SANCOLD 2025 ANNUAL CONFERENCE

Investing in the future through dams

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"Investing in the future through Dams"

Johannesburg, South Africa,

5 to 6 November 2025

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Foreword

From the Chairperson of the South African National Committee on Large Dams (SANCOLD)

I am pleased to welcome you all on behalf of SANCOLD to the 2025 Annual Conference. We are privileged to have the opportunity to host this event, exchange knowledge and experiences, and facilitate discussion on the topic of "***Investing in the future through dams".***

The preservation of water resources and the sustainable development of water infrastructure are important in South Africa as we seek to maintain and strengthen our critical water supply infrastructure. In South Africa, we have several thousand dams with a height greater than 5 metres, delivering a broad array of benefits, amongst others, water supply, power generation, flood risk management, water quality, sedimentation control, and recreational activities.

We are privileged to have Mr Tente Tente, Chief Executive of the Lesotho Highlands Development Authority (LHDA), with us at this Conference to deliver the Keynote Address. The LHDA was set up to manage the part of the Lesotho Highlands Water Project (LHWP) within the Kingdom of Lesotho's borders. The functions include, amongst others, the construction, operations and maintenance of all dams, tunnels, power stations and infrastructure. The LHDA will present several case studies and the progress of the LHWP Phase 2, which includes a tunnel and a large dam. The keynote focuses on *Lesotho Highlands Water Project as a Case Study for Transboundary Water Cooperation.*

The papers at this Conference cover a variety of topics. These include:

1. Modern technologies and methods in the design and construction of new dams
2. Climate and environmental adaption
3. Dam Rehabilitation
4. Tailings Dam Operations/lifecycles and dam closure
5. Dams for water retention or flood control
6. Geotechnical aspects of dams and water engineering structures
7. Dam Safety and Risk Management
8. Pump Storage schemes and hydropower development
9. Other

We are pleased that approximately 30% of the papers have been prepared and are being presented by our younger colleagues. The Young Persons Forum will be holding a meeting, with a social event afterwards.

Lastly, a big thank you to our Professional Conference Organiser, Qinisa, Michelle Blaeser, and the SANCOLD organising committee (Kogi Naidoo and Janice Zhang) for this event, which we trust you will enjoy.

Dr Henry-John Wright
Chairperson of SANCOLD

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Preface

I wish to thank authors for their efforts at producing and delivering quality papers of appropriate quality and relevance. We trust that the Proceedings will be a valued reference for those working in the various fields covered and that it will form a suitable basis for discussion and future development and research.

All papers submitted for the Conference were subjected to a full process of peer review and the proceedings contain only those papers that were accepted following this process. The review of the papers was undertaken by the members of the review panel acting independently on one or more assigned papers. The invaluable assistance, which has greatly enhanced the quality of the Proceedings, is greatly acknowledged. This is also most important for Continuing Professional Development accreditation.

I also would like to extend my appreciation to the conference secretariat for their assistance with organising the event.

Finally, my appreciation is also extended to the Qinisa team and support staff, especially Wilma Nortman, for the assistance of compiling these Proceedings.

Michelle Blaeser
Editor

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ADVANCES IN CFRD DESIGN AND CONSTRUCTION

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PRESENTER: JACQUES DU PLESSIS

ABSTRACT

The current ICOLD Bulletin 141 on CFRDs was published in 2010. Since then, approximately 150 large CFRDs were constructed in China alone. Many of these were constructed in highly seismic areas, in very steep valleys, and using materials previously thought not suitable. A wealth of knowledge was therefore gained over the last 15 years that is somewhat undocumented. CFRD design shifted from experience based, empirical design methods to analytical performance based approaches. The paper will discuss CFRD design evolution and highlight successful CFRDs that show when designed and constructed according to modern methods, CFRDs are resilient, adaptable, and cost effective, even on deep alluvium foundations and in highly seismic environments.

1. INTRODUCTION

Concrete Face Rockfill Dam (CFRD) design can be grouped into the following four development periods:

- 1985 to 1995 – Empirical designs.
- 1996 to 2005 – Numerical tools were developed after face slab cracking incidents at a few dams.
- 2006 to 2015 – 3D numerical analyses in which Mohale Dam played a significant role due to the large number of rockfill settlement instrumentation records that were available.
- 2016 to date – Performance based, and data driven 3D modelling based on actual rockfill settlement readings during construction.

Since 1987 China has constructed 390 CFRDs higher than 30m, 140 are currently under construction totalling 530, and approximately 90 are in the planning phase – see Figure 1. Of these, the 247m high Dashixia Dam is scheduled for completion by 2026, surpassing the 233m Shibuya Dam; currently the tallest CFRD in the world. The 280m high MJ Dam is currently being planned.

This paper discusses some of the advances in CFRD design and construction over the last 10 to 15 years. Firstly, some very high dams will be highlighted followed by design advances that made these high dams possible, even at less than favourable dam sites.

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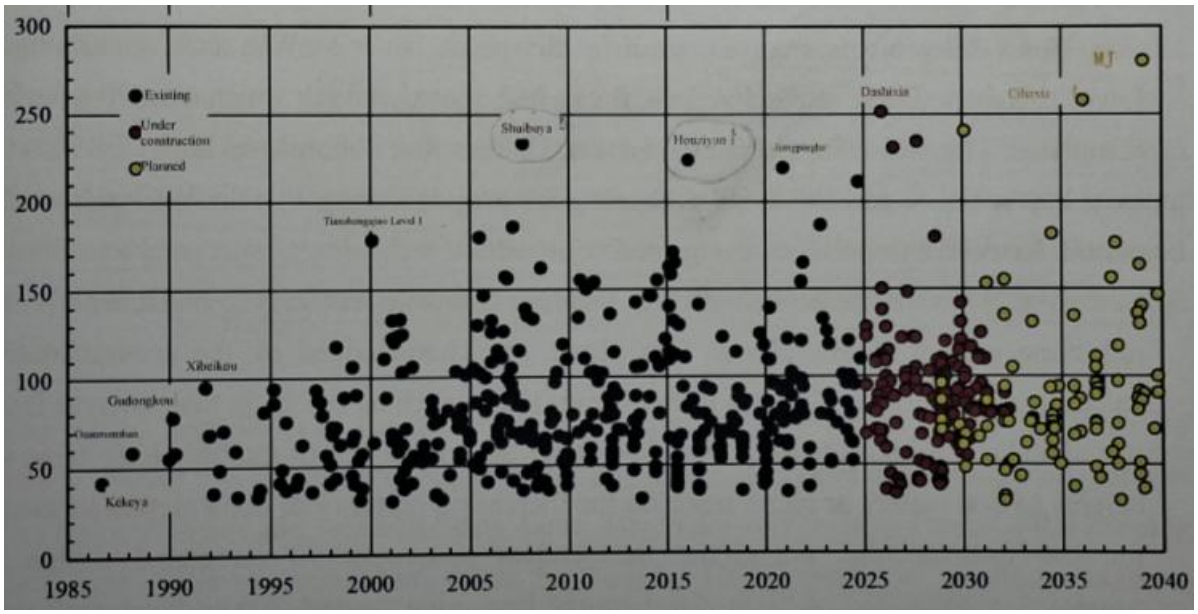

Figure 1. CFRDs in China since 1987

2. VERY HIGH CFRD EXAMPLES

Figure 2. 211m Maerdang Dam

The Maerdang Dam is located on the upper Yellow River in China. Construction presented extreme challenges due to the high altitude (>3000 masl), cold climate, and low oxygen levels. The project was completed in 2024. The first 36t roller compactors were used at Maerdang Dam.

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Figure 3. 233m Shibuya Dam

The Shuibuya Dam is located on the Qingjiang River in China. At 233 m it is currently the tallest CFRD in the world.

Figure 4. 247m Dashixia Dam

The Dashixia Dam is the world's highest CFRD under construction on the Kumarak River in China and is scheduled for completion in 2026. Note the vertical left flank and asymmetrical valley.

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Figure 5. 223.5m Houziyan Dam

The Houziyan Dam is 223.5m high and is located on the Dadu River in China. The project was completed in 2016 and is located in a highly seismic area. Note the very narrow valley and the downstream buttress of the embankment. The first 32t vibratory rockfill compactors were used at this dam.

Figure 6. 156m Zipingpu Dam

Zipingpu Dam is on the Min River in southwest China. A 7.9 magnitude quake in 2008 caused some damage to the dam. The dam is located just a few kilometers from the earthquake epicenter, and just a few hundred meters from the fault. The reservoir had to be gradually drained to permit consolidation works, but repairs were carried out relatively quickly.

3. DESIGN ADVANCES

Design advances will be discussed under the following headings: Embankment Zoning, Materials, Plinth, Face Slab, and Instrumentation and a few advances are mentioned for each. Of these, materials together with geotechnical and topographical conditions are the critical design drivers.

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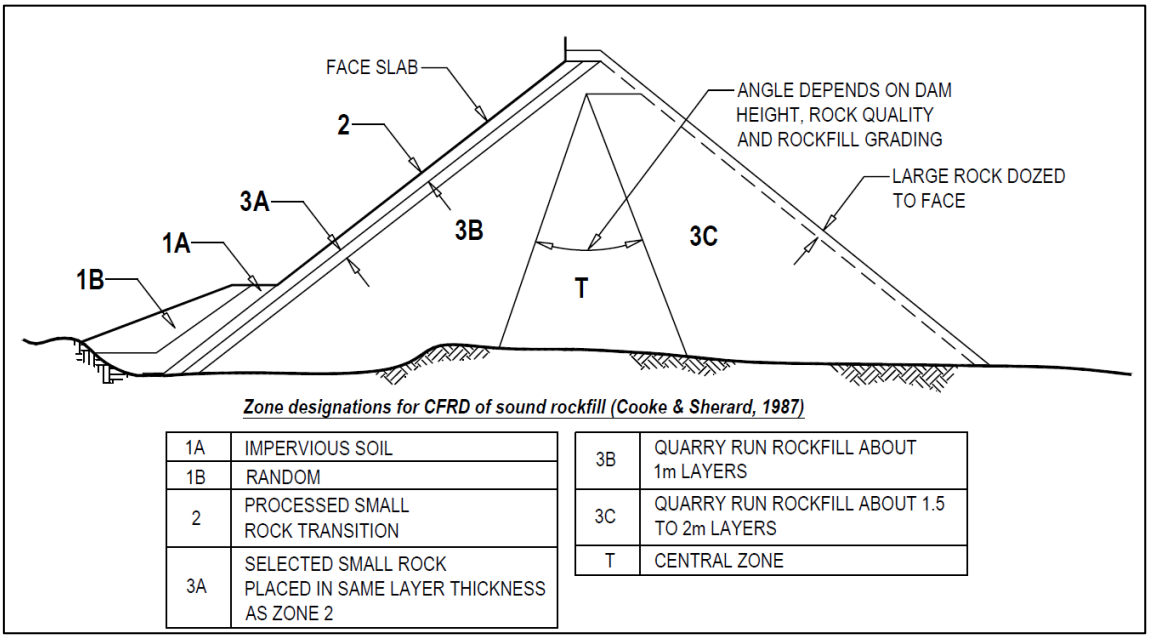
3.1 Embankment Zoning

Since 1987 zone designations by Cooke and Sherard (see Figure 7) did not change much to date (see Figure 8). The following designations are internationally recognisable:

- Zone 1 – impervious soil.
- Plinth and Face Slab membrane.
- Zone 2 – filter and transition zones under the concrete face slab membrane providing support to the face slab. Zone 2 is to provide a semi-impervious barrier preventing large leakage rates even should the face slab crack.
- Zone T – Zone T is a central zone that, depending on the dam height, rockfill quality and grading can vary in size.
- Zone 3 – bulk rockfill generally increasing in size and permeability from upstream to downstream. Rockfill settlement of this zone should be as low as possible.

Some zoning refinement includes the Zone 2B downstream of the perimetric joint between the plinth and face slab. This zone can be designed together with Zone 1A to seal possible leakage through the perimetric joint over time. Zone 3D is a drainage zone to freely drain potential dam leakage.



Zone designations for CFRD of sound rockfill (Cooke & Sherard, 1987)

1A	IMPERVIOUS SOIL
1B	RANDOM
2	PROCESSED SMALL ROCK TRANSITION
3A	SELECTED SMALL ROCK PLACED IN SAME LAYER THICKNESS AS ZONE 2

3B	QUARRY RUN ROCKFILL ABOUT 1m LAYERS
3C	QUARRY RUN ROCKFILL ABOUT 1.5 TO 2m LAYERS
T	CENTRAL ZONE

Figure 7. Zone designations by Cooke and Sherard

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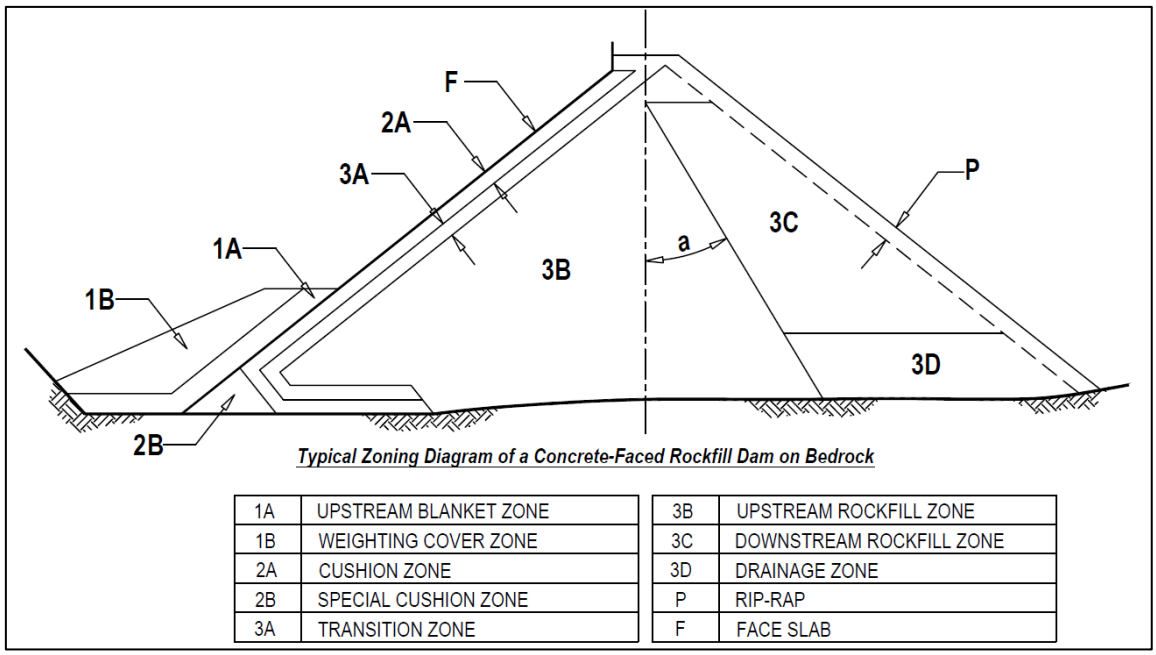

Figure 8. Recent zone designations

3.2 Materials

As stated above, materials together with topographical and geotechnical conditions are the critical design drivers. It is also related to materials where the most design advanced took place with a focus on minimising rockfill settlement as much as possible after construction and during reservoir filling, especially applicable to steep, narrow valleys. The modern focus is more on finer rockfill gradings that will result in higher moduli (less settlement) and lower porosity without negatively impacting the free drain rockfill characteristics – see Table 1.

Table 1. Example of Modern Rockfill Specifications

Embankment Zone	Particle Size Distribution				Porosity (%)	Hydraulic conductivity (cm/s)	Compaction criteria
	Max size (mm)	Fines content (P_5) (%)	Clay/Silt content ($P_{0.075}$) (%)	Coefficient of Uniformity (Cu)			
Upstream blanket	100	<50	>15	-		< 1 x 10^{-4}	>90%
Cushion material (2A)	80	35 to 50	4 to 8	-	<17	1 x 10^{-3} to 10^{-4}	Dry Density > 2.34 g/cm^3
Transition (3A)	300	10 to 30	<5	-	<18	1 x 10^{-2}	Dry Density > 2.31 g/cm^3
Bulk rockfill (3B)	800	<20	<5	>5	<19	1 x 10^{-1} to 1	Dry Density > 2.2 g/cm^3

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Rockfill and porosity target guidelines for different dam sizes were developed and are indicated in Table 2.

Table 2. Rockfill Porosity and Density Targets

Material		Cushion material	Transition material	Upstream rockfill		Downstream rockfill		Gravel fraction density
				Hard rock	Soft rock	Hard rock	Soft rock	
Medium dam	Porosity (%)	15 – 20	16 – 22	20 – 24	18 – 22	20 – 25	18 – 23	0.75 – 0.85
High dam	Porosity (%)	15 – 19	16 – 21	17 – 22	17 – 20	18 – 23	17 – 20	0.85 – 0.9
Ultra-high dam	Porosity (%)	15 – 18	16 – 19	17 – 20		18 – 21		0.85 - 1

Softer rockfill (typically UCS < 25 MPa) with good grading and in combination with hard rockfill has become acceptable. Up to 70% soft rockfill has been adopted in practice. An important design consideration is to balance material properties, the dam height, and the site geometry. The focus is more than ever to use excavated materials for construction in different parts of the embankment according to the material quality; achieving significant technical, economic and environmental benefits. After compaction however, all rockfill materials must have low compressibility, high shear strength, good permeability and adjacent material zones must comply to filter criteria. To overcome arch effects in narrow valleys a higher modulus is targeted.

Research has shown that rockfill exhibits two size dependant mechanisms: large particles are prone to breakage leading to lower material parameters. However, the interlocking effect of large particles is stronger than that of small particles that in turn result in higher material particles. The scale effect is influenced by the combined impact of these two mechanisms. Field monitoring data confirmed that actual rockfill deformations in high dams exceeds calculated values and in-situ rockfill deformation parameters are lower than given by laboratory test results.

3.3 Plinth

At dam sites where alluvial deposits are very deep, the plinth can in some cases be founded directly on riverbed gravel, but on two conditions namely: The plinth must be flexible to accommodate potential displacements, and a cut-off wall is required to prevent seepage through the gravel material – see Figure 9. This is however not a new concept as articulated plinths together with cut-off walls have been used in practise for more than 50 years. Examples of articulated plinths on alluvium deposits include:

- 110m Santa Juana Dam in Chile.
- 85m Puclaro Dam in Chile.
- 33m Los Molles, 116m Potrerillos and 136m Caracoles Dams in Argentina.
- 43m Limon Dam in Peru.

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Cut-off walls take on many forms, from plastic concrete walls to slurry walls or jet grouting.

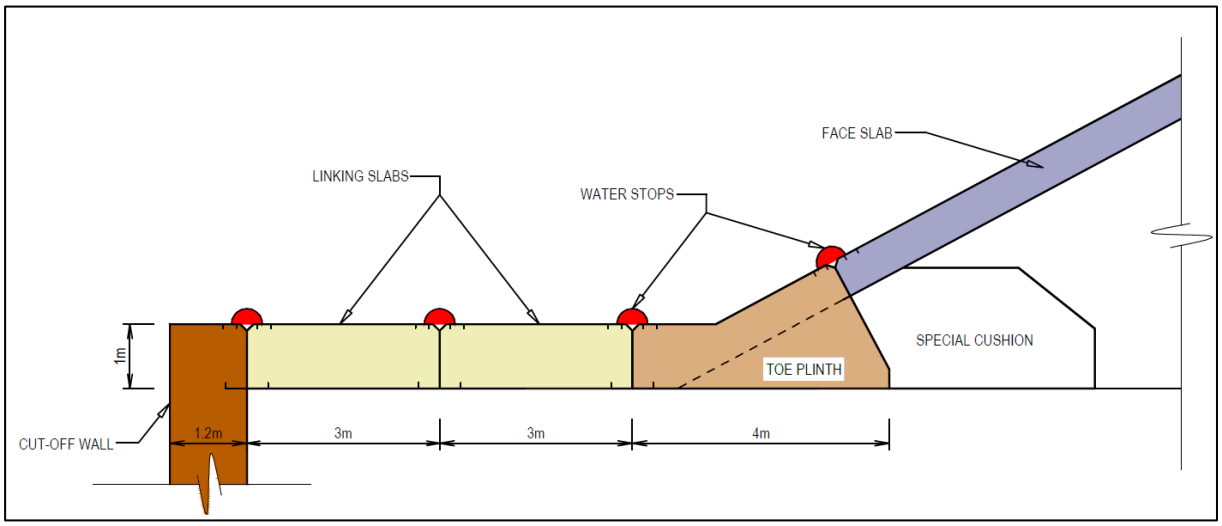

Figure 9. Example of an articulated plinth showing typical dimensions

3.4 Face Slab

At narrow and steep valleys, additional vertical face slab joints (typically 7.5m wide) must be considered to mitigate compression stresses. This concept was for example adopted at Shuibuya, El Cajon, Bakun and Mazar Dams. The modern approach to finalise face slab compression joint positions is to do performance based 3D modelling with actual rockfill moduli determined on site during construction. This underlines the importance of obtaining accurate instrumentation readings during construction. At some high dams, horizontal face slab joints with similar details as vertical compression joints, are introduced to reduce tensile stresses in the face slab.

Central face slab waterstops are not being used often anymore due to concrete honeycombing under the waterstop and easier side shuttering without waterstops extending through the joints. Copper waterstops are still used as backstop under the slab. Varies configurations and types of surface mounted waterstops are being used, from fly-ash with geotextiles to mastic and neoprene protection strips.

The perimetric joint which connects the plinth and face slab, has seen its waterstop system evolve from a single-type or "self-healing" waterstop to a combined system integrating both sealing and self-healing functions. Compared to the traditional three-layer waterstop system, most projects now retain only the bottom copper waterstop, also eliminate the middle waterstop, and reinforce the top waterstop to form a new triple waterstop system where each layer functions independently.

3.5 Instrumentation

The remarkable performance of a number of CFRDs can be partly attributed to the significant role played by monitoring dam behaviour. Instrument readings can be used to confirm design assumptions and provide design confidence going forward with ever increasing dam heights. It is standard practise to at least monitor the following:

- Rockfill settlement – internally and externally and for different zones.
- Rockfill moduli.
- Pore pressures within the embankment.
- Seepage.

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- Face slab deflection, displacement, and concrete strain.

Modern instruments include fibre optic seepage sensing by measuring the temperature of water seeping through the embankment. Where the temperature contrast between the rockfill and the seepage water is not sufficient to provide a reliable detection, a self heating cable is used that allows detection of temperature differences over large distances.

4. CONSTRUCTION ADVANCES

The construction technology for CFRDs has developed, and is continuing to develop, fast due to the simplicity, cost competitiveness and inherent safety of this dam type. Some more recent construction advances include the following:

- Ground Penetrating Radar (GPR) to evaluate rockfill compaction.
- Heavier compaction rollers up to 36t compared to typically 12t used in the past. Together with heavier rollers more compaction passes up to eight are becoming more common.
- Unmanned rollers and other construction equipment are being used for safety purposes.
- Global Network of Satellite Systems (GNSS) is starting to replace survey beacons.
- Rockfill placement rates: Due to the use of large machinery with better construction efficiency, rates up to 1 million m^3/month are being achieved and generally between 500 000 to 700 000 m^3/month.
- Fibre optic sensing is being used more regularly to record temperature, strain and measure seepage.

5. CONCLUSION

It has for a long time been known that CFRDs contain inherent safety features such as:

- The entire rockfill embankment is located downstream of the water body.
- Water load on the concrete face enters the dam foundation upstream of the dam axis.
- Uplift and pore pressure do not influence dam safety if designed correctly.
- High shear strength of rockfill compared to earthfill.
- Proven seismic resilience.
- Rockfill zoning promotes through flow.

With modern design techniques and principles, the above features were taken to another level especially now that face slab behaviour can be replicated through 3D numerical modelling and if necessary, design adjustments can be made during construction with minimal impact on the contractor and programme.

CFRDs have exceptional adaptability to complex conditions and can efficiently make use of local and sometimes marginal materials.

Global experience proves CFRD as a reliable solution when designed and constructed in accordance with modern methods, even on challenging foundations and in steep valleys or in highly seismic regions. This together with relative ease of construction, construction speed and often competitive cost, often makes the CFRD the dam of choice.

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STABILITY ANALYSIS OF KATSE DAM FOUNDATION WEDGES UNDER STATIC AND SEISMIC CONDITIONS

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3. LHDA, Maseru, Lesotho

PRESENTER: FRANCOIS SMIT

ABSTRACT

The last Panel of Expert (POE) review of Katse Dam in 2018 concluded that the original design calculations although appropriate at the time, does not satisfy all the current international accepted criteria and procedures. This specifically pertains to the abutment seismic stability during normal operation and extreme conditions. As a result, the dam owner (LHDA) initiated a project to perform a finite element analysis of the dam.

Concrete arch dams are among the most efficient and structurally complex dam types, utilizing their curved geometry to transfer hydrostatic and hydrodynamic loads into the abutments. This load transfer mechanism makes the integrity of the foundation particularly critical. Therefore, a detailed analysis of existing foundation rock wedges—formed by faults, joints, and other geological discontinuities—is essential to ensure their stability under both static and seismic loading conditions.

This paper discusses the three-dimensional foundation wedge analyses of Katse Dam. Particular attention was be given the dynamic analysis including the interpretation and presentation of results as well as cognisance given to the potential failure modes.

1. INTRODUCTION

Concrete arch dams are among the most complex and efficient structures which rely on their curved geometry to distribute the applied loading conditions into the foundation abutments. However, these dams are not inherently stable under seismic loading conditions which has been repeatedly shown under strong earthquakes (Wieland, 2014).

When designing new arch dams, or analysing existing arch dams, the sliding stability by means of credible failure mechanisms throughout the dam and foundation must be evaluated under static and seismic loading conditions (ICOLD, 2024). Concerning foundation failures, credible failure mechanisms are developed through a combination of existing faults, shears, joints and similar discontinuities which daylight at the surface of the foundation to form a foundation wedge (Londe, 1973). The sliding stability of these foundation wedges is determined by evaluating the resistance capacity, obtained through a combination of friction and cohesion, against the applied destabilising load to determine a sliding stability factor of safety.

This paper discusses the three-dimensional wedge analysis of Katse Dam subjected to both static and seismic loading conditions.

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Katse Dam is a 185 m high, wide-valley double arch dam with a crest length of 710 m. The mass concrete arch dam, located on the Malibamatso river in the Lesotho Highlands, was constructed with approximately 2.3 million m^3 of concrete and provides a reservoir storage capacity of approximately 1 950 million m^3.

2. MODEL CONFIGURATION AND METHODOLOGY

Katse Dam was constructed on a foundation comprising basaltic lavas which were formed in sub-horizontal layers of various thicknesses. The discontinuities contained within the foundation is made up mainly of sub-horizontal flow contacts, shear zones, auto-brecciated layers and subvertical stress relief joints (Tardieu, 1995).

Several foundation wedges formed by various interacting flow contacts and other features, located on the right abutment, were identified and analysed during the original design of Katse dam. The locations of these faults and layers were identified through a series of borehole investigations and geological mapping. The material properties of these layers were determined through a series of triaxial tests on the Basalt and Auto-breccia layers (LHC, 1991). At the time, the stability of the wedges was determined by estimating a range of friction angles required to maintain a factor of safety of one when only considering the effects of friction. Once obtained, these friction angles could be compared to the on-site, material tested friction angles of the rock mass to confirm whether the wedges would remain stable.

With improvements in computer computational technology and the commercial availability of user-friendly numerical modelling software packages, it is possible to conduct complex and high-resolution structural analyses of a dam and its foundation using the Finite Element (FE) method. In line with the recommendations of the POE review of Katse Dam, a detailed FE model of the Katse dam was developed including the layered foundation and the various wedges as formed by features in the rock mass. A plan view and developed downstream elevation view of the foundation are shown in Figure 1 and Figure 2.

The dam and foundation FE model mesh is shown in Figure 3. The model was created using the Midas FEA NX software and comprises 407 500 nodes and 476 400 elements. The dam and foundation were modelled as a continuum mesh body discretised with a hybrid of hexahedral, tetrahedral, pyramid and wedge 3D solid elements. The model is hexahedral dominant with other elements only used in the vicinity of sharp edges and faces.

Figure 1. Plan view of foundation wedges (Dungar et al., 2019)

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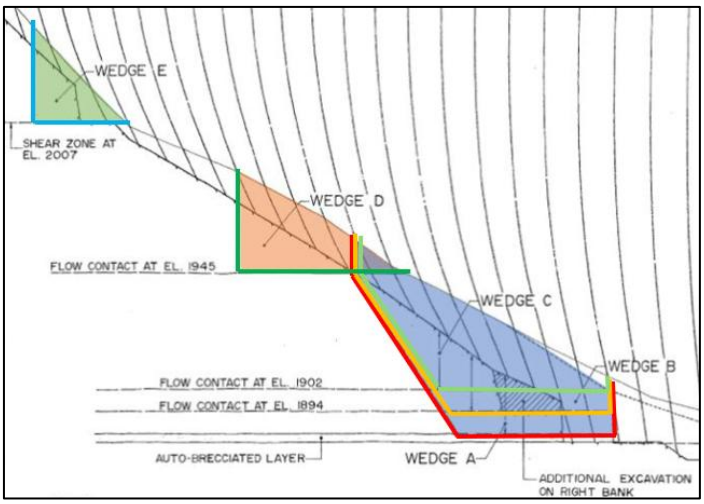

Figure 2. Developed downstream elevation of wedges A - E (Dungar et al., 2019)

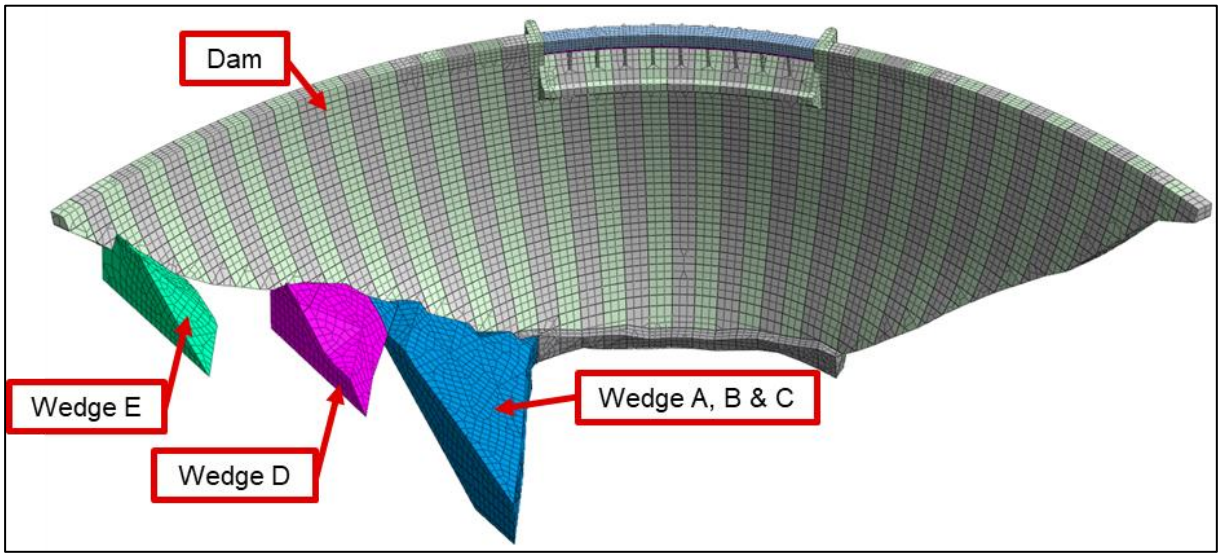

Figure 3. Top view of foundation wedge meshes

3. LOAD CASES AND MODELLING ASSUMPTIONS

3.1 Stability Criteria

Considering that South Africa does not have a design code or guidelines to regulate the design or analysis of dams, an internationally accepted standard must be adopted to determine the stability of the model (Cassells, 2024). The sliding stability criteria of EM 1110-2-2201 was adopted for the foundation stability analysis based on the assumption that that a comprehensive field investigation, involving in situ material testing, was conducted on the foundation during the design of the dam. The adopted sliding stability Factor of Safety (FoS) criteria are summarised in Table 1 as prescribed in the guidelines.

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Table 1. Sliding Stability FOS Criteria for Static and Seismic Loading Conditions (USACE, 1994)

Usual/ Normal Conditions	Unusual/ Accidental	Extreme
2.0	1.3	1.1

3.2 Applied Load Cases

Katse Dam foundation was analysed under multiple static and seismic loading conditions. In this paper, the load cases considered are limited to normal operational loading conditions, for the static load case, and an SEE level dynamic load for the seismic load case.

For the static load case, assuming normal operating conditions, the dam will be subjected to upstream and downstream hydrostatic pressures, uplift pressure and its own weight. No information was available with regards to the silt levels in the reservoir. As such, the silt load was excluded from this analysis.

The uplift pressure applied to the bottom of the dam was determined using EM 1110-2-2200 assuming a drainage efficiency of 67 %. Though the uplift pressure is applied to the base of the dam, the resulting pressures do not represent the pore pressures acting on the foundation wedges. As such, a steady-state seepage analysis was conducted to model the pore pressures within the rock mass. The results from the seepage analysis were compared to the pore pressure measurements obtained from piezometers on site to ensure that the applied pore pressures accurately represent the in-situ conditions.

The static loads adopted in the analysis are summarised in Table 2. These loads represent the loads in relation to the deepest monolith.

Table 2. Static Loads Applied to the Deepest Monolith

Loading Condition	US Head (m)	DS Head (m)	Drainage Condition	Silt Load	Self-weight	Grout Curtain
Usual	178	12	Drained	Excluded	Included	Intact

For the seismic load case, an SEE level (1 in 10 000-year event) earthquake acceleration time-history signal, was scaled and spectrum matched according to the site-specific response spectrum as derived from the Seismic Hazard Assessment. The seismic load was adopted in addition to the static loads.

A site specific Probabilistic Seismic Hazard Assessment (PSHA) was conducted, as per ICOLD Bulletin 148, considering seismic activity observed within a radius of 450 km from Katse Dam. In this analysis, the Cornell McGuire procedure was adopted for the hazard assessment (Cornell,1968; McGuire, 1976, 1978, 1978, 2004).

The horizontal elastic response spectrum as derived from the PSHA was based on the Eurocode 8 curve formations assuming 5% damping and shear wave velocity of the rock (v_s) greater than 800 m/s. The vertical response spectrum was determined assuming an average ratio of 2/3 of the horizontal response spectrum. The derived horizontal and vertical response spectrum for the SEE load case are shown in Figure 4.

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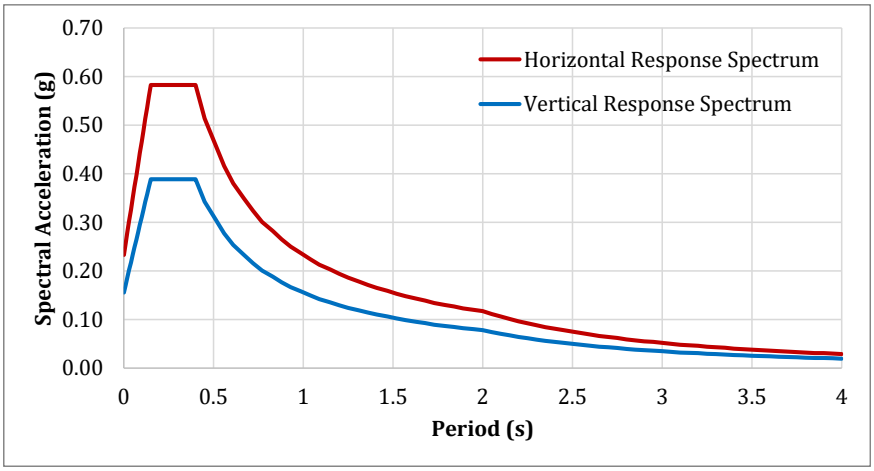

Figure 4. Vertical and Horizontal Response Spectrum

As per ICOLD bulletin 148 (2016), Eurocode 8 (2004) and several prominent design codes, three SEE level earthquakes were selected to evaluate the dynamic behaviour of the dam and foundation under SEE loading conditions. The three selected earthquakes, selected in accordance with the magnitude and distance to the epicentre, were further adapted to fit the derived response spectrum of the site. The magnitude and respective distance to the epicentre of the selected earthquakes are summarised in Table 3. The horizontal time history signal of the Sparks Oklahoma earthquake as adopted for the instream direction is presented in Figure 5.

Table 3. Earthquake Events Considered for Seismic Analysis of Katse Dam

Earthquake Event	Year	Magnitude (m_w)	Distance (km)	Vs_{30} (m/s)
Sparks Oklahoma, USA	2011	5.68	38	725
Little Skull Mountain, USA	1992	5.65	24	593
Umbria Marche, Italy	1997	5.70	28	551

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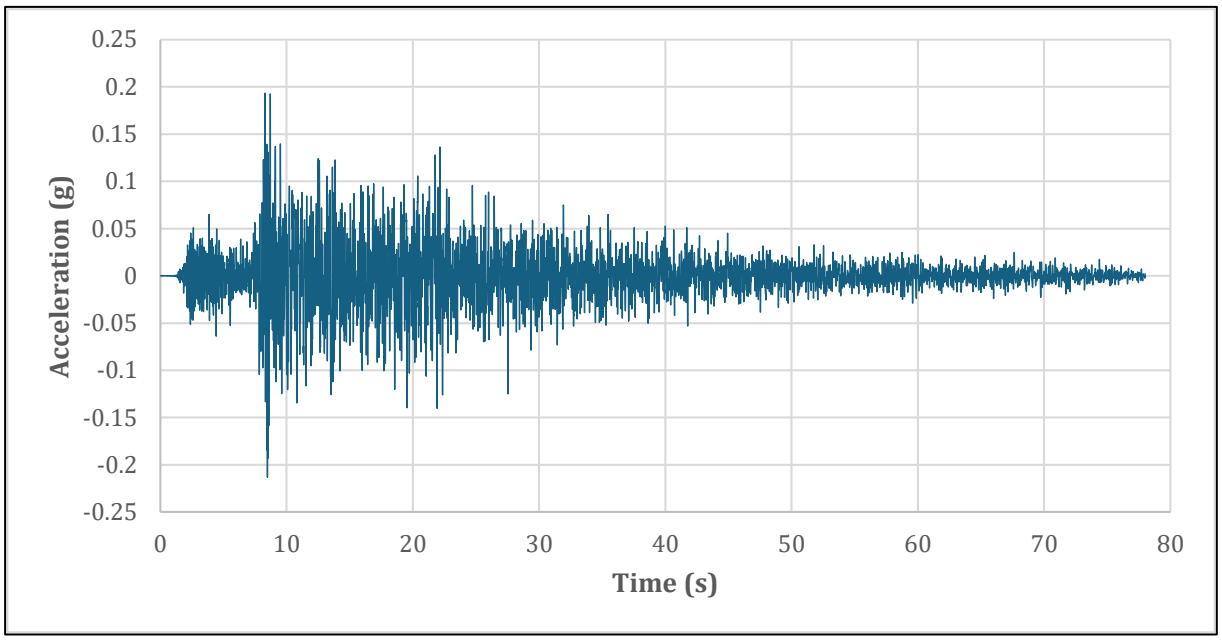

Figure 5. Sparks Oklahoma Instream Time History Singal

Rayleigh Damping, assuming 5% damping, was included in the seismic model. Westergaard added masses were used to model the hydrodynamic pressures of the reservoir during the seismic event.

3.3 Material Characteristics

The adopted concrete material properties under static and dynamic loading conditions are summarised in Table 4.

Table 4. Dam Concrete Material Characteristics Under Static and Dynamic Loading Conditions

Material Properties	Mass Concrete	
	Static	Dynamic
Unit Weight (kN/m^3)	24.5	
Elastic Modulus (GPa)	34	39.1
Poisson Ratio	0.2	0.16

Various geotechnical tests were conducted on borehole samples to confirm the foundation properties during the feasibility and design of Katse Dam. The material properties of the foundation under static and seismic loading conditions are summarised in Table 5. Figure 6 shows the location of the layers noted in the table with the colours corresponding to the layer colours. To determine the stability of the foundation wedges under static and seismic loading conditions, only the foundation wedges were assigned a mass while the rest of the foundation remained massless. A density of 2 800 kg/m^3 was adopted for the foundation wedges which was provided in the original design report. Typical densities for both Basalt and Breccia range between 1 800 – 3 000 kg/m^3 (Tenzer et al., 2011).

According to Dungar et al. (2019), the residual friction angle between the flow contacts varied between 42° and 49.5°. As such, a residual friction angle of 46° with a cohesion of 0 kPa was adopted for the stability analysis of the foundation wedges.

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Table 5. Foundation Material Characteristics Under Static and Dynamic Loading Conditions

Foundation Layer Description	Unit Weight (kN/m³)	Elastic Modulus (GPa)		Poisson Ratio	
		Static	Dynamic	Static	Dynamic
Upper right bank basaltic layer above 1975 m	28*	38	43.7	0.23	0.184
Upper basaltic layer above flow contact 1892.5 m	28*	30	34.5	0.25	0.2
Basaltic layer above 1885 m and below flow contact 1892.5 m	28*	30	34.5	0.25	0.2
Upper auto-brecciated layer (1882 m – 1885 m)	28*	10	11.5	0.35	0.28
Basalt layer between auto-brecciated layers (1869m – 1882 m)	0	25	28.75	0.27	0.216
Lower auto-brecciated layer (1866 m – 1869 m)	0	20	23	0.30	0.24
Lower Basalt layer below auto-brecciated layer < 1866 m	0	40	46	0.21	0.168

* Only adopted for foundation wedges.

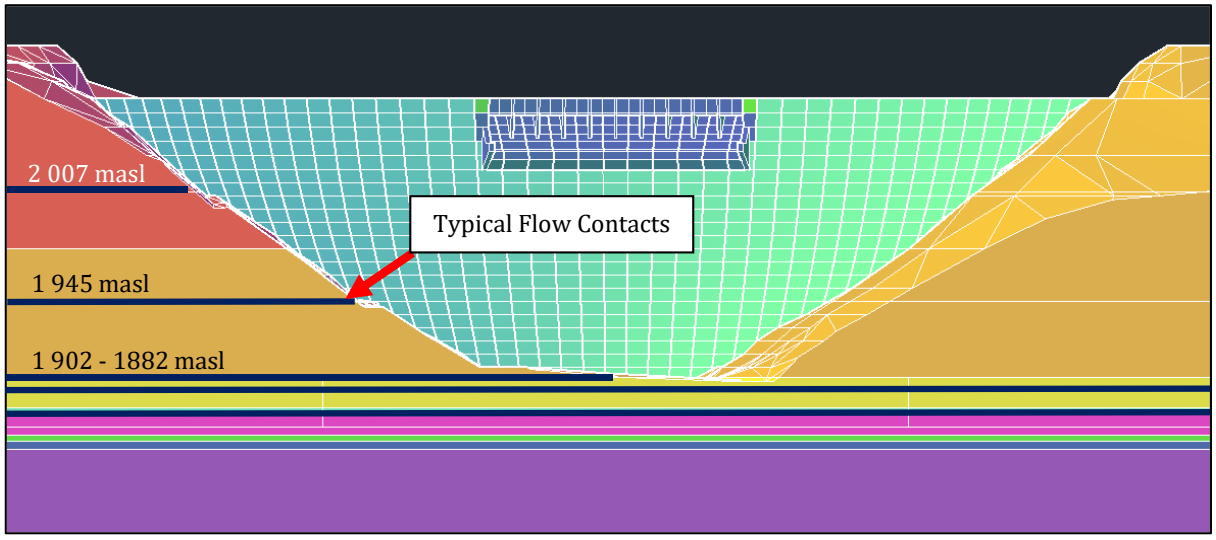

Figure 6. Section of Katse Dam Foundation Model Showing Foundation Layers

4. ANALYSIS RESULTS AND DISCUSSION

The five wedges were analysed under static and seismic loading conditions for the various load cases and maintained a FoS above the assessment criteria. Wedge C, located on the lower valley on the right bank, maintained a FoS of 2.0 under normal operating conditions and 1.6 under seismic loading conditions. As such, considering the wedge location, applied loading and the resulting FoS, Wedge C was selected to be discussed in this paper.

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4.1 Steady-State Seepage Analysis Results

The pore pressure distribution from the seepage analysis under normal operating conditions is shown in Figure 7. These results show the effect of the grout curtain in reducing the pore pressures towards the downstream. The resulting pore pressures were used as input for the stability analysis of the foundation wedges. The modelled pore pressures were calibrated against measured pore pressures.

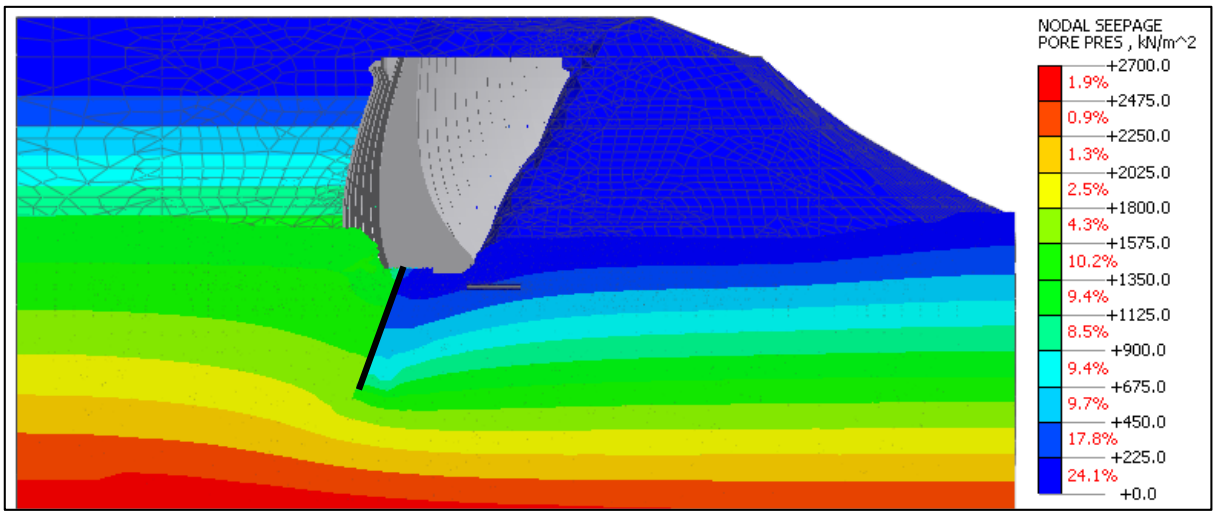

Figure 7. Pore Pressure Distribution Under Normal Operating Conditions

4.2 Static Analysis Results

The Normal stress distribution on the surface of the horizontal flow contact of Wedge C, under normal static operating conditions (FSL) is shown in Figure 8. The plot is orientated such that the heel of the wedge is located at the bottom of the figure and the toe is located at the top. These results show the thrust applied from the dam to the foundation due to the hydrostatic loading. Most of the normal stress is concentrated under the dam and reduces towards the toe of the wedge.

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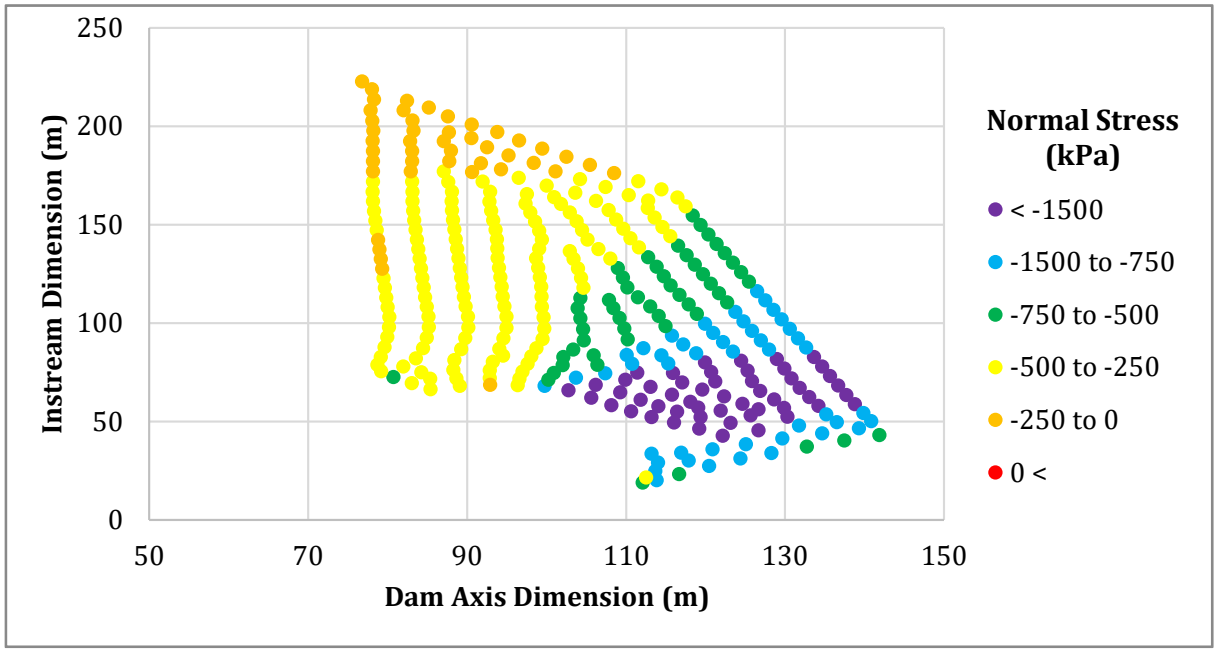

Figure 8. Normal stress distribution on the surface of the flow contact for Wedge C

The Shear stress distribution on the surface of the horizontal flow contact of Wedge C, under normal static operating conditions is shown in Figure 9. The plot is orientated such that the heel of the wedge is located at the bottom of the figure, and the toe is located at the top. Like the normal stresses, most of the shear stresses are concentrated below the dam and reduce towards the toe of the wedge. The plot shows that small shear stresses, and in some cases small tensile stresses, develop near the heel of the wedge which is a limitation of a linear-elastic analysis. In reality, minor tensile cracks would develop in the region of the tensile stresses which would redistribute these stresses across the wedge.

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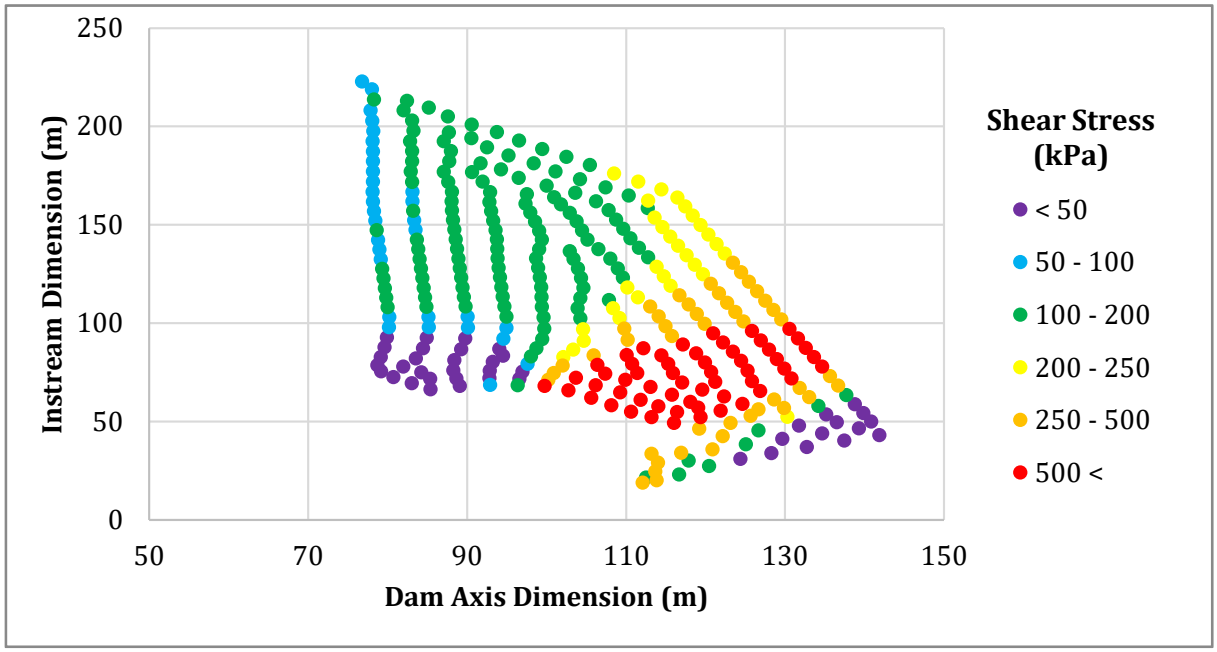

Figure 9. Shear stress distribution on the surface of the flow contact for Wedge C

The factor of safety distribution on the surface of the horizontal flow contact of Wedge C, under normal static operating conditions, is shown in Figure 10. The plot is orientated such that the heel of the wedge is located at the bottom of the figure, and the toe is located at the top. These results show that the lowest factor of safety is concentrated at the heel (where the tensile stresses develop) and toe (where the normal stresses is the lowest) of the wedge. The derived FoS for the wedge, under normal static operating conditions, is 2 which is considered stable under the adopted stability criteria.

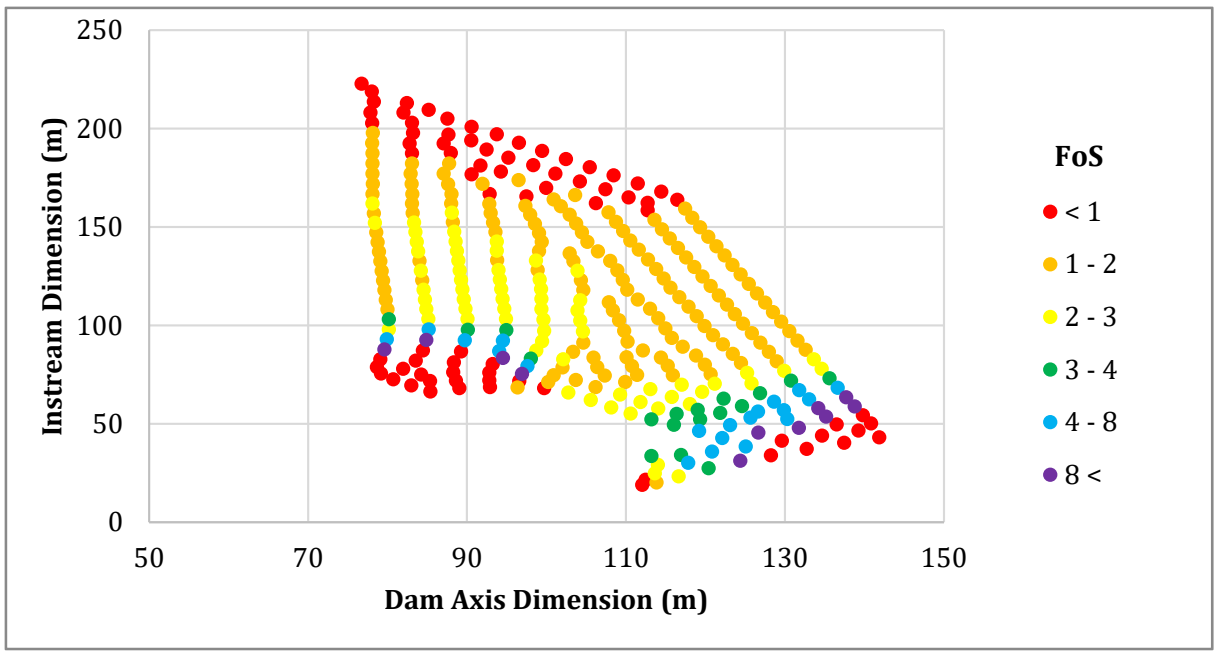

Figure 10. FoS distribution on the surface of the flow contact for Wedge B

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4.3 Seismic Analysis Results

The derived time-history factor of safety of Wedge C, under seismic loading is shown in Figure 11. These results indicate that the wedge maintains a factor of safety above the required criteria throughout the seismic event. As such, the wedge is considered stable under seismic loading conditions.

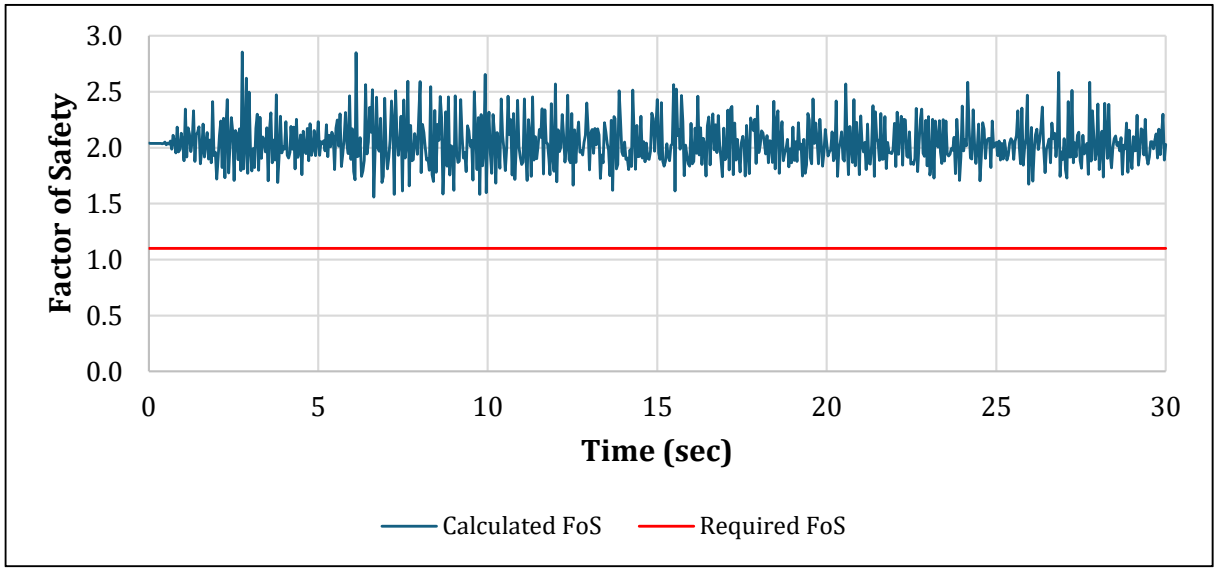

Figure 11. Wedge C Seismic Sliding FoS Results

The computed shear and resisting force time-history of Wedge C, under seismic loading is shown in Figure 12. These results indicate that the resisting forces remain larger that the shear forces throughout the seismic event, effectively indicating stability is maintained throughout the event.

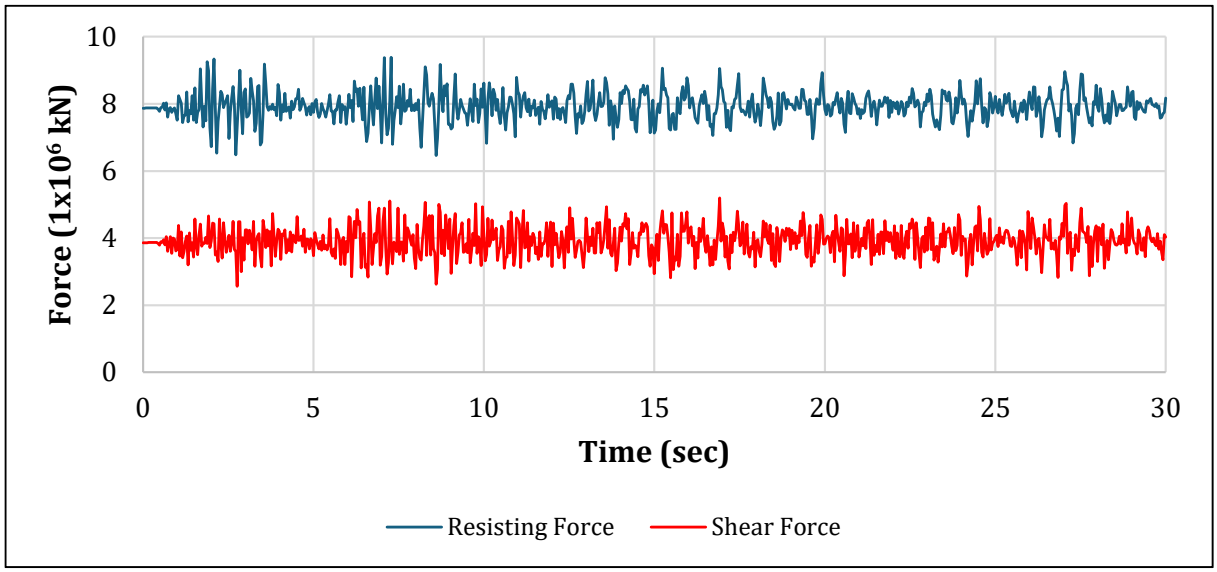

Figure 12. Wedge C Seismic Shear and Resisting Force Results

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5. CONCLUSION

This paper discussed the methodology used to model, analyse and evaluate the structural response of Katse Dam's foundation abutment under static and seismic loading conditions. The methodology and analysis techniques adopted for this study were based on accepted engineering principals using the finite element approach.

A three-dimensional FE model of the dam and foundation was developed and analysed under various loading conditions. Factors of Safety were derived from the structural response of the wedges and compared to published acceptance criteria.

The results described in this paper, under both static and seismic conditions, showed that the wedge would maintain a factor of safety above the required criteria under both static and seismic loading conditions. These results suggest that the wedge will remain stable under the applied loading conditions.

6. ACKNOWLEDGEMENTS

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CIRCULAR ECONOMY THINKING IN THE DESIGN AND STEWARDSHIP OF DAMS

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ABSTRACT

Circular economy (CE) frameworks are typically applied in sectors where waste generation is significant, and material cycles are short. Civil infrastructure, by contrast, is built to last. Dams, in particular, are designed for longevity, often serving generations with minimal physical turnover. As such, the conventional CE emphasis on waste minimisation is less applicable. However, the broader philosophy of circularity, especially the higher-value R-strategies such as Rethink, Reduce, Reuse and Refurbish, holds valuable potential in reshaping how we approach dam design, maintenance, and end-of-life planning.

This paper introduces CE as a mindset rather than a waste-driven metric and explores how dam engineers can proactively embed circular thinking into infrastructure design. What features can we incorporate today that will ease the burden on future generations tasked with refurbishing, repurposing, or decommissioning these assets? How might our current dams have benefited from such foresight in the past? Through conceptual applications, this presentation invites delegates to reflect on asset stewardship beyond design life, and encourages a shift from linear, end-state thinking to adaptive, regenerative design philosophies.

In reframing CE for dams, the goal is not to chase zero waste, but to maximise long-term value, adaptability, and resilience. By designing with the full lifecycle, including refurbishment and reuse, in mind, we can enhance the sustainability and relevance of our dams in a changing world.

1. INTRODUCTION

Among engineers, the phrase circular economy (CE) often provokes scepticism. For many, it conjures up images of bureaucratic checklists, donor-driven jargon, and buzzwords detached from the realities of practical design. We already design for longevity, adaptability, and refurbishment. We already think in terms of service life measured in centuries. Slapping a new label on good engineering practice feels like bureaucracy at best, and box-ticking at worst. CE can feel like an irritant, a distraction from real engineering. Yet, hidden beneath this terminology is a set of ideas that, when reframed, align closely with the ethos of good engineering practice.

This paper suggests that CE is not merely about waste management or recycling, but rather about designing for long-term value retention. In dam engineering, this translates into foresight: building assets that are adaptable, repairable, and easier for future generations to maintain or repurpose. The central argument is that by consciously applying CE thinking, we can not only improve the technical resilience of dams but also align our work with the broader conversation surrounding sustainability.

Further to this, CE is the language that funders, policymakers, and global institutions have chosen to evaluate projects. So rather than dismiss it, if we can reframe our existing practices in CE terms, we can speak a language that aligns with funders while staying true to the practical, common-sense engineering we already know.

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2. CIRCULAR ECONOMY PRINCIPLES AND THE R-STRATEGIES

The formal definition of CE centres around the closing of material loops, reducing waste, and regenerating natural systems. In practice, however, this has often been interpreted narrowly, with a strong emphasis on the management of waste. The archetypal example is the aluminium Coca-Cola can, endlessly being re-smelted into new cans, as implied in Figure 1. While technically circular, this sits very low on the value chain. Recycling is only marginally better than landfill disposal, since most of the embodied energy and form of the product is lost.

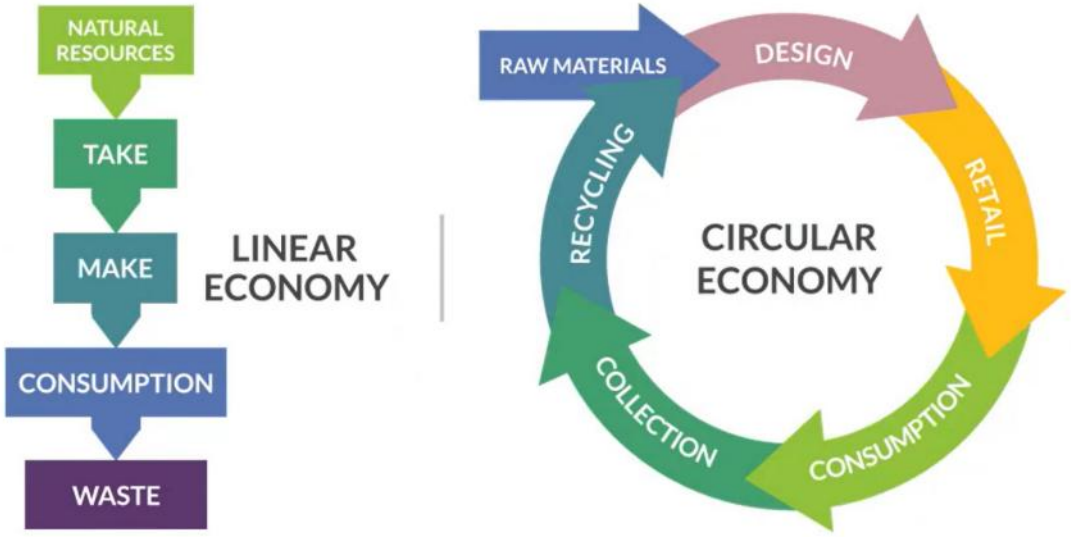

Figure 1: Shift from linear to circular economy (adapted from BCG & WBCSD, 2018)

Whilst informative, this vision of circularity is often too simplistic to be applied to our industry. When it comes to infrastructure, the situation is somewhat more complex. The "value-hill" is a visual metaphor that may be more appropriate, as shown in Figure 2.

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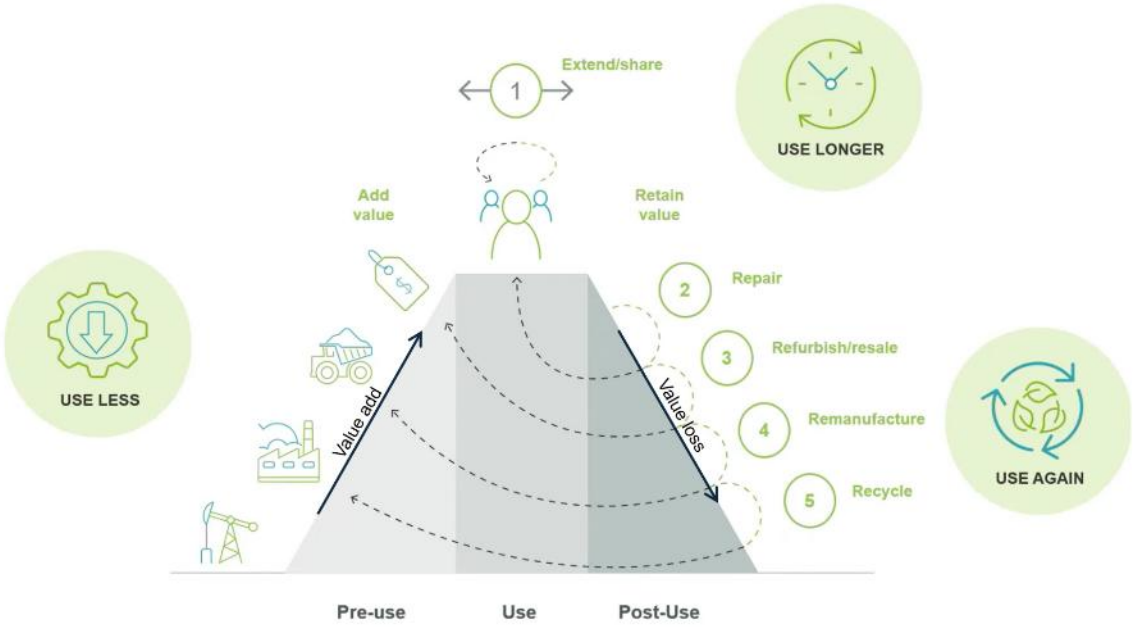

Figure 2: Value-hill concept of circular economy (adapted from Aurecon, 2024)

Value is created as a product or piece of infrastructure is designed, built, and put into service. Once in use, it sits on the "plateau" of the value hill. Eventually, as it reaches the end of its service life, it begins to move down the post-use slope, losing value.

Various interventions can however be implemented to retain the value of the service or product. Repairs bring the asset back into service with minimal intervention. Refurbishment restores it to an "as-new" condition. Remanufacturing involves partial dismantling and reconstruction, while recycling requires complete breakdown and reprocessing. The goal is to stay as high on the hill as possible, ideally on the plateau itself. In other words, extend the service life of the infrastructure and preserve its value for as long as possible.

This metaphor can be summated into the so-called "R-strategies". There can be any number of R's, though the nine listed in Figure 3 are frequently used. At the top of this hierarchy are the approaches that maximise value retention. Note that these do not focus on managing waste at the end of life but on designing products, systems, or infrastructure in ways that prevent waste, prolong useful life, and enable flexible adaptation. This focus on designing and operating assets to maximise value retention rather than waiting for their eventual disposal, is where the true opportunity lies for dams.

The guiding question then becomes: how can dam engineers embed design foresight today to preserve maximum functional and societal value for tomorrow?

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	Strategy	Description
USE LESS	Refuse	Making a product redundant by cancelling its function, or by substituting it with a radically different product. Choosing to build, buy or use less.
	Reduce	Intensifying asset use by sharing multifunctionality. Using assets with more care, and for longer. Using fewer natural resources and materials to manufacture products and assets.
USE LONGER	Reuse	Reusing a second hand or discarded asset or product for the same purpose, by a different user (with or without exchanging money).
	Repair	Repairing broken or malfunctioning equipment to enable continuation of its original function.
	Refurbish	Replacing or repairing components of an older product to upgrade quality, retaining the overall structure of the product for use in its original function.
	Remanufacture	Returning used or non-functional equipment or parts to 'as-new' or 'better-than-new' condition through a controlled industrial process.
USE AGAIN	Repurpose	Using discarded products or their parts in new products with a different function. For example, side streams or by-products from agricultural processes in new products.
	Recycle	Converting waste materials into secondary raw materials for use in manufacturing or construction.
	Recover	Recovering energy from materials.

Figure 3: Circular Economy R-strategies (Aurecon, 2024)

3. WHY CIRCULAR ECONOMY MATTERS FOR DAMS

In short, CE matters for dams because it encourages whole-life-cycle stewardship.

Dams are not short-lived consumer products. They are intergenerational assets, designed for a century or more of service. This longevity makes them appear at odds with CE, whose popular discourse revolves around fast-moving goods. Yet precisely because dams last so long, their design choices reverberate far into the future. Decisions made today will shape whether engineers in 50 or 100 years inherit assets that can be refurbished and adapted, or structures that are brittle and difficult to maintain.

When framed as value retention, CE principles translate naturally into dam practice. They echo several business logics already recognised in the field of circular economy (Bocken et al, 2016), as follows:

- **Recover value in waste:** in dams, this could mean sediment management strategies that turn waste sediment into a useful resource.
- **Reform use of resources**: using renewable energy, modular outlet components, or durable alloys that can be replaced easily.
- **Sharing/optimising use**: multipurpose reservoirs that provide water, energy, flood control, and recreation, maximising societal value.
- **Product as a service:** akin to EPC or equity-based business models, where incentives align for long-term performance and not just upfront delivery.
- **Extend lifecycles**: perhaps the most obvious, as dam engineering is about making structures last as long as possible.

Dams and CE also have a unique risk management dimension. Sizable and significant unknowns such as climate change, sedimentation, and evolving safety standards create uncertainty for dams over their lifetime. Designing dams with adaptability in mind reduces these risks and builds in resilience. In other

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words, engineers today should, at several points in their design process, take a step back and examine their designs from a broad systems-based viewpoint. This will allow them to make responsible choices that maximise the ability for future modifications to be made, if needed. Decisions made today should not create impossible challenges for the engineers of tomorrow.

One further benefit of adopting the CE mindset is that its terminology strengthens the case for acquiring funding for a project. Many development banks and climate funds now screen proposals for CE criteria. By framing dam projects with CE intentions, such as adaptability, reuse, lifecycle stewardship, etc, we as engineers can position our work to meet these funding expectations.

4. EXAMPLES OF CE STRATEGIES IN DAM DESIGN

The essence of CE in dams is to design for future flexibility. In order to illustrate this, here are some conceptual practical examples.

4.1 Sediment Management

Sediment is the silent limiter of reservoir life. Left unmanaged, it gradually reduces storage capacity and undermines the long-term value of the asset. Designing for adaptability can however mitigate this risk. Incorporating low-level outlets in the dam body or providing bypass culverts through the basin, even if not used now, would allow future engineers to retrofit a sediment passing systems. Another strategy is to repurpose older dams nearing the end of their service life as sediment traps, extending the capacity and lifespan of downstream reservoirs. Such measures prolong value across a river system, not just within a single structure, but then the structure must be designed with this future purpose in mind.

4.2 Mechanical Works and Outlet Systems

Mechanical components, pipelines, and outlet works should be as simple as practicable. Complex pipe and valve systems should never be cast directly into concrete, where they become inaccessible. Instead, they must remain fully exposed or housed in conduits, allowing for easy operation, maintenance, and eventual replacement. The use of modular and standardised fittings further supports long-term serviceability and reduces dependency on obsolete parts.

4.3 Hydrological Adaptability

Design floods are not fixed quantities. Climate change is having an impact by increasing both their magnitude and uncertainty. For this reason, dams should allow space for future auxiliary spillways, fuse plugs, or other hydraulic features. By leaving room for such future works, designers today can create options for future engineers to adapt the dam thus lowering the long-term risk of the dam.

4.4 Reuse of Construction Materials

The reuse of construction materials, such as crushed concrete aggregate, is often promoted as a CE strategy. While promising in principle, it is only feasible if the material is fully traceable. Engineers must have confidence in its performance, which requires detailed records of the original source, placement and treatment during construction, maintenance history, and long-term behaviour. Without this data, few responsible engineers would choose a reused material over a new one. Traceability and perpetual record-keeping are therefore critical if material reuse is to be successful.

4.5 Access for Maintenance

Access is the simplest and most effective form of circular design. Galleries, for those dams that have them, should be large enough to accommodate drilling rigs and other equipment needed to maintain foundation uplift relief systems. If relief holes need cleaning, redrilling, or replacement, adequate space is essential in order for this work to be carried out safely and with much higher chance of success. More

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broadly, all areas of a dam should be designed to be safely, quickly, and effectively accessible for future maintenance. The easier it is to carry out a task, the more likely it is that the task will be carried out successfully. This also aligns with human centred design principles.

4.6 Structural Foresight

All dams age. The typical first signs of this are increased seepage through the various joints, zones or barriers. These seepage or leakage zones or conduits should thus be designed not just to carry expected seepage but also some measure of future seepage, also bearing in mind that these conduits themselves may block as they age. In a similar vein, circular foresight could also include providing sleeves or gaps in the superstructure for the placement of future waterstops. An associated aspect is future raisings, i.e. ensuring that the structure and the foundations are capable of accommodating high hydraulic gradients.

4.7 Decommissioning and End-of-Life Planning

Decommissioning is seldom considered at the design stage, but it is an increasingly important issue worldwide. CE thinking does encourage designing for the eventual dismantling of a dam. For example, segmented demolition, where the water level in a dam is lowered gradually over a number of years, allows deposited silt to stabilise in place, reducing downstream risks and enabling ecological recovery of the basin. Such foresight prevents the dam from becoming a long-term liability and instead enables a safe, managed transition back to a natural state.

5. DISCUSSION: BARRIERS AND OPPORTUNITIES

Us engineers and dam practitioners, may resist CE because it feels like external jargon imposed from outside. Yet many CE principles are already present in dam design, just described in different language. Enlarged galleries, provisions for raising, sediment bypasses are all good examples of value retention. The opportunity lies in reframing this work in CE terms, both to sharpen design thinking but also to align with the expectations of funders. CE offers us a systems thinking framework that encourages us to:

- Look at the big picture as well as the detail.
- Consider time-variant patterns and future needs.
- Think in terms of connections, not just isolated structures.

Barriers remain, however.

- Designing for adaptability can increase upfront costs, which clients may not be willing to cover.
- Predicting future needs over a century is often speculative at best.
- Cultural resistance to unfamiliar terminology is real.

Yet the opportunities are substantial. Lifecycle cost savings from refurbishing rather than replacing are well-established. Alignment with CE improves access to green finance and climate funds. Above all, CE reinforces the ethic of intergenerational stewardship: ensuring that today's decisions do not leave tomorrow's engineers with intractable problems.

Measuring CE's impact in dams should move beyond waste flows and instead adopt life cycle assessments. Above all, CE requires engineers to think like stakeholders, aligning technical choices with the values and needs of the communities served.

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6. CLOSURE

Circular economy may sound like a buzzword, but beneath the terminology and jargon is a philosophy well suited to dam engineering. At its heart, CE is about retaining value for as long as possible. For dams, it is simply a language for describing what dam engineers have long valued: longevity, adaptability, and resilience. Many of these ideas are already part of normal dam practice but the challenge is to recognise them as circular and to speak about them in terms that resonate with today's funding and policy frameworks.

In the end, CE for dams is not about chasing recycling metrics. It is about demonstrating foresight, thereby retaining maximum value for society, for the environment, and for the generations of engineers who will inherit these assets long after we are gone.

7. ACKNOWLEDGEMENTS

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MULTICRITERIA ASSESSMENT – A CASE STUDY FOR SELECTING THE RADIAL GATE HOIST FOR BOSTON HPP

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ABSTRACT

Dams and hydro-power projects are generally endeavours with significant economic, environmental and other impacts that can span over years. To complicate matters there are many different role players and criteria to be considered when selecting from a wide range of options available. Choices made, especially early during project development, significantly impacts the total life cycle cost, maintainability and success of a project. One of the main considerations used by principal decision makers is generally initial cost which is not necessary the best route to follow. The final choice should rather be based on a well-balanced, and transparent method that can be well documented with input from all stakeholders. Multicriteria assessment tools initially introduced after World War II is well suited for this and becoming frequently used for hydro projects to achieve this. The selection of the hoist system using this methodology for the intake radial gate for the Boston HPP is presented as a case study.

1. BACKGROUND

Water infrastructure development such as dams and hydro-power stations generally constitutes complex multi-dispensary projects with significant socio economic and environmental impact with long life cycles. Selection of site, layout and equipment selection has a mayor effect on the maintainability, cost effectiveness and overall success over the life cycle of the plant. Generally, Engineers and designers are thinking of solutions and analysing their feasibility while clients or project financiers generally makes the final choices.

Ever increasing available solutions and complexity of choices and their effect has resulted in a desire by all parties for making and conveying decisions on a well-balanced and transparent method that can be documented with input from all stakeholders. Multicriteria assessment (MCA) methodologies established after World War II, initially in the field of mathematics, was developed with similar objectives in mind. The principles have consequently effectively been adopted into the wider field of engineering for this purpose.

A wide and varied range MCA's methodologies have since been developed. All of them however have common aspects consisting of comparing considered options based on identified criteria and ranking them based on the outcome. There are many factors than can influence the selection of a hoist and should not just be based on initial cost only. The choice of system is therefore not necessary always clear. A weighted multi criteria matrix was consequently set up for selecting the hoist system for the intake radial gate for the Boston Hydro-power station.

2. HOIST OPTIONS

The intake to the Boston Hydro-power plant (HPP) feeder canal can be closed-off using a radial gate. Thie main function being to isolate the system for maintenance purposes and protecting downstream

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systems from flood events. Under normal operating conditions the gate is kept docked in the open position and would generally only be used once a year during the annual shutdown period. Historically radial gates are generally operated using one of the following means:

- Buoyancy drives.
- Chain Drives.
- Rack and pinion.
- Wire rope and winches.
- Hydraulic drives.
- Electromechanical drives.

Based on system layout operational requirements and hoist characteristics the above choices were reduced to three viable options for further evaluation and selection. The hoists considered comprises of the cable drum hoist, hydraulic cylinder and electro-mechanical drive, each of which is described in more detail below.

2.1 Option 1 – Cable and Drum Hoist

Cable drum hoist is one of the oldest technologies and have been used for over 100 years for hoisting and lowering radial gates. The hoist generally consists of two similar, but opposite hand cable drums, arranged to lift each end of the gate using a wire rope connected to the gate that winds around a drum(s) (hoisting) or un-winded when Lowering (Refer to Figure 1). The machinery drive train typically consists of an electric motor that drives one or more gearboxes. The final output shaft is connected to line shafts or torque tubes with pinion gears that drives a bull gear attached to the wire drum.

Synchronisation between the hoist units is automatically achieved as the drive shafts are mounted to the same output axle of the gearbox. The most common type of brake system used for these hoists consist of a magnetic shoe brake. The brakes are in the normally locked condition and releases when power is provided to the hoist. The operators for Boston would generally be familiar with this type of hoist as such a system has been used at Kruisvalei HPP, also manged by them.

Table 1 provides the advantages and disadvantages generally associated with cable drum hoist as it relates to hydro-mechanical equipment.

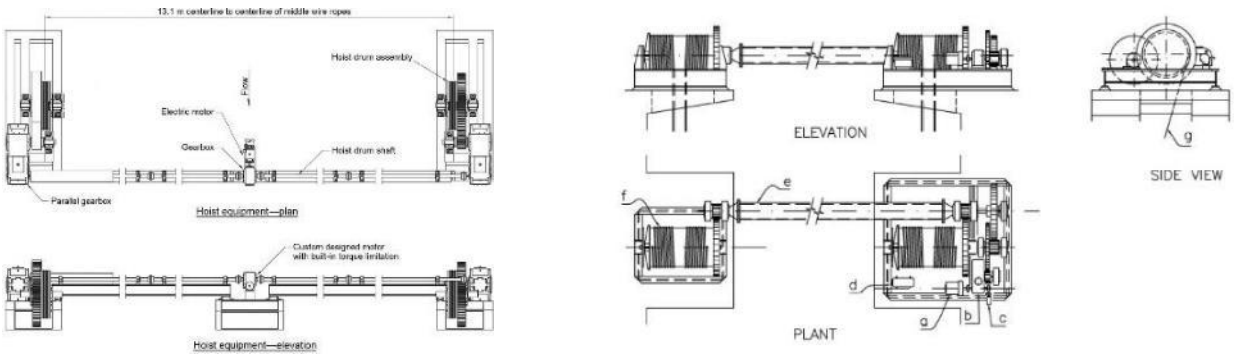

Figure 1. Typical Cable drum hoist layouts used for radial gates

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Table 1. – Cable drum hoist system advantages and disadvantage

Advantages	Disadvantage
• Operators are generally familiar with the technology as it has been around for a long time	• Rope, axles, couplings and gearing are subject to wear and may need to be frequently replaced (Expensive and time consuming).
• Hoisting point synchronising is automatically using a positive direct connection (Drive shaft) with wire ropes set at equal lengths	• Stretching of wire ropes can cause problems with the actuation of limit switches and with position indication system causing operational difficulties
• Convenient positive mechanical position indicator via direct coupling to hoist assembly	• Failure of a limit switch or of the solenoid brake can cause serious accidents
• Potential moment arm gain by mounting wire rope on upstream side. (Lintel seal requirement may negate upstream mounting of connection point.)	• Accurate setting and adjusting of cable lengths required to ensure equal load distribution at hoist points (Unequal loading can lead to a stuck gate, rope failure and or damage to gate structure
• Wire rope hoists can maintain gate in any given position without fear of drift.	• Gravity closure during electric power failure without providing additional mechanisms to prevent free falling not possible. Manual operation requires a separate load brake which adds to the cost
• Generally, more resistant to infrequent use	• Hoisting points, wire ropes, gears, sheaves, must be designed to withstand stalled condition) of the hoisting motor which can be sometimes as large as 300% of the normal pull e.g. adds to the cost.
	• Requires frequent inspection and lubrication on the gears, wire rope. Check for breaking of wire strands, stripping of screw threads on turn buckles, improper winding of rope on the drums, etc. as failure could lead to costly failure/damage
	• Flexibility of wire ropes can cause vibrational problems or variation in flow for gates operated in given positions due to wave action or other hydro-dynamic forces

2.2 Option 2 – Hydraulic Cylinders with power pack

Hydraulic operated hoist is largely becoming the preferred drive system over wire rope hoists in newer projects due to their versatility in their ability to offer variables drive loads and speeds. Larger forces can be transmitted by sizing the cylinder and operating pressures appropriately, making them more cost effective as the required lifting forces becomes higher.

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Hydraulic hoist units for wide radial gates generally consist of two hydraulic cylinders, one mounted on each pier and arranged to lift each end of the gate. Typically, the cylinders are trunnion-type, mounted in cardanic rings that are supported by hoist frames cantilevered over the side of the pier. The other end mounted to the gate using spherical bearings. Fluid power is provided from a central hydraulic power pack via hydraulic lines (Refer Figure 2). Most of the valves and controls are located at the power pack manifold and cylinder ports to minimise interconnecting piping and potential leakage failure points.

Hoist cylinders are kept in synchronism electromechanically by either, position indicators mounted internal to each cylinder providing feedback signals relative to cylinder stroke, to the control system. The system generates an error signal that is used to control a small proportional valve. The valve is used to bleed oil from the rod side of the lead cylinder, when rising, and from the rod side of the lag cylinder, when lowering. For small gates or gates that are infrequently operated, such as Boston's inlet gate, a simpler system utilizing a flow divider might provide sufficient synchronization.

Literature further indicates that these hoists also tend to be more cost effective in terms of operation and maintenance cost over cable hoists. (Sehgak, K). The systems however require the operator to be familiar with hydraulics and hydraulic circuitry. The emergency gate at Boston as well as some aspects of certain turbine is also hydraulically operated. It is thus considered that the operators would be well enough versed with these systems.

Table 2 below indicates some of the more common advantages and disadvantages associated with hydraulic hoist systems. Most of the disadvantages of these hoists can however be mitigated. If basic maintenance is done in keeping oil clean by maintaining oil filters and breathers their cost could generally be kept to the replacement cost of the filters and breathers.

Figure 2. Typical hydraulic cylinder mount and central power pack (Mobile Emergency unit centre right)

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Table 2. – Hydraulic Hoist Advantages and Disadvantages

Advantages	Disadvantage
• Hydraulic hoist can provide a push and pull operation.	• Leakage of hydraulic oil can cause environmental Hazard (Advances in bio friendly power fluid can help mitigate).
• Operational forces relatively easily adjusted by means of changing operating pressure. Relatively large forces can be obtained at low speed without the need for complicated reduction gearboxes.	• Sluggish in cold temperatures (Mitigated using correct oil for environmental temperature e.g. special oils rated to be operational to below -30ºC but is more expensive)
• Operational speed adjustment by changing flow control valve settings as well as possibility of different opening/closing speeds	• Occurrence of gate drift over time due to oil leakage across cylinder piston or other points in the hydraulic system. (Drift control circuit can overcome problem)
• End stroke cushioning relatively easily provided to prevent damage to gate.	• Cleanness and filtration of oil essential to prevent damage to valves and clogging of ports
• Maximum hoisting force can be limited by setting pressure relieve valve.	• Mechanical driven position indicator can be elaborate. Electronic position indicators not as reliable e.g. movement of gate without power would require adjustment and recalibration of units.
• Several gates (Usually 2) can be operated using the same hydraulic power pack – economic benefit. Power pack can also be located short distance from gate	• Synchronising of cylinders for wide gates requiring 2 or more hydraulic cylinders generally needs expensive electronically proportional control valves.
• Gate can be lowered without power by opening shut-off valves (Gravity controlled) and use of accumulators driven closure	• Hydraulic system control panel generally more elaborate because performance of several components such as filters, pressure switches, pumps and accumulators needs to be monitored.
• Few moving parts compared to wire rope hoist. Hydraulic fluid automatically lubricates parts thus reduced wear.	• Operation can be influenced by solar heating – Long exposed pipe layouts.
• Hydraulic power pack can easily accommodate one or more standby pumps / motors – improved redundancy in system.	
• Overall mechanical efficiency is higher for hydraulic systems (85-90%) compared to wire ropes (75-80% or less if worm gear is used)	

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Advantages	Disadvantage
• Pilot operated check valves provide an excellent means of holding gate still in a given position in conjunction with dampening effect of cylinders against hydro-dynamic forces.	

2.3 Option 3 – Electrically-Mechanical Drive

The electro-mechanical drive considers is comparable the hydraulic cylinder hoists in that they resemble hydraulic cylinder hoist and actuation. Their operation mode is however different in that load transfer is mechanical i.e. there is no oil or other fluid involved. The heart of the system is a spindle (screw actuator) driven by an electric motor through a gearbox. The directly driven component is a lead nut mounted on a threaded piston rod. The rotation of the nut causes the extension or retracing of the piston rod that, in turn, drives the gate (Refer Figure 3).

Mounting to piers and gate would be similar to that of a hydraulic hoist with synchronisation done electronically. The system also does away with the requirements for a separate hydraulic power pack and oil pipe connections etc. which could affect ancillary costs. As far as it is known these kinds of hoist have not yet been used for hydro-mechanical equipment locally. These drives are however particularly favoured by a growing number of European engineers due to advantages in the technology and their reliability.

Preferences for these drives are further partly motivated by several issues that have been experienced with hydraulic cylinders in gate drive systems. Although the initial cost of the electric motion system may be more costly than other actuator options, the increased efficiency of the total operation coupled with the little to no maintenance required over its life span could make the total cost lower in comparison with other types of actuators.

Usage of electric actuation is favoured when the application requires speed, accuracy, flexibility, and control. The no- pressure lubrication an enclosed environment gives the cylinders a long life, but also, they decrease the use of lubricants, making mechanical cylinders one of the cleanest options for gate drives. Reliability is also potentially higher, because there are simply less components to fail. Figure 3 below indicates some of the advantages highlighted in suppliers' catalogues, literature and some anticipated disadvantages.

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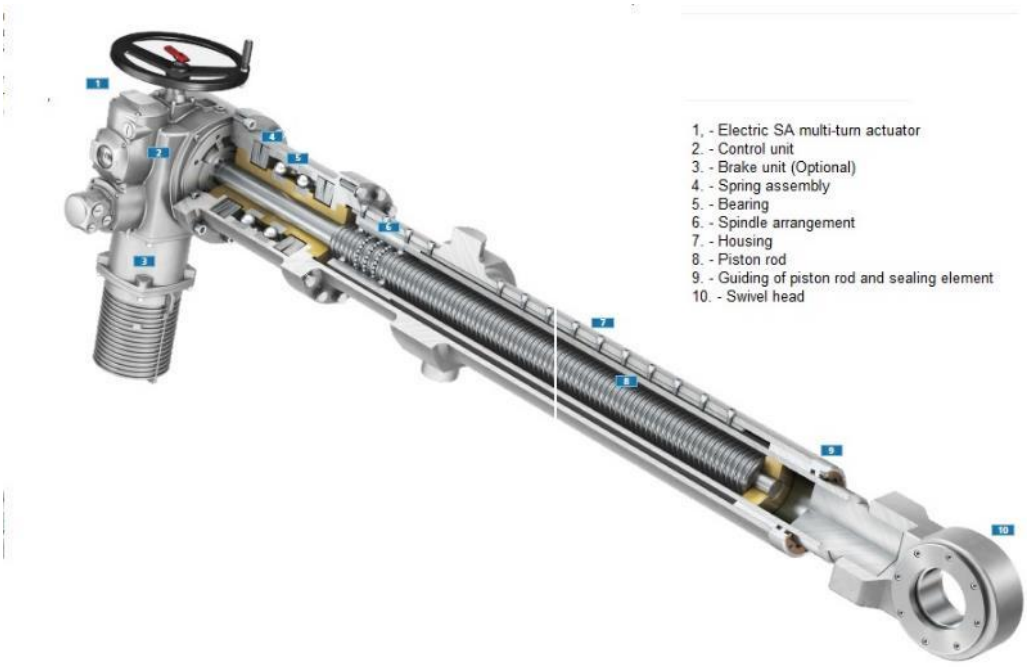

Figure 3. Typical layout of an electric actuated cylinder (Courtesy Auma)

Table 3. – Electromechanical Actuated Cylinder Hoist Advantages and Disadvantages

Advantages	Disadvantage
• Simple installation and commissioning	• Initial cost per force and stroke length higher than comparative hydraulic cylinder and Hydraulic power pack (HPC)
• Low operational and maintenance cost (Generally maintenance free for 10 years depending on operation)	• Not suitable for explosive environment. (Methane accumulation in galleries can be problematic)
• Easy integration with various control schemes. No additional external sensors and limit switches required.	• Generally, more sensitive to Vibration and shock loading. Advances design and shock absorbing support has improved shocks and load peaks vibration absorption
• Continuous travel measurement, position feedback and force monitoring generally integrated as standard	• Highly specialised technology unlikely that operator would be able to fix without supplier training or intervention
• Environment friendly contained unit with low risk of oil or lubricant contamination. IP68 with pressure less lubrication	• Increased speed lower thrust and vis versa. Limits operational parameters (Can impact size and initial cost if higher thrust and speed is required)

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Advantages	Disadvantage
• Suitable for low temperatures without additional heating or warming circuits required.	• Operation during power failure would require an external/ manual input. Generally, actuators have a build-in manual hand wheel. Synchronisation of two hoists may be problematic.
• Self-locking or non-self-locking option available. Self-locking option allows for long term position control without creep or when loss of power occurs. Non-self-locking can incorporate gravity controlled fail safe options	• Possibility of overheating with abnormal duty cycles or change in duty cycle
• Adjustable, variable speed over operation profile (acceleration, deceleration for soft stop)	• Larger cylinder bore required for similar sized rated hydraulic cylinder with associated connecting points
• Synchronisation between actuators done relatively simple by means of electronic control	
• Smaller footprint than both drum hoist and hydraulic systems requiring bridge structures and hydraulic power packs.	

3. HOIST SELECTION CRITERIA

The criteria and their weight used in setting up the Multi criteria Assessment (MCA) matrix is summarised in Table 4. The criteria, weight and scoring for the different options are discussed in more detail below. It should be noted that below criteria was deemed sufficient as baseline to compare the various options. Depending on project objectives the criteria can be expanded for items that can suitably quantified e.g. system reliability or environmental risks be factors.

Table 4. – Summary of Evaluation Criteria, scoring and importance of criteria

Criteria	Evaluation Definitions	Evaluation Score
Initial Cost *(Weight 5)*	Scoring based on the initial cost of the various hoisting systems with a variance of more than 20% using the lowest cost as base	5 = Lowest Cost 4 = <20% Higher 3 = <40% Higher 2 = <60 % Higher 1 = >60 % higher
Visual Impact (Weight 3)	Scoring based on the perceived visual impact that system will have on the plant (Items located above channel NOC considered to have an impact).	5 = No Impact 4 = Slight Impact 3 = Moderate impact 2 = High Impact 1 = Significant impact

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Criteria	Evaluation Definitions	Evaluation Score
Hoist Synchronisation (Weight 5)	Wide gates operated using two hoist points requires that lifting points are synchronised during lifting and lowering operations. Hoist system has varied complexity in achieving synchronisation and need for adjustment over its operational life.	5 = Permanent synchronised 4 = Some adjustment 3 = Moderate adjustment 2 = Complex adjustment 1 = Not synchronised
Speed control and Adjustability (Weight 4)	Hoisting speed should preferably be controllable over gate motion to reduce shock loading and impact loading on gate. Also, perceivable that requirements may change in future based on operational conditions	5 = Fully adjustable 4 = Highly adjustable 3 = Moderately adjustable 2 = Limited adjustability 1 = Not adjustable
Power Failure and Redundancy (Weight 2)	Consideration as to the operation of hoist in event of power failure or faulty hoist motor.	5 = Fully operational 4 = External Drive 3 = Limited 2 = Fail safe 1 = Not usable
Frequency of Operation (Weight 3)	Operation of the gate would generally only occur annually during planned plant outages	5 = Annually 4 = Six Monthly 3 = Three Monthly 2 = Monthly 1 = Daily
Operating and Maintenance Effort (Weight 4)	Effort in maintaining and operating equipment impacts the effort in running and success of the scheme	5 = Low 4 = Low to Moderate 3 = Moderate 2 = High 1 = Significant
Future Integration (Weight 1)	The inlet hoist is not connected to the plant control system it is not considered necessary due to the planned gate operating philosophy. It is however conceivable that some integration could be required in future.	5 = Plug and Play ready 4 = Minor Modification 3 = Notable Modification 2 = Significant modification 1 = Not Possible

3.1.1 Initial Costs

Initial costing of equipment is one of the key selection criteria and thus also given a weight factor of five. Ranking of equipment is done by providing the lowest costing system a score of five. The remainder reduced based on the percentage difference of their cost relative to the lowest. Costing of the wire rope system used in this evaluation has the highest level of uncertainty as design aspects such as the bridge for mounting and detailed aspects of the drum hoist drive train was not developed in sufficient details to obtain a firm contractor's price.

Pricing for the wire drum hoist is thus based on a high-level engineering estimate. Supplier/Contractor prices were obtained for both the electric cylinder and hydraulic hoist system. Based on the engineering estimate the rope drum hoist system has the lowest initial cost impact at R 1 562 400. This is opposite

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of the norm based on a study done in 1987 (Sehgak, K) indicating that the installed cost for rope drum hoists to be generally higher than that of hydraulic hoist units. This could be an indicator that the engineering estimate for the rope drum hoist might be on the low side (Refer Figure 4). The difference is however less than 20% than the cost for a hydraulic system at R 1 631 248 and thus scored similarly.

The electrical hoist is most expensive system at R 2 660 474. It is believed that the initial cost for the electric cylinder hoist system is strongly impacted due to the weak exchange rate as the system is mostly imported were as the other hoist units have a strong local content.

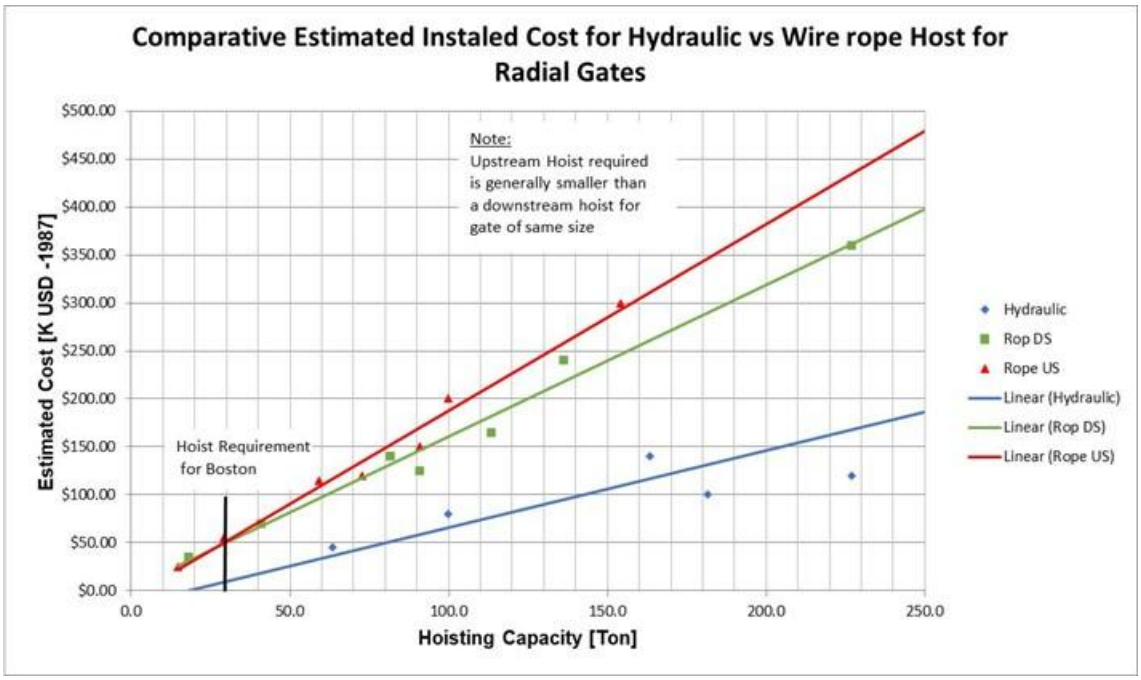

Figure 4. Cost Comparison study for hydraulic vs wire rope hoist (USD in 1987)

3.1.2 Visual Impact

Hoisting equipment impact aesthetic appearance of the plant. Visual impact was indicated as an aspect with some importance to the Client but not considered a deal breaker and thus given a weighed factor of 3. The significance of the impact is however difficult to judge as it is very subjective. For evaluation purposes the impact is appraised based perceived sections of plant that would be above the inlet canal NOC level.

Electric cylinders would have a slight impact as this would be limited to the back ends and mounting brackets. For hydraulic cylinder hoist an additional control house for the power pack is required and therefore considered to have a moderate impact. The cable drum hoist is however deemed to have a high to significant impact as it would either require locally raised canal wall section corbels with a cross-connection shaft or a steel bridge in proximity of the downstream heavy-duty traffic bridge which might make the hoist bridge structure look out of place.

3.1.3 Hoist Synchronisation

Wide gates requiring and operated using two hoist points necessitates that lifting points are synchronised during hoisting and lowering operations. This criterion significantly affects the structure and reliability of operation and thus provided with a weight of five. Electric cylinders are synchronised

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electronically by ether direct cable connection, wirelessly or combination of both (redundancy). Internal limits switches and on-board control would further automatically adjust or provide necessary error signals if fault conditions occur. Synchronisation would thus be relatively simple with slight adjustment and setting up required during commissioning or maintenance conditions.

Rope drum hoists are inherently synchronised as the drum units are linked by means of drive axles/ torque tubes. Aligning drives and adjusting wire rope length are however required during commissioning. This would also need to be checked and adjusted periodically by the operated due to cable stressing, thermal expansion/contraction and wear of the wire ropes. Rope lengths are generally adjusted using turnbuckles where the rope is attached to the gate and requires moderated skill and judgement from the artisans performing the works.

Hydraulic system uses flow dividers or expensive proportional flow control valves. Setting up and commissioning can be complex and requires a hydraulic specialist. It is further conceivable that the system would also need periodical adjustment. The more expensive proportional control valves with feedback signalling can adjust automatically. Adjustment of simpler flow control would require observation from the operator to note that gate is not lifting or lowering equally anymore (similar to wire rope).

3.1.4 Speed Control and Adjustability

Variable speed actuation of the gate is preferred as large sized equipment and mass is to be moved. They allow the operating speed to be adapted across the travel. This would allow for soft starts as well as gentle approaching of end positions. Due to the size and weight of the gate this criterion is given a weight of 4. Electric cylinders can generally be programmed directly on their modulating actuator on board control and considered highly adjustable. Hydraulic hoist speed can be adjusted using a flow control valve and combined with cushioning at the end of the cylinder strokes and deemed to be moderately adjustable. Generally, cable hoists used for radial gates are not adjustable. This could be implemented using a variable speed drive to control the electrical hoist motor. This would come at an increased system cost and complexity and thus not considered practical for this gate.

3.1.5 Power Failure and Redundancy

Operation of hydro-mechanical equipment during power failure is generally considerations of significance as it can influence safety of the infrastructure and surrounding. This is less so for the Boston HPP intake radial gate as it is normally open and generally only closed for maintenance and in the event of a high flood conditions to protect the canal. There are also further redundancies to safeguard the downstream plant in terms of an active spillway gate and emergency gate. For this project it is thus given a lower weight of two.

Hydraulic systems can generally be made fully operational during power failures by using accumulators. The use of accumulators for lifting gates from fully closed position is nevertheless considered uneconomical especially for the gate size and stroke length required for Boston. Accumulators would thus generally only be used for small lifting purposes i.e. lift gate for undocking. This can however be simplified by means of a cheaper and simple hand pump operation as the docking pin for this gate is not hydraulically driven. This would enable the operators to close the gate using gravity by manually opening the return line valve without the need for an external drive.

Opening the gate using a hand pump from the fully closed position, although feasible, would be impractical. An external drive would be required. Typically, this would consist of a petrol or diesel driven hydraulic pump that can easily be connected to drive the power pack (Refer Figure 2). Building in redundancy into a power pack is relatively simple and cost effectively by providing two motors and pumps i.e. one operating and one standby in the power pack.

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Both the wire drum drive and the electric cylinder hoist would generally be considered "fail safe" in that the gate would be kept in position in the event of a power failure. Closing or opening would however require some form of external drive (and means of overriding the brake for the drum hoists) to change gate position. Actuators driving the electric cylinders are fitted with manual overrides that can be driven by mean of a socket drive.

Both lifting and lowering can this be achieved but would however require two operators, one at each of the hoists, working in tandem and closely monitoring synchronisation. Operation is thus considered limited. Providing external drives and redundant drive motors for the drum hoist is believed to be costly for this particular gate. The drum hoist system for this gate is thus only scored as being "fail safe".

3.1.6 Frequency of Operation

Operational personnel have indicated that frequency of gate operation is a concern for them, as the gate would normally only used once a year during the annual plant shut down for maintenance. The choice of hoist unit thus needs to consider suitability and impact of low frequency of use. Both Rope drum and hydraulics are used on dam infrastructure, where in the South African context they are not operated as frequently as they should be due to negatively impact on water storage, downstream users and in Boston's case the generation of electricity. In many cases it would also only be annually before the rainy season occurs.

Frequency of use however does not only impact the reliability of the hoist but could also affect other crucial elements such as trunnion bearing friction and seal friction. The requirement for more frequent exercise could thus be more beneficial to the overall health of the gate but not taken into consideration for the purposes of this evaluation.

It is perceived that cable drum hoist can be stationary for extended periods of time without a significant effect as it does not depend on oil seals requiring lubrication or hydraulic ports that can get stuck due to being inoperative for an extended time. Although not recommended, it considered that they would be suitable for only annual operation and operational checks for the purposes of evaluation.

Hydraulic hoist has been known to be inoperative for extended periods without affecting the system. The risk in oil leakage due to hardening of seals or "sticky" valve operation however increases significantly. It is therefore considered that one should at least work the system every three months. This can be done by just moving the gate from the open to dogging position without affecting flow to the power station.

Performance of the electric cylinder if kept stationary for extended periods of time is not known. The units are however fully self-contained and enclosed without the need for a pressurised lubrication system. It is consequently therefore believed that they would also have a satisfactory to good performance, even if only operated annually, and is therefore scored similarly to that of the Cable drum hoist.

3.1.7 Operation and Maintenance Effort

Long-term operating and maintenance effort and plays a major part in the running of a scheme. The electric cylinder hoist with a correct duty cycle is rated to have high energy efficiency with little to no maintenance effort for at least ten years. The main requirement for the operation and maintenance of hydraulic systems is the cleanliness of the hydraulic fluid as the oil also service as lubrication reducing system wear. If is oil kept clean the units can have a long service life with low to moderate effort in terms of replacing and checking oil filters and replacing aged oil.

Hydraulic systems can be seen to be complex requiring skilled operators to service and fault find. There is however multiple other hydraulic systems forming part of the plant. It is thus considered that the plant

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operators at Boston would generally be well versed in these types of systems and would have the required skill.

Cable drum hoist requires frequent lubrication of all the drive axle bearings, gearbox, hoisting cable etc. Cable lubrication can be challenging as on would need to lubricate the entire cable i.e. gate needs to be run through complete open and closing cycle while ensuring that applied oil penetrates the cable sufficient. Servicing and repair gearbox could require specialist that are not necessarily available locally. Brake and brake lining needs to be checked and set. Failure of the limits has caused incorrect winding of hoist with subsequent rope failure causing compete gate structural failure. Maintenance requirement for the hoist is this considerate to be high to moderate.

3.1.8 Future Integration

The current operation philosophy and requirement of the intake gate is that it is operated manually as required by the operators. The system consequently does not require providing any feedback or control signals to the power plant main control system. This might change depending on future operational requirements. The ability of adapting the hoist for this scenario is evaluated based on the considered effort in modification required. The electro-mechanical drive is considered to be "plug and play" ready as the unit would already have usable signal input and output ports. Notable to significant modification would be required to integrate the hydraulic system and cable hoist.

4. HOIST SCORE RANKING AND SENSITIVITY

Based on the weighed scoring the hydraulic hoist solution has the highest rated score with the electric cylinder hoist and cable drum a close second and third (Refer Table 5). The closeness of the scores would suggest that there would be a high degree of sensitivity towards weighing factor and hoists bias. Reducing bias would require input and scoring from more people. Sensitivity of the results is however checked by changing scoring and weighing parameters.

Initial costing of the cable drum has a high uncertainty. A reduction in cost of 25% would favour the electro-mechanical drive while making the hydraulic hoist the least favourable choice. Increasing its cost by 25% has no effect on the hoist selection ranking. In both cases the cable drum is not the preferred hoist system. It is thus believed that the engineering cost estimate for this hoist does not penalise the cable drum evaluation.

Sensitivity towards the weight criteria is checked by setting the weight for all the criteria to be the same. This results in the electro-mechanical hoist scoring the highest with the hydraulic hoist and drum hoist respectively with a score of 94,4% and 76,9%. Although scoring generally well over all criteria the large initial cost difference and weight is reducing the appeal of the use of this hoist for Boston. The hydraulic hoist is however still relatively close second.

Visual impact is a very subjective criterion to evaluate as it would be in the eye of the beholder. One person's high impact might be defined as a moderate impact or a significant impact by someone else. Changing the cable drum scoring from 1 to 2 changes the ranking for the cable drum to 94% and reduces the electro-mechanical hoist to 93,3%. The difference between the hydraulic system and cable drum is further reduced if power failure weight is reduced to 1.

Initial cost of the electric cylinder hoist is the only real negative aspect that influences the use of the hoist at Boston. The cable drum hoist only becomes the preferred choice if the initial cost is at least 15% lower than the engineering estimate used in combination with a reduced visual impact and maintenance effort score.

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Table 5. – Weighted score and hoist ranking

Nr.	Criteria	Weight	Cable Drum	Hydraulic cylinder	Electric cylinder
1	Initial Cost	5	5	5	1
2	Visual Impact on Plant	3	1	3	4
3	Hoist Synchronisation	5	4	2	4
4	Speed control & Adjustment	3	2	3	4
5	Power failure & Redundancy	2	1	4	3
6	Min Frequency of operation	3	5	3	5
7	Operation and Maintenance effort	4	2	4	5
8	Future Integration in plant control	1	2	3	5
	Weighted Score		**81**	**89**	**83**
			91%	**100%**	**93.3%**

5. SUMMARY

Three options for hoisting systems were checked for the intake radial gate for Boston HPP. Al three hoists are deemed suitable and the choice influenced by personal preferences. From an engineering perspective the electro-mechanical hoist seems to be the best based on technical evaluation criteria. Initial costing however does not favour its selection. The cable drum would, from simplicity of operation, be favoured. It does however come at an unappealing visual impact as well as scores lower on adjustability and other operational limits. Hydraulic operated hoist based on this assessment appears to be the best compromise between cost and technical ability and thus suggested as the hoist system of choice for the intake Radial Gate. The assessment further indicates that with overall consideration the most expensive solution should not necessary be outrightly be rejected.

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Table 6 Maintenance requirements

	Wire Rope Hoist	Hydraulic Cylinder	Electric Cylinder
Before Every Operation	• Check wire rope condition • Check gearbox oil level and leakage (Sweating at glands & filling points) • Check electrical system indication lights	• Filters: check at start and end of each operation • Check for leakage (visually) • Oil tank: check fluid level • Check electrical system indicating lights	• Visually inspect cable entries, cable glands, blanking plugs for correct tightness and sealing • Check connections of cylinder to gate
Every Three Months	• Lubricate rope drum & sprocket bearings • Lubricate sliding parts (wire rope). • Check condition of sheaves for wire rope hoists		
Every Six Months	• Check Electrical wiring connections for deterioration • Check calibration of position indicators • Check condition of turnbuckles and U-bolts (gate to hoist connections) • Perform run test (If not operated frequently)	• Accumulators: Check pressure, clean parts & inspect for damage and wear • Visually check for leakage and clean outside of valves. Manually actuate several times to prevent sticking of the spool and gumming of the oil • Accessible piping and hoses: Inspect and clean outside. Check for leaks & damage. • Hydraulic Oil: Check for aging and contamination (Generally replace every 2000 hours of operation) • Check electrical wiring and connections. • Actuators: Visually inspect for leakage • Check calibration of position indicators • Perform run test (If not operated frequently)	
Every Twelve Months	• Check wire ropes thoroughly for damage, deformation and uniform tension • Check all couplings for alignment and coupling seals for damage • Check brake linings and correct extension of the solenoid brake when the brake is applied • Check for smooth operation of electric motors • Check electrical equipment including functions of circuit breakers, selector switches, push buttons, etc.	• Hydraulic Pumps: Check for smooth operation and leakage • Oil Tank: Check all accessories including cleanliness of the oil filler and level switch (remove level switch to check) • Electric Motors: Check for smooth operation • Hydraulic Cylinder Piston Rod: Inspect for Foreign particles and clean. (Also inspect for damage to the chrome plating). • Check electrical equipment including function of circuit breakers, selector switches, push buttons, etc. • Check with operator for frequent excessive drift and for sluggish operation and correct the problem areas	• Visually inspect cable entries, cable glands, blanking plugs for correct tightness and sealing • Check fastening screws between actuator and electrical cylinder and gearbox • Perform test run (If not operated frequently) • Check for corrosion or other damage and patch repair as necessary
Every Five Years	• Touch-up paint or repaint if necessary • Check slack rope and over load settings	• Touch-up paint or repaint if necessary	

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	• Load Brake: check brake adjustment and lining • Renew grease in electric motors (renew after 5000 hours of operation, if that occurs before 5 years)	• Renew grease in electric motors (renew after 5000 hours of operation, if that occurs before 5 years) • Hydraulic Pumps: Disassemble ends, check for worn parts and clean • Check working and setting of limit switches, timers, pressure switches, etc.	
Every Ten Years			• IP68 components to be lubricated every 10 to 12 years if operated rarely (open close duty cycle) • Exchange seals during lubrication

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GEOMEMBRANE FACINGS TO CONSTRUCT SAFE EMBANKMENT DAMS MINIMISING TIME AND COSTS

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ABSTRACT

Flexible geomembranes have been used in new embankment dams since 1959, since the 2000s as only upstream water barrier. They accommodate deformations and differential displacements that can damage more rigid water barriers and their connections to ancillary structures. Geomembrane facings are installed more quickly than concrete and asphalt concrete facings, site organisation is simpler, costs and environmental impact are lower, underwater repairs are possible. The paper outlines design options and discusses two recent projects: in Côte d'Ivoire, where a geomembrane substituted the original concrete facing, and in Peru, geomembrane staged installation on a tailings dam already 198 m high.

1. INTRODUCTION

Dams play an essential and indisputable role in irrigation, water supply, flood control, hydropower generation, recreation, navigation, aquaculture, and tailings management. However, climate change is likely to intensify the challenges associated with the design, construction, and maintenance of safe dams. Developing and exploiting safer, more resilient, and sustainable technologies is a responsibility of all professionals operating in the field of dams.

According to all agencies and entities, embankment dams are the most common type of dam, in the range of 74-77% worldwide. According to ICOLD, the International Commission on Large Dams (ICOLD 2025), the vast majority (67%) are earthfill dams, and 7% are rockfill dams. In earthfill dams, watertightness is generally achieved through an impervious core of compacted clay or other low-permeability materials. Rockfill dams, being pervious, need a water barrier, which can be an impervious core if material is available, or an upstream water barrier. Traditionally, this barrier was made with concrete or asphalt concrete.

After the second World War, with the improvement in the formulations and manufacturing technologies of synthetic materials, thin deformable prefabricated and practically watertight membranes (geomembranes) became available to civil engineering. At the end of the 1950s, geomembrane started being used in new embankment dams, in covered position and generally as partial or secondary water barrier. After a few pioneer projects in the early 1980s and 1990s (ICOLD 2010), where the geomembranes were installed without a cover layer, since the beginning of this century geomembrane facings are used as the only water barrier in embankment dams. The upstream position maximises safety because if the geomembrane facing is efficient there is no pore pressure or seepage pressure in the entire dam body, and the water pressure from the reservoir stabilises the upstream part of the dam. Furthermore, the geomembrane will always be accessible for inspection, and for repair if needed.

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The number and performance of projects with exposed geomembrane facings is such that the terms GFRD (Geomembrane Face Rockfill Dam) and GFED (Geomembrane Face Earthfill Dam) are becoming of common use among dam designers.

The paper, after a brief presentation of the available design options, focuses on two projects related to rockfill dams of recent or ongoing construction; examples of watertightness granted by exposed upstream geomembrane facings with similar geomembrane systems are available also for earthfill dams (Mtkvari in Georgia, Bulga in Australia, Murdhari in Albania, all waterproofed by the same material discussed in the case histories that follow).

2. WATERTIGHT FACINGS OF EMBANKMENT DAMS AND THE ADVENT OF GEOMEMBRANE FACINGS

2.1 Context and constraints

The watertight facing of embankment dams is a pivotal design choice. It not only governs seepage control, and therefore it ties directly to public safety and to environmental protection, but it significantly influences the construction schedule, the overall costs, and the long-term operation and maintenance of the dams.

Conventional solutions — CFRD (concrete face rockfill dam) and ACFRD (asphalt concrete face rockfill dam) — are well-established, thoroughly understood and can be durable. However, experience has shown that rigid facings struggle where foundation heterogeneity, differential settlements, and seismic loading, are significant. As a result, seepage can occur through cracks or holes in the facing, through failing waterstops, or due to poor interface performance where the embankment intercepts concrete structures such as plinth and intakes. CFRDs have achieved remarkable heights (e.g., Dashixia, 247 m) but have revealed some difficulties when very high dams are at stake, and slab and joint behaviour under complex deformation fields has led to cracking, spalling, and joint distress. In ACFRDs, structural defects during service include blisters, cracking, slump, and bulges related to construction, when the impervious layer is placed in more than one course. Other problems associated with asphalt concrete are stripping and instability on the slope, resulting from creep caused by a very high bitumen content. Fatigue cracking due to temperature changes, and ageing, which can be driven by temperature and UV radiation, are additional issues. ACFRDs improved mix design and interfaces, yet temperature-rate effects, creep, and aging remain concerns.

Concrete and asphalt concrete facings carry heavy site logistics (batching plants, formwork cycles), curing windows, and for asphalt concrete thermal windows, which amplify the risk of delayed commissioning, critical especially in countries where it can affect water supply or economic growth, and in mining, where tailings capacity must track production schedules. The overall cost of watertight facings is not only the summary of Capital Expenditures (CAPEX) for design, supply of materials, site organisation, construction, of Operating Expenditures (OPEX), including costs related to maintaining the facings in good operating conditions, therefore potential long-term rehabilitation expenses, but must also consider the lost revenues due to delayed commissioning. The potential costs related to carbon emission should also be considered, as discussed in section 2.3.

2.2 From rigid to flexible facings

From a technical standpoint, the main reasons to adopt a flexible geomembrane facing reside in the tensile characteristics of the material, in the design of the peripheral sealing to the plinth and to the concrete appurtenances, and in the improvements that can be attained in the design of the embankment itself.

The material commonly adopted for GFRDs and GFEDs is a composite geomembrane (geocomposite), SIBELON® CNT, consisting of a polyvinylchloride (PVC) geomembrane plasticised with high molecular

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weight branched monomeric plasticisers, heat-bonded at manufacturing to a backing anti-puncture geotextile. The discussion on the reasons for selecting a PVC geomembrane instead of other types of geomembranes is not within the scope of this paper and is extensively documented in the literature (e.g., Perazzo & Vaschetti 2022). SIBELON® geocomposites, which being produced in a controlled factory environment have constant physical and mechanical properties, exhibit an elongation exceeding 50% when the backing geotextile breaks, and an ultimate elongation > 250%, which corresponds to the geomembrane elongation at break; such values largely exceed what would be required for the settlements generally accounted for in the design of the dam, which on the contrary can exceed the resistance of concrete or asphalt concrete facings.

For both CFRDs and ACFRDs, the connections between the embankment and the concrete plinth/concrete appurtenant structures are especially critical, due to the differential settlements occurring at such connections. A flexible waterproofing geomembrane, with a monotonically increasing tension-elongation diagram such as SIBELON® CNT, has a superior capability of accommodating differential displacements (Giroud 2005). Submersible seals have been designed and tested for water heads higher than 850 m (Avila et al. 2018), and for embankment dams have been conceived to be capable of absorbing the differential displacements along stiff concrete structures.

A SIBELON® geomembrane facing permits to build the embankment on highly deformable foundations, allows for steep upstream faces, which means less volume of fill, further reducing construction times and costs, and some mitigation of the zoning design.

More detailed technical comparison of rigid and flexible facings can be found in recent literature (Cowland et al. 2025).

2.3 Project delivery, sustainability

Geomembrane facings do not require complicated site organisation, because all materials and installation equipment have relatively small volume and weight, there is no heavy transport affecting the environment and the communities, there are no constraints imposed by weather conditions. This allows flexible scheduling and faster waterproofing compared to traditional facings; installation can be staged, providing early protection against floods and earlier impoundment.

Sustainability is related to the carbon footprint at construction, to the lifecycle costs, and to possible long-term issues. The carbon emissions of concrete and asphalt concrete facings are high due to the impact of material production (cement production, steel reinforcement, refining processes that emit significant CO_2, aggregates that require quarrying, crushing, and transportation, all of which contribute to emissions), and to the energy required for installation: both types of facings are characterised by energy-intensive construction. On the contrary, polymers have lower embodied carbon per square meter, the manufacturing of geomembranes emits less CO_2, and the installation energy is lower (lightweight, less site organisation, less machinery, less traffic, faster installation). Lifecycle costs are higher for traditional facings, in terms of both of CAPEX (higher cost of materials equipment and labour) and of OPEX (routine maintenance needed). Geomembranes have the additional advantage of underwater repairability. In the long term, traditional facings may incur in carbon taxes or require offsets due to high emissions, while geomembrane facings will have a more favourable situation under carbon pricing schemes. The end-of-life disposals of concrete and asphalt concrete facings are energy-intensive, while geomembranes can be recycled, though not always economically. In summary, geomembrane systems are the most sustainable waterproofing technology for dams.

2.4 Design options

An exposed geomembrane must be anchored against uplifts (wind, unbalanced back-pressure, waves) by a diffused face anchorage. Anchorage at points, and along lines parallel to the slope, are the feasible options.

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Anchorage at points is mostly adopted when loads are not very demanding, otherwise the stresses on the geomembrane at the anchor points can become excessive. The pattern and spacing between anchors are designed based on the loads, on the characteristics of the geomembrane, and on the friction at the interface with the support layer. The anchors can be of the "Duckbill" type or of the grouted type. The Duckbill system consists of a steel rod used to insert the anchor into the embankment, and of a steel cable. After the anchor has reached the desired depth, the rod is removed, and the steel cable is pulled so that the Duckbill anchor rotates from a vertical to a horizontal position, mobilising the resistance of the soil. Traditional grouted anchors, which have higher strength and can be used also in rock subgrades, are installed by drilling a hole in the embankment, inserting the anchor after it has been cut to the desired length, and injecting the hole with grout or mortar. The SIBELON® waterproofing geocomposite is anchored to the dam face by punching/clamping the liner over the anchors. A stainless-steel disk, an anti-puncture geotextile, and a SIBELON® geomembrane cover roundel, ensure watertightness where the anchor crosses the waterproofing liner. The advantages of point anchors are short installation times (especially with Duckbills), adaptability to different site conditions, and possibility of implementing the solution even at a late stage of construction. Successful precedents at Vaité earthfill dam (2011), at some of the 18 water saving basins of the new Panama Canal (2015), at Filiatrinos hardfill dam in Greece (2015), and at Ambarau hardfill dam in Congo (2017), have shown that this type of anchorage system can be installed in any kind of subgrade, quickly and with proven reliability.

Anchorage by lines is however preferable because the distribution of stresses on the geomembrane is much more adequate (ICOLD 2010) than that achieved with point anchors. The patented anchorage systems developed by Carpi for new embankment dams are based on the concept of embedding in the finishing layer of the embankment parallel SIBELON® geomembrane anchor bands, at spacings based on the parameters mentioned above, and then achieving the anchorage of the waterproofing geocomposite by heat-seaming it upon the geomembrane anchor bands. The way of securing the geomembrane anchor bands to the embankment is selected based on the construction procedures adopted for the embankment. All configurations include a drainage layer under the waterproofing liner.

A simple and effective system for fastening the anchor bands to the embankment consists in excavating or forming in the base layer parallel trenches, in placing the SIBELON® geomembrane anchor bands in the trenches so that a part of the band protrudes from the trench, and in securing the bands by backfilling the trenches with granular material or porous concrete, which provide the ballast for the anchor bands, and drainage. This system is mostly used in embankments constructed with soil (earthfill), with mild slopes (e.g., 3H:1V), with a bedding layer for the geomembrane liner made of compacted cohesive material; however, being a very versatile technology it can be adapted to practically any type of base layer, even when construction of the dam has already started with another design. The civil and earth works do not require sophisticated skills or personnel, the anchorage system is simple and quickly installed, so overall a cost-effective way of constructing a watertight facing. Examples are Kohrang headpond in Iran (2004), Murdhari dam in Albania (2013), the bottom part of Runcu dam in Romania (a retrofit, 2016), Bulga dam in Australia (2016, Figure 1), and Pico da Urze pumped storage upper reservoir (2019).

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Figure 1. Bulga dam. From the foreground to the background, the geomembrane anchor bands protruding from the backfilled trenches excavated in the earthfill, the black synthetic drainage layer, and the light grey geocomposite waterproofing liner

In embankments constructed with large rockfill (stones, cobbles, boulders), characterised by steep slopes (typically 1.4H:1V), the bedding/drainage layer for the geomembrane liner is sometimes made with lean/porous concrete, and the trenches are pre-formed in such layer. A sophisticated application of this technology was adopted at the dams forming the upper and lower reservoirs of Pinnapuram pumped storage scheme in India (2024, Figure 2), where an Optical Fibre Cables system was installed to detect leakage flowing in the drainage layer under the waterproofing liner. The leak location system required a confinement layer of conventional concrete and a drainage layer of porous concrete. The trenches were formed in these layers, and the Optical Fibre Cables were placed in a slot created in the porous concrete.

Figure 2. Pinnapuram rockfill dam. From the foreground to the background, the trenches formed in the concrete layer, the geomembrane anchor bands protruding from the backfilled trenches, and the waterproofing geocomposite

In modern rockfill dams adopting the so-called ITÁ method, the embankment is raised placing the fill against an upstream layer of extruded porous concrete curbs. The concept of achieving anchorage by heat-seaming the waterproofing geocomposite to geomembrane anchor bands is maintained, but the bands are formed by short overlapping anchor strips nailed to each curb, and permanently anchored by the ballasting action of the rockfill against the curbs; each anchor strip is long enough to overlap on the anchor strip placed over the underlying curb, so that they can be heat-seamed at the overlapping to

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form continuous vertical anchor bands. The advantage of this method is that the curbs allow constructing in a brief time a regular slope not erodible by rainwater, which can be very steep and has constant characteristics. The curbs provide a stable and regular base layer for the waterproofing geocomposite, which is also a draining layer, and allow using a simple face anchorage method, constructed concurrent with the raising of the dam. The case histories that follow provide further details of the system, which has had several applications worldwide, such as Sar Cheshmeh tailings dam in Iran (2008), Nam Ou VI dam in Laos (2016), Rogun 3 cofferdam in Tajikistan (2020), Edevu dam in Papua New Guinea (2022), Santa Maria in Mexico and Pskem in Uzbekistan cofferdams (2022), and the dams forming the upper reservoir of Gandhi Sagar pumped storage scheme in India (ongoing).

3. CASE HISTORIES

3.1 Singrobo-Ahouaty, Côte d'Ivoire 2024

The Singrobo-Ahouaty Hydroelectric Power Project, completed in 2024 in Côte d'Ivoire, is the first, grid-ready hydroelectric power station developed and owned by Ivoire Hydro Energy (IHE), an independent power producer in West Africa. Singrobo-Ahouaty is a composite dam about 1,150 m long, that originally had a 226 m long 21 m high gravity section, a central concrete section extending over nearly 81 m and including a gated weir, a bottom outlet, and the intake, and a right section 995 m long consisting of a 30 m high rockfill dam. The rockfill dam section included several different zones: a central zone (3B) made of graded rockfill with a Dmax of 600 mm, a 3.0 m wide rockfill transition zone (3A) with a Dmax of 300 mm, a filtration layer (2B) in treated gravel with a particle size curve in the 5/31 mm range, 2.0 m wide, an upstream toe filter (2A) made of sand with a Dmax of 6 mm. At the upstream toe, a 2.0 m thick protection in treated fine material (clay/silt), covered with a layer of run-of-mine (R) 2.0 m thick. The upstream slope is 1.55H:1V. The original design for the rockfill section foresaw a facing of reinforced concrete slabs, with uniform thickness of 0.25 m and designed with several joints. The joints were to be waterproofed by a standard configuration: an embedded PVC waterstop and an external waterstop consisting of a Hypalon membrane placed on a sealing mastic and fixed by metal profiles. The slabs were to be supported by a reinforced concrete plinth, 1.0 m thick and 3.15 m wide, with an additional 0.31 m high bump for support, and anchored to the rock foundation by means of injected bars. The foundation treatment included a deep grout curtain and consolidation grouting.

Such design involved a complicated site organisation and complex and different procedures for constructing the CFRD and a concrete gravity dam. Complexity entails higher risks of inadequate execution that can jeopardise future performance, need for stringent Quality Control, longer construction times, possible late commissioning, overall higher costs. The EPC contractor who had been awarded the contract sought an alternative that would allow constructing a safer dam, would not imply such a complex organisation, and could be constructed in shorter time and at lower costs. The original design was modified, and both sections of the dam were designed as rockfill dams raised with extruded porous concrete curbs and with an exposed geomembrane facing.

To add engineering value to the project, and to provide further potential benefits in terms of site organisation, construction times, and overall economy of the project, Carpi proposed some additional modifications that were accepted and implemented. The 2 m wide filter layer + the 3.0 m wide transition layer were replaced by a single 3 m wide support layer of gravel and sand, the filter at the upstream toe was removed, as were the 2.0 m thick protection layer of fine materials + the 2.0 m thick run-of-mine cover at the upstream toe. The plinth was extended to the top of the dam on both banks, and the bump intended to support the reinforced concrete slabs could be removed. The maximum height of the rockfill dams is 30.74 m at the right section and 21.54 m at the left section.

As the embankment together with the support layer and the curbs were being raised, the SIBELON® anchor strips were placed, anchored, and heat-seamed one upon the underlying one, as already discussed, forming parallel vertical anchorage lines on the upstream face of the embankment. At Singrobo- Ahouaty the spacing between anchor lines is six meters. The waterproofing liner was then

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deployed from the top down the upstream slope and watertight heat-seamed to the anchor lines. Figure 3 clearly shows the fill material, the 3 m wide support layer, and the curbs embedding the anchor strips.

Figure 3. Singrobo-Ahouaty GFRD. 1) support layer, 2) extruded porous concrete curbs, 3) waterproofing geocomposite, 4) SIBELON® anchor strips, 5) heat-seaming of anchor strips

The support and porous concrete layers will convey infiltration water to discharge through the embankment.

The waterproofing liner is SIBELON® CNT 4400, a flexible SIBELON® geomembrane heat-bonded at manufacturing to a 500 g/m^2 nonwoven polypropylene geotextile. The peripheral seal at bottom was installed along the axis of the plinth, extending from the bottom of the upstream face to the dam crest along one side, so as to grant continuity between the waterproofing of the upstream face and the waterproofing of foundations, and connected to the concrete of the intake structure along the other side. The seal consists of 80 × 8 mm stainless-steel batten strips fastened to the concrete with chemical anchors at 150 mm spacing. At the intake structure, the peripheral seal is designed to be able to accept the differential settlement that may occur at the interface embankment/intake. At crest, the waterproofing liner is sealed in a trench filled with concrete.

Waterproofing works were executed in phases, to allow the changes of the water course through the concrete structure, from April 2023 to April 2024. A total of 37,237 square meters were installed.

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Figure 4. Singrobo-Ahouaty at completion of civil works

The project had some construction delays but overall was completed in a relatively short time. The lesson learnt is that with a geomembrane facing it is possible to recover delays caused by non-compliance with the planned schedule, which is a frequent issue in civil and earthworks. In early 2025, major civil engineering works, including the rock fill embankment, have been completed, and the two banks of the dam are connected. It is foreseen that waterproofing works will provide reliable performance, in line with the behaviour of previous projects adopting the same system.

3.2 Las Bambas, Peru, ongoing

The Las Bambas copper open-pit mine at high altitude in the Peruvian Andes, owned by MMG, has pioneered the use of the GFRD technology for tailings storage in Peru. The tailings storage facility has been designed to contain all tailings generated from processing operations, plus up to 3.5 GL of mine process water, and up to 6.5 GL of flood storage.

The tailings storage facility is approximately rectangular in shape and is located on sloping ground with hillsides rising on two sides; a L-shaped rockfill embankment forms the other two sides, as shown in Figure 5. The embankment has been formed with zoned quarry waste rock with no ore content, which integrates resource reuse. This rockfill is placed and compacted in layers, in accordance with specified requirements.

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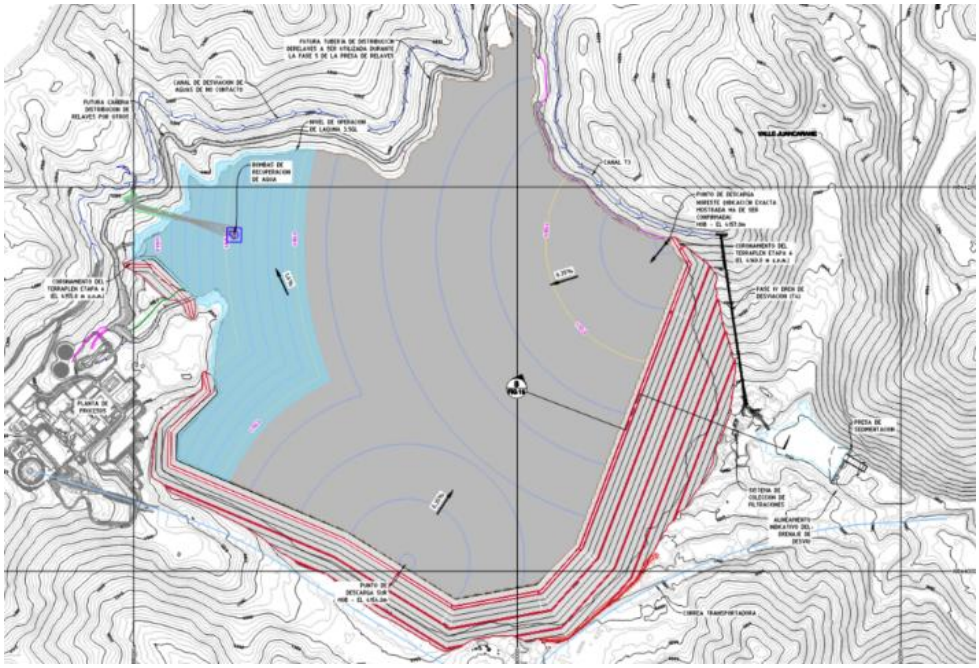

Figure 5. Las Bambas. Tailings storage facility and embankment general arrangement

The rockfill embankment, which is being raised in a number of stages using the downstream raising approach and will continue to be used for water storage throughout the operating life of the mine, has been designed as a zero-release facility, hence no provision has been made for an emergency spillway during the operational phase of the facility. A closure spillway shall be developed at completion of operation. Construction of the rockfill embankment started in 2014. Stage 1 between elevations 3932 and 4020 m a.s.l. was divided into sub-stages, Stage 2 between elevations 4020 and 4050 m a.s.l. was built continuously, as were Stage 3 between elevations 4050 and 4080 m a.s.l. and Stage 4 between elevations 4080 and 4105 m a.s.l. Stage 5 was divided into 2 sub-stages between elevations 4105 and 4120 m a.s.l. and elevations 4120 and 4130 m a.s.l. Stage 6 is currently being raised, to create a future dam height of approximately 260 m and a total crest length of approximately 4.4 km.

The dam adopts the GFRD system, with a very low permeability SIBELON® composite geomembrane as upstream waterproof barrier to ensure compliance with operational licensing conditions to retain tailings water and to control phreatic surface development within the dam embankment. The GFRD option was selected due to the lack of suitable low permeability materials to construct a clay core embankment, because of its better suitability to accommodate deformation and seismic effects (where concrete slabs are susceptible to cracking), and because of its cost effectiveness.

ATC Williams designed the geomembrane waterproofing system, incorporating details based on ICOLD experience to meet stringent performance and environmental standards. The SIBELON® composite geomembrane liner is placed on, and attached to, porous concrete curbs, which rest on a granular gravel drainage zone.

Figure 6 shows a typical cross-section of the embankment. Stage 6 incorporates geometric modifications, with the upstream slope changed from 1.7H:1V to 1.5H:1V, and the downstream slope changed from 1.75H:1V to a less conservative average slope angle of 1.65H:1V. Layers of high strength geogrids are being placed within the upstream transition to increase the stability of the slopes and to provide lateral restraint for the curbs to improve embankment response to static settlement.

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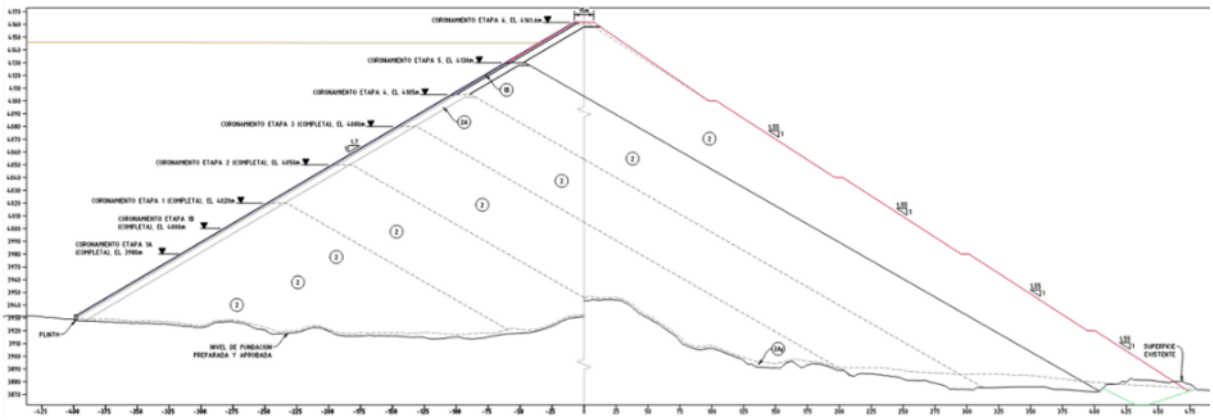

Figure 6. Las Bambas. Typical embankment cross-section

The face anchorage system for the geocomposite is the same described for the Singrobo-Ahouaty project. Figure 7 shows the substages of Stage 1: the waterproofing system has been completed in Stage 1A, and the dam is already providing the start-up water storage for the mine, while the construction of the embankment and of the curbs, and placement of the geomembrane anchor strips on the curbs, is ongoing for Stage 1B. The waterproofing liner of the upper stage will be anchored to the strips, overlapped on the already installed liner of the lower stage, and the two will be watertight heat-seamed together.

Figure 7. Las Bambas. From the foreground to the background, the extruded porous concrete curbs and PVC anchor strips forming parallel vertical anchorage lines for Stage 1B, and in the background Stage 1A completed and providing the watertight facing for impounded water, while construction of the plinth for the upper stage is ongoing

Embankment stages 1 to 5 (shown in Figure 8) used a SIBELON® CNT 3750 geocomposite (2.5 mm PVC geomembrane heat bonded during fabrication to a 500 g/m^2 nonwoven geotextile). For Stage 6 the specification was upgraded to a 3 mm PVC geomembrane heat bonded during fabrication to a 700 g/m^2 geotextile (SIBELON® CNT 4600) to enhance durability, because a longer period of exposure

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to solar radiation was anticipated, and to enhance performance in respect to long term settlements. The face anchorage system is the same for all stages. The bottom seal is of the insert type: the waterproofing liner is inserted in a slot created in the plinth and sealed with a special thixotropic mortar.

Figure 8. Las Bambas. Stage 5

Manufacturing and supply of the geocomposite liner system for Stage 6 has been completed. Construction of the Stage 6 embankment and curbs embedding the anchors strips has started, Installation of the waterproofing geocomposite is planned for two stages in 2026 and 2027.

Figure 9. Las Bambas. Commencement of Stage 6

So far, the height of the dam has reached 198 m. Its multiple stages have been carried out smoothly, according to planning, despite the delays caused by the working conditions at such altitudes, with frequent windstorms and lightning storms, harsh climate, and the altitude itself being a non-negligible factor. Once more, adopting a geomembrane facing proved essential for guaranteeing the timely completion of the works, in full compliance with the established planning.

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The dam, designed and constructed to meet the guidelines of the Australian National Committee on Large Dams (ANCOLD) related to the design of tailings dams and to seismic design criteria, is instrumented and monitored in a rigorous way, consistent with ANCOLD standards for large rockfill and tailings dams. Measurements include movement (deformation), foundation pore pressure, and seepage. Seepage is monitored through a downstream weir collecting flow from various sources. The monitoring results have been in alignment with the design predictions, which confirms that the dam and the geomembrane waterproofing system are performing well.

4. CONCLUSIONS

The GFRD technology can accommodate any schedule, constructing a flexible barrier that has low permeability, proven durability, and ease of installation, and providing a sustainable, cost-effective solution for water and tailings containment, in challenging contexts.

5. ACKNOWLEDGEMENTS

The authors want to thank IHE and MMG, owners of the projects herein presented, for their permission to publish this paper. The opinions and views presented in this paper are solely those of the authors and do not necessarily represent those of the owners.

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NUMERICAL MODELLING FOR DESIGN OF A SIDE SPILLWAY CONCRETE INFILL STRUCTURE AT YUSUFELI DAM

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ABSTRACT

Yusufeli Dam is a 275 m high double-curvature concrete arch dam on the Coruh River in the Artvin Province of the Black Sea region of Turkey. The dam has a developed crest length of 540 m and a section thickness of 8 m at the crest and 90 m at the base on the crown cantilever. The dam was constructed with conventionally vibrated mass concrete, and the structure contains 4 million cubic meters of concrete, impounding 2.1 billion m^3 of water at full supply level.

The dam has a controlled crest spillway, with three gates, which is supplemented by two controlled spillway tunnels on the right flank discharging into the river approximately 1 km downstream of the dam. The dam has a system of four mid-level outlets, which discharge into the spillway plunge pool immediately downstream. The spillway tunnels are controlled by radial gates operating on concrete peers supported by a mass concrete infill structure founded on the foundation in a gulley upstream of the right abutment.

This paper discusses the numerical modelling considerations, techniques and methodology adopted to design the control gates infill structure for sliding stability by implementing a shear key design approach. The complex loading and foundation-structural interaction of the stiff concrete infill structure placed in the gulley of the relatively flexible foundation rock mass, comprised challenging structural behaviour that was simulated using the finite element method.

1. BACKGROUND

Yusufeli Dam is a 275 m high double-curvature concrete arch dam on the Coruh River in the Artvin Province of the Black Sea region of Turkey. The dam has a developed crest length of 540 m and a section thickness of 8 m at the crest and 90 m at the base on the crown cantilever. The dam was constructed with conventionally vibrated mass concrete and the structure contains 4 million cubic meters of concrete. Yusufeli Dam is the highest dam in Turkey, the fourth highest double curvature arch dam in the world and ninth highest dam in the world.

Excavation for Yusufeli Dam commenced in 2014 and the final concrete lift in the dam body was placed in 2021. Impoundment was initiated at the beginning of 2023 and spilling of the dam is expected to occur in June 2024. With an underground power station located beneath the right abutment, the project has a hydropower generation capacity of 558 MW and annual energy production is anticipated at 1.8 billion kWh. A photograph of the dam during impoundment is shown in Figure 1.

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Figure 1. Yusufeli Dam During Impoundment

As part of the development of the spillway requirements for the dam, it was established that two gated spillway tunnels are to be constructed. The spillway tunnels are located upstream of the dam on the right abutment as shown in Figure 2. The control structure of the tunnel spillway was founded on a mass concrete infill structure placed in a local foundation gulley.

The foundation rock mass is made up of complex interwoven layers of fractured granitoid rocks and diabase dykes, with varying strength and deformation modulus properties. The fractured rock mass contains various low strength interfaces such as jointing, faults and other rock contact features arising from the fabric within it.

This paper prescribes how the large infill structure founded on the layered rock mass, was designed using the finite element (FE) method as a structural analysis tool.

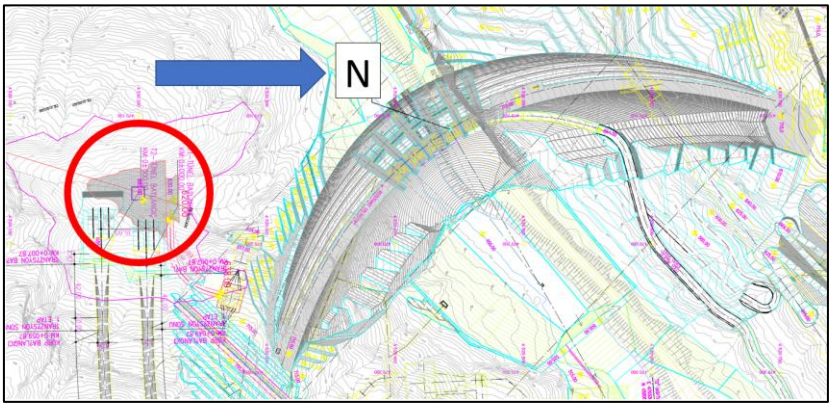

Figure 2. Plan Drawing of the Dam Right Bank Showing Spillway Infill Structure Location

2. INTRODUCTION

2.1 General

The evaluation of the infill structure required a complex three-dimensional structural analysis to be conducted to evaluate the force transfer and stress development expected to develop within the

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concrete infill structure, its underlying foundation and specifically at the potential sliding interface between the concrete and foundation rock mass. A photo of the structure during construction is shown in Figure 3.

The structural evaluation was undertaken using the finite element (FE) method. The outputs of the FEM structural analysis were used to derive the structural integrity, globally stability of the structure and compliance with serviceability operating criteria.

Figure 3. Spillway Support Infill Concrete Structure

2.2 Scope

When defining the scope of the study, consideration was given to the fact that the consequence of failure of the concrete infill has been acknowledged to be high due to the concrete infill supporting the gated side tunnel spillway concrete structure. Should the concrete infill fail, the direction of sliding failure would be toward the dam structure undermining the structural integrity of the main dam body.

This study comprised two phases, firstly the infill structure without any modifications was analysed for stability by reviewing analysis results against the applicable design criteria and where stability requirements were found to be deficient, a second phase of structural modification was implemented.

The first phase of structural stability of the infill considered the infill body mass concrete stresses, sliding and global displacement. It was found that sliding stability was inherently deficit and therefore required structural modification. The second phase of structural modification followed the establishment and design of a series of shear keys at the infill / foundation interface to increase the sliding stability to the required levels as per the design criteria.

The shear keys increased sliding capacity by allowing the infill structure to "lock" into the surrounding foundation. Although this study did not consider the integrity of the surrounding foundation, comment is provided on the expected bearing stress on the rock face at the locations of the shear keys.

2.3 Objective

Given the above scope, the overall objective of the study was to ensure the overall stability of the infill structure considering:

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- Infill mass concrete stresses,
- Sliding Stability,
- Global Displacements.

To achieve the above objective, the following steps were required to be completed form phase one of the stability review.

- Identify the possible failure mechanisms.
- Identify relevant loading conditions,
- Establish acceptance criteria,
- Develop an analysis model and technique to simulate applicable performance conditions,
- Develop a methodology to evaluate analysis results,
- Review against acceptance criteria.

To achieve sliding stability requirements, phase two was completed comprising the design of the shear keys. The goal of the shear key design was to confirm adequate structural performance of the shear keys under the equivalent global loading to that applied in phase one.

- Identify the local failure mechanisms of the shear keys,
- Modify the analysis model to include an adequate method of representing the proposed shear key configuration,
- Modify analysis model to ensure shear loading was represented suitably,
- Develop a design solution,
- Provide an interpretation of the design solution.

3. METHODOLOGY

Global failure mechanisms were identified to be a) concrete stress exceedance in terms of magnitude and extent, b) sliding failure and c) excessive deflections.

To evaluate the capacity of the infill mass concrete, principal stress results from the analyses were compared against the design criteria. The sliding resistance was assessed using the Mohr–Coulomb shear strength theory. Factors of Safety (FoS) were calculated at discrete points along the infill–foundation interface and subsequently plotted, enabling straightforward evaluation of sliding capacity in terms of both the magnitude of exceedance and the spatial extent of exceedance.

The infill structure functions in a manner similar to a gravity dam, relying on its mass for structural stability. Accordingly, EM 1110-2-2200 was used as a reference for the applicable design criteria.

It was found that inherent sliding capacity of the infill structure was not sufficient to carry the applied loading. It was therefore required that structural modification to increase sliding capacity be made. The structural modifications was in the form of a series of shear keys.

The shear keys therefore act as a structural component of the global infill structural system. The functional requirement of the shear keys is to increase sliding capacity of the infill structure.

Local failure mechanisms of the shear key configuration were identified as a) shear key concrete body stress exceedance and b) shearing along the theoretical infill / foundation interface. It is noted that the surrounding foundation is taken as competent and only applied bearing stresses are mentioned.

Since the distinct shear keys do not rely on self-weight for structural resistance, but instead on the full strength of concrete in both compression and tension, the use of EM 1110-2-2200 to establish stress

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limits was not applicable. Instead, the criteria specified in EM 1110-2-2201 were applied, allowing for a comprehensive evaluation of structural capacity based on the material strength properties of the concrete.

The resistance of the shear keys to the applied shear forces along the infill–foundation interface was designed in accordance with the requirements of ACI 318M-05 for shear reinforcement. Fixed rebar configurations were adopted to develop Capacity-to-Demand (C/D) ratios, which served as the basis for assessing adequacy. Global sliding Factors of Safety (FoS) were then determined by averaging the FoS values of the individual shear keys and applying the shear strength reduction factor specified in ACI 318M-05.

4. SHEAR KEY DESIGN APPROACH

The shear keys act as a mechanism for "locking" the concrete infill structure into the surrounding foundation (Figure 4). To achieve this, the destabilising driving forces of the infill structure had to be transferred into the shear keys and subsequently into the foundation. While not all of the destabilising force is expected to be transmitted through the shear keys, a significant proportion is, and this was explicitly accounted for in the design.

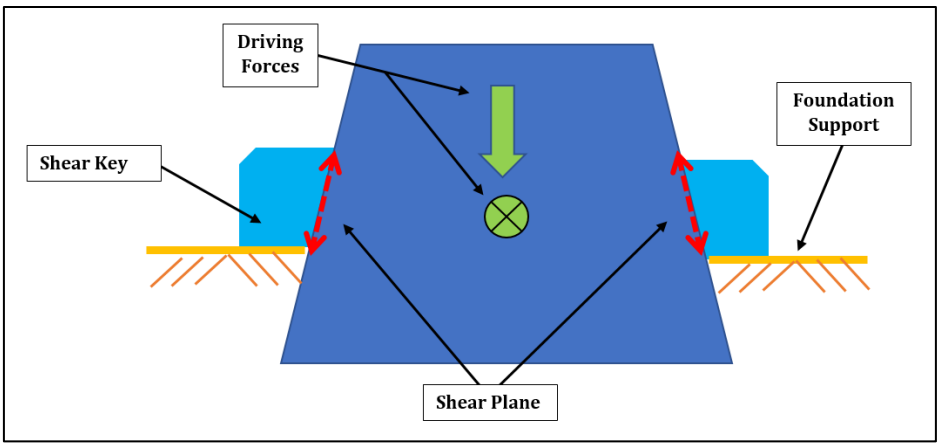

Figure 4. Mechanism of shear key support

It was therefore necessary to perform analyses to ensure that a conservative proportion of the destabilising driving force was transmitted into the shear keys. This was achieved by modelling the infill–foundation contact using interface elements with appropriate material properties, thereby controlling the transfer of the destabilising forces.

Reducing the tangential stiffness of the interfaces allows for a larger proportion of the destabilising driving force to be distributed into the shear keys. This assumption is valid as it has previously been shown that the sliding stability without shear keys is inadequate which may lead to slippage.

5. LOADS AND LOAD CASES

The loads considered for the assessment of both phases are given below:

- Gravity: taken as 9.81 m/s2.
- Hydrostatic: the hydrostatic load is taken using the reservoir full supply level of 710 masl and water density of 1000 kg/m3.

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- Uplift: the uplift loading when combined with hydrostatic was applied assuming no drainage.
- Rapid-drawdown uplift: the rapid drawdown loading followed the same mechanism as the uplift loading excepting the load assumes a drain efficiency of 50 %.
- OBE Seismic: The horizontal OBE pseudo static acceleration was taken as 0.11 g. The vertical component was then taken to be 2/3's of the horizontal.

Considering the above loads, five load cases were identified and are given in Table 1.

Table 1. Load Cases

Load Case	Load Condition	Gravity	Hydro	Uplift	Rapid Drawdown	Seismic OBE
LC1	Usual	X				
LC2	Usual	X	X	X		
LC3	Unusual	X			X	
LC4	Unusual	X				X

6. DESIGN CRITERIA

The tensile strength of concrete was calculated using ACI 207.1R-96 Clause 3.2.6 and shown in Equation 1 with f'c taken as 25 MPa.

$$f_t' = 0.32 {f_c'}^{2/3} \quad (1)$$

Where

f'c = compressive strength
f't = tensile strength

ACI 207.1R-05 Clause 3.2 states that under seismic loading the dynamic strength may be increased by 1.3 times the static for compressive and tensile strengths respectively. The design criteria in terms of sliding stability and allowable stress criteria of the concrete material for the infill concrete and structural shear key concrete is shown in Table 2 through Table 5. For sliding stability, the Mohr-Coulomb shear strength criteria for the contact interface between infill and foundation was taken as cohesion (c) = 100 kPa and internal friction angle (ϕ) = 45 ˚.

Table 2. Infill Mass Concrete Stability and Stress Criteria EM 1110-2-2200

Load Case	Load Condition	Sliding FoS	f'c (fac) (MPa)	f'cdyn (fac) (MPa)	f't (fac) (MPa)	f'tdyn (fac) (MPa)
LC1	Usual	2.0	7.50	-	*0.98	-
LC2	Usual	2.0	7.50	-	*0.98	-
LC3	Unusual	1.7	12.50	-	0.98	-
LC4	Extreme	1.3	-	29.25	-	3.17

*Due to excess restraint and the structural nature of the infill structure, zero tensile condition is not possible

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Table 3. EM 1110-2-2200 Criteria Factors

Load Case	Load Condition	$f'_{c\,(fac)}$ (MPa)	$f'_{cd\,(fac)}$ (MPa)	$f'_{t\,(fac)}$ (MPa)	$f'_{td\,(fac)}$ (MPa)
LC1	Usual	$0.3xf'_c$	-	$0.11xf'^{(2/3)}_c$	-
LC2	Usual	$0.3xf'_c$	-	$0.11xf'^{(2/3)}_c$	-
LC3	Unusual	$0.5xf'_c$	-	$0.11xf'^{(2/3)}_c$	-
LC4	Extreme	-	$0.9xf'_{cd}$	-	$0.29xf'^{(2/3)}_{td}$

Table 4. Shear Key Stress Criteria EM 1110-2-2201

Load Case	Load Condition	Load Type	$f'_{c\,(fac)}$ (MPa)	$f'_{cd\,(fac)}$ (MPa)	$f'_{t\,(fac)}$ (MPa)	$f'_{td\,(fac)}$ (MPa)
LC1	Usual	Static	6.25	-	2.74	-
LC2	Usual	Static	6.25	-	2.74	-
LC3	Unusual	Static	10.00	-	2.74	-
LC4	Unusual	Dynamic	-	13.00	-	3.56

Table 5. EM 1110-2-2201 Criteria Factors

Load Case	Load Condition	Load Type	$f'_{c\,(fac)}$ (MPa)	$f'_{cd\,(fac)}$ (MPa)	$f'_{t\,(fac)}$ (MPa)	$f'_{td\,(fac)}$ (MPa)
LC1	Usual	Static	$f'_c/4$	-	$f'_t/1$	-
LC2	Usual	Static	$f'_c/4$	-	$f'_t/1$	-
LC3	Unusual	Static	$f'_c/2.5$	-	$f'_t/1$	-
LC4	Unusual	Dynamic	-	$f'_{cd}/2.5$	-	$f'_{td}/1$

7. FE MODELLING

7.1 Model mesh

The infill structure model was discretised using a combination of tetrahedral, prismatic and pyramidal linear continuum elements. The finite element mesh is shown in Figure 5. The concrete infill and tunnel concrete structure were finely meshed to ensure satisfactory capturing of displacement and stress distribution.

Interface elements were used to represent the behaviour of concrete works and rock foundation interaction. The failure shear plane was assumed to be along the intersection surface of the original infill structure and shear keys. To capture the shear along this plane, interface elements were placed and finely discretised to enable the depiction of change in stress magnitudes over the face of the shearing plane. The shear key configuration is shown in Figure 6.

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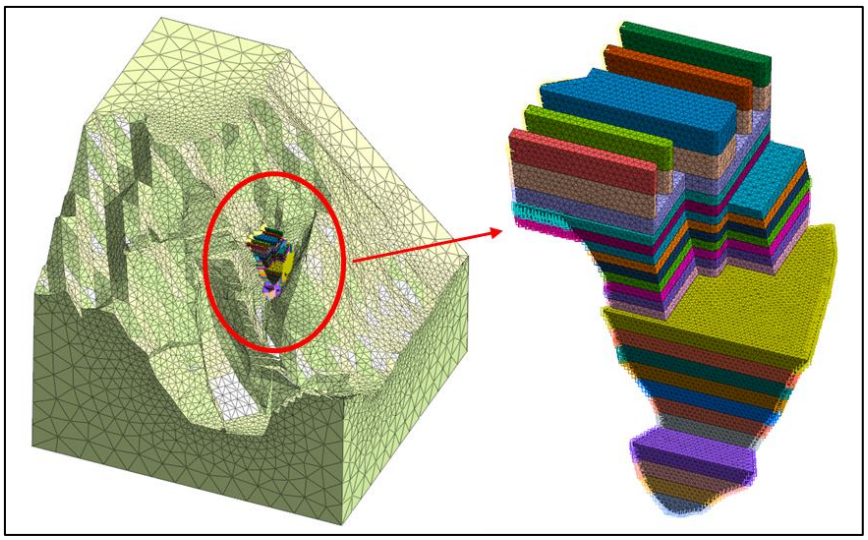

Figure 5. Finite element mesh of infill structure

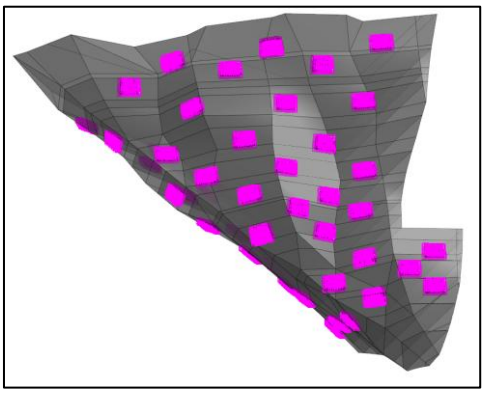

Figure 6. Infill structure CAD model with shear key configuration

7.2 Material properties

Due to the structural linear elastic nature of the analysis, only the Elastic Modulus, Density and Poisson's Ratio were required to be defined for the analysis of the structure as shown in Table 6.

Table 6. Material properties

Material	E-Modulus (GPa)	Density (kg/m^3)	Poisson Ratio
Foundation	8	0	0.3
All concrete elements	23	2 500	0.2

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8. SHEAR KEY DESIGN

8.1 Approach

Evaluation of the infill structure indicated insufficient inherent sliding capacity, and it was therefore required that stabilising methods be designed. The adequate sliding stability of the infill structure was achieved by installing a series of reinforced concrete shear keys.

The shear design of the shear keys was undertaken by firstly reviewing the performance of the concrete material on the shear plane in isolation to the shear reinforcement and following this the completion of a shear reinforcement design.

Firstly, to evaluate the performance of the concrete material on the shear plane, a Mohr Coulomb constitutive model was established. This approach allowed for the examination of the concrete material's capacity to withstand the shear stresses and tendency to exceed elastic limits in isolation of the steel reinforcement. The shear strength Mohr-Coulomb properties of the concrete, cohesion (c) and internal friction angle (Φ), were derived using equation 2 and 3 (Chen, 2014).

When reviewing these results, it was considered that the results represent only the strength of the concrete and not the addition resistance provided by the reinforcement. Following this assessment, it was possible to determine the significance the shear reinforcement contributes the overall shear resistance of each shear key.

The second potion of the design was to calculate the quantity of reinforcement required to resist the design stresses in accordance with ACI 318M-05. The results of design requirements were provided in tabular format.

$$\frac{f_c'}{f_t'} = \frac{1+sin\phi}{1-sin\phi} \tag{2}$$

$$c = \frac{f'_{c(fac)} \cdot (1-sin\phi)}{2cos\phi} \tag{3}$$

8.2 Shear Reinforcement Configuration

The shear reinforcement design was carried out in accordance with ACI 318M-05. To carry out the design, the following aspects were considered.

- A fixed configuration of reinforcement, (Figure 7). This allows for ease and speed of construction,
- All vertical and horizontal tie bars are taken as 10 mm diameter,
- Any one particular shear key would have one class of shear bar diameter,
- The diameter of the reinforcement would change dependant on applied shear stress.

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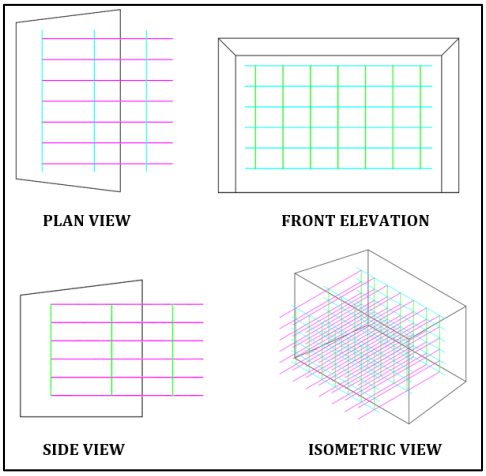

Figure 7. Configuration of shear reinforcement (*Configuration A*)

The design considered 3 different configurations of reinforcement. The only variance in the three configurations is the horizontal spacing and bar diameter. The values of horizontal spacing and bar diameter for each configuration are given in Table 7.

Table 7. Reinforcement Configurations

Configuration	Space (mm)	Bar Diameter (mm)
A	400	26
B	400	28
C	400	30
D	400	32
E	300	32

9. ANALYSIS RESULTS

9.1 FoS Results

The global FoSs computed for the infill structure, after incorporation of the shear keys, for the various load cases is shown in Table 8. It is noted that sliding stability criteria was not achieved for all load cases without the inclusion of shear keys.

Table 8. Sliding FoS for Each Load Case

	LC1	LC2	LC3	LC4
Required FoS	2.00	2.00	1.70	1.30
Calculated FoS	2.30	2.52	2.58	1.98

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9.2 Displacement and Stress Results

For purposes of this paper, comprehensive results of load case 2 (normal operating conditions) are presented and the vertical displacements and major principal stresses are shown in Figure 8. As can be seen from the results no stress limit exceedance is evident with significant capacity present for both compression and tension. Local stress concentrations were computed due to interface restraint conditions between the infill and foundation and discrete changes in geometry configuration.

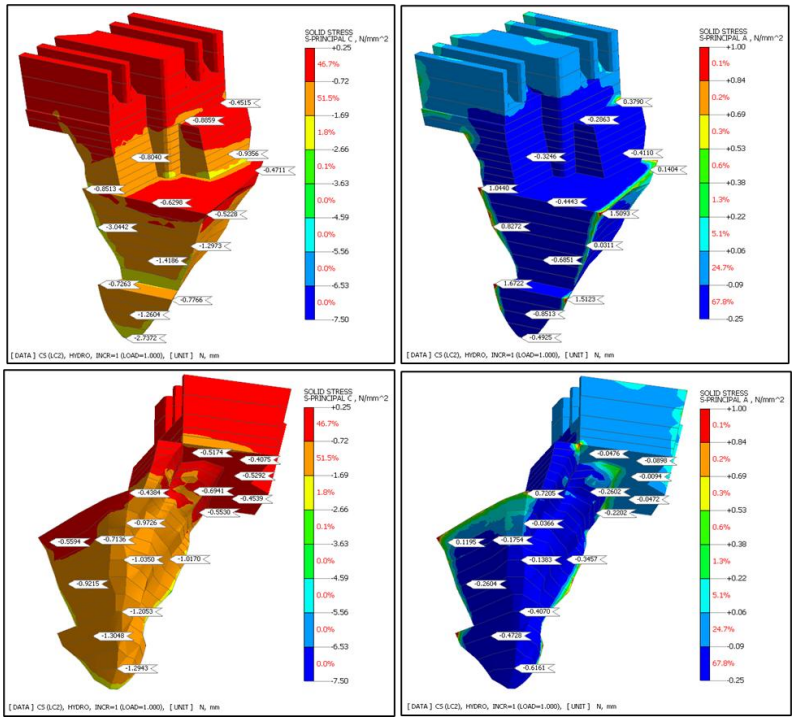

Figure 8. Minor (left) and major principal stresses (right) for load case 2

10. CONCLUSION

As part of the spillway works on the right bank of Yusufeli Dam, a concrete infill structure was required to be constructed to support the spillway intake concrete works.

Initial analyses indicated insufficient inherent sliding stability of the infill structure. It was therefore required that stabilising methods be designed. The adequate sliding stability of the infill structure was achieved by installing a series of shear keys.

The load cases considered in the design comprised the following:

- LC1: Self-weight
- LC2: Self-weight + Hydrostatic + Uplift
- LC3: Self-Weight + Rapid Drawdown
- LC4: Self-weight + OBE Seismic

All stresses within the shear key mass remained within acceptable limits. Using a Mohr Coulomb material criterion, an assessment of the concrete behaviour along the assumed shear planes was carried out in isolation of the shear reinforcement. It was shown that a steady behaviour of elastic and

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stable behaviour of the concrete along the shear plane can be expected under LC 1 & 2 and LC 3 & 4 respectively. It was also evident that shear reinforcement was required for both strength and ductility.

A fixed configuration of reinforcement was assumed for ease of construction and uniform bar diameter for each shear key.

An aspect which was considered for construction is the addition of construction and contraction joints. A Concrete infill / foundation interface regrouting system was installed. This ensured that an intact interface peak bond strength is re-established following thermal cooling and final construction.

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UNRAVELLING THE MYSTERIES OF THE KATSE DAM SEISMIC BELT AND PREFORMED JOINT

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ABSTRACT

Considering that significant developments have been made in analysis systems for arch dams since the design of Katse Dam in the 1990s, in their 2018 review, the Panel of Experts for Katse Dam recommended an updated Finite Element structural analysis to provide a clearer picture of the respective structural function and the associated level of dam safety. As a result, the dam owner (LHDA) initiated a comprehensive study to investigate the status of the dam, including a calibrated finite element analysis.

With a substantially improved ability to model non-linear features of the dam, such as the "pre-formed heel joint", the crest tension belt and the foundation shear keys, together with the use of ambient vibration monitoring calibration, the updated FE analysis provided an extremely useful tool to gain a more detailed insight into the actual structural function of the dam/foundation system. On the basis of this improved understanding, it was possible to develop a more quantitative evaluation of the safety of the dam.

The analysis comprised a suite of seismic loadings, established on the basis of ICOLD best practices, using a calibrated finite element model. A site-specific seismic hazard assessment was undertaken to accurately establish the local seismic conditions in respect of applicable earthquake loading.

In this paper, the authors discuss the methodology and outcomes of the dynamic analysis of Katse Dam, using the finite element method. The simulated behaviour of the seismic belt and preformed joint are presented, and the structural function is critically evaluated.

1. BACKGROUND

Katse Dam is a 185 m high, double-curvature arch dam on the Malibamatso River in Lesotho and forms part of Phase 1a of the Lesotho Highlands Water Project. With a developed crest length of 710 m at an elevation of 2060 mASL, the structure is located in a U-shaped valley. While neither the height, nor the crest length/height ratio of 3.8 are exceptional, the structure exhibits a relatively large upstream face surface area-to-height ratio (canyon shape factor). Construction was completed in 1997 and first impoundment was achieved in 1998.

The foundation for the dam is formed in Basalt rock, comprising a series of sub-horizontal flows of varying thickness. Shear zones are present on the abutments, while flows with weathered contacts and problematic auto-brecciated layers are located at, and beneath the base of the structure, as illustrated

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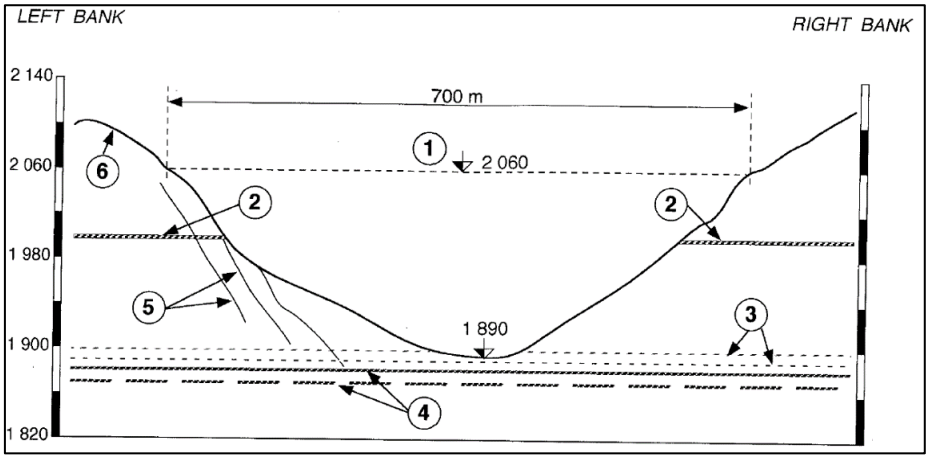
in Figure 1. A detailed account of the design considerations and structural characteristics of the dam is provided in the ICOLD Congress proceedings held in Durban, South Africa, in 1994 (Tardieu, 1994).



1 - Crest height, 2 – Sub-horizontal shear zone, 3 – Weathered flow contacts, 4 – Auto-brecciated layers, 5 – Stress relief joints, 6 – Natural ground level

Figure 1. Geological section of Katse Dam foundation valley (Tardieu, 1994)

A Panel of Experts (POE) was appointed in 2018 to perform a 10-year safety review of Katse Dam. The review indicated that the dam was well maintained and had performed adequately under the original design conditions for the duration of its operational life to that point.

The POE review concluded that, while the original design calculations for Katse Dam were appropriate for the standards and methods available at the time, subsequent advances in analysis technology now allow for the development of a significantly improved understanding of the dam's structural behaviour. The application of modern analysis systems would provide particular value in supporting and enhancing the ongoing monitoring and safety evaluation of the dam.

The analyses comprised a comprehensive structural review of the dam response under various loading conditions including dynamic earthquake loading. In this paper, the authors discuss the analysis of Katse Dam under dynamic loading, with a particular focus on the structural role of the seismic belt and pre-formed joint.

2. INTRODUCTION

2.1 Seismic evaluation of concrete arch dams

Dams are not inherently safe against earthquakes (Wieland, 2014). This has been shown repeatedly during strong earthquakes. For example, during the 2008 Wenchuan earthquake in China, more than 1 500 dams, reservoirs and power plants were damaged. After the 2001 Bhuj earthquake in India, approximately 240 dams had to be rehabilitated and strengthened. Also, during the two powerful earthquakes in the southeast of Turkey on 6 February 2023, several large embankment dams were damaged (Kijko et al, 2023).

Large concrete dams were among the first structures for which seismic analysis and design was performed (Wieland, 2008). The seismic analysis approach for dams has progressed since the 1930s when the pseudo-static analysis approach developed for the design of Hoover Dam in the USA was accepted worldwide as the standard. Since then, modelling techniques for dynamic analysis of dams

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have developed significantly and now commonly involve complex numerical modelling methods, supported by powerful personal computers and efficient solver techniques.

The pseudo-static method has various shortcomings in that it does not consider the dynamic characteristic of the dam-foundation-reservoir structural system, nor does it consider the frequency content and duration of the earthquake acceleration loading. The dynamic characteristics of the dam-foundation-reservoir system are defined by the mass, stiffness, geometrical configuration and damping of this system.

Currently the accepted seismic analysis method involves the application of an appropriate acceleration time-history signal to the structure model (ICOLD, 2010). The time-history approach utilises structural dynamic analysis principles to evaluate the dam structural response against a time-based acceleration loading input, also known as an accelerogram or time-history signal. Time-history analysis methods are best executed using numerical modelling techniques such as the Finite Element (FE) method.

2.2 Reservoir triggered seismicity

Reservoir-triggered seismicity (RTS) is a phenomenon in which the impoundment of water behind a large dam induces seismic events, including small to moderate earthquakes. This unique geophysical phenomenon has gained considerable attention in recent decades due to its potential implications for dam safety and the surrounding communities.

From the physical point of view, the generation mechanism of RTS is understood. However, it is almost impossible to predict which reservoirs will trigger earthquakes or how strong the largest RTS events will be, although by definition, they can never exceed the natural earthquakes. Most of the significant cases of RTS are associated with dams over 100 metres high – but dams just half this height are also causing RTS (McCully, 1996). According to Wieland (2017), a large dam that has been adequately designed against the Safety Evaluation Earthquake in terms of current ICOLD practices should generally be safe against RTS.

2.3 Structural enhancement components of Katse Dam

Katse Dam was constructed with three particular structural features, included to accommodate the consequences of its form, as a wide-valley arch dam, and the particular geotechnical conditions present within the foundation. The first is the "preformed-joint", which is located at the heel zone of the crown cantilever, as illustrated in Figures 2 and 3.

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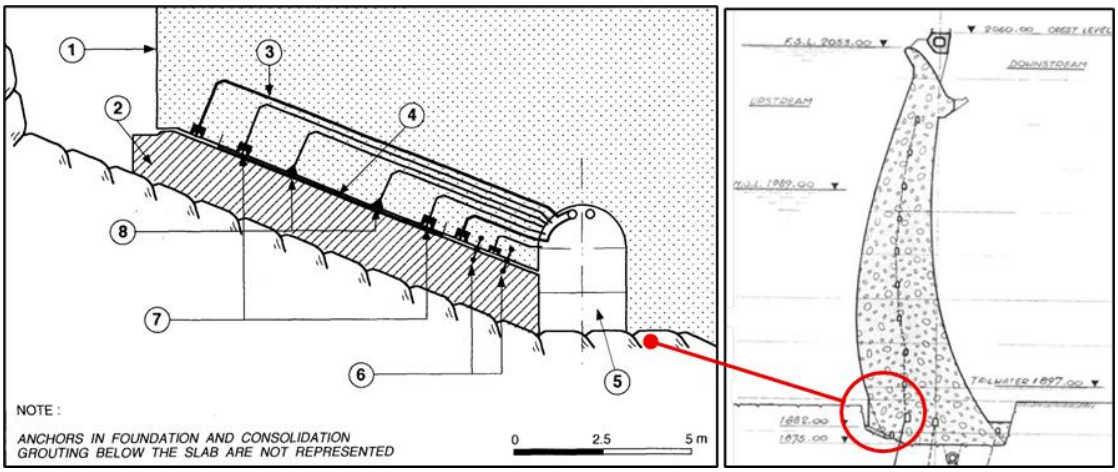
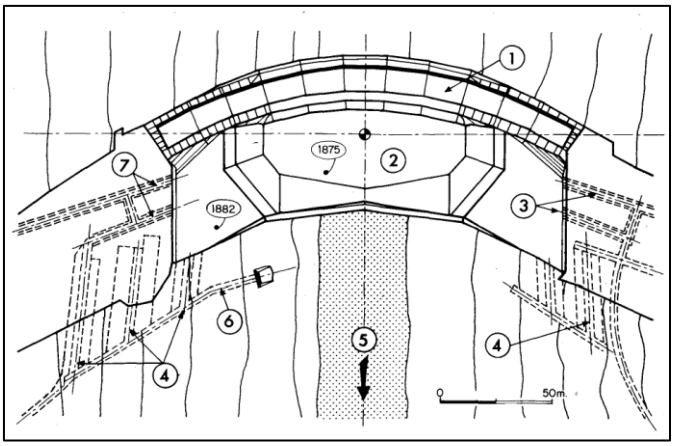


1 – Arch dam US face, 2 – Reinforced concrete slab anchored to foundation, 3 – Water feeding pipes, 4 – Membrane for grouting compartment and debonding, 5 – Control gallery of preformed joint, 6 – 500 mm waterstops, 7 – Grouting pipes, 8 – Reinjectable grouting valves

Figure 2. Pre-formed joint at heel of dam

The second feature relates to concrete shear keys that were constructed to increase shear capacity in the shear zones within each abutment, as illustrated in Figures 3 and 4.



1 – Pre-formed joint, 2 – Bottom excavation, 3 – Grouting and drainage galleries, 4 – Shear keys, 5 – Malibamatso River, 6 - Access gallery, 7 – Grouting and drainage galleries

Figure 3. Plan view of central excavations, illustrating pre-formed joint and shear key layout

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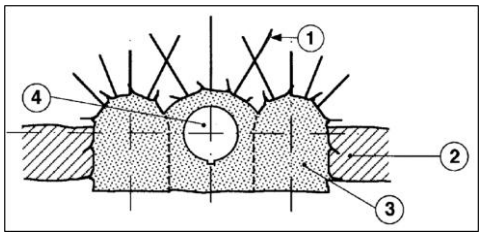

1 – Rock bolting, 2 – Auto breccia foundation layer, 3 – Concrete filled horsehoe galley, 4 – Grouting and drainage gallery

Figure 4. Section view of shear keys in Katse Dam foundation

The third feature relates to a "seismic belt" installed in the central crest of the structure, as illustrated in Figure 5.

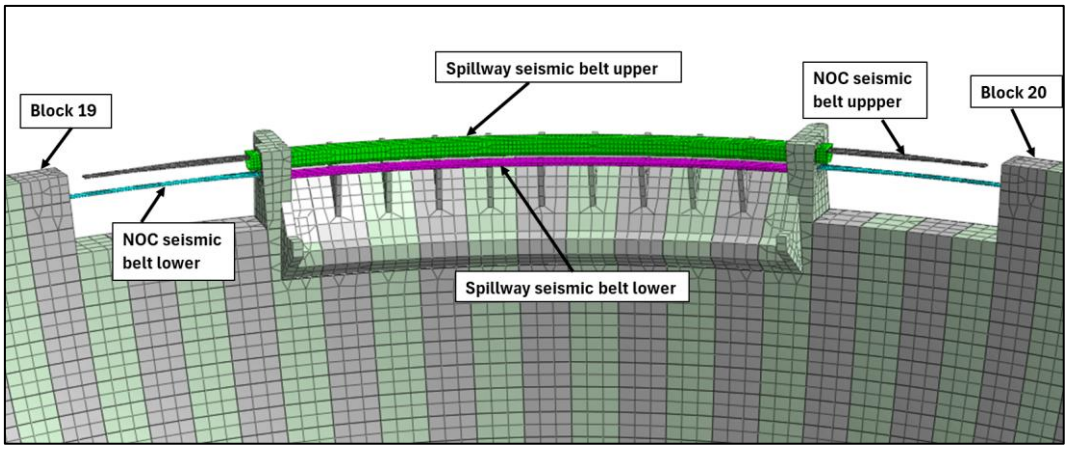

Figure 5. Seismic belt in upper dam crest & spillway bridge (concrete blocks removed for illustration)

The purpose of the pre-formed joint is to release cantilever heel tensions, which develop as the central cantilevers of the wide-valley arch structure rotate under load, and prevent the progression of tensions into the layered rock mass beneath the dam. By introducing hydrostatic pressure into this joint, a downward force is maintained onto the underlying rock mass. The shear keys obviously serve to increase shear resistance within the sub-horizontal rock mass shear zones under the thrust imposed on the abutments by the loaded dam. The seismic/tension belt resists tensions that develop along the alignment of the crest under seismic excitation and can otherwise open contraction joints and impose high vertical tension stresses on the central cantilevers of the arch structure.

2.4 Objective and scope

In this paper, the authors describe the approach and methodology adopted to conduct a dynamic structural analysis of Katse Dam under seismic loading. Within the limited scope of the paper, a specific focus on the modelling and performance of the pre-formed joint and seismic belt is included.

While the dynamic evaluation of Katse Dam included a comprehensive range of static and seismic loadings, the study addressed in this paper reports on only two load cases; namely the SEE (safety evaluation earthquake) (10 000 year event) load case and the OBE (operating basis earthquake) (1 475 year event) load case with lowered reservoir level, as the response of the dam under these loadings is significantly influenced by the seismic belt and preformed joint.

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3. METHODOLOGY

The methodology of the analysis comprised the following steps:

- Derivation of seismic performance acceptance criteria and load cases for evaluation.
- Development of suitable earthquake loading inputs, based on the findings of the site-specific seismic hazard assessment.
- Establishment of a calibrated structural FE model of the dam and foundation stiffness parameters adopted, as per the outcome of the calibration study (Cassells et al, 2024; Cassells et al, 2025).
- Performance of an eigenvalue analysis to define Rayleigh damping.
- Seismic analyses using the acceleration time-history approach.
- Post-processing of the analysis results (stresses and displacements) and an evaluation against the applicable criteria in respect of possible failure mechanisms.

4. DESIGN CRITERIA

4.1 Stress Criteria

For the purposes of the analyses addressed herein, the criteria for the arch structure were based on the recommendations of the USACE's Engineering Manual EM 1110-2-2201 titled "*Arch Dam Design",* USBR*s: Arch Dam Design Manual:* and ICOLD Bulletin 206 titled "Arch Dam Design: Structural Analysis Methodologies and Criteria (2024)". For limiting stress values, the respective recommendations are presented in Table 1.

Table 1. Allowable concrete stresses (USACE, 1994; USBR, 1977; ICOLD, 2024)

Concrete Stress	Loading Combination		
	Usual/Normal	Unusual/Accidental	Extreme
Compression	f'c/4	f'c/2.5	f'c/1.5
Tension	f't	f't	f't

The structural performance of the dam was reviewed assuming linear elastic material properties, considering the maximum allowable stresses listed in Table 2. For purposes of modelling the interfaces between the dam and the foundation and the contraction joints between the monoliths in the dam, a nominal tensile strength was adopted. On the pre-formed joint, a zero tensile strength was considered.

Table 2. Allowable concrete stress criteria

Static loading condition	Compressive stress (cube)	Tension Stress (split cylinder)	Tension Stress (modulus of rupture)	Shear Stress	Shear FoS
Usual/Normal	8.0 MPa	2.7 MPa	3.6 MPa	3.6 MPa	2.0
Unusual	9.6 MPa	2.7 MPa	3.6 MPa	3.6 MPa	1.3
Extreme (Static)	16.0 MPa	2.7 MPa	3.6 MPa	3.6 MPa	1.1
Extreme (Dynamic)	16.0 MPa	4.0 MPa	5.4 MPa	3.6 MPa	1.1

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4.2 Seismic performance evaluation

The linear seismic performance of the dam was evaluated in terms of a seismic performance curve, which compares the cumulative duration of tensile stresses in excess of the linear capacity against acceptable limits, using a cumulative duration demand-capacity curve, as prescribed in USACE's EM 1110-2-6051 "*Time-History Dynamic Analysis of Concrete Hydraulic Structures*" (2003). The single cycle demand/capacity ratio was limted to a value of 2.0 times the direct tensile strength of the concrete, as illustrated in Figure 5. At each selected location, the cumulative duration beyond basic elastic capacity is established by multiplying the value of each stress exceedence by the associated time of exceedence, as derived through the time-history analysis.

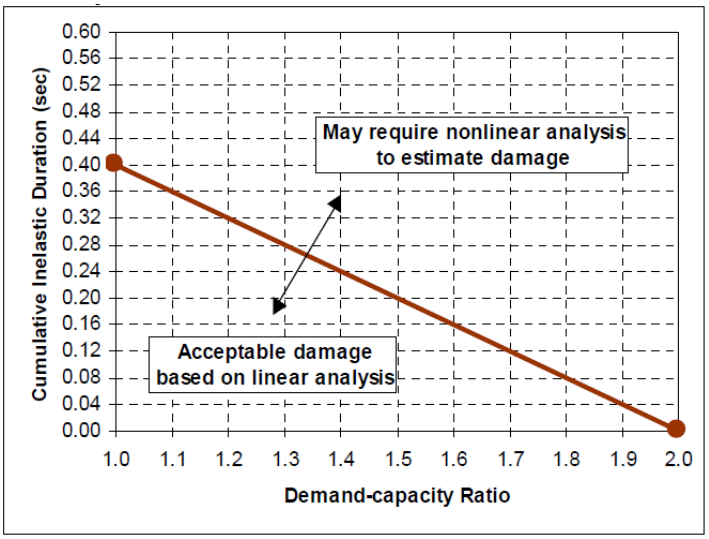

Figure 5. Performance curve for linear elastic analysis of arch dams (USACE, 2003)

5. SEISMIC LOADING CONDITIONS

5.1 Seismic hazard

A site-specific Probabilistic Seismic Hazard Assessment (PSHA) for Katse Dam was completed by Professor Andrzej Kijko, based on historical seismic activity observed within a radius of 450 km of the Katse Dam site. The PSHA study was undertaken using the conventional Cornell-McGuire procedure.

The horizontal elastic response spectra curves derived through the PSHA are shown in Figure 6 and are based on the Eurocode 8 curve formation, with 5% damping, assuming the dam is located on rock with a vs_{30} equal to 800 m/s. The curves indicate the dynamic acceleration response of a single degree of freedom system for the Safety Evaluation Earthquake (SEE) (1 in 10 000-year return period) and the Operating Basis Earthquake (OBE) (1 in 475-year return period).

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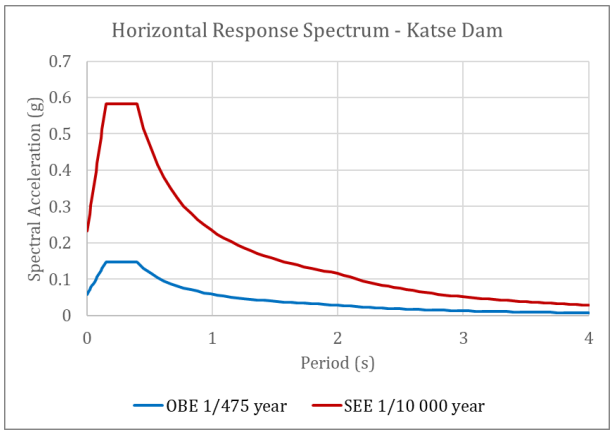

Figure 6. Horizontal Elastic Response Spectra Curves – Katse Dam

5.2 Seismic load cases and selected earthquake signals

Two critical seismic load cases adopted for analysis and discussed in this paper comprise the SEE event and the OBE event with low reservoir level as shown in Table 3. For SEE loading conditions (load case 1), three analyses (a, b and c) were performed, considering three different earthquake signals as per ICOLD recommendations (Wieland, 2017; USACE 2003). The three earthquake signals adopted for a, b and c comprised Sparks Oklahoma, Little Skull Mountain and Umbria March. The Sparks Oklahoma event was found to be the most critical. For load case 2, the Sparks Oklahoma OBE scaled event was considered. Static loadings included gravity (self-weight), hydrostatic and uplift, as per USACEs guidelines.

Table 3. Seismic load cases

Load Case	Loading Condition	Upstream Water	Tailwater Level	Drainage/ Uplift	Silt	Self-weight	Earthquake Loading
1 a,b,c	Extreme	FSL (2053 m)	1897 m	Drained	No	Yes	SEE (1/10 000)
2	Unusual	Min (1989 m)	1897 m	Drained	No	Yes	OBE (1/475)

Load case 2 is important for an arch dam, as it considers OBE loading with low reservoir conditions, under which an upstream acceleration response of the dam will instantaneously open contraction joints, which can in turn induce significant tensions and potential instability in the upper cantilevers (Figure 7).

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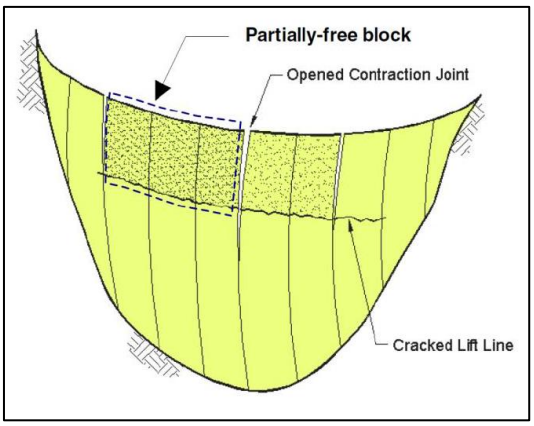

Figure 7. Mobilisation of partially free concrete block during dam response (Ghanaat, 2004)

Frequency and amplitude-based spectral matching was applied to the selected earthquake time-histories in the time-domain, using wavelet adjustment to match the response specta to the site-specific response spectra. The time-history acceleration signal for Sparks Oklahoma, as applied for instream loading, is shown in Figure 8. Cross-stream and vertical signals were also incorporated in the 3-dimensional acceleration loading.

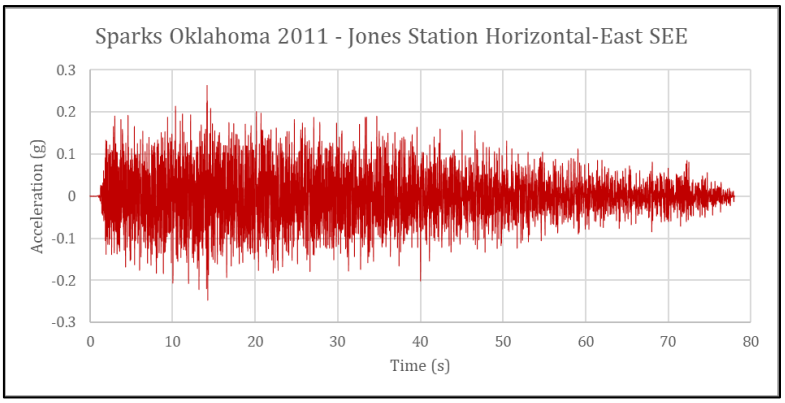

Figure 8. Sparks Oklahoma Earthquake acceleration time-history – East Horizontal SEE

6. FINITE ELEMENT MODEL

6.1 Material stiffness parameters

The dam concrete and foundation rock mass materials were modelled assuming linear elastic constitutive material parameters as shown in Table 4, with the exception of discrete material interfaces at the dam-foundation contact and on the contraction joints between dam blocks. The respective dam and foundation material properties were obtained through a comprehensive model calibration process (Cassells et al, 2025). As a result of the high strain rate associated with seismic loading, the dynamic elastic modulus was estimated by increasing the static value by 15 % (FERC, 2002; ICOLD, 2009; ACI207.1R-17, 1996). The static Poisson's ratio for concrete was reduced by 20% for the dynamic analysis (Bruhwiler, 1990).

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Table 4. Dynamic Material properties of Katse Dam concrete

Dam Material	Elastic Modulus (GPa)	Poisson Ratio	Unit weight (kN/m3)	Compressive Strength (MPa)
Mass Concrete	39.1	0.16	24.5	24
Seismic Belt Concrete	57.5	0.16	25	36

The calibrated deformation parameters for the layers of rock mass material making up the foundation are shown in Table 5 and illustrated in Figure 9.

Table 5. Katse Dam foundation dynamic material parameters

Foundation layer description	Location	Density (kN/m^3)	Elasticity Modulus (GPa)	Poisson ratio
Lower Basalt layer below auto brecciated layer (A)	Below 1 866 mASL	0	40	0.21
Lower auto-brecciated layer (B)	Between 1 866 & 1 869 mASL	0	20	0.30
Basalt layer between auto-brecciated layers (C)	Between 1 869 & 1882 mASL	0	25	0.27
Upper auto-brecciated layer (D)	Between 1 882 & 1 885 mASL	0	10	0.35
Basaltic layer (D)	Above 1 885 and below flow contact at 1 892.5 mASL	0	30	0.25
Upper basaltic layer (E)	Above flow contact at 1 892.5 mASL	0	30	0.25
Upper right bank basaltic layer (F)	Above 1 975 mASL	0	35	0.23

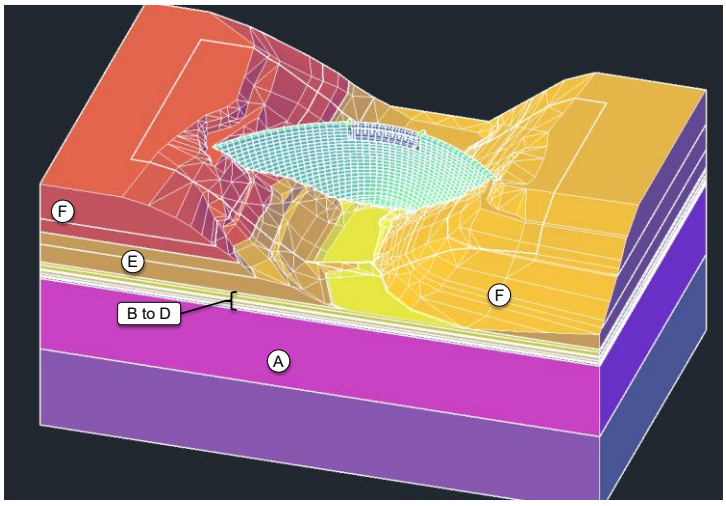

Figure 9. Section of Katse Dam foundation model showing foundation layers

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For purposes of the non-linear material modelling, a tensile strength capacity of 0.2 MPa was assumed at the contraction joints between dam monoliths, while a zero tensile strength was assumed at the pre-formed joint at the heel of the arch.

6.2 Finite Element Structural Mesh

The FE model of the dam and foundation was created using the Midas FEA NX software. The dam and foundation were modelled as a continuum mesh body, discretised with a hybrid of hexahedral, tetrahedral, pyramid and wedge 3D solid elements. The model is hexahedral dominant, with other elements only used in the vicinity of sharp edges and faces. The dam and foundation FE model are shown in Figure 10 and comprises 240 000 nodes and 460 000 elements.

The lower limit of the foundation continuum model extends to a depth twice the dam height below the lowest dam foundation. The horizontal outer limits of the foundation model extend to 1.5 times the dam height from the upper left abutment, upper right abutments and the most upstream face of the centre arch (FERC, 1999).

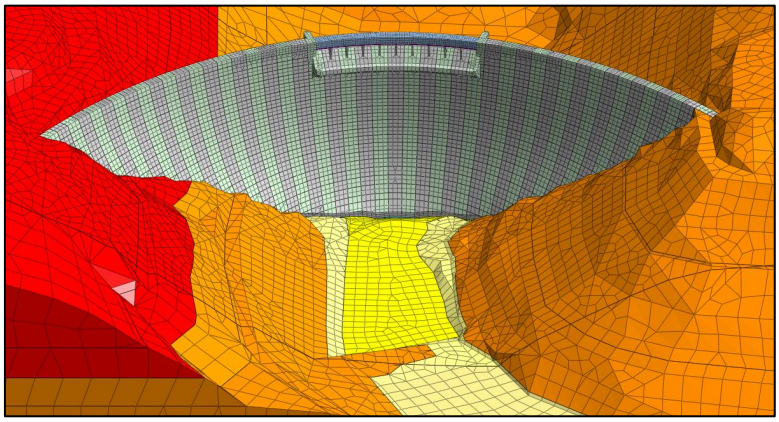

Figure 10. Katse Dam and foundation FE model

The incorporation of the seismic belt and pre-formed joint into the FEA model is shown in Figure 11, where smaller element sizes were adopted to capture the finer details of these components.

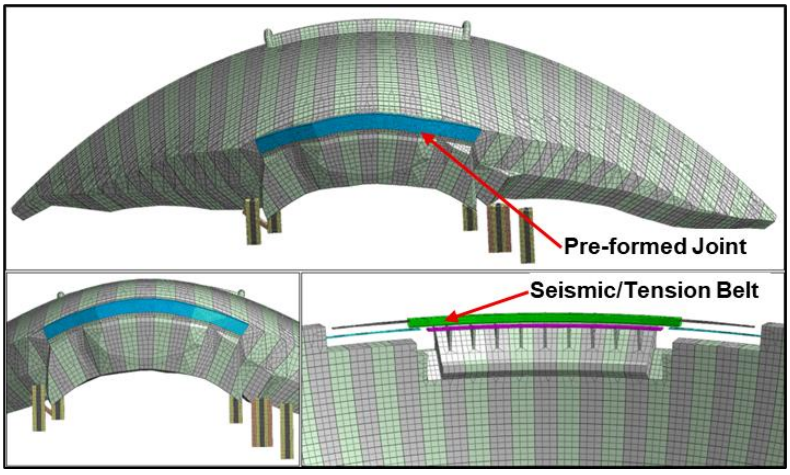

Figure 11. Finite Element structural mesh of seismic belt and preformed joint

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6.3 Hydrodynamic loading and damping

For dynamic loading, suitable Westergaard added mass elements were modelled on the upstream face of the dam to replicate the effect of the dam reservoir during dynamic excitation. It has been shown in published literature that the Westergaard Added Mass approach using a massless foundation is conservative and acceptable (ICOLD, 2021).

According to Wieland (2022), damping in linear-elastic dynamic analysis of concrete dams under strong ground shaking will typically indicate a value between 5 % and 12 %. Consequently, the applied Rayleigh damping of 5% for the dam-foundation structure under seismic loading is considered conservative.

7. ANALYSIS RESULTS

7.1 Results Summary

A summary of the linear elastic seismic analysis results for load case 1 and 2 is shown in Table 6. Minor exceedances of allowable tensile stresses were found in the vertical tensile stress response at the heel of the dam under load case 1 and in the horizontal tensile stress response along the arch near the crest.

Table 6. Seismic analysis critical results

Load Case	Arch Stress (MPa)		Cantilever Stress (MPa)		Minor principal Stress (MPa)		Peak radial displacement (mm)
	U/S face	D/S face	Heel	Toe	U/S face	D/S face	
1a	-7.5	-7.7	5.0	-6.9	-8.9	-12.5	104
2	3.0	3.1	1.1	1.0	-3.2	-6.4	43

Structural non-linear analyses were conducted to investigate the impact of stress redistribution associated with the function of the preformed joint, the contraction joints and the seismic belt. Under shaking motion in the non-linear analyses, significant tensile forces ultimately manifest as momentary openings of the contraction joints, as the nominal tensile stress capacity is exceeded. The results of non-linear analyses indicated that the dam has sufficient structural capacity to safely redistribute unfavourable tensile stresses under the critical seismic loads.

7.2 Appraisal of the pre-formed joint

While opening of the pre-formed joint was confirmed in the analysis displacement results, as shown in Figure 12 (left), operation of the joint to release tensile stresses can clearly be seen in the reduced vertical tensile stresses at the heel of the arch structure. A comparison of the vertical stress state on the upstream face under the maximum static load case with and without the pre-formed joint is also presented in Figure 12 (right). As can be seen, a maximum vertical tensile stress of the order of 2.5 MPa at the heel is effectively redistributed. A similar redistribution of tensile stresses at the heel of the dam under earthquake acceleration loading, in the downstream direction, was demonstrated.

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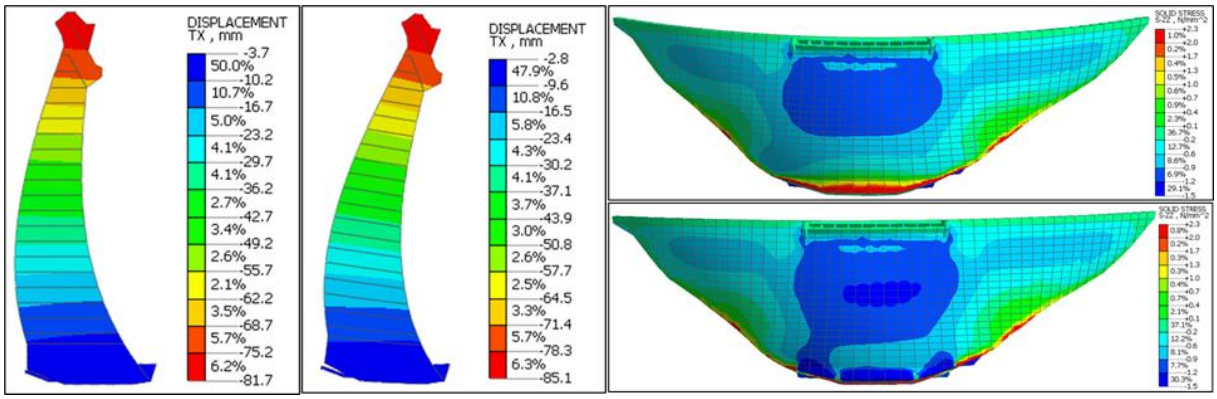

Figure 12. Radial deformation of dam crown cantilever excluding and including preformed joint (left) and upstream vertical stress state of dam excluding and including preformed joint (right)

7.3 Appraisal of the seismic belt

A comparison between the modelled structural deformation response of the dam under seismic load case 2, assuming both omission and inclusion of the seismic belt, is shown in Figure 13. The displacement plots indicate the upstream displacement of the crest of the dam structure during the simulated seismic acceleration event. The applicable response mode causes the development of tension along the dam axis, which results in the opening of the contraction joints. The seismic belt, however, can be seen to anchor the dam monoliths together, mitigating the development of vertical stress and consequential horizontal cracking on the downstream face.

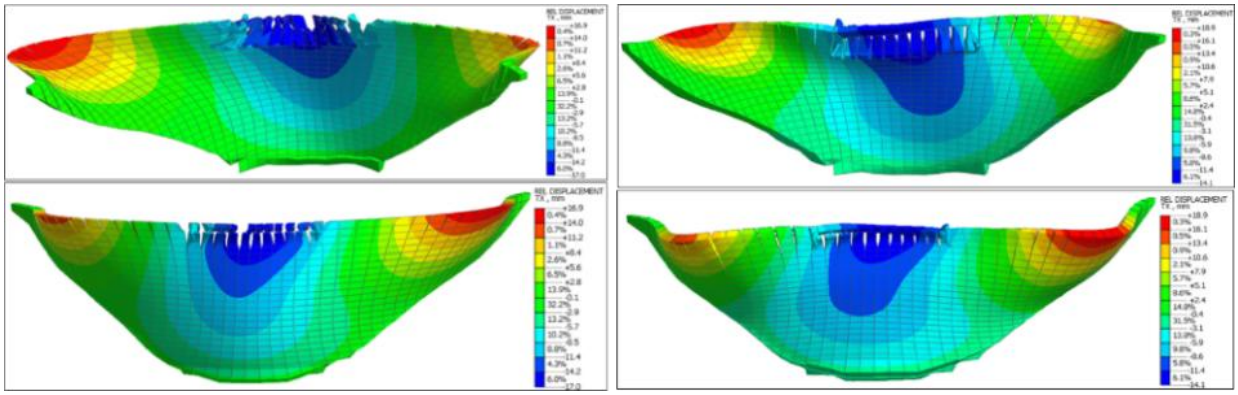

Figure 13. Exaggerated displaced shape of dam showing momentary opening of joints with omission (left) of seismic belt and inclusion (right) of seismic belt in FE model features

The time-history structural response of the dam under the Sparks Oklahoma OBE event without the tension/seismic belt is shown in Figure 14. At 13 seconds into the earthquake event, the analysis predicted a situation of failure, as the contraction joints open and cause all the cantilevers to act independently. A maximum instream displacement of 780 mm is predicted, with maximum vertical tensile stresses of the order of 30 MPa.

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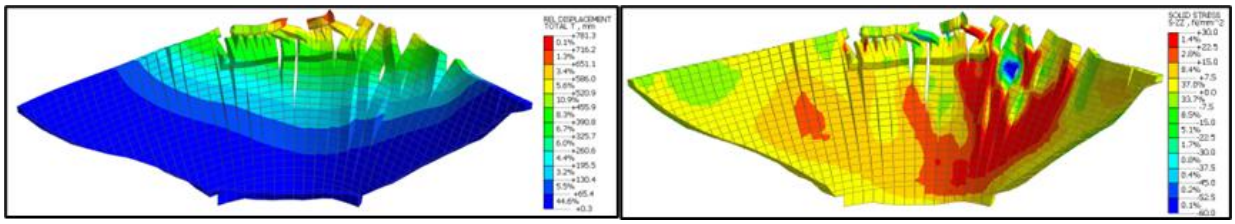

Figure 14. Displaced shape (left) and vertical stress (right) showing joint openings at 13.2 s

Including the seismic belt, as illustrated in Figure 15, can be seen to prevent the central contraction joints at the crest from opening, which in turn effectively limits consequential vertical tensile stresses to below the concrete tensile strength.

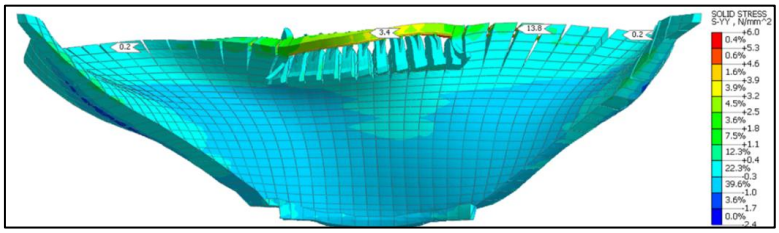

Figure 15. Horizontal hoop stresses at time step 13.2 s with seismic belt

8. CONCLUDING REMARKS

A comprehensive structural evaluation of the Katse Dam arch structure and foundation was undertaken under static and dynamic loading conditions using finite element analysis procedures. Overall, the analyses successfully confirmed that the structural performance and global stability of the dam under current conditions meets present-day international standards of safety.

Under extreme loading conditions, minor exceedances of the allowable tensile strength were indicated at the heel of the dam for linear elastic modelling. Applying a non-linear tensile limit mechanism at the pre-formed joint, subsequent non-linear analyses confirmed fully acceptable structural behaviour. The preformed joint allows the base of the crown cantilever of the dam to separate from the foundation at the upstream heel, increasing thrust into the arches and giving rise to the development of tensions further up the abutments on either side. All consequential tensile stresses, however, remain comfortably within the analysis criteria.

The modelled linear elastic dynamic response of the dam demonstrated the development of significant horizontal tensile stress along the axis of the arch, clearly confirming the need for the crest tension/seismic belt.

Although not exceeding the allowable maximum tensile stress for the concrete, the analyses confirmed that the computed tensile stresses under low reservoir and seismic loading exceed the negligible tensile strength of the contraction joints, causing them to experience momentary opening. Analysing a low reservoir level seismic load case with structural non-linearities confirmed the acceptable performance of the dam structure. The analyses further confirmed the effectiveness of the seismic belt in limiting joint openings and controlling consequential downstream face tensions to within acceptable limits. Considering the above it appears that the inclusion of a seismic belt in the Katse Dam was beneficial and improved the seismic performance of the dam, especially under low reservoir conditions.

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ADVANCING CLIMATE CHANGE COMPLIANCE IN TAILINGS FACILITIES: THE ROLE OF STOCHASTIC CLIMATE MODELLING AND GISTM FRAMEWORK

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PRESENTER: LEON DE KOOKER

ABSTRACT

As climate change continues to intensify the frequency and severity of extreme weather events, tailings storage facilities (TSFs) in mining operations face increasing vulnerability and regulatory scrutiny. This paper addresses the urgent need for robust climate change compliance strategies by exploring the combined application of stochastic climate modelling and the Global Industry Standard on Tailings Management (GISTM) framework. Stochastic modelling provides water engineers with an essential tool for probabilistic risk assessment, enabling the identification of rare but high-impact events that may threaten facility safety. The GISTM framework complements this approach by establishing a governance structure that embeds transparency, stakeholder engagement and risk-informed decision-making (ICMM et al.,2020). Together, these tools support adaptive water management strategies, ensuring resilience of TSFs in the face of uncertain climatic futures. The paper demonstrates how this integration enhances operational safety, informs infrastructure design and aligns tailings governance with evolving regulatory and societal expectations.

1. INTRODUCTION

Tailings storage facilities represent some of the most complex structures within mining operations. Their safe functioning requires careful integration of geotechnical, hydrological and operational considerations. Water management is central to this challenge, as uncontrolled stormwater inflows, high phreatic levels and inadequate freeboard can all compromise stability (SANCOLD, 2015). Traditionally, design parameters for TSFs have relied heavily on historical climate records, under the assumption that the past provides a reliable indicator of future variability. However, the accelerating effects of climate change are eroding the validity of this assumption (IPCC, 2021).

The Intergovernmental Panel on Climate Change (IPCC) has reported an increasing likelihood of intense rainfall events across many mining jurisdictions (IPCC, 2021). For water engineers responsible for mine residue facilities, these changes translate directly into higher risks of overtopping, dam break and downstream flooding. At the same time, regulatory bodies and investors are applying unprecedented scrutiny to climate risk management. Mining companies are increasingly required to demonstrate climate-resilient designs and provide evidence that operations are aligned with global standards, including the GISTM (ICMM et al.,2020). This paper explores how the integration of stochastic climate modelling with the GISTM framework can provide a technically sound and governance-driven approach to climate resilience.

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2. CLIMATE CHANGE AND MINE RESIDUE FACILITIES

2.1 Hydrological Vulnerability

The performance of TSFs is intrinsically linked to rainfall, evaporation and catchment inflows. In regions where rainfall intensities are increasing, stormwater runoff can exceed the capacity of diversion channels or spillways, leading to excessive ponding on the facility. This reduces freeboard, increases seepage pressures and elevates the risk of embankment instability (ANCOLD, 2019). Climate-driven variability in evaporation rates also affects water balance models, complicating predictions of pond size and decant requirements.

These challenges are not hypothetical. Several high-profile TSF failures worldwide have been triggered or exacerbated by water mismanagement under extreme weather conditions (ICOLD, 2013). The consequence of failure is catastrophic, both environmentally and socially and as such water engineers must anticipate not only common conditions but also extreme, low-probability events.

2.2 Regulatory and Social Pressures

The mining industry operates under increasing environmental and social scrutiny. Climate resilience is now a regulatory expectation, with compliance frameworks in jurisdictions such as Canada, South Africa, and Australia requiring explicit consideration of climate change in TSF design. Moreover, investors and insurers are demanding transparent demonstration of risk management practices. In this context, mine residue departments are at the forefront of delivering technically credible and socially responsible solutions.

3. STOCHASTIC CLIMATE MODELLING FOR MINE RESIDUE FACILITIES

3.1 Moving Beyond Historical Records

Deterministic approaches to climate analysis, which typically apply fixed design storms such as the 1:100 year rainfall event, are insufficient under rapidly changing conditions. Stochastic modelling offers an alternative by simulating thousands of potential rainfall and runoff sequences, producing a distribution of outcomes rather than a single value. This probabilistic lens is crucial in identifying tail risks which are rare events with potentially severe consequences.

3.2 Application to Water Engineering

For water engineers, stochastic modelling has direct applications in:

- **Water balance modelling,** where inflow variability can be explored under a spectrum of future scenarios;
- **Freeboard design**, by quantifying exceedance probabilities for extreme storms (SANCOLD, 2015);
- **Spillway and decant infrastructure sizing,** through simulation of rare but plausible peak inflows; and
- **Emergency preparedness planning**, by evaluating consequences of outlier events and designing appropriate contingency responses.

These models are particularly valuable in regions with limited historical rainfall records, where reliance on the past would otherwise provide a false sense of security. By framing uncertainty explicitly, stochastic tools enable adaptive designs that are robust to a wider range of futures.

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4. THE GLOBAL INDUSTRY STANDARD ON TAILINGS MANAGEMENT (GISTM)

4.1 Principles of the Standard

The GISTM, released in 2020, sets out 15 principles that collectively establish a framework for safer tailings management (ICMM et al., 2021). Central to these principles is the recognition that TSFs pose material risks to human safety, the environment and mining companies' reputations. The standard requires operators to adopt a lifecycle approach, encompassing planning, construction, operation, closure and post-closure.

4.2 Relevance to Water Engineering

For mine residue departments, the GISTM elevates water management to a strategic priority. Key expectations include:

- Ensuring water levels are controlled within safe operating limits at all times;
- Demonstrating climate resilience through documented design assumptions (ANCOLD, 2019);
- Implementing monitoring systems that can detect changes in water balance or freeboard conditions; and
- Engaging stakeholders transparently, particularly when climate risks may influence downstream communities.

The GISTM thereby creates a governance framework that compels engineers to integrate technical rigour with stakeholder accountability.

5. INTEGRATING STOCHASTIC MODELLING WITH GISTM

The combination of stochastic climate modelling and the GISTM framework creates a complementary system that addresses both the technical and governance dimensions of climate resilience. Stochastic modelling provides the probabilistic evidence base needed to quantify risk, while GISTM ensures that such evidence is systematically embedded into decision-making processes and operational controls.

For instance, probabilistic rainfall analyses can be used to establish operational triggers for freeboard management. Under GISTM, these triggers are formalised within Trigger Action Response Plans (TARPs), ensuring that operators respond consistently to emerging risks (SANCOLD, 2015). Similarly, stochastic outputs can be spatially integrated with Geographic Information System (GIS) tools to illustrate potential flood extents. This visualisation supports the stakeholder engagement processes mandated by GISTM, helping to communicate complex risks in accessible ways.

6. CASE APPLICATIONS

6.1 Freeboard and Extreme Rainfall

A TSF in Southern Africa subject to highly seasonal rainfall provides a useful example. The original design was based on a 1-in-100-year event, but stochastic modelling revealed a non-negligible probability of larger events occurring within the facility's operating life. Aligning with GISTM's requirement for continuous improvement, the operator revised freeboard allowances, resized spillways and updated operational protocols, improving resilience without compromising storage capacity.

Overtopping risk often arises not only from a single extreme storm but from sequences of smaller, seemingly negligible events that cumulatively stress the facility. Stochastic modelling enables this to be tested by generating multiple realisations of the rainfall time series that preserve the observed statistical

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properties while varying the timing and magnitude of events. Figure 1 shows an Mpumalanga Rainfall record alongside stochastic realisations that maintain the underlying statistics.

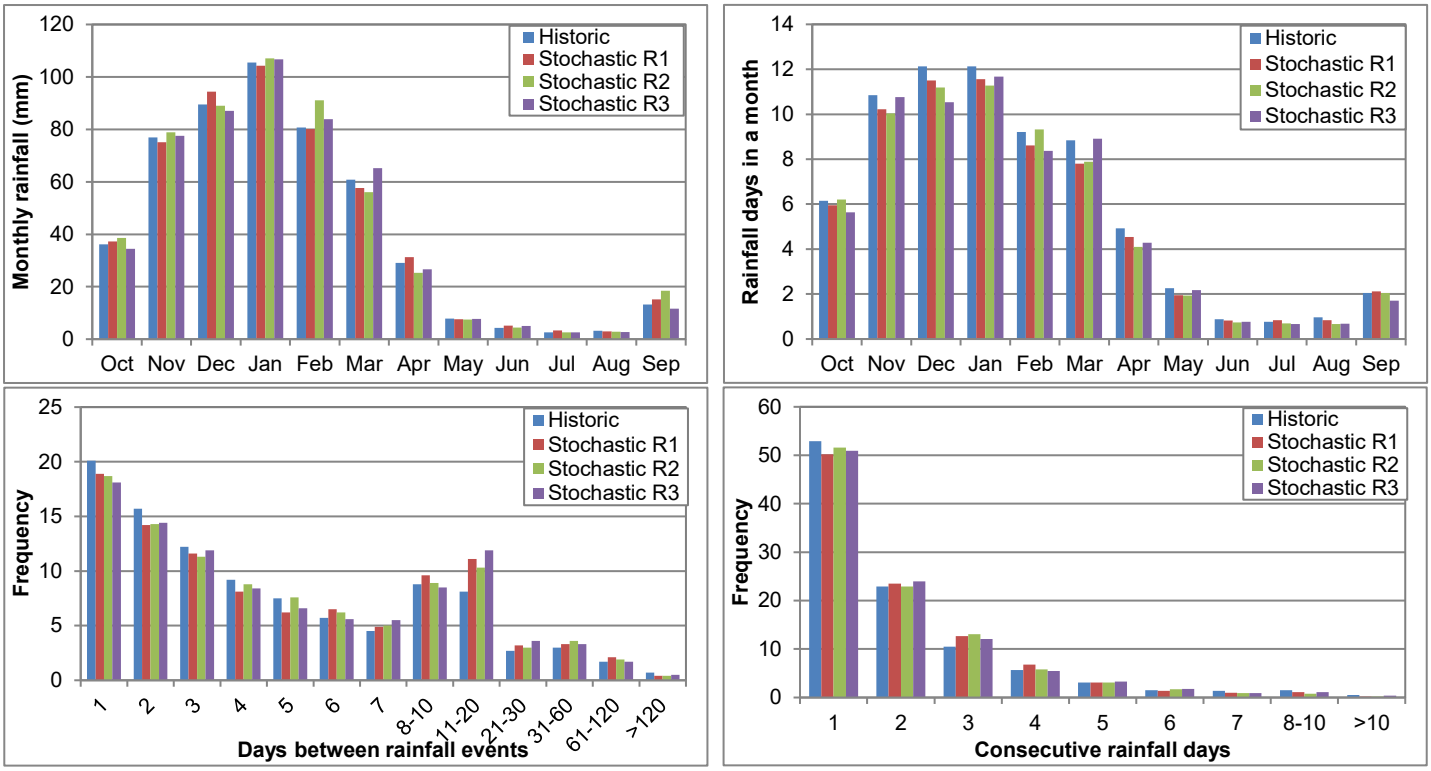

Figure 1. Rainfall record statistics historic vs stochastic

Figure 2 illustrates how these realisations reorder and rescale storms across the record, allowing the facility to be stress-tested across multiple event horizons.

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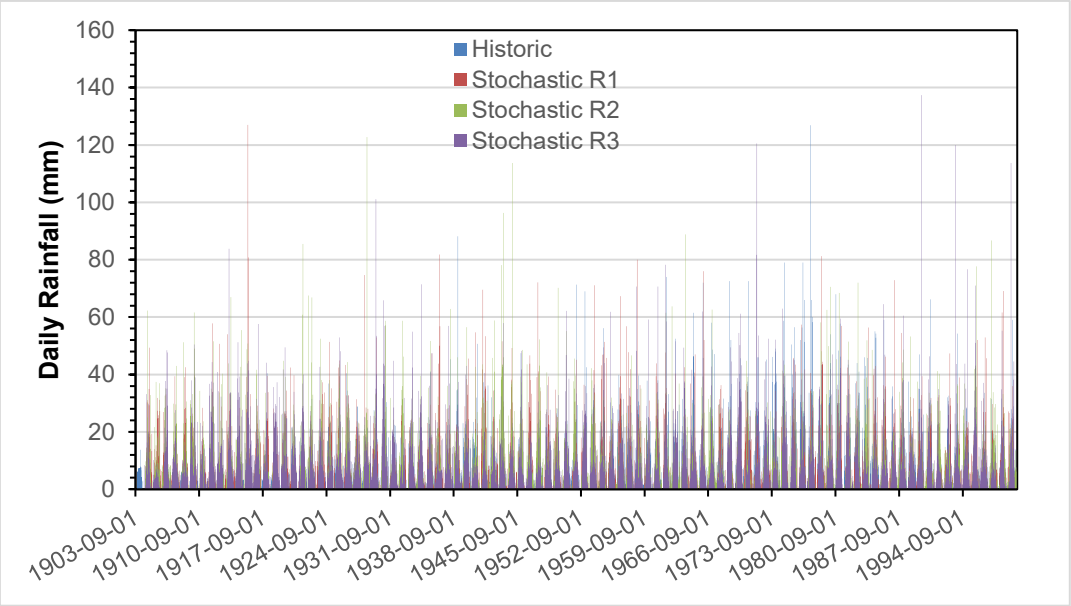

Figure 2. Daily Rainfall historic vs stochastic

6.2 Closure Planning Under Climate Uncertainty

Stochastic modelling has also been applied to closure planning, where water covers are used to suppress dust and improve environmental performance. Future climate scenarios showed that under certain rainfall distributions, water cover volumes could either exceed expectations or dry out entirely (IPCC, 2021). By applying GISTM's requirement for long-term stewardship (ICMM et al., 2020), closure designs were adapted to include contingency drainage and water supplementation strategies, ensuring compliance across a range of climate futures.

7. IMPLICATIONS FOR WATER ENGINEERS

The integration of probabilistic modelling with the GISTM has practical implications for water engineers in tailings operations. It shifts practice toward probabilistic thinking, where designs are tested against both typical conditions and extreme, low-likelihood outcomes. Engineers must also translate model outputs into governance instruments such as TARPs, monitoring plans, and stakeholder reports.

This modelling enables operators and the EoR to assess how the TSF behaves over its life and whether it remains compliant. Figure 3 shows simulated pool levels and demonstrates that, with appropriate TSF water management, compliance can be achieved and maintained throughout the facility's life.

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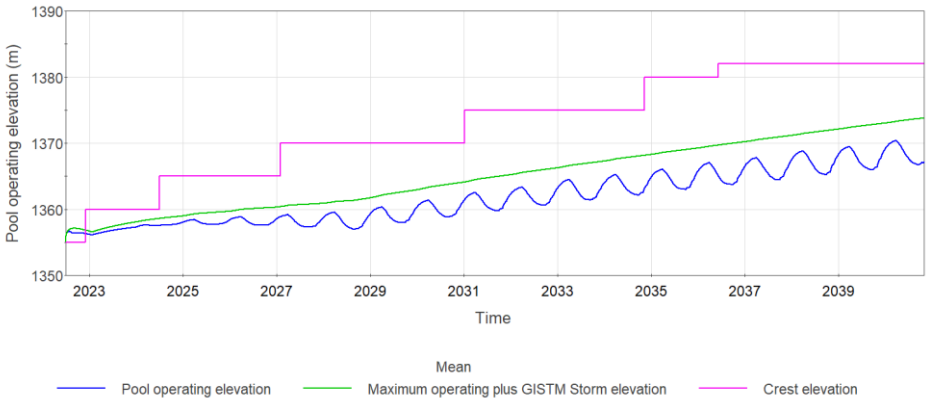

Figure 3. Pool operating elevation for TSF facility indicating GISTM compliance

Additionally, this integration broadens the engineer's role beyond technical design. Engagement with regulators, investors, and communities becomes a core part of practice, with modelling outputs providing the evidence base for transparent risk communication. This positions water engineers as both technical specialists and communicators of risk.

8. CONCLUSIONS

The mining industry faces a dual challenge: adapting its operations to climate change while demonstrating compliance with evolving global standards. For TSFs, water management lies at the heart of this challenge. Stochastic climate modelling provides the tools to quantify and manage uncertainty, while the GISTM framework ensures these insights are embedded in a culture of accountability and safety (ICMM et al., 2020).

By combining these approaches, water engineers can design and operate mine residue facilities that are resilient, compliant and sustainable. This integration not only addresses immediate operational risks but also supports the long-term social licence to operate for mining companies. In an era of climatic uncertainty, proactive adaptation is essential, and the mine residue department has a central role in leading this transformation.

9. ACKNOWLEDGEMENTS

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EVALUATING THE ROLE OF ESG IN ENHANCING CLIMATE CHANGE ADAPTATION AND SUSTAINABILITY FOR APPLICATION IN LHWP PHASE II

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ABSTRACT

Communities affected by large-infrastructure projects like the Lesotho Highlands Water Project (LHWP) are increasingly becoming aware of the detrimental and long-term social and environmental impacts of dam construction. Driven by environmental protection and socio-economic beneficiation, the implementing authority's 10-year strategy highlights its dedication to fostering sustainable socio-economic beneficiation through the development and management of water resources and electricity generation. However, amidst the increasing climate challenges and risks, the sustainability of LHDA's efforts might be threatened throughout the project-cycle. The study evaluates the role that ESG frameworks can play if incorporated to promote climate resilience and sustainability in large-scale infrastructure projects. The study will apply an evaluative research design, employing a qualitative methodology through a review of academic literature and published reports on ESG and climate resilience, alongside semi-structured interviews with LHWP role players and community leaders. The study hopes to inform policy adjustment and offer recommendations for effective ESG-driven decision-making.

1. INTRODUCTION

The Kingdom of Lesotho and The Republic of South-Africa have invested heavily in constructing dams for the transfer of water to the Gauteng province in South Africa, and to drive economic development in the rural areas of Lesotho. Implementing the Lesotho Highlands Water Project (LHWP) Phase I and II has resulted in investment of billions of maloti, and formidable environmental, social and financial impacts have been experienced thus far. It is no secret that the success and sustainability of large-scale development projects are prone to extreme climate risks and different scholars argue that to address these drastic challenges, there is a growing need to integrate ESG in decision-making processes and infrastructure investment (Owojori & Anwana, 2024). Despite the growing global sustainability issues, the LHWP remains a thriving economic project in the mountain kingdom. This paper accounts for preliminary findings of an on-going research that aims to systematically answer: to what extent does the integration of ESG frameworks in the LHWP Phase II enhance climate resilience and sustainability in Lesotho and South Africa?

In these uncertain times, with climate change and its increasing threat to development, there is a need to address such drastic changes, there is a growing need to integrate and consider Environmental sustainability, Social responsibility and Governance (ESG) in decision-making processes (Owojori & Anwana, 2024). We have seen how the world has been transitioning from industrial and profit-driven development to a more sustainable way of development, where the environment is prioritized and this has in-turn resulted in the establishment of frameworks and principles that capture the essence of ESG which fully recognize, from all angles, the essence of what is truly meant by sustainable development

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(Işık et al., 2024). The environmental aspect of ESG motivates for governing principles to address the impacts of large scale infrastructure projects on the physical environment and shared resources which, in the case of LHWP is assets such as rangelands, sand, water etc (Trahan and Jantz, 2022).

According to Eskantar et al. (2024), ESG gained popularity in the 1990s and 2000s when it was used as a tool for measuring the sustainability and impact of investments. Hilson (2024) suggests that in the early 2000s, this framework was adopted by companies to additionally identify sustainability risks, and, unlike corporate social responsibility, ESG has been argued to be less political. The financial sector views it as a tool that can help identify and manage climate-related risks and opportunities (Hilson, 2024).

The burning of fossil fuels and industrial practices have historically accelerated climate change by increasing greenhouse gas emissions. This has contributed to global warming and its associated risks such as rising sea levels, extreme weather events, and ecosystem destruction that can potentially disrupt business continuity. The integration of ESG criteria is important in evaluating these risks and providing guidance for investment decisions (Oliver, 2024). According to Riyath & Jariya (2024), in developed nations like Europe, some parts of Asia and USA, there is a growing tradition of companies from different industries applying the ESG tool to guide their entire operations. It has been argued that assessing ESG in the construction industry in particular, should be done from all angles, from procurement of goods and services to the actual implementation of projects (Riyath & Jariya, 2024) In the European Union (EU), ESG principles have been integrated into regulations as a tool to address climate change and promote climate resilience, and it is a requirement that companies are transparent about their impacts on the environment, the communities in which they work and governance (Stamelos, 2023). Riyath and Jariya (2024) further argue that the balance between an organisation's impact on the protection of the environment, how it treats its stakeholders and how it follows ethical regulations on how it is run, is just as important as the economic value the project has. In other words, balancing financial goals with social impact is critical (Oliver, 2024). Additionally, ESG has the potential to attract climate adaptation investors, which essentially improves a company's long-term success.

Companies such as ExxonMobil Inc, a petroleum company in the United States, used ESG frameworks to build a green business focusing on biomass technology, not only is it bringing in financial returns, it also supports Sustainable Development Goals (SDGs) by ensuring that Greenhouse gases are reduced (Chang and Kim, 2022). Additionally, non-financial components or benefits of ESG integration are not limited to an improved reputation among stakeholders, it is paramount to acknowledge the complexity of large-scale infrastructure projects, where different industries are inter-connected for a holistic goal. Chang and Kim (2022) argue that considering ESG at organization level and management level, especially during the construction phase is essential.

ESG is indeed steadily becoming a fundamental practice globally, for which a variety of analytical frameworks have already been developed (Eskantar et al., 2024). This approach is argued to guide businesses and nations towards sustainable economic growth that is both socially and environmentally responsible (Mukhiyayeva et al., 2024).

In answering how ESG can be strategically embedded in megaprojects to promote sustainability in and influence climate change adaptation, a preliminary desk-top review of articles obtained from three databases has been done so far. The paper proposes an integrated conceptual model by linking SDGs to LHWP's social and environmental's key impact areas. Finally, this study's preliminary findings may be of significant value to the practitioners and policy-makers in incorporating sustainability dimensions in the management of megaprojects.

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2. RESEARCH METHOD

2.1 Research design

The design of this research is evaluative in nature, it focuses on determining the effectiveness and relevance of ESG integration in large projects to promote climate resilience and achieve sustainable development (Trymata, 2025). The evaluation will be specific to the LHWP, the unique environmental conditions and people affected. In this way it aims to provide useful information that will guide LHDA policy makers and improve existing environmental and social programmes within the LHWP (Trymata, 2025). At this stage, data collection is still ongoing, and the current analysis provides only initial insights into opportunities for aligning LHWP practices with the UN Sustainable Development Goals (SDGs).

2.2 Data collection methods

The study employs a qualitative research methodology which will involve several stages. Firstly, an extensive literature review of academic and other published work exploring key concepts, theories and frameworks, as well as case studies about ESG and climate resilience. This will be followed by conducting semi-structured interviews with key stakeholders, which include role players within the LHWP, including LHDA Management, consultants and contractors, project managers, environmental managers and members of the Project Management Unit (PMU). The interviews will provide the flexibility of having a mix of structured questions and open-ended questions allowing interviewees to provide their insights and opinions (George, 2022). Additionally, a focus group discussion with community leaders from affected local communities will be conducted to ascertain the collective community perspectives (George, 2022).

3. PRELIMINARY FINDINGS: RETHINKING INVESTMENT FOR CLIMATE ADAPTATION AND SUSTAINABILITY

It is important to remember that at this stage of this research project, secondary data collection is ongoing, while structured interviews with key informants and focus group discussions with affected communities are yet to be conducted. Therefore the current analysis reflects early insights, with further empirical evidence expected from stakeholder engagement. The preliminary quantitative findings of article reviews revealed the following:

3.1 Project sustainable strategy at micro-level

Integrating environmental, social and governance principles at all operational levels is essential for sustainable socio-economic beneficiation. This includes detailed assessments of current project impacts on the environment, particularly pertaining to resource use. There should be specific, measurable sustainability objectives for each contract under the LHWP. Ensuring that sustainability objectives are embedded into all service provider's core business.

Referring to SDGs will support the LHWP to develop sustainability programmes that can be communicated to stakeholders, including project investors (Kim and Chang, 2022). Regardless of the multifaceted nature of the LHWP, there are non-financial and non-engineering factors that affect the success of a project, these influence the overall performance of the project. Social responsibility and employee health and safety in one particular are elements that remain relevant. ESG is therefore important to incorporate as core to enhance positive image of the project while maintaining the sustainability of each company within this sector (Kim and Chang, 2022).

3.2 Standard Reporting frameworks – Sustainability and ethical impact

According to United Nations Office for Project Services (UNOPS) (2022), infrastructure development is responsible for approximately 79% of total greenhouse gas emissions and 88% of climate change related impacts. These highlight the need for serious accountability and accuracy in reporting to support

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the efforts to combat climate change. Additionally, UNOPS (2022) stresses that climate adaptation involves adjusting systems and practices to cope with the changing climate, and acknowledges that climate-related shocks and stressors have undermined the stability of infrastructure systems and their ability to operate and provide essential services for communities (UNOPS, 2022). Investment in sustainable infrastructure would therefore be a major component of mitigation strategies in LHWP, for example investing in clean energy, and water treatment, irrigation systems and other climate resilient infrastructure is crucial.

4. CONCLUSION

The integration of ESG considerations into large infrastructure projects is not just a moral imperative. Given the challenges of climate change, it can also be seen as a strategic move for organisations like LHDA to enhance their reputation and manage risks effectively. The multifaceted benefits of a sustainability oriented strategy creates a pathway to the future profitability and success of global infrastructure development (Sciarelli, Cosimato, Landi & Landolo, 2021).

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ENVIRONMENTAL FLOW REQUIREMENTS AND ECOLOGICAL STATUS DOWNSTREAM OF KATSE DAM

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ABSTRACT

Lesotho Highlands Development Authority (LHDA) implements its Environmental Flow Requirements (EFR) policy and procedures to maintain the ecological state of rivers, which provide ecosystem services to communities downstream of dams in response to the Lesotho Highlands Water Project (LHWP) Treaty of 1986. LHDA operates in a sustainable way by preserving aquatic biota through monitoring the impact of releases on river ecosystems in April and October each year. Katse Dam(KD) operating rules release low flows, freshets, or floods, depending on decisions made to maintain the health of the ecosystem downstream. EFR informs multicriteria decision-making at LHDA that dictates the trade-offs between allocating water for electricity generation, ecological reserve, supply of raw water to South Africa, and maintaining dead storage. Environmental experts monitor the trajectory of change from targeted ecological condition at proximal and distal sites. The present ecological state (PES) is derived from an integrated analysis of data using models based on drivers and responders of ecological change. Drivers include hydrology, water quality, and geomorphology, while responders comprise riparian vegetation, fish and macroinvertebrates. The 2023/2024 PES at EFR 2 was 43.4%.

1. INTRODUCTION

Rivers are aquatic systems that provide a range of ecosystem services. They are complex systems within which geomorphological, hydrological and ecosystem processes interact (Dollar et al., 2006). There is a need for an interdisciplinary approach to understanding rivers as ecosystems and to protect them from exploitation due to ever-increasing pressure to provide cities with clean water from dams globally. Barriers such as dams disrupt the nature of rivers as continuous systems and impact the seasonality, size, duration and frequency of river flows, thereby, affecting the ecological good and services provided by these rivers (Van Vuuren, 2012). Thus, it is this concern that necessitated participation of scientists, engineers, environmental managers and social scientists to enable sustainable development and understanding of the allocation of water for ecological reserve and EFR in dam projects (Arthington et al., 2023).

The EFR can be described as the quantity, the timing and quality of freshwater flows and levels necessary to sustain aquatic ecosystem which in turn support human cultures, economies, livelihoods, and wellbeing (Arthington, et al., 2018). At the base of maintenance of environmental-flows (EF) is the understanding that the riverine environment is not to be considered in competition with other users, but it is at the origin of the water itself. Hence for development of water resources to be sustainable, the source of the water should be prioritised in all aspects (King and Louw, 1998). Changes in frequency and size of floods, brought about by construction of flow impeding structures affect the ecological goods and services provided by these rivers through reduced biodiversity and fishing and recession agriculture and livestock rearing. These modifications increased the vulnerability of people whose livelihoods depended on these rivers. According to Van Vuuren (2012), dam designers develop operating rules geared to maximize the abstraction of water for the highest possible economic benefit.

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In recent history, appreciation for ecological value for aquatic ecosystems and their role of river systems in the supply of water and the maintenance of livelihoods of downstream communities has increasingly become a binding focal point in the design and construction of dams. River flow alterations for water abstraction need to be balanced with environmental flows (Kumar and Jayakumar, 2020).

The development of dam operating rules shifted focus to addressing the concern by Kumar and Jayakumar (2020) that maintaining only low flows without consideration of the wider range and timing of the flows, is not sufficient for the maintenance of the health of the river ecosystems and their services. According to Hughes and Ziervogel (1998), South African water law was amended to specify that a part of the natural flow regime of a river should be reserved for environmental maintenance during the planning stage of major water development schemes. To benefit the planning stage of dams, Hughes et al. (1997) developed the Instream Flow Requirements (IFR) model that derives a flow regime in the form of monthly flow values, to sustain a river in a desired future state. These values come as either low (baseflows), high (freshets and floods) flow requirements. However, the ultimate pattern of actual releases introduces flow variability that reflect natural climatic occurrences within the catchment. The IFR results into a modified flow regime which informs dam release authorities about the desired set releases to maintain the river in a desired future condition (Hughes et al. (1997).

The evolution of methods for the management of environmental flows was driven by the rising demand for water abstraction to meet the needs of the growing population. According to Brown and King (2011) South African (SA) become increasingly aware that the construction of large-scale dams impacts the ecosystem of rivers when the subject of environmental flows and their assessment was brought in discussion in Southern Africa in the late 1980s. It was at this time integrated into legislation as part of dam operating rules in many countries. The issue of IFR for river maintenance was first addressed nationally in South Africa in 1997 (King and Louw 1998).

The 1980s marked a necessary shift in the aspiration to intensify water abstractions to provide and meet growing demand. The paradigm shift in policy to directed efforts to protect the riverine ecosystem from overexploitation informed the emergence of various EF models and methods to manage the water resources from a holistic point of view. The first and most comprehensive national attempt to manage EF and sustainable river functioning using a scientific method was an assessment of the Instream Flow Incremental Methodology (IFIM) by King and Tharme (1994) developed by the US Fish and Wildlife (King and Louw, 1998). However, IFIM inadequately addressed the DWAF (1991) policy objective to manage complete riparian and instream components of river ecosystem and describe a flow regime that could not only sustain ecosystem health but also address the needs of downstream communities for good water quality and quantity. Eventually, it was evident that a new method that could sufficiently identify and describe the timing, duration, frequency, quantity and quality of the EF specific for a river was necessary to integrate as part of any water resource development (King and Louw 1998). The basis of the approach was to develop a method that could describe a flow regime that maintains the desired ecosystem state. The situation necessitated the evolution of a Building Block Methodology (BBM) developed from inputs by water managers, environmental managers, engineers and social consultants who contributed technical information on social aspects, scientific, hydrological and hydraulics aspects of the water development projects.

Following the development of the BBM, a scenario-based method was developed by researchers in South Africa (SA) with the objective of deriving a holistic approach to environmental flows (King et al., 2004). The new method, called Downstream Response on Imposed Flow Transformations (DRIFT), was able to predict the economic and social impacts of river changes on local users of river ecosystems. The method addressed intra-annual and inter-annual flow regime and all living and non-living aspects of the river ecosystem from its headwaters to discharge into the sea (King et al., 2004). The advantage of the DRIFT was that it described the different kinds of flow (low flows, small floods, large floods and flow variability) and their significance to the river ecosystem functioning. DRIFT comprises four modules namely: (1) the biophysical module, which describes the present nature and functioning of the ecosystem and to develop predictive capacity under various flow modifications; (2) the sociological

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module, identifies the subsistence population at risk and how the flow modifications impacts their use of ecosystem services; (3) The third module integrates the biophysical and sociological module and (4) the module looks into the costs of mitigation and compensation that result from the results of application of the first three modules (King et al., 2004).

In the case of KD, the objectives of DRIFT were to establish the expected ecological condition of the downstream rivers ranging from natural to severely modified, to identify resources valued by the downstream communities and to predict the effect of the yield of KD. The information provided by implementing DRIFT including the effect of the royalties received by Lesotho and the cost of compensating the downstream communities for predicted in river resources was key in the establishment of the first EFR Policy and procedures under Contract 648 of the LHWP (Brown and King 2000). In the preceding years before implementation of the EFR Policy and Procedures Contract 648, the need to determine applicable releases to protect the downstream ecosystem became evident during the Environmental Impact Assessment(EIA) of the Mohale Dam in 1994 (Brown and King, 2011). While World Bank identified and pointed out the significance of establishing a scientifically valid Environmental flows program, most Non-Governmental Organisations (NGOs) lamented the absence of such a program from design phase of the LHWP (Metsi Consultant, 2000).

KD is a strategic water resource with multiple uses including raw water supply, electricity generation and aquaculture production (SMEC 2019). It is an engineering marvel of the 20th Century, constructed high in the Maloti mountain of the Kingdom of Lesotho, hence its "water tower" designation by many. In 1987 following the signing of the LHWP Treaty in 1986, the Lesotho Highlands Consultants Consortium was appointed by the LHDA for site investigation to define and characterise the geology of the site and understand the foundation conditions to inform the design and optimize the exact location of the concrete curvature arch (McKellar 1994). The scarcity of rainfall in South Africa coupled with rapidly deteriorating water quality in the Vaal River system motivated the planning and execution of LHWP (Metsi Consultant 2002, Wright, 2006).

According to LHDA (2020) EF only came into the discussion in the mid1990s when the design stage of the KD was completed and the dam was already under construction. The LWHP Treaty (GoL, 1986b) provides for compensation of all parties and communities affected by the project both upstream and downstream. Article 7(18)(GoL 1986b) compels the project authorities to "effect all measures to ensure that members of the local communities in the Kingdom of Lesotho, who will be affected by flooding, construction works or similar project-related causes, will be able to maintain a standard of living not inferior to that obtained at the time of first disturbance, provided that such Authority shall effect compensation for any loss to such member as a result of such project-related causes not adequately met by such measures".

The Treaty obligations are binding and inform the establishment of the environmental flow Policy and procedure according to Article 15(GoL 1986b), which states:

"The Parties agree to take all reasonable measures to ensure that the implementation, operation and maintenance of the Project are compatible with the protection of the existing quality of the environment and, in particular, shall pay due regard to the maintenance of the welfare of persons and communities immediately affected by the project".

The Treaty also provides for the maintenance of minimal flow in rivers according to Article 6(9)(GoL 1986) of the LHWP Treaty. Despite the fact that the first environmental flows emphasised maintenance of minimal flow level in rivers, with the perception all the river ecosystems are conserved as long as the levels is kept above a certain critical level, later eflows assessments methods emphasized the significance of the entire flow regime that consists of large floods, small floods, low flows and medium flows (Acreman and Dunbar, 2004).

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The objective of this paper is to highlight the findings of the EFR biophysical monitoring undertaken during the 2023/2024 downstream of the KD. The Paper also seeks to; (1) Illustrate that LHDA Environmental Flow Requirements (EFR) policy and procedure, complies to international best practice and protocols; (2) illustrate that the robust seasonal (April, October) monitoring collects a vast amount of data for reporting annually and informing management decisions on compensation; (3) illustrate that the findings inform management interventions and decisions to strike a balance on trade-offs between social needs (fish, wood, good water quality and continuous flow of rivers downstream of our dam walls) as well as water delivery obligations that feature in the LHWP Treaty.

2. STUDY AREA

The construction of KD began in early 1991 deep in a gorge along the Maibamatso River 2km downstream of the confluence with the Bokong River a major tributary (Ramsighn et al. 1998). The dam's construction on coordinates 29.3369 °S, 28.5061°E (Brown and King, 2011) was completed in 1997 (Ramsighn et al. 1998). Morphometric characteristics of the dam are as shown in Table 1. The dam is a 185m high curvature concrete arch dam with full supply level capacity of 1950 x 106 m^3 a-1 a full supply level of 2053masl (Brown and King 2011). The dam construction was completed in 1998 (Brown and King 2011).

Table 1. Morphometric characteristics of KD

Morphometric characteristics	Statistics
Catchment area	1869 km2
Capacity at full supply level	1950 x 106 m3
Surface area at full supply level	38.5 km2
Hight above foundation	185 m
Crest length	710 m
Total length	Approximately 35 km
Maximum width	900 m
Intake tower position upstream of Damwall	18 km2

The dam was fitted with low level outlet below which was the tail way pond to cushion the impact of large overflows from the dam to maximise dam safety. The low-level outlet and the control gates were installed to regulate the reservoir level as achieved during the construction of the dam and afterwards (McKellar, 1994). However, Brown and King (2011) lament the design restriction of the outlet structures to release a mid-range inter annual floods to facilitate optimal redistribution of flows as percentage of the of the mean annual run-off (MAR) to effect EFR effectively.

2.1 Approach and Scope

The construction of KD came at a time when the BBM and DRIFT methods were being tested for applicability on large-scale dam construction projects (Brown et al. 2000). DRIFT was developed in actual water resource development projects including the Palmiet River and Breede River in South Africa, and in the LHWP (Metsi Consultants, 2000). The strength of the DRIFT method relied on its predictive capacity, where specialists used data to predict consequences. The DRIFT method is a holistic approach that incorporates a social assessment (King et al. 2003). In the development of DRIFT, the focus of each specialist was on predicting how flow manipulations will impact that component of the

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ecosystem. Another advantage of DRIFT includes public consultation for consideration of the wider macro-economic implications of the operational scenarios. Finally, at the end of the process, following inputs from all stakeholders and scenario creation, the management class for the river is defined and the targeted river condition comes out of expert deliberations. (King and Louw 1998). These advantages were prominent in the use of DRIFT method to work out the baseline condition of KD EFR downstream of LHWP structures.

LHDA contracted consultants to determine EFR for the project in 1997 to 2002. The project utilised the predictive and socio-economic scenario definition capability of the DRIFT method. According to the EFR Policy and Procedure LHDA (2020), a prominent aspect of the study was the estimation of the potential losses to communities and associated mitigation due to reduces flows. The inputs for the estimation were value of the resource loss such as fish, wild vegetables, medicinal plants, wood, and mitigation of impacts such as increased public and animal health risk (LHDA, 2020). A rigorous public consultation process was undertaken in 16 downstream communities to find information to create scenarios and feed into the DRIFT model (King and Brown, 2011).

While the technical driver for the EFR release remains DRIFT (King et al., 2002), EFR sites are monitored within an Adaptive Management System (AMS) that captures changes from reference conditions and thus informs management interventions. The nature of the AMS being an ideal approach to management of natural systems (Williams and Brown, 2014) calls audit and review of the EFR Policy and procedures in a frequency of five years to improve management of the project such that following the review and external audit, LHDA adapts their interventions to ensure continuous improvement (LHDA, 2020). LHDA undertakes internal annual evaluations of the impacts of releases and produces an annual report detailing the results of the EFR annual monitoring (LHDA, 2020). The external audits are provided with these annual reports to inform their external audit process.

2.1.1 Ecological Classification

Eco-classification is a process used to determine the present river condition of biophysical factors of the river relative to reference conditions. Ecostatus refers to the overall river features, its characteristics and the riparian zone enabling the river to provide the requirements for existence of certain types of fauna and flora which provide river users with ecosystem goods and services. Ecostatus of a river/site represents the determined state by use of drivers (hydrology, geomorphology and physico-chemical (WQ)) and responses (Fish, macro invertebrates and riparian vegetation) (Kleynhans and Louw, 2008). The categorisation of Present Ecological Status(PES) is based on integration of scores generated by models as indicated in Table 2.

Table 2 presents the EC categories from A to F where A denotes no change (pristine river condition), E denotes Seriously Modified and F stands for critically/extremely modified state in relation to reference conditions. The EC determination of fish response discipline takes into consideration diversity of native/reference species, introduced species, flow, cover, velocity-depth classes, instream migration barriers and physico-chemical conditions of water. Macroinvertebrates include diversity in taxa and biotopes, flow, velocity and physico-chemical conditions. Riparian vegetation takes into consideration abundance, vegetation cover, species composition, population structure and recruitment (Kleynhans and Louw, 2008).

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Table 2. Ecological Categories and Description of Ecological Conditions (Kleynhans, 1998).

Ecological Category	Generic Description of Ecological categories	Arbitrary Guideline Score (%of Maximum Theoretical Total)
A	Unmodified /Natural, close to natural or close to predevelopment conditions within the natural variability of the system drivers: hydrology, physico chemical and geomorphology. The habitat template and biological components can be considered close to natural or predevelopment conditions. The resilience of the system has not been compromised. The system and its components are in a close to natural condition most of the time. Conditions may rarely and temporarily decrease below the upper boundary of a B category.	>92 - 100
A/B	The system and its components are in a close to natural condition most of the time. Conditions may rarely and temporarily decrease below the upper boundary of a B category.	>88-<=92
B	Largely natural with few modifications a small change in the attributes of natural habitats and biota may have taken place in terms of frequency of occurrence and abundance . Ecosystem functions and resilience are essentially unchanged.	>82-<=88
B/C	Close to largely natural most of the time. Conditions may rarely and temporarily decrease below the upper boundary of a C category.	>78-<=82
C	Moderately modified. loss and change of natural habitat and biota have occurred in terms of frequencies of occurrence and abundance. Basic ecosystem functions are still predominantly unchanged. The resilience of the system to recover from human impacts has not been lost and its ability to a moderately modified condition following disturbance has been maintained.	>62-<=78
C/D	The system is in a close to moderately modified condition most of the time. Conditions may rarely and temporarily decrease below the upper boundary of a D category.	>58-<=62
D	Largely modified. A large change or loss of natural habitat, biota and basic ecosystem functions have occurred. The resilience of the system to sustain this category has not been compromised and the ability to deliver ecological goods and services has been maintained.	>42-<=58
D/E	The system is in a close to largely modified condition most of the time Conditions may rarely and temporarily decrease below the upper boundary of a E category. The resilience of the system is often under sever stress and may be lost permanently if adverse impacts continue.	>38-<=42
E	Seriously modified. The change in natural habitat template, biota and basic ecosystem functions are explosive. Only resilient biota may survive and it is highly likely that invasive and problem (pest) species may dominate. The resilience of the system is severely	>23-<=38

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	compromised as is the capacity to provide ecological goods and services. However, geomorphological conditions are largely intact and but extensive restoration may be required to improve the system's hydrology and physico-chemical conditions.	
E/F	The system is in a close to critically modified condition most of the time. Conditions may rarely and temporarily decrease below the upper boundary of an F category	>18-<=23
F	Critically /Extremly modified. Modifications have reached a critical level and the system has been modified completely with an almost complete change of the natural habiat template, biota and basic ecosystem functions. Ecological goods and services have largely been lost. This is likely to include severe catchment changes as well as hydrological, physico-chemical and geomorphological changes. In the most instance, the basic ecosystem functions have been destroyed and the changes are irreveersble . Restoration of the system to a synthetic but sustainable condition acceptable for human purposes and to limit downstream impacts is the only option.	<18

2.1.2 Locations of EFR sampling sites

This paper focuses on presenting how the 2023/2024 EFR monitoring findings inform management interventions in carrying out EFR policy and procedure recommendations. The LHDA monitors EFR downstream of its structures at nine(9) EFR sites (Figure 1). The paper shall focus on findings of site EFR 2 because it is the proximal site located immediate downstream of KD.

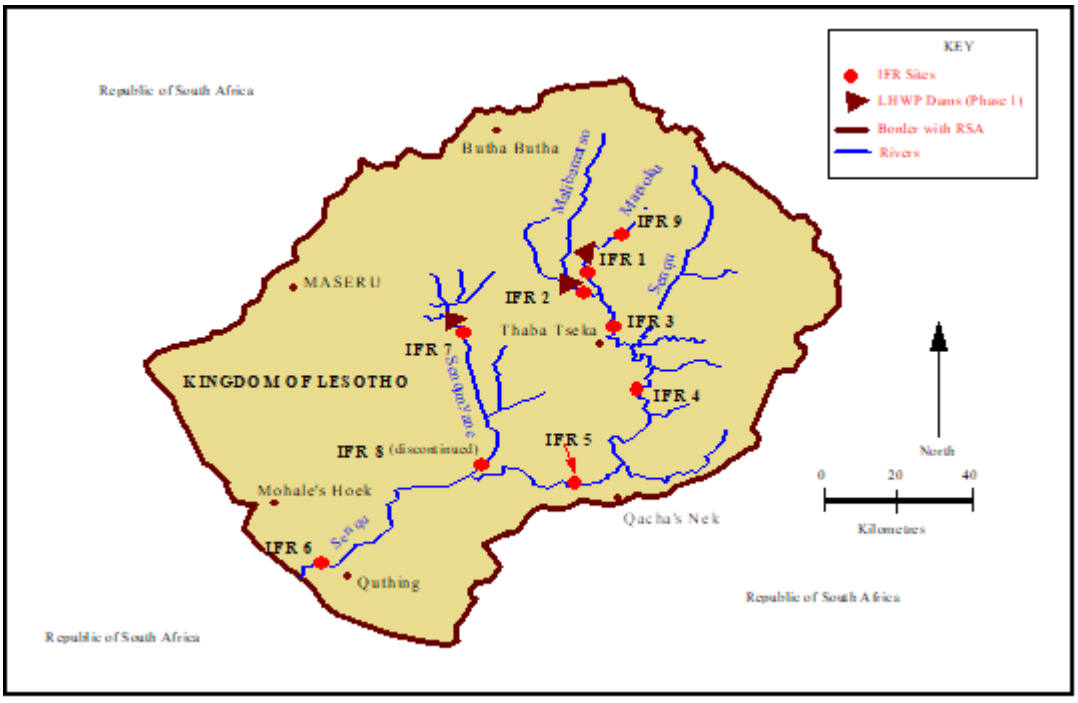

Figure 1. Location of EFR sites with EFR 2 downstream of Katse Dam

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3. METHODS

3.1 Audit and Review of EFR Policy and Procedure

EFR is subjected to an external audit every five years. The first assessment of the EFR was undertaken through LHDA Contract 648 and Contract 678 from 1996 to 2002 (King, 2000). The LHDA Contract 1237 was implemented in 2005 to 2007 (LHDA, 2004). The contract 1237 focused on revision of the River condition classes, development of specific monitoring protocols, reporting template as well as training in monitoring purpose and techniques (Pemberton and Brown, 2007). The LHDA contract 1250 EFR Audit was undertaken in 2007 by the Institute of Natural Resources (INR), SA. The recent EFR audit LHDA,1323 was conducted by JG Afrika and Ground Truth in 2019 and concluded in 2020. These audits served to inform advances in methods, management interventions and conformity to international best practice LHDA (2020). Among these changes include the introduction of Geomorphological Assessment Index (GAI), Vegetation Assessment Response Assessment Index (VEGRAI), Fish Response Assessment Index (FRAI) and Macroinvertebrates Response Assessment Response Assessment Index(MIRAI) models to determine the PES at as EFR site as outlined in Kleynhans and Louw (2008). An integrated assessment of drivers and pressures is also applied to determine the deviation of the PES of the habitat integrity of a river system from a reference state Kleynhans et al., (2008). The habitat integrity is divided into riparian and instream.

The objectives of the LHDA 2019 audit was to present the principles and methodology to determine EFR, provide guidance on releasing water to downstream rivers in accordance to downstream annual runoff and set dam operating rules to provide the agreed EFR allocations for water and to schedule EFR allocations on a daily basis (LHDA, 2020). The EFR Policy and Procedure audit (LHDA 2020) outlines the ten-step operational procedure for EFR calculations as well as the Monitoring programmes with detailed monitoring protocols.

3.2 Present Ecological Status(PES)

According to Graham et al., (2023) the PES is the current status of an aquatic system relative to its ideal natural state, also called the reference condition. The PES of each discipline is expressed as an ecological category (EC), a continuum of states from an Unmodified or Natural state (an A category river) through to a Critically or Highly Modified state (Table 3). The overall PES of the river is expressed as an integrated state, called the EcoStatus (Kleynhans and Louw 2008). LHWP keeps a continuous trend on changes it imparts on the rivers downstream, such that management decisions of compensation compacts are fully reliant on the EFR annual reports. Ecological Categories or integrated PES scores can be determined for each EFR monitoring site. These PES scores and ecological categories can be directly related to the target river condition classes to assess whether EFR targets are being met over time. This paper presents the modelled EcoStatus was based on an integration of the MIRAI, FRAI and VEGRAI results (Kleynhans and Louw, 2008). The present and targeted river ecological status are presented in Table 3.

3.3 Monitoring EFR Components

A team of LHDA experts in fish, water quality, hydrology and geomorphology work together to monitor river health downstream of LHWP dams at a frequency state in Table 3. These assess ecological status by monitoring drivers and responders of ecological change in the April high flow season and the October low flow season. Data from this biannual monitoring is analysed using standard and approved models developed by Thirion, (2007) and Kleynhans and Louw (2008). Results thereof are compiled into an EFR report whose findings serve to inform the ecological impact of releases downstream of dams and serves to safeguard river health downstream. The releases form part of the dam operating rules and come as low flows, freshets or floods depending on the decisions made to maintain the health of the ecosystem downstream.

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Table 3. Biophysical data collection activities for monitoring river conditions

	Frequency	Discipline	Data
Drivers	October	Geomorphology	Diversity and quality of physical habitat, GAI
			Cross section survey
			IHAS
	Continuous	Hydrology	Dam release schedules, and flow records at monitoring sites
		Water Quality	DO, Temp, pH, Conductivity and Nutrients
	Reactive Survey		Faecal coliforms and E. coli
Responders	April	Riparian Vegetation	woody and non-woody cover and fixed-point photographs/aerial imagery
			VEGRAI,
	April and October	Macro Invertebrates	MIRAI
			SASS 5 and Indicators species
		Fish	Species composition, relative abundance and recruitment
			FRAI
			Integrated Habitat Assessment Integrity

3.3.1 Geomorphology

The river channels were evaluated and assessed on the loss or gain of pool depth and habitat diversity and occurrence of bank erosion while studying the morphological sequences. A theodolite was used to monitor the permanent cross sections on the key features such as the pool, riffle and run/rapid and a cross-section sketch of transect were drawn. The PES value (in terms of geomorphology) was obtained from the GAI model (Rowntree, 2013). Category by GAI model based on three drivers, which are connectivity, sediment supply and channel stability (determining bed and bank stability). The model combines the weightings and the ratings of the three drivers to provide a final PES and the results are provided as categories A to F and as percentage scores.

3.3.2 Hydrology

LHDA adopted a policy of effecting downstream releases on a near real-time basis. EFR releases are set at 12.1% of inflows at KD (LHDA 2023). It is expected that in the end, the KD EFR releases would average 67.1MCM, which is 12.1% of an MAR of 554.8MCM. the information from Table 4 is sourced from LHDA (2023) and provides a summary of targeted releases as per the LHWP policy on environmental flows.

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Table 4. Environment flow releases as percentages of annual flows

EFR Site	Associated Control Structure	At EFR Site		From Control Structure		
		MAR[(1)] (million ^{3}/a)	Total Annual EFR as % Runoff	MAR (million m^3/a)	Annual EFR Releases	
					As % Inflow	As % Runoff at EFR Site
2	KD	576.0	15.3	554.8	12.1	11.6

3.3.3 Water Quality

Water quality monitoring is undertaken biannually during low flows in October and high flows in April. Measurements for in-situ water quality parameters are done using data loggers and water is sampled as per monitoring procedures at each EFR site. These water samples are kept in cold preservation until they are submitted to the laboratory for analysis. The laboratory results and in-situ variables are analysed to form an input into the water quality aspect of the report.

3.3.4 Macroinvertebrates

Macroinvertebrates samples are collected from suitable biotope types (stones, vegetation, gravel, sand and mud) at each EFR site using SASS5 net, placed in a SASS tray, identified, and recorded in a SASS5 scoring sheet according to procedure (Dickens et al., 2002). These are scored accordingly and assigned number of taxa (NOT), SASS5 score and average score per taxa (ASPT) on SASS5 score sheet. To obtain an ecostatus at a river reach, indices are determined through use of MIRAI index modelling. The modelling approach is based on rating the degree of change from natural on a scale of zero (0) (no change) to five (5) (maximum relative change) for various metrics. According to Kleynhans and Louw (2008), ecological classifications are based on presence and frequency of occurrence of macro-invertebrates is evaluated by totalling the weighted score and expressing this as a percentage of the maximum. The MIRAI achieves this by subjecting data to impact of hydrology, geomorphology, and water quality on macroinvertebrates assemblage. The ecological classification value obtained indicates the percentage change away from the expected reference. This percentage value is used to place the EC in a particular category that ranges from A to F as shown in the Table 2.

3.3.5 Fish

For fish response monitoring, an integrated habitat assessment is conducted to gain insight on general land use, habitat cover rating, water abstraction, geomorphic change, erosion, instream migration barriers, pollution and encroachment by alien plant species. The site is divided into three (3) sub-sites that represent different habitat types. Habitat types are identified in terms of water depth and velocity as slow-deep, slow-shallow, fast-deep and fast-shallow; to cater for different hydraulic conditions preferred by different fish species (Kleynhans, 2007). River depths below 0.5 m were considered shallow, while those above were considered deep.

3.3.6 Vegetation

Delineated sampling plots within the EFR sites were monitored and the assessment was done within the plot's boundaries. . The VEGRAI model was computed using the results and observations from site to workout the Eco-status.

4. RESULTS

Results come in the form of trajectory of change on instream variables, including fish and macroinvertebrates from targeted river condition. These results follow changes in the hydrology, water quality and geomorphic feature in the river channels. Riparian vegetation also changes from time to time

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due to geomorphology and hydrology. These biotic and abiotic changes are integrated into a PES for the EFR site 2.

4.1 Hydrology

The Environmental Flow Requirements (EFR) are calculated daily and adjusted accordingly based on the amount of river inflows. Based on Tables 5 the targeted flow releases were generally maintained throughout the reporting period except for January 2024 where the system experienced significant spilling at both reservoirs. According to the EFR policy, these spills are considered part of the EFR and are accounted for as part of Actual EFRs.

Table 5. Monthly variation of environmental flow releases from KD in 2024

Month		Estimated Total Target (MCM)	Actuals (MCM)			End of Month Variance		Monthly Releases Variability
			Dam Release	Spills	Total	(MCM)	%	
Jan		3.44	3.44	7.89	11.33	7.89	+229	+199
Feb		1.99	1.99	0	1.99	0	0	0
Mar		1.08	1.08	0	1.08	0	0	0
Apr		2.07	2.07	0	2.07	0	0	0
		8.58			16.47	7.89	91.9	49.75

4.2 Geomorphology

The geomorphology of the EFR site 2 was impacted by the releases as shown by the distribution of instream sand bars that separate two water channels in Figure 2 below. The water channel is characterised by wandering pattern caused by the deposition of sediment from the left and right sides of the water channel. This is a result of badly degraded catchment that is actively eroding. Due to sediment deposition, there is development of insert benches (Figure 2) and encroachment by the alien wood species. According to Tooth and Viles (2014), the presence of terrestrial species on insert benches indicates absence of water flow. On the positive side, the presence of terrestrial species on the insert benches stabilises the riverbanks preventing scouring and bank erosion but can also attracts livestock (Wan Mohtar et al., 2017)

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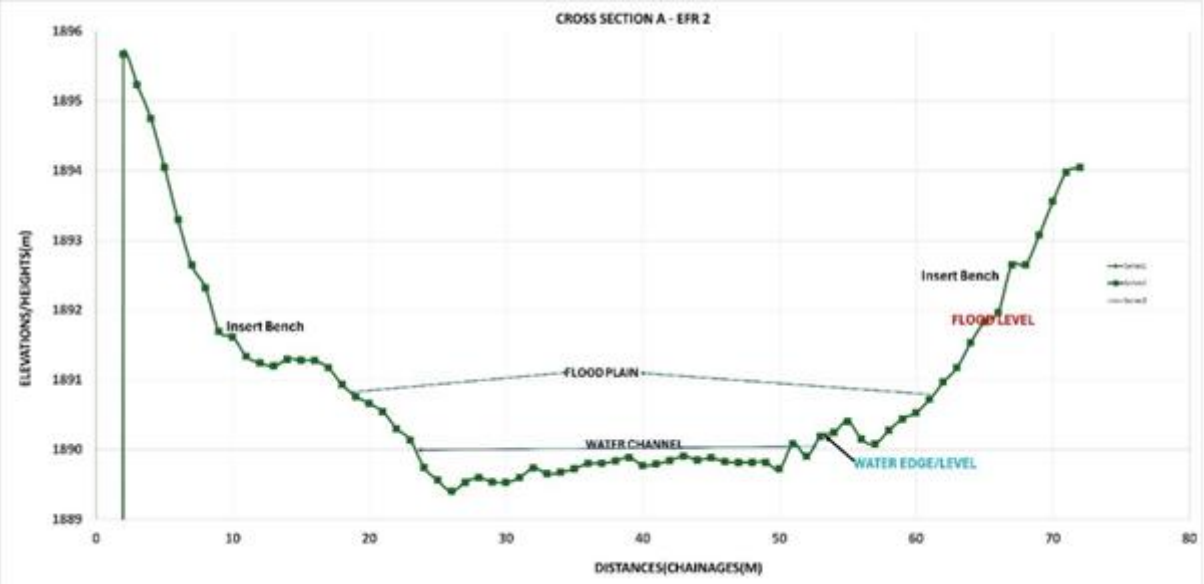

Figure 2. Section A – run from right hand side to left hand side of the channel

4.3 Water Quality

EFR 2 is a proximal site located 2 km downstream of Katse Dam wall. Temperatures at the site are lower than 13℃ throughout all seasons with have potential to cause thermal stress on fish. TSS and chemistry parameters are low and within the acceptable limits. Water is released from the lower hypolimnion layer of the Dam and with characteristic lower temperatures. The electrical conductivity is below the levels observed prior to damming of Malibamatšo indicating that the water is released from a dam which has the effect of settling sediments and ions.

4.4 Macroinvertebrates

Eco-status of IFR 2 with respect to MIRAI models has decreased from 66.2% in May 2022 to 65.4% in April 2023. Macroinvertebrates with a preference for cobbles and gravel such as *Trichorithidae* and *Blephariceridae* which were historically present at the site prior construction has also diminished while the emergence of *Chirinomidae* could be attributed to persistent low flows at Malibamatšo River below Katse Dam. Lack of flashing floods provides a condition for macroinvertebrates with preference for slow moving water and for algae to thrive.

4.5 Fish

The reference condition for EFR 2 provided the preferred habitat for *L. aeneus*, *A. sclateri* and *L. capensis* which occurred at highest abundance and frequency, historically. In 2024, the habitat consisted of rocky stretches and relatively steady flow preferred by *A. sclateri,* muddy pools and running water of large rivers preferred by *L. capensis* and clear flowing water of large rivers, deeper pools, backwaters, fast-shallow habitats with rocky/gravel substrate and absence of slow-shallow habitats as preferred by *L. aeneus* (Arthington et al., 1999; Skelton, 2001). Therefore, siltation of pools and substrate, proximity of the site to instream migration barriers (dam wall and weir), comparatively low water temperatures, restricted flow and reduced river width together with long-term invasion by non-native predacious species contributed to the observed disappearance of two reference fish species, *L. capensis* and *A. sclateri*, which were respectively last recorded in 2016 and 2014. The decline in the abundance of *L. aeneus*. is attributed to the non-native predacious *O. mykiss*, which is favoured by constantly low water temperatures (annual average =10°C) and reduced flow conditions (LHDA, 2006; Arthington et al. 1999)

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was recorded at a declining abundance. This site also attained an ecological class of F(14.2%), hence is critically modified.

4.6 Vegetation

EFR 2 being proximal to dam wall, showed a constant trajectory of deterioration over the monitoring years and there has been a significant downward trend on *Gomphostigma virgatum* and *Salix fragilis* since 2018 while *Salix mucronata* has been fluctuating. *Artemisia afra* has been declining slightly since 2018. A sharp decrease in *G. virgatum* and *S. fragilis* might be due to reduced channel observed during monitoring season and recruitment seedlings is not sufficiently maintained by river flow. Significant instream deposition of sediment encroached the channel and formed sand bars invaded by *Rumex woodii*. This condition inhibited recruitment of woody riparian obligates species. The VEGRAI calculation on the other hand indicated a moderately modified condition with category C(67.2%) Eco-status.

4.7 Present Ecological Status

The Overall PES result for EFR 2 is calculated from instream response of fish and macroinvertebrates and riparian response of vegetation to hydrology and geomorphology changes following releases shown in Table 5 as dictated by dam operating rules depicted in Table 4. The PES result for each component is shown in Table 6.

Table 6. River Ecological Classification at EFR 2 for different components in 2024

Component	Baseline condition at EFR 2 as per policy	Projected Policy status	EC component % score	Geomorphological change	Trend
GAI	B	D	56.7 (D)	Largely modified	On target
MIRAI	B	D	65.4(C)	Largely modified	On target
FRAI	B	D	14.2 (F)	Critically modified	Worse
VEGRAI	B	D	62.2(C)	Moderately modified	Above target

The PES monitoring results with associated trajectory of change from reference condition indicated in Table 7.

Table 7. Integrated Present Ecological Status of IFR sites for 2024

EFR site	Pre-dam condition	Predicted condition	2021 – 2023 Ecological category	Integrated Ecostatus 2011-2023	2023 – 2024 Ecological category	Integrated Ecostatus 2023-2024	State	Actual relative to Prediction
2	B	D	48.1	D	43.4	D	Largely modified	On target

5. CONCLUSION

PES results of 43.4% for the proximal site, EFR 2, is on target against a predicted condition of D This demonstrates that LHDA appreciates rivers as living systems that provide ecosystem services to communities and organisms that rely on it for survival. Environmental flow releases from the LHWP reservoirs play a crucial role in maintaining the ecological integrity of riparian life in the downstream of

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the impounded rivers. Habitat for fish and macroinvertebrates is restored while instream and riparian seedlings are uprooted during release of floods. The entire ecosystem thought deteriorating in some proximal and distal sites, shows gradual restoration following heavy rains and flood releases that scoured sediment in the river channels downstream. The entire EFR project demonstrates that LHDA complies to international best practice standards and frameworks. LHDA carries out its operations in a sustainable way that cares for the environment and communities affected by the impoundments that restricted the natural river flow. LHDA continues to consult Experts to undertake external EFR audits according to Treaty obligations and prescriptions LHDA also trains its component specialist to monitor EFR according to new advanced methods recommended by external audits

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RESOLVING DRAINAGE DESIGN CONFLICTS IN A DAM RAISING PROJECT

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ABSTRACT

The Tzaneen Dam, built in the 1970s, requires upgrades to meet modern standards. A key design component, the downstream toe drain, was re-evaluated as part of the raising project. However, discrepancies between the original design drawings and the actual constructed system resulted in design changes during construction. Comprehensive investigations, including surveys, excavations, inspections, and water quality tests, were undertaken to determine existing conditions and finalise the alternative design for implementation. This paper highlights lessons learned, emphasising the importance of accurate as-built records, thorough site investigations, and adaptable approaches in both design and construction phases.

1. BACKGROUND

Tzaneen Dam, located in the Mopani District of eastern Limpopo Province, is a composite structure with a central concrete gravity spillway and flanking zoned earthfill embankments. The dam has a crest length of 1 140 m, with the concrete gravity section 90 m long and 51 m above riverbed level. The embankments have a maximum height of 40 m. The dam was constructed in 1977 to secure water in a water-scarce region (ARQ, 2023).

As part of the Groot Letaba River Water Development Project (GLeWaP), the Department of Water and Sanitation (DWS) initiated the raising of Tzaneen Dam to increase the Full Supply Level (FSL) by 3 m to RL 726.99 m, expanding storage from 157.3 million m³ to 194.7 million m³. This involves modifications to the spillway, non-overspill crests, and ancillary works (ARQ, 2023).

Geotechnical investigations undertaken during the raising design revealed that the existing embankments did not meet modern safety requirements for a large Category III dam. To address these concerns, the raising design incorporates both crest elevation raising and embankment strengthening. The embankment non-overspill crest will be raised by 1.7 m to RL 732.30 m (ARQ, 2023). Strengthening measures include extension of the clay core, flattening of the downstream slope from 1V:2H, to 1V:2.6H with additional selected fill, and the construction of an upstream reinforced concrete retaining wall. With the flatter downstream slope, the works include the extension of the blanket and toe drainage system.

2. INTRODUCTION AND PROBLEM DEFINITION

The designs for the Tzaneen Dam Raising toe drain system relied on the original 1970s design drawings, but detailed site investigations revealed that the as-built drainage system did not conform to the documented layout. Discrepancies such as unrecorded connections, blocked pipes, variations in invert levels, and the combining of stormwater and embankment drainage flows became evident during excavation and testing. These findings highlighted significant uncertainty in the functionality and reliability of the existing system, creating risks for both dam safety and construction planning.

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In response, a comprehensive re-evaluation of the embankment drainage system was undertaken. The investigations included surveys, flow testing, and material inspections to establish the actual condition, configuration, and performance of the toe and blanket drains. The results informed the development of a revised drainage design that introduced two coordinated networks: an embankment pipe system extending beyond the new embankment toe and a blanket/toe drain system configured for sectional monitoring, separation of flows, and improved safety.

The extent of the discrepancies meant that most of the existing drainage infrastructure required replacement, which influenced project sequencing, planning, and costs. Close collaboration between the engineer and contractor was essential to implement redesigns in real time, manage uncertainties, and maintain progress on site.

This paper documents the investigations, design modifications, and construction implications, while highlighting key lessons learned. These include the importance of accurate as-built documentation, thorough site investigations, adaptable design strategies, and effective team collaboration in achieving dam safety and project objectives. By sharing both technical outcomes and practical experiences, the study offers guidance for future dam rehabilitation and upgrade projects.

3. ORIGINAL DESIGN DRAWINGS AND DAM CONSTRUCTION

The original 1970s design drawings for Tzaneen Dam provided a simplified representation of the embankment drainage system, as illustrated in Figure 1 and Figure 2. The embankment was constructed with a central inclined clay core, supported by a chimney and blanket filter comprising sand material. This filter was intended to intercept seepage from the upstream side and convey it downstream, where it discharged into a gravel toe drain running along the embankment toe. The toe drain was configured with several layers of fine and progressively coarser material, with embedded slotted pipes that are connected to a series of manholes. Seepage through the embankment and the foundation is collected and measured in the manholes, which ultimately drains laterally into the river.

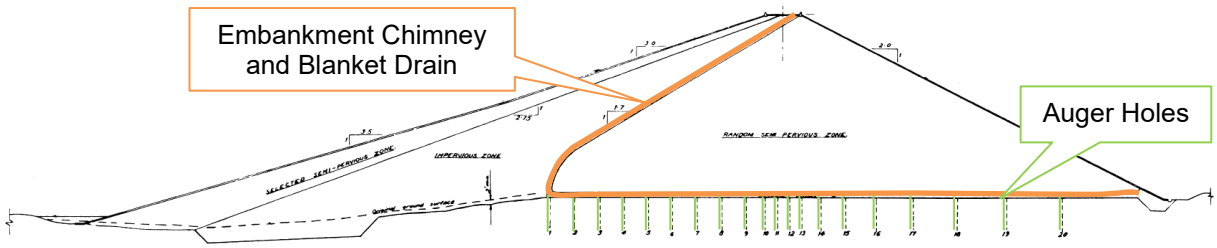

Figure 1. Embankment Drainage Section

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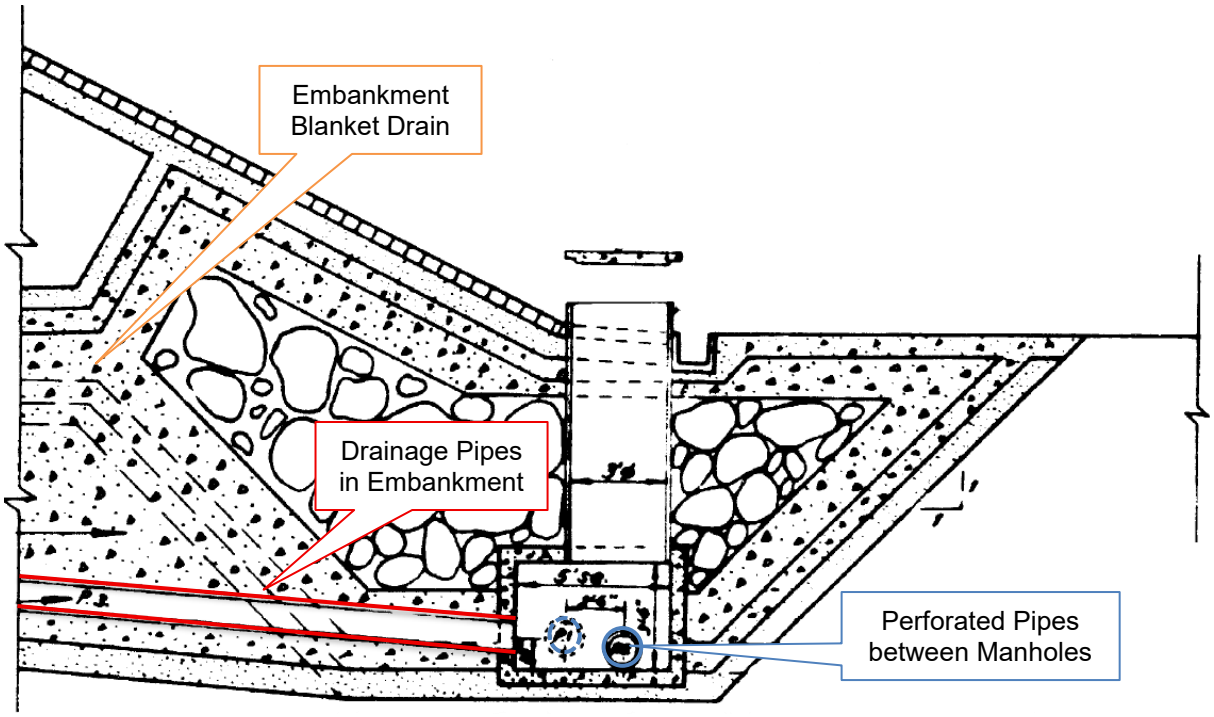

Figure 2. Toe Drain Section with Manhole

In addition to the blanket drain and toe drain arrangement, the original drawings indicated that the toe drain is fed by slotted drain pipes embedded in shallow trenches at particular intervals that run beneath the embankment for an indeterminate distance to the upstream (see Figure 3). These slotted pipes were detailed as relief drains intended to capture water pressures acting within the embankment foundation. The design drawings indicated that the pipes were positioned inside gravel material, aligned with auger-drilled holes in the foundation, allowing uplift water to be relieved and safely transferred into the manhole system.

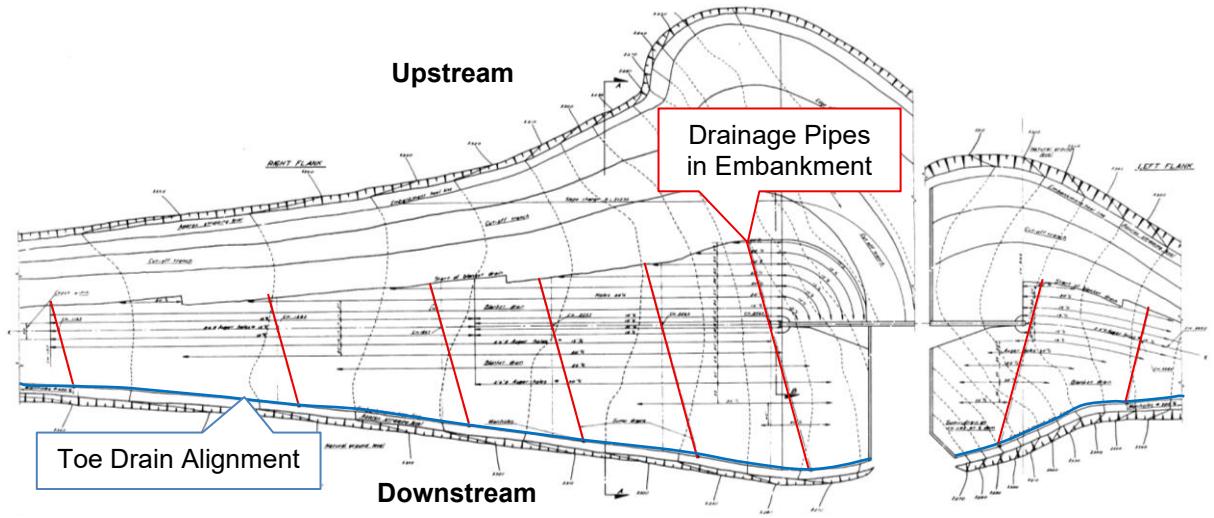

Figure 3. Plan Layout of Auger-drilled Holes and Embankment Pipes

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The downstream manholes therefore served multiple functions:

- Conveyance of seepage collected in the chimney and blanket filter via the gravel toe drain;
- Collection of water from the perforated foundation relief pipes; and
- Discharge of surface runoff and stormwater from the downstream embankment face and the access road at the embankment toe.

While this arrangement reflected contemporary design practice of the 1970s, subsequent investigations (see Section 4) revealed that the actual as-built conditions are more complex, and in some cases deviate from the original intent. The presence of additional connections, blockages, and mixed-use of drainage pathways demonstrated that reliance on the original design drawings alone is insufficient, and that an alternative drainage design is required based on the verified site conditions.

The design of the new embankment toe drains, and blanket drain was initially based on the available 1970s design drawings of Tzaneen Dam. These drawings formed the basis of the preliminary design work and assumed that the original drainage system had been constructed in accordance with the documented details.

However, during site planning for the construction of the new drainage system, it became evident that the original drawings did not accurately reflect the as-built conditions. These uncertainties made it clear that reliance on the 1970s records alone would not be sufficient for the successful integration of the new drainage works. Consequently, a series of site investigations was initiated to establish the actual condition, extent, and functionality of the existing toe drainage system. The findings of these investigations formed the basis for re-evaluating the proposed drainage designs and developing a solution aligned with the true site conditions.

4. SITE INVESTIGATIONS

As part of obtaining clarification on the existing toe drainage system, detailed site investigations were undertaken to confirm the layout, function, and condition of the existing toe drainage systems along both the left and right embankments.

4.1 Drainage Network Surveys

Surveys were carried out on the left and right embankment toe drain systems. The work involved identifying and recording the positions of existing manholes, establishing pipe alignments, and capturing invert levels at each structure. The results of this exercise are presented in Figure 4 and Figure 5, showing the spatial distribution of manholes, pipe connections, and outlet locations.

4.2 Visual Inspections

Visual inspections were carried out to assess the condition of the drainage systems along both the left and right embankments. At the outset, seepage and continuous saturation were observed on the left embankment, indicating potential deficiencies within the subsurface drainage network. In contrast, the right bank showed no significant signs of drainage-related issues.

The investigation included a series of manual inspection activities, such as opening and examining manholes, monitoring drainage performance during rainfall events, and identifying the location and functionality of existing drainage structures along the embankments. The following key findings were recorded:

- The left embankment side wall showed clear signs of water seepage, confirming saturated conditions within the embankment. Detailed inspection revealed that the two lowest manholes were completely flooded.

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- The embankment paving on this side appeared noticeably darker than that of the right bank, and the left-bank platform remained continuously wet due to water emerging from the embankment, indicating possible deficiencies within the subsurface drainage network.
- Multiple manholes, catchpits, and pipes were identified along the left and right embankment toe. These structures serve as collection and inspection points for the interconnected drainage pipes.
- Several manholes, with sumps, were found to be filled with water and silt, predominantly on the left bank. This gives an indication of insufficient maintenance over the years, and possibly the accumulation of silt in drainage pathways, reducing the effectiveness of the drainage system.
- Each manhole contained multiple pipe inlets and outlets, which were found to serve multiple unknown purposes. The complexity of this configuration contributed to difficulties in tracing the pipe routes and diagnosing blockages.

4.3 Water Flow Testing

To verify the direction and functionality of the drainage system, flow tests, with a white colour agent, were conducted at each manhole, starting from the upstream sections and progressing downstream to the outlets discharging into the river. These tests confirmed connectivity between the manholes and identified the dominant direction of flow within each pipe. Multiple pipes were identified, and these include inlet pipes from the embankment foundation drainage, connector pipes between manholes, stormwater channel connections, and outlet pipes to lower-lying manholes. The flow paths are illustrated in Figure 4 and Figure 5.

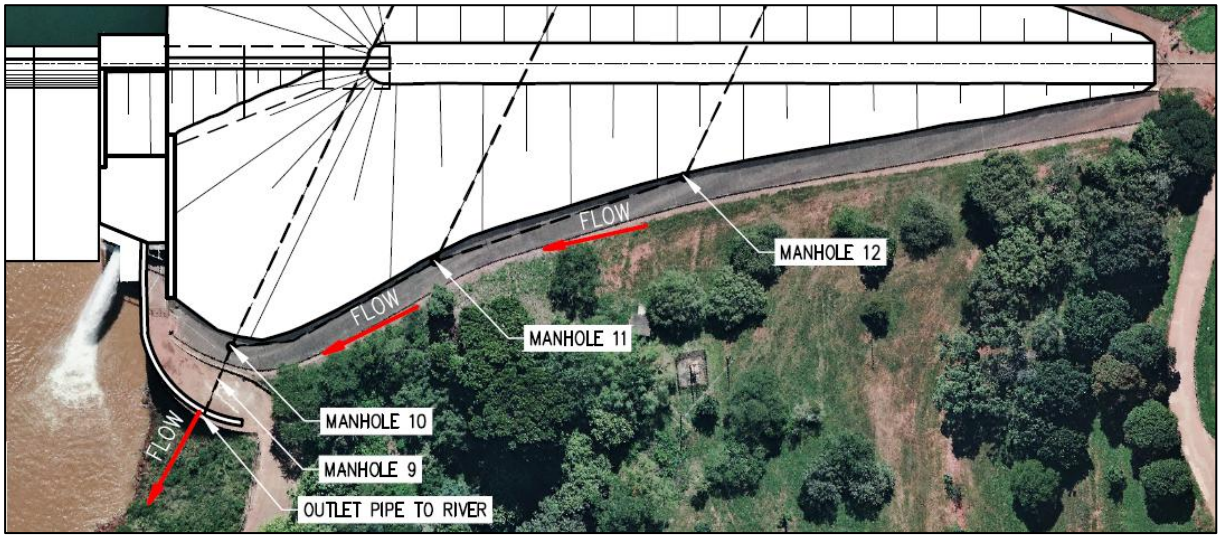

Figure 4. Manhole Flow Tests - Left Embankment

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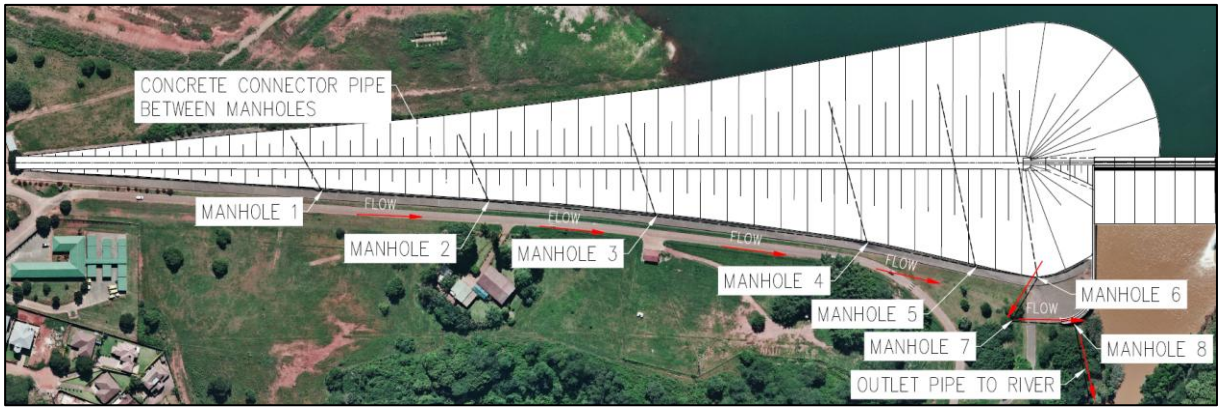

Figure 5. Manhole Flow Tests - Right Embankment

4.4 Drainage Excavations and Restoration

Targeted excavations were performed around selected manholes to expose and log the surrounding toe drain material profile. This provided clarification of the filter and drainage material placed during construction, including the gradation, thickness, and condition of the filter layers. Additional excavations were undertaken along the alignment of the toe drainpipes to establish the depth, dimensions, and material type. Photographic records are shown in Figure 6.

Figure 6. Excavations - Manhole and Toe Drain Pipes

In an attempt to restore functionality of these manholes, several dewatering attempts were undertaken by continuous pumping from the affected manhole into a 18,000-litre water tanker to determine the location of the outlet pipe as seen in Figure 4. However, despite these efforts, water levels remained constant, indicating that the system was not discharging, while inflow from the embankment and surrounding area into the manholes was still significant.

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Figure 7. Left Embankment - Cleaning of Blocked Manholes

Following rigorous cleaning and repeated dewatering attempts of the manholes and connected pipes, the internal configuration of the left bank manholes was mapped and confirmed. It was subsequently determined that the cause of the persistent saturation was a blocked outlet pipe, which prevented effective drainage of the embankment. Figure 8 below shows the reconfiguration of the drainage system behind the left bank sidewall.

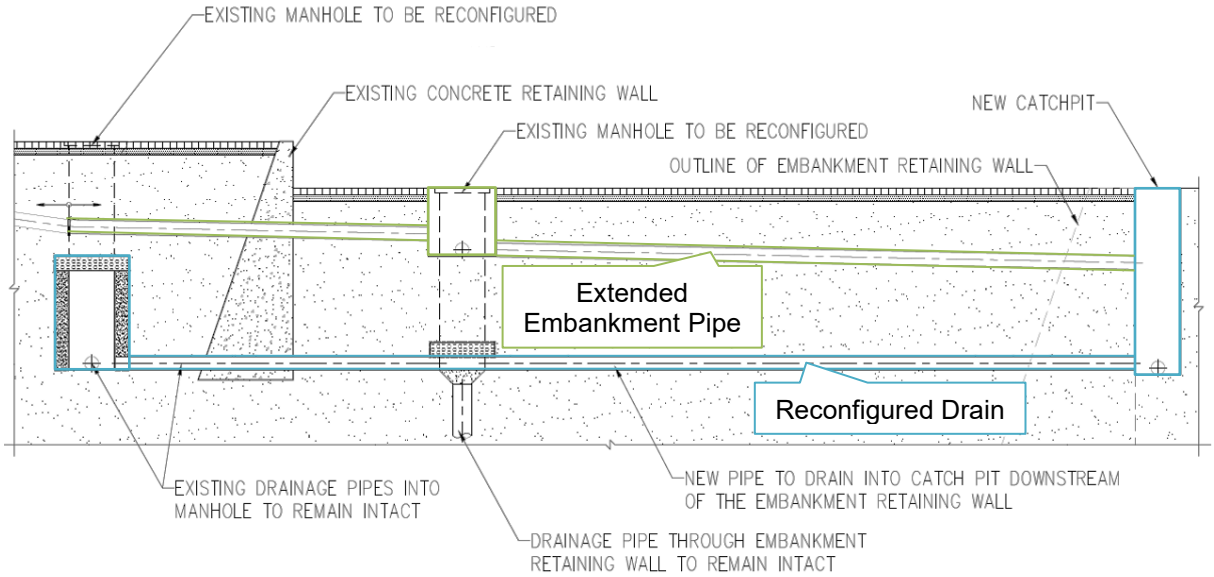

Figure 8. Left Embankment - Drainage Rerouting Section

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4.5 Summary

The site investigations revealed several drainage design features and conditions that were either unclear or inconsistent with the original design drawings. Key findings included:

- The downstream road stormwater discharges directly into the toe drain manholes;
- The configuration of manholes draining through or around the spillway sidewall was determined;
- The invert level of the embankment relief pipe is higher than that of the toe drain pipe, rather than aligned at the same level;
- The presence of 600–1000 mm deep sumps in most manholes, where silt and water accumulate between consecutive structures.

In addition to these observations, the investigations provided essential insights into the actual condition and configuration of the existing toe drainage system at Tzaneen Dam. Surveys confirmed the locations, levels, and connectivity of manholes and pipes, while water flow testing clarified the actual flow directions within the network. Excavations further revealed the underlying material profiles, construction methods, and pipe functions, offering a more accurate understanding of the as-built system than previously available from historical records.

The findings highlighted several shortcomings compared with the assumptions made during the preliminary design stage. These included partial blockages, inconsistencies in invert levels, and variations in pipe depth and alignment. Furthermore, the investigations demonstrated that the drainage system is more complex than shown on the original drawings, with undocumented connections and additional drainage elements present. Consequently, it was determined that the initial proposals, which relied on incomplete records, were inadequate. A revised design was therefore required to ensure the effectiveness of the dam drainage system.

5. REDESIGN OF NEW DRAINAGE SYSTEM

Following the site investigations, a more accurate understanding of the existing drainage network was obtained, allowing for the development of a revised drainage system design. The refurbishment of the toe drain system is required due to uncertainty regarding its current functionality after nearly 50 years of service. Over time, inadequate maintenance likely led to blockages, sediment accumulation, and possible pipe damage, reducing the effectiveness of the original drainage network. Furthermore, the limited number of manholes in the existing system restricted access for inspection and cleaning.

The primary objectives of the alternative design were:

- Removal of compromised, blocked, or redundant components in order to restore full hydraulic capacity and prevent future clogging or malfunctioning of the system.
- Reconfiguration of the drainage system into distinct, separated flow networks to ensure clear separation of stormwater, foundation drains, and blanket drains. Allowing each to operate independently without cross-connections.
- Independent monitoring of drainage elements by isolating the flows from embankment foundation pipes, existing blanket drains, new blanket drains, and stormwater runoff. Each element is monitored individually, also improving operational reliability.
- Ease of access for maintenance was prioritised to enhance the serviceability of manholes and pipes, enabling efficient cleaning, inspection, and repair, and thereby ensuring the long-term performance of the drainage system.

To achieve these design objectives, the revised design was structured into two drainage systems and is discussed in Sections 5.1 and 5.2 below.

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5.1 Embankment Pipe Drainage Extension

The existing perforated pipes within the embankment, which collect foundation drainage, will be extended in a straight line beyond the raised embankment toe into newly constructed manholes. Each manhole will be equipped with gauge plates to enable flow measurement and monitoring. The manholes will be interconnected laterally and will ultimately discharge into the existing lowest downstream manhole, which will drain directly into the river. This arrangement will provide a clearer flow path and will ensure that embankment seepage can be distinguished from other drainage elements. By extending the pipes in the same orientation as originally constructed within the embankment, easy access will be maintained for rodding and cleaning in the event of blockages. The plan layout of the drainage network can be seen in Figure 9 below.

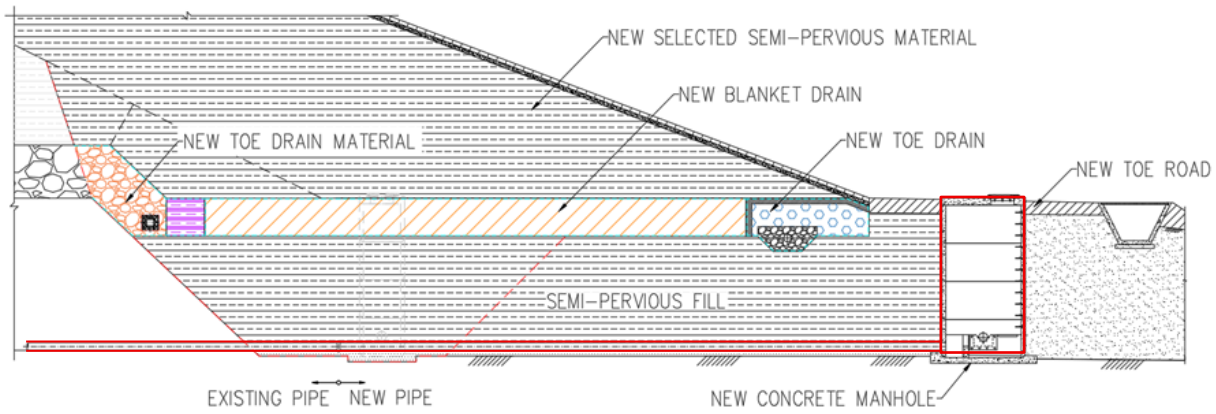

Figure 9. Section - Embankment Drainage Pipes Extension

5.2 Blanket and Toe Drainage System

The second network comprises the new embankment drainage system, configured to follow the standard layout of a blanket drain and rockfill toe drain. The blanket drain will consist of a sand filter layer, 1 m thick, fully encased in NW6 geotextile to prevent clogging and migration of fines. This layer will tie into a rockfill toe, providing additional drainage capacity and structural stability. To ensure no hydraulic interference between the new and old systems, the new blanket drain will be separated from the existing blanket by an impervious material strip, 1 m wide, running continuously along the contact line.

Each manhole along this system is designed to collect water from a 50 m section of the new blanket drain layer. Within each manhole, settling bays are incorporated upstream of the gauge plates to allow for sediment to settle before water flows through the gauge plate. These traps will require regular cleaning as part of routine maintenance. The flows then pass through gauge plates, allowing accurate measurement and monitoring of seepage rates.

The 50 m segmentation of the blanket drain ensures that seepage inflows can be traced to specific portions of the dam embankment. This modular arrangement provides a diagnostic benefit, as any abnormal flows detected at a manhole can be linked to a defined zone of the embankment, making it possible to isolate problem areas for further investigation or remedial action. The existing blanket drainpipes will be reconfigured and segmented in the same 50 m intervals to discharge into these new manholes, ensuring uniformity and improved hydraulic control across the system. From the manholes, the water flow is conveyed through outlet pipes that daylight into the downstream road drainage channels.

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It is important to note that the road drainage channels also carry stormwater runoff from the embankment slopes. This stormwater will not be rerouted into the seepage drainage system. Instead, it will flow directly into the downstream catchpits and ultimately discharge into the river. This clear separation ensures that embankment seepage flows remain distinct from stormwater runoff, thereby improving the reliability of monitoring data. This two-network system (illustrated in Figure 10) therefore achieves full separation between embankment seepage flows and stormwater drainage. The revised design not only addresses the shortcomings of the original construction but also establishes a more robust and monitorable system aligned with modern dam safety requirements.

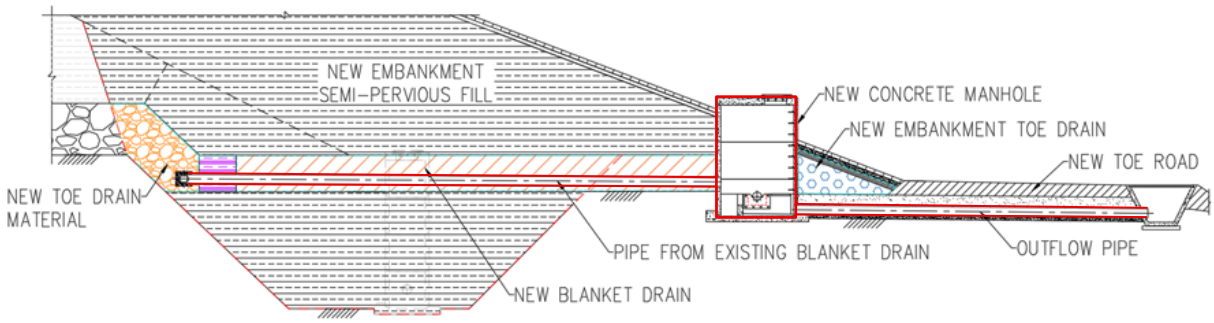

Figure 10. Section - New Toe Drainage System

6. LESSONS LEARNED

This project design component highlighted several key lessons in the context of dam rehabilitation and drainage system redesign:

- The absence of reliable as-built drawings from the original 1970s construction proved to be a significant limitation. Assumptions made from design drawings did not reflect actual conditions, resulting in unexpected challenges during construction. Accurate and detailed as-built documentation is critical for future rehabilitation projects.
- The discovery of inconsistencies, such as pipe blockages, hidden connections, and unrecorded stormwater integration, emphasised the need for comprehensive site investigations before finalising alternative designs. These investigations provided the understanding of the true drainage network and informed the revised design.
- Several site investigations, as described in Section 4, such as the manhole position surveys, visual inspections, and water flow tests, are non-invasive in nature and could have been carried out during the initial design stage in collaboration with the dam maintenance team.
- When potential unknowns are evident, comprehensive planning for these uncertainties should be built into the construction works programme. This includes allowing for additional time to accommodate unforeseen excavation conditions, both of which can demand extra time and resources. Without proper planning, such requirements remain unaccounted for, leading to delays and additional project costs.
- Substantial investigations during the design phase should be undertaken where possible. If not, time during construction must be allowed as explained above.
- The initial drainage design, developed solely from the original design drawings, proved inadequate once actual site conditions were revealed. This highlighted the importance of flexibility in both design and construction planning, enabling engineers to adapt quickly to on-site findings in close coordination with the design office.
- The revised design demonstrates the importance of separating drainage components into distinct networks. This approach allows for more reliable monitoring, better maintenance, and enhanced dam safety.

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6.1 Impact on Construction and Project Delivery

The discoveries made during site investigations had direct implications on construction activities, scheduling, and project costs:

- The construction program required repeated re-planning to accommodate unexpected findings and to integrate the revised design. The complexity of the existing drainage systems introduced additional delays, as each stage required careful verification and adjustment before work could proceed. Flexibility in planning was therefore essential to maintain progress under these evolving conditions.
- The extent of the drainage deficiencies meant that most of the existing system required replacement rather than partial rehabilitation. This marginally increased the projected cost of the works.
- The redesign process depended on close collaboration between the engineering team and the contractor. The contractor's role in excavating, exposing existing systems, and assisting with real-time problem-solving was crucial for the engineer to understand the conditions and adapt the design.

6.2 Associated Risks

The project carried several risks arising from both technical challenges and construction pressures, which had direct implications for dam safety:

- The limited timeframe for redesign placed significant pressure on the engineering team, increasing the risk of errors or incomplete assessments.
- In efforts to save time and reduce costs, simplified or shortcut design alternatives were mooted (but not implemented), creating potential risks to long-term performance, safety, and compliance with dam engineering standards.
- While the original dam drainage infrastructure was consistent with design practice at the time, limitations in the system and subsequent maintenance challenges have resulted in vulnerabilities that were revealed during investigations.
- Inadequate maintenance over decades allowed blockages and deterioration to accumulate, reducing the effectiveness of the drainage system and masking the system's true condition.
- Blockages in the existing drainage system led to saturation in the embankment zones, increasing pore water pressures and creating potential instability risks.
- The combination of blocked drainage, saturated embankment conditions, and outdated infrastructure directly threatened the safety and integrity of the dam, making proper redesign and reconstruction critical.

7. CONCLUSION

This project illustrates the critical importance of accurate records, thorough investigations, and adaptability in both design and construction. While the absence of reliable as-built information introduced significant challenges, the collaborative approach adopted by the project team enabled the successful development of a revised drainage system design. These lessons underscore the need for detailed documentation, intensive site investigations, and flexible design approaches in future upgrade projects.

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The authors thank DWS for their permission to publish this paper. The opinions and views presented in this paper are, however, those of the authors and do not necessarily reflect those of DWS.

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CALCIUM ALUMINATE CONCRETE - A UNIQUE PROTECTIVE MATERIAL AGAINST EROSION & CAVITATION DAMAGES

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ABSTRACT

In hydraulic structures with flowing water at high speed, corrosion due to abrasion, shock or cavitation is a general problem. This water often contains solid particles which increase the abrasion and shock to the exposed surfaces. Typical structures exhibiting these problems are inlets, spillways, weirs, sediment tunnels and stilling pools. To protect these areas different technologies are currently employed. Each with their own challenges, advantages and disadvantages. Typical technologies are standard concrete, abrasion resistant plates, steel plates or organic coatings.

In recent years a new solution which addresses the abrasion, shock and cavitation issues was introduced to the market, although used in projects globally, it is still only known within a small community. This innovation is a new type of concrete based on calcium aluminate cement and calcium aluminate aggregates. This unique combination of materials produces a concrete with significantly improved properties, which provide major benefits for hydraulic structures exposed to high degrees of abrasion.

1. BACKGROUND

The calcium aluminate division of Imerys, formerly Lafarge Aluminates, patented the calcium aluminate cement in 1908 and has studied its use in detail in different applications since that time. In these studies it was found that the abrasion resistance is improved by the use of calcium aluminate cement. One application field for such materials is the protection of hydraulic structures.

2. TYPE OF ATTACK

Hydraulic structures are attacked through a variety of different mechanisms. Beside the standard attacks for concrete like freeze/thaw or carbonation, there are attacks which can only be observed at hydraulic structures.

2.1 Erosion

Erosion is the action of surface processes such as water flow that removes soil, rock, or dissolved material from one location on the Earth's crust and then transports it to another location where it is deposited.

Erosion can take place in small areas like water outlets of an air conditioning due to soft water or at spillways by water with a high load in it.

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Figure 1: Dissolving attack of a concrete by soft water

2.2 Cavitation

Cavitation in fluid mechanics and engineering normally refers to the phenomenon in which the static pressure of a liquid reduces to below the liquid's vapour pressure, leading to the formation of small vapor-filled cavities in the liquid. When subjected to higher pressure, these cavities, called "bubbles" or "voids", collapse in an implosion and can generate shock waves that may cause damages. Cavitation can only be observed with flowing water at high speed.

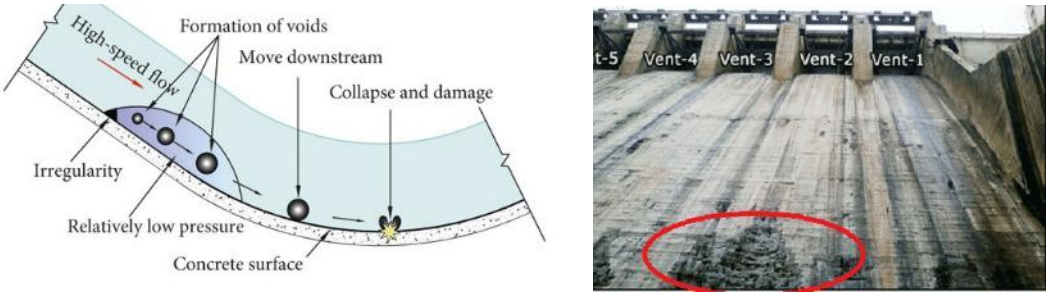

Figure 2: Illustration of the mechanism of cavitation (left); example for cavitation damages at a concrete structure (right)

2.3 Impacts / shocks

Impacts are repeated blows and absorption of energy which can lead to cracking and spalling. Impact scenarios can also be classified into low velocity impact and high velocity impact.

Impacts can be caused by the load inside flowing water or water falling from height.

Figure 3: Example for a load transported by a mountain river

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3. USED MATERIALS

To protect new concrete structures or to rehabilitate damaged ones, there are currently different technologies employed to protect these areas. Each with their own challenges:

- Standard concrete with an abrasion resistance class of M3 (this is based upon the standard for industrial floors) is often specified.
- Special high strength concrete or even ultra-high strength concrete UHPC.
- Abrasion resistant plates (e.g. Basalt) attached to the exposed surface.
- Anchored steel plates.
- Organic resins.

If materials with a higher erosion and cavitation resistance are specified, today, our main building material is OPC based concrete. Most developments are focused on this building material.

Beside OPC, other materials with high abrasion resistance are steel plates, organic resins and granite blocks. All solutions are widely used, especially as repair solutions. Steel shows the highest abrasion and cavitation resistance of all systems, but it is not easy to install. The plates must be fixed by anchors to the substrate, and the joints must be welded. Due to the thickness of the steel plates normally the geometry of the substrate is limited.

Granite blocks must be glued to the substrate and in most cases additional anchors are used to fix the blocks to the substrate. The critical point for granite blocks is the joint filling material as it has normally less erosion and cavitation resistance than granite. This oftentimes leads to the joints being washed out by erosion. In the worst case, as one consequence, further erosion can take place below the blocks, uplift pressure can occur, and full blocks can be torn out. Coatings and mortars based on organic resins are normally applied on dry surfaces which increases the complexity of application. As most resin products are watertight, moisture from the substrate can lead to a failure of such products.

OPC solutions show a wide range of performance depending on their design. High abrasion resistant concrete designed for industrial floors show no good performance in hydraulic applications. The abrasion resistance increases with the increase of compressive strength. To achieve similar abrasion resistance as granite, UHPC must be used which requires a high level of production control.

Approximately 30 years ago a new concrete design was introduced which offers superior properties. This concrete is based on Calcium Aluminate cement and uses Calcium Aluminate aggregates. The concrete can be placed in all geometries, by the use of chess board placing method the joints can be minimised, and its abrasion and cavitation properties are excellent.

4. LAB TESTS

For testing abrasion resistance, there are many standards and test protocols in the industry for different applications (Liu 2024). Every test gives its own results, and the results cannot be directly compared between the test methods.

4.1 ASTM C 1138 - Abrasion test

As a standard ASTM C 1138 (figure 4) is established for underwater abrasion of concrete. It is known that the abrasion resistance for OPC concrete improves with the increase of compressive strength. The addition of fibres, steel or PP, have only limited effect on the abrasion resistance (Table 1). In contrast, the use of the special CAC concrete, based on calcium aluminate binder & aggregates, shows a very high abrasion resistance. There is an improvement of more than 70% when compared with OPC

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concrete of 90-100 MPa strength. As ASTM C 1138 is an abrasion test for concrete, there are no test data on other abrasion resistant materials such as steel or granite.

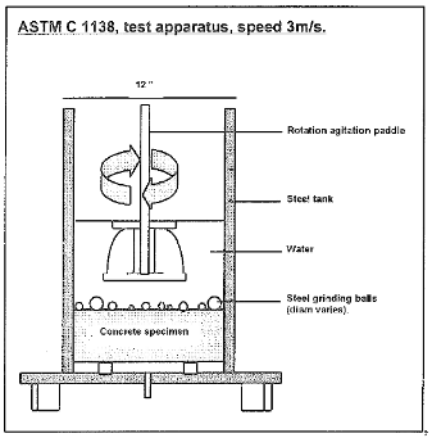

Figure 4: ASTM C 1138 test method: water velocity 3 m/sec; duration: 24h & 72h

Table 1. Mass lost after 72h - ASTM C 1138

Material	28d CS	Mass lost in g	reference
Special CAC concrete	80 MPa	15-40 g	Cabiron 1998
OPC with MS	90 MPa	110 g	Ojha 2021
OPC with MS plus SF	97-100 MPa	102-120 g	Ojha 2021
OPC with MS plus PF	85-90 MPa	108-110 g	Ojha 2021
OPC with MS	100 MPa	373 g	Horszczaruk 2009
OPC with MS plus SF	91-99 MPa	303-381 g	Horszczaruk 2009
OPC with MS plus PF	98 MPa	265 g	Horszczaruk 2009

4.2 C.N.R. Abrasion test

C.N.R., Compagnie nationale du Rhône, developed an internal test (figure 5). A water sand mix is projected on a sample in a 45° angle with a water velocity of 10 m/sec. for a test duration of 75 min. As a reference, glass is used (Dumas 2024) and the abrasion resistance index for glass is set to 1. In this test different materials were tested (Table 2) including OPC concrete, CAC concrete, steel and granite, with their abrasion resistance compared. As expected, steel has an abrasion resistance of 0.04, CAC concrete (0.38-0.75) has even a comparable resistance than granite (0.4-0.8). As seen in ASTM C1138 OPC concrete has a lower abrasion resistance index of 2 than CAC concrete or granite, which was expected.

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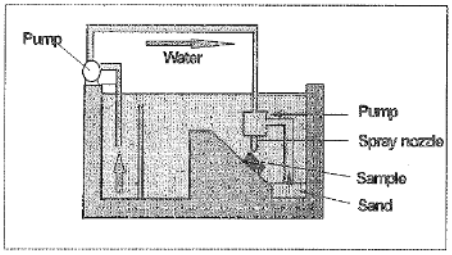

Figure 5: C.N.R. abrasion test method: water velocity 10 m/sec; duration 75 min.

Table 2. Abrasion test - C.N.R.

Material	Abrasion index	reference
Glass	1	Cabiron 1998
CAC concrete	0.38-0.75	Cabiron 1998
Granite	0.4-0.8	Cabiron 1998
OPC with MS	2	Cabiron 1998
Steel	0.04	Cabiron 1998

4.3 CERG - Cavitation test

Beside the abrasion resistance, CAC concrete has an additional benefit as it also shows a high cavitation resistance. This property was tested at GEC Alsthom - CERG (Cabiron 1998). A water jet of 110 m/sec. and 54 bars was blasted at a 90° angle towards a sample for a duration of 2 min (figure 6). Strong cavitation occurs and erodes the sample. In practice, normally the water velocity does not exceed 10 m/sec.

In this test, a high strength concrete was compared to the special CAC concrete. In figure 7 on the left side, the OPC concrete samples are heavily damaged. Single aggregates are torn out of the concrete matrix as the forces of cavitation are higher than the bonding strength of the aggregates in the concrete matrix.

In contrast, the CAC sample shows only very small defects. The sample shows a discoloration due to the high energy impact, but no large aggregates are lost. The reason is that the binder and aggregates are both reactive and form a chemical bonding during hydration which is much stronger than the bonding based on adhesion as is the case in OPC concrete. This chemical bonding is responsible for the high abrasion resistance as well as the cavitation resistance.

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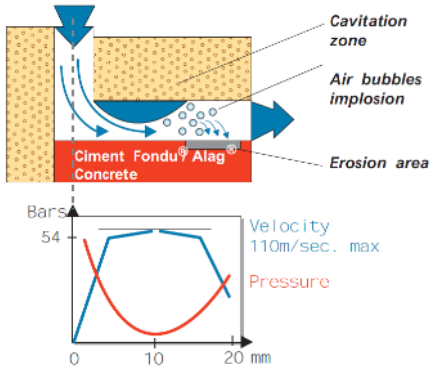

Figure 6. CERG cavitation test method:chamber : water velocity 110 m/sec; pressure 54 bars; duration 2 min.

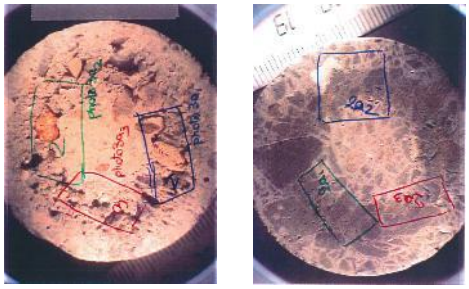

Figure 7. Samples after test in CERG chamber: High Strength OPC concrete (left); CAC concrete (right)

4.4 C.N.R. - Impact test

Due to the chemical bonding of cement matrix and aggregates, the shock or impact resistance, as the third important property, is also increased. C.N.R., Compagnie nationale du Rhône, also developed a test method to evaluate different materials regarding this property (figure 7). A 1m wide drum rotates with 1kg metal balls inside. 1kg metal balls are dropped 2700 times on the specimen and the volume lost is measured.

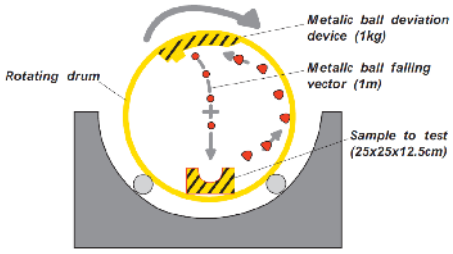

Figure 8: C.N.R. impact test method: 1kg metal ball; 1 m height; 2700 impacts

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Table 3. Impact test - C.N.R.

Material	volume loss	reference
Special CAC concrete	< 120	Cabiron 1998
OPC based concrete	< 300	Cabiron 1998
Granite	< 100	Cabiron 1998

The results show that the special CAC concrete has nearly the same impact resistance as granite. An OPC based concrete shows a lower impact resistance as the bonding forces between cement matrix and aggregates are lower than the one in CAC concrete.

5. CONCLUSION

With special CAC concrete, a little known but widely used material, is available which offers unique properties. It combines the high abrasion & cavitation resistance like granite and the simple application technology of standard concrete. Due to the fact that it is normally mixed on the jobsite, it allows its use in areas with low infrastructure. In practice this type of concrete proved its superior properties versus OPC based concrete.

With 30 years of practical experience, it is proven that the rehabilitation cycles can be prolonged which can contribute to an economic and sustainable benefit.

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7. THE AUTHOR

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COMPREHENSIVE DESIGN REVIEW OF STEENBRAS LOWER DAM

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PRESENTER: DANIEL RAJNA

ABSTRACT

Steenbras Lower Dam is a 28 m high concrete gravity arch dam, originally built in 1921, and raised twice in its lifetime. During the raising of the crest in 1953 the wall was strengthened via a suite of post-tensioned steel cables extending vertically from the crest through the dam to the foundation. The dam has cracking due to concrete swelling and other signs of distress and there is a concern that the anchor cables may be compromised. A project was initiated to conduct a full design review and investigation into the structural integrity of the dam which entailed a variety of investigations including geotechnical drilling, concrete testing, ASR swelling testing, ambient vibration monitoring, exposure of the anchor cable blocks, hydrology and hydraulic modelling. This paper looks at the process of the design review and the lessons learned in relation to drawing the final conclusions.

1. INTRODUCTION

Steenbras Lower Dam is a 28 m high concrete gravity arch dam, originally built in 1921, and raised twice in its lifetime. In 1953, during the second raising of the crest, a parapet wall was added and the main wall was strengthened via a suite of post-tensioned, fully grouted, steel cables extending vertically from the crest through the dam to the foundation. (see Figure 1).

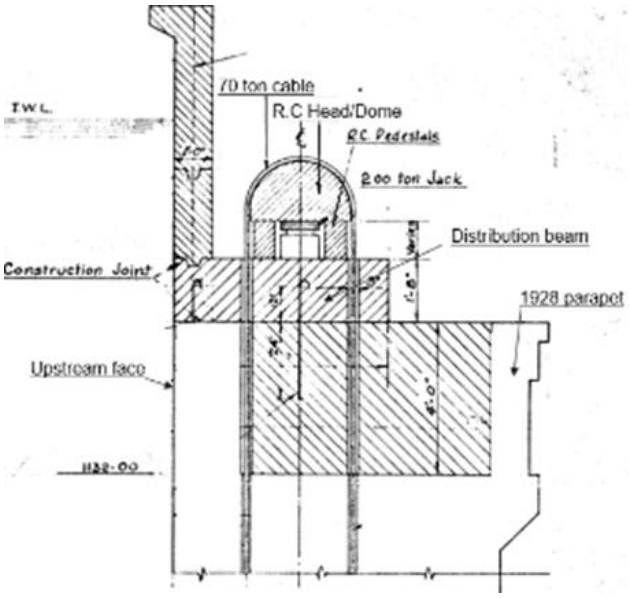

Figure 1. Excerpt from DWG WQ08 1952 - Raising of Steenbras Dam

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Concerns regarding cracking along the crest and potential anchor cable corrosion prompted a detailed investigation into the dam's condition. The investigation aimed to evaluate the integrity of the anchor system, characterize the concrete for Alkali-Silica Reaction (ASR), assess foundation conditions, and analyse the dam's performance under various stability scenarios.

Results from these investigations were incorporated into a Finite Element model which was calibrated and validated using information from ambient vibration testing and geodetic movement monitoring data.

This case study examines these numerous investigations and how they tie together in assessing the structural behaviour and integrity of the dam.

2. INVESTIGATIONS

2.1 Anchor Cable Investigation

Cracking along the crest of the dam related to the onset of concrete swelling, created concern that the cables had been exposed to corrosion and it was decided to open the anchor blocks and assess the cables as well as to take concrete cores from the wall to determine the extent and presence of ASR. Eight anchor blocks were partially demolished to reveal their cables, each tensioned over a dome on top of two pedestals (see Figure 2).

2.1.1 Visual assessment and testing

A thorough visual assessment of the exposed cables found that a significant number of cables (up to 15%), mostly on the domes but some near the base of the pedestals, were partially corroded or even snapped. It is important to understand that whatever happens to the cables above the distribution beam, even if totally snapped, has no bearing on the actual tension in the cables, which are fully grouted in, however the question arises that perhaps some corrosion has extended into the vertical sleeves which would start to influence the overall strength of the cables.

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Figure 2. Cables exposed in anchor block (2023)

2.1.2 X ray scans

X ray scans were done on the exposed parts of the cables and results confirmed that they all had some measure of corrosion both externally and internally, which means that even where there is no observed corrosion there has been internal corrosion. Clearly, the significant macro cracking in the anchor blocks together with cracks in the parapet wall allowed ingress of water to contact and cause corrosion of the cables.

2.1.3 Reactivity tests

Corrosion reactivity tests were also done on the metal components namely the cable runner, cable sleeve and cable itself which show that all three components are similar, consisting of low carbon steels which are not expected to react with each other. This ruled out corrosion between metals and thus the corrosivity in the cables is likely a reaction between the concrete and the metal, electrically coupled by the presence of moisture or water.

2.2 Anchor cable analysis

Although the state of the cables below the distribution beam remains unknown, there is a possibility, though unlikely, that seepage through the dam, particularly at the 1921/1927 interface which currently collects in the gallery, could potentially reach the cable sleeves and cables at a much lower depth in the concrete and cause corrosion.

Based on the worst corrosion seen on the cable heads, using the conservative assumption that this has happened further down into the wall, a percentage loss of 20% strength in the cables was explored during the 2D stability analysis of the dam that followed this investigation.

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2.3 Concrete Investigation

Intact concrete cores were taken from the wall of the original dam 1921, first raising (1927) and the second raising (1954) and subjected to a suite of mechanical tests as well as Alkali Silica Reaction (ASR) testing which concluded the following:

2.3.1 Alkali Silica Reaction testing

Initial observations, later confirmed by petrographic analysis, indicated that the greywacke aggregate in the cantilever wall and steps of the 1954 raising is considerably more susceptible to ASR than the sandstone aggregate of the 1921 and 1927 concrete and the advanced state of ASR in elements containing the 1954 concrete implies that parts of the greywacke concrete structure may no longer be fully structurally competent – in this case, the walkway concrete.

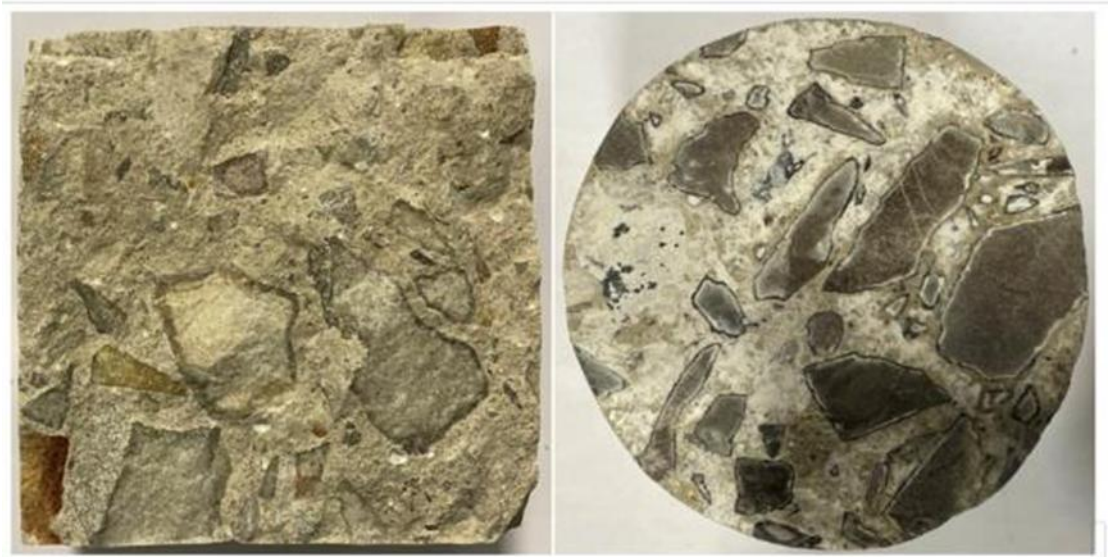

Figure 3. Reaction rims on sandstone aggregates (left). White ASR gel on greywacke concrete (right)

The petrographic analysis was done on concrete core samples taken from the toe, gallery and steps which confirmed evidence of deterioration due to alkali silica reaction. The extent of deterioration ranged from slight damage in the case of the toe concrete (1927), to slightly more extensive in the gallery concrete (1921) to extensive damage with significant expansion and cracking in the case of the step concrete (1954).

2.3.2 Mechanical properties of the concrete

The mechanical strength properties of the intact concrete of the main 1927 sandstone aggregate dam structures were found to be generally adequate for a dam of this nature and age. Consequently, the behaviour of the dam as a structure is expected to be governed more by the major cracking, related often to the slow ASR in the main structure concrete, as well as the prominent vertical and horizontal cracks that have developed more recently in the greywacke aggregate concrete (1954) of the steps and cantilever wall sections over time.

2.3.3 Alkali Silica Reaction swell testing

Overall, the average expansion of the specimens made from greywacke aggregates (cores extracted from the walkway) at 0.090% is substantially higher than for the specimens made from sandstone

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aggregates (0.032% to 0.042%). This confirms the initial observations that the sandstone concrete is expanding more slowly than the greywacke and that these expansions will continue over time.

A wide scatter was observed in the results for the category of specimens made from the greywacke aggregates (1954). This is to be expected in view of the highly variable nature of the ASR phenomenon especially with the reactivity of greywacke, and the variable nature of the damage experienced by the structures.

For use in swell loading for FEA stability analysis the ultimate ASR expansion was modelled as 0.15% for the 1954 greywacke concrete, 0.07% for the 1921 sandstone concrete and between 0.07% and 0.09% for the 1926 sandstone concrete.

2.4 Geotechnical Investigation

A dam foundation geotechnical investigation was undertaken which comprised the following:

- A review of available information.
- Fieldwork consisting of drilling 5 boreholes on the downstream toe and an Electrical Resistivity Tomography (ERT) Survey.
- Laboratory testing.
- Analysis and interpretation of the data.

Results showed the following:

- A relatively thin soil cover blankets the site. The dam is founded on shallow felspathic sandstone of moderate to high strength starting from 5 m below which improves with depth.
- Most of the cables are anchored in the strong rock though a few may be anchored within the moderately strong rock which extends some 11 m below the ground on the right flank.
- The main properties of the rock mass as determined in the investigation and later used in the FEA are shown in Table 1 below.

Table 1. Foundation properties of the rock mass

Material properties	Moderately strong sandstone	Strong sandstone
Tensile strength	0.101 MPa	0.503 MPa
Stiffness (Modulus of deformation)	14.4 GPa	42.3 GPa
Cohesion	2.8 MPa	9.2 MPa
Friction angle	37.2 °	39.3 °

2.5 Monitoring and Sensors

2.5.1 Geodetic prism remote monitoring findings

A variety of sensors have been set up on the dam walkway and parapet wall as part of a demonstration project to showcase the use of remote sensors and continuous data monitoring some years ago. The instruments include fixed prism targets, GPS beacons, temperature, and water level sensors.

An analysis was done on radial movements of the fixed displacement prism sensors located on the crest of the dam to determine whether there has been any significant unexpected movement of the dam

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structure over any period since the start of remote monitoring (2018) and to see what factors are influencing displacement of the structure at various points.

Figure 4. Relative summer displacements along the crest

The dam wall displaces in a manner typical of a gravity arch dam. Displacement readings show a downstream movement with a decrease in temperature (particularly coupled with an increase in water level), or upstream movement with an increase in temperature (particularly coupled with a decrease in water level).

Overall, radial displacement in summer is highest at the arch (18 mm), decreasing along the gravity flanks towards the abutments where there is practically no movement. Near the end of the arch and beginning of the left flank the crest prism (US 6) appears to move very slightly radially opposite to the arch apex which is something that is often seen in arch gravity dams.

The arch and gravity flanks displace as expected responding to ambient temperature and water level. Interestingly, the temperature appears to be the main driving factor of the movement with water level fluctuation playing a lesser role. This could be attributed to the fact that the water fluctuation is kept to 7 m. The displacements do not appear to be excessive.

2.5.2 Ambient vibration monitoring (AVM)

Ambient vibration monitoring (AVM) of the dam wall was performed to provide data on the natural frequencies and modes shapes of the structure which are used to help calibrate the stiffness in the finite element model (FEM). Twenty-two highly sensitive accelerometers were set up along the crest of the dam as shown in Figure 6 to measure the frequency response spectra in a radial upstream-downstream direction. Full scale tests were conducted four times between December 2022 and December 2023 during which period the water level reportedly did not fluctuate significantly.

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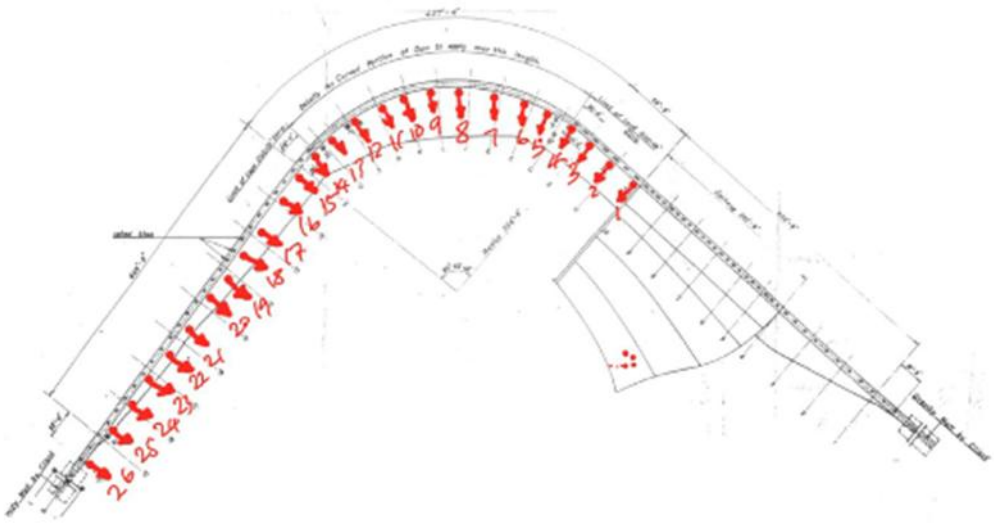

Figure 5. Location of the accelerometers along the crest for AV measurements

The main findings were:

- Nine modes of vibration were positively identified by the accelerometers acting in the radial direction, six of which are shown in Figure 6 below.
- From the mode shapes, the left flank of Steenbras Lower Dam is stiffer than the right flank.
- The natural frequencies and mode shapes were used to calibrate the FE model of the dam.

Frequency	Date	Shape type	Mode shape
10.00Hz	Dec 2022	Anti symmetrical	
10.61	Dec 2022	Symmetrical	
12.10Hz	Dec 2022	Anti symmetrical	
12.5Hz	Dec 2023	Symmetrical	
13.33Hz	Dec 2023	Symmetrical	
14.28Hz	Dec 2023	Symmetrical	

Figure 6. Some mode shapes in the radial direction that were identified

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2.6 Stability Analysis – 2d

A stability analysis was done on a spreadsheet, using information from drawings and from the determinations of the concrete investigations. The two more critical sections of the dam were analysed, namely the tallest gravity section and the spillway section. This type of analysis is more limited than the finite element analysis however it is extremely useful in picking up major design flaws.

Various critical load scenarios including selected flood events, uplift pressures and earthquake events combined with various cable strength losses were analysed for stability ahead of the Finite element analysis.

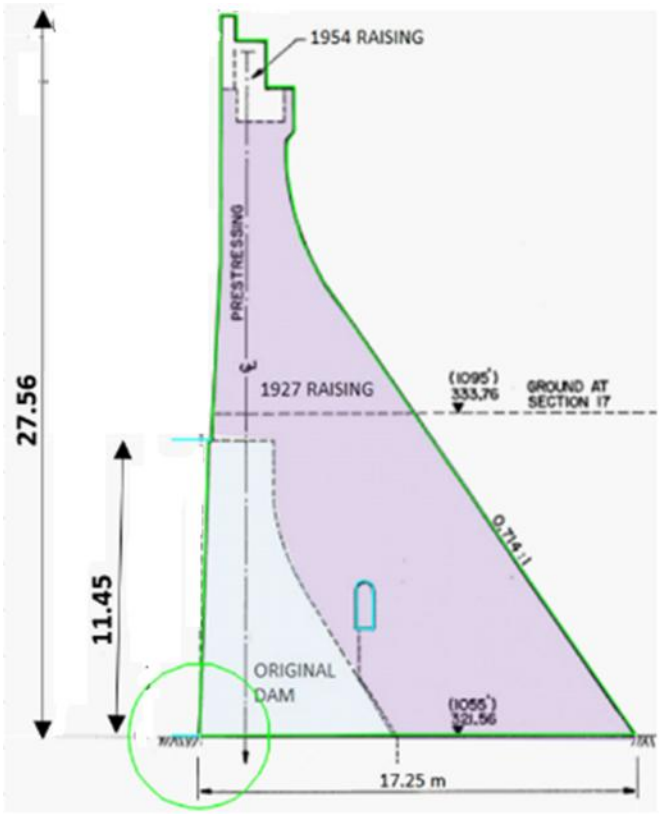

Figure 7. Section showing full dam structure

2.6.1 Full dam structure at highest straight gravity section.

The wall was found to be stable under all conditions assuming there is no strength loss in the cables. With 20% loss in strength, the dam generally complies, but for one extreme earthquake (Maximum credible earthquake) loading case. This would not pose imminent danger however should the cables have a 40% loss in strength there is cause for concern with respect to sliding. The OBE earthquake with associated return period of ±145 years was also used in the load scenarios however the wall remained stable under these loadings.

2.6.2 Spillway structure – straight gravity section.

This section was found to be stable and complies in all cases up to a 20% loss in cable strength.

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2.6.3 Part of dam structure above 1921-1927 horizontal contact at highest gravity section.

This part of the wall was found to be stable under all conditions assuming there is no strength loss in the cables. With up to 20% loss in strength this part of the wall is still generally stable and complies comfortably in all cases except in the RDF case though the FoS values are still reasonable.

2.6.4 1927 & 1954 structure only at highest gravity section.

The contact of this section between the two different aged concretes is drained to capture any seepage. For analysis purposes the section was assumed conservatively to be cracked throughout. Assuming a 20% cable strength loss results in sliding FoS values that do not comply however there is no failure i.e FoS < 1. The overturning FoS values comply no matter what the strength of the cables.

2.7 Finite Element Analysis (FEA)

A 3D finite element model was set up for critical flood, temperature and ASR loading, using the following inputs listed below.

- Surveyed dimensions and drawings.
- Temperature fluctuation model.
- Rock mass stiffness from the geotechnical investigation.
- Concrete stiffness from the concrete core investigation.
- Concrete swell estimation from the ASR swell testing.

The foundation and wall stiffnesses for the model were calibrated using the determined Ambient Vibration Modelling frequency modes. The temperature model was calibrated from recorded temperatures and recorded seasonal radial displacement of the crest wall since 2018.

The model was used to determine the behaviour of the dam in terms of stresses and displacements under certain critical loading conditions and to assess the integrity of the post-tensioned cables, but the overall stability was not assessed. Seismic loading was not considered as part of the model.

3. ANALYSIS AND CONCLUSIONS

Field measurements including borehole drilling and testing, Electrical Resistivity Tomography (ERT) Surveying. and ambient vibration testing confirm the general variation of foundation stiffness, i.e., the left flank is slightly stiffer than the right flank.

Laboratory tests for ASR confirm that ASR is pervasive throughout the dam elements. However, it is very slow acting in sandstone aggregates 1921 and 1927 concrete and does not resemble the destructive ASR seen in the concrete steps and parapet wall that contain the greywacke aggregates (1954 concrete).

The Table Mountain sandstone used as the coarse aggregate in the original and first raising of this dam is minimally reactive but nevertheless does result in slow swelling of the concrete that can in the future result in more surface cracking. That said, swell test results indicate that 84% of the swelling has already occurred thus further swelling is expected to be minor.

A 2D stability analysis shows that under all required critical load scenarios the dam is stable if the cables are working properly and generally complies with stability requirements up to a 20% loss in cable strength. At 40 % loss in strength there would be cause for concern particularly at the main wall section for the unusual and extreme flood /earthquake events.

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Cracking on the downstream face of the dam wall (1928) and on the 1954 section of the wall has been caused by swelling due to the combination of ASR and thermal stresses. However, since most of the swelling due to ASR has occurred, further cracking will be limited.

The finite element model showed that the foundation stiffness is independent of the cable strengths and thus the stiffness of the foundation and wall could be calibrated using the AVM frequencies, assuming the cables were intact.

Based on the critical loads considered (flood, temperature and ASR), the FE model shows that the prestressing cables are in good working order. The stress induced in these cables is well within the design limits, thus the main dam is considered stable for the critical load scenarios as calculated in the 2D analysis.

The finite element model did however confirm high stresses in the curved part of the 1.2 m high 1954 parapet wall which has been structurally compromised to some degree and may need attention in future.

The FE model calibration process showed that the foundation stiffness to wall stiffness ratio (Ef/Ec) is approximately 0.9, making the foundation relatively stiff. This value, being close to 1 is favourable, resulting in better seismic safety for the dam. The 2D analysis does not take this into account explicitly but does use a pseudo-static calculation (Westergaard,1933) for estimation of earthquake loadings, bearing in mind that it is extremely conservative.

4. LESSONS LEARNED

The condition assessment and investigations of the dam that culminated in the finite element model were designed to create the best model possible for use in assessing the structural integrity and any change in the dam going forward. The model does not assess the overall stability but produces useful stresses and displacements that can be used, with careful monitoring going forward to predict future behaviours.

Exposing the cables was revealing, however it did not give meaningful information on what is going on within the grouted in cables and couldn't be incorporated into the finite element model. To determine the state of corrosion of the full length of the cables, destructive testing of selected cables would likely be necessary, although this is a very expensive solution.

The geodetic radial movement of the sensors on the crest was reasonably useful for calibrating seasonal radial displacements however it is not reliable for long term displacement. The readings only describe the very top of the structure and geodetic readings at selected points on the downstream toe should be implemented.

Online continual displacement data is very cumbersome to extrapolate and the fact that the parapet walls have visibly moved upstream over the years is not detectable. For absolute movement over the years, it is recommended that the prisms or fixed beacons are read manually relative to a fixed beacon at least quarterly and logged.

Field measurements on the sections of the dam would be advantageous to check actual swelling rates reliably for input into the finite element model.

Notwithstanding the limitations of not definitively knowing the state of the cables, this suite of studies provides an excellent benchmark for the long-term safety management of the Dam.

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5. ACKNOWLEDGEMENTS

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HAZELMERE DAM RAISING – UNIQUE DESIGN SOLUTIONS TO THE RAISING AND SAFETY OF THE DAM

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PRESENTER: DUANE BOOYSE

ABSTRACT

Launched in 2012 and commissioned in 2024 at a construction cost of R820 million, this strategic project is a marvel of world class engineering. The main focus of the project was to raise the dam spillway from 86 m.a.s.l. to 93 m.a.s.l., increasing its storage capacity from some 17 million m³ to 37.1 million m³, ensuring water supply to KwaZulu-Natal's Dolphin Coast.

The Category 3 dam, designed (and constructed) to strictly comply with Dam Safety Regulations, now features a number of world firsts, namely: the world's tallest constructed Piano Key Weir (PKW), significantly increasing its spillway discharge capacity; the highest capacity ground anchors installed through a dam wall (91-stranded, 279 kN/strand), globally; the longest ground anchors installed in a dam wall, globally (some 90 metres). Structural reinforcements included with deep rock anchors also incorporated the need for bespoke stressing jacks to be manufactured and utilized in the stringent anchor testing and stressing process (in addition to the destructive test anchor phase of the project). Lastly, real-time data management software and 3D digital sensors, ensure continuous remote monitoring of the pressure transducer hydraulic load cells, installed at a selection of ground anchors) and digital 3D crack meters, for structural integrity.

1. APPLICATION OF PKW TECHNOLOGY AT HAZELMERE DAM

PKW technology has been studied in detail and the theory has been well developed and documented with dimensional ratios of the complex configuration optimised and defined. At Hazelmere Dam we retrofitted the PKW concept to an existing structure as the physical constraints of the existing dam spillway prevented the designers from rigidly adhering to the prescribed dimensions. We were able to optimise the application of the preferred PKW design proportions by testing initial theoretically selected parameters in a hydraulic model study. (Ingerop South Africa et. al. 2014).

1.1 Basic relevant characteristics of Hazelmere Dam

Hazelmere Dam was designed and built between 1975 to 1976 with provision for the future raising of the water storage level. The intention was for the raising of the dam to be accomplished by the installation of radial gates for which provision was made in the existing structural concrete arrangement. (Botha et. al. 2013 & Ingerop South Africa et. al. 2014).

The crest level of the original uncontrolled spillway (FSL) was at 86 m.a.s.l. and the planned 7 m raising of the spillway would increase the FSL to 93 m.a.s.l. The dam is slightly curved in plan with a radius of 725 m and comprises 32 concrete monoliths each approximately 15 m wide and ranging from 15 m to 45 m in height. The ogee spillway was located on 7 of these blocks above the river channel and was 103 m long. An aerial view of Hazelmere Dam, pre-construction, is shown in Figure 1. (Botha et. al. 2013).

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Figure 1. Hazelmere Dam prior to the Dam Raising

Existing vertical construction joints between the blocks were sealed with rubber waterstops and during construction the contractor had to experiment with the joining of new waterstops with the old. (Botha et. al. 2013).

Increased water demand in certain areas of Durban North, exclusively supplied with water from Hazelmere Dam, dictated that the FSL of the dam be raised by the planned 7 m in order to improve the yield, so that demand is met by a supply that is within acceptable safety margins. (Botha et. al. 2013).

Although provision was made in the original design for a future raising of the FSL by means of radial gates, this was considered by the current owners and operators to be inappropriately expensive to install, operate and maintain. As a result the designers were required to investigate alternatives. Apart from the normal financial restrictions for this kind of operation, the two major considerations in determining the eventual preferred option were, firstly, the extremely short time of concentration (ToC), of 6 to 12 hours, for the dam catchment area and, secondly, the limited height difference of 1 m between the proposed new FSL and the NOC of the dam wall. (Botha et. al. 2013).

The ToC restriction demanded that an uncontrolled spillway be preferred but the physical constraint of the limited freeboard between FSL and NOC militated towards mechanically operated gates in order to cope with the sheer volume of the safety evaluation flood (SEF). (Botha et. al. 2013).

In the search for alternatives to mechanical gates the designers examined the possibility of using labyrinth fusegates but the same freeboard constraints meant that the likelihood of the regular toppling and loss of fusegates was very high. This was unacceptable as it would result in the regular compromise of storage capacity and increase in operation cost as well. (Botha et. al. 2013).

This dilemma was resolved by the introduction of the PKW, as one of the main features of the PKW is to minimize the head required to pass flood waters through a given spillway. The ingenious design of the PKW offered a perfect solution to the imperatives faced in managing flood waters through the Hazelmere Dam spillway. (Botha et. al. 2013).

Four basic parameters were then assumed to define the design criteria for the Hazelmere Dam PKW structure (Botha et. al. 2013 & Ingerop South Africa et. al. 2014):

- The 1:200 year flood must pass through the spillway section without overtopping the existing NOC section of the dam.
- Floods with a return period as large as possible, preferably the regional maximum flood (RMF), should be passed over the PKW structure without overtopping the existing 1.5 m parapet wall on the NOC section.

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- The SEF must be safely routed over the spillway with limited overtopping of the NOC and the parapets on the NOC.
- Under the above circumstances, the dam wall stability criteria must not be compromised (accounted for by ground anchors for improved stability, reinstated grout curtain to account for uplift pressure, as well as toe protection in the event of limited over-topping).

The PKW was positioned on the upstream side of the ogee to maximize contribution to the stability of the wall and to ensure that most of the discharged water impacted the ogee spillway where the existing Roberts splitters are positioned. It has been visually assessed in the hydraulic model study that this will facilitate adequate energy dissipation. Figure 2 shows the final constructed PKW on Hazelmere Dam with a PKW positioned on the wall, during a flood event in 2022.

Figure 2. Operational Raised Hazelmere Dam with a PKW

1.2 Theoretical considerations

It is normally more advantageous to make use of a larger flow depth to optimize the design of a spillway because flow through a spillway is defined by flow depth to the power of 3/2. This is especially so when the spillway length is limited as is the case at Hazelmere Dam. (Botha et. al. 2013).

When the spillway length is limited and the maximum allowable flow depth is likewise restricted, the only alternative is to increase the effective length of the crest. A PKW is particularly effective in this regard as it is possible to increase the overflow crest length by up to 7 times the effective length of the spillway. Hazelmere Dam is an existing gravity concrete dam with both a fixed spillway width and restrictions regarding the temporary storage depth of water above the fixed FSL. (Botha et. al. 2013).

Using the available theoretical equations published in various papers by researchers from Algeria, France and the USA who sought to optimize the hydraulic efficiency and structural stability of the PKW, a configuration for the use of a PKW at Hazelmere Dam could be defined. (Botha et. al. 2013).

The main relevant characteristics of the proposed PKW are as set out below and shown in Figure 3 and 4 (Botha et. al. 2013):

- The existing spillway length was 103 m consisting of 6 blocks 15 m wide, and a spillway centre block 13 m wide, separated by vertical construction joints.
- The raising of the FSL above the existing ogee crest is 7 m.
- The 1:200 year flood to be passed without overtopping the NOC is 1660 m³/s.
- P was determined as 9 m, where P is the height of the PKW.
- Pm was determined as 6 m, where Pm is the Maximum height of the PKW walls (Point of intersection).

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- Wc should have been 10.8 m, where Wc is one cycle width but was fixed as 6.5 m for the centre block and 7.5 m for the remaining 6 blocks in order to accommodate the fixed block widths and their associated vertical construction joints.
- Wi/Wo was determined as 1.315 where Wi and Wo are the internal widths of the inlet and outlet key respectively.
- PKW top length B is 21.6 m.
- L/W was determined as approximately 6.5 where W is the width of the spillway and L the effective length of the top of the PKW walls on their centre lines.
- The wall thickness at the top of the wall is 0.5 m.

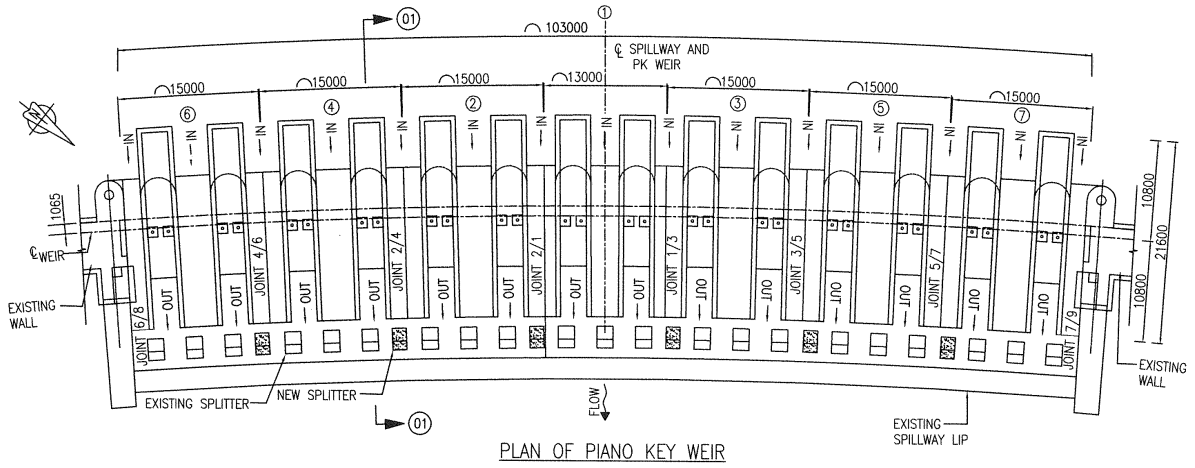

Figure 3. Sketch plan of Hazelmere Dam structure

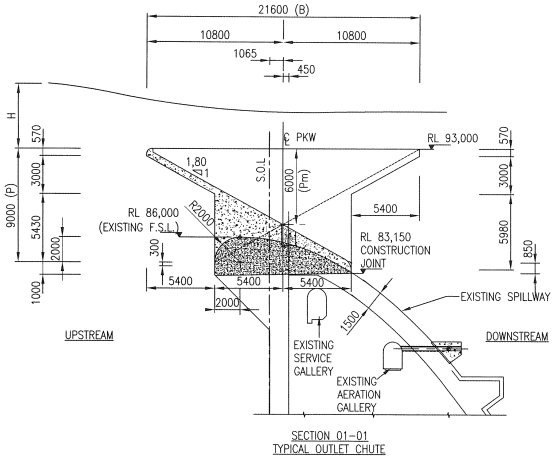

Figure 4. Sketch section of Hazelmere Dam structure

A hydraulic model of the proposed PKW structure was then constructed in the water research laboratory of the DWS in Pretoria and used to test several configurations to determine the optimum unit flow through the weir. (Botha et. al. 2013 & Ingerop South Africa et. al. 2014).

These models were for Pm-values of 4.8 m to 7 m. The 7 m high walls were associated with a B-value of 25.2 m and a Wc-value of 12.6 m, while the 4.8 m high walls were for a B-value of 17.28 m and a Wc-value of 8.64 m. The linear scales used varied between 1:21 and 1:44.

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The data is summarized as follows:

The discharge coefficient is defined by the equation below (Lempérière 2009):

$q=CH\sqrt{Pm}$

Where:

- q = Discharge per meter of spillway (m^3/s)
- C = PKW discharge coefficient
- H = Upstream hydraulic head – head water level (m)
- Pm = Maximum height of PKWeir walls (Point of intersection) (m)

The C-values reported are determined using the head (H) for q = 40 $m^3/s/m$ for all layouts. The H-values varied between 3.7 m and 5.2 m. The C-values were in the range of 3.5 to 4.4. The best fit equation indicates a C-value of 4.0 for L/W-ratio of 5 and that C varies according to L/W to the power 0.25. (Botha et. al. 2013 & Ingerop South Africa et. al. 2014).

The single data point residing below the minus 5% curve is for a layout of equal incoming and outgoing channels. To minimize the required head, the incoming channel should be approximately 55% of the cycle width, which means that the outgoing channel is only 80% as wide as the incoming channel. (Botha et. al. 2013).

Other features like rounding of crest and inlet-walls seem useful to reduce the required head, but remain within the ±5% range. (Botha et. al. 2013 & Ingerop South Africa et. al. 2014).

After the initial results were obtained, the hydraulic model was adjusted several times to optimize the required structural layout. The model also contained a study of the energy dissipation downstream and on the existing ogee to determine minimum modification (breaking out of concrete etc.) of the structure. Finally a configuration with a 7.5 m cycle width for the 15 m wide blocks and a 6.5 m cycle width for the 13 m centre block, was selected and arranged over the full width of the existing spillway. This configuration was selected to optimize flood passage whilst ensuring that existing construction joint positions were accommodated in the new PKW. The result as determined in the laboratory modelling for the final configuration is shown below with unit discharge of the overall project flow against the actual project hydraulic head (Figure 5). (Botha et. al. 2013).

The discharge rating curve was then developed to determine the flow depth (water level in the dam) discharge relationship. This was very important to determine the flow depth for extreme flood flow conditions as the raising of the existing structure will be done by ensuring stability of the structure against overturning and sliding by installation of rock anchor cables. The discharge rating curve is shown below (Figure 6). (Botha et. al. 2013).

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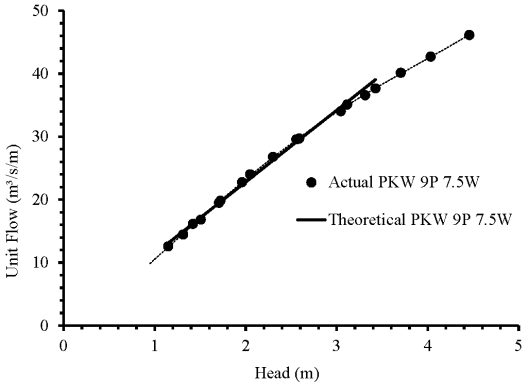

Figure 5. 7.5 m cycle width all data

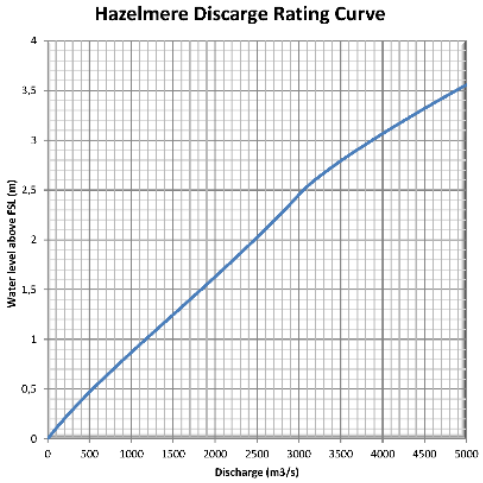

Figure 6. Discharge rating curve

1.3 Structural requirements

After the flow rating curve had been determined, the structural requirements to optimally position the new PKW on top of the existing ogee crest was necessary. As far as we know the PKW height, P, of 10m is presently the highest constructed in the world. (Botha et. al. 2013).

The following aspects had to be considered for the structural placing of the PKW on the existing ogee concrete structure (Botha et. al. 2013):

- Positioning of the PKW on top of the existing ogee to ensure adequate energy dissipation during flow.
- Configuration and placement in an upstream-downstream direction to optimize the effect it would have on the stability of the structure.
- Proper anchorage of the PKW to the existing ogee.
- Only 1 m freeboard is available between the new FSL and NOC to pass the 1:100 year flood.
- Proper placing of construction joints to coincide with the existing construction joints on the ogee.

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1.4 Final design

The final design, after optimization, has the following characteristics which are depicted in the above sketches (Figures 3 & 4) (Botha et. al. 2013 & Ingerop South Africa et. al. 2014)):

- FSL is 93 meters above sea level (m.a.s.l.).
- NOC concrete level is 94 m.a.s.l.
- NOC with existing parapet left in place is 95.5 m.a.s.l.
- FSL top level length of each PKW is 21.6 m.
- 12 full standard cycle 7.5 m wide PKWs and 2 full cycles of 6.5 m.
- Total effective spillway length of PKW is approximately 675 m.
- Unit discharge capacity at 95.5 m water level is 29 $m^3/s/m$ length.
- Total discharge capacity with water level at 95.5 m is 3000 m^3/s.
- The safety evaluation flood (SEF) is 4300 m^3/s.
- Maximum water level at SEF is 96.23 m which determined the design water level for all structural considerations.
- At SEF, the PKW is almost completely drowned, making the effective spillway length approximately 103m.

2. STABILITY OF STRUCTURES

2.1 Stability Analysis

Analysis revealed that the normal loading conditions for the new FSL (93.00 m.a.s.l.) of the dam in its raised condition were very close to the extreme conditions in its current status and that the dam wall structure could not be considered safely stable under these circumstances. The reason for the apparent new conclusions could be attributed to the uncertainties in regard to the underlying geological conditions, which seemed to have deteriorated, coupled to the modern practice of designing Category 3 Dams to SEF conditions equivalent to the Probable Maximum Flood (PMF) and not the Regional Maximum Flood (RMF). Under Dam Safety inspection analyses, the existing status was assessed against the original design criteria but in the process of raising a dam like Hazelmere Dam, design for the new condition had to comply with the latest dam design practices. This meant that the wall structure was in need of improved stability, even in the current status for extreme conditions, and that it very definitely required even further stabilizing measures for the future raised condition. (Booyse 2015 & Ingerop South Africa et. al. 2015).

A further area of concern regarding the stability of the structure was that artesian water conditions were detected when new cored boreholes were drilled in the drainage gallery, especially in the area of the existing spillway structure as well as the apron area downstream of the toe of the spillway. This was unexpected as the existing drainage/pressure relief holes showed no evidence of significant water flow in the foundation below the dam or build-up of water pressure. During a preliminary investigation the water pressure measured, however, was in the order of 10 m which corresponds to about 1/3 of the water depth – confirming a load case as normally predicted by Westergaard. (French Committee on Large Dams 2002) Three fundamental factors were considered in determining acceptable levels of stability (Booyse 2015 & Ingerop South Africa et. al. 2015):

- Sliding
- Overturning
- Bearing Pressure (stress) at the toe and heel of the dam wall

2.2 Post-tensioned anchors

From the stability analysis which considered findings in the Geotechnical Report and the strength of the concrete in the dam wall, as well as the theoretical values for the sub-surface conditions, it was

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determined that post-tensioned anchors installed through the dam wall could safely be designed to improve the dam wall stability. Due to the fact that the dam wall is made up of separate concrete blocks connected via shear keys, each block was analysed separately in order to determine how many post-tensioned anchors would be required in each block to render the factors of safety and heel pressures acceptable according to modern dam safety practice. (Booyse 2015) A total number of 83 rock anchors were utilised across the dam wall. The largest of these anchors were 91-stranded, 279kN/strand anchors with a free length of 86m and a fixed length of 9m (anchored some 40m into the bedrock underlying the concrete dam wall structure. Considering the theoretical unlimited lifespan of a Category 3 Gravity Dam, the sustainability of the post-tensioned anchors is vitally important. It was determined that the lifespan of a correctly designed, installed and protected post-tensioned anchor could be in the vicinity of 120 years, which was seen to satisfy the sustainability criteria according to the Approved Professional Person (APP) responsible for the safety of Hazelmere Dam. (Booyse 2015) The post-tensioned anchors were thus to be designed as double protected permanent anchors and installed by an experienced, expert Contractor in strict adherence to BS8081:1989. (BS8081:1989 1989) It was also viewed as imperative that a comprehensive maintenance and monitoring procedure be drawn up and carried out in order to ensure the sustainability of the post-tensioned anchors and longevity of the Hazelmere Dam structure. (Booyse 2015 & Ingerop South Africa et. al. 2015).

3. REINSTATEMENT OF GROUT CURTAIN

The scope of the grout curtain reinstatement included the following:

- Drilling and water pressure testing of rotary cored exploration boreholes at the dam to about 60m below gallery floor level. These were required to explore the geological conditions down to the invert levels of the anchor holes.
- Drilling and grouting from the gallery, the spillway section and most of the left flank and lower right flank.
- Drilling and grouting from the dam crest at the upper left and right flanks in areas where the gallery is not present, or where steep gallery floor slopes prevented gallery drilling.
- Reinstatement of the vertical drainage curtain in the gallery only.

The reinstatement of the grout curtain was carried out over an extended duration due to the extensive number of splitting holes required on the lower left flank and in the spillway section. The grout curtain was reinstated along the entire length of the left flank although it was initially expected that parts of the upper left flank would not require any grouting. For the reinstatement of the grout curtain, 9 229m of drilling (including redrilling of quaternary holes) was done and 132 134 litres of grout was used. The total cement consumption required for the grout curtain reinstatement was 132 134 kg, which relates to an average grout take of 7,9kg/m2, which is significantly less than the average grout take of 19,5kg/m2 during the initial grout curtain construction.

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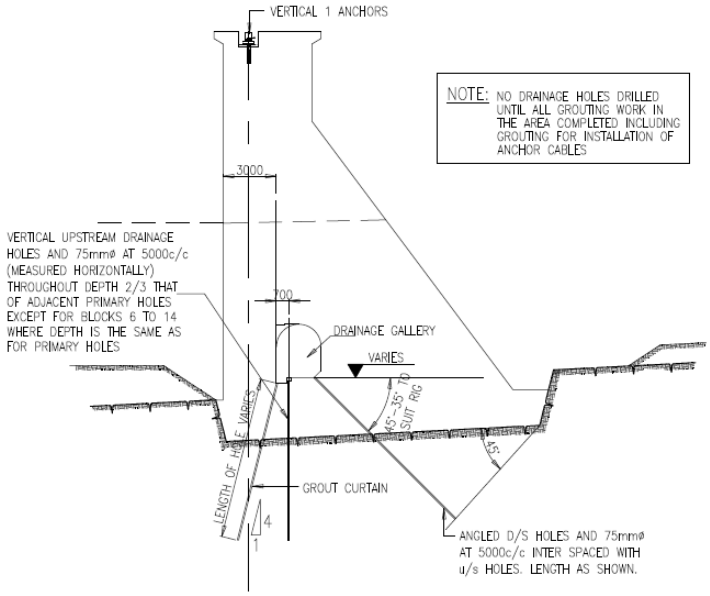

Figure 7. Typical Section for Curtain Grouting and Drainage Holes

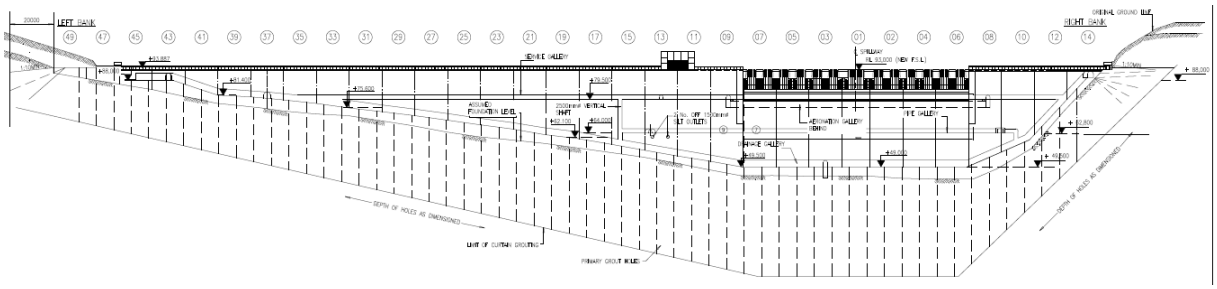

Figure 8. Grouting Arrangement

4. CONSTRUCTION

The planned construction process, which included the improvement of dam stability by installation of multiple strand rock anchors, which will be routed through the base of the PKW, was started in July 2015 and the dam commissioned in December 2024. The PKW constituted about 7000 m³ of concrete and comprised about 40% of the construction costs, which was predominantly split between the PKW and rock anchors. Water levels in the dam had to be carefully controlled to maintain water supply to the WTW and accommodate construction activities despite the limited outlet capacity of 42 m3/s, which is very small if compared to the possible floods.

4.1 Phased impoundment

Often the increased impoundment of water is of a pressing nature and it is an advantage to enable the client to receive the benefit of early impoundment.

The PKW let itself well to this urgency and at Hazelmere Dam we were able to offer early impoundment that allowed 13 000 000 m^3 to be stored a full season ahead of scheduled impoundment.

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Early impoundment was accomplished by arranging the construction sequence in such a way as to close the spillway by completing all base structures up to Stage 6 in the "Construction Sequence" below.

4.2 Construction sequence

Stage 1 of the construction sequence, consisted of the demolition of the piers, deck, lintel beam and 3 m of the ogee spillway (Figure 9).

Figure 9. Stage 1

Preparing the platform for the construction of the PKW took place in stage 2 (Figure 10).

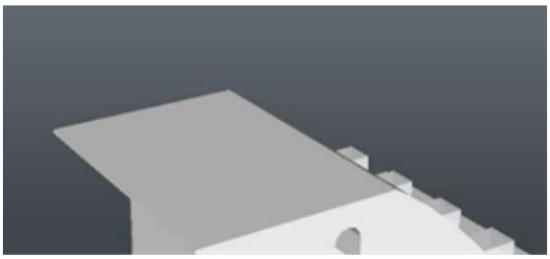

Figure 10. Stage 2

Stage 3 was the first lift of the PKW (Figure 11).

Figure 11. Stage 3

In stage 4 the support work for the installation of the anchors was constructed, followed by the installation thereof (Figure 12).

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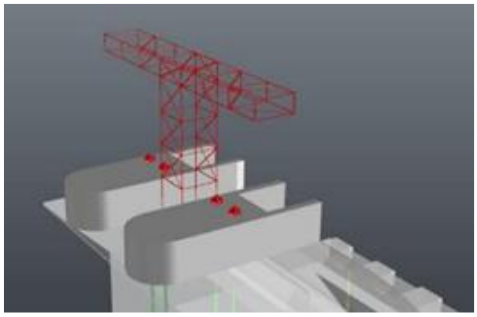

Figure 12. Stage 4

The second lift of the PKW was constructed in stage 5 (Figure 13).

Figure 13. Stage 5

The PKW was constructed, in stage 6, to the level to which the first phase of water impoundment could take place (Figure 14).

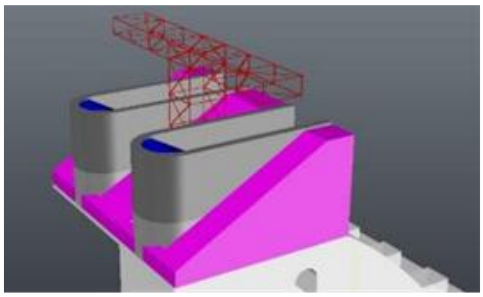

Figure 14. Stage 6

The construction of the PKW walls took place in stage 7 (Figure 15).

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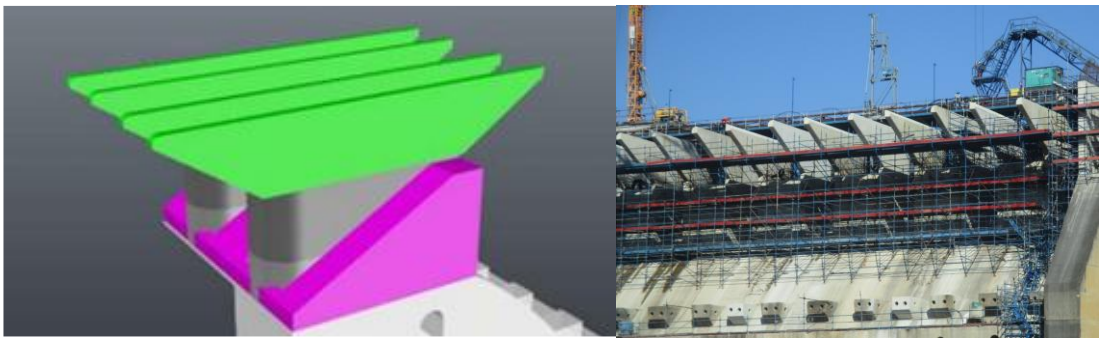

Figure 15. Stage 7

In stage 8 the final construction of the inlet and outlet key floor slabs were done (Figure 16).

Figure 16. Stage 8

4.3 Post Tensioned Ground Anchors

In order to verify theoretical calculations, destructive anchor tests were carried out at various locations at the toe of the dam. This assisted in determining the behaviour of the underlying bedrock against the high anchor loads, as well as determining behaviour characteristics of the ground anchor components.

Figure 17. Stressing of the test anchors

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Once verified, the process of installing the ground anchors commenced, in strict adherence to the BS8081:1989 design standards. Given the aggressive environmental conditions of the Kwazulu-Natal Coastal area and the required lifespan of the ground anchors, it was imperative to ensure sufficient corrosion protection to the ground anchors. For this reason, the anchor installation process followed a stringent quality management process to ensure that the anchors were double corrosion protected. This corrosion protection process included:

- Waterproofing of the 355mm diameter diamond-head drilled anchor boreholes.
- Waterproofing of the HDPE anchor ducting.
- Zero tolerance for compromise on the HDPE anchor strand sheathing.
- Introduction of corrosion inhibiting compound at locations where the anchor's steel components were not protected by HDPE ducting or sheathing (e.g. at the anchor head).

The final installation process of the post-tensioned ground anchors included the following:

- Drilling and waterproofing of the anchor boreholes.

Figure 18. Anchor Borehole Drilling

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- Installation and waterproof testing of the corrugated fixed length and smooth free length HDPE ducting.

Figure 19. Ducting Installation

- Assembly of the ground anchors.

Figure 20. Anchor Assembly

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Figure 21. Anchor Assembly

- Transport and installation of the ground anchors.

Figure 22. Anchor Transportation

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Figure 23. Anchor Transportation

Figure 24. Anchor Installation

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Figure 25. Anchor Installation

Figure 26. Anchor Installation

Figure 27. Anchor Installation

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- Stage grouting of the ground anchors fixed and free lengths.

Figure 28. Stage Grouting Inner and Outer Ducting

- Installation of the anchor heads and load monitoring equipment.

Figure 29. Anchor Head Assembly

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- Stressing, testing and locking off of the ground anchors.

Figure 30. Anchor Stressing and Testing

Figure 31. Anchor Stressing and Testing

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- Corrosion protecting and capping off the anchor heads.

Figure 32. Corrosion Protection at Anchor Head

4.4 Realtime Data Management

A selection of the ground anchors was installed with pressure transducer hydraulic load cells, connected to a remote digital readout system, allowing for real-time management of the load data. This means that while a flood is being routed through the spillway, the dam owner can monitor the performance of the anchors (and dam wall) via a remote digital platform. Similarly, the installation of digital 3D crack meters, monitored by a similar system, allows for real-time reading of data.

Figure 33. Read-out Cabling from Load Cells

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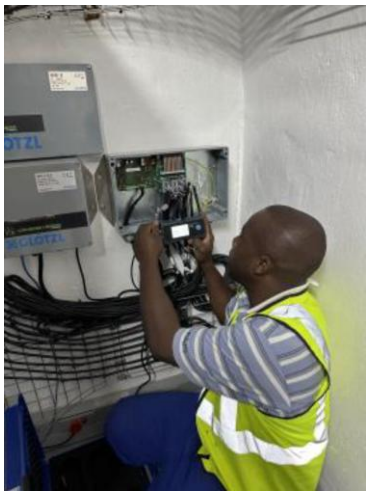

Figure 34. Load Cell Connection to Server Room

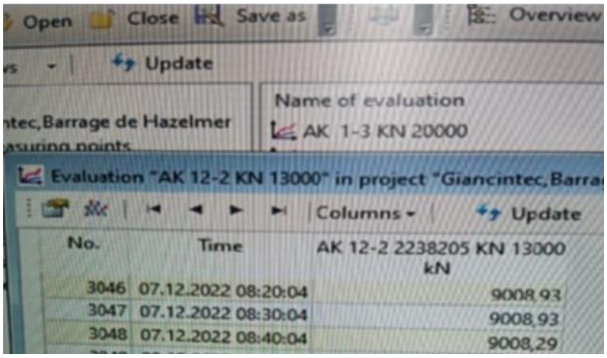

Figure 35. Remote Read-out of data

5. SUMMARY OF IMPORTANT PROJECT CRITERIA

5.1 Complexity of System, Including Process Engineering Aspects

The system's complexity is exemplified through the integration of the world's tallest constructed PKW structure, designed to accommodate massive flood volumes without overtopping the non-overflow crest (NOC). The unique PKW geometry required complex hydraulic modelling to achieve desired performance, especially considering the fixed spillway block widths and constraints posed by the existing vertical joints. Additionally, the system includes:

- 91-stranded ground anchors, each tested and stressed using custom-built hydraulic jacks,
- A multi-phase impoundment strategy requiring real-time coordination between old and new hydraulic systems,
- Compatibility between the new spillway profile and the legacy infrastructure.

5.2 Overall System Efficiency and Reliability

The redesigned system offers significantly improved hydraulic discharge efficiency, with the PKW ensuring robust flood attenuation even during extreme events (1:100-year flood of 1360 m³/s). Ground anchors penetrating up to 90 meters into bedrock ensure unprecedented structural reliability. The dam

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now provides a redundant, passive overflow mechanism that functions even in the absence of human intervention or power—unlike traditional gated systems.

5.3 Health and Safety Considerations

The design eliminated the need for radial gates, thereby removing operational and mechanical failure risks under emergency conditions. Health and safety were further ensured through:

- Advanced pre-grouting and waterproofing procedures,
- Strict on-site quality control including destructive anchor testing,
- Full compliance with South African dam safety regulations and international codes (e.g., BS 8081:1989).

5.4 Cost Effectiveness

Through innovative design elements like the PKW and bespoke anchoring systems, the project minimized long-term operational costs while maximizing performance. Eliminating mechanical gates reduces lifecycle maintenance. The reliance on passive hydraulic structures over mechanical systems provides long-term savings in operational expenditure. Given the scale and technological advancements, the R820 million cost represents exceptional value in comparison with international equivalents.

5.5 Maintainability

The design emphasizes low-maintenance components:

- The PKW is a gravity-based structure requiring minimal upkeep.
- Anchors are monitored remotely using embedded 3D digital sensors and load cells, meaning that the on-site system should not need to be disturbed in order to take readings. This also reduces the need for continual reinstatement of site components (e.g. corrosion protection).
- The real-time digital monitoring system allows for predictive maintenance, avoiding catastrophic failures and ensuring ongoing structural integrity.

5.6 Environmental Considerations

The raising of the dam enhances climate resilience without requiring a complete overhaul of the original structure. Key environmental features include:

- Minimal ecological disruption due to a phased impoundment strategy,
- Prevention of uncontrolled floods protects downstream ecosystems,
- Use of environmentally friendly grouting and water testing methods during anchor installation,
- Remote monitoring systems reduce the need for regular invasive inspections.

6. CONCLUSION

The Raising of Hazelmere Dam stands as a flagship example of modern dam engineering innovation—demonstrating how precise, site-responsive design can solve complex structural and hydrological challenges while setting a new global standard in efficiency and sustainability.

6.1 Site-Specific Design Solutions and Problem Solving

The Hazelmere Dam site presented a formidable design challenge. The original dam's curved geometry, limited freeboard of only 1 m between the proposed Full Supply Level (FSL) and the Non-Overflow Crest

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(NOC), and rapid time of concentration of the catchment meant that conventional gated spillways or fusegate systems were impractical.

To overcome these constraints, the design team introduced a Piano Key Weir (PKW)—retrofitted onto the existing concrete ogee spillway. This innovation provided the required flood discharge capacity while maintaining an uncontrolled spillway and ensuring structural stability.

Crucially, the PKW's ability to pass large flood volumes at lower hydraulic head allowed the design team to achieve the 7 m raising of the FSL without significantly increasing the upstream water level. This reduced the extent of land inundation and avoided costly expropriations that would have been necessary with a conventional gated or labyrinth spillway system. The reduced head requirement directly translated into a smaller reservoir footprint, environmental preservation of riparian zones, and a measurable economic advantage—making the PKW solution both technically superior and financially efficient.

Complementing this hydraulic innovation, the dam's stability was ensured through the installation of post-tensioned ground anchors, the longest and highest-capacity anchors of their kind globally, penetrating up to 90 m into bedrock. Together, these elements delivered a resilient structure fully compliant with modern Dam Safety Regulations.

6.2 Innovation and Design

The project's innovation extended far beyond structural geometry. Hydraulic model optimisation at the Department of Water and Sanitation's laboratory enabled the refinement of PKW dimensions to meet both flood routing and freeboard criteria while keeping within the dam's existing monolith spacing. Custom-fabricated stressing jacks, double corrosion protection and bespoke anchor assemblies were developed to accommodate exceptional design loads and ensure a service life exceeding 120 years. Integration of digital 3D crack meters and pressure-transducer load cells into the structure now enables real-time remote monitoring, ushering in a new era of digital asset management for dam infrastructure.

This combination of hydraulic ingenuity, structural reinforcement and digital monitoring has produced a dam that is safer, more maintainable and more intelligent than its predecessor - an achievement that epitomises engineering innovation applied to legacy infrastructure.

6.3 Lessons Learned

The Hazelmere project reaffirmed the importance of adaptive design and iterative problem solving when dealing with existing infrastructure. Retrofitting advanced technologies into a constrained site required flexibility, precision and close collaboration across structural, hydraulic and geotechnical disciplines.

Among the key lessons were:

- Economy through innovation: By employing a PKW that required less headwater, the project achieved the desired yield increase with minimal environmental and social disruption, avoiding unnecessary land expropriation and reducing costs.
- Hybrid thinking: Combining modern composite systems with traditional gravity dam design offers sustainable, safe and cost-effective outcomes.
- Digital continuity: Embedding smart monitoring systems ensures continuous safety oversight and predictive maintenance, extending the structure's useful life.

6.4 A Global Standard in Dam Engineering

The Raising of Hazelmere Dam exemplifies world-class South African engineering - balancing innovation, economy and environmental responsibility. The project achieved multiple global firsts:

- the tallest constructed PKW.

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- longest and highest-capacity post-tensioned (and restressable) anchors through a dam wall.
- integration of real-time structural monitoring within a rehabilitated gravity dam.

Beyond its technical achievements, the project delivered a sustainable economic outcome, increasing water security for more than one million people while saving money in avoided expropriation costs and preserving upstream land and ecosystems.

The Hazelmere Dam now stands as a global benchmark in dam raising design, demonstrating how locally engineered solutions can deliver international-level excellence—combining hydraulic innovation, digital intelligence and socio-environmental responsibility into a single, enduring achievement.

7. ACKNOWLEDGEMENTS

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In addition, the inputs throughout the project from the APP, Mr. Adam Botha, as well as the Anchor Specialist, Dr. Devon Mothersille were invaluable in completing this successful project.

The opinions and views presented in this paper are, however, those of the authors and do not necessarily reflect those of "the owner".

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SOUTH AFRICAN SPILLWAY GATES USED IN INDIA

P D Townshend
1. Amanziflow Projects, Johannesburg, South Africa

PRESENTER: R K VON HOLDT

ABSTRACT

An Indian company approached South African (SA) company Amanziflow Projects to use its automatic control gates on river weirs and dams which are more robust, reliable with negligible maintenance compared to risk prone electro-mechanical gates. The SA developed TOPS gate has now been installed on three structures in two separate states in India to retain storage for irrigation and to pass the very large floods experienced in India. Many more projects with these gates have already been approved. This case study covers the need for such gates, the design, fabrication, and installation of gates in India. These are pioneering projects in India using SA technology.

1. INTRODUCTION

India is a huge country and has the largest population in the world. With 1,4 billion people it is 22 times South Africa (SA) population and with a land area of 3 times SA, (Google) India's people density of 426 people per square kilometre, when compared to only 51 people per square kilometre in SA, indicates it is highly dependent on food and water to sustain the population. Farming is the backbone of rural India and agriculture employs about 40% of the workforce.

Whilst India has many large dams situated in hilly terrain with long distribution systems for irrigation, India also makes extensive use of small dams and weirs along rivers to supply water for nearby farmers and human development. India has an average rainfall of 1180mm/year whereas SA is 460mm/year. (Google) The India monsoon is 4 months of the year and is characterized mainly by long dry period and short heavy rains leading to flooding. With long dry season, agriculture and farming is heavily reliant on water supply and irrigation.

During British reign in early 1900's, use of falling shutter gates on river weirs was predominant to capture and retain surplus water between monsoon seasons. (Figure 1). Over the years, this system proved to be unsatisfactory. Currently extensive use is made of electro-mechanical gates. However, these types of gates have high maintenance requirements, dependent on electrical power supply, and have many risks of failure. (Townshend 2019).

2. BARRAGES AND RIVER WEIRS

There are thousands of such river structures in India. Most are fixed concrete or masonry weir, but hundreds more have flexible gates to increase storage for the long dry season. Of these types of gates, electro-mechanical radial or vertical lift gate are the majority. Other types include rubber dams, fuse gates, and a locally designed tilting gate.

Many older masonry weirs built in the colonial period in early 1900's have steel falling shutters which are bottom hinged steel panels supported by a diagonal strut to a shear pin or bracket. (Figure 1). These gates are raised at the end of the monsoon season and manually dropped open at the end of the dry period to allow for monsoon floods. This operation is dangerous with the operator harnessed with a

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safety rope. This system, whilst used for more than a century, does not provide assurance of supply with the climate changes being experienced around the world.

Figure 1. Existing weir with falling shutter panels to increase storage between Monsoon periods

These barrages have the usual problems with:

- Gates not opening to pass floods when required.
- Gates getting flushed away in floods.
- Sediment build -up.
- Faulty electrical or control systems.
- Lack of maintenance.
- Human error.

India has suffered dam failures, some with large loss of life, mostly due to faulty electro-mechanical gates. (Leyland 2018 & 2023). The current requirement is for many river weirs and barrages to be raised to meet increasing demand for water. The installations requirement is for gates to open automatically to pass floods, debris and sediment and to close automatically to retain the full reservoir capacity. The gates need to be devoid of electro-mechanical actuation, with minimal risk of failure, minimal maintenance, and without operator intervention.

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3. SOUTH AFRICAN INVOLVEMENT

In 2019, an Indian company Universal Hydro Structures (UHSPL), contacted Amanziflow Projects (AP) of South Africa to use the TOPS automatic spillway gate on Indian rivers. Having determined that the current barrage gates were not totally satisfactory, they found the TOPS gates through an internet search, as a suitable gate which met all the necessary requirements. AP participated with UHPSL in an international conference and exhibition in Bhubaneshwar, State of Odisha, in 2019 where considerable interest was received in the automatic spillway gate. A few proposals were accepted, one being the Bhakra weir.

4. BHAKRA WEIR

The Weir is on the Bhakra River, in the Rampur District in the State of Uttar Pradesh and serves two large irrigation canals.

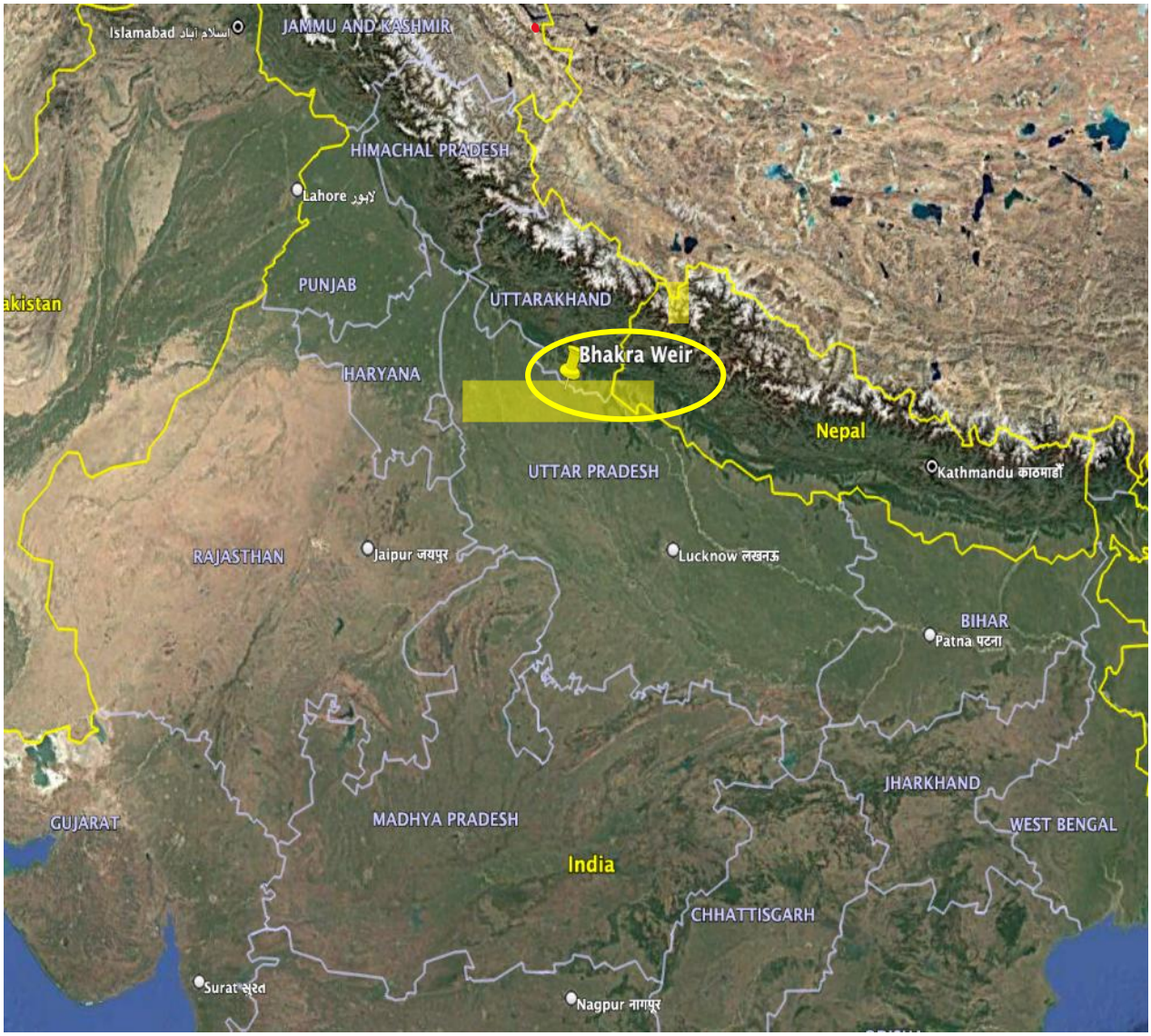

Figure 2. Location of Bhakra weir in India

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As far as irrigation is considered, Bhakra Weir is a very significant structure. It was constructed in 1926 by Harcourt Butler and was popularly known as Harcourt Butler Weir. This 100-year-old weir is a 60-metre-long stone masonry weir with an average of 2m high. It is solidly built and has withstood many very large floods. The weir was fitted with1.2m high drop panels to increase storage capacity in the non-monsoon period.

The civil work was carefully designed to prevent any damage to this old stone masonry weir. A special cutting machine was used to remove the piers, which prevented damage using heavy jackhammers.

The weir was jacketed with reinforced concrete and a concrete spillway constructed downstream of the weir. The piers for the gates were incorporated into the spillway construction. Energy splitters blocks were constructed to prevent downstream damage.

5. HYDROLOGY

The Bhakra River is a seasonal river with heavy flows in the monsoon season of 4 months, and then low flow out of monsoon period of 8 months.

The maximum flood is 1200m³/sec with an overtopping of the weir of 4.6m. During the low flow season the gates remain closed to provide storage for irrigation.

6. DESCRIPTION OF THE GATES

The four gates are fitted between new concrete piers as shown in figure 3. The trunnions are supported on steel lattice piers which were cast into concrete after the gates were installed.

The 4 gates are equally sized as 13.5m long and 2m high. The axle is 2.4m vertically above the top of the gate and is set back on the upstream side by 3.1m to be above the maximum flood level. The gate in elevation is shown in figure 3 and in section in figure 4.

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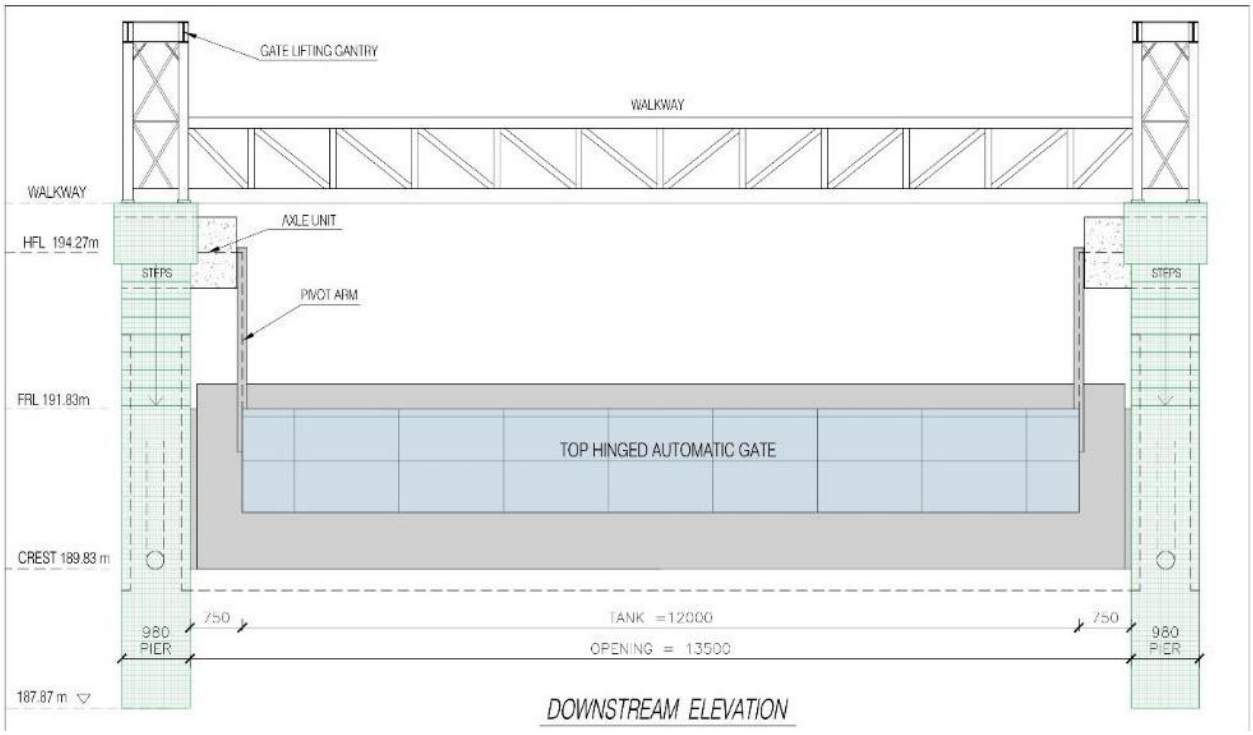

Figure 3. Downstream elevation on one gate

The gate consists of a front face against which the water level (WL) acts to open the gate. A ballast tank attached to the front face is filled with water from the upstream reservoir and acts as the counterweight to keep the gate closed against the upstream hydrostatic WL to retain the full supply level (FSL). The gate is connected by pivot arms to the trunnion axle unit.

The trunnion axle unit has Vesconite as the bush which is manufactured in South Africa. It is a hi-lube, low-friction, high strength, non-corrosive, non-metallic material and does not require lubrication.

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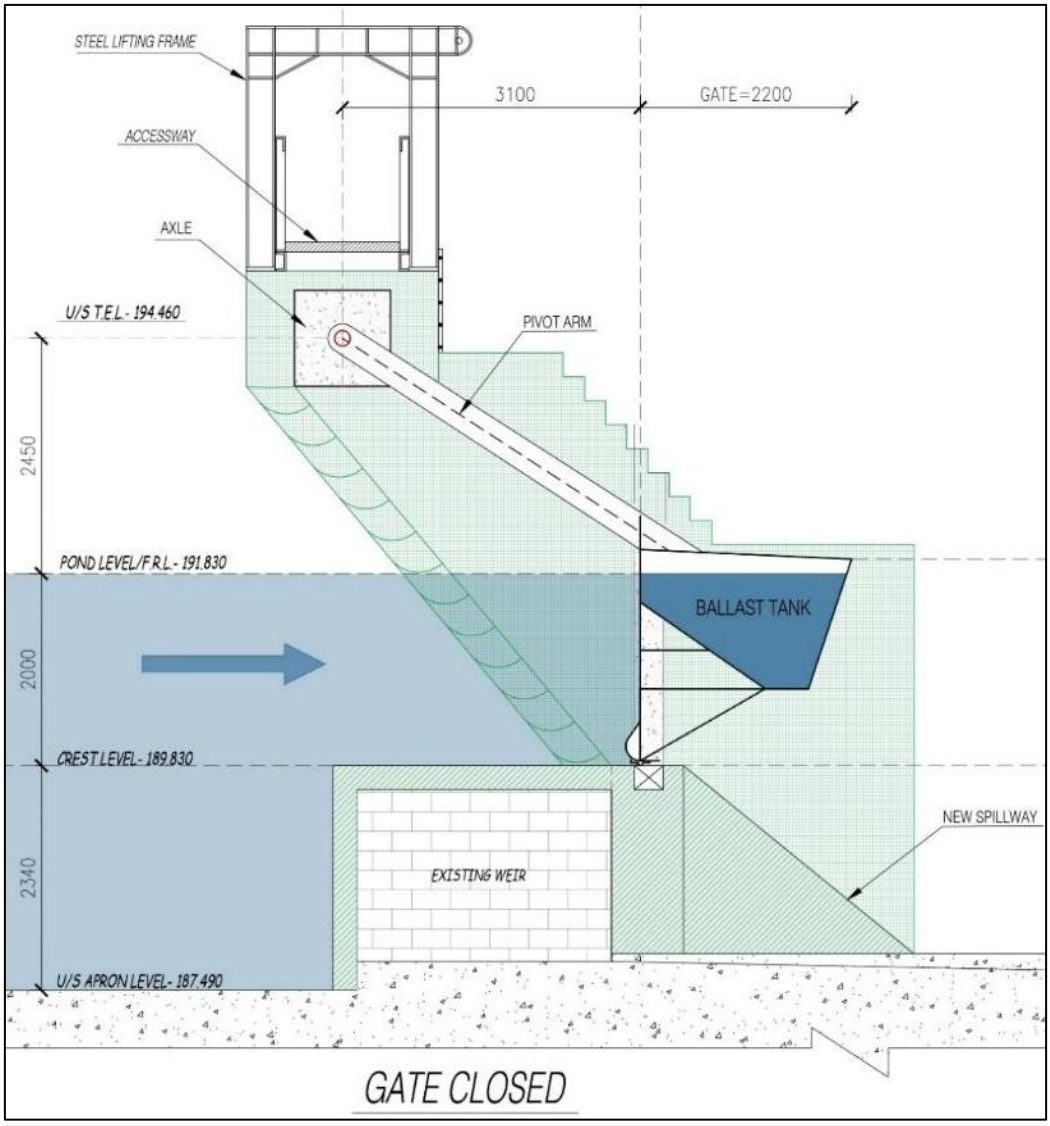

Figure 4. Gate closed to retain the FSL

As the WL rises in a flood, the increase in the upstream force will rotate the gate outwards and upwards. As it rotates, water decants from the ballast tank to lighten the counterweight. The gate will continue to rotate upwards as the WL increases, until at the HFL, the ballast tank is empty and the gates floats on the HFL as showed in figure 6. (Townshend 2012).

As the flood recedes, the gate rotates downwards and inwards, and the ballast tank fills so that the gate will close to retain the FSL in the reservoir. The TOPS gate therefore retains the FSL which is unlike other fuse gates where the increased storage is drained away. The TOPS gate is an effective fuse type gate in that it will respond to increased WL from floods to open automatically without external actuation and operators. It will then remain on the spillway and will close automatically to retain the FSL.

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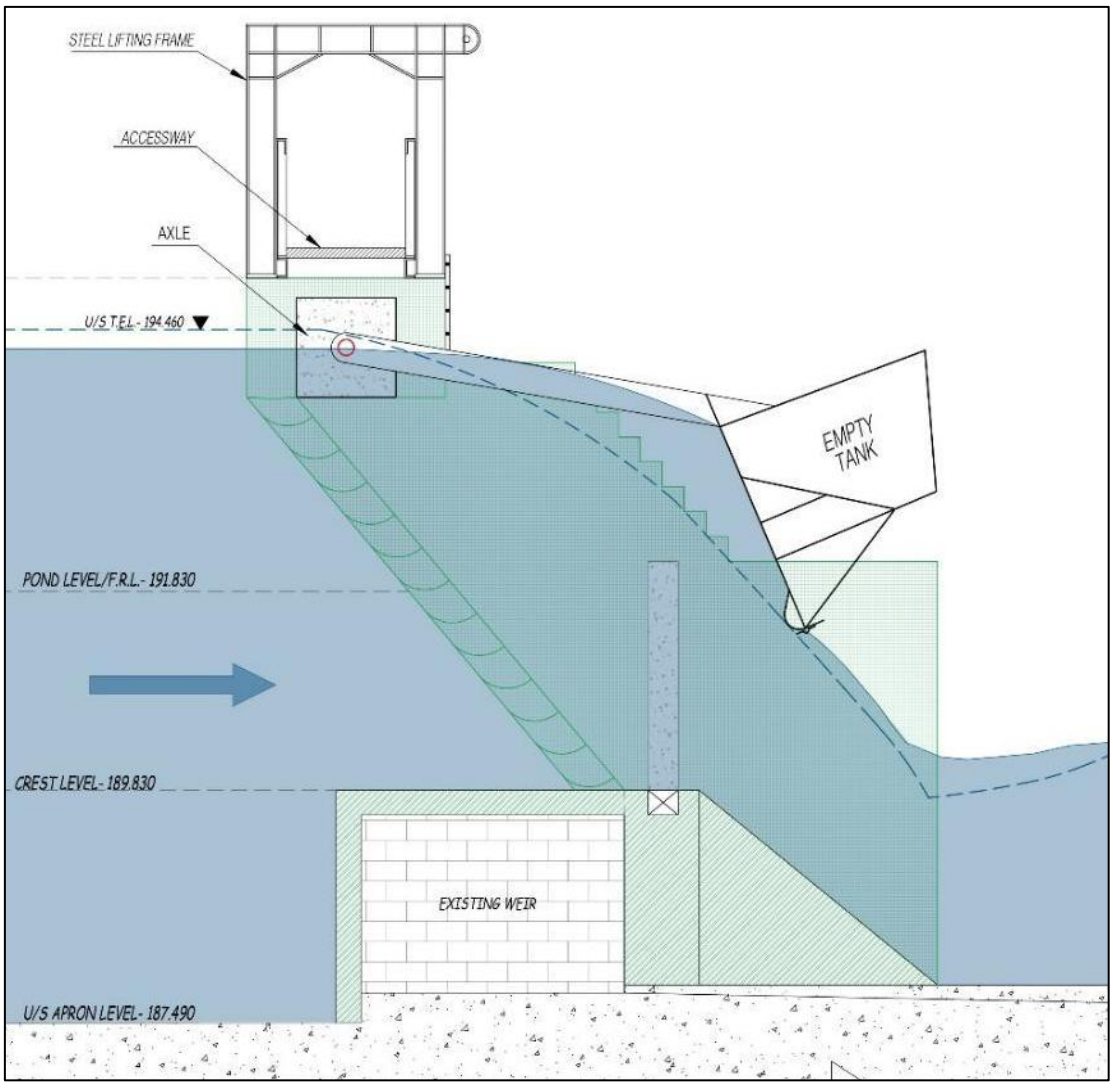

Figure 5. Gate fully open to pass the maximum flood

7. DESIGN

The gate is balanced between the upstream opening forces and the counterweight closing forces, using customised software. The gate shape is like a box girder with a large moment of inertia. Consequently, the stresses and deflection are low. This allows the TOPS gate to close off larger spans than the bottom hinged gates and electro-mechanical gates.

The structural design is performed using 2D and 3D structural software.

8. PERFORMANCE OF THE GATES

The gates close off the full span between the piers of 13,5 m. In the closed position they retain the FSL in the reservoir of 2m. The gates are set to open 100mm above the FSL. The middle gate has a siphon

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discharge from the ballast tank. This allows the middle gate to open before the other three gates. The siphon gate is lower than the other three gates to allow flotsam to flow over the gate before it opens.

The gates can be opened manually by opening a valve on the ballast tank. This will be done occasionally to sluice out accumulated sediment and debris from the reservoir. Further the pier is fitted with a lifting frame which is used to lift the gate clear of the water, if required. These are additional features to the automatic functioning of the gate and provide means to lift the gates, which is the procedure during the maintenance activities, (figure 6).

The gates are provided with redundancy siphons to ensure the gates open when required.

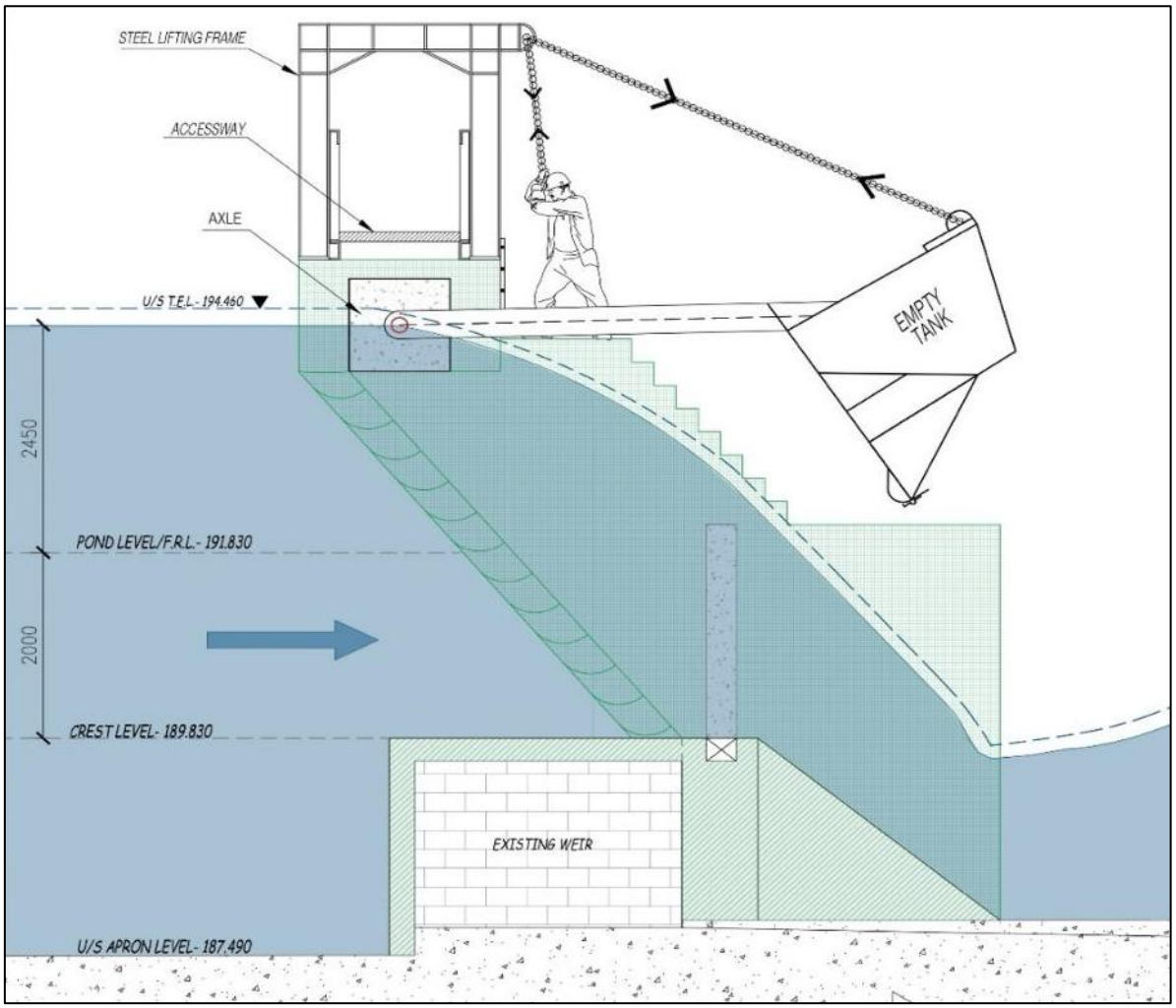

Figure 6. Lifting gate clear of water.

9. FABRICATION AND INSTALLATION

The gates were designed in SA by Amanziflow Projects as well as the fabrication drawings. The gates were fabricated by Indian company Universal Hydro Structures Pvt. Ltd. (UHSPL) under license to Amanziflow Projects.

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The gates are in mild steel. The corrosion protection is a good quality twin pack polyamine paint applied under strict quality assurance procedures. The gates are assembled complete in the fabrication works and tested before dispatch to site. The seals are fitted in the workshop together with the seal plates. The seals have embedded PTFE lining to minimise seal friction as the gate opens.

The gates were installed as one unit with the seals plate attached to protect the seals against damage during the long transport and during installation. The pivot arms are then bolted and connected to the trunnion units. Once the gate was levelled and checked, the seal frames were concreted in.

When the reservoir reached FSL, the gates were opened manually to demonstrate its workings to open and close as part of the wet commissioning. (Figures 7 and 8)

Figure 7. Gates open to pass the river flow

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Figure 8. Gates closed to hold 2m extra water in the reservoir

10. CONCLUSION

The South African designed and developed top hinged TOPS gate has found acceptance at the Bhakra River Weir in India. It offers advantages over electro-mechanical type gates and the other automatic gates in that it opens and closes automatically and requires minimal maintenance or operator intervention.

It is suitable to handle large and quick response flood peaks caused by cyclones and heavy monsoon rains as well as opening to pass flotsam and sediment to maintain adequate storage in the reservoir They are also fitted with back up discharge siphons to ensure redundancy for the gates to open.

This is the first South African gate of its kind in India and is viewed with great interest. There are many more scheduled for the next few years. There are however many installations of the TOPS gate in Southern Africa. These barrage gates need to be considered in South Africa to provide a series of barrages on the river for storage along a river rather than building or raising dams.

11. ACKNOWLEDGEMENT:

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MULTI SMALL DAM PROJECT - TAKUM EDUCATION INSTITUTE, NIGERIA

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PRESENTER: RYAN THOMSON

ABSTRACT

This paper presents the integrated design and engineering of a multi-dam system at Takum Education Institute (TEI), a first-of-its-kind tertiary education facility in Taraba State, Nigeria. The use of small dams surrounding the campus was developed for multi-purpose use, primarily for recreational and overall aesthetic appeal, but also to address critical needs in water storage for agricultural pilot projects, flood mitigation, and sustainable campus development. The scope comprises assessing the best use cases for the available water, given the confines and logistics of the university campus. Innovative features include an HDPE-lined dam with subsurface drainage networks for near-potable water recovery at outlet chambers, desanding beds, dealing with campus accessibility, interconnected treatment systems for both fish farming practices and potable water to campus. The design integrates environmental aesthetics with robust hydraulic performance, supporting aquaculture, irrigation, and recreational use. Geotechnical investigations, seismic considerations, and sustainable construction practices underpin the system's long-term viability. This project exemplifies a holistic approach to dam engineering in a semi-urban African context with very little to no data available, balancing infrastructure, ecology, engineering standards and community needs.

1. INTRODUCTION

Tembo Power Engineering (TPE) was presented with a multiple small dam architectural concept (Figure 1) surrounding a university campus. The campus' conceptual layout and initial water features were developed by the project architect. The scope of works required by the client included optimising the water balance of the university and residences, whilst remaining cognisant of the dams' recreational use and aesthetic features. The campus is situated near the confluence of Stream 1 (3.7 km^2) and Stream 2 (4.2 km^2), which join less than 1 km downstream. Both streams are non-perennial, and the initial proposition, by the client and their architect, was to have four small dams as numbered below, surrounding the campus.

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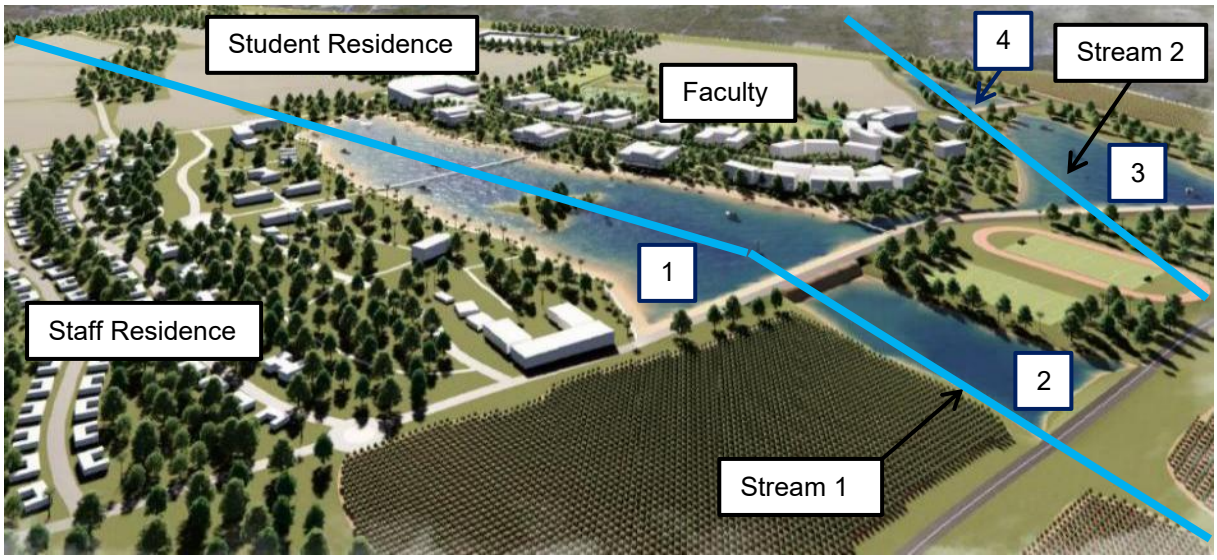

Figure 1. TEI 3D Aerial View. Photo: courtesy of Peerutin Karol Architects

In addition, the client had envisioned the "look and feel" of the campus to follow that of the International Institute of Tropical Agriculture (IITA) headquarters in Ibadan, Nigeria. In other words, as natural looking as possible (Figure 2), whilst ensuring minimal water level change, cost-effective storage and functional use of the water resource to the various campus facilities.

Figure 2. IITA Forest Centre, Ibadan, Nigeria (Courtesy of @IITAForest)

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The design concepts presented herein and eventual infrastructure adopted for the project are not new; however, the complexity of the project was in aligning with client expectations in a continuously changing campus layout. This was due to multiple entities (the engineering, architecture and landscape teams) working on the same development simultaneously. An important technical challenge included dealing with substantial shallow subsurface flows on Stream 1.

To provide a timeline of decisions, the structure of this paper follows three main phases, which are discussed in more detail separately:

- *Kick-Off* – Data gathering and justification (aligning the preliminary concept with technically feasible solutions).
- *Phase 1* – Concept development and early works project costing (with options).
- *Phase 2* – Detailed Design of Preferred Concept.

2. KICK-OFF – OVERCOMING EARLY DESIGN HURDLES

At the project kick-off, it was realised that the lack of on-site data would need to be resolved within the tight deadlines of the project. Lack of geotechnical information and rainfall data/streamflow data were identified as key aspects to validate. Geotechnical information can be determined relatively quickly (in relation to obtaining reliable new collect, hydrological gauged information. It is cases such as these that the engineer needs to evaluate from the site information comprising a site walk-over, geotechnical information gathered/shared and the expected infrastructure (project size and nature), what further information could be available in a short time period to further develop the concept. On the other hand, without site-specific rainfall and climatic data, it would take months or even years to develop highly accurate hydrological studies. Again, it is in such cases that the engineer has to evaluate the project timelines and make use of reasonable methods applicable to the catchment, which evidently has characteristics that will change in future, given the urban development of both the University and the Village Town of Takum. The following can be said about the early information needed and collected.

2.1 Geotech

Preliminary geotechnical investigations (test pitting and trenching along dam alignments) were conducted using a 20t excavator. The preliminary investigations were carried out in order to determine if there were adequate founding conditions. Unfortunately, not all the planned test pits and trenches were carried out due to the difficulty of access, mainly when it came to crossing the streams. However, it did allow for justification that the marshy conditions would not be appropriate for some of the proposed infrastructure. It was for this reason that a combined spillway between the two streams on suitable foundations, as detailed later, was the preferred solution. The marshlands were later traversed during the dry season to confirm the material assumptions made.

2.2 Hydrology

Many available sources on the subject matter conclude that flood frequency analysis and realistic design flood estimations associated with a level of risk are pertinent in the economic planning, safe design and operation of various critical hydraulic structures. However, this can be elusive when on-site rainfall and streamflow data are not available, as per the case in establishing flood events for the respective spillways.

An event-based approach was used to simplify the estimation of catchment conditions, given the size of the catchment as well as the probable future development of the catchment. The Soil Conservation Service (SCS) Method was preferred to the Rational Method in this instance as it is less sensitive to user inputs, whereby if not used circumspectly, runoff coefficients and rainfall intensity can yield results further from reality. In addition, the SCS method is deemed more appropriate for the smaller agricultural catchments and has largely replaced the Rational Method in the USA (a jurisdiction with which the client

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is more familiar) whilst still allowing the computation of the entire hydrograph and is recommended for both urban and rural catchments with areas < 10 km^2 (Smithers, 2012).

2.3 Concept inconclusive:

The design had to progress, understanding that the campus layout and functionality were still being finalised by others. This meant that in some instances, conservative assumptions had to be made whilst also having input into the technical implications of a shifting layout.

3. PHASE 1 - CONCEPT IMPROVEMENT (IN A CHANGING DESIGN LANDSCAPE)

3.1 Dam Sizing / Planning

A first stage design consideration was to establish what the available water volumes would be in the respective dams. Therefore, typical stage storage curves (i.e. surface area (m^2) and storage capacity (m^3) in relation to spillway elevation) were established for each of the dams, including an option to combine dam 1 and dam 3 (Refer back to Figure 1 for dam numbering).

Whilst the stage storage curves are not presented in this paper, this led to the conclusion that there was little benefit to having Dam 4 (13 000 m^3) as it was worthwhile raising the combined Dam 1 & 3 option (450 000 m^3) to impound closer to the proposed Full Supply Level (FSL) of Dam 4. In other words, the removal of a single dam upstream was largely compensated for by raising this combined option by a mere 0.5 m. Besides the advantage of the savings in the construction of one combined spillway for Dam 1 and Dam 3, instead of one for each dam, another advantage was the ability for the now combined storage of both Dam 1 and Dam 3 to benefit and maximise overall water retention should one stream receive more rainwater due to a more localised rainfall. It is worth mentioning at this juncture that the client insisted on keeping Dam 2 (10 000 m^3) for flexibility in fish farming activities.

A second stage design consideration was determining which type of spillway would optimise the solution. On this design step, it was important to seek risk and functionality guidance from the client. The two main instructions received were that the spillway must consider aesthetics as far as possible, as well as minimise the freeboard surrounding the dam to give the impression of a "full dam".

In terms of matching these minimum design requirements to an appropriate flood categorisation, the SANCOLD Guidelines to Floods (Report No. 4) and Freeboard (Report No. 3) were consulted and applied. All 4 dams fell into "small" height categories.

The hazard rating from potential loss of life was defined as "low", but the potential economic loss was defined as "significant". Hence, a Class (Small) with Hazard Rating (significant) resulted in a Category II (Q_{100}) being assigned to the combined Dam 1 and 3 option. Although in terms of height and actual economic loss, Dam 2 fell in a lower Category, it did not make sense to design Dam 2 for a lesser flood, and hence the Significant rating was carried across all three dams. Through sharing of known past project spillway types, a duckbill-shaped spillway was opted for the combined Dam 1 and 3 option, to meet both low freeboard (through use of an elongated crest length) as well as aesthetics (client preference).

Dam 2 was considered of less aesthetic importance but served functions such as (i) supplemental storage for fish farm activities, (ii) access across the stream to service buildings and (iii) dissipation of flood water from the combined 1 and 3 spillway chute. At this point in time, the design of Dam 2 continued with the intent to use Armourflex® or a similar material to establish a more natural-looking embankment with a crest that was both overtoppable and driveable. It would later be proved that Dam 2 did not provide sufficient storage to be of any benefit to the holistic water management plan.

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3.2 Water Usage Planning

A complete water services review of the campus and residence was done. This included calculating water demand for potable needs – 1 ML/day (Redbook – Part II, Section J), wastewater - Average Wet Weather Flow (AWWF) and stormwater (restricted to locally available material), amongst other services such as roads (pavement design by others) and electrical services (design completed by others).

In determining what water could be used for the relevant processes without extensive treatment, the client's instruction was that the dams should be used first as a feature of the university and second as a source of water. To achieve both a feature and meet the university's water requirements, a solution was required that minimises the water loss (to keep said appearance of a full dam), whilst allowing for the replenishment / re-circulation of lost water. Hence, the option for an HDPE-lined dam with a network of sub-surface drains below the liner was selected. The intent is to collect the filtered subsurface flows in a common outlet chamber and to supplement this flow by means of a series of boreholes as the main source of potable water for the university. The number of boreholes would be determined later based on achieved pumping rates and campus demand. Furthermore, the boreholes could be increased in stages to match the growth in campus development.

In order to maximise this process, a shallow retaining wall along the campus facing perimeter of the combined Dam 1 and 3 option was introduced to reduce the potential evaporation area and waste of campus space resulting from a retreating or flooding shoreline. For perspective, the difference between the high water mark and dry season water drop of say 0.5 m would result in almost 20 m offset in some zones along the banks of the reservoir, which was deemed aesthetically prohibitive to the campus layout. To further reduce the impact of receding shoreline and maintain the water level in the dam to maintain the "dam full" visuals, the filter zones and some of the fishbone underdrains (under the HDPE liner) discharged into a chamber fitted with a set of submersible pumps returning the water to the dam with the option to pump into a 1000 m^3 ground storage reservoir (if excess flow was available or when the dam is full) for other usage. The water distribution network is detailed further in the section to follow.

3.3 Minimum Design Considerations for a Multi-purpose, HDPE-lined dam

It should be noted that ponds constructed off-channel allow for their construction in a more controlled environment. However, lining of dam(s) that are still in channel requires further considerations around future de-sanding, damage during floods, construction in wet conditions, and predominantly where shallow sub-surface flows are present, etc. The attenuation ponds with desanding capability upstream of the dams are discussed in the section to follow.

In addition, as the inflow would be low to negligible, particularly in the dry season, the lined dam(s) also require "treatment" in the form of aeration fountains to aerate the water in order to reduce algae growth and reduce the risk of bad odours arising from the collected silt. In effect, aerating fountains circulate bottom water to the surface, where toxic gases exchange for oxygen. Compressor units were therefore incorporated in the design to be housed around the perimeter of the reservoirs with aeration tubing running along the bottom above the liner and aerators placed at various locations across the reservoir basin. This topic was discussed in detail with the respective suppliers.

Lastly, the fixing of the liner to the dam perimetric boundary needed to be robust, whilst cognisant of the aesthetics (i.e. not visible at any points along the perimeter) and not be subject to forces from the incoming flood water. In order to summarise the design outcomes for the reader and document decision timelines, refer to the matrix presented below.

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Table 1. Early Stage Decision Matrix

Design Decision	Analysis / Design & Cost Check	Outcome
Dam Quantity & Quality	Stage Storage against comparable cost(1)	A larger combined dam option with a shared spillway.
Dam and Spillway Type	Least freeboard (i.e. long spillway) and aesthetics. Dam alignment traversed marshland closer to streams, limiting founding options.	Earth embankment with a shared Duckbill spillway at an appropriate location with a stepped concrete chute(2).
Optimising Water Available	Evaporation loss, borehole costs, HDPE liner costs, Subsurface network costs	Reduced surface area and shoreline retreat through a vertical perimeter wall. Underdrain connected to outlet chamber to offer re-circulation to dam alongside supplemental boreholes.
Dealing with HDPE liner disadvantages in a multi-use context	(i) Dealing with sediment from incoming streams. (ii) Liner Maintenance. (iii) Dealing with stagnation during dry season. (iv) Dealing with liner visibility and "hippos".	(i) Raised culvert embankments with cleaning capability. (ii) Dam outlet chamber for liner maintenance. (iii) Aeration fountains to supplier specifications. (iv) Subsurface perforated pipe and bubble drain network below liner(3).

1. *Comparable cost refers to cost estimations at various elevations based on similar past projects.*
2. *The client did not want a simple Ogee weir and did not want a simple gabion chute (appearance).*
3. *The liner was connected to the base of the perimeter wall with a stainless steel strip as discussed below.*

4. PHASE 2 – DETAILED DESIGN OF THE PREFERRED SOLUTION

With the concept design approved, the next phase included the detailed design of the various components, including the integration of the various multi-use features. For example, the preliminary design discussed the use of a water tower on campus. Up until this design phase began, it was always a point of discussion that was "kicked further down the road". Based on the calculated demand of 1 ML/day, and the understanding that, typically, in urban areas, water stored on site should not exceed 2 days (48 hours), a 1 ML ground reservoir (24 hours) was selected because both the boreholes and low lift pumps had redundancies. The water demand also decreases significantly during the university holidays, so the water will tend to stay in the tanks for longer during those periods. For completeness, in a more remote location (which is not the case here), 4 days would be preferable (96 hours). Usually, more than 5 days of storage is not advisable, as water standing in reservoirs for long periods of time tends to attract algal growth.

To ensure a more appropriate and constant pressure delivery system across the campus, a water tower was initially conceptualised. The water tower should ideally be sized for 4 to 8 hours of Average Annual Daily Demand (AADD), so 170 kL to 340 kL. It was initially planned to be a raised steel Abeco tower or similar. However, Tembo proposed to relocate the reservoir to a nearby hilltop, outside of the property boundary. This offered both a simplified, on-ground, concrete reservoir with potentially much lower maintenance costs, placed "out of view" from the campus, which was preferred by the client. The client managed to secure the necessary land for the implementation of the ground reservoir and associated

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infrastructure. In order to keep critical infrastructure on campus (for security and reduced maintenance complications), the following water plan was recommended.

4.1 Water Use Planning

The University water system considered the following, as represented by Figure 3:

- The underdrain network (orange) collected at the sole outlet chamber (fishbone type structure not shown to simplify the illustrative nature of Figure 3). This catered for the collection of a semi-treated source - (filtered water to the 1000 m^3 reservoir, supplemented by boreholes as required).
- The 1000 m^3 reservoir (stationed at level 260 m) includes a set of low-lift pumps to convey the filtered water (light blue) to a 350 m^3 intermediate reservoir (stationed at level 280 m) before being treated by a package plant and further pumping the potable water to a 350 m^3 hilltop reservoir (stationed at level 340 m).
- The hilltop has a high elevation and was considered too far in distance for complex maintenance (> 1.0 km and hence not shown); therefore, it was preferred to keep the package plant and high-lift pump set fully on campus. The intent is for the 350 m^3 reservoir on the hilltop to have very little to no power requirements, as the line would be gravity-fed back to the campus. Only a 12 V DC power supply, provided via a mounted solar panel, is required for the remote 5G CCTV and reservoir level probe.
- Water is to be gravity-fed to the campus (purple). A pedestrian bridge, across Dam 1, will accommodate the potable water supply pipe. An option to offtake at the intermediate 350 m^3 reservoir for garden standpipes only was allowed to reduce unnecessary package plant usage.

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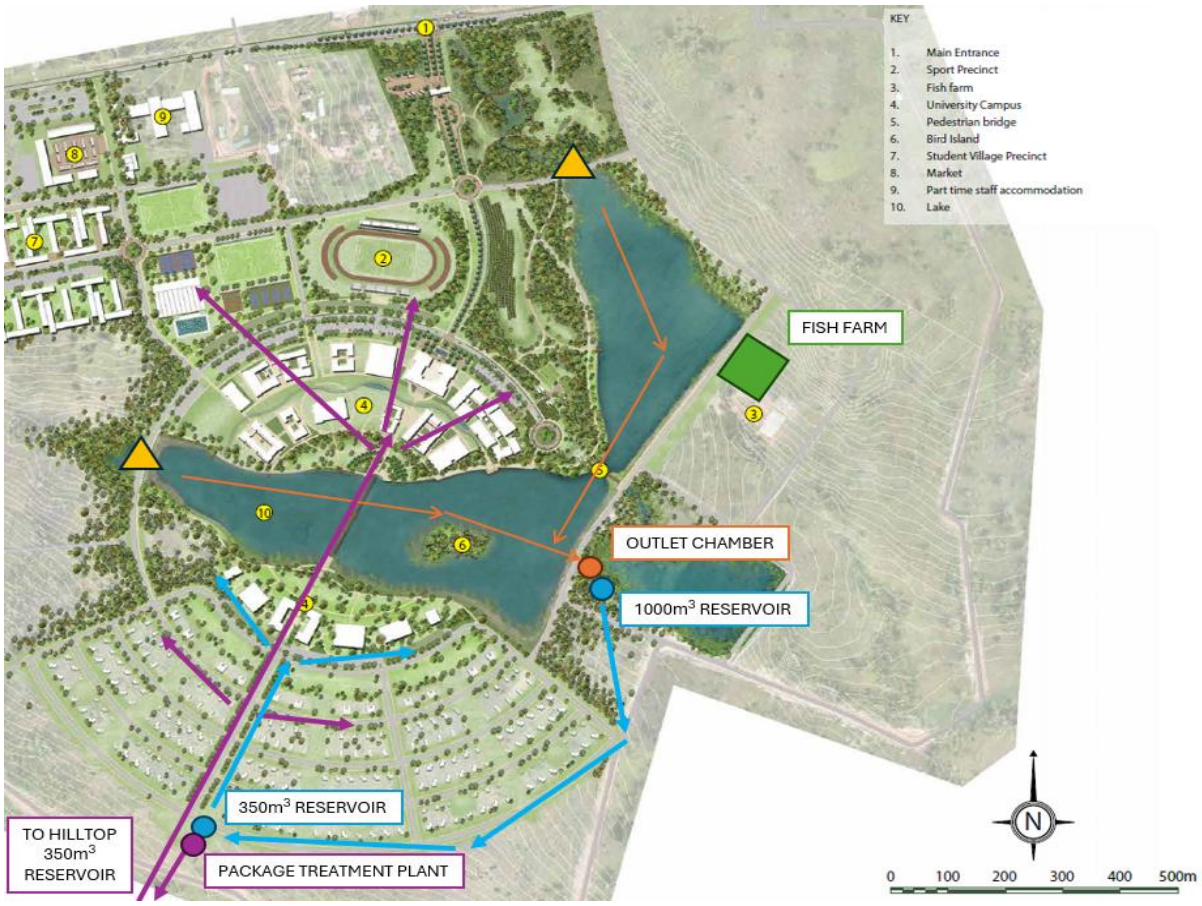

Figure 3. Campus Water Plan. Photo: courtesy of Landmark Architects

4.2 HDPE liner planning

Planning the HDPE liner connections to the perimeter was a "back and forth" process with the landscape architects, the HDPE specialist suppliers, the client and the contractor on site in order to achieve the desired aesthetics. The opted solutions (as shown in Figure 4 below) met both the desired edge treatment, robust liner connection and underdrain maintenance capability. Approximately 24 ha of 1.5 mm HDPE geomembrane was ordered. Respective suppliers tend to have their own requirements for air release in accordance with their Intellectual Property (IP). This ranges from various arrays of pop-up pressure relief valves or air release pocket mechanisms aligned to geofabric wrapped bubble drain (gas venting strips). Aesthetic constraints had to be considered in the appropriate air release methods. For the benefit of the reader, minimum standards to become acquainted with include SANS 1526: Liner Composition, SANS 10409: Design, selection and installation of geomembranes, as well as GRI-GM13 (GRI GM29).

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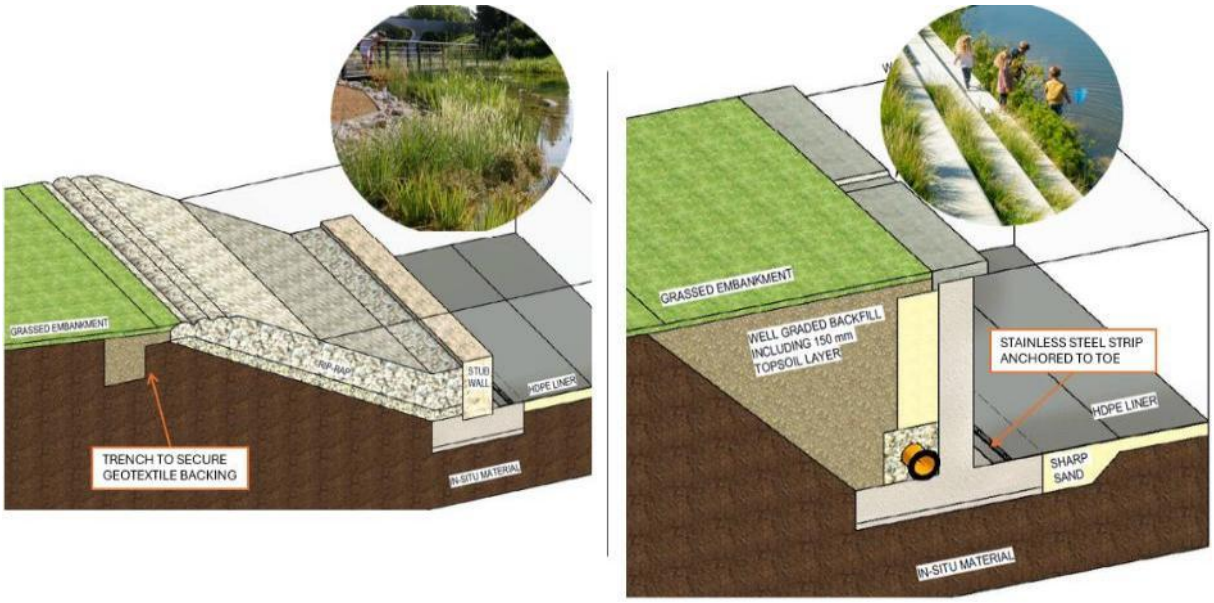

Figure 4. HDPE liner planning and edge treatment details (hard & soft feel)

4.3 Subsurface collection and outlet chambers

The outlet chamber is not shown as all the elements are difficult to capture in a single, meaningful image. The role of the outlet chamber was to allow for (i) the drawdown and maintenance of the dam as well as (ii) subsurface water collection (below the liner and toe drain). Some key aspects that were considered are:

- The outlet was split into a raw water collection chamber and a filtered water collection chamber (from underdrain and toe drains), which were kept separate for treatment flexibility.
- The outlet and subsurface pipes running through the embankment were encased in a concrete trench with a minimum 0.5 m wide "collars" at 12 m centre to centre to mitigate the risk of piping in the foundations.
- The raw water chamber discharged back to the river, with a t-off option for fish farm/future irrigation activities for the planned small-scale pilot irrigation fields for research purposes.
- The filtered water chamber included a set of submersibles, which pumped to the nearby 1000 m^3 reservoir (for storage) or a t-off option for back into the dam (for water level top-ups during the dry season).

4.4 Dealing with sediment and stormwater

As stated previously, dealing with a lined reservoir still connected to its natural watercourse needs careful consideration. Not just in the context of the incoming streams, but campus stormwater runoff too.

In principle, the culverts located upstream of the two reservoirs (see orange triangles - Figure 3 were designed with a sloping floor with the inlet section raised above the FSL to offer an attenuating pond-like infrastructure to trap sediments from incoming non-flood flows. This included a driveable, upstream riverbed section to allow for frequent cleaning of trapped sediment (as flushing options are, for obvious reasons, not valid). The outlet section of the culverts was submerged below FSL for visual and energy dissipation reasons. In addition, an extended apron slab on the downstream side was included to reduce any effect of high velocity flow on the liner connection, which would cause lifting/ripping of the liner.

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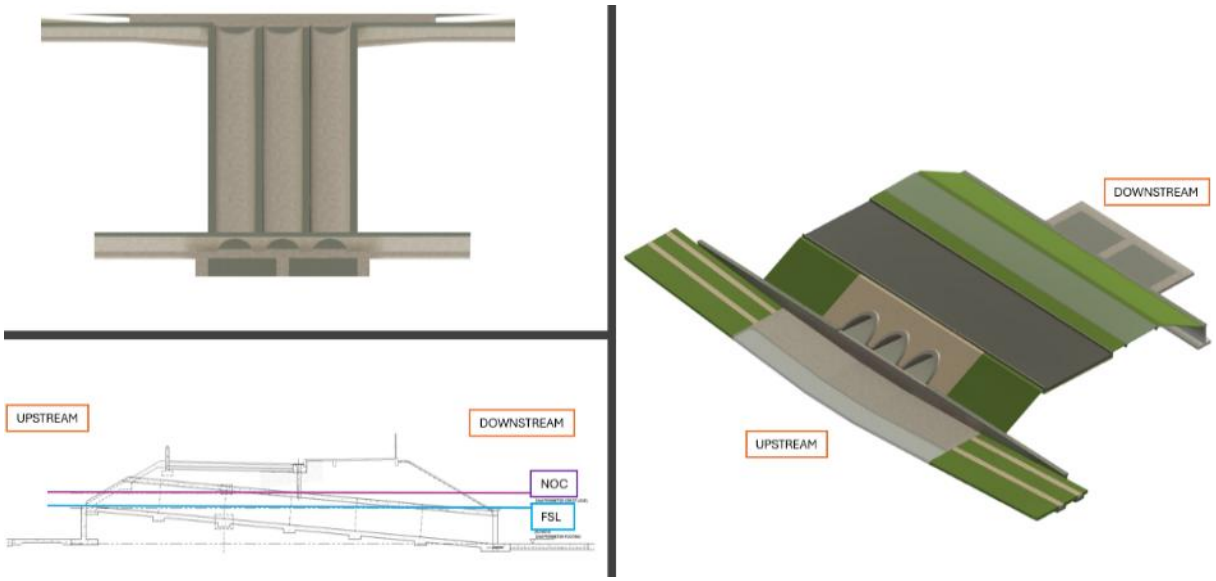

Figure 5. Upstream attenuation ponds (culvert crossings)

Regarding stormwater around the reservoir perimeter, the client did not want any open channel draining directly into the reservoir for both visual and safety concerns. Therefore, a network of stormwater culverts with several types of inlets and separation chambers was provided. A key component was the inclusion of separation chambers that can collect rubbish (litter) and sediments (with ease of access for maintenance). This was done primarily to reduce sediment-laden stormwater from directly running into the dams. The principle of the separation chambers constructed on site follows closely that of the Contech Debris Separating Baffle Box (DSBB), which is well documented online.

4.5 Spillway and Bridge Considerations

The spillway section geometry was analysed using CADAM3D® software as shown in Figure 6 below.

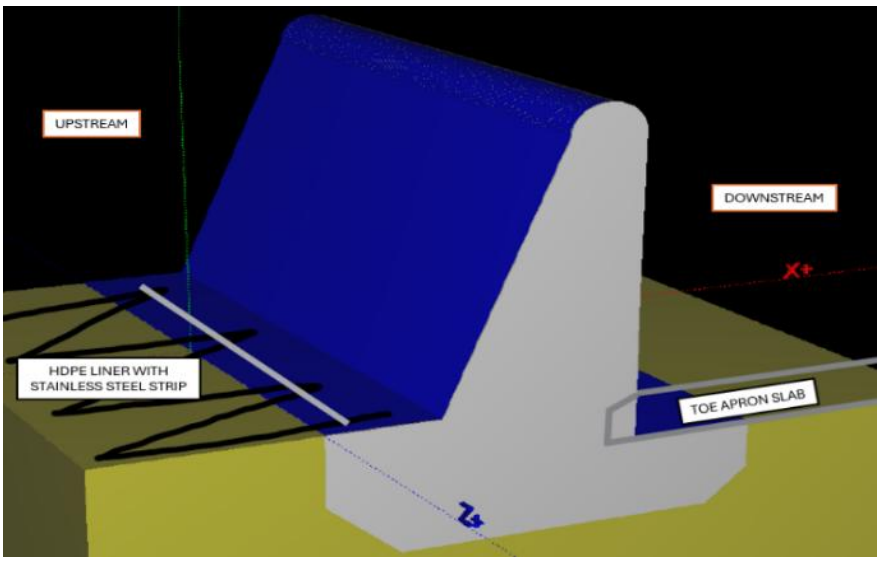

Figure 6. Duckbill section geometry (CADAM3D®)

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The analysis is not elaborated herein as the process and results criteria are well documented in the Engineering Manual (USACE, 2005). Although the spillway was analysed under the gravity method, anti-crack reinforcement was provided along the entire spillway wall because all impounding infrastructure needed to be constructed timeously to close the dam during the dry season, whilst certain elements remained unclear at the time, such as:

- Which exact locations the pedestrian bridge supports would sit, raising point load variances later on.
- A late decision by the client to include slightly lowered crest sections on either side of the bridge (for more concentrated spill during lower flows), with an increased downstream Tail Water Level (TWL), to give the impression of a waterfall sound. This meant lowering the toe for the downstream apron slab to be "tucked in" – potentially yielding unclear stresses at the base of the wall during higher uplift.

The final constructed spillway infrastructure is as presented in Figure 7 and Figure 8 below.

Figure 7. Duckbill spillway with pedestrian bridge, traffic bridge and discharge chute to river

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Figure 8. Downstream view of the spillway during dam first filling

4.6 Aeration planning

Similar to the potable water treatment package plants, the aeration and fountain equipment, as well as the supporting infrastructure, need to be optimised with the preferred supplier early and were not finalised at the time of preparing this paper. However, provision has been made for the necessary infrastructure.

Whilst the request for proposals and Offers are not detailed herein, the key metrics for a designer to consider are in specifying the maximum turnover of the waterbody for the least continuous power usage (kW) over time. This will give a good sense of operational cost and not just the initial capital required to procure, install and commission the equipment.

4.7 Sewer Network

In general, the internal sewer network was constructed from Class 34 uPVC pipes (Heavy duty, 300 kPa stiffness) of sizes varying from 160 mm to 355 mm (based on in-country availability). The network of pipes was laid in straight sections despite the circular layout of the university buildings, with all bends occurring at precast concrete manholes of a minimum diameter of 1 m internally. The network was designed with a median length of 60 m between manhole sections to allow for pipes to be unblocked via conventional rodding techniques. Where sections were required to be longer, the absolute maximum length was 80 m.

Perhaps the most important aspect to note was that some sections of the sewer network were required to pass under the HDPE liner of the dam, and these sections were encased in concrete to minimise the ingress of water, or contamination of the groundwater that will feed into the water collection chambers. Careful attention was paid to the waterproofing of manholes located next to the dams (< 50 m). It was recommended that manhole rings be sealed with an approved joint putty between the interlocking rings, and a 300 mm torch-on seal (Fidia 3 mm or similar) along the exterior of the manhole ring.

The treatment of sewerage will be by way of phased package plants. However, the entire network is gravity-driven as it was deemed that any pumping of sewerage would not be suitable, as any failure or temporary downtime of such a system would not be acceptable in such an environment.

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CONTROL THE UNCONTROLLABLE USING DAMS

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ABSTRACT

Dams, man-made structures that can be used to control the uncontrollable act of nature, floods. Is it possible to apply our technical skills, knowledge and creative minds to combine the function of a dam to be used for water supply as well as reducing the effects of floods in our rivers and municipal stormwater systems? Floods not only affect the natural river course, but also the infrastructure around it and could cause loss of life. In addition, new residential estates and commercial developments require stormwater retention ponds/dams to be built before entering the municipal system. This paper will investigate and determine how it is possible to optimise man-made structures, especially dams to mitigate the potential hazard and risk of floods on communities and infrastructure in South Africa.

1. DEFINITIONS

Technical and engineering terms and definitions are listed below:

- "**Recurrence Interval**" (RI) and return period means the average time period/internal between rainfall or flood events that will be exceeded once.
- **"Greenfield development"** means infrastructure built on undisturbed land.
- **"Brownfield development**" means land that has previously been used/developed for a specific purpose and typically have environmental challenges.
- **"Sponge city"** means a city which uses natural features to reduce surface runoff and hence reduce flooding.
- **"Urbanisation"** means an increase in communities that moves from rural/isolated areas to urban/developed areas.
- **"Spillway"** means the components of a dam where water discharges from the dam basin before overtopping the non-overspill crest.
- **"Non-overspill crest"** means the top/horizontal section of an embankment, wall crest or structure.

2. INTRODUCTION

Civil infrastructure around the world is developing at an unprecedented rate with ever-increasing cities and metropolitans expanding to keep up with population- and economic growth. The result is more undisturbed, natural and pervious areas are being deforested to accommodate these developments. These natural pervious areas are therefore declining at a fast rate. How can we manage and ensure that the increased runoff from these new developed areas won't destroy the pre-developments' municipal stormwater infrastructure, and potentially cause severe flash flooding? Stormwater detention dams are but one method used worldwide to control floods.

Stormwater management plans and design guidelines do not yet exist for many smaller South African towns and Municipalities (*Drakenstein Municipality, Western cape, 2024*). Currently the stormwater requirements for residential developments follow the same principle that is used in the major

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metropolitan municipal by-laws such as the City of Cape Town, Tshwane and Johannesburg. Developments in rural areas differ tremendously from those constructed in large cities and so should the stormwater requirements. This paper focusses on resiliency-based stormwater design practices to attenuate and control the volume of the expected floods using detention dams or water-retaining structures.

3. STORMWATER ATTENUATION HISTORY

Studies show that designing to control the increased runoff from developed areas were never a need, nor seen as a potential flood risk, until the 1970's. By-laws in the United States and Europe were developed to regulate and control stormwater due to the increased effect that urban development had on the existing municipal drainage systems as explained in the "*Two decades of stormwater management evolution*" (Ben Urbonas, 1999).

South Africa soon followed suite after the Constitution (1996) and the National Water Law (1998) came into effect. Cape Town developed a stormwater management by-law in 2005 and in 2010 Johannesburg enforced their stormwater management by-law. Similarly, "*The South African Guidelines for Sustainable Drainage Systems*" (Water Research Commission (WRC), 2013) was published and in 2023 the WRC worked with several educational and engineering firms to develop a design guideline called "*A best practice for design flood estimation in Municipal areas in South Africa"* (WRC, 2023).

4. THE DILEMMA

4.1 Cause and effect

Newtons third law of motion states that for every action, there is an equal and opposite reaction. Similarly, for every new developed square meter the runoff increases (due to impervious / hard surfaces) relative to its pre-developed natural surface / state. Not only are digital technology, artificial intelligence and computer-aided software developing at an exceptional pace, towns and cities are also continually expanding their infrastructure, even after thousands of years. This infrastructure includes roads, housing, shopping malls, residential estates and schools, just to name a few. The effect is not only that the available area is being developed (deforesting natural vegetation and habitat), but these developments also contribute to global warming due to material manufacturing such as concrete. One of the results of global warming is the change in rainfall patterns and mean annual rainfall experienced in specific areas. In addition, climate change factors have only recently been included in flood hydrology design calculations as mentioned in *"The impact of climate change on flooding",* (CSIR, 2019).

The dilemma is that the cause is the effect and visa-versa; therefore, alternative methods and innovating design practices are required.

4.2 Degree of uncertainty

Flood hydrology and stormwater management remains a continual study field and a growing science. Associated designs are never exact and hence open for interpretation, manipulation and human error. Even though software (such as PCSWMM, Hydrology Studio, Autodesk software and HEC – RAS, just to name a few) for calculating flood peaks have been developed, there is still a large degree of uncertainty with flood hydrology and design requirements. It is therefore imperative that designers, scientists and consultants develop their own stormwater design judgement through collaborative experiences, training and lessons learnt.

The road to success is always under construction (Albert Einstein).

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5. CURRENT INFRASTRUCTURE & GUIDELINES

5.1 International

5.1.1 Georgia Stormwater Manual

The "Georgia S*tormwater Management Manual, Volume 2*" (2001) provides a detailed design approach, and engineering principles applied to stormwater management systems in urban environments.

In addition to conventional stormwater management practices such as detention and retention ponds, the manual discusses an Extended Detention (ED) method. This method provides an effective strategy for mitigating channel erosion and managing post-development runoff. The storage potential in, for example the wet ED ponds, micropool ED ponds and shallow ED marshes (wetlands) are used for water quality requirements such as settling of pollutants. According to the manual, two outlet structures are typically included in a storage facility design for any pond type, namely a principal outlet and a secondary or emergency spillway. Conventionally the emergency spillway is designed to only bypass the floodwater that exceeds the design capacity of the principal outlet and hence the volume of the pond. The potential downstream damage in the watercourse or infrastructure due to overflowing of the emergency spillway is considered and designed accordingly.

The design and principle of multiple storage ponds are described and depicts that the runoff could be increased or decreased based on different downstream locations. It is therefore proposed to perform a downstream peak-flow analysis to accurately design the ponds. A comprehensive design procedure is outlined within this manual to assist practitioners and designers in the accurate and consistent development of stormwater detention pond systems. The methodology proposes a step-by-step process covering hydrologic and hydraulic analysis, sizing criteria and outlet structure capacities.

Estimated storage volumes can be determined from triangular-shaped hydrographs as obtained with the Rational Method using the equation (2.2.6) in Figure 1 below from the "*Georgia Stormwater Management Manual",* (Jones et al, 2001).

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The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5T_I(Q_i - Q_O) \quad (2.2.6)$$

Where: V_S = storage volume estimate (ft^3)
Q_i = peak inflow rate (cfs)
Q_O = peak outflow rate (cfs)
T_i = duration of basin inflow (s)

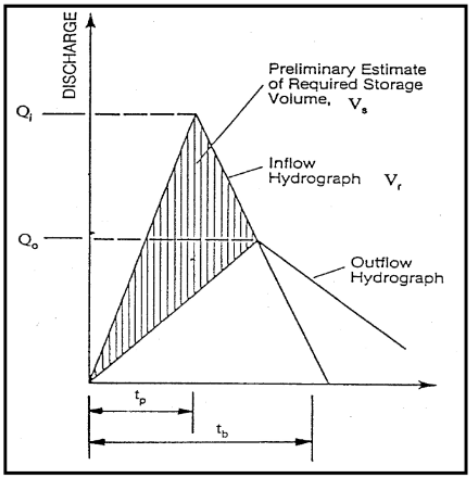

Figure 2.2.4-1 Triangular-Shaped Hydrographs (For Preliminary Estimate of Required Storage Volume)

Figure 1: Estimated storage volumes from triangulated hydrographs (Jones et al, 2001)

5.1.2 Sponge cities

Developed cities like Wuhan and Bishan-Ang Mo Kio Park in China and Singapore respectively have already adopted alternative resilient techniques to reduce the runoff for developed areas by creating a "Sponge city".

"Sponge cities" are defined as cities which are designed to absorb rainwater directly into the areas where it falls, thus reducing runoff and flood waters accumulating on hard surfaces. Examples of these techniques include permeable pavements, green roofs, bioswales and retention ponds/lakes.

The principles and techniques can be incorporated in South Africa, but given the current state of infrastructure, it is estimated that it would take a couple of years before sponge cities will be feasible in metropolitans and rural areas.

5.2 Local (South Africa)

Currently, there are two main focus areas in stormwater management plans and designs in South Africa, namely:

- Adequate water quality; and
- Flood protection downstream of the site by attenuation of the peak flows (quantity of water runoff).

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This paper focusses on the latter item above, being the management of the increased quantity of runoff off a developed site. According to the "*Road drainage manual*" (SANRAL, 2006), Section 3.3.3.1 urbanisation typically increases the runoff rate by 20-50% compared to natural conditions. The function of a detention dam or alternative water-retaining structure is therefore to attenuate this increase in runoff and release it over a longer period of time, reducing the risk of flooding downstream of the development.

Drakenstein Municipality rather recently developed a short guideline, and standard called the "*Engineering Services: Municipal standards*" (date unknown) under the Civil Engineering Services Department. This guideline only has a few design requirements with regards to the stormwater management plan:

- All detention/retention/irrigation dam overflows to be carefully designed with sufficient flood capacity downstream of the overflow, and so as to prevent erosion;
- No development/construction is allowed below the 1:50 yr flood level;
- For any development between the 1:50 and 1:100 yr flood levels there must be a disaster management plan;
- As far as possible stormwater runoff from erven must be able to discharge into the road, where this cannot happen the provision must be made for the drainage of such erven.

The details pertaining to the design of detention dams are not described in this guideline. Understanding what the municipality's stormwater management requirements are is an important factor in the design process. It was mentioned that the municipality is in the process to compile and publish their own stormwater by-law. One example as to why clarity is needed can be explained when the municipality requested that a detention dam must be constructed to retain or attenuate the complete or total 1:50 year flood for at least the 15-minute full storm duration. The volume calculation for the detention dam should account for the release of the 1:20 year pre-development without overtopping the "emergency" spillway.

"*The South African Guidelines for Sustainable Drainage Systems (SuDS)*" (University of Cape Town, 2013) states that "In an attempt to maintain pre-development conditions, stormwater runoff should be controlled and treated as close to its source as possible". Detention or retention ponds (called Regional SuDS controls) are referred to as the last line of defence for the management of stormwater before entering the municipal system or natural watercourse. The main objective of detention ponds is to control the post-development flood by discharging the calculated pre-development flood peak over an extended period of time. In other words, the existing municipal stormwater infrastructure must still be capable of handling the pre-development flood peaks even after post-development is completed.

Various computation design tools are available to assist in making the design process faster and more efficient as listed in Table 1 and Table 2 from SuDS, 2013.

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Table 1. Potential modelling tools available for design criteria computation

	Public education	Research	Developing Sizing rules for devicecs	Planning of land use in catchments/cities	Preliminary design of a subdivision or site	Preliminary design of a regional controls	Detailed design of regional drainage system	Detailed deisgn of subdivision or site	Site Layout and materials selection
MOUSE									
MUSIC									
P8									
SLAMM									
StormTac									
SWMM									
PCSWMM									
UVO									
WinDes (Quint. Only)									
Key					Model is suitable for use		Model is marginally suited for use		Model is not suited for use

Source: SuDS (2013)

Table 2. SuDS component capabilities for selected design models

	Impervious reduction	Ponds and wetlands	Soil protection	Reduction of contaminant generation	Infiltration trenches/bores	On-site detention tanks	Swales	Run on	Rain tanks	Bioretention, rain gardens filtration devices	Permeable paving	Green roofs
MOUSE												
MUSIC												
P8												
WinSLAMM												
StormTac												
SWMM												
UVO												
WinDes (Quint. Only)												
Key								Model is suitable for use		Model is marginally suited for use		Model is not suited for use

Source: SuDS (2013)

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The designers and practitioners should be well acquainted with the applicable program before using it for detail design purposes. It is also advised that designers incorporate sustainable and resiliency-based design practices as mentioned in Section 6.

In addition, typical considerations for designing detention ponds found in the SuDS includes:

- The local catchment hydraulics and hydrology;
- The possible use of floating islands;
- The implementation of appropriate safety structures, including disease vector controls;
- The prevention of dangerously steep ground slopes around the pond perimeter;
- The prevention of erosion at the inlet.

Additionally, resources such as the "*A best practice guideline for design flood estimation in municipal areas in south Africa"* (WRC, 2023) states a few resilient design guidelines and control measures. Table 3 stipulates the SuDS objectives, limitations and recommended flood return periods for different development types. Many smaller municipalities are adopting these requirements without the necessary technical background to apply the requirements to special and unique situations.

Table 3. City of Cape Town Criteria for SuDS objectives (City of Cape Town, 2009)

SUDS OBJECTIVES		Greenfield Developments *and* Brownfield and Existing Development Sites located in catchments of sensitive receiving water systems	Brownfield and Existing Development Sites > 50 000 m^2	Brownfield and Existing Development Sites 4000 m^2 – 50 000 m^2 *and* Total impervious area (exist & new) > 15% of site	Brownfield and Existing Development Sites < 4000 m^2 *and* Total impervious area (exist and new) > 600m^2
CONTROL QUANTITY AND RATE OF RUNOFF	*Protect the stability of downstream channels*	24 hour extended detention of the1-year RI, 24h storm event	24 hour extended detention of the1-year RI, 24h storm event	Combination of on-site and regional off-site measures to achieve requirements as for development sites >50 000m^2	On-site runoff control measures not required but encouraged where practicable Regional off-site runoff control measures to be provided to achieve requirements as for development sites > 50 000m^2
	Protect downstream properties from fairly frequent nuisance floods	Up to 10-year RI peak flow reduced to pre-development level	Up to 10-year RI peak flow reduced to pre-development level		
	Protect floodplain developments and floodplains from adverse impacts of extreme floods	Up to 50-year RI peak flow reduced to existing development levels. Evaluate the effects of the 100-year RI storm event on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts through detention controls and / or floodplain management	Up to 50-year RI peak flow reduced to existing development levels. Evaluate the effects of the 100-year RI storm event on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts through detention controls and / or floodplain management		
		Developments adjacent to floodplains must adhere to the requirements of the Floodplain and River Corridor Management Policy			

Source: WRC (2023)

Metropoles further use the "*Floodplain and River Corridor Management Policy"* (City of Cape Town, 2009) to classify hazard risks associated with flooding in the context of water flow depth and velocity. Figure 2 graphically produces a clear distinction between low and high-risk hazard for floods. This graph is used by competent and experienced consultants to determine flood lines and the type of developments allowed by municipalities.

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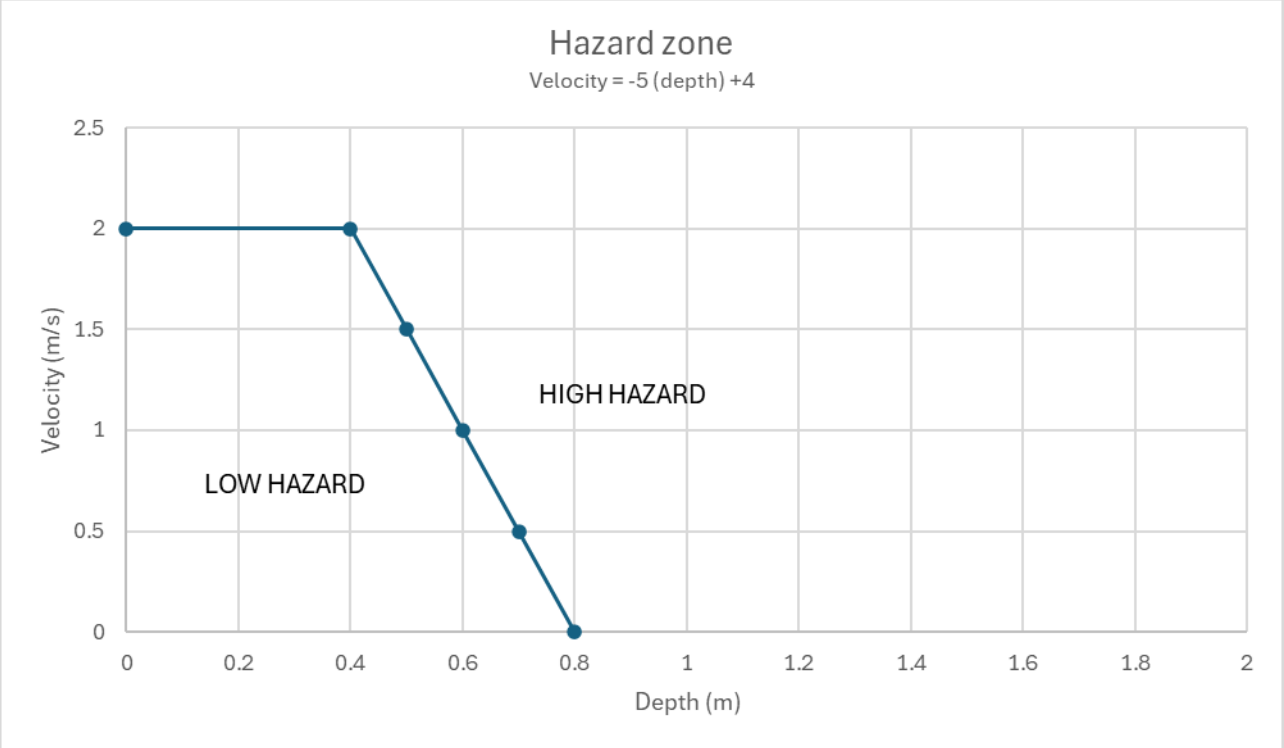

Figure 2. Flood Hazard Zones (City of Cape Town, 2009b)
Source: WRC (2023)

5.2.1 Environmental and safety aspects

SuDS (2013) states that three primary areas of concern with regards to detention ponds are transportation nodes, schools and informal dwelling areas. Examples of practical design considerations include gentler side slopes with specifically chosen vegetation growth to prevent slips and a "barrier" for pedestrians and emergency exit mechanisms for pedestrians to evacuate a pond etc.

One method of determining the flood hazard potential is by evaluating the flood lines downstream of the development. These flood lines can also be utilised to prepare emergency preparedness plans and early warning systems in a town of area inside the flood line.

6. RESILIENCY-BASED DESIGN FOR ATTENUATION DAMS

6.1 Philosophy

The existing methods (as presented above), research and technology is a stepping stone and solid foundation on which to build and refine resiliency-based design practices. Sponge cities and other innovative methods to control floods are also on the rise in South Africa and should be considered when designing the stormwater systems for new cities.

To date, attenuation dams/ponds for new residential developments states that the total (100%) volume of a 1:50 year flood must be contained for the entire storm duration (or at least 15-minutes) (*Drakenstein Municipality Civil Services Department, 2023*). To satisfy this requirement, large dams (likely also with a safety risk) would be required which requires adequate land and sufficient funds. Residential development has limited open areas and hence limited space for large dam structures and basins.

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The question therefore is, can attenuation ponds be designed for a percentage attenuated? For example, attenuate the 80th percentile of the required flood and not the total (100%) runoff volume of the post-development flood peak? Or differently expressed; design detention dams for smaller return periods, i.e. floods occur more frequently, but then retain 100% of the post-development runoff volume for at least a 15min storm duration without overflowing the "emergency" spillway. The 1:20 year pre-development flood is still released via an outlet pipe through the embankment or structure. The above philosophy and percentage attenuation question must further be examined and tested before it can be answered.

Furthermore, the downstream flood risk and hazard potential (both in the natural watercourse and directly into the municipal stormwater system) should be evaluated during the zero-freeboard flood (maximum spillway capacity) over the spillway versus the capital cost to construct a large dam. In other words, compare the consequence of flooding through the spillway (80th percentile attenuated) to the storage volume capacity of the detention dam required to retain the complete 1:50 year flood. The current understanding is that the spillway only works or overflows during an "emergency". This could include scenarios where the primary outlets are blocked, or when the flood exceeds the typical 1:50 year design peak flow rate.

It is therefore proposed that resiliency-based design practices should consider using the spillway as a secondary outlet (primary outlet being the pre-development releases), instead of only an "emergency" outlet, if the downstream hazard potential is low.

Further alternative resiliency-design approaches are proposed and discussed in the following chapters.

6.2 Alternative methods/approaches

6.2.1 Using existing dams

Cases where existing farms in rural areas are sub-divided into residential zones; the remainder of the farm continues with its farming activities and uses its existing instream dams, primarily for irrigation. In reality, these irrigation dams act as a "buffer mechanism" that routes the additional runoff caused by the developments upstream from it. SANRAL "*Road drainage manual*" (2006), Section 3.2 however, stipulates that "the flood peak is calculated using the entire catchment characteristics and contribution to the runoff at the outlet". This implies that the surface areas of upstream storage structures such as dams should not be subtracted from the local catchment area calculation. The effect is that the potential routing capability or "buffer effect" of the above-mentioned water retaining body is neglected in the flood peak calculation for the detention pond. This is a conservative approach, and dams have attenuation potential even when it is full (at FSL). The recommended safe rising water level inside the dam is restricted by the freeboard of the dam and the capacity of the spillway can be calculated at certain heights using hydraulic formulae.

It is therefore proposed that the potential flood attenuation of the upstream dams, and other water retaining bodies (within the same watercourse as the proposed detention dam) be incorporated in the peak inflow (Q_{in}) determination into the detention dam volume calculation. One quick method is using the peak discharge attenuation ratio formula ($Q_{out}/Q_{in} = 0.99\text{-}5.56\ A_r/A_c$) as stipulated in the "*Review of the* selection *of applicable flood capacity for dams in South Africa in the context of dam safety", 2007*. To obtain the Q_{in} for the individual upstream water bodies, it is recommended that the peak flood inflow at the detention dam be proportionally reduced by its own catchment area contribution to the larger detention dam's catchment area. For example, the detention dam has a catchment area of 1 km^2 and the calculated inflow into the detention dam is 10 m^3/s. There is an existing dam upstream of the detention dam with a catchment area of 0.5 km^2 which falls within the 1 km^2 of the detention dams' catchment area. Since the upstream water bodies' catchment is half (0.5) that of the detention dams' catchment, the expected peak flow is also half, ie 5 m^3/s, given that the catchment characteristics are similar.

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Alternatively, recommended software programs must be used to model the complete drainage system, including the upstream water body characteristics, therefore incorporating their routing potential. The resultant flood peak at the detention dam can then be used to calculate the required volume to retain the complete 100% return period before water is released into the municipal stormwater systems or natural watercourse.

6.2.2 Detention dams in series

In many scenarios, the available area on a rural development for detention ponds is restricted by municipal building lines and infrastructure. The need may therefore arise to construct and design detention dams in series in order to retain the desired flood. The required volume to retain the complete 100% of the desired flood is calculated by the sum of each dams' volume capacity. The topography will ultimately dictate whether two smaller embankments instead of one larger one will be feasible. Additional studies should be conducted to determine whether the dams in series will indeed pose a benefit in retaining the desired flood, as states in the "*Georgian Stormwater Management Manual", 2001*.

6.2.3 Multi-functional depressions

The use of depressions below natural ground level is an alternative to building an embankment above the ground. The principle is that a local depression with a relatively large surface area functions as the basin of a detention dam. During the dry months these areas are used for sporting grounds, parks and recreational activities such as soccer, hockey and picnics etc. But during the rainy seasons, the area is prohibited from the public, and the basin is filled by natural runoff and attenuates the incoming flood via canals inside a catchment area. The basin is sloped and shaped in such a way that water drains to the outlets and discharges at a much slower rate into the existing municipal systems.

In addition, the playing surface can be raised above the surrounding areas to ensure water drainage to desired low points, and it would also aid in clearing debris and avoid siltation on the playing field. The outlets should be well designed to prevent overtopping of the entire surrounding area. Safety considerations are critical when designing these types of detention ponds as potential drowning exists if no early-warning system exists or if individuals are unable to exit or evacuate the detention pond.

Disadvantages of these depressions are the growth of numerous pathogens such as Giardia Cryptosporidium which are common in faecal polluted water which is prone to detention ponds. Utilising local depressions as detention ponds presents minimal structural risk; however, the predominant issue lies in their potential impact on water quality due to limited filtration and increased pollutant accumulation.

Alternatively use floodplains and wetlands to attenuate the desired floods (larger flat surface areas results in smaller embankment heights and more evaporation) while ensuring an environmentally friendly habitat. Examples of these in the Western Cape include the Tygerberg waterfront and Driftsands dam attenuating floods in the Elsieskraal river and Kuilsrivier respectively.

6.2.4 Multi-level discharge structures

In addition to the "emergency spillway" which currently only overflows during flood events exceeding the design flood return period, a multi-level discharge structure situated on the upstream side of the dam can be designed to discharge different flood peaks, while still attenuating the incoming flow between outlets. One example of such a discharge structure is shown in Figure 3. This method will discharge flood waters at a much slower rate rather than having one primary outlet (pre-development releases) at the basin invert of the detention dam and one "emergency spillway" closer to the non-overspill crest. The potential risk to downstream infrastructure using this method is crucial since more water will be discharged into the municipal system or natural watercourse, but the advantage is that it is done at a much slower rate (time) reducing the risk of flash floods and backwater in the municipal stormwater system.

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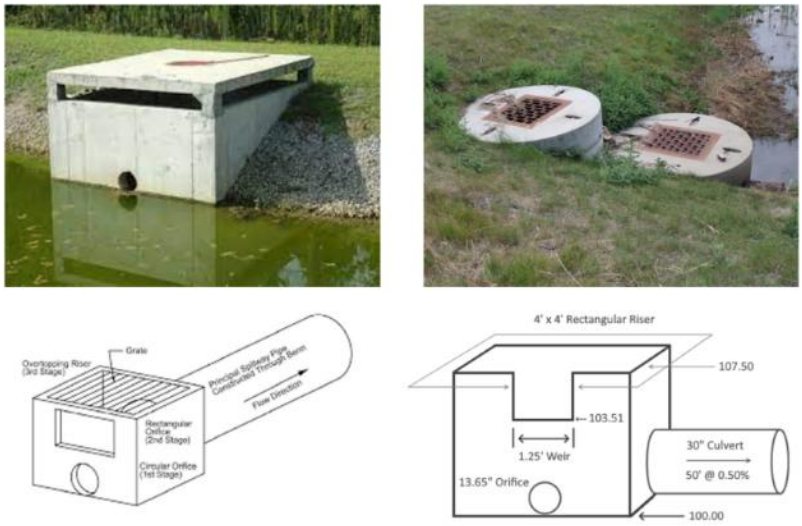

Figure 3. Example of a multi-level discharge structure (Krest Engineers: 2025)

7. CONCLUSIONS AND RECOMMENDATIONS

Flood hydrology and stormwater infrastructure is a growing civil engineering field and is continually developing. Over the years, it has been improved, refined and updated after numerous studies and research. This paper is only the tip of the iceberg in terms of resiliency-based design practices.

South Africa has had to re-look at how stormwater is managed and designed not only for new developments, but also its existing infrastructure. The recent floods (2025) in Kwa-Zulu Natal and the droughts in Cape Town (2017) highlighted the need for designers to adapt, improve and incorporate alternative stormwater techniques and methods. Global climate change factors are also considered a fundamental component to ensure the sustainable and resilient stormwater designs will be applicable for future flood peak requirements.

A few recommendations to consider for the next resilient detention dam design:

- Design the detention dam to attenuate smaller recurrence intervals and design the spillway system to discharge the routed residual of the 1:50 year flood, if the downstream flood hazard is low;
- Incorporate the attenuation potential of the existing water bodies upstream of the detention dam to accurately calculate the post-development design flood;
- Depending on the available topography, design detention dams in series to attenuate the desired flood before entering the municipal stormwater system or natural watercourse;
- Incorporate multi-functional depression areas in the stormwater management design which includes open spaces, parks and sports facilities to attenuate the desired flood;
- Design the outlets/spillways using multi-level discharge structures inside the detention dam.

A flood remains a natural occurrence that we could determine (fairly accurately) and then design applicable infrastructure to minimise the effects thereof. But controlling the uncontrollable remains a challenge and the need for resiliency-based design practices to create, develop and improve stormwater systems are evident in South Africa. The above alternative resiliency-based design approaches must further be examined and tested before it can be published in municipal by-laws.

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URBAN FLOOD RISK REDUCTION USING ATTENUATION DAMS

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PRESENTERS: MATT BRAUNE / MONICA BOTHA

ABSTRACT

The current urban environment is rapidly changing due to more high-density developments within municipal areas. Additional climate change and sporadic, more intense storm events as South Africa has experienced this recent rainy season has caused an increase in flooding problems and damage to property. This combined with financial constraints increases the pressure on municipalities to solve urban flooding problems in a more cost-effective manner. A project involving the remediation of flooding problems in a residential estate within the City of Tshwane has highlighted the benefits and cost savings achieved when considering flood attenuation in conjunction with Sustainable Urban Design Standards (SUDS). This approach included the use of an attenuation dam, diversion berms, as well as swales and natural floodplains thereby being able to mitigate the flood risk at an affordable cost.

This paper presents a case study which highlights the significant benefits of solving urban flooding problems using both flood attenuation dams and the SUDS principles. The paper also gives details on how the flood control measures were designed, constructed and how they performed during an extreme 1:100-year storm event that occurred during February 2022.

1. INTRODUCTION

It has been observed over the past few years and in particular the rainy season of 2021/22 that weather patterns have changed which cause more sporadic and more intense rainfall events within South Africa as well as other countries. In view of this stormwater drainage systems have become more important to drain excess stormwater and to prevent flooding and damage to property.

A shortcoming often encountered when planning urban developments is the lack of attention given to the drainage of stormwater once the development has been completed. A further shortcoming is defining upstream future urbanisation within the drainage catchment which causes an increase in stormwater runoff along both natural as well as artificial drainage systems. This in turn causes an increase in the flood levels and hence a higher flood risk.

The above shortcoming and lack of integrated stormwater management was the main cause of an increasing risk of flooding and frequent flood damage within the Zwavelkloof Private Estate causing major damage to private properties. To assist the Estate in reducing the flood risk an integrated stormwater master plan study was done to determine an economically viable and environmentally friendly solution. The approach adopted as well as the implementation of the flood control measures and operation thereof during recent extreme flood events is discussed and illustrated below.

2. INTEGRATED STORMWATER MASTER PLANNING

It is of utmost importance to first carry out an integrated stormwater master plan (SWMP). The main objective of a SWMP is to define the entire catchment draining into the study area, establishing the

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current and anticipated future development within the catchment and then quantifying the problem by compiling a hydrological model of the catchment to determine expected peak flow rates. A hydraulic study of the existing drainage network was then carried out to define the current capacity of the network. From the above information the extent of the problem as well as identification of remedial measures could be determined. Relevant information and results of the study are given below.

2.1 Study Area Locality

Zwavelkloof Private Estate is situated to the south of Saal Street, Olympus AH Gauteng as shown in Figure 1 below. The site is situated within a low naturally low-lying area which forms a major flow path during storm events.

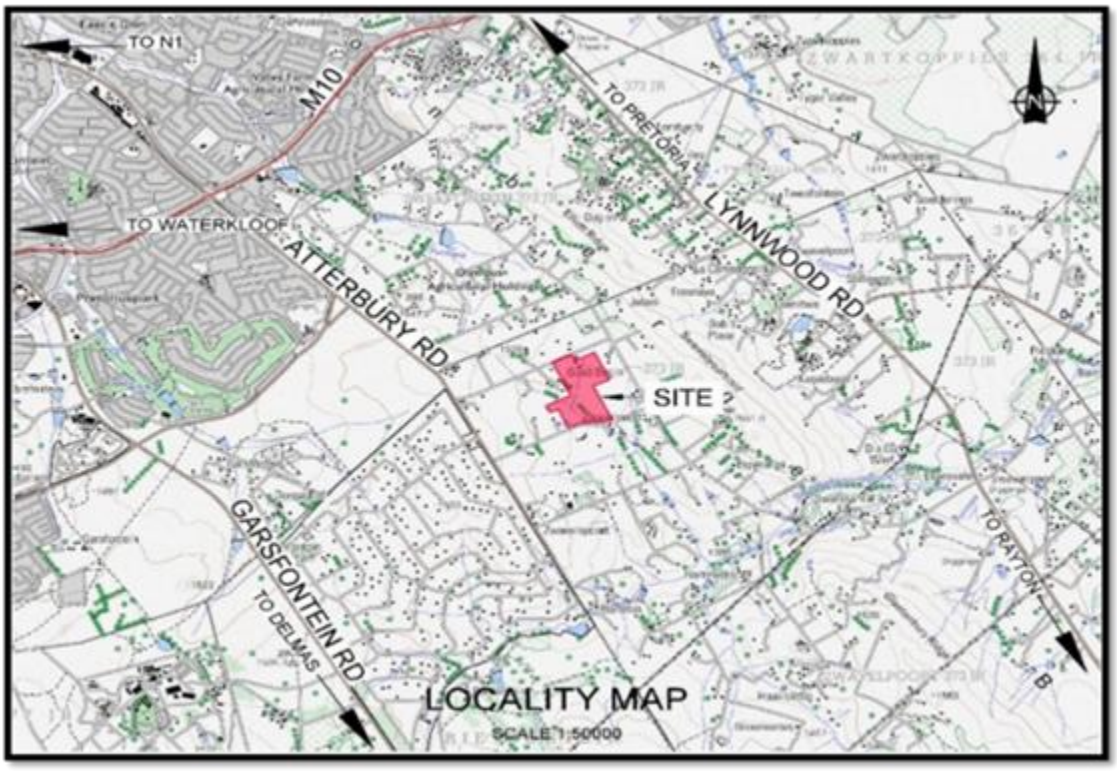

Figure 1. Locality of the Zwavelkloof Private Estate *(Swavelkloof Private Estate Stormwater Study, 2018*)

It was now important to define the entire upstream catchment as well as current and future planned land-use. This information was obtained from the City of Tshwane planning department and is shown in Figure 2 below.

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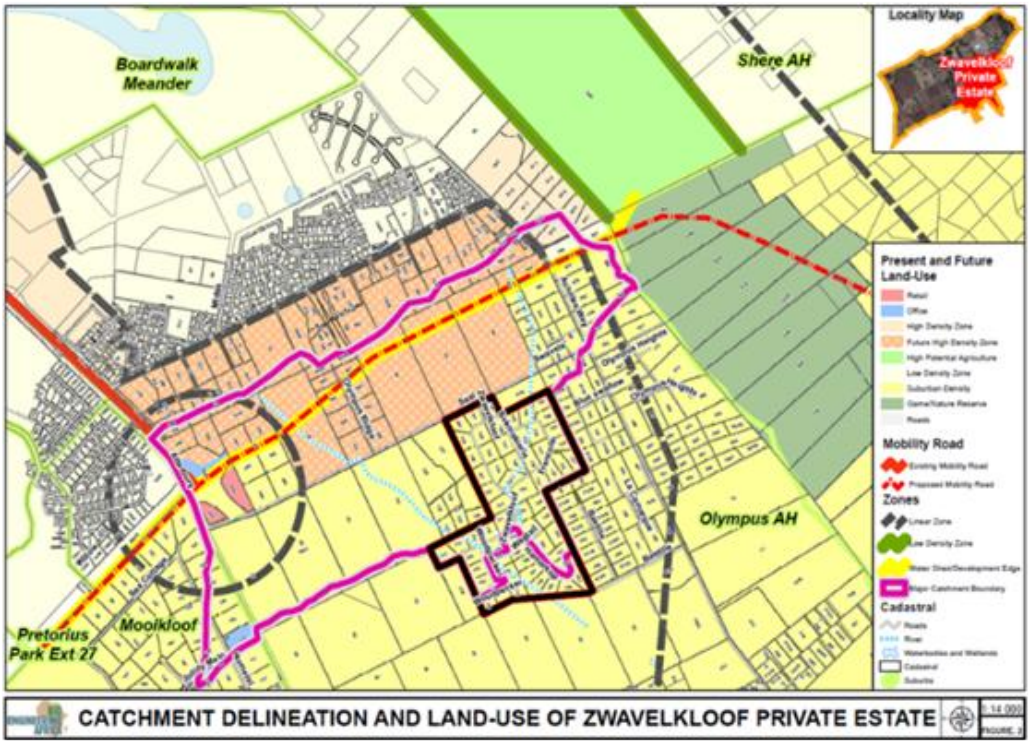

Figure 2. Catchment Delineation and Land-use (City of Tshwane Spatial Development Framework, 2020)

It is observed from the above figure that an upstream catchment of about 1.4 km^2 drains into the estate causing a significant stormwater runoff volume. It is also observed that about 65 % of the catchment would still be developed causing and even higher impact on the stormwater runoff peaks and volumes.

2.2 Hydrological Modelling

The PCSWMM Hydrological model was used for modelling of the catchment in order to obtain runoff peaks and volumes as selected node points. The schematic layout of a typical drainage network including road overflow and swale flows are shown on Figure 3 below and a brief summary of the model input parameters is given below.

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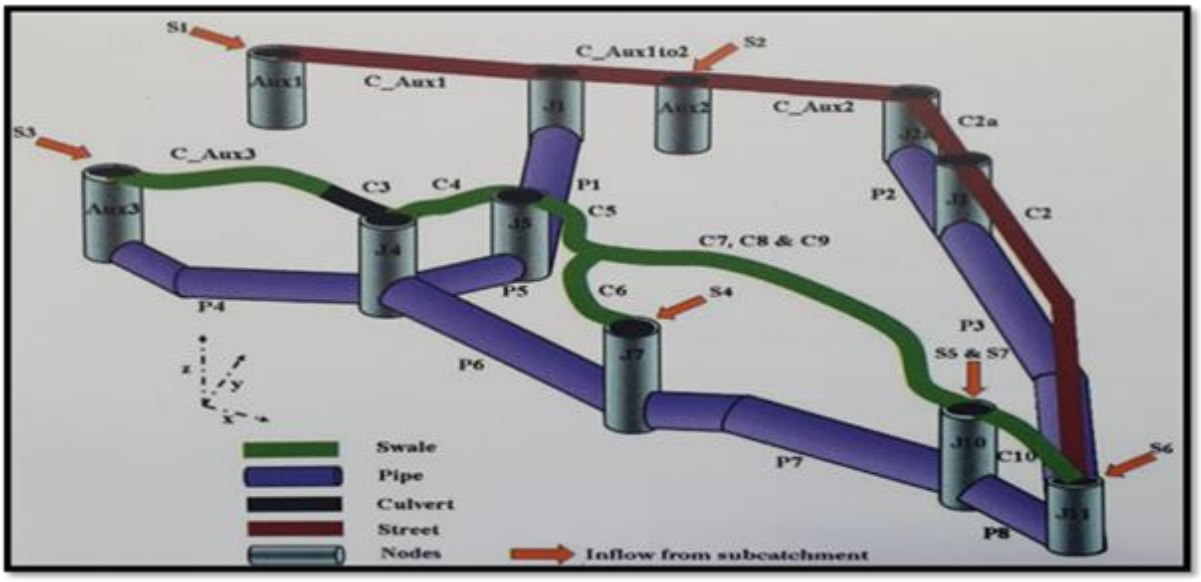

Figure 3. Typical PCSWMM hydrological model drainage network (PCSWMM version 7.1 Hydrological Model Guidelines, 2024)

2.2.1 Storm Rainfall

Relevant 24-hour storm rainfall for various return periods was obtained from the nearby South African Weather Bureau (SAWS) weather station. The 24-hour storm rainfall varied from 53 mm to 135 mm for a 1:2 year and 1:100-year storm event, respectively.

2.2.2 Existing Drainage Network and Sub-catchment Definition

Sub-catchments were now determined based on the layout of the existing developments as well as existing drainage network. The defined drainage network and associated sub-catchments are shown on Figure 4 below.

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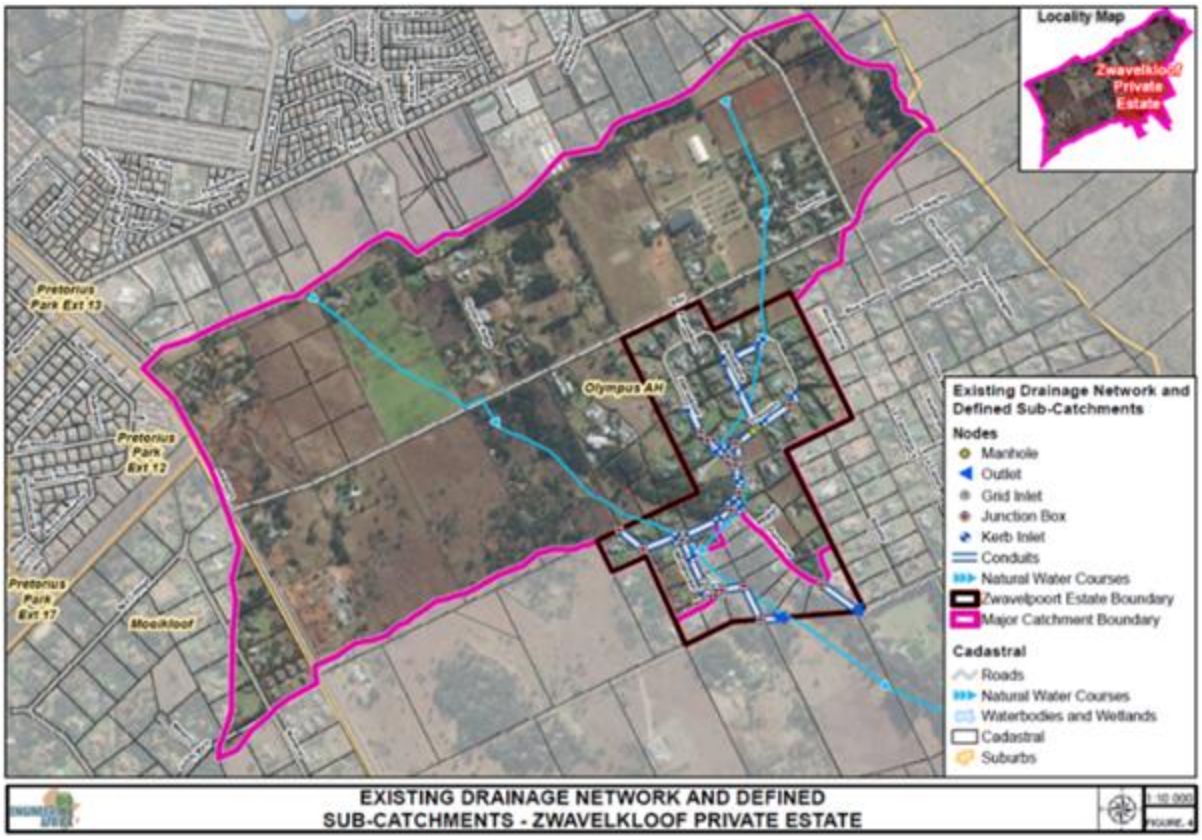

Figure 4. Existing Drainage Network and Watercourses (*Swavelkloof Private Estate Stormwater Study, 2018)*

2.2.3 Design Peak Determination and Design Standards

Before carrying out the hydrological modelling relevant design standards had to be determined. The design standards were based on the City of Tshwane guidelines for stormwater drainage systems as follows:

- The minor system defined as the underground pipe network and kerb /grid inlets must cater for at least a 1:5-year storm event.
- The major system defined as the minor drainage stem plus road overflow must cater for at least a 1:25 year storm event.

2.2.4 Existing Drainage System Assessment and Compliance

An assessment of the existing drainage network compliance was now carried out by comparing the design flows with the hydraulic capacity of the drainage network. The compliance of the network is shown graphically on Figure 5 below.

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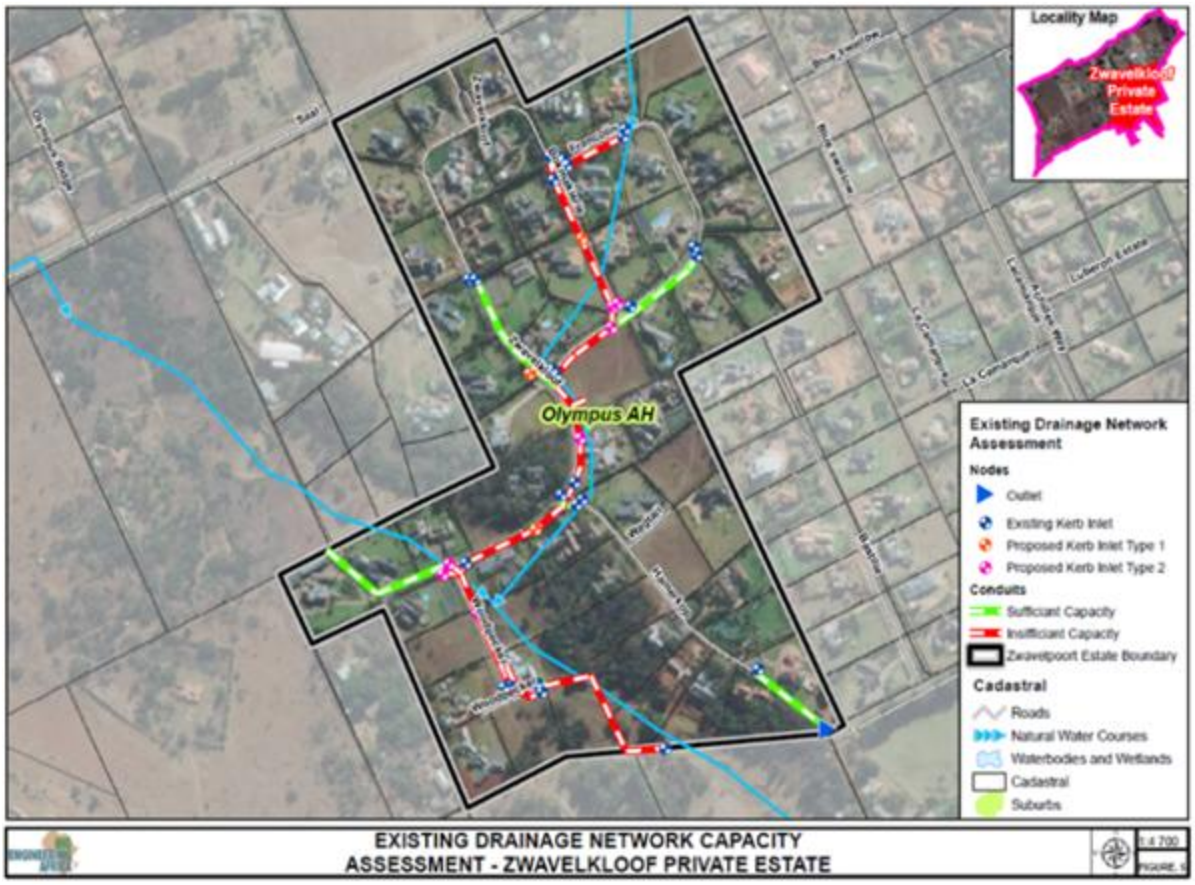

Figure 5. Drainage Pipe Network Capacity Assessment (Swavelkloof *Private Estate Stormwater Study, 2018)*

3. FLOOD REMEDIATION MEASURE ALTERNATIVES

Having now defined both the required design flows as well as shortcomings of the existing drainage network enables one to identify and design alternative remediation measures. The approach as well as selected remediation measures are discussed below.

3.1 Approach and Alternative Remedial Measures

Due to budget constraints for the capital works several remediation measure options needed to be considered in order to obtain both a practical as well as afforded remediation alternative. In view of the above as well as to satisfy the City of Tshwane the following remediation measures were considered:

- Alternative 1: Convectional upgrading of the existing pipe network by replacing the undersized pipes with larger dimeter pipes.
- Alternative 2: Implementing the Sustainable Urban Design Standards (SUDS) approach consisting of a combination of attenuation facility and upgrading a portion of the drainage system to prevent further flooding of the development.

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3.1.1 Alternative 1: Conventional Approach

This approach included a new design of all the undersized stormwater pipes as well as kerb inlets to handle at least a 5-year storm event. This approach would have ungraded a total length 1038 m of concrete stormwater pipes and implementing about 55.5m additional kerb inlets. The capital cost estimate was determined to be about **R 30 million.** This amount without the assistance from the City of Tshwane was impossible to fund by the Private Estate.

3.1.2 Alternative 2: Attenuation Dam and SUDS Approach

This approach included the possibility of using on site flood attention to reduce the peak flows entering the downstream undersized drainage network. An attenuation dam with a controlled bottom outlet and a side spillway was proposed.

The dam was classified in terms of the SANCOLD Dam safety guidelines (1990) as a Small size dam with a Low Hazard Potential rating. Considering the above classification the Recommended Design Discharge (RDD) should be between and 20–50-year flood event. Considering that the stormwater network should handle at least a 25-year storm event, a 25-year RDD was adopted.

Based on the SWMM Hydrological model the 1:25-year peak flow is reduced by about 40 % from 7 m^3/s to 4 m^3/s. The dam consists of an earth fill embankment as well as a small retention area with suitable wetland vegetation. The main dam design details are summarised in Table 1 below. The layout of the dam is shown on Figure 6 below. This cost for this alternative was **R 3,2 million** which is considerably less than Alternative 1.

Table 1. Summary of Attenuation Dam Design Parameters

Maximum Full Supply Volume	1500 m3
Dam Safety Evaluation flood (SEF) RMF$_{\Delta}$	15 m3/s
Recommended Design discharge (RDD)	7 m3/s
Spillway (depth x width)	0,75 m x 8 m
Dam embankment height	3,2 m

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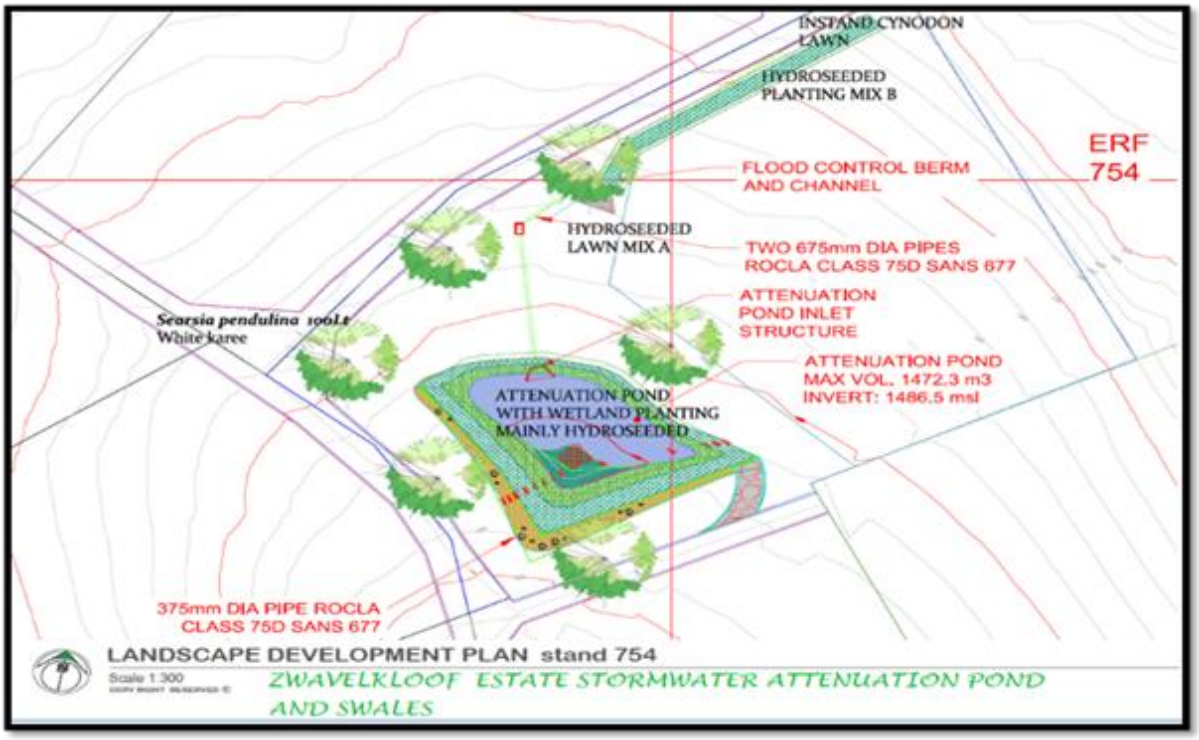

Figure 6. Attenuation dam layout (Swavelkloof Private Estate Stormwater Study, 2018)

Typical Design Details of the outlet chamber and the dam embankment are shown on Figure 7 and 8 below, respectively.

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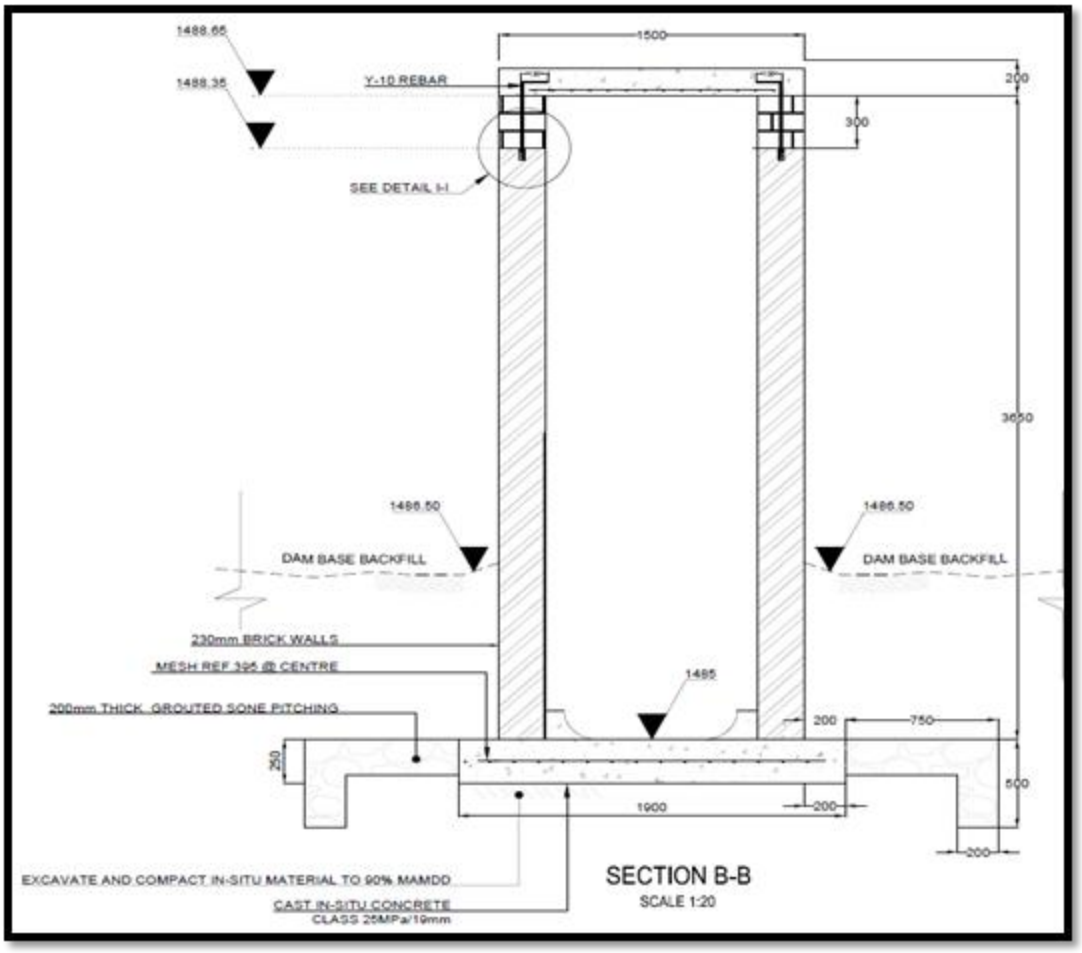

Figure 7. Attenuation dam-controlled outlet structure (Swavelkloof Private Estate Stormwater Design Report 884/001, 2020)

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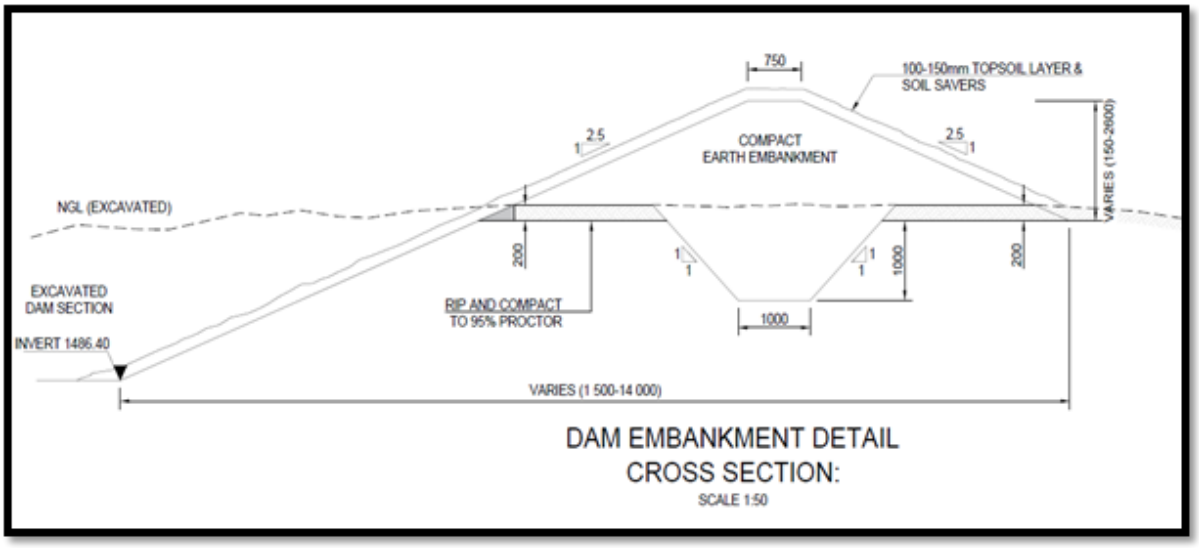

Figure 8. Attenuation dam earth embankment (Swavelkloof Private Estate Stormwater Design Report 884/001, 2020)

3.2 SUDS Approach and Additional Flood Control Measures

The SUDS approach is to minimise the directly connected impervious areas in an urban development and to implement natural and environmentally friendly control measures with the use of flood control derms and grassed waterways to reduce the energy of stormwater runoff. Even with the attenuation dam not all stormwater runoff downstream of the dam could be contained within the existing drainage network. Considering the above several additional environmentally friendly and natural flood remediation measures were implemented as described below.

The following additional control measures were therefore considered:

- Construction of a flood control swale and channel routing additional excess road flow into the attenuation dam as shown in Figure 6 above.
- Construction of side inflow flood control berms to route the stormwater into naturally low-lying areas thereby reducing the risk of flooding developments.

The designed and constructed diversion channel /Swale is shown in Figure 9 below.

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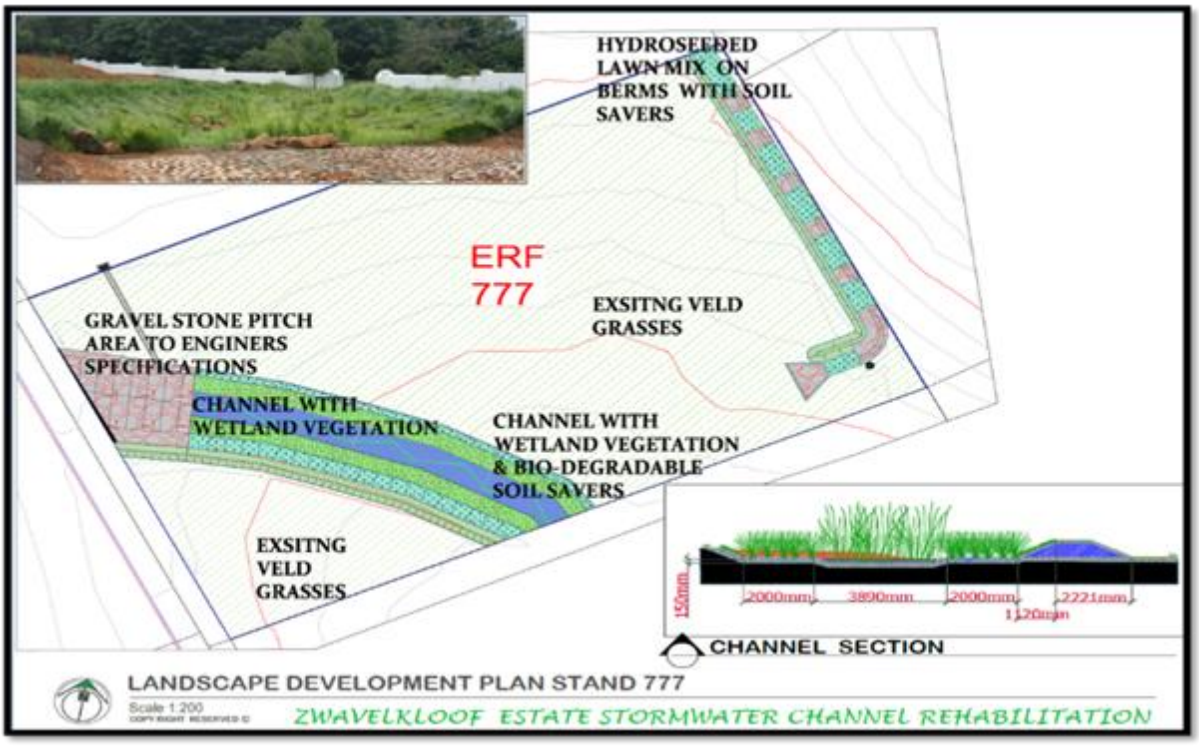

Figure 9. Constructed diversion Channel/Swale (Swavelkloof Private Estate Stormwater Study, 2018)

3.2.1 Hydraulic Design Aspects of Swales and Diversion Channels

When planning and designing naturally vegetated channels and swales it is of upmost importance to reduce flow velocities to threshold limits based on the type of vegetation cover. The following main aspects were considered:

- Reducing the flow velocity and energy by constructing natural rocks upstream of the swale.
- Where possible ensuring a subcritical flow with a Froude Number < 1,0 is obtained.
- Limiting flow regime changes from supercritical to Sub-critical flow which would create a hydraulic jump causing erosion.
- Maximising your wetted perimeter.

3.2.2 Landscaping and Vegetation Cover Design

An important aspect when design natural channels and/or swales is the correct and suitable erosion protection during the construction phase and prior to the establishment of the vegetation cover. The following main aspects and design considerations were taken into account:

- Utilising bio-degradable soil savers for erosion protection.
- Planning and designing suitable erosion protection covers and topsoil conditions that are both robust and sustainable.
- Planning and implementing irrigation systems to ensure the sustainability of the erosion protection measures.
- Implementing a maintenance plan for the upkeep of the erosion cover.

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- Planning and designing suitable erosion protection covers and topsoil conditions that are both robust and sustainable.
- Planning and implementing irrigation systems to ensure the sustainability of the erosion protection measures.
- Implementing a maintenance plan for the upkeep of the erosion cover.

3.2.4 Dam Embankment Vegetation Cover and Maintenance

The dam embankment cover consisted of a 200 mm topsoil layer with mainly Cynodon & Erogrotos Teff Mix. The bottom and wet area of the dam consists of a typical wetland mix for purifying the stormwater as well as silt control. In addition, an underground drip irrigation system was implemented to ensure a constant and suitable moisture condition.

3.2.5 Diversion Channel/Swales Vegetation Cover

The diversion/swale erosion cover consisted of a 200 mm topsoil layer with a Hydroseeding mix of Cynodon & Erogrotos combined with Eragrotis Gummiflua and Watsonia Borbonica.

4. THE UPGRADED DRAINAGE SYSTEM OPERATION AND RESILIENCE

The implemented upgraded drainage system has now been in place since 2022 and has survived two 100-year flood events with 145 -160 mm rainfall in 24 hours as well as one 50-Year flood event. A few pictures of the drainage system are given in Figure 10 and 11 below.

Figure 10. Attenuation dam at construction stage and after rainy season

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Figure 11. Stormwater diversion channel/swale

5. CONCLUSION

From this case study it can be concluded that making use of flood attenuation including a SUDS approach can significantly reduce capital expenditure by implementing a combination of environmentally friendly solutions at a much lower cost and therefore more acceptable to the community.

6. ACNOWLEGDEMENTS

The authors wish to acknowledge the opportunity given by the Zwavelkloof Homeowners Association to assist in making the estate a safer place to live in during storm events as well as the support and approval of the City of Tshwane Roads and stormwater division.

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FLOOD LINE MAPPING AND RISK ASSESSMENT IN THE RURAL EASTERN CAPE: A COMPARATIVE STUDY OF THREE MAJOR DAMS (MTHATHA, XILINXA AND NCORA DAMS) UNDER FAILURE SCENARIOS.

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2. Department of Water and Sanitation, Pretoria, South Africa

PRESENTER: AYANDA TSITSA

ABSTRACT

The impact of climate change has recently become a critical concern in the dam engineering sector, particularly in the context of dam safety. In South Africa, the recent erratic trends of rainfall are now compelling dam owners to consider adaptability and safety of communities downstream of dams. The observed impact following major rainfalls has highlighted the vulnerabilities of rural communities downstream of dams, some of which have developed informally over time, often within floodplains. This paper presents dam break inundation mapping under various breach scenarios of three major dams (Mthatha, Xilinxa, and Ncora) located in the rural catchment of the Eastern Cape. With the utilization of hydrodynamic modelling, the study compares the ***extent and impact of inundation*** as a result of the conservative 1:100-year flood and the more frequent 1:20-year and 1:10-year design floods. The analyses revealed that zones of vulnerability and extent of inundation compare marginally across the different scenarios. The study showed that a series of non-development zones should be identified in due time to counter for the shifting climate baselines and provision should be made of actionable insights for emergency planning.

1. INTRODUCTION

1.1 Background

Climate change has increasingly influenced hydrological patterns globally, resulting in altered rainfall distribution and intensity (Nhamo, 2019, Thoithi et al (2022). These changes have been reported to have significant implications for dam safety and flood risk management (Fluixá-Sanmartín, et al, (2018), Sayers et al (2013), Muyambo et al (2023), Sayers et al (2013)). Between 1959 and 2021, up to a total of 40 devastating flood events have been reported in the South Africa (Busayo et al, 2022; Thithi et al, (2022), Mudefi (2023). The effect has been evident, especially in rural areas where communities often develop informally along floodplains (Naido et al, 2022).

Whilst this is the case, urban areas have also been severely affected. The prevalence of flood related disaster events has underscored the need to re-examine disaster management frameworks within rural communities. Unlike metropolitan municipalities, rural municipalities do not have a strong drive for by-laws and policies that safeguard against development along floodplains. As a result, thereof, this highlights the need to incorporate flood data into existing rural developments to depict new inundation boundaries therefore managing potential local hazards and preparedness for major flood events.

"The South African National Water Act - Part 3, Section 144 (1998) supports this in that it requires that the public is notified about flood-related information and unsuitable areas for development with respect to flood risks to minimise the impacts and vulnerability". A key element in this regard is the determination

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of risk estimation in quantifying the impacts of flooding (ICOLD, 2005). The demarcation of the 100-year flood line is the preferred method of making people aware of the possibility of inundation of existing properties, whilst the 50-year flood line is good for use as a development control line (Alexander, 2000). The 10-year flood becomes important as well due to its conservatism which may depict moderate floods events that are constantly experienced. Whilst a sunny day dam break is typically preferred in most standards and global jurisdictions, for the purpose of this paper the 10-year and 20-year flood are considered as the most conservative to depict an event that would result from a frequent but moderate rainfall event.

1.2 Risk assessment

The evaluation of risk is conducted globally by dam owners and regulators using several methodologies available in literature. The main purpose of risk analysis is the transparent treatment of the uncertainty of the causes and consequences of dam failure (ANCOLD 2003b). ICOLD (1995) proposes a standards-based approach (SBA) reported to reduce failures by 0.5%. Whilst this is the case several other methodologies have been proposed in different jurisdictions, prioritising the risk related to what is considered as the most important impact metric (economic, environmental, socio-economic, loss of life etc). Oosthuizen et al (2000) provides a detailed methodology of the Level-0 risk assessment that is widely accepted across the dam safety community. Whilst the methodology is robust, based on the recent flooding incidents in both the Eastern Cape and KwaZulu-Natal provinces, this study presents a rural risk assessment that considers the interdependencies between two most relevant aspects for such communities - the *loss of life* and *social impact*, rather than the environmental impact and others emanating from a flood event. In this context, to address climate change influence on dam safety management, two impacts are analysed as a basis: population at risk and social impact.

1.3 Disaster Preparedness and Mitigation

Due to inadequate disaster preparedness, the disaster response during the April 2022 floods was inefficient and ineffective. The findings were justified by the loss of many lives as well as the extensive damage and destruction caused by the floods to property and infrastructure. Two sources of literature alluded to the following:

- A lack of robust disaster preparedness and mitigation strategies, with an overemphasis on response and recovery rather than proactive measures was identified post the KwaZulu-Natal Floods (Naido et al., 2022).
- Unplanned urbanization and inadequate infrastructure were significant contributors to the disaster's severity, as many homes were built on floodplains and wetlands (Mudefi, 2023).

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Figure 1. Development along the banks of Mthatha River (Algoa FM, 2025)

To model the downstream impact of flooding, dam practitioners often use hydraulic models to evaluate one and two-dimensional unsteady flow, and these have proven accurate predictions for resulting inundation (Wang et al., 2015; Sowiński, 2006). For this study, the same procedure will be followed for the assessment and comparison of the extent and impact of inundation.

2. STUDY AREA

The comparative study was conducted at three Category III dams in the Eastern Cape, located in Idutywa, Cofimvaba and Mthatha all within a radius of approximately 80 km from each other – see Figure 2. Ncora Dam is located along the Tsomo River, Xilinxa is located along a tributary of the Great Kei River whilst Mthatha Dam is located along the Mthatha River. Construction of the dams occurred between 1970-1977 and the infrastructure is currently owned and maintained by the Department of Water and Sanitation.

The study areas were chosen due to their importance within the rural municipalities in the Eastern Cape particularly for regional water supply and rural development– see Table 1 and Figure 2. Despite the proximity of the three dams, they each exhibited distinct hydrological behaviour influenced by the variability in catchment size, rainfall, and length of water course - Table 2. The most adverse consequences of flooding were considered to emanate from breaches primarily from overtopping and release of water into the outlet works system. For this study, due to the unavailability for a range of data related to the dams, the capacity, height of the dam wall, type of spillway, and downstream area were considered as the most crucial and guiding parameters for the prediction of the breach outflow.

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Table 1. Case study dams (Size class: Large, Hazard potential rating: High and Category III)

Study area	Type	Spillway type and width	Height	Capacity	Purpose
Xilinxa Dam	Zoned earth fill embankment	Concrete side channel and auxiliary spillway	24,5 m	14.84 million m^3	Amathole water scheme and industrial use
Mthatha Dam	Homogenous earth fill embankment	Concrete lined horseshoe shaped side channel with ogee crest spillway	38,0 m	254 million m^3	Supply to Eskom and domestic purposes
Ncora Dam	Concrete gravity	Uncontrolled ogee spillway	39,5 m	147.28 million m^3	Supply to the Ncora Flats Irrigation Scheme.

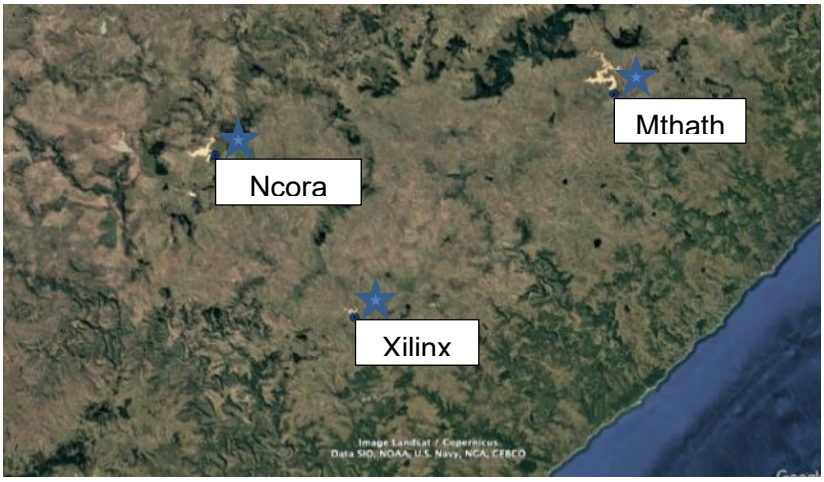

Figure 2. Locality map of the study area (Google Earth, 2025)

Table 2. Case study dams: Key hydrological characteristics

Characteristic	Xilinxa Dam	Mthatha Dam	Ncora Dam
Drainage region	S70C	TR001	S50E
Mean Annual Precipitation (mm)	668	1050	669
Effective catchment area (km^2)	205	886	1782
Longest watercourse length (km)	22.8	74	132
Time of concentration (h)	6.0	24	30
RMF Region & K_{RMF}	5.2	5.2	5.0
Regional Maximum Flood (m^3/s)	1861 (2022)	3760 (2010)	4221 (2020)

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3. OBJECTIVES

This study aims to address the following key objectives:

- Model dam break scenarios under various flood conditions (1:100, 1:20 and 1:10-year floods).
- Assess inundation extent and impact of downstream communities.
- Provide recommendations for zoning and emergency response aligned with climate adaptation.

4. METHODOLOGY

The methodology adopted for performing dam break analysis is shown in Figure 3 and the following subsections explain the steps followed in hydraulic modelling, model simulation, and floodplain mapping.

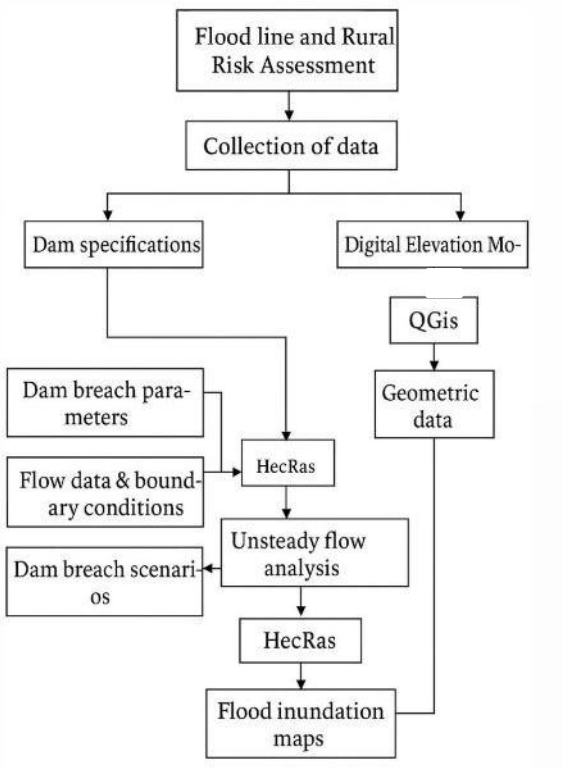

Figure 3. Flood line assessment and Rural Risk Assessment

- **Software:** The dam breach analysis models were simulated using HEC-Ras 2D analysis. Pre-processing of the elevation data of the respective dams was conducted on QGis software.
- **Digital elevation model and terrain**: Topographic data (10 m contours) used to map the floodplain and dam was sourced from CDNGI Geospatial Portal data sets, an open-source website by the National Geospatial Information division of the Department of Rural Development and Land Reform. QGis was used to process the data into a TIN (Triangulated Irregular Network) suitable for analysis in HEC-Ras. The contour maps were projected onto the Lo19 (Hartebeeshoek) projected coordinate system.

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- **Description of dams and Manning roughness parameters**: The dam registration information and as-built drawings were used to obtain dam specifications such at the height of the dam, length, and type of the spillway. Manning roughness parameters were estimated by comparing observed river conditions with the reference table in Chow (1959). The following were adopted for analysis:

Table 3. Manning's n for channels (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
Flood plains Cultivated area - no crop	0.020	0.030	0.040
Rock cuts jagged and irregular	0.035	0.040	0.050
Flood plains Pasture, no brush – short grass	0.025	0.030	0.035

- **Dam breach parameters**: Due to uncertainty with regards to the shape of the breach weir and the resulting weir coefficient, a conservative weir coefficient was determined from Chow (1959) (C_d = 1,4 for earth dams and C_d = 1,6 for concrete dams). Characteristics of the breach were determined following recommendations from SANCOLD (1990). The parameters shown on Table 4 were used in the model.

Table 4. Recommended breach characteristics (SANCOLD Guidelines on Dam Break Floods, 1990)

Parameter	Description	Recommended value – Gravity dams	Recommended value – Earthfill dams
B	Average breach width (m)	Some multiples of monolith widths, with B < 0,5 W	0.5 H < B < 3H
Z	Average side slope	0	0.2 < Z < 2
t	Time of failure (hrs)	0.2	0.5 < t < 3.0

Where W : crest length and H : ham height

- **Flood hydrograph:** The observed hydrograph of actual historic floods from the most recent hydrology were used. For dams that did not have this data, the hydrographs were estimated using the Standard Unit Hydrograph (SUH) at $2t_c$ assumed to closely fit the flood peaks and volumes of previous flood events. The dams were conservatively assumed to be at Full Supply Level (FSL) when the breach occurs. It was assumed that the breach of each dam would occur when the maximum inflow reaches the dam wall.
- **Unsteady Flow Analysis Parameters**: In all scenarios, a steady state warm-up period was run before the start of the unsteady flow analysis, to fill up the respective dam to full supply level prior to routing the observed inflow hydrograph. All analyses were run at a computational interval of 30 seconds. Detailed output (hydrographs and inundation maps) was generated at 5-minute intervals to maintain stability of the computations. The analysis was run, for a period corresponding to the time of concentration (t_c) of the observed hydrographs.
- **Flood inundation maps**: Results from the analysis were generated on HEC-Ras and exported to a shape file and .kml formats for visualisation and analysis in Google Earth software.

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5. DISCUSSION AND RESULTS

5.1 Hydrology

An important boundary condition in inundation mapping is the determination of the inflow hydrograph, which describes the discharge routed through a dam during a flood event. This study considered, the recommended flood peaks for the 1:100, 1:20, and 1:10-year return periods. These were developed by scaling the observed hydrographs proportionally to the recommended peak discharges. The 1:100-year is widely adopted in dam safety practice as a credible design flood for assessing the potential consequences of a flood event (DWAF, 2009; FEMA, 2013; SANCOLD, 2011, ICOLD, 2018; DWS, 2010). Analysis of the hydrological data of the three dams, indicated that the adoption of the 1:100-year flood as described in literature would result in more significant inundation extents based on peak flows compared to the observed inflow hydrographs and lower recurrence flood intervals – see Figure 4 to Figure 6. As a result, the 1:100-year flood was used as a benchmark flood for the inundation extents.

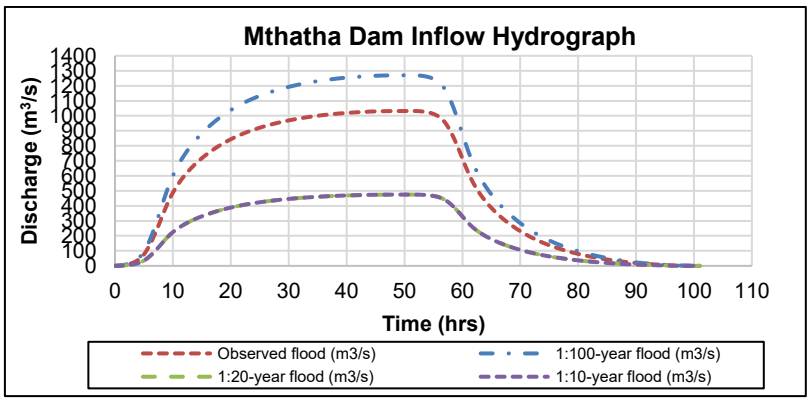

Figure 4. Mthatha Dam Inflow hydrographs

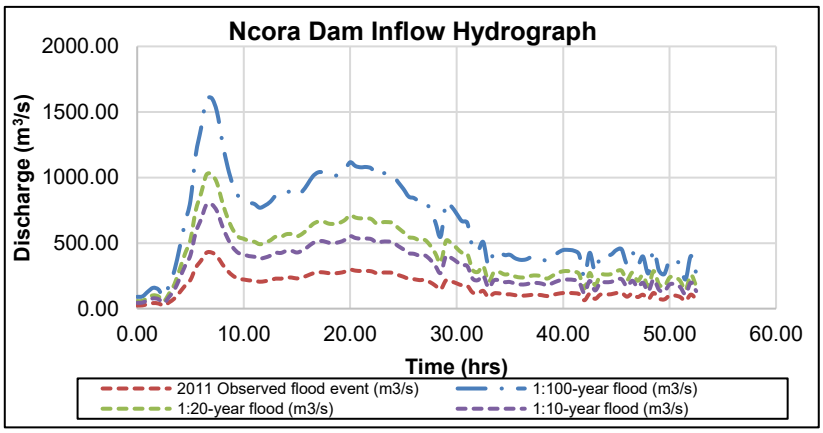

Figure 5. Ncora Dam Inflow hydrographs

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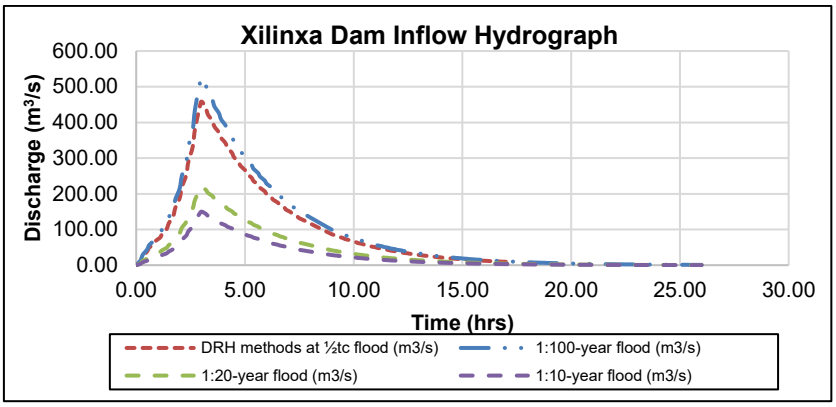

Figure 6. Xilinxa Dam Inflow hydrographs

5.2 Flood inundation mapping

A HEC-Ras 6.6 two-dimensional hydrodynamic numerical model based on the momentum equation was performed for all three scenarios using as built information from available sources. The models were discretised into flow areas with 5 x 5 grid cells compared to 10 m contour intervals. Downstream surface conditions were represented by varying the Manning's roughness coefficient (n) as described in Table 3. To develop robust models, the topographic survey data was analysed to define accurate terrain and boundary characteristics (length of spillway, reduced level of FSL, Non Overspill Crest (NOC) level etc). The dam walls were modelled as weir structures with geometric properties closely matching as-built configuration. The calibrated models were then used to simulate unsteady flow analysis and compare the hydraulic response of the dams under the 1:10, 1:20 and 1:100-year flood scenarios to evaluate flow behaviour, inundation extent, and downstream impacts.

5.2.1 Mthatha Dam

Analysis of the Mthatha Dam inundation extents demonstrated under all three flood scenarios, significant impact of the events would occur primarily along the Mthatha River flood plains. This would result in the inundation of Mt Pleasant, extending into the built-up areas of the town. The surrounding terrain along the river line. It was also observed that unlike the two other dams, the importance of identifying non development zones relative to the different flood event would result in minimal comparison. This was attributed to the fact that the 1:100-year flood line was not regulated by local authority during development thus increasing the exposure to flood hazards regardless of the scenario – see Figure 7**.** Even with the delineation of the 1:20, 1:10 year inundation boundaries, the vulnerability the would be less variable. An important observation was that priority in this instance would have to be a balance between emergency response mechanisms and flood-based allocation of future development. The population at risk was estimated to be more than 5 000 people with a potential loss of life of more than 100 people. Compared to other rural dams. More infrastructure damage would result in the excess of R 55 000 000.

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Figure 7. Mthatha Dam - Flood inundation mapping

5.2.2 Xilinxa Dam

Inundation mapping results showed that for all three flood scenarios, DWS abstraction works and the Sihlabeni-eSantini Bridge would be inundated. The surrounding terrain of the dam is moderately elevated between 817 – 771 RL, limiting lateral spread of flood waters to adjacent villages located at approximately 881 RL compared to 771 RL along the river line. For Xilinxa Dam the incremental difference in extents from the 1:10-year (220 m^3/s) to the 1:20-year (300 m^3/s) flood is 19 m, while the increase from 1:20-year to 1:100-year flood is 33 m – see Figure 8 A. In the event of the occurrence of either of the three flood scenarios, inundation of arable land or land earmarked for development would not be at risk – see Figure 8 B.

From the analysis of the maps, only areas earmarked for farming around KwaMlonyeni village would be at risk while those up to Qima would be outside of the boundary. The population at risk is estimated at between 1000-2000 people, with an expected loss of life exceeding 5 people. Even though infrastructure damage would be limited, disruption of the Department of Water and Sanitation (DWS) abstraction infrastructure would affect domestic and agricultural water supply, resulting in temporary service delivery and quantifiable economic impacts.

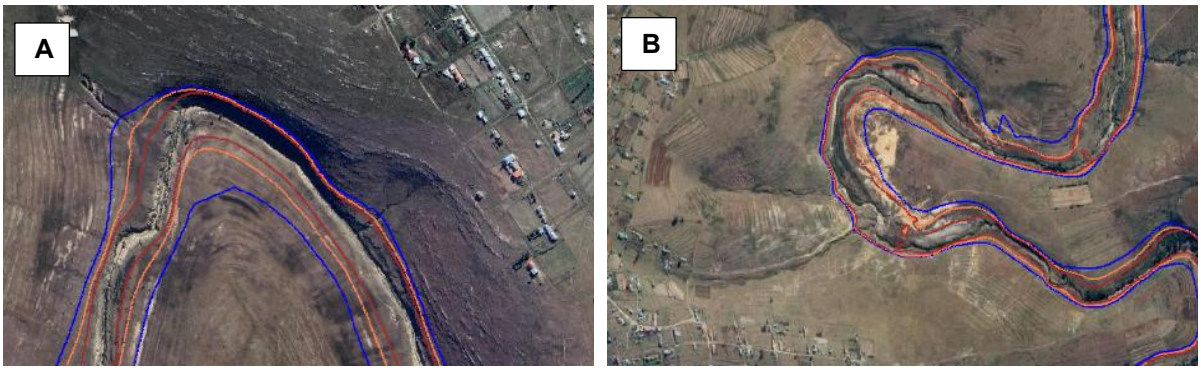

Figure 8. Xilinxa Dam - Flood inundation mapping

5.2.3 Ncora Dam

The downstream flood plain of Ncora Dam showed variable topography varying between RL 1058-1000 m, with a more complex morphology with steep elevations gradients as also observed along Xilinxa River banks. Along the inundation boundary of Ncora dam, no immediate settlement was located and in this scenario no high risk development areas were identified. The nearest village to the dam wall,

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kuGongqo is located at a difference in height of approximately 100 m from the river line, outside of these inundation extents – see Figure 8 A. The 1:20-year flood peak (795 m^3/s) was approximately 20% higher than the 1:10-year flood peak (1 025 m^3/s). The 1:20 year flood extent compared to the 1:10 year flood, suggesting a relatively marginal increase in the downstream flood plan with both boundaries coinciding with each other. In contrast, the 1:100 year flood inundation boundary demonstrated a 40 m average increase in width from the 1:20 year flood inundation boundary demonstrating a more pronounced difference.

A notable observation was that a bridge dividing the villages which serves as a critical emergency response and transport route would be inundated – see Figure 8 B. This suggested that for Ncora Dam, emergency response should be prioritised contrary to the relocation of future development zones. The population at risk is estimated also between 1 000 - 2 000 people. Although the immediate population exposure is low, the inundation footprint would compromise the Ncora Irrigation Scheme, which relies on inter catchment transfer for the irrigation of approximately 300 ha. The associated loss of water conveyance capacity at the dam would lead to measurable economic and operational impacts, especially considering that Ncora Dam forms part of a hydropower cascade system utilised at both the Ncora and Collywobbles Eskom Power Stations.

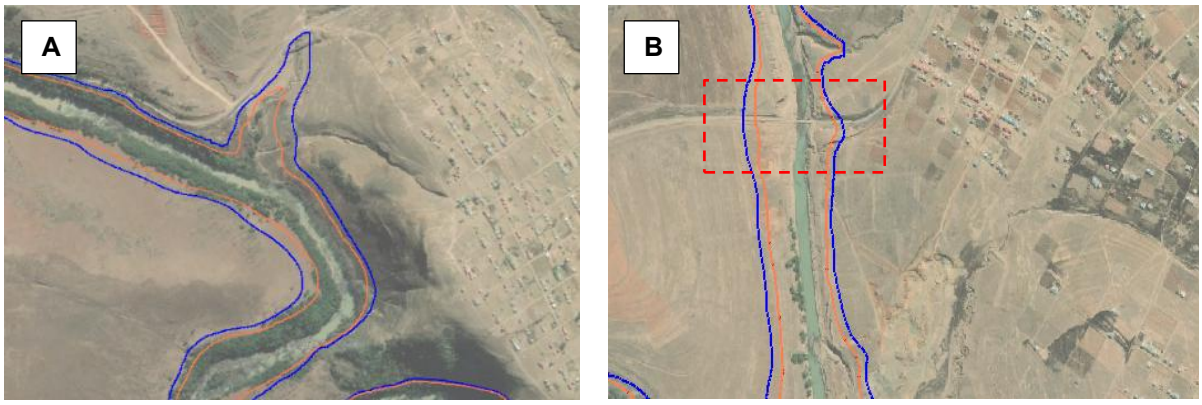

Figure 9. Ncora Dam - Flood inundation Mapping

6. CONCLUSION

Climate change-induced variability in rainfall necessitates a shift in dam safety and flood risk management strategies in the Eastern Cape. This study highlights the vulnerability of downstream rural communities and emphasizes the need for emergency planning to enhance resilience, compared to the identification of adaptive development zones except in the case of the Mthatha Dam scenario.

The comparative analysis of the flood inundation boundaries across the three study dams demonstrated that the 1:100-year flood provides a conservative estimate for the determination of a higher potential hazard for heavily developed urban areas. In contrast, rural assessments showed a limited variation in the effects of inundation extent between the different flood frequencies. This suggests that for rural catchments such as that of Ncora Dam and Xilinxa Dam, characterised by stable topography and low development densities, lower frequency floods based on observed flood hydrographs can be sufficiently used as a basis for defining evacuation routes, emergency response zones and land use planning.

However, Mthatha Dam provided a contrasting scenario due to its historical encroachment of the 1:100-year flood line as promulgated by the National Water Act. The results showed that regardless of the contributing flood scenario, the relatively flat terrains in analysis underscores the urgency for proactive

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zoning and planning. These areas require more robust inundation mapping since the areas provides no buffer for direct exposure to flooding.

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SUMMARY OF THE FINDINGS FROM THE STUDY ON ENHANCING SPILLWAY EFFICIENCY DISCHARGE COEFFICIENTS: INSIGHTS FROM HISTORICAL MODELS AND CASE STUDIES OF OGEE SPILLWAYS

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ABSTRACT

This paper summaries the initial finding from the study that hypothesises that consistent, quantifiable trends can be identified from historical model studies and prototype data, reflecting how specific non-ideal conditions impact spillway performance. These trends can be incorporated into modified theoretical formulations, such as correction factors or adjustment parameters, to improve the accuracy of spillway discharge estimations for diverse configurations and site-specific conditions. This paper aims at highlighting the limitations of the spillway hydraulic theory and showcasing the deviations from practice.

1. INTRODUCTION

The design of hydraulic structures has benefited significantly from technological advances over the past two decades. The widespread availability and growth of Computational Fluid Dynamics (CFD) software, coupled with increased computing power and cloud-based resources, have revolutionised the analysis potential for engineers. Furthermore, the advent of Cloud server computing has further democratised access to such analysis. Nevertheless, during the early stages of dam and spillway design, such advanced tools are often impractical due to time and cost constraints. Preliminary design decisions therefore continue to rely heavily on empirical equations and hand-based calculations.

1.1 Problem Statement

Early spillway design relies on idealised discharge equations, but South African model/prototype evidence shows systematic deviations under non-ideal conditions (approach angle, offset, pool depth, geometry). These errors propagate into freeboard, cost, and safety decisions. No comprehensive framework exists locally to quantify and correct these deviations across spillway types. This gap motivates the current research.

1.2 Aim and Research Objectives

The research aims to analyse historical South African physical model studies and prototype performance data spanning the past four decades, in order to identify key trends that influence the hydraulic efficiency of spillways. By comparing theoretical discharge predictions with empirical findings, this research seeks to reduce the current discrepancy between calculated and observed spillway performance, thereby improving design accuracy. The ultimate goal is to derive practical correction factors that can enhance the reliability of flow calculations, and to develop concise design guidelines that improve hydraulic calibration practices in future spillway designs. This paper however, aims at highlighting the limitations of the theory and showcasing the deviations from practice.

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1.3 Hypothesis

Traditional hydraulic design methods in dam engineering primarily rely on two-dimensional flow assumptions and idealised geometries. However, real-world conditions, especially the complex three-dimensional flow dynamics in natural approach basins, often lead to deviations from these simplified theoretical predictions.

1.4 Scope and Limitations

This study hypothesises that consistent, quantifiable trends can be identified from historical model studies and prototype data, reflecting how specific non-ideal conditions impact spillway performance. These trends can be incorporated into modified theoretical formulations, such as correction factors or adjustment parameters, to improve the accuracy of spillway discharge estimations for diverse configurations and site-specific conditions.

This study focuses on the hydraulic efficiency of spillways by examining historical physical model studies and prototype data from South African dams over the past four decades. The analysis is limited to publicly accessible reports and documented studies, primarily from the Department of Water and Sanitation (DWS), academia, and relevant consulting engineering firms. The study considers both standard and non-standard spillway types, including ogee, broad-crested, labyrinth, and Piano Key configurations, across various dam types (e.g., earthfill, rockfill, concrete gravity, and arch dams).

The scope includes an investigation of key hydraulic variables that influence spillway performance, such as approach flow angle, pool depth, spillway location on the dam wall, and design flood magnitude (Recommended Design Flood and Safety Evaluation Flood).

Limitations of the study include the following:

- Data Availability: The analysis is restricted to historical reports that were obtainable at the time of research. Some model studies may lack complete data sets or sufficient metadata on physical model scaling and prototype calibration.
- Model Consistency: Variation in modelling standards, documentation quality, and spillway definitions across decades may introduce inconsistencies in the data.
- Generalisability: While the goal is to identify overarching trends, some site-specific findings may not be broadly applicable without context-specific adjustment.

1.5 Research Methodology

The greater research follows a structured methodology comprising the following key phases:

- Literature Review.
- Data Collection and Compilation.
- Parameter Identification and Classification.
- Trend Analysis and Interpretation (highlighted in this paper).
- Development of Correction Factors (Not discussed in this paper).
- Validation and Recommendations (Not discussed in this paper).

1.6 Literature Review

1.7 Introduction to the literature

Standard theoretical formulations (e.g., USBR, USACE) provide baseline curves for idealised ogee and sharp-crested spillways, but empirical evidence from physical model and prototype studies often reports systematic deviations under non-ideal conditions. These deviations, linked to factors such as approach

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flow angle, pool depth ratio, and spillway offset, are well-documented in international datasets and South African model studies. Yet, no integrated framework currently consolidates these findings.

1.8 Spillway Classification

Spillway hydraulics are typically categorised into three main types of spillways namely linear, lateral and non-linear spillways. This classification reflects the distribution of flow along the crest and the hydraulic principles governing discharge.

Linear spillways are typically aligned perpendicular to the flow on the dam wall, with the assumption that the flow is distributed uniformly across the spillway crest. Thus, there is a linear relationship between the unit flow and length of the spilling edge. Linear spillways come in many forms and even allow for curved edges as long as there is an assumed equal distribution of flow over the spillway or rather a negligible variation. The ogee spillway is the most common form, shaped to match the natural trajectory of the nappe under design head conditions. Other examples of linear spillways are sharp or broad crested weirs, although these are less common in larger dams where ogee spillways dominate due to stability and efficiency.

Lateral spillways are oriented to guide flow horizontally before it is discharged downstream, producing spatially varied flow over the crest of the spillway. The unit flow changes over the length of the spillway and therefore requires a more fundamental hydraulic analysis as opposed to the standard weir relationship. Their efficiency typically decreases at high unit flows as a result of the submergence effects in the bucket.

Non-linear spillways are designed to increase the crest length within a limited width by folding the crest planform into a labyrinth or piano-key configuration. The non-linearity is derived from the fact that the unit flow is not uniformly distributed across the length of the spillway and is dependent on the inlet and outlet conditions because of the unique cross-flow interactions caused by the structure.

1.9 Flow Principles and Discharge Coefficients

In open channel hydraulics, a control refers to a section of the flow where conditions can be uniquely related to upstream head and discharge, i.e., a Q-h relationship exists. Controls may be natural (channel controls) or artificial (structural controls). Channel controls occur in long, stable reaches where geometry and slope maintain near-normal flow, unaffected by downstream conditions.

Spillways therefore serve as structural controls. They are designed to force the flow through the critical state (critical flow depth, y_c), defined by a Froude number equal to unity, which ensures a consistent relationship between upstream head and discharge

The standard sharp-crested weir equation, widely presented in classical hydraulics, provides the foundation for spillway hydraulics (Ven te Chow, 1959):

$$Q = CLH^{\frac{3}{2}} \qquad \text{Equation 1}$$

where: C = Discharge coefficient of the spillway,
L = Length of the spillway (m), and
H = Potential energy head upstream of the weir, with crest level as datum (m).

The derivation of this relationship is based on simplifying assumptions, including a constant horizontal velocity component and an idealised nappe profile. The ogee spillway shape was later developed to replicate this nappe trajectory under design head conditions. Early empirical work illustrated how deviations from these assumptions influence performance. Rehbok (1929) proposed that the discharge coefficient should vary with relative head:

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$$C = 3.27 + 0.40\left(\frac{H}{P_s}\right) \qquad \text{Equation 2}$$

where P_s is the weir height (m). Rouse (1936) investigated the performance of the discharge coefficient equations and found that they describe weir behaviour well for relative head ratios of $H/P_s < 10$. For cases with $H/P_s > 15$, representing shallow approach pools relative to high flow levels, he recommended the alternative expression:

$$C = 5.68\left(1 + \frac{H}{P_s}\right)^{1.5} \qquad \text{Equation 3}$$

Although Equations 5 and 6 were originally developed for weirs rather than large spillway structures, they clearly demonstrate the influence of pool depth (H/P_s) and upstream face geometry on the discharge coefficient.

Another important parameter is the effective crest length because it directly scales discharge capacity in Equation 4. If contraction effects are not properly accounted for, changes in discharge may be incorrectly attributed to the discharge coefficient (C_d) rather than to geometry. Accurate estimation of the effective crest length (L') is therefore essential to distinguish between genuine hydraulic efficiency effects and purely geometric reductions in flow capacity. Ven te Chow (1959) expressed pier contraction effects as:

$$L' = L - KnH \qquad \text{Equation 4}$$

where: L' = Effective length (m),
L = Actual length of the structure, without physical widths of pier (m),
k = Contraction coefficient as a function of H and pier shape,
n = Number of contractions.

Ven te Chow (1959) assumes a contraction coefficient of 0.1 in Equation 4, thus assuming a uniform reduction factor, regardless of pier geometry.

The United States Bureau of Reclamation (USBR) (1987) refined the effective length equation by introducing geometry-specific contraction coefficients:

$$L' = L - 2(nK_p + K_a)H_e \qquad \text{Equation 5}$$

where: Kp = pier contraction coefficient,
Ka = abutment contraction coefficient,
He = Actual energy head above the spillway, inclusive of the velocity head component (m).

The USBR guidelines (1987) refine the effective crest length calculation by introducing shape-dependent contraction coefficients for piers (K_p) and abutments (K_a). These coefficients are account for the geometry of structural elements and their effect on flow contraction.

1.10 Ogee Spillway hydraulics

The ogee spillway equation is a direct adaption of the standard weir theory equation (Equation 1), with the total height defined to include the velocity component.

This formulation of the equation is consistently applied in Ven te Chow (1959), USBR (1987) and USACE (1990), but each organisation developed distinct approaches for estimating C. For the purpose of this paper and the research undertaken only free-flow conditions are considered, although it is appreciated

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that there are additional modification to the equation to account for downstream and submergence effects of ogee spillways.

For the purposes of this paper the graphs and functions for the representative approaches are excluded for the purposes of keeping this paper concise. Each graph and equation can be found in the respective publications.

USBR (1987) presented the base discharge coefficient, C_0, as a function of the ratio of pool depth to head (P/H_0). The C_0 increases steadily once $P/H_0 > 1$, but drops rapidly when $P/H_0 < 0.5$. Additionally, USBR provides a modification factor, C1, for the base coefficient of discharge as determined by the unit flow ratio, H_e, to the design flow, H_0. Assessing the modification factor there is a notable increase in the C1 and thereby the discharge coefficient as the unit flow increases. The opposite is also true, that as the unit flow decreases in relation to the design flow the modification factor decreases. The final modification factor considered by USBR, for free flow conditions, considers the approach slope. The modification factor, $C_{inclined}$, for the base coefficient of discharge as determined by the ratio of pool depth to design flow height for different upstream approach slopes is given as three curves, each representing variations in the approach angle to the crest with the vertical plane as reference. The three curves given are for slope ratios of 3:3, 2:3 and 1:3 representing steep to shallow approach angles.

Ven te Chow (1959) assumed a constant base coefficient of 4.03 (imperial units, equivalent to 2.2249 SI), but illustrated multiple modification factors C for the various upstream pool depth ratios in a single combined chart. Unlike USBR (1987), Ven te Chow (1959) did not separate base and unit flow modification factors, instead embedding them in families of curves. Similarly to USBR, Ven te Chows does include a modification for the approach flow slope.

USACE (1990) developed families of discharge coefficient curves for ogee spillways with vertical and sloped upstream faces. This is the first departure from both USBR and Ven te Chow, as the slope effects are treated as inherent properties in the USACE (1990) method rather than applied separately. These curves give the base coefficient C_0 as a function of the pool depth ratio P/H_0 and also show how coefficients vary when the actual head (H_e) differs from the design head (H_0), i.e., for discharges other than the design discharge. This dual presentation means that the graphs can be read both as the base coefficient chart and as the H_e/H_0 correction equivalent to the USBR (1987) and Chow (1959) approaches.

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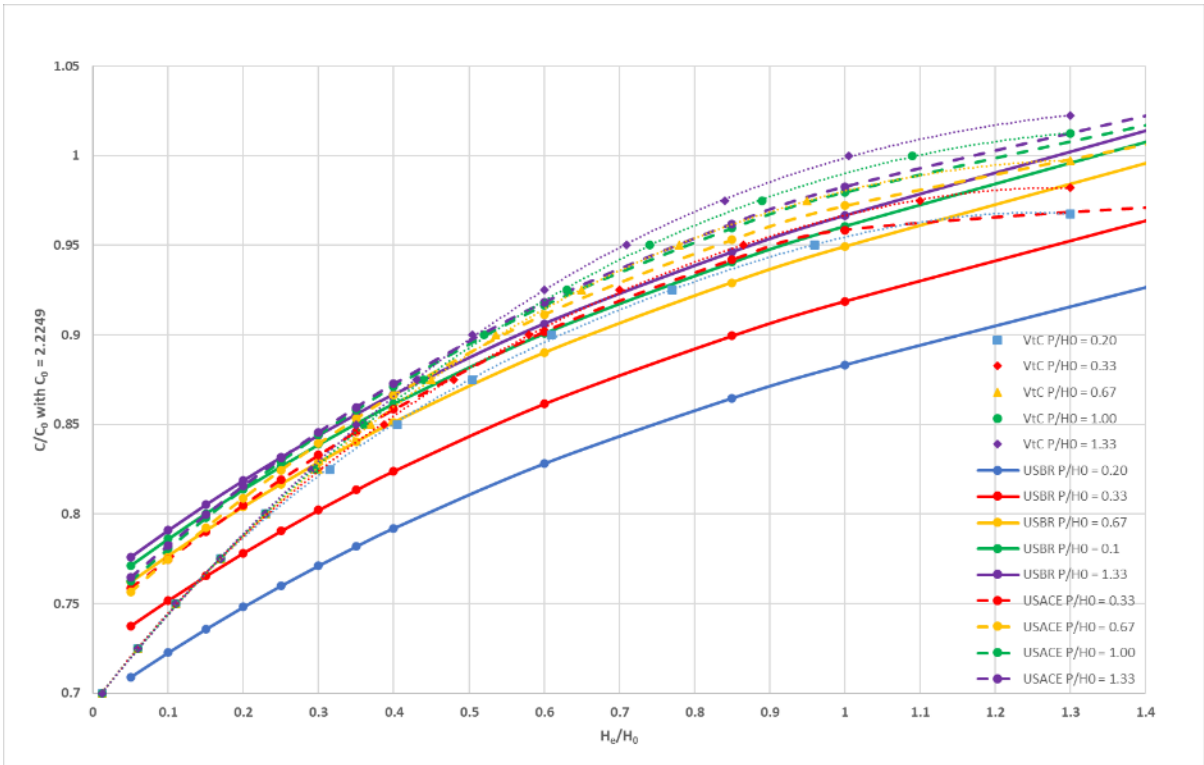

Figure 1. Discharge Coefficient Comparison between the various methods

Overlaying the results from USBR (1987), Ven te Chow (1959), and USACE (1990) highlights systematic differences but also areas of convergence. At shallow relative pool depths $(P/H_0 < 1)$ USBR curves show the sharpest decline in discharge coefficients, whereas Chow and USACE predict comparatively higher values. At deeper pool ratios $(P/H_0 < 2)$, the three datasets converge within about 3%, suggesting that discrepancies diminish as submergence increases. These contrasts are illustrated in Figure 1, which overlays the coefficient curves from the three methods on a common axis. The figure shows that at design head conditions $(H_e/H_0 = 1)$, discrepancies can reach up to 8% in shallow pools, with USBR (1987) predicting lower values than Ven te Chow (1959) and USACE (1990).

The treatment of slope effects also highlights key differences between the reference sources. Figure 2 compares the correction factors from USBR (1987) and Ven te Chow (1959) for approach angles of 1:3, 2:3, and 3:3. While the 1:3 and 3:3 slopes show broadly similar trends, the 2:3 slope diverges most strongly between the two methods. Ven te Chow's curves tend to decline at higher H_e/H_0, whereas USBR's curves flatten toward a horizontal asymptote. This highlights not only numerical differences but also different assumptions about how efficiency scales with slope and relative depth.

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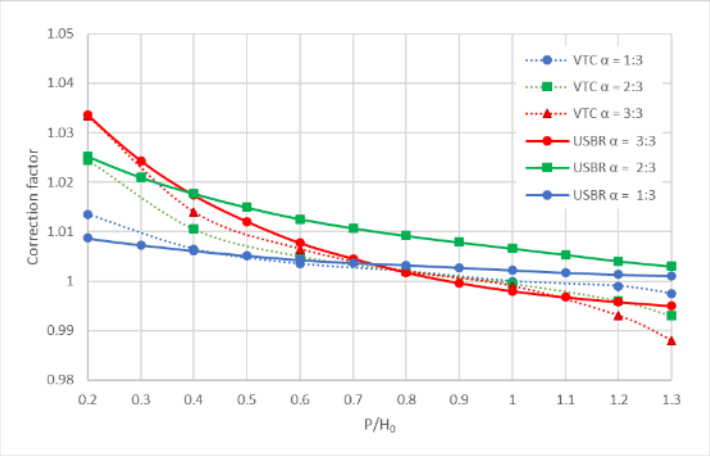

Figure 2. Slope Modification comparison between the various methods

Taken together, Figure 1 and Figure 2 provide a consolidated view of the agreements and discrepancies between the three main reference methods. This synthesis demonstrates that while broad convergence occurs at deeper pools, method choice under shallow or sloped approach conditions can affect the design discharge coefficient and, consequently, prototype spillway dimensions.

2. AVAILABLE INFORMATION

The model study reports for the various dam that were utilised for this research are provided in Table 1. While a great many model studies were conducted over the history of the Department of Water and Sanitation, these following dams where the only reports that were available and that had sufficient results and information available from which to extract. This is an unfortunate limitation of information as the results of non-standardised and un-archived information.

The individual model studies and calibrations are not directly referenced in this paper but a general reference is provided in terms of access to information from the DWS. (Department of Water and Sanitation, 2025).

Table 1. Dataset information

No.	Name	Author	Date	Shape	Model Study	DWS Calibration
1	Pongola	ECH Sellick	05/1964	Ogee	X	X
2	De Mist Kraal	R H Marnewick	*06/1987*	Ogee	X	X
3	Wolwedans	HP van Staden	*09/1991*	Ogee	X	X
4	Driekoppies	Chunnett, Fourie and Partners	*02/1988*	Ogee	X	X
5	Grootdraai '77	WCS Legge	08/1977	Ogee	X	X
6	Grootdraai '79	WCS Legge	06/1979	Ogee	X	X
7	Inanda	MJ Sienkiarski	02/1985	Ogee	X	X
8	Nungwana	R Cai	11/1995	Ogee	X	X

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No.	Name	Author	Date	Shape	Model Study	DWS Calibration
9	Knellpoort	WS Rowlston	12/1987	Ogee	X	X
10	Wagendrift	N/A	04/1962	Ogee	X	X
11	Kogelberg	KJM Jordaan	09/1983	Ogee	X	X
12	Albert Falls	BE Hollingsworth,	11/1974	Ogee	X	X
13	Rhenosterkop	MJ Sieniarksi,	02/1983	Ogee	X	X
14	Hazelmere	JC Hafensche	11/1973	Ogee	X	X
15	Kouga	N/A	03/1960	Ogee	X	X
16	Vaal	JM Jordaan	08/1987	Ogee	X	X
17	Nwamitwa	HJ Wright	03/2017	Ogee	X	-
18	Krugersdrift	KE Olsen	09/1969	Ogee	X	X
19	Heyshope	N D Killian	03/1963	Ogee	X	X
20	Stompdrift	R Immelman	12/1979	Ogee	X	X
21	Roodekopjies	N D Killian	12/1979	Ogee	X	-
22	Clanwilliam	S Hartley	02/1994	Ogee	X	X
23	Qedusizi	JL Enslin	09/1996	Ogee	X	-
24	Gcuwa	C Hattingh	06/2023	Ogee	X	-
25	Elandsdrift Barrage	N/A	03/1972	Ogee	X	-
26	Zalu	R De Lange	10/2020	Ogee	X	-
28	Roodeplaat			Ogee	-	X
29	Mearns Weir			Ogee	-	X
30	Zaaihoek			Ogee	-	X
31	KillBurn			Ogee	-	X
32	DeHoop	HJ Wright	06/2015	Ogee	X	X
33	Loskop			Ogee	-	X
34	Blyderiver			Ogee	-	X
35	Flag Boshilo			Ogee	-	X
36	Kleinplaas			Ogee	-	X
37	Nandoni			Ogee	-	X
38	Duiwenhoks			Ogee	-	X
39	Peitersfontein			Ogee	-	X
40	Nahoon			Ogee	-	X
40	Laing			Ogee	-	X

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3. COMPARISON OF RESULTS

Figure 3 shows the model study results for the twenty-seven dams. The dotted lines represent the individual curves and the points representing the unique unit flow ratios that were compared. The dashed green lines represent the upper and lower bound standard deviation, top and bottom lines, and the solid green line represents the data-mean in the center.

It is important to note, that the coefficient of discharge is dependent on the design head, H_0, for each dam. In the model study reports much of the information had not been standardised, and as many of the studies focussed on a retrospective capacity analysis, the design head was not provided. This limitation of the information necessitated an additional comparison in the analysis process. That aspect was to compare the curves for various assumed design head. For the purposes of explanation, the results presented are for the assumption that the design head is equal to the maximum dry freeboard, $H_{FB\text{-}dry}$. That is to say that the design head is the full height between the spillway sill level and the non-overspill crest. It is understood that this may not be the case in practice, but it does represent the uppermost boundary of the potential assumptions. Additional comparisons are completed for the assumptions of $H_0/H_{FB\text{-}dry}$ equal to 90%, 80%, 70% and 65%. It is with this underlying understanding in mind that the maximum H_e/H_0 ratio will shift according to each assumption case. This is because the maximum spillway capacity should not overtop the dam. In Figure 3 the design head and maximum unit head ratio is 1 because of the aforementioned assumption.

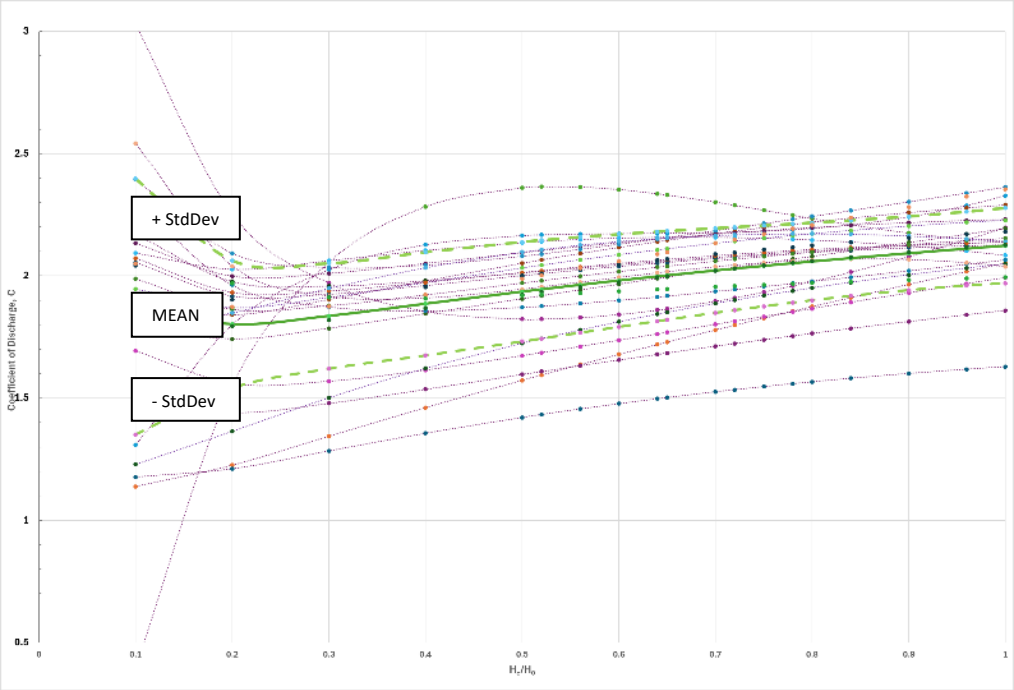

Figure 3. Ogee model Study results (whole dataset) for H_0 = 1.0 x $H_{FB\text{-}dry}$

The results shows that the expected efficiency at lower unit flow ratios is greatly divergent with large variations, both the positive and negative. This however is not the range of concern for this study, which is between the design flow and maximum flow conditions, ie: H_e/H_0 greater than 1.

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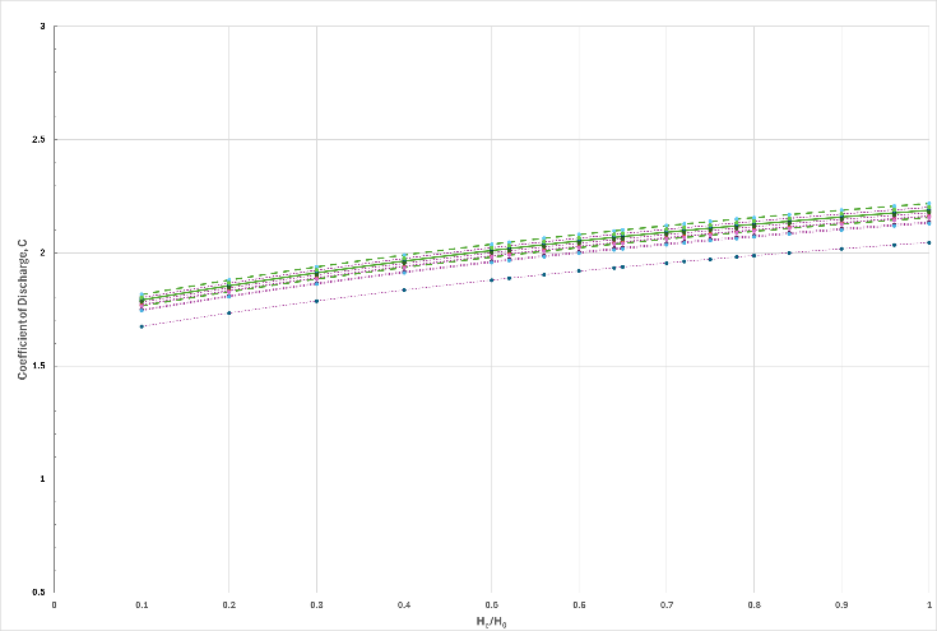

Figure 4. USBR theoretical coefficients (whole dataset) for H_0 = 1.0 x $H_{FB\text{-}dry}$

Figure 4 shows the results for USBR's *Design of Small Dams'* theory applied to the same dataset of twenty-seven model studies. In a similar manner as presented before the dotted lines show the individual results. The dashed green lines show the upper and lower standard deviations, and the solid green line shows the data-mean for the USBR theoretical results. In Figure 4, unlike the model study dataset the USBR results are orderly and uniform showing a condensed grouping of results with negligible variations. This pattern of uniformity is seen with each theoretical approach's results to varying degrees. The figures was intentionally plotted on same scale as the Figure 3 to further emphasise the unpredictive nature of the practical results in comparison to the uniformity of the corresponding theory. This aspect in-and-of itself highlights the limitation of theory to comprehensively describe the complexity of spillway hydraulics.

Figure 5 shows the comparison between the theory and model datasets for the individual USBR study. The graphed results for USBR's *Design of Small Dams*, Ven Te Chow's *Open Channel Hydraulics*, USACE's *Design of Hydraulic Spillways*, were also assessed for this study, each showing similar trends. The mean of the datasets and both upper and lower standard deviation bands are shown for each comparison by the three solid lines. As noted in the previous sections the importance of the graphs is not in the specific dam data but rather in the aggregated trend that is developed. It is therefore that the respective individual dams aren't given in the legends for the graphs.

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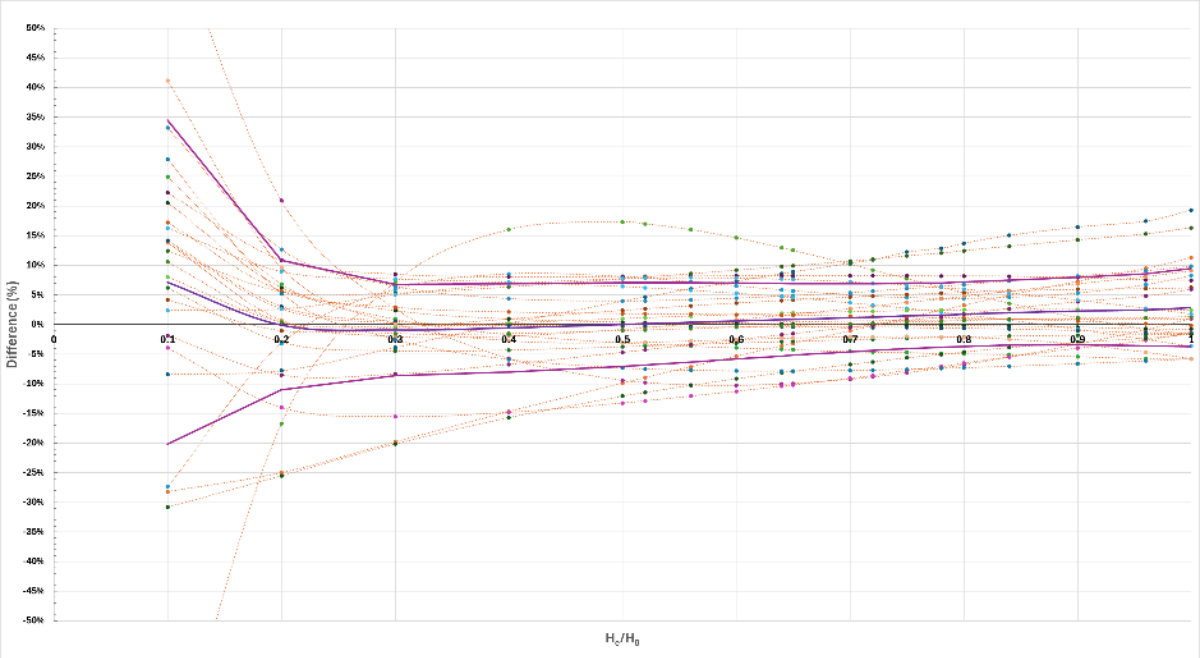

Figure 5. % Difference comparing the individual Model study to the Theoretical results (whole dataset) for H_0 = 1.0 x $H_{FB\text{-}dry}$

The results presented in Figure 5 highlight the whole dataset, which includes curved spillways, spillways with piers and gated spillways, although only the free-flowing, ie: ungated, conditions were assessed.

$$Difference\left(\frac{H_e}{H_0}\right) = \frac{C_{practical}}{C_{theory}} - 1 \qquad \text{Equation 6}$$

where: $C_{practical}$ = Coefficient of discharge for the model study or calibration.
C_{theory} = Coefficient of discharge for the different theoretical approaches.

Figure 5 shows the comparison of the theoretical mean-curves against the model study's mean curve with use of Equation 6. This method of comparison is basis of results information for this study. From the equation it is important to note that a negative difference shows an overestimation of the theoretical discharge coefficient, because the equation results in a greater denominator value, and thereby negative when reduced by 1. Assessing the positive-negative position of the results therefore infers the over- and under-estimation of the theory in comparison to the physical dataset results.

To interpret the various effects that the physical factors of the spillway have on the results different scenarios are compared. Table 2 shows the different scenarios that were derived from the whole dataset. These scenarios are grouped based on the geometry, location of the spillway and approach flow angle to highlight the potential of each better. The base condition is the straight, centered and un-obstructed ogee spillway as that represents the most classical definition of the assumption of a spillway. This assumption removes the outlier ani fringe results that may skew the mean results.

Furthermore, to assist with assessing the results, simplified 'diagnostic' charts where derived showing the same model study and DWS calibration comparison but only representing the design flood and maximum flood for each assumption. The solid lines in each represent the comparative difference for the maximum flood and the dotted lines for the design flood. Although these diagnostic charts only show condensed information many of the same trends are evident in these diagnostic charts.

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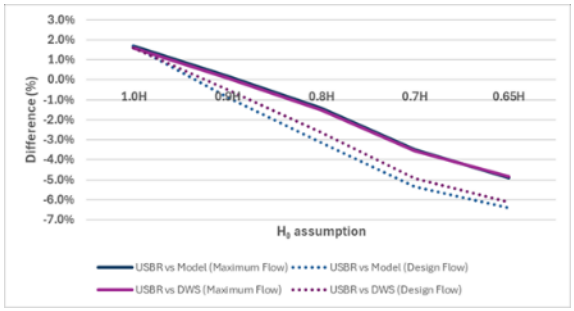
Table 2. Analysis Scenarios

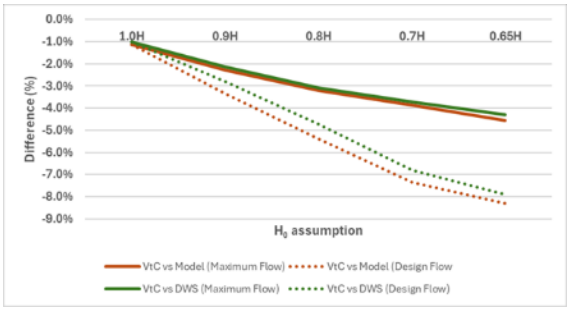
Number	Grouping	Scenario	Geometry	Location	Approach flow angle
1	Base	Base dataset	Straight and Un-obstructed	Center	75-105
2	Geometry	Straight and Center	Straight	Center	All
3		Curved	Curved	Center	All
4	Location	Straight and Non-Center	Straight	Left/Right	All
5	Angle	Direct Approach	Both	All	75°-105°
6		Angled Approach	Both	All	60°-120°

3.1 Base Scenario

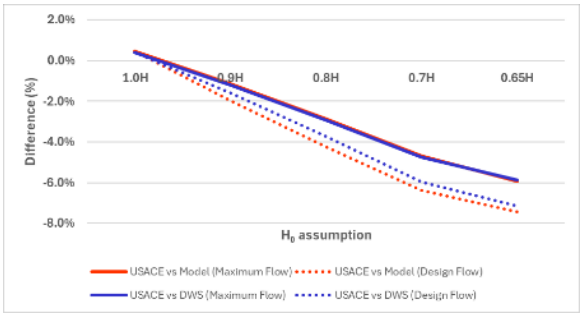
The Base Scenario dataset comprises of eleven different dams (eight model studies and eight DWS calibrations). Assessing the Base scenario, it is evident that the difference between the Model study and DWS calibration comparisons are near-negligible. This is evident for all the approaches.



(USBR)



(VtC)



(USACE)

Figure 6. Summary of % Difference comparing the Individual results to the Model study and DWS Calibration results (Base Scenario) for H_0 = {0.65-1.0} x $H_{FB\text{-}dry}$

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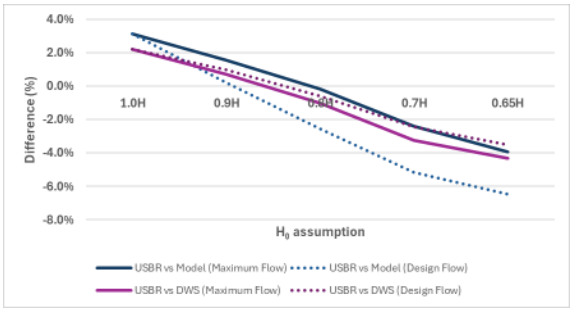
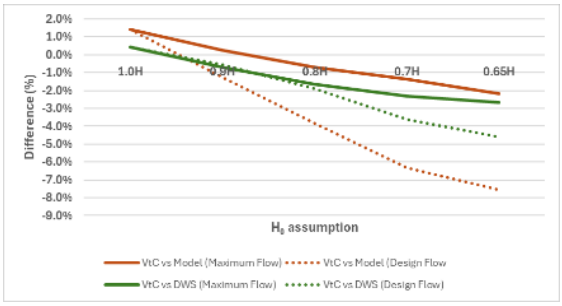
For all the base conditions it is evident that each theoretical approach overestimated the flow. This is an interesting result as it would be expected that for this classical setup of the ogee there is uniformity between the theory and practical data. Although it is clear that both Ven te Chow and USACE both overestimate for whole or most of the assumption range. It is however, appreciated that if the design head was known many of the results for the USBR tend closer to the 0% difference for this case.

3.2 Scenario 2

The Scenario 2 dataset comprises of fifteen dams (eleven model studies and twelve DWS calibrations) of which three are gated spillways. Comparing the base scenario with Scenario 2 shows the effect of including non-direct flow over the spillway as well as the addition of piers in the spillway.



(USBR)



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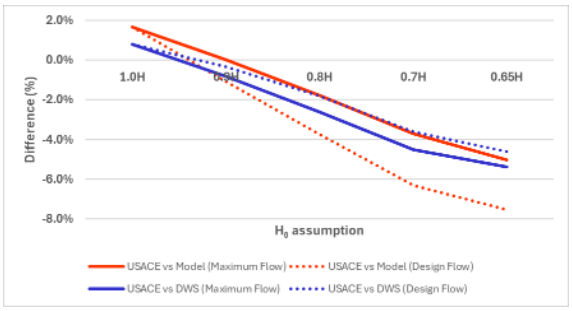

(USACE)

Figure 7. Summary of % Difference comparing the Individual results to the Model study and DWS Calibration results (Scenario 2) for H_0 = {0.65-1.0} x $H_{FB\text{-}dry}$

For each of the approaches there is an upward translation of both dataset results. The maximum flow for the model study dataset shows a clear difference while the design flow for the DWS data has the greatest difference when compared to the base scenario.

When evaluating the results of the three approaches it is evident for Scenario 2 the greatest effect on the results is seen in the with the DWS calibration design flow condition. The positive translation of the results is also clear throughout the results to varying degrees.

3.3 Scenario 3

Scenario 3 dataset comprises of ten dams (six model studies and ten DWS calibrations). Comparing the Base Scenario with Scenario 3 shows the effect of curved spillways. In the base scenario and reflected in Scenario 2 there is an apparent overestimation of the efficiency by the theory, whereas with Scenario 3 there is an evident translation of the results to show the theory underestimating the

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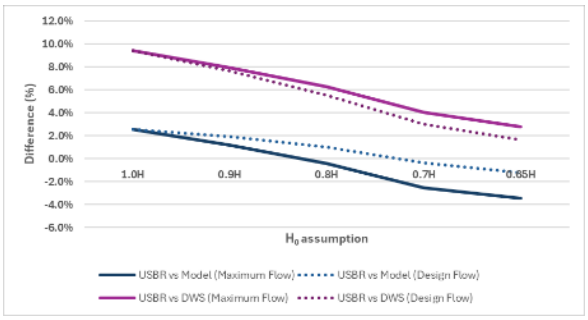
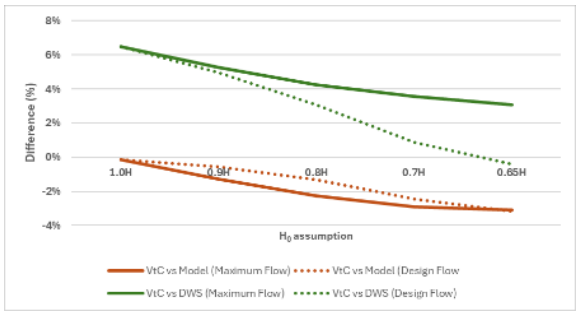
performance of the curved spillway. This underestimation is most prominent with the DWS calibration results which took a notable positive translation for each approach.



(USBR)



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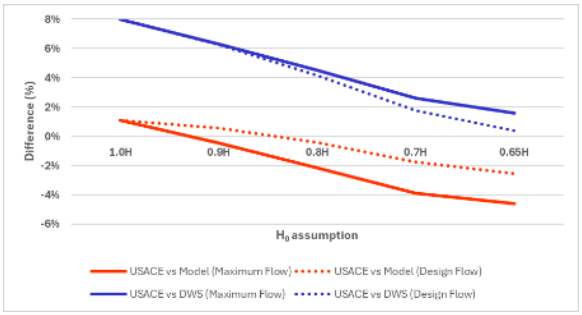

(USACE)

Figure 8. Summary of % Difference comparing the Individual results to the Model study and DWS Calibration results (Scenario 3) for H_0 = {0.65-1.0} x $H_{FB\text{-}dry}$

Assessing the four approaches in Scenario 3 the results consistently show the greatest difference with the DWS calibration dataset. This is reflected in the significant nominal difference between the scenario results for the DWS calibrations. Further insight is drawn from the reduction of slope consistently seen for the design flow conditions, which highlights the reduced range of result values across the design head assumptions. This is, in-part, expected as the discharge modifications (approach slope and unit head) are at unity for the design flow condition. It is reasoned that the variation between the model study and DWS calibration results are in part attributed to the limited number of studies, but it also the nature of the typical location of curved spillways. Curved spillways are typically on arch dams which are located against solid rock abutments in a valley-type locations. This promotes deep and direct approach flow over the spillway, with limited cross-flow in the approach flow conditions.

3.4 Scenario 4

Scenario 4 dataset comprises of seven dams (six model studies and five DWS calibrations) of which three are gated spillways. Comparing the base scenario to Scenario 4 shows the effect of non-centered spillways. The results highlight a clear inversion of the dataset comparison. The model study results are underestimated and the DWS calibration results tending closer to the 0% difference axis for each approach. This highlights an interesting observation in the performance of the model study results.

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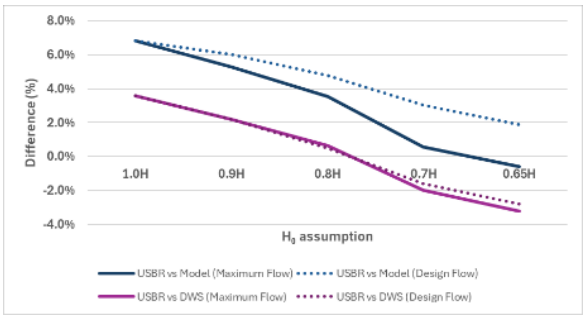
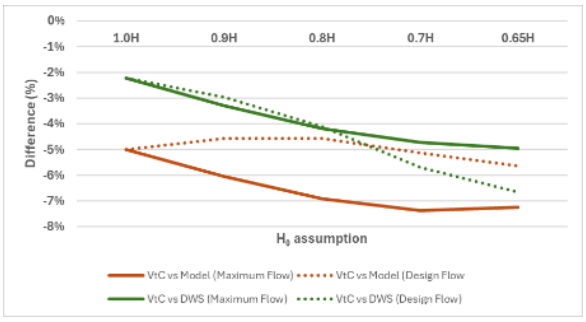


(USBR)



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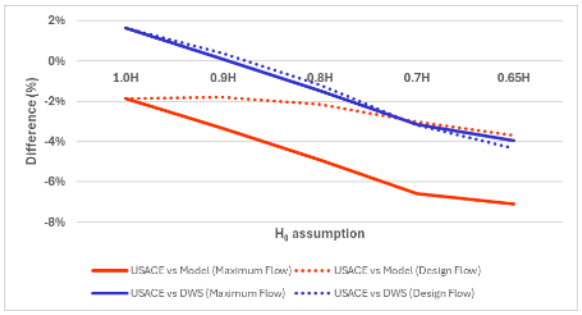

(USACE)

Figure 9. Summary of % Difference comparing the Individual results to the Model study and DWS Calibration results (Scenario 4) for H_0 = {0.65-1.0} x $H_{FB\text{-}dry}$

When interpreting the results the first factor that is appreciated is the limited dams in the dataset that thereby increases the sensitivity of the statistical results and comparison.

3.5 Additional Scenarios

In the greater study additional scenarios were also conducted and evaluated. These studies however, are not included in this paper as the aim is achieved in highlighting the deviations from the theoretical approaches as a result of the practical configurations of the spillways. This is achieved through the first 4 scenarios.

4. CONCLUSION AND RECOMMENDATIONS

This paper clearly shows the variations found in literature for the standard ogee theory. This variation between approaches is important to note because in the absence of standards, each approach being from reputable and established sources, carries the same significance. This therefore introduces a potential variation of results based solely on the theoretical approach utilised.

The base scenario highlights a critical correlation between the model studies and the DWS calibrations where there is alignment of the results. Furthermore, the base scenario highlights the critical deviation of the theory and practical datasets for each theoretical approach. It is noted in the base scenario discussion that the average difference between the theory and practice may yield more unified results if the design head is known but it is very clear that each approach has a different trend and relationship to the practical results. The USBR results have the most balanced average difference, where the whole

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range of Ven te Chow's key results show an overestimation of the theory. USACE has skew toward overestimation by the theory.

The results from Scenario 2, 3 and 4 very clearly shows the variation of the results because of the physical and practical configurations of the spillway. Each of these scenarios are susceptible to limited available information for each dataset but do highlight the variations not only from the different theoretical approaches but also between the design and maximum flow conditions as well as between model study findings and DWS calibrations.

From this paper the primary recommendation is for a South African standard approach for the hydraulic design of spillways to be developed and/or adopted. Although there is a clear connection between the SANCOLD and ICOLD standards and the USBR approach this is not a national standard and therefore allows for discretion of the engineer.

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A PROPOSED CLASSIFICATION SYSTEM FOR DAMS IN ANGOLA

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ABSTRACT

The classification of dams is crucial for ensuring their safety, as it helps assess the risks and hazard potential associated with each dam and guides the implementation of necessary activities or safeguards. Currently, a dam classification system tailored for Angola is being developed, taking into account the country's unique economic, social, and geographical contexts. The proposed categories draw inspiration from established methodologies, including those from ICOLD, South Africa, and Mozambique. This paper outlines the proposed Angolan dam classification system and details the rationale behind its development.

1. INTRODUCTION

As a result of the inherent risk posed by most dams to the public, to property or to the quality of water resources, the design, construction, operation and decommissioning of dams are nowadays typically governed by legislation in many countries. The legal framework for dam safety assurance serves to establish minimum standards, along with duties, roles, and responsibilities, for assuring the safe development and operation of dams (Wishart et al, 2021). Whilst such legislation cannot remove the risks completely, the potential for harm to the public, damage to property or to the quality of water resources can be greatly reduced.

Most laws dealing with dam safety recognises the fact that the risk posed by dams vary and the associated legal requirements therefore typically also varies. It is also generally accepted that some dams may pose no risk or the risk may be considered too low to warrant any regulatory compliance. Dams that need to comply with safety legislation are typically classified/categorised and different regulatory requirements will apply to dams in different categories. For dams with a lower risk (and associated classification) the legal requirements are typically less stringent than for dams with a higher risk, and vice versa. This ensures that greater effort is put into the design and management of dams with higher risk, whilst at the same time ensuring that dams with a lower risk are designed and managed to acceptable standards.

2. ANGOLAN CONTEXT

2.1 Governance

The Ministry of Energy and Water (MINEA) was established by presidential decree in May 2014 as an Auxiliary Ministerial Department of the President of the Republic and of the Executive. The purpose of MINEA is to set and coordinate national policies in the energy and water sectors. The National Institute of Water Resources (INRH) was established later that same year as a subordinate of MINEA, responsible for regulating the Angolan water resources and implementing the national water resources policy.

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PRODEL-EP is a Public Electricity Production Company that was created in 2014 by presidential decree to manage energy production. Most of the Angolan dams aimed at hydroelectric power generation are managed and operated by PRODEL-EP.

2.2 Dams in Angola

Dam safety regulations are typically developed and tailored to the country in which they are intended to be applied. Factors such as climate, hydrology, topography, geology, availability of construction materials, technical expertise, institutional frameworks and socio-economic priorities all influence how dams are designed, maintained, operated, and monitored.

Angola has more than 1300 rivers, excluding springs, with a total combined length of more than 57 000 km spread over eleven hydrographic regions (Covele, 2020). Most of the rivers originate from the Central Plateau in the Huambo region.

Based on the current inventory of dams, Angola has at least 189 dams, with this number likely to increase with the identification of additional (smaller) dams. Dams in Angola primarily serve to provide a secure water supply for domestic use, irrigation, or as a source to generate hydroelectric power. Although the majority of the dams are utilised to supply water for domestic and irrigation purposes, most of the larger dams (by vertical height and reservoir capacity) are used to generate hydroelectric power. The latter contributes to more than 70% of the electricity generated within the country (IEA, 2023). To this end, Biopio, Cambambe, Capanda, Chicapa, Gove, Laúca, Lomaúm, Luachimo and Matala Dams are the prominent dams that currently contribute towards generating hydroelectric power. The current total installed capacity of 3 890 MW will increase to 6 062 MW once the Caculo Cabaça hydropower plant, currently under construction, is complete.

The sizes in terms of vertical wall height of dams in Angola range from a few meters to more than a 100 m, with Laúca Dam and Capanda Dam being the highest, with heights of 132 m and 110 m respectively. These dams also have the largest reservoir capacities in the country, 5 482 million m³ and 4 795 million m³ respectively. Most of the larger dams, including Laúca Dam and Capanda Dam, are located in the north of Angola, east of Luanda in the Cuanza region.

2.3 Population

Angola has a population of approximately 39 million people. The country has historically been a largely rural country, but with increasing urbanisation the urban population now accounts for just under 70% of the total population. The 2014 census showed a population density varying from under 4 persons/km^2 in the Cuando Cubango province in the south-east of the country to over 400 persons/km^2 in the capital city Luanda on the northern west coast. The major population centers include the coastal cities of Luanda, Lobito, and Benguela, as well as inland hubs like Huambo and Lubango.

The average household size in Angola is approximately 6.45 members. Nearly 25 percent of households have seven (7) or more members, while single-person households account for 9.4 percent of the total. (Statistica.com, 2025).

3. DAM CLASSIFICATION WITHIN THE CONTEXT OF RISK MANAGEMENT

Dam safety legislation is essentially a form of risk management to ensure that the construction and operation of dams comply with certain safety standards. To this end, the classification of dams is normally part of this risk management process. For the purpose of this paper, the risk posed by dams will be defined as follows:

$$Risk = Probability\ (of\ failure) \times Consequences\ (of\ failure)$$

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Using the above definition, the risk associated with a dam can be reduced by either reducing the probability of failure, the consequences of a failure or both.

3.1 Reducing probability of failure

With dam safety legislation the probability of failure can be reduced by the inclusion of various requirements in the legislation. Typical key requirements, in broad terms, include that designs be done by professional dam engineers, design drawings and reports contain the necessary information, existing dams undergo periodic safety inspections, dams are managed using an approved operation and maintenance manual and any rehabilitation works are done in a controlled manner by suitably qualified people.

Whilst the above requirements significantly improve the safety of dams if the legislation is implemented, dam safety legislation has traditionally followed a standards-based approach, meaning that if certain standards or criteria are met, the level of risk associated with a dam is deemed acceptable, even if the level of risk itself is not quantified. When a new dam classification system is considered to be included in legislation, it should be remembered that it forms part of the overall dam safety management framework. The classification system should be such that it enables appropriate regulations to be included in the legislation for each class or category of dam. It should seek a balance between achieving an acceptable level of safety for all dams, based on the risk posed, whilst at the same time not overregulate dams, leading to possible unnecessary expenditure.

3.2 Reducing consequences of a dam failure

Many countries implement legislation to limit development in floodplains, often prohibiting occupied structures below the 50-year or 100-year recurrence interval floodline. However, dam safety legislation does not normally regulate downstream development due to the relatively low risk associated with dam failures, compared to natural floods. Additionally, there is often already some development downstream of a proposed dam. Ideally, dam safety legislation should focus on managing the risk at the dam itself, thereby enabling safe downstream development rather than restricting it.

Timely warnings during dam failures can also significantly reduce loss of life, as seen in the case of the failure of Teton Dam. Although more than 700 houses were totally destroyed and more than 3000 damaged, there were only 11 fatalities. Emergency preparedness and response/evacuation plans should outline communication procedures and details of key contact persons to ensure authorities and persons at risk are alerted. Whilst it is normally the duty of the dam owner to inform relevant authorities and persons at risk living immediately downstream of the dam, the logistics of evacuating people are normally managed by the relevant authorities.

4. GENERAL CLASSIFICATION CONSIDERATIONS

The aspects typically considered by countries with dam safety legislation for the classification/ categorisation of dams can be divided into two main groups:

- The physical characteristics/geometry of the dam wall and/or reservoir, including the height of the dam wall, the storage volume of the reservoir, etc.
- The expected consequences in case of a dam failure, including possible loss of life, financial losses and environmental damage.

The second set of parameters are sometimes referred to as the hazard potential of the dam. Many countries consider both dam characteristics and consequences of failure in the classification process, whilst other only consider one of these groups, resulting in the dam having either a characteristics-based category or a consequence-based category.

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5. CLASSIFICATION BY DIFFERENT COUNTRIES

5.1 General

For the purpose of this study the classification systems as implemented by various countries have been reviewed for comparison purposes. Whilst there are some similarities in the classification system between some countries, the differences are notable and the systems adopted do not appear to be linked to factors such as date of implementation of the legislation, or external aspects such as continent, GDP, etc.

All the dam safety legislation from the various countries studied for the purpose of this chapter accepts that not all dams need to be regulated by legislation, with the smaller dams with minimal to no risk to downstream developments normally not falling under the dam safety regulations.

Different countries and in some instances states within the same country use different terms for dams that need to comply with their relevant dam safety legislation, for instance *"declared dam"* (NSW, Australia), a *"referable dam"* (QLD, Australia), and *"dam with a safety risk"* (South Africa & Namibia).

Dams that need to comply with legislation are normally recorded in a register, which is then updated by the governing authority from time to time, as required. For ease and consistency in this study, dams that need to comply with a country's statutory dam safety requirements will be referred to as "registered" dams but note that the terminology varies between countries.

5.2 Legal framework

The legal framework used for the implementation of statutory dam safety requirements varies from country to country, including national or state acts, decrees, regulations or circulars. The legal framework for dam safety should be in line with the specific country's existing legislative framework. In many cases there is an overarching act dealing with water resources in general, which makes provision for the promulgation of decrees or regulations in terms of the act. This allows more flexibility to update regulations as considered necessary.

In almost all countries with dam safety legislation the owner of a dam is responsible for complying with the relevant dam safety legislation, whether it be privately owned or owned by the national government or state.

5.3 To regulate, or not to regulate?

As stated previously, it is generally accepted that there are dams that do not pose a risk or the risk is considered too low to warrant any regulatory compliance, i.e. need not be registered.

In most countries the height of a dam wall as well as the reservoir volume are generally used as the main parameters to determine whether a dam needs to be registered or not. If the maximum height of the dam wall and/or the volume of the reservoir is greater than a specified value, the dam needs to be registered and should comply with the relevant dam safety legislation.

With regard to the height of a dam, it is important that a clear definition for the height be provided in the legislation.

However, very often the legislation will also make allowance for the registration of dams based on the possible consequences in case of a dam failure, with the implication that some dams are registered even though they do not require registration in terms of the dam wall height or reservoir volume.

From Table 1 hereafter it will be noted that the criteria to decide whether a dam should be registered (and regulated by legislation) can vary from country to country. In many countries dams smaller than

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15 m do not fall under the dam safety legislation by default but may possibly be registered based on the consequences of the failure of the specific dam.

In many countries a dam height of 15 m forms part of the criteria for registration, which can be assumed to originate from the ICOLD definition of a large dam, which is a dam with a vertical height greater than 15 m (from the lowest foundation level to the top of the dam) or a dam between 5 m and 15 m in height that impounds more than 3 million m^3.

Whilst some countries only include dams higher than 15 m (by default), other countries include much smaller dams. As an example, any dam higher than 2 m in France will fall under the country's dam safety legislation.

In conclusion, there is no distinct international trend insofar the minimum criteria for registration of a dam is concerned and country specific conditions and preferences play a large role.

5.4 Factors considered in classification

As stated, the two main factors considered in the classification of dams by most countries are the dam statistics and the consequences in the event of a dam failure. In 2012 ICOLD's Committee on Dam Safety compiled the results of a study on dam safety regulation practices worldwide, based on data received from 44 countries. For the purpose of this study, the number of countries was increased by 5, bringing the total to 49. The additional countries are all in Africa, including Namibia, Zimbabwe, Mozambique, Ghana and Sierra Leone. These countries include countries where dam safety legislation was only introduced after 2012. The number of countries from each continent included in the ICOLD study as well as the additional countries are shown in Table 2. The criteria used for classification are shown in Table 3.

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Table 1. Criteria for registration of dams in selected countries

Austria	Height > 15 m or Storage capacity volume > 500 000 m^3
Brazil	Height > 15 m or Storage capacity > 3 000 000 m^3 or reservoir of hazardous materials or Medium or High risk category
France	Height > 2 m
Great Britian	Reservoir volume > 25,000 m^3
India	Height > 15 m or Height > 10 m, with at least one of the following: Crest length > 500 m Storage capacity > 1 000 000 m^3 Discharge capacity > 2 000 m^3/s dam has specially difficult foundations, or dam is of an unusual design.
Mozambique	Height > 15 m or Height > 10 m & Storage capacity > 1 Mm^3 or Discharge capacity > 2 000 m^3/s or High potential damage index
Namibia	H > 5 m
Portugal	Height > 15 m or Storage capacity > 1 00 000 m^3 or any Class I dam
South Africa	Height > 5 m and Storage capacity > 50 000 m^3
Zimbabwe	Height > 8 m or Storage capacity > ½ million m^3

Table 2. Distribution of countries in 2012 ICOLD study & additional data

Region	Number of Countries
North America	3
South America	3
Europe	25
Asia	8
Africa	8
Oceania	2

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Table 3. Classification criteria used by countries

Classification based on:	No. of Countries	Percentage (%)
Geometry only	17	37
Consequence only	12	24
Geometry and Consequences	17	33
Other	3	6

With the exception of Oceania (Australia and New Zealand) all the regions as listed in Table 2 have countries where only the dam statistics are considered in the classification process. Using only the dam statistics has the advantage that the registration and classification is a relatively simple process, specifically if the relevant dimensions (height, volume etc.) are known. However, classification by only using the dam statistics does not mean that the consequences of a possible dam failure is ignored. The potential loss of life and damages in the event of a dam failure are related to the flow velocity, flow depth and the warning time following the event. These parameters are related to the main characteristics of the dam, specifically the height and reservoir volume, and although the specific consequences of a dam failure are not estimated as part of the classification procedure, the consequences are indirectly taken into consideration.

Classification based on the consequences of a dam failure only is used by several countries. Depending on the countries' guidelines, the type of consequences to be considered for the dam classification can vary. Possible parameters that are typically considered include persons at risk, loss of life, direct financial losses and damage to infrastructure, indirect economic losses and environmental consequences. The hazard classification based on consequences of a dam failure is independent of the probability of failure, or the efficiency of any emergency plans in place, in case of a dam failure.

6. HAZARD CLASSIFICATION

6.1 General

The consequences of a dam failure can be significant and can include loss of life (LOL), damage to houses, infrastructure and farmlands as well as environmental damage. When considering the consequences of a dam failure for hazard classification purposes, the two aspects generally considered are potential loss of life and the damage by the dam break flood, which can include both economic losses as well as damage to the environment.

For classification purposes, there are various methods to express the consequences (LOL, damage etc.) due to a dam break, but to simplify the process, the consequences are normally grouped based on the level of severity. Such grouping is then used to determine the associated hazard classification, as shown in Table 4.

Table 4. Hazard classification (example from South African dam safety regulations)

Economic loss / Loss of life	Low	Medium	High
0	Low	Significant	Significant
to 10	Significant	Significant	High
More than 10	High	High	High

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6.2 Dam break analysis

When using a typical grouping methodology as shown above, it may be possible to get an estimate of the potential damage and LOL (in the event of a dam failure) for classification purposes without doing a dam break analysis, such as where a large dam is situated just upstream of an urban area or in the case of a small dam with no development downstream. Over the last 2 decades various tools such as Google Earth and other Geographic Information System (GIS) have become readily available for such purposes. Care should however be taken to use recent satellite photographs representative of the development downstream of the dam, if any.

For new dams, especially larger dams, undertaking a dam break analysis can be made compulsory by means of legislation. Where a significant number of existing dams however need to be classified, such as in the case of Angola, making the determination of a dam break flood compulsory for all dams may notably delay the classification thereof.

That said, where there is uncertainty with regard to the expected LOL or damage resulting from a dam break to the extent that that the anticipated LOL or damage cannot be confidently grouped in the relevant category, a dam break analysis will have to be performed to determine the anticipated floodline. It should be stressed that although there are numerous methods for estimation of a dam break flood peak (and the associated flood line), the estimation of the flood peak is very dependent on assumed parameters such as the time to breach, which can significantly influence the peak flow. The time to breach and peak flow also depend on the failure mechanism (piping, overtopping, etc.) The peak flood and floodline should therefore be considered an estimation at best and for classification purposes adopting conservative parameters are advisable. From the plotted floodline, the LOL and damage is then estimated. The estimation of the LOL and possible damage resulting from a dam break are discussed hereafter.

6.3 Loss of life

Various empirical and statistical methods have been described in literature for estimation of loss of life in the event of a dam failure. An important distinction between the terms "Loss of life" (LOL) and "Persons at risk" (PAR) should first be noted. While PAR refers to the number of people who would be exposed to flooding in the inundation zone should a dam failure occur, LOL refers to the number of fatalities that actually occur, and is normally taken as a percentage of the total PAR.

A prominent method of determining the loss of life is known as the Bureau of Reclamation Method by Wayne J Graham, written in 1999 for the US Department of Interior's Bureau of Reclamation and is documented in DSO-99-06.

Aspects to consider when estimating the PAR from inundation maps include the following:

- If an Emergency Preparedness Plan (EPP) and evacuation procedures are in place.
- Anticipated warning time (along the floodline) for PAR.
- Occupancy data (i.e. estimated number of occupants per house/dwelling), which is typically influenced by socio-economic factors.

6.4 Damage

Similar to the LOL, it may not be necessary in some cases to undertake a dam break analysis in order to estimate the damage caused by a dam break flood for classification purposes, depending on the complexity of the grouping/categorisation of the damage.

For classification purposes the anticipated damage is generally expressed in relative terms, based on the description of the severity group (Low, Medium, High, etc.). The risk of using such grouping however is the subjectiveness thereof, with varying definitions of what "low", "medium" or 'high" may mean. To

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reduce the subjectiveness, guidelines for categorisation should be provided. An example of such guidelines is the guidelines provided by ANCOLD (ANCOLD, 2003), as shown in Table 5.

As an alternative to the description of economic losses above, the category (severity) of losses may be expressed as a percentage of the maximum economic losses expected for a dam break in the relevant country. If the maximum losses is *x* million in the relevant currency, then a "low" economic impact may for instance be defined as 5% or less of *x* million. Such method however will require the estimation of losses in monetary terms.

Table 5. Dam hazard impact (consequences)

Severity / Parameter	High	Medium	Low
Economic losses	Excessive economic loss such as serious damage to communities, industrial or commercial or agricultural land or facilities, important utilities, the dam itself or other storages downstream	Appreciable economic loss, such as damage to limited land areas, secondary roads, minor railways, relatively important public utilities, the dam itself or other storages downstream	Minimal economic loss, such as farm buildings, limited damage to agricultural land, minor roads, etc.
Likelihood of repair of dam	Repairs to dam not practicable. Dam essential for services.	Repairs to dam practicable or alternative sources of water/power supply available.	Repairs to dam practicable. Indirect losses not significant
Environmental impact	Substantial to irreversible reduction of a species/habitat or National or State significance with little prospect of recovery to pre-impact conditions.	Reduction of a species/habitat with regional significance.	Impacts to abundance of flora and fauna in the affected environment limited to a localised effect as species and habitat abundant in the region.

7. PROPOSED DAM CLASSIFICATION SYSTEM FOR ANGOLA

The proposed classification system for Angola considers both dam geometry and consequences of a dam failure to provide a category for a given dam. The details thereof are discussed hereafter.

7.1 Size classification

The proposed criteria for the size classification of Angolan dams are presented in Table 6 hereafter.

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Table 6. Proposed size classification in terms of dam height

Size	Definition
Small	5 m < H ≤ 12 m
Medium	12 m < H ≤ 30 m
Large	30 < H ≤ 45 m
Very Large	H > 45 m

In line with typical international practice, small dams less than a specific height and with a storage capacity less than a specific volume are proposed to be excluded from the register of dams subject to classification and possible dam safety legislation. A height of 5 m and a volume of 50 000 m^3 is considered appropriate in this regard. It should be stated however that whilst dams less than 5 m in height would not automatically require classification, in certain cases dams smaller than the proposed 5 m may need to be classified where such a dam poses a known and notable risk to the area downstream of the dam.

Dams greater than 5 m in height are by default assumed to pose some risk to the area downstream. Distinction is further made between "Medium" and "Large" dams, as dams with a height greater than 30 m are assumed to pose a notable greater risk than dams with a height in the order of 15 m, requiring more stringent control in the design and management thereof. Lastly, a fourth size class is proposed for dams in excess of 45 m high. This size class will include a number of Angola's hydropower dams, and damage to such dams or in worst case a dam failure will not only cause extensive direct economic losses and possible loss of life, but the secondary impact due to loss of power to the grid may also be substantial. The operation and management of such dams should include an extensive dam safety monitoring (instrumentation) system.

7.2 Hazard classification

To classify a dam into a specific hazard category, the loss of life and potential damage downstream are proposed to be used as parameters. The proposed hazard classification system is shown in Table 7.

Table 7. Hazard Classification

Damage / PAR	Low	Medium	Significant	High
0	Low	Low	Medium	Significant
1-10	Medium	Medium	Significant	High
10-100	Significant	Significant	High	High
>100	High	High	High	High

With reference to Table 5, the following definitions for the consequences classes are proposed. For classification purposes the lesser of the consequences in each severity class should govern, whether it be financial or environmental consequences.

- Low

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Damage to some dwellings/houses possible, but no loss of life or major structural damage expected. Minor damage to roads possible. All damages repairable within 3 to 12 months from dam break event. Erosion within the river channel, but no notable erosion outside of the main river channel.

- Medium

  Notable damage to some dwellings / houses, with possibility of entire house/dwelling being destroyed by flood waters. Possible risk to industrial buildings. Deposition of silt in some locations in flood plain. Erosion outside of main river channel and notable deposition of silt. Repair of flood damage anticipated to take more than a year.

- Significant

  Significant damage to dwellings/houses, with high risk of complete destruction due to flood waters. Damage to industrial/commercial buildings and agricultural land or facilities. Erosion outside of main river channel. Extensive deposition of silt in flood plain. Possible impact on power production.

- High

  Significant damage to at least 100 dwellings/houses, with high risk of complete destruction due to flood waters. Damage to industrial/commercial buildings or extensive loss of agricultural land. Erosion outside of main river channel. Extensive deposition of silt in flood plain. Communities in need of temporary relocation. Notable impact on power production in country. Repair of flood damage requiring funding at national level.

7.3 Proposed dam classification system

Following from the proposed size classes and hazard classification, four dam categories (I to IV) are proposed for Angola, as indicated in Table 8 below.

Table 8. Dam Classification System

Hazard / Size	Low	Medium	Significant	High
Small	I	II	II	III
Medium	II	II	III	III
Large	II	III	III	IV
Very Large	III	III	IV	IV

8. CONCLUSION

This paper presents a proposed dam classification system for Angola, considering international practice and local conditions. The system makes provision for a varying degree of oversight for small dams with little risk to very large dams requiring stringent criteria for the design, construction and management thereof. It is envisaged that the INRH will initially use a classification system for the classification of their larger dams, whereafter it is preferably that the system be legislated, and all dams be registered (as applicable) for dam safety purposes.

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AMBIENT VIBRATION TESTING OF KATSE DAM: LESSONS AND OBSERVATIONS

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ABSTRACT

The last Panel of Expert review of Katse Dam in 2018 concluded that the original design calculations although appropriate at the time, does not satisfy all the current international accepted criteria and procedures. This specifically pertains to the abutment seismic stability as well as the overall long-term behaviour of the dam wall during normal operation and extreme conditions. As a result, the LHDA initiated a project to perform a finite element analysis of the dam as well as evaluate the monitoring system and monitoring results and determine threshold values for the monitoring system.

This paper will discuss the in-situ ambient vibration measurement of Katse Dam which is the main input used for the dynamic calibration of the 3D finite element analysis. Specific attention will be given to the methodology used as well as the results. It will also discuss the issues experienced and the lessons learnt.

1. INTRODUCTION

Katse Dam is a double curvature concrete arch dam situated on the Malibamatso River in Lesotho (see Figure 1). The dam is part of the Lesotho Highlands Development Authority (LHDA) Water Projects that supply water to South Africa. The dam is 185m high and has a crest length of 710m. It consists of 47 blocks, with each block 15 m wide. The maximum thickness of the dam at the base is 60 m, and the thickness at the crest is 9 m. Construction of the dam started in January 1991 and was completed in April 1997.

In 2018, a Panel of Experts performed a 10-year review of the dam (Dungar et al: 2019). The review recommended the investigation of the abutment seismic stability as well as the overall long-term behaviour of the dam wall during normal operation and extreme conditions including the development of threshold values for the extensive monitoring system of the dam. Consequently, the LHDA initiated a project to satisfy these recommendations by performing a 3D finite element analysis of the dam and its foundation as well as evaluate the monitoring system and monitoring results and determine threshold values for monitoring system.

Not only has the dam has multiple galleries at different elevations over the height of the dam (see Figures 2 and 3), but it also contains a number of special design features which could have an influence on the dynamic behaviour of the dam. These include the following (see Figures 2 and 3 for the location of the preformed joint and the seismic belt respectively):

- Shear keys was installed underneath the dam on both flanks to bridge the auto-breccia rock layer downstream of the central foundation;

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- A preformed joint was provided on the upstream side of the lowest part of the arch to force tension cracking in the concrete instead of the weaker foundation;
- A seismic belt (heavily reinforced hidden beams) was included at the central portion of the crest of the arch around Gallery A stretching across the uncontrolled spillway from Block 15 on the right flank to Block 16 on the left flank. The purpose of this belt is to tie the blocks to avoid out-of-phase movement between the blocks in the event of an earthquake and thus prevent damage at crest level.

Figure 1. Downstream view of Katse Dam

2. RATIONAL FOR AMBIENT VIBRATION TESTING

The use of vibration testing of dams to obtain their as-built dynamic properties for the validation of theoretical models dates back to the 1960s (Hall, 1988). The premise for vibration testing is that the dynamic characteristics of a dam, i.e., natural frequencies and mode shapes, are related to its structural properties, i.e., mass and stiffness. These parameters are system parameters, and they are directly related to the structural behaviour of the dam (stiffness, mass and boundary conditions). The mode shapes are largely dependent on the geometry of the dam, while the natural frequencies are largely dependent on the mass and stiffness of the dam.

By fine-tuning the theoretical finite element model to match the measured dynamic characteristics as closely as possible, the resulting theoretical finite element model would be a good replica of the as-built dam. Extensive research has been conducted at the University of Cape Town into the dynamic

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behaviour of large concrete arch dams (based on the methodology used for the theoretical analysis of bridges) since 2007 (Moyo & Oosthuizen, 2013, Moyo et al, 2016, Moyo & Nyoni, 2020, Bukenya & Moyo, 2023, Nyoni & Moyo, 2023). This research highlighted the use of the results of ambient vibration testing as an essential first step in the calibration of the theoretical model in the proper behavioural analysis of existing dams.

The calibration steps therefore adopted for this project at Katse Dam involved the following:

- Calibration of the dynamic properties using the results of the AVM testing and available material properties obtained during design and construction (Cassels et al, 2025a);
- Thermal calibration of the concrete structure (Cassels et al, 2024). Observed thermal behaviour was available from the static monitoring results, and
- Lastly, deformation calibration/validation using the results of the static monitoring (Cassels et al, 2025b).

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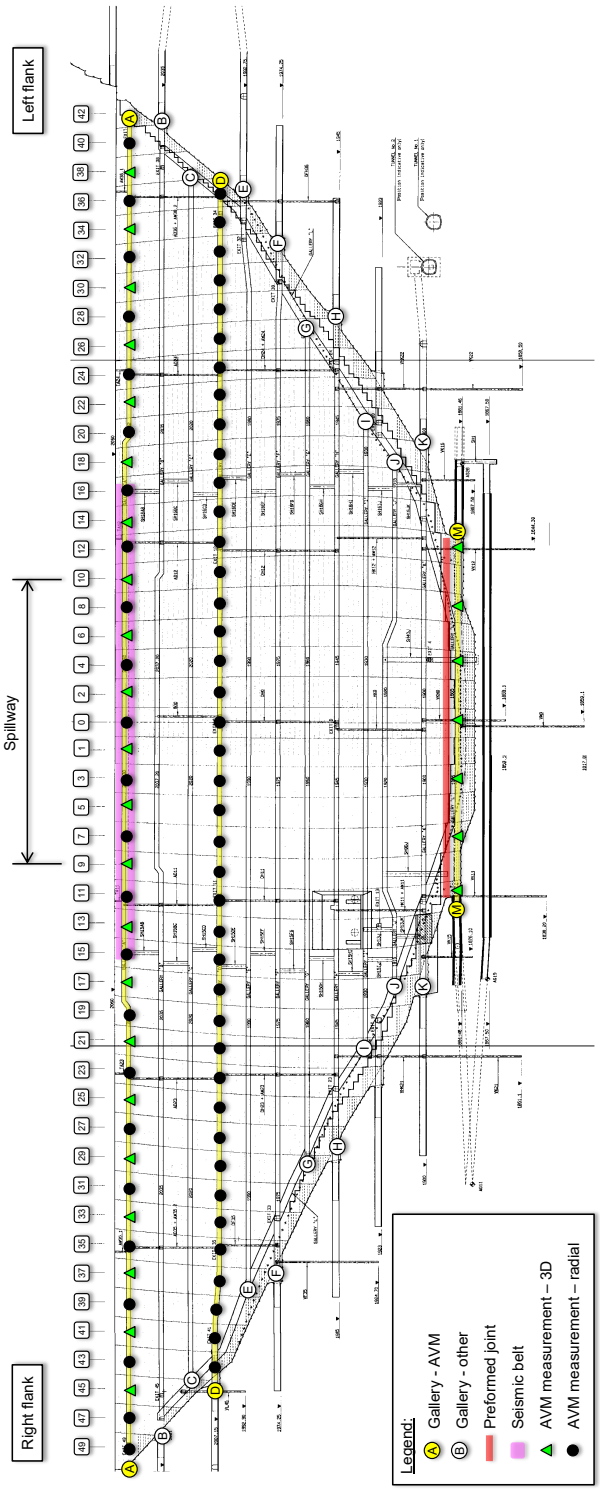

Figure 2. Downstream elevation

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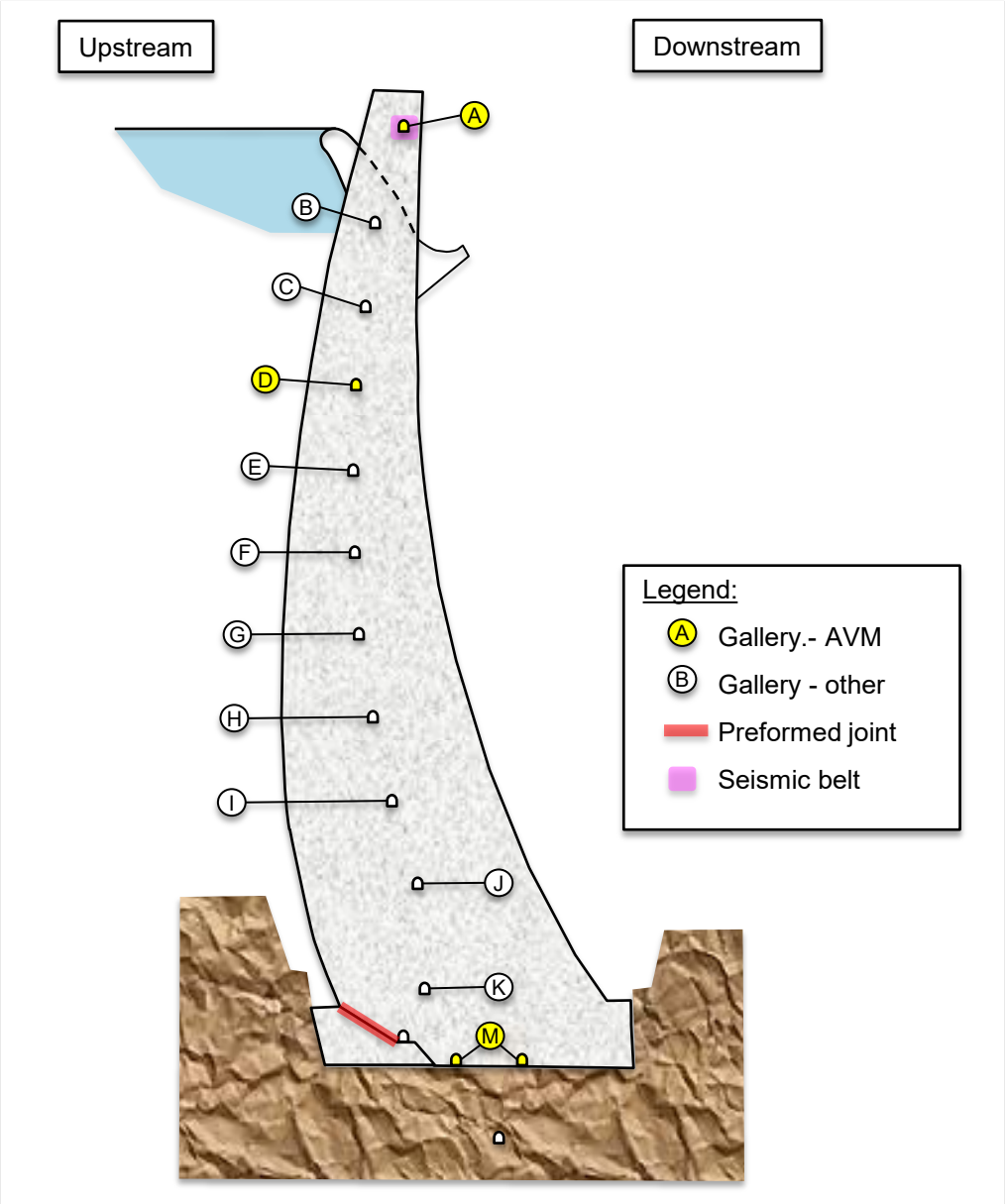

Figure 3. Section through block 11

This "new" approach to calibration has been successfully used for the analysis of a number of dams including Oanob Dam in Namibia (Hattingh, Moyo, Shaanika, Mutede, & Muir, 2019), Roode Elsberg Dam (Moyo et, 2013, Moyo et al 2016, Moyo et al, 2024) and Kouga Dam (Moyo & Nyoni, 2020) in South Africa.

It is important to note that this differs from the standard methodology of using only the results of static monitoring for the calibration of the theoretical model, which generally results in changes to the stiffness of the structure and/or its foundation to achieve calibration.

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3. AMBIENT VIBRATION TESTING

3.1 Introduction

Ambient vibration testing is not new in dam engineering. Severn et al (1980), Clough et al (1984) and Brownjohn et al (1986) conducted forced as well as ambient vibration tests on a number of arch dams and gravity dams with limited success. The main reason cited for the poor results was that dams are very stiff structures that are not easily excited by ambient forces such as water waves and wind. Darbre et al (2000) reported successful periodic ambient vibration testing of Mauvoisin Dam, followed by continuous monitoring for 180 days (Darbre & Proulx, 2002). Darbre & Proulx (2002) demonstrated that the dynamic properties of dams vary with the water level. The success of Darbre's tests was mainly due to developments in vibration measurement technologies. Coupled with developments in vibration measurement technologies were developments in techniques for extracting dynamic properties from ambient vibration data. These techniques are classified as time-domain, frequency-domain, and time-frequency domain methods. The process of extracting the dynamic properties from ambient vibration measurements is referred to as operational modal analysis (OMA).

Ambient vibration testing relies on operational loads, such as wind, ground motion, water waves, and operating mechanical equipment, to excite the dam. Only the sensors and the data acquisition system are needed for ambient vibration testing. However, due to the lack of control of the ambient excitation and the low-level vibrations they introduce, ambient vibration signals should be recorded over a longer time than forced vibration tests.

It is important to note that ambient vibration testing are normally carried out in three steps:

- A theoretical assessment of expected modal properties of the dam;
- An initial preliminary ambient vibration measurement to determine whether the frequency response functions of the dam would be measurable under operating procedures; and
- Followed by a full-scale ambient vibration testing.

For this project at Katse Dam, ambient vibration testing was performed to determine the dynamic properties of the dam, assess if the seismic belt affects the dynamic behaviour of the dam and to obtain the response of the structure at the foundation level.

3.2 Theoretical assessment

Effective ambient vibration testing requires a preliminary evaluation of the expected dynamic behaviour of the dam. Where a finite element model of the dam exists, it can be used to provide initial estimates. In the absence of a finite element model, initial estimates can be obtained from theoretical and empirical methods. The two essential parameters to estimate for test planning are the dam's first natural frequency and the expected sequence of mode shapes**.**

3.2.1 Fundamental natural frequency

Two empirical methods were used to estimate the first natural frequency of the dam with a full reservoir. The first method (described by equation (1)) was proposed by Priscu et. al. (1985), and the second method (described by equation (2)) was proposed by Kou et. al. (2007).

- $f_o = \frac{1}{0.1+0.2\left(\frac{H}{100}\right)}$ (1)
- $f_o = \frac{1}{0.06683+0.00227H}$ (2)

In each case, H represents the height of the dam in metres.

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For Katse Dam the estimated first natural was determined as 2.13 Hz based on equation (1) and 2.05 Hz based on equation (2). An error margin of up to 10% can be expected from these estimates since the empirical equations from which they are determined are based on fitting curves to experimental data.

3.2.2 Mode Shape sequencing

The United States Bureau of Reclamation (1977) classifies arch dams based on the ratio of base thickness (b) to structural height (h) as follows:

- Thin arch if b/h is less than 0.2;
- Medium-thick arch if b/h is between 0.2 and 0.3; and
- Thick arch if b/h is greater than 0.3.

Nyoni & Moyo (2023) showed that the sequence of mode shapes is influenced by the thickness of the arch dam and the valley shape. The results of their study were used to predict the sequence of mode shapes for Katse Dam.

Based on the United States Bureau of Reclamation (1977), Katse Dam is on the lower margins of a thick arch (height to base ratio is 0.32). The valley is U-shaped and its classification, based on the crest length to height ratio of 3.8, is therefore between a narrow and wide valley type according to Nyoni & Moyo (2023). Based on these characteristics, the sequence of the mode shapes of Katse Dam were initially postulated either follow the expected pattern for thick arch dams or that of medium-thick arch dams shown in Table 1 (Nyoni & Moyo, 2023).

Table 1. Theoretical mode shape sequence of thick and medium-thick arch dams (Nyoni & Moyo, 2023)

Mode	Medium-thick arch	Thick Arch
1	Asymmetrical	Symmetrical
2	Symmetrical	Asymmetrical
3	Symmetrical	Symmetrical
4	Asymmetrical	Asymmetrical
5	Symmetrical	Symmetrical

3.3 Initial preliminary measurement

It is important to note that this step is very important. In addition to the preliminary ambient vibration measurement to determine whether the frequency response functions of the dam would be measurable under operating procedures, this first site visit also assess access, identify potential risks, check the availability of power and lighting in the galleries and identify any requirements for equipment modifications and/or new equipment. The preliminary measurement also provides an indication of the amount of time required to perform the full measurement as well as the sampling rates.

During the site visit it was evident that unfortunately the lift was out of service and unlikely to be operational for the full AVM measurement. As a result, the equipment would need to be carried by hand between galleries, which was to be a challenging task considering the size of the dam. Fortunately, there is direct access to some galleries, including galleries A, D, and M which let to the decision to use these galleries for the full AVM measurement. This all worked out well as result of Gallery A's proximity to the seismic belt and the crest of the dam; Gallery D represents a typical gallery below the crest away from the seismic belt and Gallery M would provide the response of the dam at the lowest foundation level (provide information on the foundation stiffness in relation to the dam's stiffness).

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During the site visit, preliminary AVM measurements were taken in Gallery A inn blocks 5, 17 and 19 on the right flank. The reservoir water level was 0.57 m below the full supply level during this measurement. The instrumentation for the preliminary measurements consisted of three force balance accelerometers with a nominal sensitivity of 8.8V/g, a purpose-made power supply and a 24-bit National Instruments dynamic signal analyser. It is important to note that the signal analyser, used for the initial site visit, comprise a light portable system with a dynamic measurement range capability significantly lower than the system used for the full measurement. The successful measurement of low-frequency vibrations by this system did indicate that the frequency response functions of the dam will be measurable by the full-scale system under operating conditions from wind excitation alone.

The frequency response spectra results obtained from Gallery A in the frequency range between 0.5 Hz and 12 Hz are shown in Figure 3. Only three dominant peaks are evident within this frequency range. The first peak is at 1.80 Hz; the second peak is at 2.37 Hz and the third is at 3.00 Hz. When comparing it with the results of the full measurement, it is evident that for example the second natural frequency (which is close to the first natural frequency) was missing and no natural frequencies were not evident beyond the fourth natural frequency. This clearly highlights the importance of using measurement equipment with the appropriately dynamic measurement range capability.

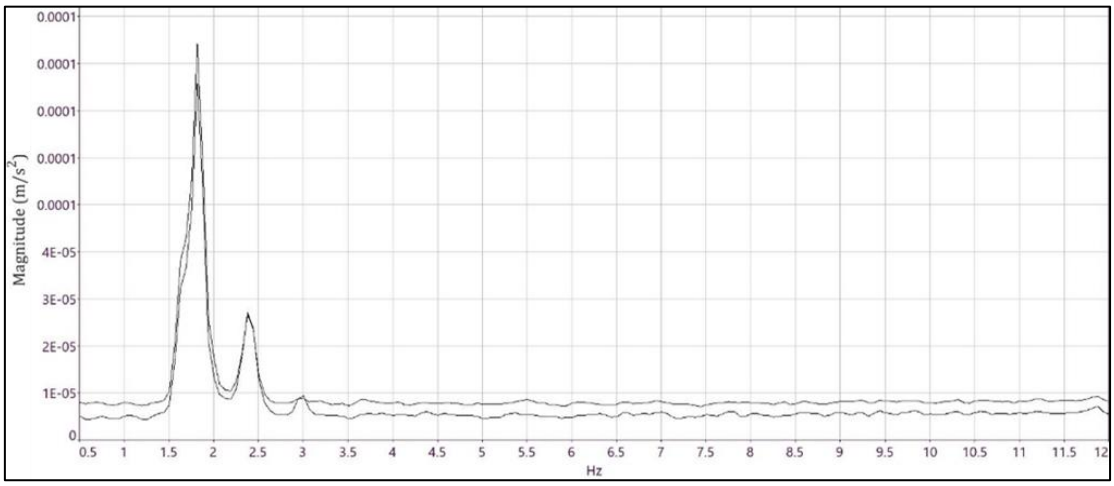

Figure 4. Frequency response spectra measured in Gallery A in the radial direction during the preliminary measurement (Blocks 5 and 17)

3.4 Full measurement and results

3.4.1 Methodology

The full AVM measurement were carried out at in the galleries at three different levels (Galleries A, D and M – see Figure 2) as determined during the initial preliminary measurement. From a sequencing point of view Gallery A was firstly measured followed by Gallery M and ending with Gallery D.

In Gallery A measurements were taken for all the blocks. For selected blocks (basically every second block – 22 in total) 3D measurements were taken (radial, tangential as well as vertical directions). For the rest of the blocks (24 blocks) only radial measurements were taken. In Gallery D radial measurements were taken for all 41 of the blocks at this level. In Gallery D 3D measurements were taken at selected blocks (7 blocks of the 13 blocks) to capture any possible natural ground motions or reservoir induced motions at the foundation level.

The full AVM measurement took 5 days. The reservoir water level ranged from 1.52 m to 1.78 m below full supply level during the measurement. Based on the initial preliminary measurement, it was evident

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that the frequency range of interest would be below 20 Hz, and thus, a sampling rate of 64 Hz was chosen, with each measurement set lasting at least one hour. The radial measurements in Galleries A and D were performed in two sets under two different wind conditions with wind speeds of approximately 25km/h and 40km/h respectively.

The measurement system consisted of 22 force balance accelerometers with a nominal sensitivity of 8.8V/g, a 24-channel purpose-made power supply, a 24-channel Dataphysics signal analyser and a suite of cables. It is important to note that the dynamic measurement range capability of this signal analyser is significantly higher than the system used for the preliminary measurement.

The accelerometers were mounted on heavy aluminium plates with three legs to ensure that each accelerometer was level. Multiple setups were required for Gallery A and D to cover the entire span of the dam with the 22 accelerometers. Dataphysics software was used to record and save the data. Modal analysis software (ME'Scope) was used to extract natural frequencies and mode shapes from the ambient vibration measurements. The instrumentation setup used for this measurement involved over a kilometre of cabling.

A crucial aspect of ambient vibration testing is data quality assurance and control. To this end data was constantly checked and evaluated during the tests. If any of the measurements did not meet the quality checks, the cause of the problem was investigated and fixed, and the test was repeated.

3.4.2 Results

The typical frequency response spectra results in the radial (upstream/downstream) direction obtained in Gallery A are shown in Figure 4, with at least 9 distinct modes of vibration evident. Figure 5 shows the first six natural frequencies and the corresponding mode shapes extracted from ambient vibration acceleration measurements taken in Gallery A.

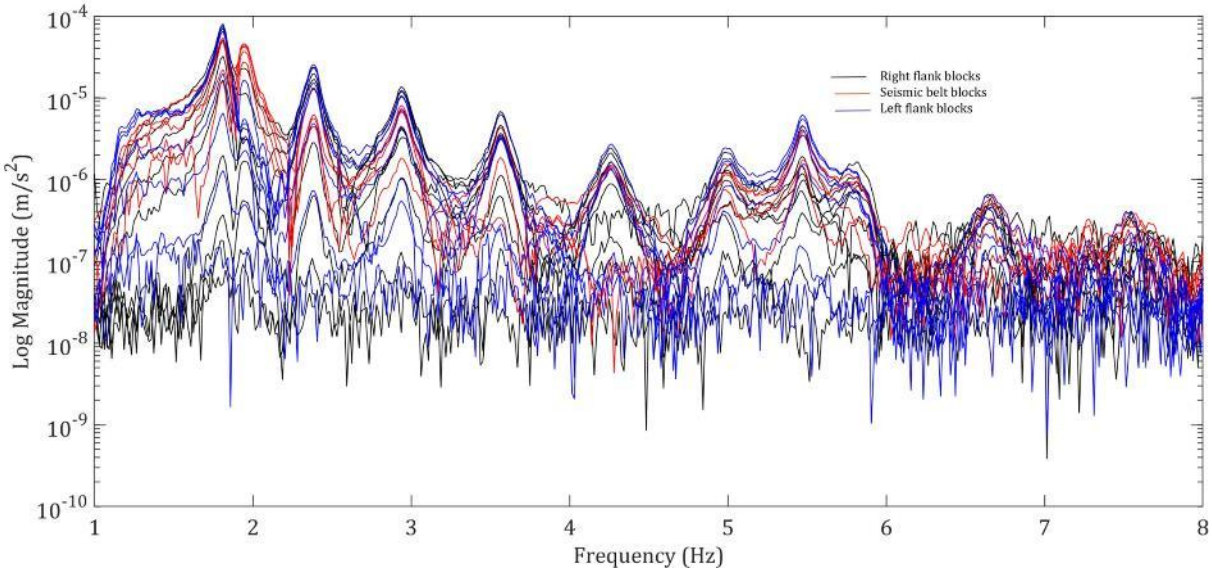

Figure 5. Frequency response spectra measured in Gallery A in the radial direction

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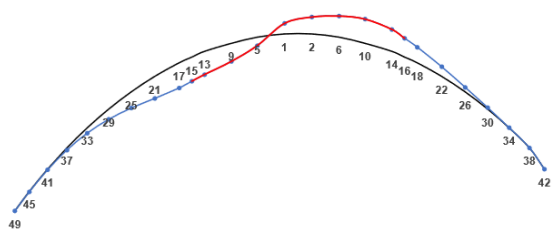
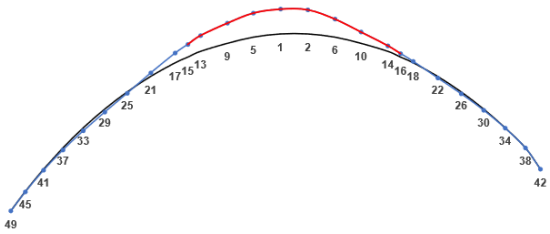
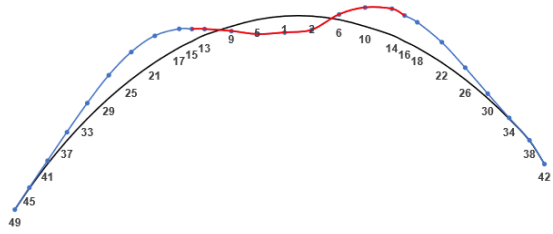


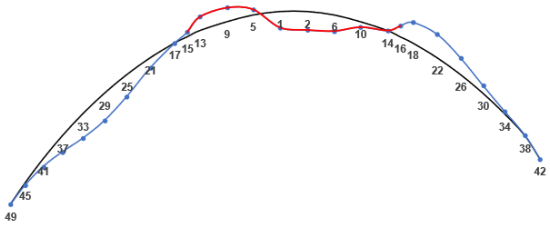
Mode 1: asymmetrical mode, 1.81Hz



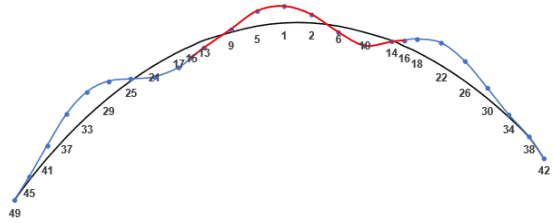
Mode 2: symmetrical mode 1.95Hz



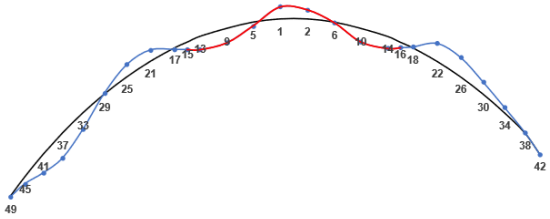
Mode 3: symmetrical mode, 2.38Hz



Mode 4: asymmetrical mode: 2.94Hz



Mode 5: symmetrical mode: 3.56Hz



Mode 6: asymmetrical mode: 4.27Hz

Figure 6: Natural frequencies and mode shapes measured in Gallery A (seismic belt is shown in red)

The modal sequencing of the first 5 modes observed in Gallery A follows the dynamic behaviour typical for a medium thick arch in a wide U-shaped valley (Nyoni & Moyo, 2023). The 6th mode at 5.47 Hz however are different and is dominated by the left flank. This mode was also captured in Gallery D, confirming that it is a structural mode. All peaks identified in the spectra obtained for the tangential direction in Gallery A were associated with frequencies in the radial direction. Thus, there were no independent tangential modes observed. Some peaks were evident in the vertical direction between Block 14 on the left flank and Block 13 on the right flank (coinciding with the spillway portion of the dam) in Gallery A. All these were the same frequencies as those observed in Gallery M (see below). Furthermore, based on the results from Gallery A, no distinct dynamic behaviour as a result of the influence of the seismic belt was evident.

The mode shapes and their sequence as well as the natural frequencies observed in Gallery D were the same as results from Gallery A. This confirmed that the results obtained in Gallery A apply to the whole dam. Overall, based on the measured mode shapes, the right flank appears to be stiffer than the left flank. This is due to the asymmetrical geometry of the arch and the stiffer right abutment foundations.

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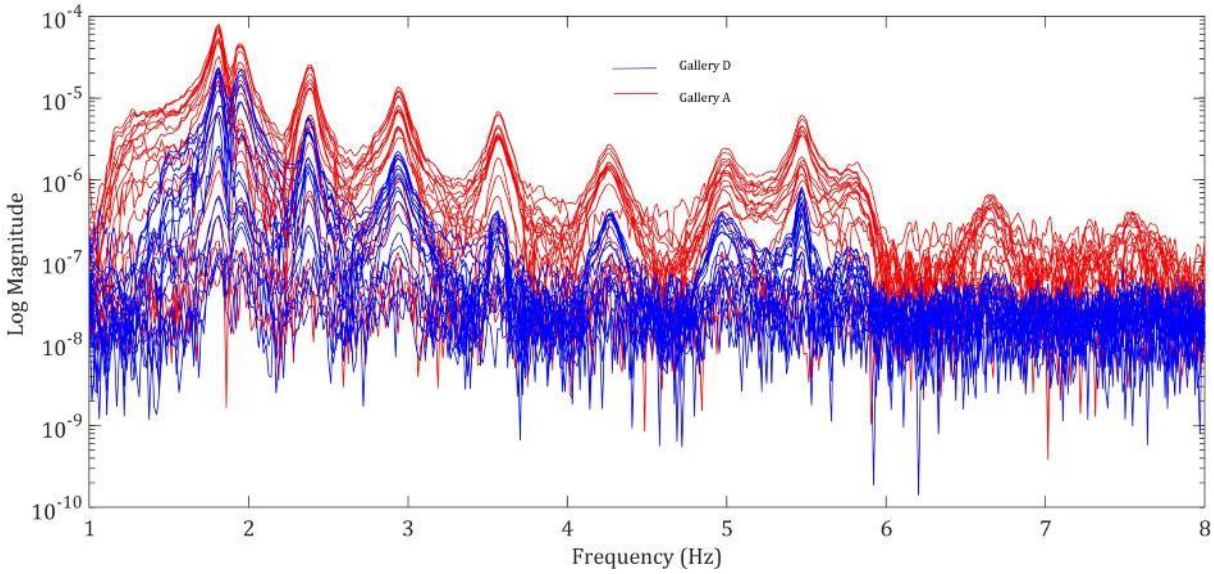

Figure 7. Frequency response spectra measured in Gallery A and D in the radial direction

From the results of Gallery M four harmonic excitations associated with three pumps operating at the foundation level in the dam was observed. Some other odd peaks were also observed. The narrow band of each of these peaks suggests that they are not structural responses. The results showed that there were no independent low-frequency structural modes at the foundation level. This indicates that the stiffness of the foundation material is approximately equal to or greater than the stiffness of the dam. Additionally, since the response measured at the base of the dam proved to be not significant, there was no need to test an additional lower-level gallery.

4. CONCLUDING REMARKS

From the results of the AVM measurement of Katse Dam the following important conclusions were possible:

- The ambient loads operational loads (pumps and releases though the outlet works) were sufficient to obtain relevant ambient vibration monitoring results.
- This study highlighted the importance of performing and initial site visit for planning purposes including performing and preliminary AVM measurement. Future AVM measurements should always include this important step.
- The importance of using a signal analyser with an appropriate high dynamic measurement range during a full detail AVM measurement. Failing this the measurement results most probability would miss some important natural frequencies.
- A comparison between the observed dynamic behaviour and the result of initial theoretical assessment confirms the relevance of the available theoretical methods. In other words, these theoretical methods used provides appropriate results that can be used with confidence in the initial planning of such a measurement. With regards to the modal sequencing proposed by Nyoni & Moyo (Nyoni & Moyo, 2023), the first 5 observed modes follows the dynamic behaviour typical for a medium thick arch in a wide U-shaped valley. The 6th mode however are different.
- Although the results of Gallery and D confirm similar behaviour it is still highly recommended for a structure like Katse Dam to perform measurements at different levels to confirm the validity of the observed dynamic behaviour and possibility identify non-structural behaviour.

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- The mode shapes and operating frequency response functions confirm the following:
    - There is no identifiable influence of the seismic belt on the radial behaviour of the dam; and
    - The right flank appears to be stiffer than the left flank. This is due to the asymmetrical geometry of the arch and the stiffer right abutment foundations.
- The measured response of the dam at foundation level did not indicate any dam structural modes indicating that the stiffness of the foundation material is approximately equal to or greater than the stiffness of the dam. The importance of performing such a measurement at foundation level should also be noted.

5. ACKNOWLEDGEMENTS

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EVALUATION OF THE INTEGRITY OF STEENBRAS LOWER DAM POST-TENSIONING CABLES USING UPDATED FINITE ELEMENT MODEL

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PRESENTER: PROF P MOYO

ABSTRACT

This paper reports the condition assessment of the post-tensioning cables of Steenbras Lower Dam, a 28 m high post tensioned gravity dam with a curved central section located in Gordon's Bay in the Western Cape. A three-dimensional finite element model was developed for the cable assessment, updated using information from ambient vibration testing, laboratory tests and field investigations. The FE analysis showed that the stresses induced in the post-tensioned tendons and their anchorage are within the design limits.

1. INTRODUCTION

This paper reports the evaluation of the post-tensioning cables of Steenbras Lower Dam using a three-dimensional updated finite element model as part of a project implementing the recommendations from the sixth dam safety evaluation of the dam. The assessment of the post-tensioning cables of Steenbras Lower Dam has been a concern from as far back as 1977 when the first assessment of the condition of the post-tensioning cables was done by breaking open three blocks to try inspecting and assessing the condition of the cables. They were found to be in sound condition (Denys et al, 2014). The challenge is that there is no proven method to assess grouted post-tensioned cables and rock anchors (Denys et al, 2014). Grouted post-tensioned anchors must resist four principal modes of failure: (i) steel tendon tensile failure; (ii) bars pulling out of the grout; (iii) rock-grout interface failure and (iv) rock mass uplift (Brown, 2015). The evaluation of the dam's grouted post-tensioned cables to these failure modes is the subject of this paper. The overall assessment of the stability of the dam against sliding, overturning or seismic analysis (critical for this dam) did not fall part of the scope of this study.

This paper is organised as follows: Section 2 describes the case study concrete dam under consideration; Section 3 details the methodology implemented in this paper; Section 4 presents the results; Section 5 a discussion and Chapter 6 presents the conclusions of the paper.

2. STEENBRAS LOWER DAM

The Steenbras Lower Dam (Figure 1) is located in the Hottentots Holland Mountain Range Southeast of Gordons Bay in the Western Cape Province. The dam was completed in 1921, raised in 1927 and in 1954 (Figure 2). The dam has a curved central section with an arc length of 124m and post-tensioned gravity flanks with a total length of 288m, resulting in a crest length of 412m. The gravity section of the dam was post-tensioned, and the cables grouted during the 1954 raise. The dam's maximum height from the lowest level on the downstream side of the wall is 28m. The uncontrolled spillway is 67.7m, and it is located on the left flank of the dam.

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Steenbras Lower Dam lies below Steenbras Upper and forms part of The City of Cape Town's water supply systems. In addition, the Steenbras Dams also provide water to neighbouring areas, as well as the 180-megawatt Steenbras Hydro Pump Station. Steenbras Lower Dam is affected by alkali-aggregate reaction (AAR) and the resulting deterioration well documented by Denys et al (2014).

Figure 1. Steenbras Lower Dam

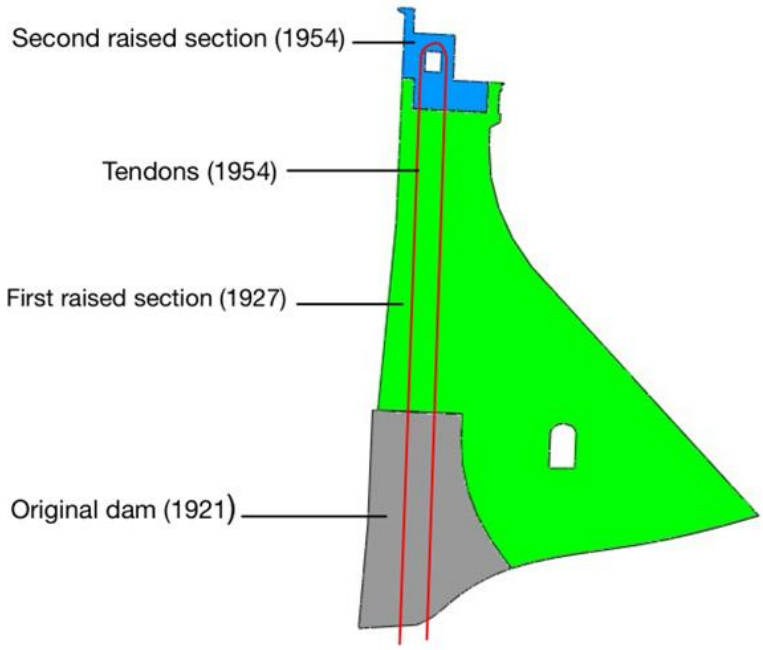

Figure 2. Vertical section through block 11

3. METHODOLOGY

The approach adopted in modelling the structural behaviour of Steenbras Lower Dam included the incorporation of measured or observed information in the modelling process through calibration and validation of the finite element model. The developed finite element model combined 1) information from existing drawings 2) results of laboratory tests of concrete and rock samples extracted from the dam, 3)

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ambient vibration testing, 4) geodetic monitoring data and 5) local weather data to produce the best mathematical representation of the geometry, material properties and loading on the dam.

3.1 Finite Element Modelling

3.1.1 Model Development

The 3D FE model of Steenbras Lower Dam was developed in Abaqus FE Software. The geometrical configuration of the dam was derived from drawings and technical papers on the dam. Figure 3 and Figure 4 show the developed three-dimensional deformable dam system consisting of the concrete dam wall, rock foundation, reservoir and prestressing tendons.

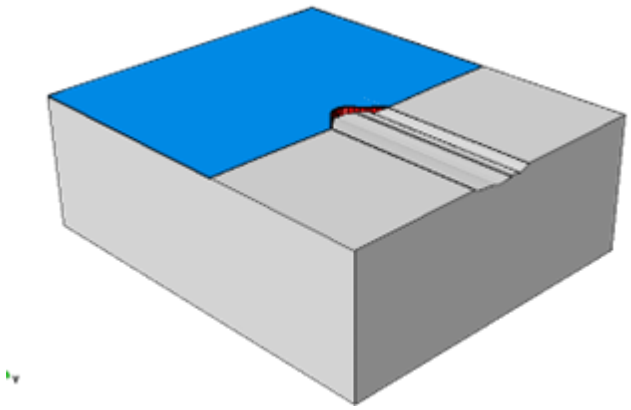

Figure 3. Dam-foundation-reservoir system

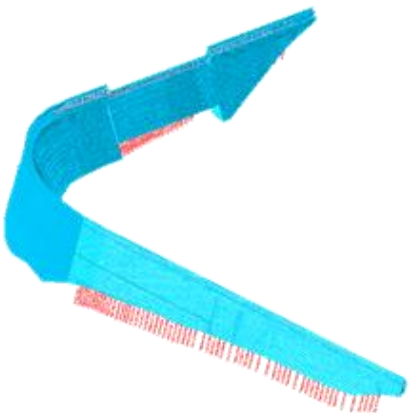

Figure 4. Concrete dam wall with the prestressing tendons

3.1.2 Material Properties

Table 1 provides a summary of the material properties used in the FE analysis.

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Table 1. Summary of material properties used in the finite element model

Material	E (GPa)	f_c (MPa)	f_t (MPa)	α (10^{-6}/K)
Concrete (1921)	40.5	73.5	4.00	11
Concrete (1927)	28.5	50.3	3.20	11
Concrete (1954)	24.6	55.2	3.73	11
Rock (right flank)	24	-	-	-
Rock (left flank)	28	-	-	-
Prestressing tendons	205	-	1450	-

3.1.3 Loading

Thermal Loading

Thermal loading conditions were obtained from OPTRON monitoring data. The data included temperature measurements from August 2013 to March 2022. Figure 5 shows the sun's path around the site on a typical shortest day (21 June) and typical longest day (21 December). The radiation and irradiation energy was applied on the dam wall according to the exposure times on the dam surface. The heat conduction properties of concrete used in the analysis are given in Table 2.

Figure 5. Sun's trajectory (https://www.suncalc.org)

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Table 2. Summary of heat conduction material properties used in the finite element model

Property	SI units	
Specific heat, c	j/(kg.K)	912
Thermal conductivity	W/(m K)	2
Convection coefficient	W/(m^2 K)	23.2
Emissivity, e		0.88
Solar absorptivity		0.65

Alkali-Silica Reaction

Long-term swelling tests showed that all the aggregates used in the Steenbras Lower Dam construction were reactive, with the reactions more than 84% and less than 0.06% expansion potential remaining. The model proposed by Saouma & Perotti (2006) was used to estimate concrete expansion due to alkali-silica reaction (ASR). The model assumes that the evolution of volumetric strain under temperature T, is defined by the expansion curve;

- $$\varepsilon(t)=\left(\frac{1-e^{-\left(\frac{t}{\tau_c(T)}\right)}}{1+e^{-\left(\frac{t-\tau_l(T)}{\tau_c(T)}\right)}}\right)\varepsilon(\infty) \quad (1)$$

The parameters (τ_c), (τ_l) and $\varepsilon(\infty)$ defined as the characteristic time, latency time and ASR ultimate strain respectively, are estimated from the experimentally derived curves and summarised in Table 3.

Table 3. Expansion curve parameters

Location	Current strain $\varepsilon(0)$	Ultimate strain $\varepsilon(\infty)$	Latency time (days)	Characteristic time (days)
Concrete (1921 & 1927)	0.26	0.295	22523	5822
Concrete (1954)	0.33	0.395	13375	6087

Hydrostatic Loading

The hydrostatic load included the hydrostatic pressure on the dam wall and the uplift pressure as shown in Figure 6.

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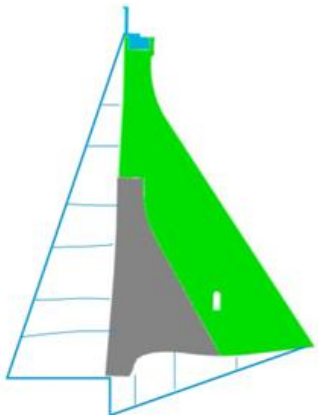

Figure 6. Hydrostatic load

3.2 Finite Element Model Updating

A multistep finite element model updating procedure ((Hattingh et al, 2019) (Moyo et al,2023)) was used as follows:

- **Calibrating of dynamic properties**: updating of the foundation stiffness and the wall stiffness using ambient vibration measurements. The fluid-structure interaction approach was preferred for modelling the water reservoir. The main objective of this step was to minimise the difference between the measured modal parameters (natural frequencies and mode shapes) and theoretical values (at a given water level).
- **Calibration of thermal behaviour:** updating the thermal behaviour of the dam wall using temperature monitoring data. The main objective of this step was to minimise the difference between measured and theoretical temperatures.
- **Calibration of seasonal deformation:** updating the dam's displacement at a given water level and time of year using deformation monitoring data such as geodetic measurements and pendulum readings. This step included the calibration of the long-term AAR-induced deformations. At the end of this step, the coefficient of thermal expansion of the concrete was identified. It is important to note that the stiffness of the dam was not to be adjusted as much as possible in this step, any adjustments to the stiffness are not to change the dynamic behaviour of the dam (i.e. natural frequencies and mode shapes).
- **Model validation:** validation of the calibrated finite element model using long-term monitoring static and dynamic measurements. The model was used to predict the dynamic properties and displacements of at least two historical measurements.

4. RESULTS AND DISCUSSION

4.1 Model Calibration

4.1.1 Dynamic Calibration

Table 4 shows the measured and theoretical values of four natural frequencies of the dam obtained using updated stiffness values. The corresponding mode shapes are shown in Figure 7. There was good agreement between the measured values and theoretical values. Based on these results, the updated foundation stiffness and wall stiffness was used throughout the analysis as representative values.

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Table 4. Measured versus theoretical natural frequencies

Label	Measured frequency (Hz)	Theoretical frequency (Hz)	Difference (%)
1	9.47	9.44	3%
2	9.89	9.95	-6%
3	12.09	12.14	-5%
4	14.2	14.155	4%

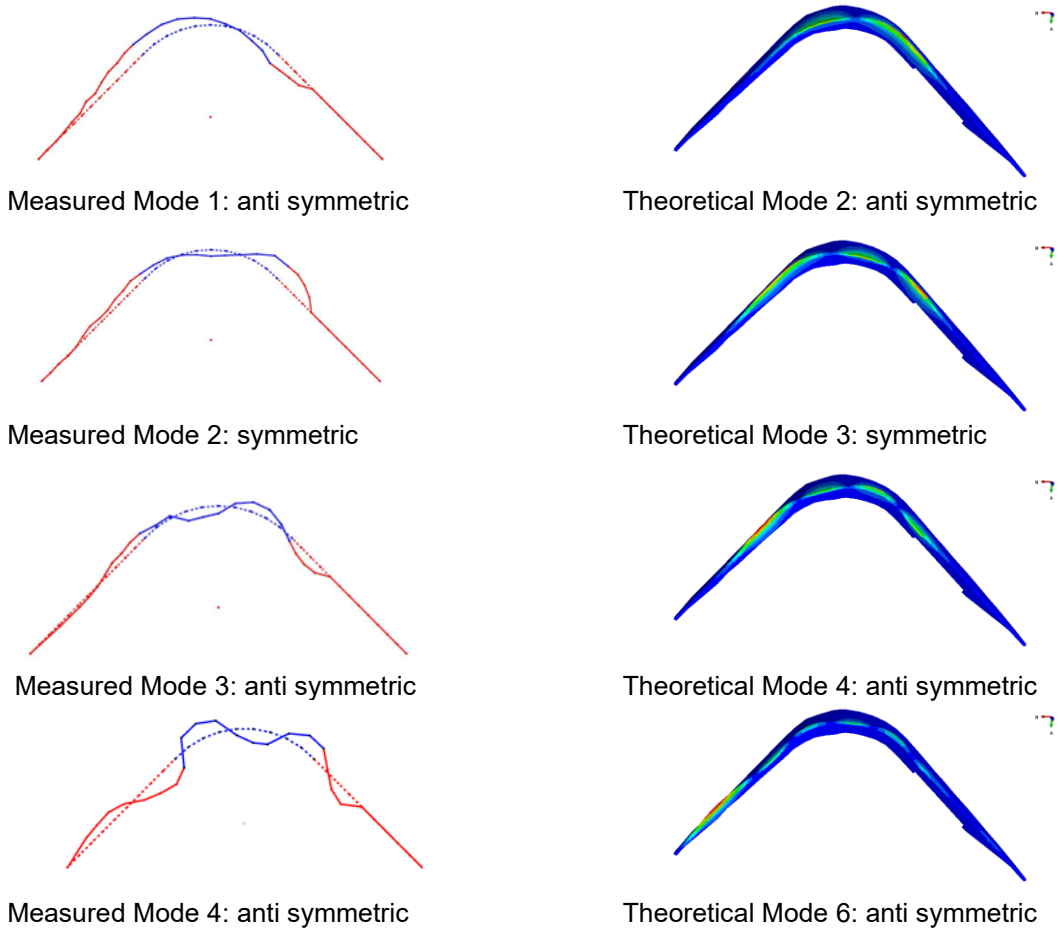

Figure 7. Identified measured vs theoretical mode shapes

4.1.2 Thermal Calibration

Ideally, the obtained temperature distributions should be compared to actual field measurements. This data was not available for Steenbras Dam. Thus, temperature gradients were used to estimate seasonal thermal displacement and stress. Figure 8 shows the thermal displacement of the dam for 1) the dam full in winter, 2) the dam full in summer and 3) the dam under the hydrostatic, gravity and prestressing

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loads. Based on these load scenarios, the seasonal displacement of the full dam is about 10mm (Table 5). While the accuracy of the OPTRON geodetic measurement was not fully established, the measured seasonal displacements were of the order of 11mm. It must be pointed out that the elastic properties established above, and the coefficient of thermal expansion were determined through an iterative optimisation process.

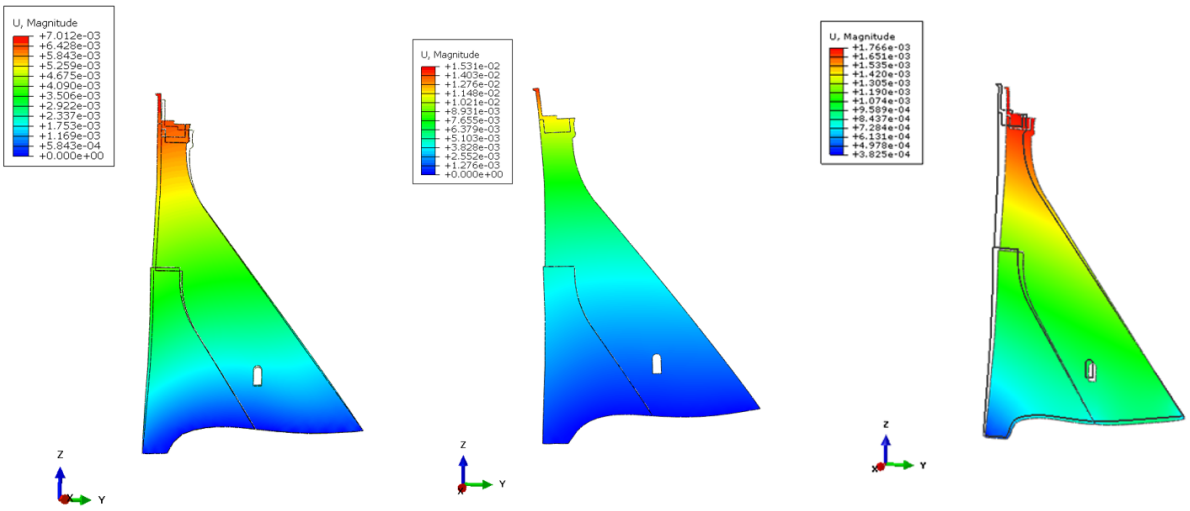

Figure 8. Seasonal displacements

Table 5. Summary of seasonal displacements for the dam full scenario

Load Case	Displacement (mm)
Summer dam full	15.30
Winter dam full	7.01
Hydrostatic, gravity and pre-stressing	1.77
Seasonal range (dam full)	10.01

4.1.3 Alkali-Silica Reaction

The main wall has been permanently displaced upstream by between 5mm and 11mm, while the 1954 wall has been permanently displaced upstream by about 37.9mm. The main wall has been permanently displaced vertically by less than 6mm, while the 1954 wall has been displaced by 8.7mm.

4.2 Stress Analysis

4.2.1 Thermal Stress

The summer and winter vertical stress distributions are shown in Figure 9. The dam's downstream side is under vertical tensile stresses with potential cracking on the downstream face and the 1954 wall. The dam's upstream side is under vertical compression (Max 7.9MPa).

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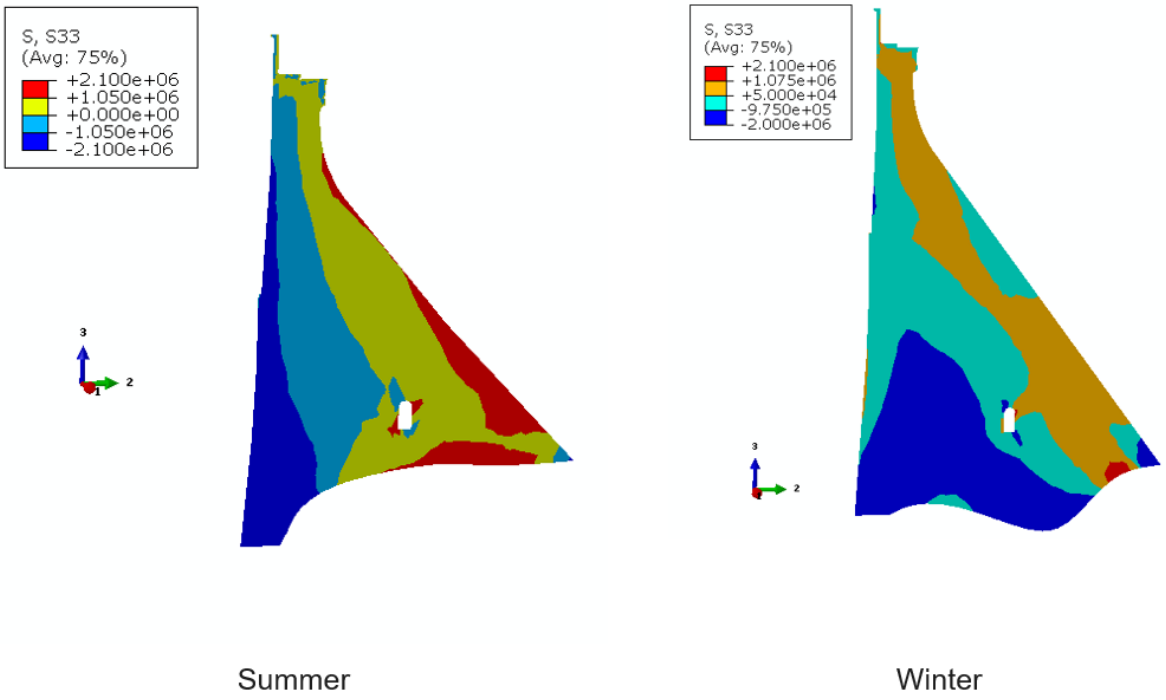

Figure 9. Thermal stress distribution

4.2.2 Alkali-Silica Reaction

Vertical and tangential stress distributions are shown in Figure 10. The dam's downstream side is under vertical tensile stresses with potential cracking on the downstream face and the 1954 wall. The dam's upstream side is under vertical compression (Max 7.9MPa). The tangential stress is compressive, highest in the 1954 wall.

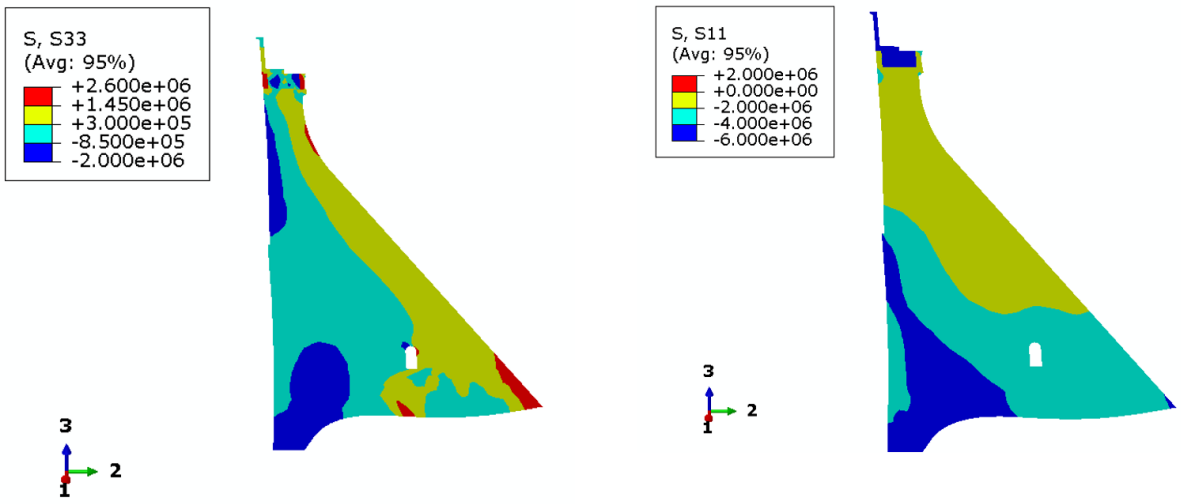

Figure 10. Vertical and tangential stress distribution at the centre section of the curved part of the dam

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4.2.3 Post-tensioned Tendons

Three critical parameters that were checked are the yield stress of the tendons (1450MPa), pullout shear stress between tendons and the grout (1-3 MPa) and the pullout shear stress between tendons and the rock (0.83-2.24MPa). Figure 11 shows the tensile stress in the tendons for the flood load case. The maximum tensile stress is 813MPa, well within the yield stress of the tendons.

Figure 12 and Figure 13 shows the rock's shear and tensile stress respectively, at the critical tendon anchor points in the left and right flanks. In both flanks, the tensile stress in the grout around the anchorage of the tendons is less than 0.26 MPa, while shear stress is less than 0.15 MPa. The tensile and shear stress at the tendon anchorage was less than the recommended design limits of 1 MPa, and 0.33 MPa, respectively.

Under the considered load combinations, the force resultants in the tendons are less than the design limits.

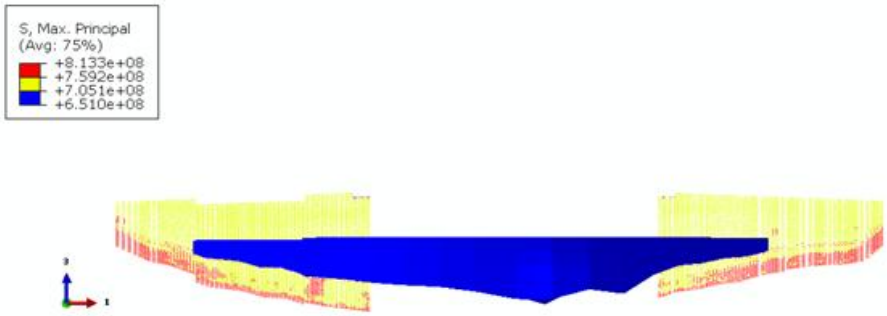

Figure 11. Stress in post-tensioned tendons

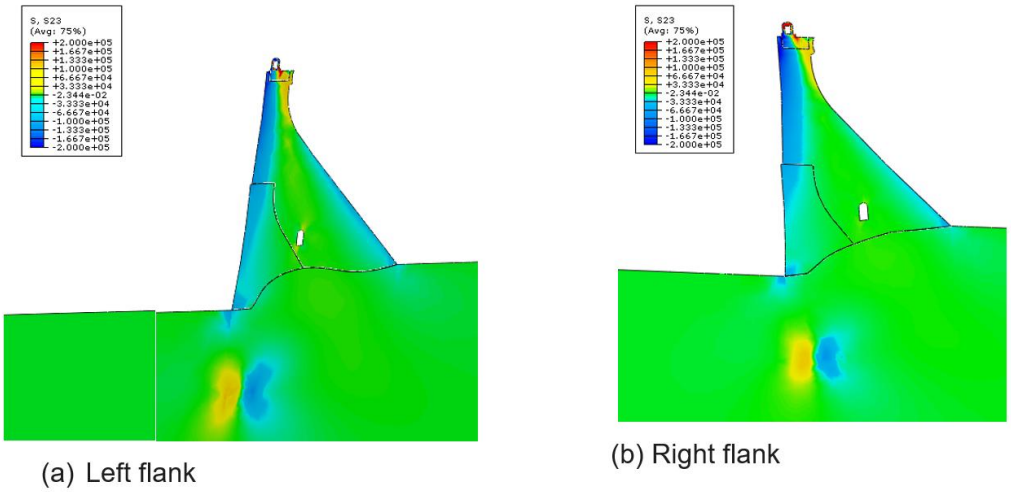

Figure 12. Shear stress at tendon anchorage

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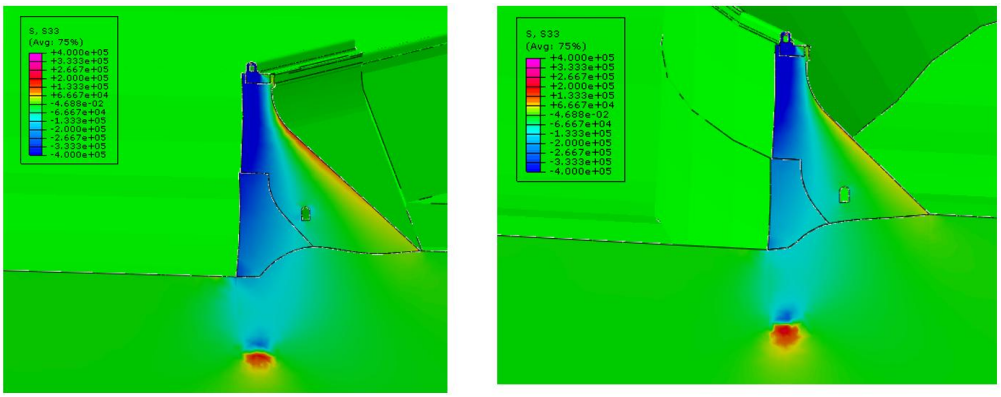

(a) Left flank (b) Right flank

Figure 13. Vertical tensile stress at tendon anchorage

5. DISCUSSIONS

For the four potential failure modes under investigation: (i) steel tendon tensile failure; (ii) bars pulling out of the grout; (iii) rock-grout interface failure and (iv) rock mass uplift (Brown, 2015):

The stresses obtained from the updated finite element model for each failure mode did not exceed the defined limits as per the above discussion. The other important check that was carried out was for the tensile stresses on the upstream face of the wall to check the potential of cracking and moisture ingress that could result in loss of bond along the length of the cable. Figure 13 shows that the dam is in compression on its upstream face for the flood load case.

6. CONCLUSIONS

This paper presents results of the condition assessment of the Steenbras Lower Dam post-tensioning cables using a three-dimensional finite element model of the dam to investigate the dam's behaviour under various loading conditions and to assess the integrity of the post-tensioned cables and the dam wall. Based on the analysis it was concluded for the stresses obtained the prestressing tendons are expected to be in good working order. Cracking on the downstream side of the wall is due to the combined effect of swelling due to ASR and thermal stresses. However, since most of the swelling due to ASR has occurred, further cracking will be limited. It will therefore be critical to carry out a stability check including a seismic analysis.

7. ACKNOWLEDGEMENTS

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RISK INDEXING AS A GATEWAY TO RISK ANALYSIS IN DAM SAFETY

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PRESENTER: ODON HUMAN

ABSTRACT

Dam safety management in developing countries is often constrained by limited data availability, technical capacity, and financial resources. This paper presents a method for integrating several risk informed approaches designed to support risk-informed decision-making (RIDM) for dam safety, particularly in data-poor environments. The method provides a gateway to RIDM approaches in the application of dam safety, particularly in countries where dam safety is still in its nascent state. It enables systematic identification, screening, and prioritization of potential failure modes (PFMs) using structured scoring methodologies. The methodology is particularly suited for environments where full quantitative analysis is impractical, offering a pragmatic approach to informing the prioritizing of dam safety interventions and improving regulatory oversight where resources are limited.

1. INTRODUCTION

The safety of dams is a critical concern worldwide, particularly in regions where infrastructure is aging and downstream populations are increasingly vulnerable. In developing countries, the challenge is often compounded by limited data availability, resource constraints, and a lack of sufficient advanced technical capacity. Traditional risk assessment methods, which often rely on extensive data and sophisticated modelling, supplementary investigations and a large experience-knowledge pool, are frequently impractical in such contexts. Yet, the global trend in RIDM in dam safety and the subsequent drive for implementing such approaches into nascent national dam safety management programmes still under development. The data-, knowledge- and experience gaps creates difficulty in the effective implementation of such programmes. Recognizing this gap, several approaches and tools, such as semi-quantitative risk analysis, risk-screening, and modified potential failure modes and affects analyses have been developed.

The methodology described in this paper was developed as part of the dam safety management programme under development in Angola and integrates several of these techniques and tools in a single approach. It is meant to provide a structured, transparent, and adaptable framework for assessing dam safety risks. The methodology is intended to support professionals in identifying and prioritizing potential failure modes (PFMs), quantifying the urgency of safety interventions, and facilitating consistent regulatory evaluations. It is poised as a gateway for introducing "risk" terminology and thinking, as these can often be jargon heavy and challenging to assimilate while remaining simple and practical. The following sections describe the thinking behind aspects of the method, without going into too much detail, some limitations and practical considerations for application elsewhere.

2. DEVELOPMENT PHILOSOPHY AND STRUCTURE

The development of the semi-quantitative risk analysis methodology was guided by several key principles: simplicity and approachability, systematization, transparency, and adaptability. These principles were selected to ensure that the methodology could be applied effectively in environments

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where data is scarce, technical expertise may be limited, and decisions must often be made under (additional) uncertainty.

The methodology integrates elements of potential failure modes analysis (PFMA), event tree logic, and risk indexing to create a coherent structure for semi-quantitative evaluation. The elements from which it is composed are therefore nothing new. However, their integration and use of the outputs are more novel. Such as using the indexing system to inform relative probabilities of failure due to particular failure modes.

The methodology designed to allow for application at specific dams' individual PFMs as well as across a portfolio of dams. It is divided into three parts, each representing a progressive level of detail and analytical depth.

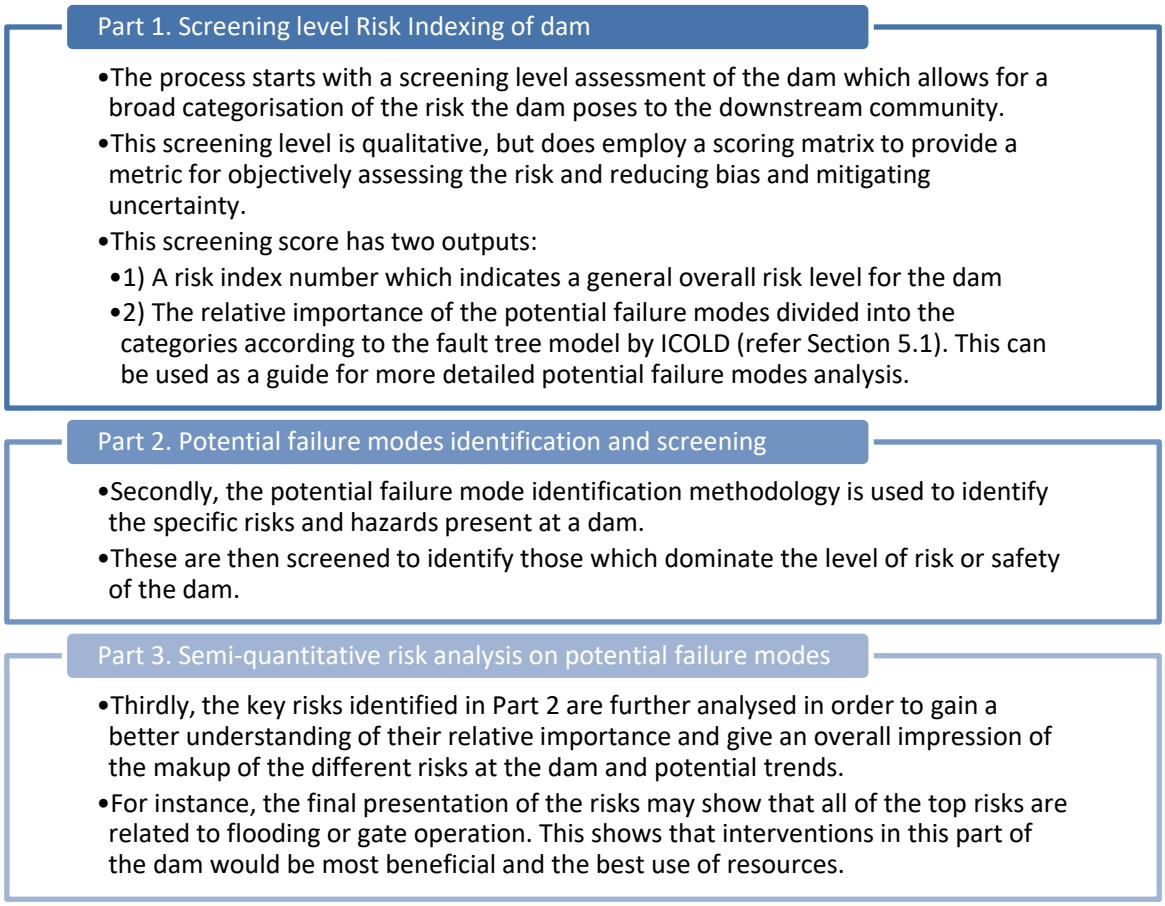

Figure 1. Structure of the methodology

3. PART 1: SCREENING-LEVEL RISK INDEXING

Risk indexing is typically used for high-level risk scoring of individual dams, enabling comparison across a portfolio. These methods are described in several documents such as those by Zielinski et. al. (2022), World Bank (2021). It assigns each dam a numerical score (Risk Index) based on key factors which vary, depending on the indexing system used. The resulting Risk Index helps prioritize dams for further investigation, actions and equipment for monitoring, or investments for upgrades further studies and

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modifications. It's not a substitute for detailed analysis, but a practical tool for RIDM at the portfolio level. This methodology's score is derived from two primary dimensions: the *potential hazard* posed by the dam and its *risk category*.

- *Potential Hazard* is assessed based on reservoir volume, potential loss of life, and environmental and socio-economic consequences. These factors are scored using defined criteria, with higher scores assigned to dams with greater potential impact.
- *Risk Category* considers technical characteristics (TC), existing conditions (EC), and the presence of safety plans (SP). It reflects the likelihood of failure based on structural, operational, and environmental factors.

Each dimension is scored independently, and the final risk index is calculated as the product of these scores. The resulting index provides a coarse but useful measure of relative risk. This approach allows for rapid screening of dams to identify those warranting further investigation.

Beyond the typical use for prioritisation, the risk indexing in this methodology is also used to provide a relative weighting of the importance of the PFMs identified. Even as part of a fully quantitative probabilistic risk analysis, relative likelihoods still often need to be used to estimate probabilities of certain PFMs. Therefore, this approach might even be further developed to supplement such detailed analyses. The output of the weighting process can then be illustrated as a slice of the total theoretical likelihood of failure, as illustrated in Figure 2.

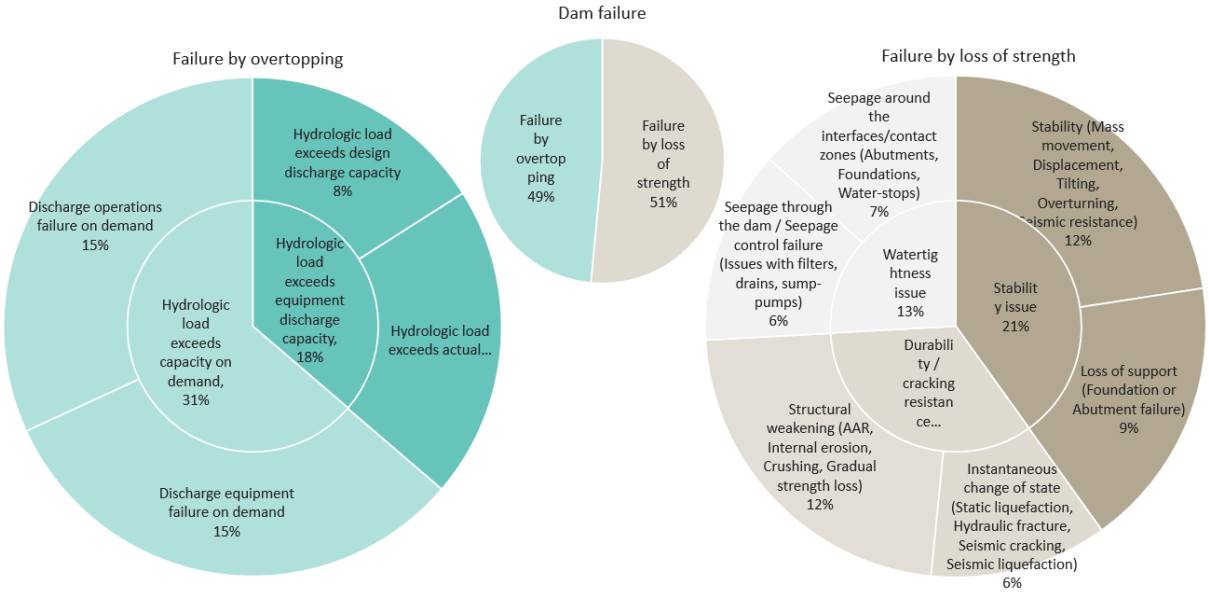

Figure 2. PFM relative importance weighting through risk indexing

In order to do this, each of the criteria comprising the *Risk Category* are mapped to the ICOLD Fault tree model of hazards and failure modes (ICOLD, 2017), shown in Figure 3.

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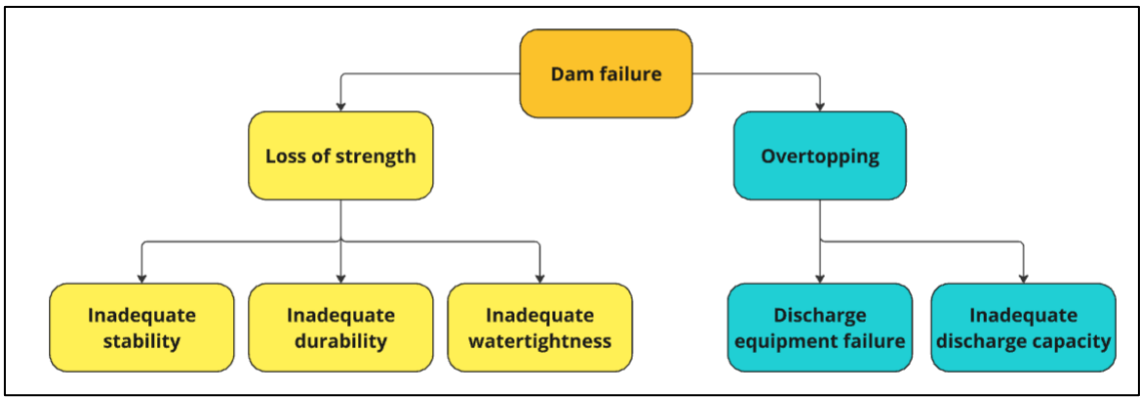

Figure 3. Fault tree model of hazards and failure modes (ICOLD, 2017)

The mapping of the PFMs to the Risk Categories is shown in Table 1. When the screening process has been completed, the scores contribute to the weight of each of the PFMs.

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Table 1. Mapping of Risk Categories to potential failure modes

Potential failure modes / Risk category	Stability	Loss of support	Instantaneous change of state	Structural weakening	Seepage through the dam /Seepage control failure	Seepage around the interfaces/contact zones	Hydrologic load exceeds design discharge capacity	Hydrologic load exceeds actual discharge capacity	Discharge equipment failure on demand	Discharge operations failure on demand
TC: Technical characteristic										
Dam age	x	x	x	x	x	x			x	x
Inflow design flood							x	x	x	x
Length					x	x				
Foundation	x	x	x	x		x				
Height					x	x				
Type	x	x	x	x	x	x				
EC: Existing conditions										
Seismic design	x	x	x	x	x	x	x			
Installed flow control equipment (gates)								x	x	x
Outlet flow control equipment condition									x	x
Spillway structure and reliability	x	x		x			x	x	x	x
Seepage	x		x	x	x	x				
Deformations and settlements	x	x		x	x	x				
Slope deterioration		x	x	x						
SP: Safety Plans										
Design documentation	x	x	x	x	x	x	x	x	x	x
Operation & Maintenance manual	x	x	x	x	x	x			x	x
Organization, staffing and qualifications	x	x	x	x	x	x	x	x	x	x
Safety inspections, monitoring and reporting	x	x	x	x	x	x			x	x

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Dam safety reports, analysis and interpretation	x	x	x	x	x	x	x	x	x	x

4. PART 2: IDENTIFICATION AND CLASSIFICATION OF PFMS

The second part of the methodology focuses on systematically identifying all plausible PFMs for a given dam. The goal is to isolate the most critical PFMs that could lead to uncontrolled release of water. The process follows a fairly standard approach for PFM workshops, and includes the following steps:

- Forming a multidisciplinary working group to ensure diverse expertise.
- Reviewing all relevant dam data, including safety evaluations, design documents, and historical performance, as well as site-specific factors such as geology, hydrology, and downstream consequences.
- Identifying all plausible failure modes, regardless of perceived likelihood.
- Screening PFMs to focus on those with the greatest risk contribution.

The identification process should be exhaustive, ensuring that all credible failure mechanisms are considered. Each PFM is described in detail, including its load scenario, failure mechanism, and structural impact. Once identified, PFMs are screened and classified into three categories:

- Class A: PFMs that are imminent or already occurring. These require immediate attention and, if not already being addressed, should trigger urgent intervention, not further analysis.
- Class B: PFMs that are credible and require further analysis. These are the primary focus of the semi-quantitative risk analysis and are prioritized for detailed evaluation.
- Class C: PFMs that are not credible under current conditions. These are documented for completeness but do not require further analysis unless conditions change.

5. PART 3: SEMI-QUANTITATIVE RISK ANALYSIS OF CREDIBLE PFMS

This step builds on the outcomes of Part 2 where the PFMs which have been classified as credible (Class B) are then subjected to a semi-quantitative risk analysis using a Risk Priority Number (RPN) methodology. The concept of the RPN was introduced and shared largely through the World Bank Laying the Foundations documents (World Bank, 2021), and India DRIP programme (General Water Commission, 2019). The RPN is simply calculated as:

$$RPN = (LI \times RI \times CLI) \times (CI \times CCI)$$

Where:

- LI: Likelihood Index, reflecting the probability of failure based on available evidence and expert judgment. Additionally, the relative importance is guided by the indexing process as described above.
- RI: Rapidity Index, assessing the time available for detection and response, with rapid failures posing greater risk. It reflects the time available for detection, warning, and response. PFMs that could lead to rapid failure with little or no warning are assigned higher scores, reflecting their increased risk.
- CLI: Confidence in LI, accounting for the quality and reliability of the likelihood estimate.
- CI: Consequence Index, evaluating the potential impact in terms of loss of life, environmental damage, and economic cost.

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- CCI: Confidence in CI, reflecting the certainty of the consequence estimate.

Confidence indices (CLI and CCI) are used to adjust the RPN based on the quality and reliability of the underlying data. Lower confidence in the estimates results in higher RPNs, reflecting the increased uncertainty and potential for underestimating risk.

The method is intended to be used by multidisciplinary teams, including dam engineers, hydrologists, geotechnical specialists, and emergency planners. Calibration of the scoring system is essential to ensure consistency and reliability. Calibration is further achieved through training workshops, peer review, and iterative refinement based on field experience. A key part of reducing subjectivity and expert bias and improving reproducibility, is the establishment of a guide for the scoring process. To this end each factor is scored using defined criteria, the selection of which is guided by a rubric for each criteria; only the likelihood is discussed further below in more detail.

5.1 Likelihood estimation

Likelihood estimation is a notoriously subjective process in which many cognitive biases are manifested and deserves further unpacking. Two main issues are noted with regards to likelihood estimation (and evaluation) by Vick (2014), and these problems are present, whether the persons are experts or not:

- Very large numbers and very small numbers are outside ordinary experience and therefore difficult to comprehend accurately;
- Humans tend to have an overconfidence bias, meaning they think they are more sure of their estimates than they should be.

In an attempt to mitigate this issue in the Likelihood Index estimation and communication processes, two measures were implemented. The first measure is that the likelihood index has defined annual failure likelihood ranges divided into "bins" along with qualitative descriptions for guidance, as shown in Table 2. There is no standard in the description and values for terms to describe likelihood (e.g. "unlikely" vs. "Very unlikely"). Therefore, several likelihood ranges and descriptions were investigated in the literature including those by: USBR (2015), INDIA CWC Guidelines (2019), World Bank Technical note 5 (2021), FEMA screening level risk analysis and Vick (2014). The ranges by Donelly et. al. (2022) were found to be well gradated with suitable number of ranges provided with the maximum and minimum likelihood values bookended by typical design values for floods and the typical likelihood of failure for a well-constructed and maintained dam as above.

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Table 2: Likelihood estimation guide

Descriptor	Annual Failure Likelihood	Definition of likelihood
Very High	>= 10^{-2}	There is direct evidence or substantial indirect evidence to suggest that failure has initiated or is very likely to occur.
High	10^{-4} to < 10^{-2}	The fundamental condition or defect is known to exist; indirect evidence suggests it is plausible; and key evidence is weighted more heavily toward "more likely" than "less likely".
Medium	10^{-5} to <10^{-4}	The fundamental condition or defect is known to exist; indirect evidence suggests it is plausible; and key evidence is weighted more heavily toward "less likely" than "more likely".
Low	<10^{-6} to <10^{-5}	The possibility cannot be ruled out, the fundamental condition or defect is postulated. Evidence indicates it is very unlikely.
Remote	<10^{-6}	Several events must occur concurrently or in series to cause failure, and most, if not all, have negligible likelihood such that the failure likelihood is negligible.
Not possible	0	Failure modes ruled out because the physical possibility does not exist, or information came to light which eliminated the concern that had generated the development of the failure mode; or the required load and subsequent structural response is considered so remote as to be non-credible or not reasonable to postulate.

It is noted that the context of the likelihood of the potential failure modes becoming reality is one of a long time span. A typical likelihood of failure for a well-constructed and maintained dam with no known deficiencies is postulated between 1 x10^{-5} and 1 x10^{-6} (Oosthuizen, 2002).

The second measure implemented considers the problem with the collapse of consequence and likelihood into a single value, which may not differentiate between high-consequence-low-likelihood and low-likelihood-high-consequence risks. A visual representation and therefore interpretation of the RPN as a consequence-likelihood (or f-n) graph ameliorates this to some extent. Further, most often risk is shown on graphs as a point value for likelihood and consequence, implying high precision. However, in reality there are uncertainties regarding estimation of both these values and they are therefore more accurately described as distributions and the value for each parameter is the most likely one in that distribution. The depiction of the risk in terms of exact numbers is therefore considered somewhat misleading, particularly for a high level methodology as described here. Therefore, risk may be shown in the graphs as a shaded area rather than a point. This shaded area is adjusted with the Confidence Index (CI), where a lower CI (i.e. when there is less certainty) the shaded risk area increases in size. Inversely, when there is more certainty in the input parameters and estimation, a higher CI decreases the size of the shaded risk area.

6. LIMITATIONS AND PRACTICAL CONSIDERATIONS

While this approach offers a structured and simplified risk informed decision making framework, it has inherent limitations. The semi-quantitative methodology simplifies complex dam systems and introduces a degree of subjectivity, particularly in the scoring of likelihood and consequence indices, as well as biases in the initial definition of scoring systems. Risk levels are gradated coarsely, and the treatment of uncertainty, despite the introduction of confidence indices, still has a measure of subjectivity.

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Several specific limitations with such simplified methodologies are noted:

- Data Dependency: The accuracy of risk scores is highly sensitive to the availability and quality of dam data. Incomplete or outdated records can lead to misleading results.
- Subjectivity: Expert judgment plays a central role in scoring, which can introduce variability and bias.
- Simplification of Complex Systems: Dam behaviour and failure mechanisms are reduced to discrete categories, potentially overlooking interdependencies or rare failure modes.
- Confidence Index Challenges: While confidence ratings aim to reflect uncertainty, they are themselves subjective and may not fully capture data reliability.
- Portfolio Aggregation: Such tools are not optimized for statistically robust aggregation of risk across large dam portfolios, limiting its utility for national-level prioritization.

These limitations underscore the importance of using a broader, integrated risk management strategy and they should complement not replace engineering judgment, stakeholder engagement, and where feasible, more detailed quantitative analyses. Despite their constraints, such tools remain valuable for rapid prioritization and decision support, particularly in contexts where time, data, or resources are limited.

7. CONCLUSION

The standard toolkit for risk informed decision making includes methods such as potential failure modes analysis, probabilistic analyses, event and fault tree methods, which all offer valuable insight into decision making for improved dam safety. However, these tools and methodologies have been aimed and developed in data-rich environments where they can be rigorously applied in a quantitative manner. Consequently, these approaches are ill-suited to environments where data, or resources for collecting additional data, as well as expertise, might be lacking; particularly in nations who are in the process of developing a dam safety programme.

Recent developments in simplified approaches and semi-quantitative tools such as risk indexing offer bridging tools to move towards risk informed decision making where the supporting resources might not be abundant enough. However, these tools are fragmented and do not speak to each other. The methodology described weaves together several of these tools to provide a structured framework for identifying and prioritizing PFMs, analysing them and supporting transparent and defensible decision-making. Its simplicity makes it particularly valuable in developing countries, where traditional methods may have limited applicability and traction.

While not a substitute for detailed analysis, the tool offers a pragmatic starting point for risk assessment and resource allocation. It empowers engineers and regulators to make informed decisions that enhance dam safety and protect downstream communities. Lastly, it provides a starting point for the introduction of risk informed decision-making.

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USING ACCELERATED RESIDUAL EXPANSION TEST METHOD TO PREDICT THE LONG TERM EFFECTS OF ALKALI SILICA REACTION (ASR) ON CONCRETE DAMS.

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ABSTRACT

Accelerated Alkali-Silica Reaction (ASR) residual expansion tools are laboratory based methods aimed at quantifying the remaining ASR in concrete. These tools can be useful in providing a vital insight to dam owners and practitioners on the correct prognosis of ASR affected structure, thus leading to the correct intervention, i.e. monitor, rehabilitate or decommission the dam should the need arise. Furthermore, these tools can be incorporated into existing monitoring systems in order to formulate better dam safety management strategies for dams known to suffer from ASR. Three large dams belonging to the Department of Water and Sanitation were used as a case study to assess the viability of incorporating accelerated residual expansion into the predictive models to quantify the remaining swelling reaction.

1. INTRODUCTION

The Department of Water and Sanitation (DWS) is the custodian of South Africa's water resources. It owns over 320 dams that are classified as dams with a safety risk according the Section 117(a)(i) of the National Water Act no.36 of 1998. To ensure the safety of these dams, the Department of Water and Sanitation is required to comply with Regulation R139: Safety of dams, which outlines the responsibilities of each dam owner to ensure public safety. Through, the mandatory dam safety evaluations conducted over the years, some concrete dams on the departmental database were found to exhibit the presence of alkali-silica reaction (ASR). The majority of these dams are located in the Cape Peninsula, with few in the Free State.

Although South Africa has a very comprehensive legislative frameworks and guidelines on the management of dams with a safety risk, these do not cover how dam owners should care for dams that have been found to have suffered from ASR. This poses a serious challenge for dam owners as ASR tends to affect the durability of these concrete dams over time, thus reducing the serviceability of the affected structures (Oberholster, 2009).

While is it widely accepted that the reaction will cease to be active when all alkalis in the concrete's pore solution have been used, however, it remains a challenge to predict the point at which this reaction will stop. Compounding this challenge, some structures, have exhibited unusual reaction patterns, where the reaction does not seem to be exhausted even after 6 decades in service (Nyoni, 1998). These cases of unusual ASR patterns highlight the need to formulate comprehensive dam safety management tools that can quantify the residual expansion of ASR taking into account the influence of chemical and mineralogical composition of the concrete.

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To evaluate the residual expansion of concrete due to ASR, three concrete dams belonging to the Department of Water and Sanitation were used in the study. The details of the dams are highlighted in Table 1.

Table 1. Dams forming part of study

No.	Name of Dam	Description of dam wall	Location	Year completed (Age)
1.	Kouga Dam	69 m high double curvature arch wall	Patensie, Eastern Cape	1969 (56 years)
2.	Poortjieskloof Dam	33 m high double curvature arch wall	Montagu, Western Cape	1955 (70 years)
3.	Kleinplaas Dam	22.5 m Concrete gravity wall	Stellenbosch, Western Cape	1982 (43 years)

2. DIFFERENT TYPES OF CHEMICALLY INDUCED EXPANSIONS IN CONCRETE

The dominant expansive chemical reactions in concrete are, Alkali Aggregate Reaction (AAR), Delayed Ettringite Formation (DEF) and External Sulphate Reactions. AAR can further be classified depending on the elements driving the reaction, i.e. if the aggregate has silica elements then the reaction would be Alkali-Silica Reaction (ASR) and when the aggregate has carbonate elements then the reaction would be Alkali-Carbonate Reaction (ACR). In South Africa, ASR is the dominant reaction with hardly no known recorded case of ACR. As such the focus of the study was on ASR.

2.1 Alkali-Silica Reaction

Alkali Silica Reaction is defined as a chemical reaction in either concrete or mortar between hydroxyl ions (OH^-) of sodium and potassium alkalis and certain siliceous rocks and minerals such as opal, chert, microcrystalline quartz, and acidic volcanic glass, found in some aggregates (Thomas et al., 2013). For ASR to take place, the following three conditions must be met (Fanijo et al., 2021):

- The presence of reactive siliceous components in aggregates (both coarse and fine fractions),
- Sufficient alkali content from cementitious materials, and
- The presence of moisture, temperature and additives.

ASR mechanism is a multi-stage process involving 1) the dissolution of metastable silica, 2) formation and gelation of colloidal silica, and lastly 3) swelling of gel (Rajabipour et al., 2015). According to Charlwood et al. (1991) the alkaline pore solution of the cement paste, containing almost entirely sodium, potassium and hydroxide ions with very low concentration of other ions such as calcium, diffuse from cement paste into reactive (poorly crystallised) siliceous aggregates, causing the dissolution of the reactive silica and producing an amorphous calcium alkali silica gel ($CaO\text{-}Na_2O/K_2O\text{-}SiO_2\text{-}H_2O$) containing small amounts of calcium.

Reaction rims around aggregate particles are initially formed, followed by the deposition within the particles of the gel, which absorbs water and swells (relative humidity content above about 80% is required), inducing stress and micro-cracking in the aggregate. This, in turn, causes expansion and micro-cracking in the surrounding cement paste, with alkali silica gel flowing into the cracks and pores. Near the aggregate the gel contains less calcium and swells to a greater extent. As the gel moves through concrete, it picks up more calcium, which reduces its swelling potential. The ASR sequence described above is illustrated in Figure 1.

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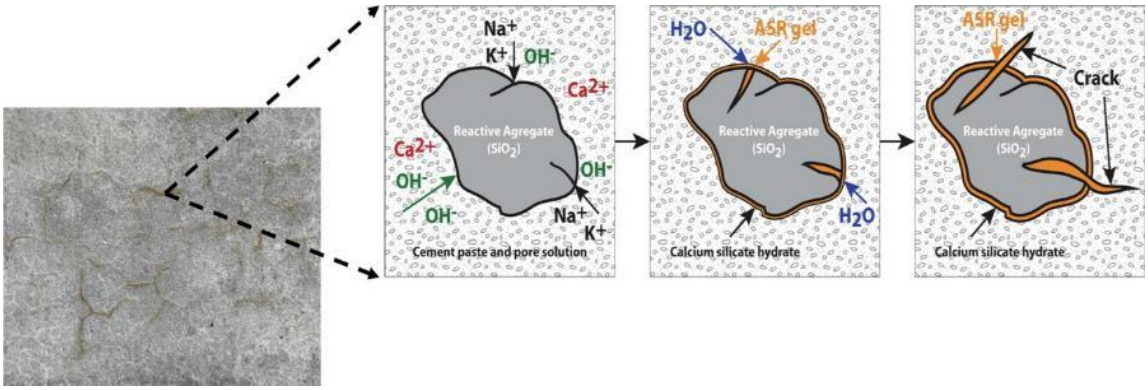

Figure 1. Alkali-Silica Reaction sequence (internet)

2.2 Alkali-Carbonate Reaction

Oberholster (2009) defines alkali-carbonate reaction as a chemical reaction between alkali pore solution and certain argillaceous dolomitic limestones and argillaceous calcitic dolostones. He further notes that the harmful expansion is attributed to a dedolomitisation reaction that occurs when the alkali hydroxides react with small dolomite crystals in the clay matrix. In certain instances, the uptake of moisture by dry clay material may also cause expansion of certain dolomitic limestones which have been opened up by the dedolomitisation reaction. This type of reaction in South Africa can be regarded as rare in South Africa. Li et al. (2022) describe the chemistry of the reaction mechanism as the reaction of alkali with dolomite to produce brucite ($Mg(OH)_2$), calcite ($CaCO_3$) and soluble salt. The newly formed salt can continue to react with calcium hydroxide ($Ca(OH)_2$) from the cement pastes to form soluble alkali, allowing the reaction to continue until the dolomite reacts completely.

2.3 Delayed Ettringite Formation

Taylor et al. (2001) define Delayed Ettringite Formation (DEF) as a formation of ettringite in cementitious material by a process that begins after hardening is substantially complete and in which none of the sulphate comes from outside the cement paste. Incomplete crystallisation of ettringite during the initial stage of hydration enhances the solubility of ettringite, resulting in the formation of ettringite crystals at a later stage during the hardening of concrete material in the presence of water, which consumes internal sulphate and may cause expansion (Sanjeewa and Asamoto, 2024). DEF occurs in concrete exposed to high temperature typically between 70 to 80°C during curing. At these high temperatures, mono-sulphate is formed and higher than normal sulphate is absorbed on the C-S-H phase. The sulphate is then released from the C-S-H as the concrete cools down, and in the presence of water it will react with the mono-sulphate to form ettringite (Stehle, 2018).

3. TOOLS FOR QUANTIFYING ALKALI SILICA REACTION (ASR) IN CONCRETE

The tools for quantifying ASR residual expansion can be grouped into two main categories, i.e., diagnostic tools and prognostic tools. Damage Rating Index (DRI) and Stiffness Damage Test (SDT) are diagnostic in nature and useful in quantifying the reaction that has occurred to date, while Residual Expansion is prognostic and is useful in predicting the remain ASR reaction. These tools are briefly described below.

3.1 Damage Rating Index (DRI)

Sanchez et al. (2017) defines damage rating index as a microscopic analysis performed with the use of a stereomicroscope (about 15-16x magnification) where damage features, generally associated with ASR, are counted through a 1 cm² grid drawn on the surface of a polished concrete section (see Figure

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2). The number of counts corresponding to each type of petrographic features is then multiplied by weighing factors, whose purpose is to balance their relative importance towards the mechanism of distress, such as ASR.

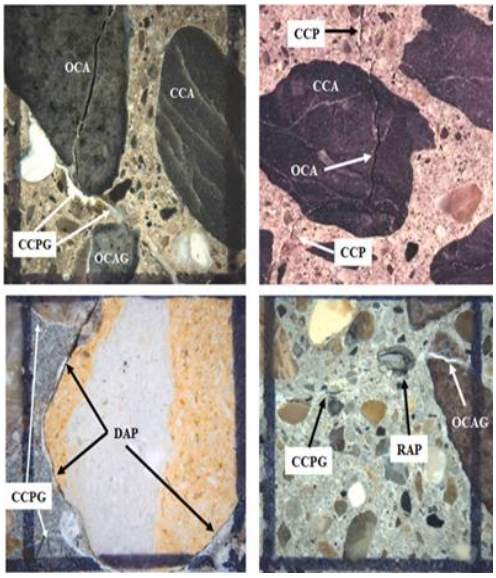

Figure 2. DRI test method analysis (Sanchez et al. (2017)

3.2 Stiffness Damage Test (SDT)

The stiffness damage test was developed to assess the change in stress/strain response of the concrete damaged by alkali-silica reaction or overstress from impact, thermal or other loading (Chrisp et al., 1989). The test procedure as initially proposed by Crouch involved loading of specimen in compression in five loading/unloading cycles at a fixed load of 5,5 MPa and a loading rate of 0.1 MPa. According to Sanchez et al. (2016), Smaoui et al suggested that fixed load be increased to 10 MPa to better quantify damage in concrete due to ASR. Although the revised method as proposed by Smaoui et al. covered a wide range of aggregates, its limitation was that it was based on just one concrete mix-design (i.e. Concrete Prism Test (CPT) as per CSA A23.2-14A or ASTM C1293).To overcome the limitation of the test procedure as proposed by Crouch, Sanchez et al. (2017) proposed the following revisions to the procedure:

- The SDT should be carried out with a percentage of the concrete strength (i.e. 15%, 20%, 30% and 40% at 28-day strength) instead of using a fixed load of 5.5 or 10 MPa.
- The 40% of the design strength is recommended for assessing the concrete damage as it distinguishes damage due to ASR from non-ASR damage.
- Parameters such as hysteria area (HA), plastic deformation (PD) over the five cycles and modulus of elasticity are to be used for diagnostic output results of the test in conjunction with,
- The Stiffness Damage Index (SDI) and Plastic Deformation Index (PDI) which take into account the ratio "dissipated energy/total energy" implemented in the system, providing better the real "damage" of the affected materials, rather than relying on absolute HA or PD values.

3.3 Residual Expansion (RE)

Martin et al. (2017) describe residual expansion (RE) test as a tool used to assess the remaining internal swelling reaction that might occur in affected structures. Numerous test methods have been developed

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to quantify the remaining internal swelling of affected structures or components. Typically, these tests are carried out on cores extracted from affected structures. These tests are conducted in a controlled temperature and humidity in the laboratory. The output results are then used to evaluate the prognosis of the affected structures and are sometimes used as input data in numerical models to predict their mechanical behavior.

To predict the future expansion of concrete from RE test data, the macroscopic phenomena of ASR are modelled within the framework of the porous continua theory, extended to reactive partially saturated porous media. In order to take into account the diffusion conditions such as when the ASR develops, the constitutive model represented by Eq.3.1 was developed and calibrated based on extensive experimental studies done by Larive (Benitez et al., 2022). This phenomenological model was used in this study to estimate the residual expansion of concrete. Figure 3 shows a sigmoidal chemical expansion-time curve plotted using Eq. 2.1.

$$\varepsilon(t) = \varepsilon(\infty)\frac{1 - e^{-\left(\frac{t}{\tau_c}\right)}}{1 + e^{\left(-\frac{t-\tau_l(T)}{\tau_c(T)}\right)}} \tag{3.1}$$

Where:

$\varepsilon(\infty)$ is the AAR strain at the end of the reaction

τ_c is the characteristic time

τ_l is the latency time of ASR swelling

T is temperature in Kelvin (°K)

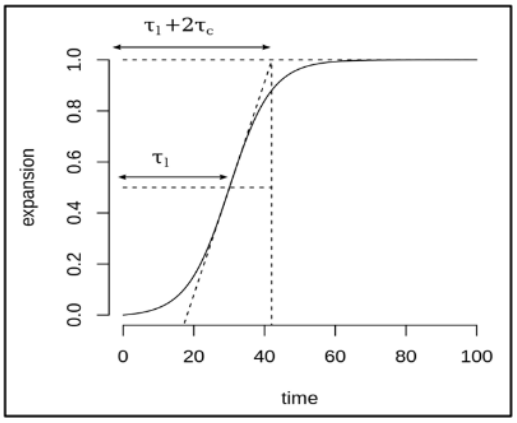

Figure 3. Expansion modelled as a function of time, modified from (Giorla, 2013)

4. APPROACH TO STUDY METHODOLOGY

The steps followed in the study can be summarised as follows: 1) Recovery of core samples, 2) testing of mechanical properties of concrete such as compressive strength, tensile strength and static elastic modulus, 3) determination of concrete pore solution, 4) determination of mineralogical composition of concrete and lastly, 5) conducting accelerated ASR RE test.

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4.1 Recovery of core samples

Locations for core recovery were chosen such a representative results for the analysis are obtained as recommended in ICOLD Bulletin 79 (Bertacchi et al., 1992). Other factors considered included, the degree of concrete deterioration at specific zones in the structure, variability of construction material such as faces or structural elements exposed to the external environment and those that are located inside the galleries away from direct sunlight. Typical setup used for core recovery is shown in Figure 4.

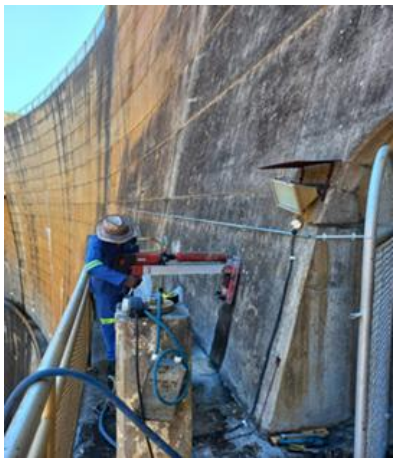

Figure 4. Photo showing core recovery exercise

4.2 Mechanical properties testing

The effect of ASR on concrete is the loss of durability and the reduced serviceability of the affected structures. Therefore, it was considered important for this study to assess the current strength parameters of concrete considering the impact of ASR which might have diminished some of these parameters.

Compressive strength tests were performed on prepared cores 100 mm ± 0.1 mm diameter x 100 mm height in size according to SANS 5865:2006. The mass of the core samples was recorded before the crushing test. The cores were then crushed using an automated compressive testing machine that complies with EN 12390-4 at a loading rate of 0.3 MPa until failure. The maximum failure load was recorded for each core sample. The mean compressive strength was determined using Eq. 4.1. The results obtained from the test are given in Table 4-1.

$$\sigma_{cc} = \frac{F}{A_c} \tag{4.1}$$

Where

σ_{cc} is the compressive strength, in megapascals

F is the maximum load at failure, in Newtons

A_c is the cross-sectional area of the specimen on which the compressive force acts, in square millimeters

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Tensile splitting tests were performed on prepared cores 100 ± 0.1 mm diameter x 155 ± 0.1 mm height in size according to SANS 6253:2006. The rate of loading during testing should be 0.03 MPa. The mass of the core samples was recorded before the crushing test. The cores were then crushed using an automated compressive testing machine (Figure 5) that complies with EN 12390-4 at a loading rate of 0.3 MPa until failure. The maximum failure load was recorded and the mean tensile strength was calculated using Eq. 4.2.

$$f_{ct} = \frac{2F}{\pi \times l \times d} \tag{4.2}$$

Where

F is the maximum load at failure in Newtons,

l is the length of the specimen, in millimetres and

d is the cross-sectional dimensions of the specimen, in millimetres

Figure 5. Setup for tensile splitting test

Static elastic modulus tests were performed on prepared cores 100 ± 0.1 mm diameter and 195 ± 0.5 mm in height according to BS 1881 part 121. The cores were tested using automated compressive testing machine shown in Figure 6, that complies with BS 1881: Part 115. The maximum load obtained from compressive strength was used to determine the upper loading stress for static elastic modulus test, which is taken as a third of maximum compressive stress. The start load or lower stress load of 0.5 N/mm^2 was applied to the sample at rate of 0.7 N/mm^2 until the upper stress load is reached. The cycle of loading and unloading was repeated 3 times until there were no changes in the values of upper and lower strain values. The static elastic modulus was calculated using Eq. 4.3.

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Figure 6. Setup for static elastic modulus

$$\frac{\Delta\sigma}{\Delta\varepsilon} = \frac{\sigma_a - \sigma_b}{\varepsilon_a - \varepsilon_b} \quad (4.3)$$

Where

σ_a is the upper loading stress in N/mm^2

σ_b is the basic loading stress (= 0.5 N/mm^2)

ε_a is the mean strain under upper loading

ε_b is the mean strain under basic stress

The results obtained from mechanical strength tests are given in Table 2. When the static elastic modulus was re-tested at the end accelerated RE tests, it showed 5 to 7% strength decrease.

Table 2: Mechanical strength properties

	Kouga Dam	Kleinplaas Dam	Poortjieskloof Dam
Mean compressive stress (MPa)	40.1	33.2	33.9
Mean tensile stress (MPa)	4.42	3.32	4.96
Mean elastic Modulus (GPa)	35.19 (32.75)*	29.06 (29.06)	28.36 (26.73)

*Elastic modulus test conducted at the end of RE tests.

4.3 Determination of concrete's alkali content, pore solution and pH.

In this study, the immersion test method was preferred, this was motivated by the need to minimise the leaching of alkalis which can prematurely end the reaction. It was therefore important to determine the 'in-situ' alkali concentration of the pore solution for each dam forming part of the study so that this concentration can be replicated in the laboratory for ASR residual testing. Cold Water Extraction (CWE) was used to determine the pore solution of the hardened concrete due to its simplicity, and it can preserve unstable minerals during the extraction process. The CWE process consist of the following 4 steps; 1) grinding the concrete to powder, 2) leaching of the powdered sample by introducing it to a

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known amount of deionized water for 5 minutes 3) filtration, and lastly, 4) analysing the extracted solution and calculations (Plusquellec and De Weerdt, 2017). The results obtained for Alkali content, pore solution composition and pH are given in Table 3.

Table 3: Test results

Dam ID	Alkali content kg/m^3	pH	Pore solution concentrations
Poortjieskloof Dam	6.71	13.41	NaOH[1] = 10.49; KOH[2]= 13.92
Kouga Dam	2.97	13.04	NaOH = 4.96; KOH = 5.79
Kleinplaas Dam	4.84	13.24	NaOH = 8.52; KOH = 8.95

[1] = Sodium Hydroxide; [2] = Potassium Hydroxide

NB: If alkali content is 2.2 kg/m^3 < 3.8 kg/m^3 concrete is moderately reactive, if >3.8 kg/m^3 then concrete is highly reactive. These outcomes correlate with visual observations on site. The pH of greater than 13 that the cement used had a high alkali content containing entirely Na+, K+ and OH- ions.

4.4 Determination of Mineralogical composition of concrete

The mineralogical composition of concrete samples were determined using x-ray diffraction (XRD) and x-ray fluorescence (XRF). Saraugi (2024) defines X-ray power diffraction (XRD) as a rapid analytical technique used for phase identification of mineralogical composition or crystalline structure of samples. The material to be analysed is finely grounded, homogenised and average bulk composition is determined. The procedure is based on constructive interference of monochromatic X-rays and a crystalline sample. The X-rays are generated by a filtered cathode ray tube which produces monochromatic radiation, collimated to concentrate, and directed towards the sample. It is this interaction of incident rays with the sample that produces constructive interference when conditions satisfy Bragg's law represent by Eq. 4.4.

$$n\lambda = 2dsin\theta \quad (4.4)$$

X-ray fluorescence (XRF) was used to determine the percentage volume of minerals in concrete and also validate the concretions obtained through XRD. The procedure involves irradiating the sample by an intense x-ray beam, which causes the emission of the fluorescent x-rays. The emitted X-rays are detected by either energy dispersive or wavelength dispersive detector. The energies or wavelength of the emitted X-rays are used to identify the elements present in the sample, while the concentrations of the elements are determined by the intensity of the X-rays. Table 4 give the results of mineralogical composition of the cores.

Table 4: Mineralogical composition of concrete (mass%)

	Kouga Dam	Kleinplaas Dam	Poortjieskloof Dam
Quartz	96.3	48.1	73.9
Plagioclase	-	24.6	8.6
Chamosite	-	2.9	-
Mica	3.7	24.4	5.9
K-Feldspar	-	-	11.5

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As can be seen from Table 4, the concrete in all dams contains minerals in the aggregate known to be reactive.

4.5 Evaluation of ASR residual expansion

In this study, the pore solution immersion method was used for accelerated residual expansion tests as opposed to air testing at relative humidity > 95% and temperature at 38°C. The adopted procedure is similar to Canada, Laval University methods as proposed by Fournier and Sanchez (2021). The key differences were between the approach used in this study and the Canada, Laval University method was, test temperature, solution composition and duration of the test. The immersion test according to Canada, Laval University method is carried out using 1 mole of Sodium Hydroxide solution (1N NaOH) at 38°C for over 12 months. For this study, the concentration of the pore solution used was equivalent to the concentration in the actual structure.

The samples were immersed in an upright position inside the container filled with pore solution as shown in Figure 7. The samples were then placed in an oven and tested at 60°C for a period of three (3) months. Expansion readings were taken on a weekly basis. To measure the lateral expansions the dial gauge (Figure 8) is zeroed using zeroing steel plate. Then a reference reading is taken at a reference bar (Figure 9). The reference bar reading should be between 0.125 to 0.130 microns, if it is not, then the dial gauge must be zeroed again and a new measurement taken at the reference bar before readings are taken at the sample. At the time of writing this paper, the results of the accelerated expansion were still being processed.

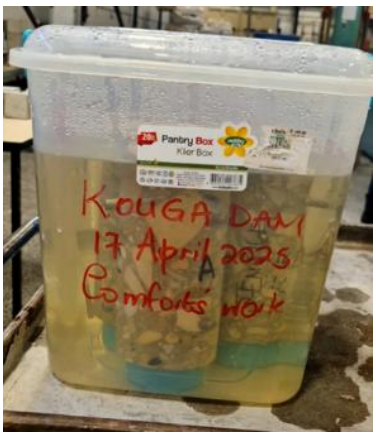

Figure 7. Samples immersed in pore solution

Figure 8. Dial gauge used to measure core expansion

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Figure 9. Reference bar

5. CONCLUDING REMARKS

Based on the work done to date on the study, the following concluding remarks are made;

- It is important for dam safety engineers and owners to work towards developing a proper ASR dam safety management strategy to assist in decision making when dealing with structures that are known to suffer from ASR.
- The study confirmed the reduction of elastic modulus by 5 to 7%, this is an expected reduction in structural durability until all the alkalis are used and the reaction stops.
- The XDR and XRF tests confirmed through mineralogical composition, the presence of reactive minerals in the core samples. Petrographic analysis is still being conducted to positively confirm ASR.
- Once the laboratory data on residual expansion tests is processed, the latency and characteristic time will be determined, thereafter the remaining ASR swelling will be quantified. This is crucial in estimating the long term effects of ASR on structure.
- Preliminary assessment has noted some drawbacks with residual expansion tests especially when testing at 60°C. These include ASR gel exudation which lead to under estimation of actual swelling and the premature halt of the reaction.
- The swelling measuring apparatus should be automated where possible and the frequency of taking measurement should be daily. This will enable the measurements to pick up the transition points when the ASR gel becomes less viscous and starts exuding from the core sample.
- Accelerated expansion tools should be used in conjunction with other monitoring activity such as geodetic survey to ensure there is correlation laboratory and field data.
- Lastly, it must be noted that the accelerated RE tools are not standardised therefore the results cannot be replicated.

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USING UNDERWATER SURVEYING AT DRIEKLOOF DAM FOR DAM SAFETY PURPOSES

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ABSTRACT

For most existing dams, the condition of the underwater slopes is unknown, as it is below the reservoir water level. As a result, when performing a dam safety evaluation, it is assumed that there are no issues with these inundated slopes. The correct use of multibeam echo sounder (MBES) technology allows for highly accurate three-dimensional (3D) point cloud data to be acquired of underwater structures. This data can be used to identify any issues with any inundated structure.

The paper will not only discuss the use of a 3D multibeam underwater survey at Driekloof Dam during the latest dam safety evaluation, as both slopes of this 47 m high embankment dam are largely inundated. But will also highlight the importance of using a holistic approach by also considering the observations of the visual inspection, the importance of also performing a topographical survey, as well as analysing the impact of reservoir water levels on the structural stability of the dam. The results will be discussed, and the advantages of using this technology for dam safety purposes will be highlighted.

1. INTRODUCTION

Driekloof Dam is located in the upper reaches of the Sterkfontein Dam reservoir (see Figure 1), approximately 27 km south-southwest of the town Harrismith in the Free State Province, and functions primarily as the upper storage for power generation in the Tugela-Vaal pump storage scheme. The dam was designed, built and completed in 1979 and is owned, operated and maintained by the Department of Water and Sanitation.

The rockfill embankment consists of a central clay core bordered by rockfill on either side. Soft, sandstone rockfill with a doleritic protective shell is provided as the bottom half of the upstream side, as well as the complete downstream side, while rockfill of doleritic origin has been provided as the upper half of the upstream side. The clay core is protected by multiple sand filters. The rockfill embankment has relatively steep upstream and downstream slopes of 1 V: 1.4 H except for the downstream baffled spillway face, which has a slope of 1 V: 2 H. An uncontrolled broad-crested concrete slab spillway is located in the centre of the wall. Concrete baffles have been provided on the downstream side.

Driekloof Dam embankment crest design level was supposed to be at RL 1 702.44 m according to the available as-built drawings. Sterkfontein Dam's full supply level is set at RL 1 702.00 m. Driekloof Dam embankment crest design level, therefore, was supposed to have a minimum freeboard of 0.44 m.

The previous inspection done at Driekloof Dam in 2009 (Oosthuizen & Hattingh, 2009) identified extensive damage to the downstream side of the crest of the embankment. At the time, this damage was attributed to wave action and extremely high operating levels of Sterkfontein Dam.

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2. RESERVOIR WATER LEVELS

One of the often-forgotten aspects of the dam-related monitoring is water reservoir levels. Unfortunately, the majority of the industry does not give any cognisance to the behaviour of the reservoir water levels when performing dam safety evaluations. Inundation of the non-overspill crest of Driekloof Dam was already identified during the previous dam safety evaluation of 2009 (Oosthuizen & Hattingh, 2009) as a major risk. This risk, unfortunately, was not investigated in more detail.

Figure 1. Driekloof Dam from the right bank (courtesy of Gary Davis)

As part of the recent dam safety evaluation, the reservoir water level of Sterkfontein Dam (which is representative of the reservoir water level in Driekloof Dam) was evaluated. It was evident from the analyses that Sterkfontein and Driekloof Dams are being operated with high water levels for extended periods (see Figure 2). From the results of the annual exceedance probability analysis of the reservoir water level from 2002 (see Figure 3), the water level has exceeded the lowest embankment crest level and submerged the Driekloof Dam embankment 30% of the time since 2002. Also of importance is that the full supply level of Sterkfontein Dam has been exceeded 14% of the time since 2002.

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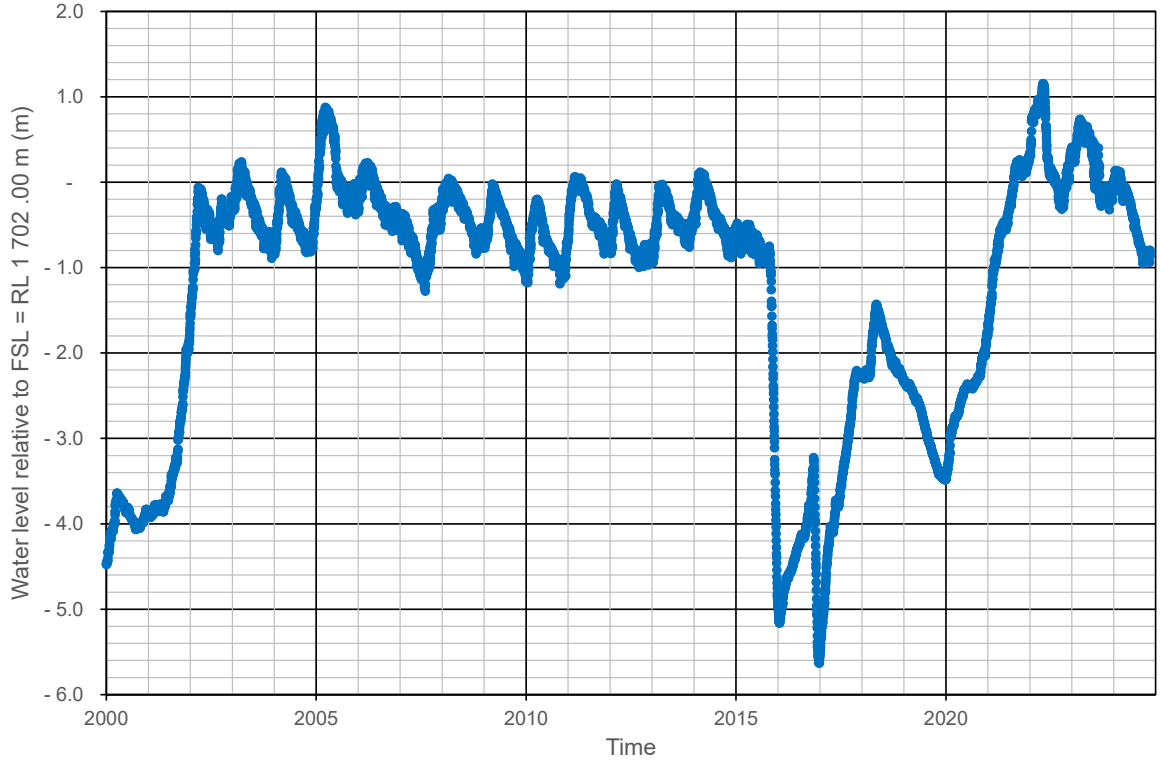

Figure 2. Relative reservoir water levels of Sterkfontein Dam since 2000

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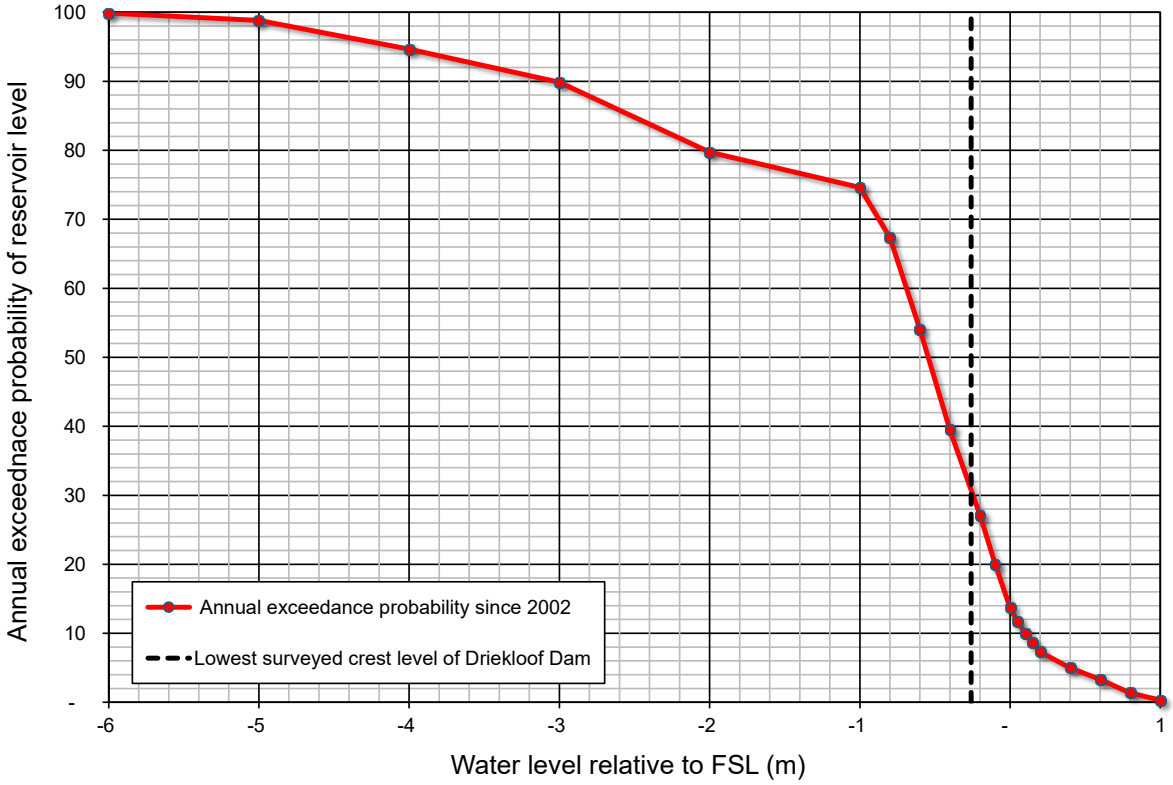

Figure 3. Annual exceedance probability of the reservoir water level of Sterkfontein Dam

3. OBSERVATIONS MADE DURING VISUAL INSPECTION

As mentioned above, the previous inspection done at Driekloof Dam in 2009 (Oosthuizen & Hattingh, 2009) identified extensive damage to the downstream side of the crest of the embankment. At the time, this damage was attributed to wave action and extremely high operating levels of Sterkfontein Dam. The last two inspections in 2003 (Van der Spuy, 2004) and 2009 (Oosthuizen & Hattingh, 2009) also only accessed the right flank due to high water levels in Sterkfontein Dam inundating the spillway of Driekloof Dam.

For the latest visual inspection conducted as part of the compulsory dam safety evaluation (Anderson, 2025), access to a boat was organised. This enabled access to the left flank of the embankment as well. Only the crest was visible during the inspection (see Figures 1 and 4). More extensive damage was evident to the crest of the embankment on both flanks during the inspection (see Figure 4 for an example).

Closer investigation, including the use of bio-location techniques (Hattingh & Oosthuizen, 2016), indicated that the disturbed riprap and round shapes of the damaged areas indicated slip surfaces (instead of the previously assumed wave action). The tops of multiple slip surfaces were visually observed on the downstream slope, with a few on the upstream slope, and their extent was confirmed by bio-location. On both the left and right embankment crests, the top of the concrete wall on the core was also exposed owing to erosion caused by the flows over the crest during high reservoir water levels in Sterkfontein and the daily operation of the pump storage scheme.

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4. SURVEYING

During November 2024, a multibeam underwater survey of the embankment and surrounding area, as well as a topographical survey of the crest of the dam wall, was conducted by Underwater Surveys (2024). The survey was conducted according to International Hydrographic Bureau (IHO, 2022) and THM11 (DOT, 1997) specifications. The specified vertical datum for the survey was the South African Geoid 2010 (SAGeoid2010). All heights were determined relative to the Driekloof Datum sourced from the Department of Water and Sanitation. As all the datums used for the surveys were referenced to Hartebeesthoek94 Datum (WGS 84), no datum transformation was required.

Figure 4. View of the crest of Driekloof Dam during the 2025 inspection

4.1 Topographical survey

From the available geodetic survey results (1979 to 2001 - unfortunately, no surveys have been conducted since 2001), total settlement since the end of construction of around 250 mm is evident for the embankment. The spillway section showed less settlement (approximately 220 mm) over the same period. Most of the observed settlement is evident from 1979 to 1988. Since 1988, the rate of observed settlement has been very small.

From the results of the topographical survey conducted in 2024, the following was evident (see Figure 5):

- The non-overspill crest of the embankment is up to 0.7 m below the design level of RL 1 702.44 m at the lowest points, close to the central spillway, on both flanks. The tops of the concrete spillway walls are between 0.39 m and 0.43 m below the design level of RL 1 703.0 m. As a result, it was concluded that no provision was made during construction for a camber to allow for long-term settlement.

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- More settlement is evident on the downstream side compared to the centre of the crest and the upstream side.

It is important to note that this survey tied into the reference network for Driekloof Dam as well as Sterkfontein Dam, including the reservoir water level gauging. Many surveys done for dam safety management purposes are only done on a local reference system without tying into the dam's reservoir water level gauging reference system.

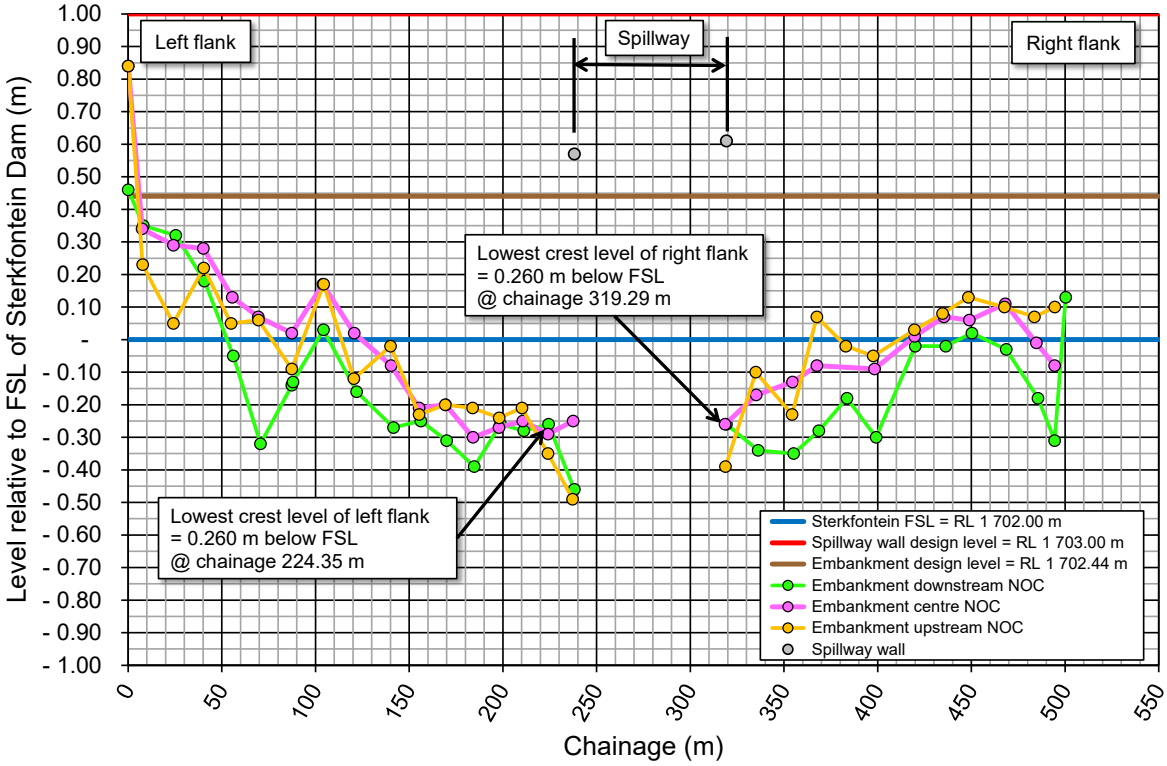

Figure 5. Topographical survey results of the crest of Driekloof Dam (2024)

4.2 Underwater survey

The underwater survey was conducted with a 6.7 m ski-boat customised for survey operations.

The following equipment was used:

- Positioning (Ashore 22/11/25): Leica 1200 RTK Base Station
- Positioning (Ashore): Leica 1200 RTK Rover
- Positioning (Ashore 18/12/25): Leica GS16 RTK Base Station & Leica GS16 RTK Rover
- Positioning (Afloat primary): SBG Ekinox RTK GPS Antenna 1
- Motion sensor: SBG Systems Ekinox-D INS
- Navigation and acquisition system: QPS Qinsy v9.6.5
- Data acquisition: QPS Qinsy v9.6.5
- Sounding system: R2Sonic 2024 Multibeam System (Ultra High Resolution – 400 kHz)
- Sound velocity probe: Valeport SWiFT Sound Velocity Profiler
- Sound velocity sensor: Valeport SVS Sound Velocity Sensor

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Thanks to the use of high-end sensors, an advanced calibration procedure, and robust algorithm design, the survey achieved an accuracy of at least 0.05°. Given the specialist and unknown nature of the project, as well as the potential complexity of the observations, the highest possible accuracy range was used for this survey.

To achieve this, the following were of relevance:

- The multibeam echosounder system comprises a transmitter array, a receiver array and processing and interfacing hardware. The transmitter array transmitted a single wide across-track and narrow along-track pulse of sound. The receiver array and associated hardware computed the precise angles, relative to the array, for each receiver beam, correcting for array bias due to sound speed variances in the water at the sonar array. Bottom tracking was achieved utilising a combination of advanced phase and amplitude-based algorithms (R2Sonic LLC., 2023) (IHO, 2005).
- To account for the 6 degrees of freedom associated with solid body motion, sound speed variances, mechanical installation and bringing the data into a "real world" coordinate system, additional peripheral sensors were included.
- The conversion of multibeam echosounder data from sonar relative to "real world" positions and depths required that the online systems compute the precise position of the acoustic centre of the multibeam echosounder array as well as the pitch, roll and yaw of the array. This was achieved by applying Euler's Equation for 3D Rigid Body Dynamics to the Global Navigation Satellite System (GNSS) position and motion sensor data. The multibeam echosounder data was then corrected for all angular offsets (both static and dynamic), underwent ray tracing (application of Snell's Law to correct for changes in sound speed within the water column) and finally plotted, in near real time, to the navigation and survey suite (R2Sonic LLC., 2023) (IHO, 2005) (Hare, 1995) (Lekkerkerk & Theijs, 2011b) (Lekkerkerk & Theijs, 2011c).
- Due to the relatively steep slopes to be surveyed, for best coverage, the multibeam echosounder installation was customised to allow data to be acquired above the actual array (to the water surface) as well as by measuring perpendicular to the slopes by changing the angle of the array. This innovative method, whilst adding to processing time due to surface detections and multipath noise, has proven successful on multiple campaigns, allowing for 100% data coverage (above and below water) when utilised in conjunction with mobile laser scanning in intertidal zones (such as harbours) and a data gap of >0.5 m on stable water (dams). Regular deployment of the multibeam echosounder (array facing down) would have proven ineffective, as this would provide data to a maximum of 10˚ below the array.
- Also, for this case, instead of using a new, lightweight portable multibeam echosounder, the older, less portable multibeam echosounder was used, which is known to provide a significantly better point cloud definition. The lightweight system may be of use for general mapping purposes but is insufficient for high-definition data analysis, which is critical for dam wall inspections.
- All data was acquired using Real-time Kinematic (RTK) GNSS positioning and processed using Post Processed Kinomatic (PPK) GNSS positioning. All positions were linked to one available ground control point (DK SL1). The base station height was verified against two other ground control points (SL76 and BM8 – BM8 had only elevation information available). The height verification of BM8 showed a height variation of 0.004 m. SL76 showed both a horizontal (0.17 m) and vertical variation (0.040 m). A similar variance in SL76 was observed on both the bathymetric and topographic survey campaigns. As no alternate control points were available, the survey was performed relative to SL1, for which the horizontal accuracy could not be verified to better than 0.17 m.
- The sound velocity profiles were also measured prior to commencing the data acquisition on both the upstream and downstream sides of the embankment. The measurement includes variables such as temperature, pressure and salinity from the surface to the bottom. The

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data is then converted into sound velocity profiles that are used in the calculation process. It is important to note that the data set acquired for this sound velocity calibration could be used as input into reservoir thermal analysis, which is an essential component of the numerical analysis of concrete dams.

- Sonar alignment (Patch Test) calibrations were also performed on both sides of the wall, as the sonar had to be recovered to cross between dams. Inertial Navigational System initialisation and alignment were only required once, as this system remained functional throughout the period of the survey.
- Finally, the quality of the data was computed in terms of uncertainties (total horizontal, total vertical and total propagated uncertainties). The IHO Special Order Survey limits were applied (IHO, 2022; IHO, 2005; Lekkerkerk & Theijs, 2011a). Despite the steep slopes, the 0.5 m gridded data passed a cross-check verification with a standard deviation of less than 0.18 m. The following uncertainties were evident:
    - Total Horizontal Uncertainty of 0.87 m (2DRMS); and
    - Total Vertical Uncertainty of 0.08 m (1DRMS) with a Slope Corrected Uncertainty of 0.25 m.

From the results of the underwater survey, the following was evident:

- It was not obvious from the normal methods used to visually evaluate the survey results whether there were any issues with the slope stability of the embankment. Careful consideration, including the evaluation of the changes in slope, highlighted slope stability issues on both the upstream as well as downstream slopes (see Figures 6 and 7 for a presentation of the slope changes). The extent of the slope failures was clearly evident as a result (see Figure 8). This extent and confirmation would have been unknown without these results.
- Some other useful observations include:

    - Several baffles appear to be damaged or missing on the spillway.
    - Erosion is evident at the toe of the spillway on the downstream side.
    - Some boulders dislodged above the full supply level on the right bank are also evident.

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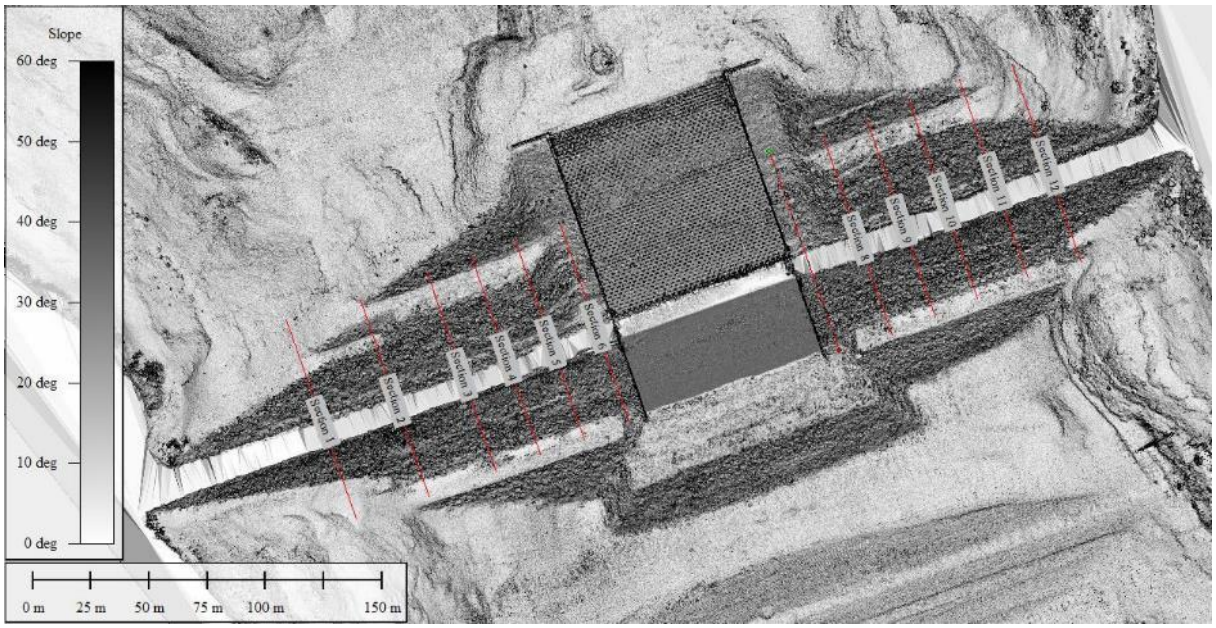

Figure 6. Underwater survey results: Slopes

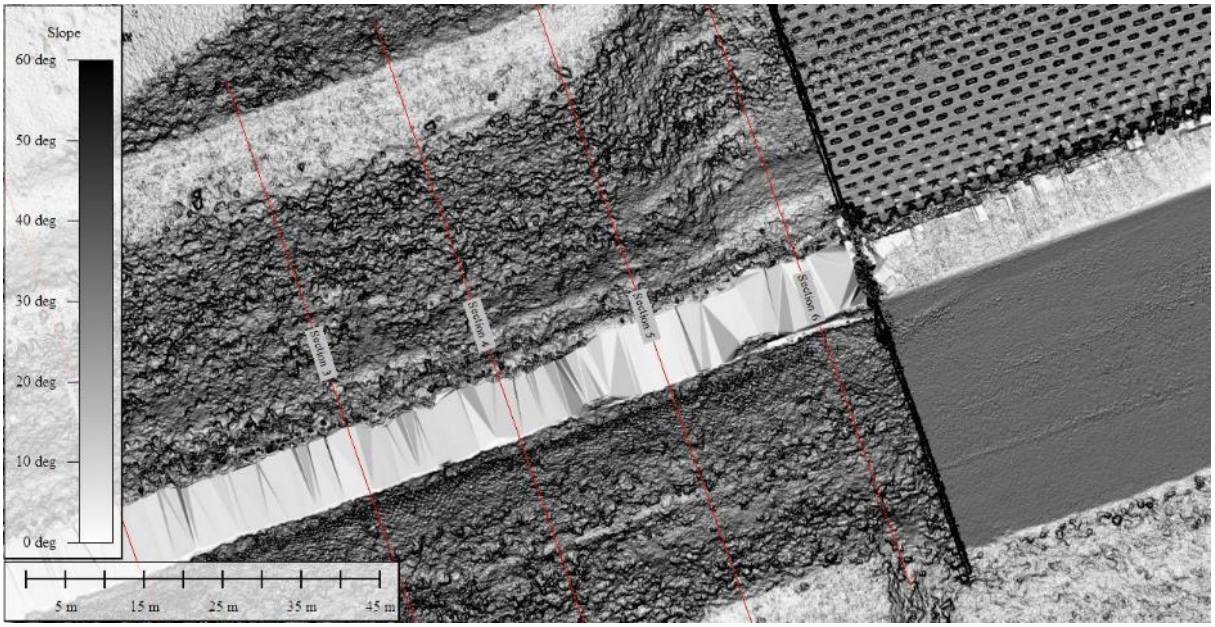

Figure 7. Underwater survey results: Slopes: Detail of left flank

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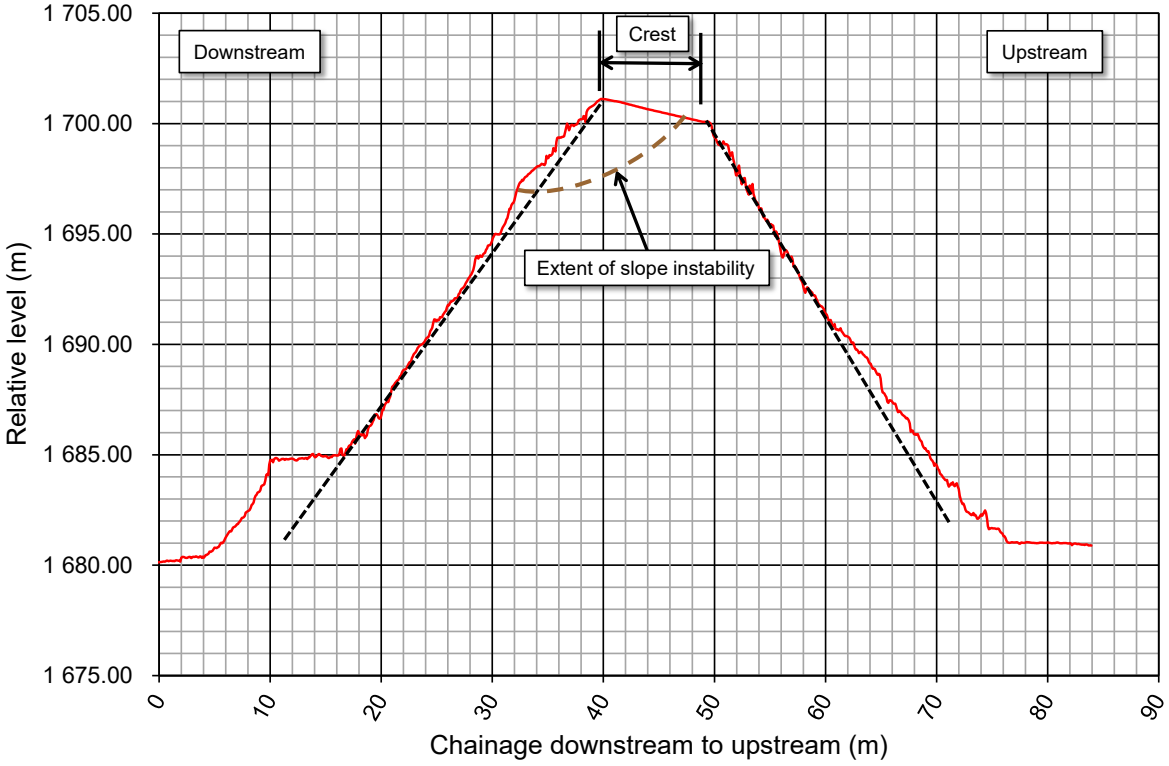

Figure 8. Underwater survey results: Section through left flank

5. CONCLUDING REMARKS

From the results of the surveying exercise and the dam safety evaluation, the following important conclusions were possible:

- The combined results from the topographical and multibeam underwater surveys, as well as the visual observations and the use of bio-location techniques, confirmed the origin of observed damage to the crest of Driekloof Dam – slope failures on both the upstream as well as downstream sides. Without the multibeam underwater surveys, the extent of the slope failures would not have been known.
- Future applications should consider reference topographical surveying as well as lidar surveying above the water level to establish a holistic framework for evaluation of the behaviour.
- Importance of proper reference surveying, tying into the reservoir water level monitoring references system, not just for underwater surveying but also for normal surveying for dam safety purposes. Also, the importance of evaluating reservoir water levels during dam safety evaluations, especially the impact and duration of reservoir water levels.
- When performing underwater surveys, it is essential to do the following:
    - Perform appropriate calibration measurements using a sound velocity profiler to take into account temperature changes along the vertical water profile, as well as for sonar alignment if more than one water body is scanned.
    - Consider the accuracy of the survey as well as the importance of proper ground controls.

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  - The proper conversion of multibeam echosounder data from sonar relative to "real world" positions and depths using international best practice.
  - Changing the deployment of the multibeam echosounder to make provision for relatively steep slopes.
- Future use of the results of sound velocity calibration (vertical temperature profile) as input into reservoir thermal analysis, which is an essential component of the numerical analysis of concrete dams.
- This case history again highlights the importance of a proper survey for dam safety purposes to determine as-built characteristics.
- A dam engineer should clearly communicate his/her needs to surveyors. Careful consideration and interaction/discussion afterwards is essential.

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LESSONS LEARNT FROM THE IMPLEMENTATION OF THE FINITE ELEMENT ANALYSIS OF KATSE DAM

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ABSTRACT

The last Panel of Expert review of Katse Dam in 2018 concluded that the original design calculations although appropriate at the time, does not satisfy all the current international accepted criteria and procedures. This specifically pertains to the abutment seismic stability as well as the overall long-term behaviour of the dam wall during normal operation and extreme conditions. As a result, LHDA initiated a project to perform a finite element analysis of the dam as well as evaluate the monitoring system and monitoring results and determine threshold values for the monitoring system.

This paper will discuss the project implementation methodology followed including the calibration procedure for the finite element analysis. Specific attention will be given to the importance of certain tasks for example the technical procurement methodology and other tasks performed during the execution like the confirmation of the as-built dimensions, the ambient vibration measurement, the site specific seismic hazard analysis and using an experienced expert for review. It will also discuss the issues experienced and lessons learnt during the execution of the project and propose improvements to the process.

1. INTRODUCTION

Katse Dam (see Figure 1), located on the Malibamatšo River in Lesotho, forms part of Phase 1A of the Lesotho Highlands Water Project. Construction of the 185 m high double curvature arch dam started in January 1991 and was completed in April 1997. Reservoir impoundment took place between October 1995 and March 1998. The dam is owned by the Lesotho Highlands Water Commission and operated by the Lesotho Highlands Development Authority (LHDA).

In 2018, a Panel of Experts performed a 10-year review of the dam (Dungar et al: 2019). The review recommended the investigation of the abutment seismic stability as well as the overall long-term behaviour of the dam wall during normal operation and extreme conditions including the development of threshold values for the extensive monitoring system of the dam. Consequently, the LHD initiated a project to satisfy these recommendations by performing a 3D finite element analysis of the dam and its foundation as well as evaluate the monitoring system and monitoring results and determine threshold values for monitoring system.

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2. PROJECT METHODOLOGY

2.1 Procurement methodology

Due to the specialise nature of the project, the LHDA involved one of the members of the 2018 Panel of Experts as a specialist to provide technical assistance during both the procurement as well as implementation of the project. His involvement included the following:

- Assistance with the development of the scope of services as well as technical evaluation criteria,
- Assistance with the technical review of the proposals as part of the technical evaluation panel,
- Assistance during the project implementation including:
    - Provision of all the documentation and information sourced during the 2018 Panel of Experts (including the updated monitoring information). It is important to note that difficulty was experience during the 2018 Panel of Experts to source information as the LHDA is currently populating its information database. The project team also experienced similar issues when trying to source additional information from the dam LHDA's information database,
    - Review of all the task reports (including the inception report) as well as the final project report.

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Figure 1. Downstream view of Katse Dam

Typically for projects of this nature, the scope should not be developed by people that have no knowledge or experience of the nature or effort required for proper execution. Many dam owners of large dams may overlook this important aspect when executing projects of similar nature to an extend that the scope of services fail to provide enough information and guidance to the prospective bidders for the compilation of their proposals.

Also, even if evaluation criteria is provided, there are normally a disconnect between the scope of services, the required experience and the evaluation criteria requiring a subjective evaluation of the proposals. In the majority of cases the people who are doing the technical evaluation of the proposals (evaluation panel), generally do not have the specialist knowledge or experience to properly execute such a subjective evaluation of the technical proposals resulting in an award of a project to the "wrong" team.

As the scope of services is the main component on which the procurement as well as execution is based, the following was considered in developing the scope of services:

- Although background of Lesotho Highlands Water Project, information on Katse Dam background information on the behaviour of Kaste Dam as well as the objectives of the project was given, no detail technical information was shared with the prospective bidders,
- A detail scope of services was provided including the identification and description of each of the main tasks needed to execute the project. In this case 9 tasks were identified including

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inception as well as capacity building and training. The expected reporting requirements were also specified,

- As a project of this nature relies on appropriately experienced specialists, 5 key experts were identified. For each expert the minimum relevant experience, the required experience as well as the minimum professional qualification was specified. Furthermore, a distinction was made between whether a requirement was an absolute requirement (using the term 'must have') or whether it was a reference point for evaluation purposes (using the term 'should have'). Furthermore, it was required that each expert participated in at least one similar project over the last 10 years for a concrete arch dam at least 150 m high. The CVs for each expert also was required not to be more than 3 pages focussing on only relevant experience,
- A guideline/specification was given for the preparation of the technical proposal to ensure both a level playing field when evaluating the technical proposals as well as simplify the evaluation (in other words enabling comparing apples with apples when performing the evaluation). This guideline was directly related to the evaluation criteria provided. This guideline gave specific guidance on the composition of the technical proposal as well as the maximum length of each of each section as follows:
    - Introduction (2 pages),
    - Past experience (10 pages),
    - Methodology (12 pages),
    - Team capability (5 pages), and
    - Capacity building and training (2 pages).
- The bidders were also required to state what tools and software they would be using for the project.

As already stated above, in many cases the technical evaluation of similar projects is done subjectively by an evaluation consisting of people that don't have the relevant experience and knowledge on the specialist subject matter. The following main technical evaluation criteria categories were considered:

- Past relevant experience of company,
- Approach, methodology and project organisation,
- Team composition,
- Capacity building and training,
- Tools and software,

Each of these main technical evaluation criteria were further divided into relevant sub-criteria **(see Table 1 below).**

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Table 1. Technical evaluation criteria

Criteria no.	Criteria description	Points
1.	**Past Relevant Experience**	**25**
1.1	Long-term behaviour under normal as well as extreme load conditions including seismic loads of large concrete arch dams	4
1.2	Numerical modelling especially 3D finite element analysis of large concrete arch dams, their foundations and reservoir water interaction	5
1.3	Site specific probabilistic seismic hazard analysis	4
1.4	Ambient vibration testing and analyses	4
1.5	Evaluation of the existing monitoring information including multiple regression analysis	4
1.6	Implementation of monitoring evaluation system (threshold values)	4
2.	**Approach, methodology, and project organization**	**30**
2.1	Compliance with Scope of Service	5
2.2	Appropriate and detailed method statement for each task	15
2.3	Detailed work programme, project organization staff inputs and deliverables	10
3.	**Team Composition**	**30**
3.1	Team Leader/Specialist Dam Engineer	10
3.2	Numerical Modelling Specialist	4
3.3	Seismic Specialist	4
3.4	AVM Specialist	4
3.5	Surveillance Specialist	4
3.6	Technical Support Staff	4
4.	**Capacity building and training (effective knowledge transfer)**	**10**
5.	**Tools and software**	**5**
	TOTAL	100

To assist the technical evaluation panel to evaluate each proposal as objective as possible (instead of the normal subjectivity), a scoring guideline was developed by the technical assistance specialist for each sub-category. This was discussed and agreed on by the technical evaluation panel. This scoring guideline made sure that only the relevant information provided (whether it was relevant experience or relevant information pertaining to the methodology or tools and software to be used), actually was scored. It left very little room for improper technical evaluation as a result of a lack of knowledge and experience.

For example, for the "Past experience" main category, "Long-term behaviour under normal as well as extreme load conditions including seismic loads of large concrete arch dams" sub-category (maximum of 4 points), the following scoring guidance was provided:

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- 2 projects or more in the last 10 years of concrete arch dam more than 150 m high – 4 points,
- 1 project in the last 10 years of concrete arch dam more than 150 m high – 3 points,
- 2 projects or more in last 10 years of concrete arch dam more than 100 m but less than 150 m high – 2 points,
- 1 project in last 10 years of concrete arch dam more than 100 m but less than 150 m high – 1 point,
- 2 projects or more in last 10 years of concrete arch dam more than 70 m but less than 100 m high – 1 point.

For the "Team capability" main category provision was made for different options for example for the "Team leader/ specialist dam engineer" sub-category, the following scoring guidance was provided considering the number of concrete arch dams with their maximum heights and whether this included behavioural analysis or not):

- More than 15 years of relevant experience with more than 2 concrete arch dams higher than 150 m, including behavioural analysis – points 10,
- More than 15 years of relevant experience with more than 2 concrete arch dams higher than 150 m, no behavioural analysis – points 8,
- More than 15 years of relevant experience with more than 2 concrete arch dams higher than 100 m, including behavioural analysis – 8 points,
- More than 15 years of relevant experience with more than 2 concrete arch dams higher than 100 m, no behavioural analysis – 6 points,
- More than 15 years of relevant experience with more than 2 concrete arch dams higher than 70 m, including behavioural analysis – 4 points,
- Any other experience that does not fulfil any of the above – 0 points.

The scoring guideline therefor made sure that those with proper experience or proper descriptions of their methodology is properly rewarded compared to those with lessor experience and/or insufficient descriptions of their methodology.

Each of the technical evaluation panel still independently evaluated each of the technical proposals using the scoring guideline. This methodology followed proved to be very successful and is highly recommended to other dam owners to ensure that the correct technical evaluation of the proposals are done.

The procurement process followed the standard LDHA procurement procedure. For this project it included the following:

- A single step process with a compulsory virtual briefing session for which attendance had to be registered to receive a link to meeting,
- The LHDA only advertised the project on their website. It is important to note that for a specialist project like this, there are less than 20 consulting companies in the world that will be able to successfully execute all the components of such a project. A few days before the compulsory briefing session it became clear that only a couple of local South African consulting companies were aware of the tender. The technical assistance specialist then reached out to other international companies through his international network especially his involvement in relevant ICOLD technical committees, to make them aware of the tender, and
- As mentioned above, no additional technical information was provided to any of the bidders that attended the compulsory briefing session.

In hindsight it would have been advisable to provide each of the bidders with more relevant technical information. This would have led to a better understanding of the complexity and potential issues. A two-step procurement process involving an initial expression of interest phase and then a second formal

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bidding phase by those bidders that proved their technical proficiency would have been better for a project of this nature as more relevant technical information could have been provided to those participating in the second bidding phase by signing non-disclosure agreements.

It is also advisable for future specialist projects to make all the relevant consulting companies aware of such a tender well in advance.

2.2 Finite element analysis calibration

The current international best practice pertaining to the calibration steps used in such a finite element analysis of a concrete arch dam generally involves the following:

- Firstly, static calibration using the results of static monitoring of the dam for the calibration of the theoretical finite element model. This typically results in changes to the stiffness of the structure and/or its foundation to achieve calibration without consideration of the actual inherent stiffness of structure and its foundation (in other words in many cases the calibrated stiffness used for the concrete in these models is not representative of the concrete used), and
- Secondly, thermal calibration also using the results of the static monitoring of the dam for the thermal behaviour of the dam.

The dynamic properties of dams, such as natural frequencies and modes shapes, have long been recognized as essential parameters for calibrating finite element models of as-built dams during dam safety evaluations (Servern et al, 1980). Extensive research has been conducted at the University of Cape Town into the dynamic behaviour of large concrete arch dams (based on the methodology used for the theoretical analysis of bridges) since 2007 (Moyo & Oosthuizen, 2013, Moyo et al, 2016, Moyo & Nyoni, 2020, Bukenya & Moyo, 2023, Nyoni & Moyo, 2023). This research highlighted the use of the results of ambient vibration testing as an essential first step in the calibration of the theoretical model in the proper behavioural analysis of existing dams. The calibration steps adopted for this project involved the following:

- Calibration of the dynamic properties using the results of the AVM testing and available material properties obtained during design and construction. In a perfect world considering the age of the concrete and the known time-dependent nature of properties of concrete, additional tests should have been performed on the concrete during this project,
- Thermal calibration of the concrete structure (Cassels et al, 2024). Observed thermal behaviour was available from the static monitoring results, and
- Lastly, deformation calibration/validation using the results of the static monitoring.

This "new" approach to calibration has been successfully used for the analysis of a number of dams including Oanob Dam in Namibia (Hattingh, Moyo, Shaanika, Mutede, & Muir, 2019), Roode Elsberg Dam (Moyo et, 2013, Moyo et al 2016, Moyo et al, 2024) and Kouga Dam (Moyo & Nyoni, 2020) in South Africa. It is important to note that this "new" approach was specified in the detail scope of service provided as part of the Terms of Reference to each of the prospective bidders.

This "new" approach was successfully implemented during this project. It is important to note that these three calibration steps are discussed in more detail by other technical papers presented at the SANCOLD conferences of 2024 and 2025 as follows:

- Calibration of the dynamic properties (Cassels et al, 2025a),
- Thermal calibration of the concrete structure (Cassels et al, 2024), and
- Deformation calibration/validation using the results of the static monitoring (Cassels et al, 2025b).

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2.3 Other important tasks

Not only did this project use a "new approach" to the calibration of the finite element model, but other important tasks were also included that are not always included when performing such an analysis. These included:

- A laser and underwater sonar scan to confirm the as-built dimensions of the dam and provide appropriate input for the compilation of the finite element model (Schalekamp et al, 2023). As part of this task baseline of the plunge pool was also performed as recommended by the 2018 Panel of Experts evaluation. This will in future be used as a baseline to determine the extent to which erosion is taking place during significant spilling events over the uncontrolled ogee spillway,
- As a major motivation of the finite element analysis was to evaluate the dynamic behaviour of both the dam wall as well as the foundation (specifically wedge stability of the right flank), using the appropriate seismic loading was of utmost importance. As a result, a site specific seismic hazard analysis was carried out (Kijko et al, 2023) as the last site specific seismic analyses was carried out during design. As is the case with flood hydrology, it should be standard practice to update the site specific seismic hazard analysis on a regular basis (every 5 to 10 years) as new information becomes available,
- To enable the dynamic calibration of the finite element analysis, an ambient vibration measurement was performed at the dam (Moyo et al, 2025). From the results of the ambient vibration measurement, the natural frequencies and modes shapes for each of the natural frequencies were determine. These results also provide an insight into the dynamic behaviour of the structure and the inherent stiffness of the dam wall and its foundation.

3. ISSUES EXPERIENCED DURING EXECUTION

As is the case with all projects, some issues were experienced during the execution. The following is of reference for this paper (some of the issues experienced during the procurement process was already discussed earlier):

- As already mentioned above, not enough technical information was made available during the bidding process. In fact, only the Terms of Reference, which include the detail scope of services, was provided to the bidders. Also, it became evident during the procurement as well as execution of the project that even the detailed scope of services did not contain enough technical information. That left the bidders with uncertainty leading to assumptions being made. As was mentioned under the procurement methodology, this could have been negated by using a two stage procurement process and the use of non-disclosure agreements during the second bidding phase by the chosen consulting companies,
- Similar to the 2018 Panel of Experts, the project team also experienced issues to source additional information from the LHDA digital document library in addition to the information provided by the technical assistance specialist who was part of 2018 Panel of Experts. This is not a unique issue. Similar issues may be experienced at other dam owners. Dam owners should give due attention to the development of their digital libraries especially the naming and description of drawings and documents,
- Difficulty was also experienced by the project team to source all the monitoring results as the current system consists of numerous excel spreadsheets. LHDA is planning to eliminate these difficulties through future implementation of a web based monitoring information system.

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4. CONCLUDING REMARKS

The project team of ARQ Dams, Trial Surveys, Natural Hazard Assessment Consultancy, University of Cape Twon, OFITECO and Civil Engineering Laboratory with the help of the relevant LHDA staff members successfully implemented the project. From the results of this successful implementation, the following important conclusions were drawn:

- Procurement management:
  - The use of a technical assistance specialist proficient in the technical subject matter ensured that not only was the correct technical methodology followed during implementation but also assisted during the procurement phase to provide an appropriate scope and ensure that the most capable technical team was chosen during the technical bid evaluation. It is recommended to other dam owners to consider the use of such a specialist for projects of this nature,
  - The technical procurement methodology proved to be very efficient. It is highly recommended that dam owners consider using the methodology requiring the bids to follow specific formats as well as the use of a technical scoring guideline for the technical bid evaluation team to ensure proper technical evaluation,
  - Dam owners should consider using a two stage procurement process and the use of non-disclosure agreements during the second bidding phase by the chosen consulting companies to ensure that the bidders have sufficient technical information to prepare proper technical proposals that consider all relevant known technical issues,
- The use of the "new" calibration approach during the finite element analysis by including the dynamic calibration of the model as a first step using the results of the ambient vibration measurement, again proved to be successful. It is highly recommended that any future finite element analysis of an existing concrete dam wall should use this calibration approach,
- It is also recommended that projects of this nature also include a laser and underwater sonar scan to confirm the as-built dimensions of the dam, a site specific seismic hazard analysis as well as an ambient vibration measurement to determine the dynamic properties of the dam wall and the inherent stiffness of the dam and its foundation,
- As availability of information plays an integral part of risk management, dam owners should give due attention to the development of their digital libraries especially the naming and description of drawings and documents to enable easy access, and
- Finally, for a dam of this size with more than 1 000 measuring points, the use of spreadsheets might not be answer – rather a dedicated web based monitoring information system.

5. ACKNOWLEDGEMENTS

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CALIBRATION OF THE FINITE ELEMENT MODEL OF KATSE DAM

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ABSTRACT

The last Panel of Expert review of Katse Dam in 2018 (Dungar et al: 2019) concluded that the original design calculations although appropriate at the time, do not satisfy all the current international accepted criteria and procedures. This specifically pertains to the abutment seismic stability as well as the overall long-term behaviour of the dam wall during normal operation and extreme conditions. In addition, advancements in numerical modelling techniques since the 1990s—when the dam was designed—warranted a state-of-the-art structural evaluation. As a result, the LHDA initiated a project to perform a finite element analysis of the dam as well as evaluate the monitoring system and monitoring results and determine threshold values for the monitoring system.

A key component of the structural evaluation involved developing a finite element model of the dam and foundation for undertaking state-of-the-art simulations of the dam behaviour under a suite of loading conditions.

The final calibration step of the finite element (FE) model of the dam through a structural back-analysis, comparing simulated deformations with those measured on site using the results of the earlier dynamic and thermal calibration steps. Model parameters were validated by aligning the numerical predictions with observed dam displacements. This paper presents the static calibration methodology, highlighting the challenges encountered and key lessons learned.

1. BACKGROUND

Katse Dam, located on the Malibamatšo River in Lesotho, forms part of Phase 1A of the Lesotho Highlands Water Project. The dam is a 185 m high, wide-valley, double-curvature arch dam with a developed crest length of 710 m giving it a crest length to height ratio of 3.8. The dam is founded in a U-shaped valley and has a relatively large surface area to height ratio (canyon shape factor). It consists of 47 blocks, with each block 15 m wide. The maximum thickness of the dam at the base is 60 m, and the thickness at the crest is 9 m. Construction of the dam started in January 1991 and was completed in April 1997. A comprehensive account of the various design considerations and characteristics of the dam can be found in the conference proceedings of the ICOLD Congress held in Durban, South Africa in 1994 (Tardieu, 1994).

According to Tardieu (1994), the foundation of the dam consists mainly of basaltic lavas of the Drakensberg formation. The lava flows have been deposited nearly sub-horizontally and vary in thickness – from 1 m to 45 m with a median thickness of 3 m. The dam abutments consists of more than 60 flows. The main discontinuities are composed of the sub-horizontal flow contacts; a sub-horizontal shear zone two horizontal layers of auto-brecciated basalt and some vertical stress relief joints. The extent of these different components are given in Figure 2.

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Furthermore, the dam contains a number of special design features which could have an influence on the dynamic behaviour of the dam. These include the following:

- Shear keys was installed underneath the dam on both flanks to bridge the auto-breccia rock layer downstream of the central foundation;
- A preformed joint was provided on the upstream side of the lowest part of the arch to force tension cracking in the concrete instead of the weaker foundation;
- A seismic belt (heavily reinforced hidden beams) was included at the central portion of the crest of the arch around Gallery A stretching across the uncontrolled spillway from Block 15 on the right flank to Block 16 on the left flank. The purpose of this belt is to tie the blocks to avoid out-of-phase movement between the blocks in the event of an earthquake and thus prevent damage at crest level.

2. RATIONAL FOR STATIC CALIBRATION

The structural evaluation of Katse Dam was undertaken considering the current conditions and state of the dam and foundation. To ensure the model characteristics and properties correctly represent the current conditions on site, the finite element model required validation of the assumptions on which it is based.

Extensive research conducted at the University of Cape Town into the dynamic behaviour of large concrete arch dams (based on the methodology used for the theoretical analysis of bridges) since 2007 (Moyo & Oosthuizen, 2013, Moyo et al, 2016, Moyo & Nyoni, 2020, Bukenya & Moyo, 2023, Nyoni & Moyo, 2023) highlighted the importance of using a three step calibration process. This "new" approach to calibration has been successfully used for the analysis of a number of dams including Oanob Dam in Namibia (Hattingh, Moyo, Shaanika, Mutede, & Muir, 2019), Roode Elsberg Dam (Moyo et, 2013, Moyo et al 2016, Moyo et al, 2024) and Kouga Dam (Moyo & Nyoni, 2020) in South Africa.

The calibration steps therefore adopted for this project at Katse Dam involved the following:

- Calibration of the dynamic properties using the results of the AVM testing (Moyo et al, 2025) and available material properties obtained during design and construction (Cassells et al, 2025);
- Thermal calibration of the concrete structure (Cassells et al, 2024). Observed thermal behaviour was available from the static monitoring results; and
- Lastly, deformation calibration/validation using the results of the static monitoring (this paper).

3. STATIC CALIBRATION METHODOLOGY

Deformation provides the most direct indication of the complex behaviour of concrete arch dams; therefore, interpreting observed deformation data and comparing it with numerical modelling results is a critical component of dam safety monitoring (Wang et al., 2019). The ability to predict "regular" dam deformations under prevailing conditions, and to compare these predictions with observed responses, is fundamental to effective dam surveillance.

A concrete dam in normal operation is subjected to seasonal variations in reservoir level and temperature, driven by inflows, abstractions, and climatic conditions. These variations induce elastic deformations of the structure that follow cyclical patterns, typically manifesting as increased downstream displacement during periods of higher reservoir levels and/or lower ambient temperatures.

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More than any other dam type, arch dams are particularly sensitive to temperature effects, with downstream crest displacements under maximum temperature drop conditions often comparable in magnitude to those induced by maximum hydrostatic loading (Daoudu et al., 1997). Katse Dam is located in a continental climate characterised by extreme seasonal temperature variations. Hot summers and cold winters result in large differences between seasonal maximum and minimum temperatures.

For Katse Dam, the typical seasonal operating behaviour indicated that approximately 50% of the total crest displacement between seasonal extremes can be attributed to temperature effects, while the remaining 50% is associated with hydrostatic loading. This relationship was established for reservoir levels varying between the Full Supply Level (FSL) of 2053 mASL in summer and 8 m below full supply level in winter.

To calibrate the finite element model against the static deformations of the prototype dam, the thermomechanical and stiffness properties used in the model were verified to accurately represent the current characteristics of the dam.

The first parameter calibrated in this static calibration was the thermal expansion coefficient of the dam concrete. This commenced by evaluating the accuracy of the thermal expansion value assumed in the original design and adjusting it as necessary to ensure that the FE model replicated the measured deformation response under thermal loading. Through iterative validation, the thermal expansion coefficient was modified until the simulated thermomechanical behaviour closely matched the observed prototype response.

The second parameter calibrated was the static modulus of elasticity of the dam and foundation material. It is important to note that the first step in the global calibration process has already determined moduli of elasticity for both the concrete as well as foundation using the results of an ambient vibration measurement (Cassells et al, 2025). Due to the low strain rate and low level of strain development experienced during ambient vibration testing, the static elastic modulus of the finite element model was expected to be identical or very similar to that derived by dynamic calibration. Under low strain conditions dynamic and static elastic modulus are similar (Shkolnik, 2007).

The static elastic modulus calibration process initially adopted the dynamic elastic modulus from the results of dynamic calibration and confirmed the accuracy of this, by testing the associated static deformation response of the dam's finite element model against the measured response of the dam.

Calibration of the dam's static finite element model was considered to be achieved when the deformation response of the model closely matched that of the prototype under identical external loading conditions. For this study, a correlation tolerance of 10% between predicted and measured deformations was deemed acceptable.

The following important assumptions were made at the onset of this static calibration process:

- For thermal calibration under operating conditions, the dam was assumed to have reached a seasonal equilibrium state.
- Post-construction, the dam was assumed to be in the operational temperature state whereby the time-based temperature state is driven by heat transfers at the surface (convection) and through the material (conductance) of the dam and foundation. Changes in temperature state are driven purely by seasonal fluctuations in ambient and water temperatures.
- The load scenarios were chosen with a short enough time period so that there should be no significant permanent deformation be observable.

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4. OBSERVED LONG TERM BEHAVIOUR OF KATSE DAM USED IN CALIBRATION

4.1 Reservoir water levels

The observed reservoir levels of Katse Dam are shown in Figure 1. The reservoir data shows that apart from a dry spell from 2004 to 2006 and another lengthy dry spell from 2015 to 2022, on average the reservoir level usually reaches full supply level (2 053 mASL) in late summer (wet season) and drops around 9 m below full supply level (2 044 mASL) late in winter (dry season). The lowest reservoir level ever recorded was 51.5 m below full supply level and the highest spilling level was 3.5 m above full supply level.

4.2 Temperature Conditions

Daily maximum and minimum air temperature measurements are taken at Katse Dam as shown in Figure 2. Historical temperature data for the dam is available from January 1995 to current day, with the exception of the period from July 2004 to March 2013, when the thermometer station was inoperable. The red curve presents the daily maximum temperatures and the blue curve daily minimum temperatures. The typical average daily air temperature cycle is depicted by the green curve. The historical data plot shows that maximum summer air tempearatures could reach 30° C and while the winter minimum air temperature could go below -5° C.

4.3 Displacement

The deformation of the dam is measured by both pendulums as well as geodetic survey. The layout of these monitoring systems is given in Figure 3.

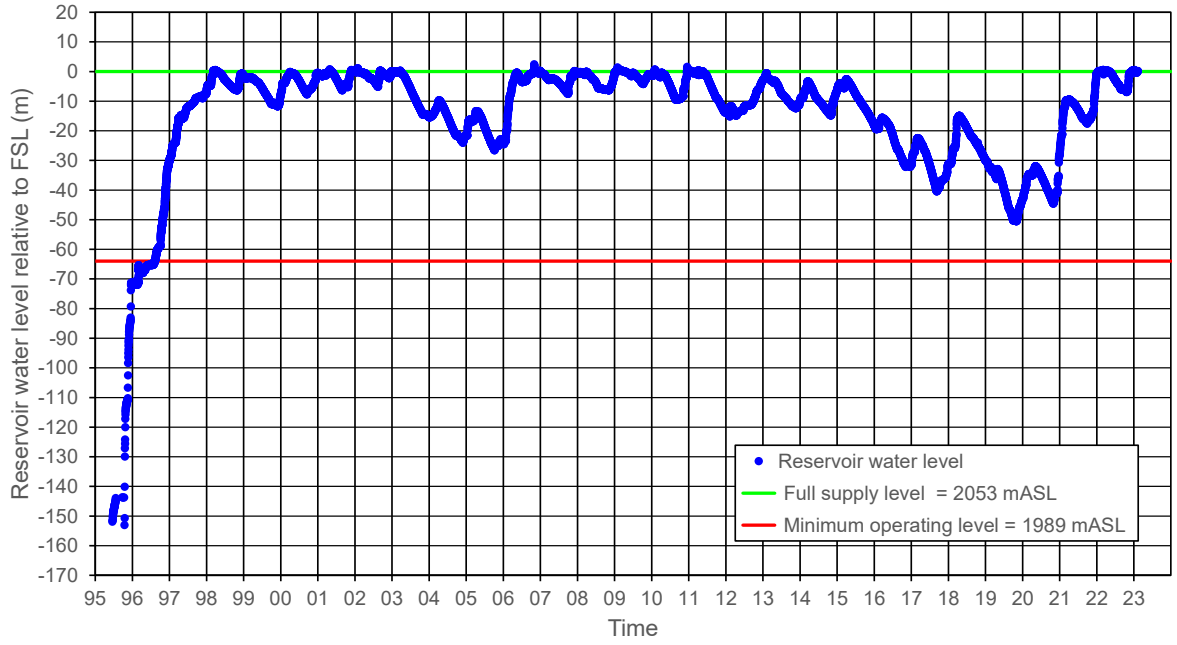

Figure 1. Katse Dam reservoir water levels

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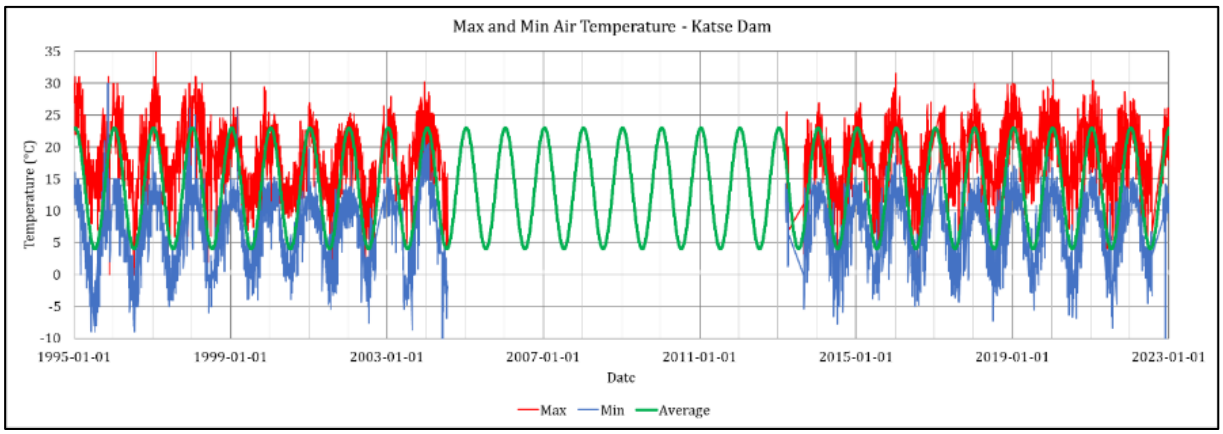

Figure 2. Historical air temperature data for Katse Dam

4.3.1 Geodetic survey

A comprehensive geodetic survey network was installed at Katse Dam consisting of the following (Pretorius et al, 2001) (see):

- A triangulation network consisting of 5 primary beacons, 8 secondary beacons, 8 arch dam balcony pillars, 4 gallery exit brackets, 19 parapet targets on the crest, 6 optical alignment monitoring points, 24 crest benchmarks and 2 altrimetric wire anchors.
- Precise traverses on the crest and inside Galleries D, H and K installed in the centre of every other block, and in the drainage and access galleries on both flanks.
- The inverted and hanging pendulum system form an essential part of the geodetic surveying system as they are connecting the different levels of the traverse system with each other.

The geodetic surveys are taken on a six-monthly basis at times of the year when the dam is assumed to be at its maximum and minimum operating temperature state (under peak summer and winter temperature states). The geodetic survey results for the crest of the dam for load scenarios 1A and 1B are shown in Figure 4.

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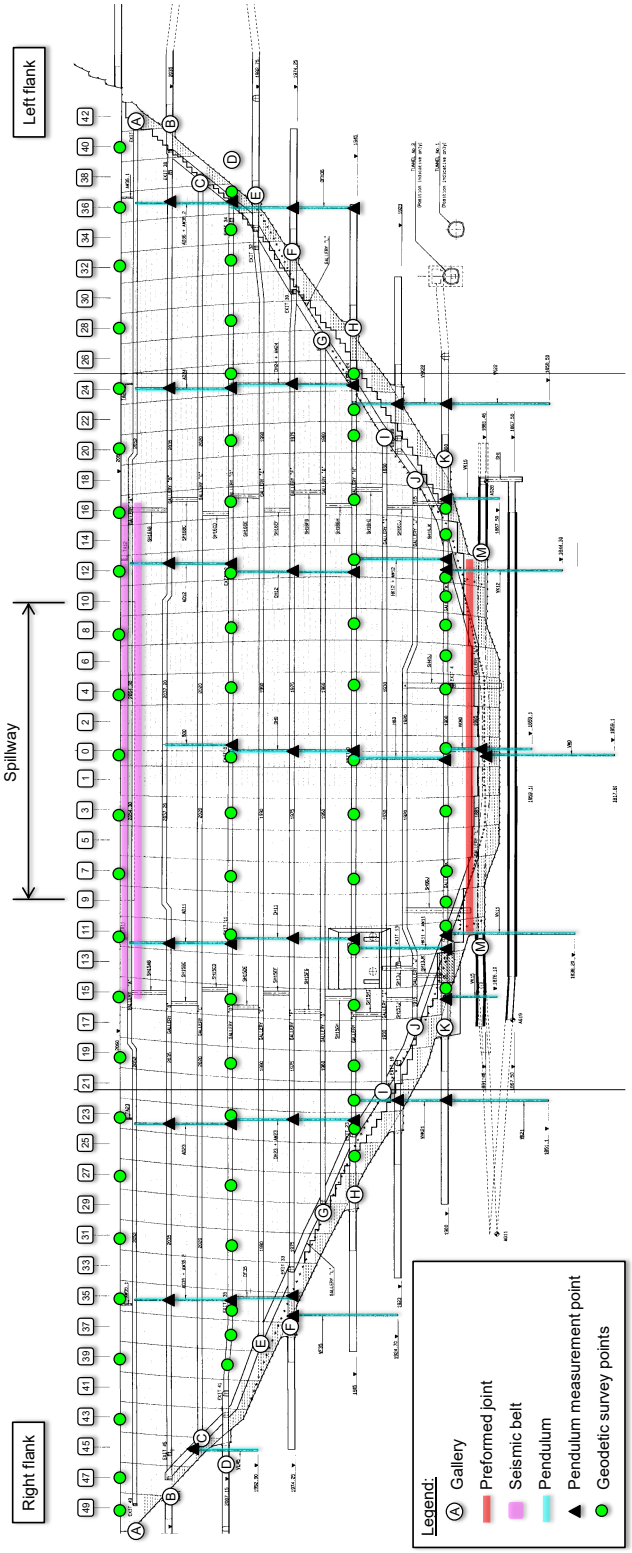

Figure 3. Downstream elevation with geodetic survey and pendulum layout

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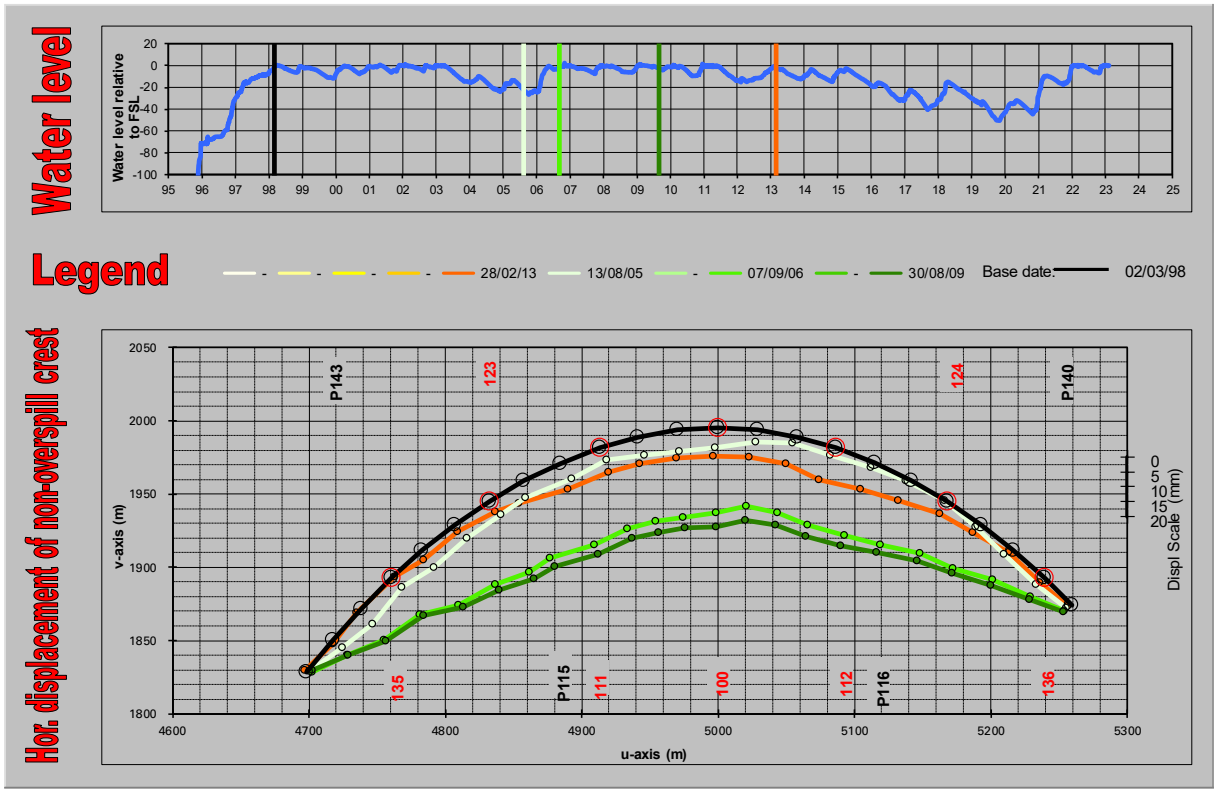

Figure 4. Geodetic survey: Non-overspill crest: Horizontal displacement: Load conditions 1A and 1B

4.3.2 Pendulum Measurements

Katse Dam has an extensive pendulum system comprising both hanging and inverted pendulums. A total of 44 pendulum instruments were installed in the dam's galleries at various elevation levels at approximately one-eighth points along the dam axis in blocks 35, 23, 11, 0, 12, 24 and 36 as shown in Figure 3.

5. FINITE ELEMENT MODEL

The finite element model of the dam and foundation was created using the Midas FEA NX software. The dam and foundation were modelled as a continuum mesh body discretised with a hybrid of hexahedral, tetrahedral, pyramid and wedge 3D solid elements. The model is hexahedral (8 Gaussian integration points) dominant with other elements only used in the vicinity of sharp edges and faces. The dam and foundation finite element model comprises 240 000 nodes and 460 000 elements. This complex model used a sound meshing strategy with very high resolution of elements within the dam body and the foundation zone in close proximity of dam body and low-resolution elements on the outer extents of the foundation.

The outer extents of the foundation model were constrained against translation accordingly, with the bottom face constrained against translation in all three directions of the cartesian plane, X, Y and Z. The side faces of the foundation mesh were constrained against horizontal lateral translation.

The material parameters for the dam concrete, as assumed at design stage, were obtained from laboratory testing of specimens cast and cured in the on-site laboratory under controlled conditions. The

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modulus of elasticity and Poisson ratio of the dam concrete were obtained from laboratory deformation testing of concrete specimens.

Foundation material parameters of the Katse Dam site, such as elastic modulus and Poisson ratio, were derived from a combination of laboratory and in-situ geotechnical tests done before and during construction of the dam. The results of which were used to determine foundation material parameters used for the finite element model (see Table 1). The corresponding geological model incorporated in the finite element model is shown in Figure 2.

Table 1 Katse Dam baseline foundation material parameters

Foundation layer description	Location	Density (kN/m^3)	Elasticity Modulus (GPa)	Poisson ratio
Lower Basalt layer below auto brecciated layer (A)	Below 1 866 mASL	0	40	0.21
Lower auto-brecciated layer (B)	Between 1 866 & 1 869 mASL	0	20	0.30
Basalt layer between auto-brecciated layers (C)	Between 1 869 & 1882 mASL	0	25	0.27
Upper auto-brecciated layer (D)	Between 1 882 & 1 885 mASL	0	10	0.35
Basaltic layer (D)	Above 1 885 mASL and below flow contact at 1 892.5 mASL	0	30	0.25
Upper basaltic layer (E)	Above flow contact at 1 892.5 mASL	0	30	0.25
Upper right bank basaltic layer (F)	Above 1 975 mASL	0	38	0.23

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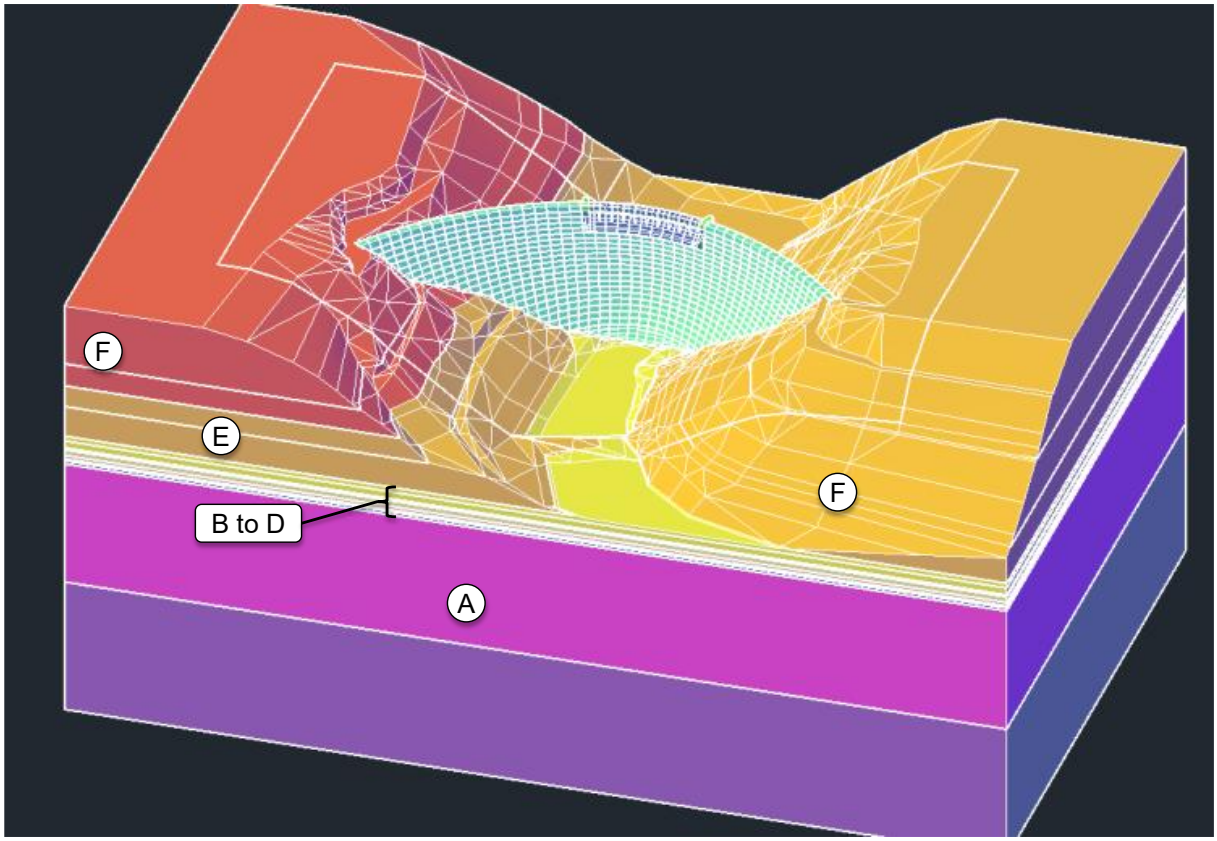

Figure 5. Katse Dam 3D finite element model with foundation detail

5.1 Loading Scenarios

Under normal operating conditions, the structural behaviour of an arch dam should remain within the linear elastic stress-strain range, and consequently the principle of superposition of loads can be invoked for structural analysis (Gunn, 2001).

To calibrate the FE model, loading scenarios were devised with the intention of modelling the dam deformation behaviour in terms of its relative elastic response between two historical points in time, for which the dam behaviour has been documented and can be used as baseline reference data.

Separate loading scenarios were formulated to isolate the thermomechanical response of the dam due to temperature changes only, and the structural stiffness related mechanical response of the dam due to changes in hydrostatic loading only. This was achieved by selecting historical loading scenarios whereby the change in deformation was driven purely by either temperature change conditions or changes in hydrostatic loading conditions.

Four loading scenarios were postulated for calibration of the finite element model as shown in Table 2. Due to differences in frequency of the geodetic surveys and the pendulum measurements, separate thermomechanical and hydrostatic loading scenarios were each analysed for baseline deformation data obtained from both survey and pendulum measurements. Calibration of the finite element model under multiple scenarios in reference to baseline data from multiple measurement instruments enhances the reliability of this static calibration. It is important to note that the recommended concrete characteristics from load scenarios 1A and 1B were tested against the pendulum deformations in load scenarios 2A and 2B.

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Table 2: Load scenarios static calibration

Load scenario	Calibration parameter	Baseline data source	Start point		End point	
			Date	Relative water level	Date	Relative water level
1A	Thermal expansion coefficient	Geodetic	2013-02-28	-2.4 m	2009-08-30	-2.4 m
1B	Structural Stiffness	Geodetic	2005-08-13	-22.4 m	2006-09-07	-1.1 m
2A	Thermal expansion coefficient	Pendulum	2022-02-01	-0.1 m	2022-08-04	-4.4 m
2B	Structural Stiffness	Pendulum	2020-08-12	-38.1 m	2022-08-04	-4.4 m

Water level relative to full supply level

6. CALIBRATION RESULTS

6.1 Loading scenarios 1A and 1B

The finite element model results of the deformation at the crest of the dam for the iterative analysis steps under loading scenario 1A are presented in Figure 6. The deformation curves represent outputs from a parametric study assessing the sensitivity of dam displacement to different thermal expansion coefficients, ranging from 6 x 10^{-6} °C^{-1} to 11 x 10^{-6} °C^{-1}. These results are compared with measured deformations obtained from geodetic surveys. The analysis indicates that a thermal expansion coefficient of 8.75 x 10^{-6} °C^{-1}, when applied in the finite element model, provides the best agreement between simulated and observed dam deformations. Similar curves were also derived for the deformation of Galleries D and H. The results for these elevation levels confirmed the same agreement between simulated and observed dam deformations with a thermal expansion coefficient of 8.75 x 10^{-6} °C^{-1}.

The finite element model results of the deformation at the crest of the dam for the iterative analysis steps under loading scenario 1B are presented in Figure 7. The deformation curves represent outputs from a parametric study assessing the sensitivity of dam displacement to different elastic moduli, ranging from 23 GPa to 49 GPa. These results are also compared with measured deformations obtained from geodetic surveys. The analysis indicates that a dam elastic modulus of 34 GPa, when applied in the finite element model, provides the best agreement between simulated and observed dam deformations. Similar curves were derived for the deformation of Galleries D and H. The results for these elevation levels confirmed the same agreement between simulated and observed dam deformations for modulus of elasticity of 34 GPa. It is important to note that this result is the same as what was concluded from the dynamic calibration (Cassells et al, 2025).

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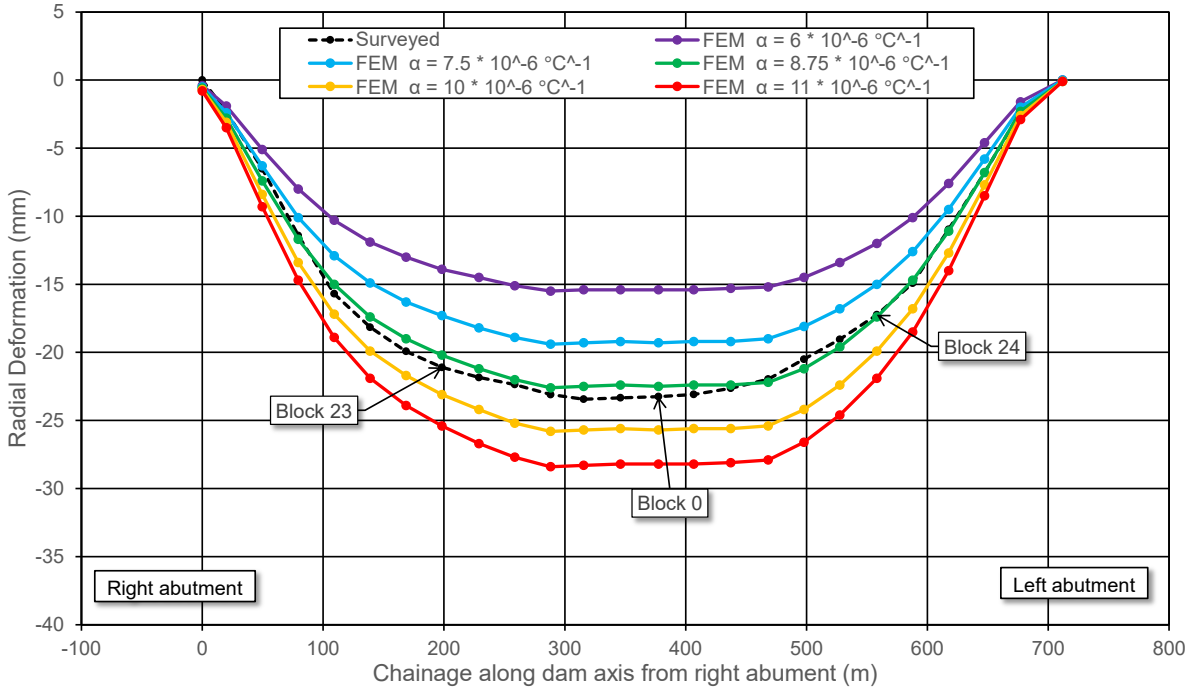

Figure 6. Scenario 1A: Non-overspill crest: Different thermal expansion coefficients

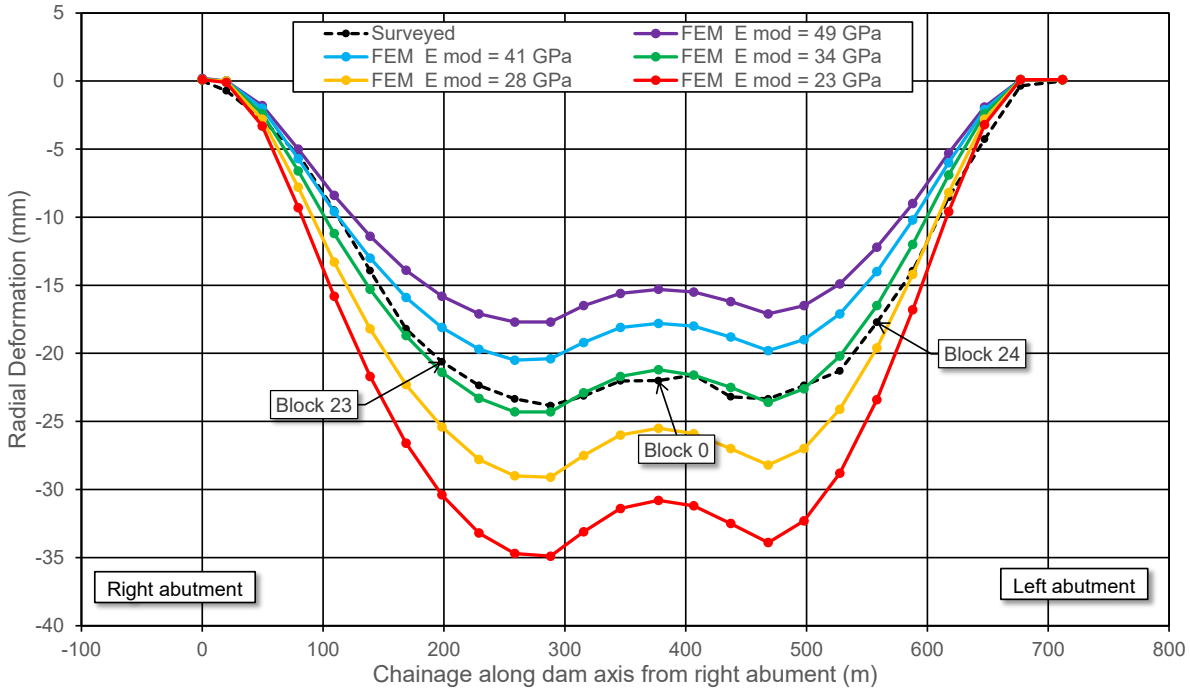

Figure 7. Scenario 1B: Non-overspill crest: Different modulus of elasticity

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6.2 Loading scenarios 2A and 2B

A comparison of the computed radial deformation results of the dam for loading scenario 2A against the cumulative radial pendulum measurements is shown in Figure 8. The results show that the assumed thermal expansion coefficient of 8.75 x 10^{-6} $°C^{-1}$ provides accurate predictions of the measured cumulative pendulum displacements under thermomechanical loading scenario 2A. Most modelled deformations are within 1 to 1.5 mm of the measured displacement with a single comparison showing a difference of 2 mm (at the crest of block 35).

A comparison of the computed radial deformation results of the dam for loading scenario 2B, against the cumulative radial pendulum measurements is shown in Figure 9. The results show that the assumed dam elastic modulus of 34 GPa provides accurate predictions of the measured cumulative pendulum displacements under hydrostatic loading differences. Most modelled deformations are within 2 mm of the measured value except for block 24, indicating that the finite element model marginally underpredicts the displacement of the dam in the region of block 24 assuming a modulus of elasticity of 34 GPa or that there is a pendulum measurement error in block 24.

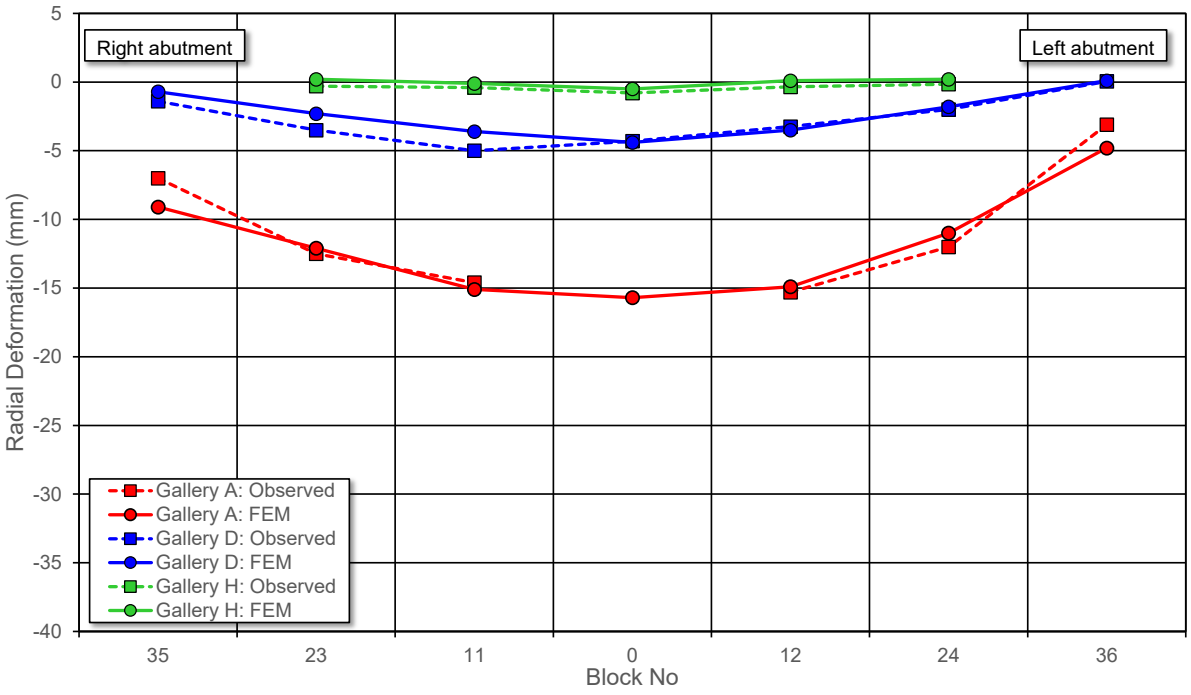

Figure 8. Scenario 2A FEM deformation of dam against pendulum measurements

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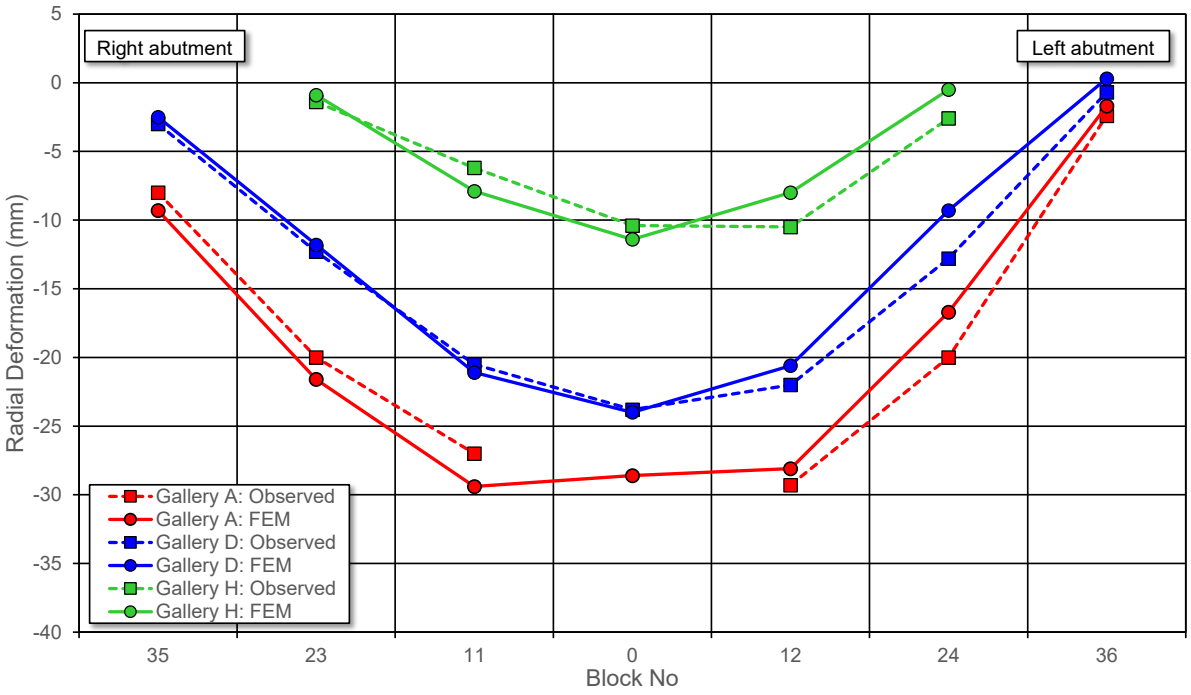

Figure 9. Scenario 2B FEM deformation of dam against pendulum measurements

7. CONCLUDING REMARKS

From the dynamic calibration of the finite element model of Katse Dam, the following conclusions were possible:

- From the results of the static calibration, for the concrete that used basalt as aggregate, a thermal expansion coefficient of 8.75 x 10^{-6} C^{-1} and a modulus of elasticity of 34 GPa was determined.
- The statically calibrated modulus of elasticity of 34 GPa is the same as the modulus of elasticity determined in the dynamic calibration step (Cassells et al, 2025).
- It was possible to model the behaviour of the dam within a tolerance of 10% compared to the observed deformation behaviour of the dam using both the results of geodetic survey as well as the pendulums.
- This project has highlighted the importance of performing all three calibration steps – dynamic, thermal as well as static, to obtain a proper validated finite element model.

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MOUNTAINS TO MEGAWATTS - SCREENING OF POTENTIAL PUMPED STORAGE SCHEME SITES

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ABSTRACT

Pumped storage schemes have been identified as a key role player in current and future energy generation worldwide because of the global shift towards renewable energy generation, increasing energy costs and net zero carbon emission commitments to combat climate change. South Africa's mountainous topography and vast land mass provide the elevation and area required for dam siting as well as the current energy landscape; great potential exists for investment into pumped storage options as enablers for variable renewal energy storage in South Africa.

Using GIS algorithms, the South African Hydropower Atlas (SAHA) has identified 49,986 pairs of potential pumped storage hydropower sites with generation potential of approximately 31,000 MW from overlapping sites. This research developed an evaluation methodology and Pumped Storage Site Evaluation Tool to allows the user to input 7 topographic parameters to conduct the energy and financial screening assessments of sites, effectively turning "mountains to megawatts".

1. BACKGROUND AND INTRODUCTION

Energy supply and consumption are the most important factors influencing economic growth due to the requirement of energy for every form of economic activity to function (Kumba and Olanrewaju, 2022).

Countries are moving towards a more sustainable energy future by evolving the energy market to include more variable renewable generation. This aims to address the increased energy requirements due to increased population growth and industrialization (Gielen et al., 2019). Furthermore, renewable energy generation aims to mitigate issues of carbon dioxide emissions and climate change by providing clean energy to the society as highlighted in the Sustainable Development Goal 7, the Paris Agreement and South African National Development Plan (Akinbami et al., 2021).

Worldwide, the energy market is evolving to include more variable renewable energy generation sources including intermittent options such as wind and solar photovoltaic cells as generation fluctuates on daily and seasonal timescales depending on the effects of cloud cover and weather patterns. This intermittency has the potential to result in grid instability.

On 28 April 2025, large scale grid black outs were experienced in Spain, Portugal and Southern France with an estimated loss of 2.2 GW of generation. This impacted over 50 million people and an estimated €1.3–1.6 billion in economic losses due to disrupted transport, communications, healthcare and cold-chain dependent industries. This outage was a result of the electricity grid running high on renewables (55% solar and 12% wind) but had low dispatchable spinning reserve (Seeber, 2025).

The South African energy landscape currently faced rolling blackouts because of a shortage of generation capacity and ongoing breakdowns across Eskom's ageing fleet of coal-fired power stations

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(Department of Minerals and Energy, 2021). Further abroad, the Russian invasion of Ukraine resulted in European energy costs surging by 115% for gas and 237% for electricity over the period of 23 February 2022 to 31 July 2022. This resulted in increasing food prices, supply chain disruptions and reduced economic growth (Ferriani and Gazzani, 2022). Furthermore, the energy landscape in South Africa is complex owing to diverse sources of energy generation, renewable energy expansion, increasing energy prices, unique tariff structures and energy shortages.

Pumped storage hydropower is a grid-scale energy storage solution that supports the integration of variable renewable energy sources. It provides spinning reserve services portion of the plant's capacity that is online, synchronized to the grid and ready to provide power immediately if there is a sudden loss of generation or an increase in demand.

1.1 PUMPED STORAGE SCHEMES

Pumped storage schemes are a proven solution for high capacity, long duration energy storage capabilities. Worldwide, dependencies on electricity increase with increasing initiative to decrease reliance on carbon emission sources of energy generation. Pumped storage schemes have become increasingly important in ensuring grid stability and reliability in recent times.

Pumped storage hydropower produces a cleaner energy by utilizing difference in height between two reservoirs and time-shifting excess available energy in the form of gravitational potential energy for later use (HydroWires, 2021). Pumped storage schemes are operated by utilizing electricity supply to lift water to an upper reservoir during off-peak times and then generate electricity by releasing water from the upper reservoir to the lower reservoir through a turbine in times of greater demand. Figure 1 illustrates the typical layout of a pumped storage scheme.

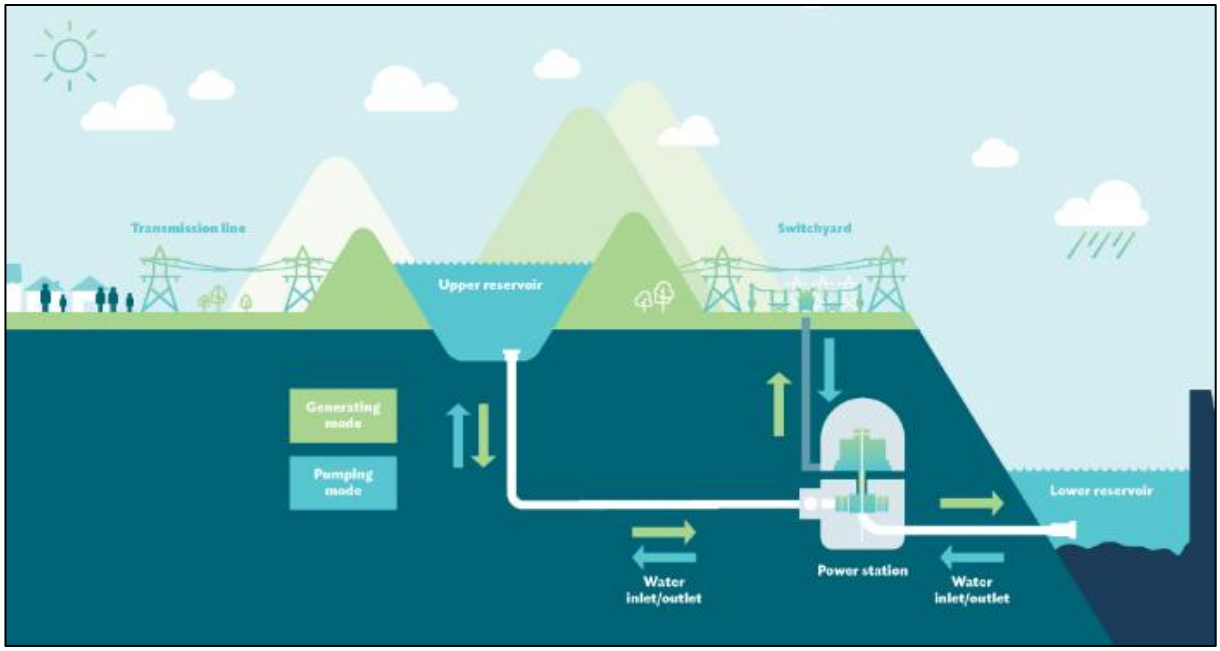

Figure 1. Typical layout of a pumped storage scheme (Hydro Tasmania, 2023)

South Africa's mountainous topography, geological conditions and vast expanse of area which provide head and favourable requirements for dam siting, as well as the complex hydraulic conditions, provide opportunities for efficient energy generation solutions through pumped storage schemes.

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1.2 SOUTH AFRICAN HYDROPOWER ATLAS

The South African Hydropower Atlas (SAHA) was developed by the University of Pretoria with the aim to identify suitable, potential sites for various types of hydropower generation. SAHA aimed to make information regarding suitable, potential sites available to present opportunity for renewable energy opportunities and to encourage the exploitation of hydropower as a renewable energy source which could increase energy generation to the South African energy grid (van Dijk et al., 2022). SAHA identified various types of hydropower generation ranging from pico, micro, macro across South Africa at various locations including pumped storage schemes, wastewater treatment works, water treatment works, storage schemes, conduit hydropower, run-of-river, hydrokinetic and weirs (Bekker et al., 2021).

Using Geographic Information Systems (GIS) with defined search criteria, the Australian National University identified 616,000 potentially feasible pumped storage scheme sites around the world. It is estimated that the potential energy storage of 23 million GWh (Blakers et al., 2017). 49,986 upper and lower reservoir pairs of sites for defined energy generation capacity potential ranging from 2GWh to 150GWh with an associated storage time of 6 hours and 18 hours as listed in Table 1.

Table 1: Total number of pumped storage scheme sites

Reservoir storage time	Number of sites for associated energy generation capacity					
	2 GWh	5 GWh	15 GWh	50 GWh	150 GWh	Number of sites
6 h	5,440	9,687	7,402	4,580	0	27,109
18 h	0	7,992	8,151	4,985	1,749	22,877
Total						49,986

This research was integrated into the South African context by addition to the South African Hydropower Atlas (SAHA). Figure 2 shows the numerous identified potential pumped storage sites in the SAHA. Different coloured dots represent sites of reservoir storage time and associated energy generation capacity.

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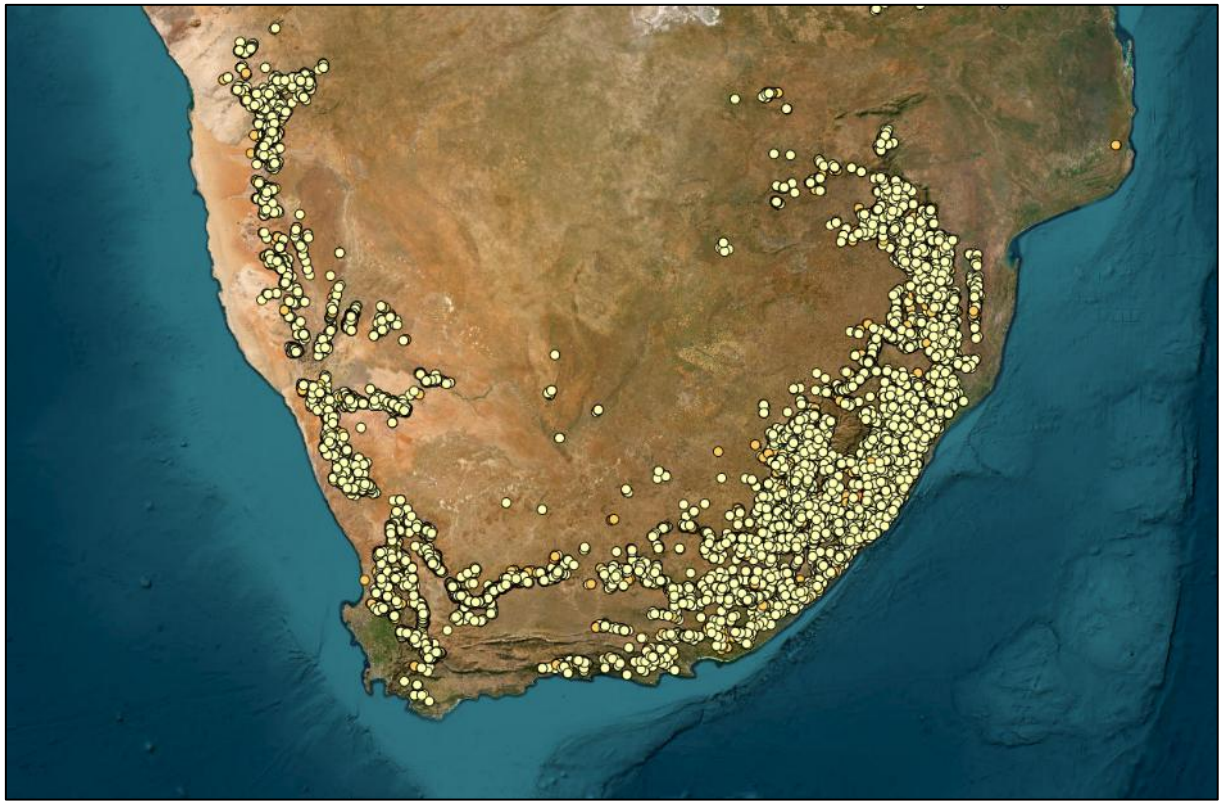
Figure 2. Numerous identified potential pumped storage sites in the South African Hydropower Atlas (Bekker et al., 2021)



2. NEED FOR A PUMPED STORAGE SITE SCREENING PROCEDURE AND TOOL

While all sites included in SAHA are feasible, certain sites demonstrate higher financial viability than others and thus require screening to identify sites of greater development potential.

To date, global tools and criteria used for pumped storage scheme site evaluation have been developed for broad contexts that separate geospatial screening from financial evaluation (Van Veenhuyzen, 2023). This highlighted a need for a pumped storage scheme site screening tool tailored to the constraints relative to the context of South Africa's energy landscape.

It would be time-consuming to evaluate all sites on an individual basis. The developed evaluation methodology and Pumped Storage Site Evaluation Tool to allows the user to input 7 topographic parameters to conduct the energy and financial screening assessments of pumped storage scheme sites, effectively turning mountains to megawatts.

3. PUMPED STORAGE SCHEME SCREENING PROCEDURE

The developed evaluation process comprises four key stages: hydraulic assessment, energy assessment, costs and revenue calculation and financial assessment. The outputs from the hydraulic and energy assessments serve as inputs to the financial assessment.

Figure 3 shows the pumped storage scheme site evaluation procedure that was developed to enable a relative financial comparison of sites to identify those that may be more financially viable for development to further investigate for development purposes.

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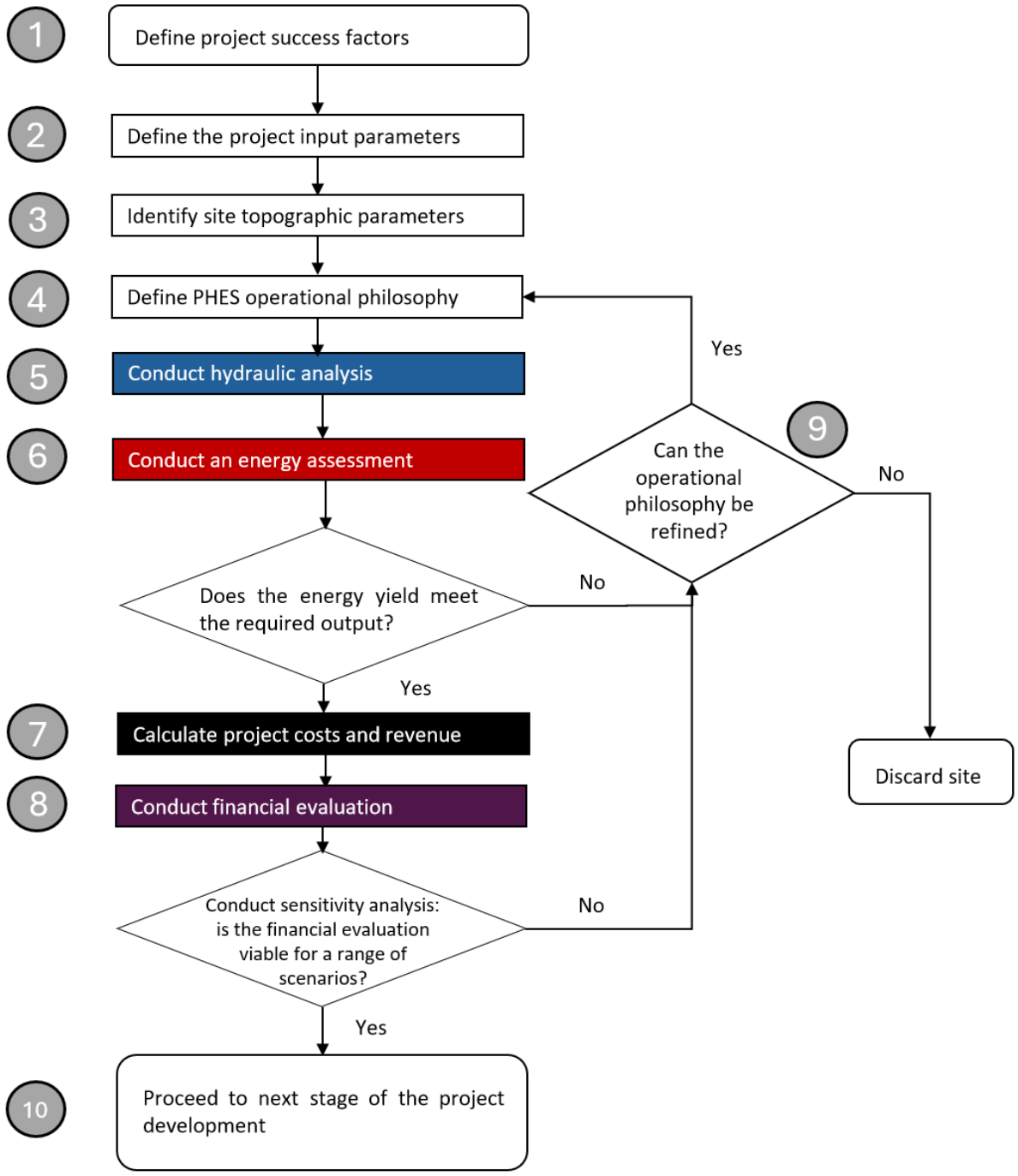

Figure 3. Developed pumped storage scheme site screening procedure flow diagram

3.1 TOPOGRAPHIC AND HYDRAULIC ANALYSIS OF SITES

Topographic and hydraulic input parameters are required to conduct the hydraulic analysis of a pumped storage scheme site. Topographic input parameters can be acquired from the SAHA dataset or a user-defined site. The following topographic input parameters are required:

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- A unique, user-defined site identifier name.
- Head difference between the upper and lower dam sites (m).
- Horizontal distance separation between the identified sites (km).
- Upper and lower reservoir volumes (GL).
- Upper and lower reservoir dam wall heights (m).
- Upper and lower reservoir dam wall material volume required (GL).
- Upper and lower reservoir dam wall length (m).
- Latitude and longitude of the dam sites (decimal degrees).

3.2 ENERGY ASSESSMENT OF SITES

Energy assessment of the sites involved the determination of energy generated versus energy consumed during pumping. This is dependent on the operational philosophy of the pumped storage scheme. The total annual pumping and generated energy was calculated by multiplying the daily pumping and generation values by the number of arbitrage days, excluding unplanned maintenance days. The round-trip efficiency was calculated as the ratio of the energy generated to the energy consumed in pumping mode represented as a percentage.

3.3 COST AND REVENUE DETERMINATION

The profitability of a pumped storage scheme is fundamentally driven by the principle of energy arbitrage. This is the process of leveraging cheaper electricity prices to pump water during off-peak periods and capitalising on generating and selling electricity at higher prices during peak demand periods.

Revenue was calculated from tariffs for the different time-of-use periods under the ESKOM Megaflex tariff structure. These tariffs show the significant variability in electricity pricing, with peak rates reaching up to 6 times higher than off-peak rates during the high-demand season and up to 2.5 times higher during the low-demand season. This substantial price differential illustrates the case for pumped storage schemes, where energy arbitrage can be effectively leveraged to capitalise on these fluctuations.

The financial assessment involves calculation of revenue from selling generated electricity and infrastructure costs. Capital costs are highly sensitive to the unique topographic, geological, and logistical characteristics of each site. Capital costs that were included in evaluation of pumped storage schemes at concept level included the expenses associated with the construction and installation of:

- Dam walls.
- Penstock.
- Machine hall.
- Electromechanical equipment.

Operation and maintenance costs were applied across the lifetime of the pumped storage schemes.

3.4 FINANCIAL ASSESSMENT OF SITES

A financial evaluation is to be conducted on the sites for the purpose of assessment of the profitability and financial viability of the envisaged project. A pumped storage project is considered financially viable when its projected revenues are adequate to cover both capital and operational expenditures, while also yielding a satisfactory return for investors measured using the following metrics of financial assessment:

- Levelised Cost of Storage
- Net Present Value
- Cost benefit ratio
- Internal Rate of Return

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Sensitivity analysis to identify and evaluate critical variables and parameters that significantly influence the project's performance. These variables serve as key drivers of the outcome in both positive and negative scenarios due to the potentially variable nature of factors such as exchange and inflation rates.

4. DEVELOPMENT OF THE PUMPED-UP TOOL

The Microsoft Excel-based Pumped Storage Site Evaluation Tool was developed to meet the need for a practical tool that enables efficient calculation of pumped storage scheme costs and financial parameters, supporting informed site development decision-making. Figure 4 shows an extract of the Pumped Storage Site Evaluation Tool's results dashboard.

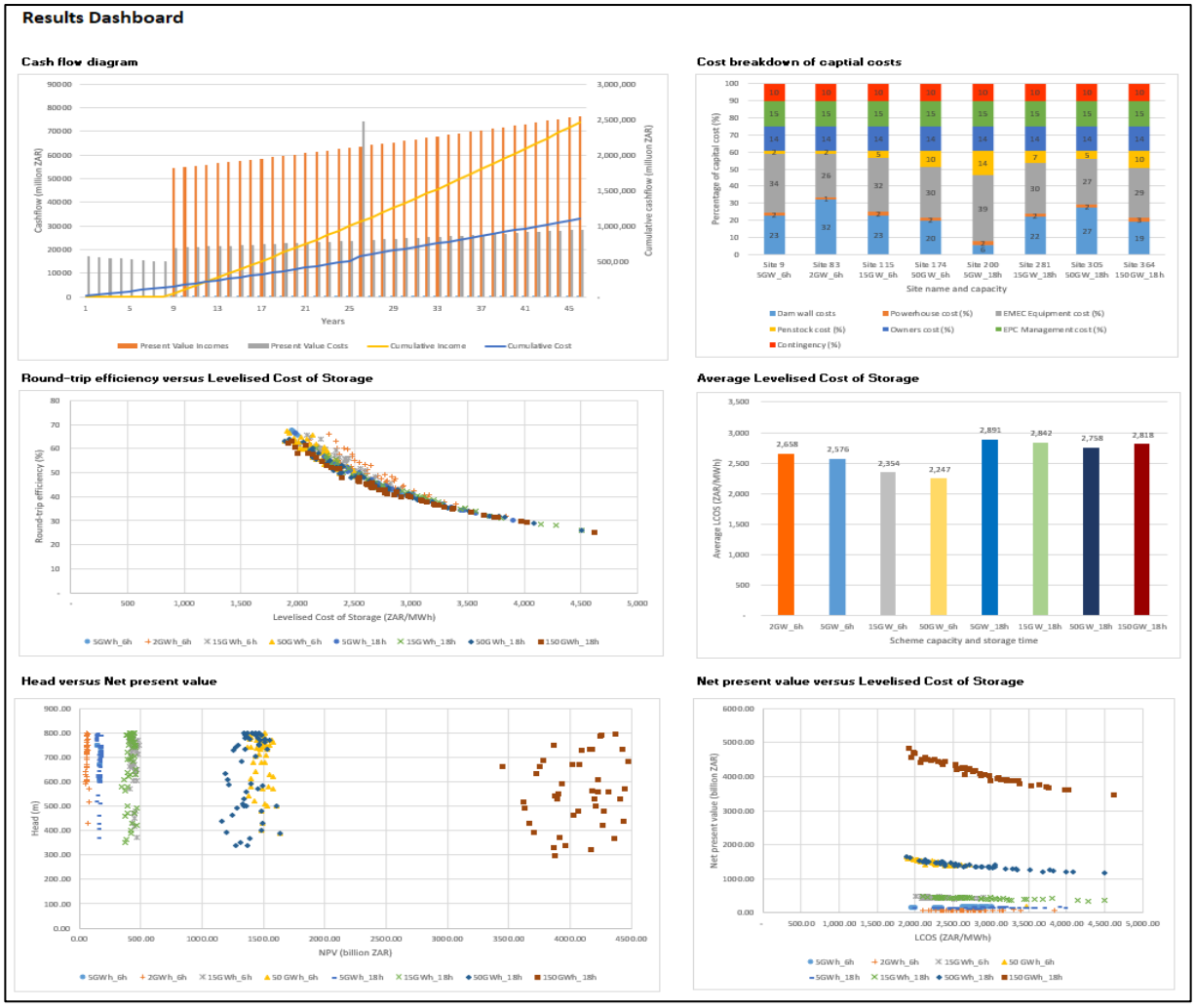

Figure 4. Pumped Storage Site Evaluation Tool results dashboard

Use of the Pumped Storage Site Evaluation Tool allows the user to evaluate hydraulic analysis, energy assessment and financial evaluation to provide a common basis for financial comparison of pumped storage scheme sites. This allows more efficient use of time by aiding in the prioritisation of sites to investigate by providing a tool for screening vast numbers of potential sites in shorter time periods which would result in increased resource utilisation.

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This ultimately identifies opportunities to expand the market for future pumped storage hydropower schemes that could provide ancillary services of grid stabilisation benefits and enabling variable renewable energy storage.

5. APPLICATION OF THE PUMPED-UP TOOL

The Pumped Storage Site Evaluation Tool was applied to sites within the South African Hydropower Atlas (SAHA) dataset. Summaries of the calculation of the cost of infrastructure development, levelised cost of energy and cashflow projections for sites are provided to demonstrate the outputs of the tool.

5.1 Cost of infrastructure development

Pumped storage schemes have high initial capital cost requirements at the start of lifetime of the asset. Figure 5 illustrates the breakdown of total project costs across multiple pumped storage sites, with each component expressed as a percentage of overall expenditure.

Despite variations in storage capacity and generation potential among individual sites, the relative contribution of individual cost components is consistent. Electromechanical equipment dominates the expenditure as is expected for high head hydropower plants, followed by the civil works requirements for the dam infrastructure.

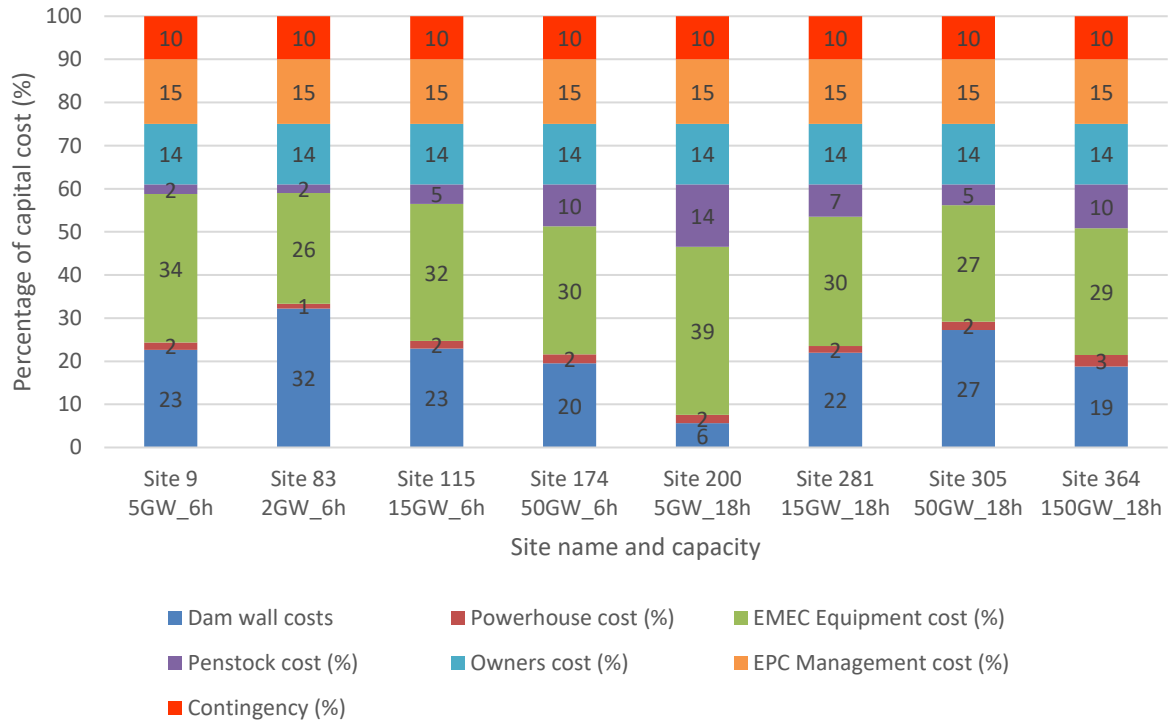

Figure 5. Percentage costs per component for selected sites

5.2 Levelised cost of storage (LCOS)

LCOS spreads the total lifetime cost across the total amount of energy generated over the project's lifetime. Larger pumped storage schemes benefit from economies of scale, achieving lower unit costs per megawatt of installed capacity. Smaller schemes have higher unit costs but require less initial capital

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investment but deliver less energy. These effects can offset one another, resulting in LCOS results that remain within a narrow range across projects of different sizes.

Figure 6 shows that even if a site has more storage and higher capacity, the overall costs are also higher, however the energy generated is also higher. When both are scaled up, the ratio of cost to energy generated works out to be in a similar range.

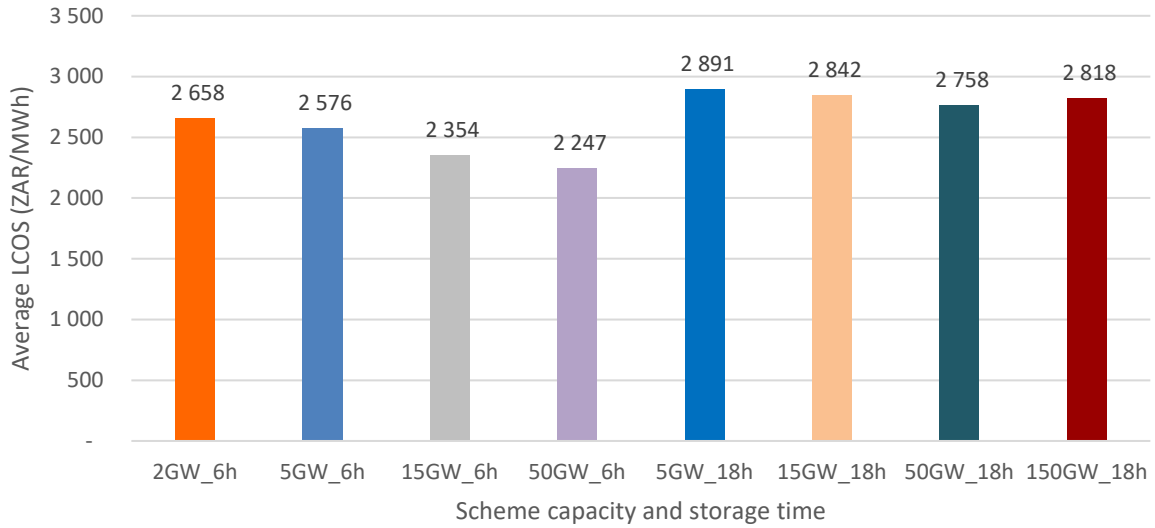

Figure 6. Average LCOS for sites analysed

5.3 Cashflow projection

Figure 7 shows the cashflow diagram generated for a screened site. The purpose of the cashflow diagram is to enable developers or investors to anticipate project cost requirements. It serves as a tool to compare investment costs against expected income returns, thereby supporting informed decision-making. All cash flow values have been converted to present values to provide a consistent and realistic financial representation across the full project lifecycle.

The cashflow diagram shows significant capital expenditure during the initial user-inputted project input parameter of a user defined construction period of 4 years. Once the pumped storage scheme is operational, the scheme begins to generate revenue, with cumulative income surpassing cumulative costs at the breakeven point in Year 9. Annual operation and maintenance costs and the major refurbishment of the electromechanical equipment scheduled for Year 25 are shown in the cash flow diagram

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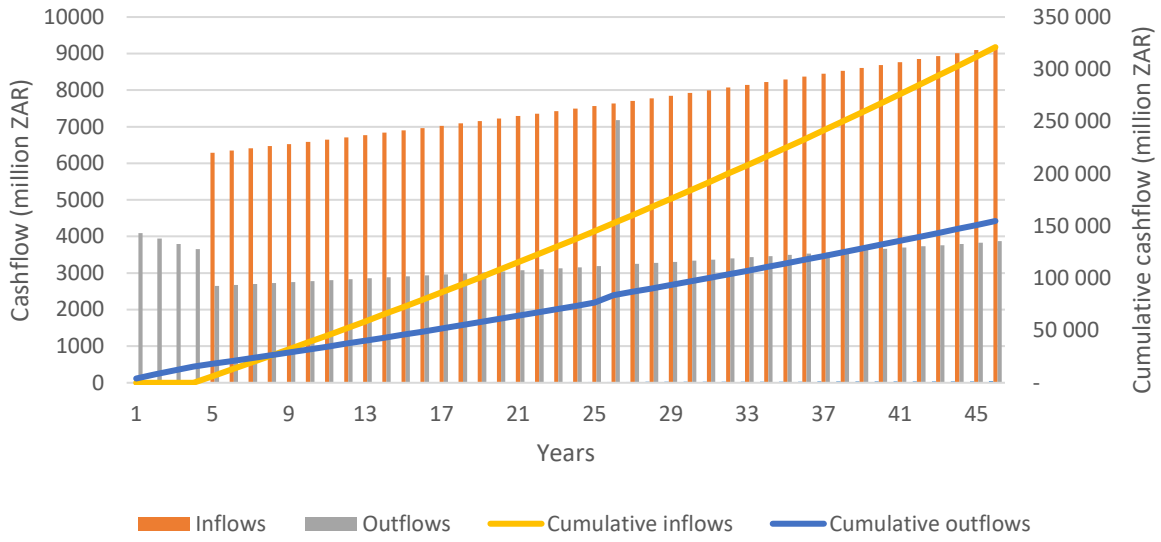

Figure 7. Cashflow projection for Site 1 generated by the Pumped Storage Site Evaluation Tool

6. CONCLUSIONS AND RECOMMENDATIONS

The developed evaluation methodology and associated tool enable early-stage financial screening of pumped storage scheme sites using 7 topographic inputs to screen sites with the highest potential for further development, essentially turning "mountains into megawatts".

The evaluation procedure performs screening of potential pumped storage scheme site. This is done by estimating the life cycle costs and benefits and, by using those values, calculating the net present value, benefit-cost ratio, levelised cost of storage and internal rate of return of the proposed project.

The application of the developed tool can provide decision-makers, developers, infrastructure owners and energy generation utilities to input site parameters to screen sites using hydraulic analysis, energy assessment and financial evaluation to provide a common basis for financial comparison of sites.

This research was conducted at screening level to select identified sites from SAHA for further investigation. Detailed investigations are required to be undertaken for each identified site to determine site specific geological conditions, hydrology, yield analysis, feasibility and selection of the dam wall type, environmental and cultural heritage studies. Without conducting due diligence on each of the above-mentioned required studies, project costs could be affected.

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RESETTLEMENT CONSULTATION: LESSONS LEARNT ON LHWP II - THE CASE OF MASAKONG VILLAGE IN MOKHOTLONG, LESOTHO

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PRESENTER: MAKANANELO KUMALO

ABSTRACT

Construction of large infrastructure has negative impacts on the lives of the affected communities, including physical displacement, which is often accompanied by expectations of improved living conditions and compensation. During the project feasibility study, Masakong village, although located above the reservoir demarcation line, was earmarked for relocation to allow for construction of labour camps and construction workshops. However, subsequently, it became evident that the resettlement process for the village would delay the appointment of construction contractors, and a decision was made by the project, in consultation with the community, to establish a permanent relocation site at Masakong (in-situ relocation). During the consultations that spanned over several years, expectations were raised and sometimes conflicting messages communicated from different project teams, thus creating expectations and eventual risk of reputational damage. This led to erosion of trust and communities disrupting construction works on site due to their dissatisfaction, with the involvement of Civil Society Organisations, lenders and development partners. Ultimately, the project is implementing a development benefit for the community. Experience has shown that effective stakeholder engagement from the onset, proper documentation and continuous communication with affected communities and households, is essential to manage community expectations and protect the project image.

1. INTRODUCTION

The Lesotho Highlands Water Project (LHWP) is a multi-billion Maloti/Rand bi-national project which was established by the Treaty of 1986 signed between the governments of the Kingdom of Lesotho and the Republic of South Africa. The LHWP includes large scale civil engineering, socio-economic and environmental components. The LHWP will harness the water resources of the highlands of Lesotho through the construction of a series of dams and tunnels for the mutual benefit of the Kingdom of Lesotho and the Republic of South Africa. The first phase (Phase I) of the four-phased project was completed in 2003, and the second phase (Phase II) is currently underway, with the construction of advance infrastructure (roads, powerlines, telecommunications and accommodation) completed and the construction of the Polihali Dam and Transfer Tunnel at advanced stages.

Implementation of Phase II of the LHWP will require land acquisition for occupation by contractors appointed by the Project. In order to adhere to the legal and administrative framework, as well as international best practice in terms of commitments to affected communities, the Lesotho Highlands Development Authority (LHDA) – the implementing arm of the LHWP within Lesotho- engaged Consultants to plan and implement measures to address resettlement impacts associated with the implementation of Phase II, through the development and implementation of Resettlement Action Plans (RAP). In keeping with international best practice, avoidance of economic and/or physical displacement is the overarching principle that has been applied by the LHDA.

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The Phase II Feasibility Study identified 72 villages potentially affected by the Polihali Reservoir and approximately 2,700 households will be affected by several components of the project, as per Table 1 below. More than 1,800 households will be affected as a result of the loss of land and natural resources (economic displacement) and the loss of residential structures (physical displacement), Of these, approximately 350 households will be physically displaced due to inundation and secondary impacts associated with the impoundment of the Dam.

Table 1: Number of Households Affected by Polihali Dam Construction Components

Project Component	Affected Households	
	Number	Percentage
Polihali Western Access Corridor (PWAR and Bulk Power Supply)	**537**	**19.9**
Polihali North-East Access Road	**131**	**4.9**
Polihali Site Establishment and Reservoir Area*	**1,858**	**68.9**
Transfer Tunnel West**	**169**	**6.3**
Total	**2,695**	**100.0**

Large-scale infrastructure projects such as the Lesotho Highlands Water Project (LHWP) play a central role in national development strategies. While these projects often bring substantial economic benefits, they are also associated with significant social and environmental costs. One of the most challenging issues relates to displacement and resettlement of communities affected within the project footprint. Such processes disrupt livelihoods, alter cultural and social systems, and create expectations of compensation or improved living standards. If not well managed, these dynamics may generate grievances, erode trust, and may cause delays or even failure of projects.

This paper presents the case of Masakong village, which is economically displaced by the construction of LHWP II. Although the village was located outside the reservoir inundation zone, it was earmarked for relocation to make way for construction activities. The case illustrates how inconsistent communication and prolonged consultations created unrealistic community expectations, ultimately disrupting construction activities and exposing the project to reputational risk. It also highlights the importance of robust stakeholder engagement and communication strategies.

The purpose of this paper is twofold: (i) to document the Masakong resettlement planning process as a case study of adaptive project management, and (ii) to identify lessons for practitioners managing community expectations in large infrastructure projects.

2. CONTEXT

Globally, development-induced displacement has been recognised as one of the most sensitive aspects of infrastructure projects. The World Bank's Environmental and Social Framework (2017) and the International Finance Corporation's (IFC) Performance Standard 5 on Land Acquisition and Involuntary Resettlement (2012) provide guidance to minimise negative impacts and ensure affected persons are not left worse off. These standards emphasise the importance of consultation, timely compensation, livelihood restoration, and—where possible—the provision of development benefits.

Literature on resettlement highlights recurring challenges: underestimation of social impacts, delays in land acquisition, inadequate or inequitable compensation, and weak communication with affected

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populations (Cernea, 1997; Scudder, 2005). Community expectations often extend beyond compensation to include improved access to services, housing, and employment opportunities. When project teams fail to manage these expectations transparently, mistrust and resistance may follow.

In Africa, where infrastructure projects often intersect with vulnerable rural communities, the risks of reputational damage are heightened by the involvement of international lenders, civil society organisations, and advocacy groups. The Masakong case contributes to this body of knowledge by showing how an in-situ resettlement decision was both a solution to project delays and a +trigger for community tensions.

3. PROBLEM STATEMENT

Masakong is located along newly developed access roads to Polihali Village (an accommodation facility built for the project) and the Polihali Operations Centre (the future centre of control for the Polihali Dam) and in the vicinity of other advance infrastructure components, with 47 households, as depicted in Figure 1 below.

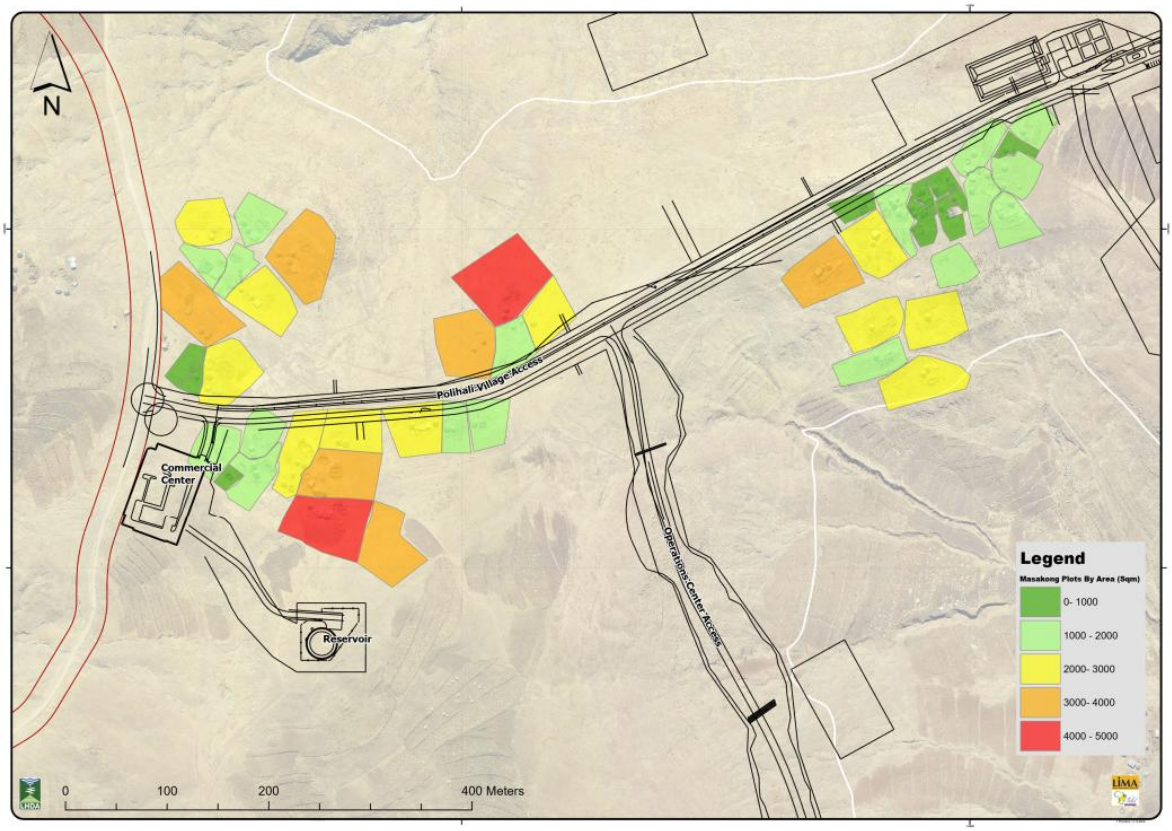

Figure 1: Masakong and Ha Tlhakola Villages and Homestead Layouts

Although the village is above the Reservoir Demarcation Line (RDL; 2,080 metres above sea-level), it was earmarked for resettlement during the Feasibility Study phase for the establishment of labour camps and construction workshops (blue polygons in Figure 2 below). The villagers were consequently informed (prior to 2013) that they are in an area required for Project developments and therefore to be resettled.

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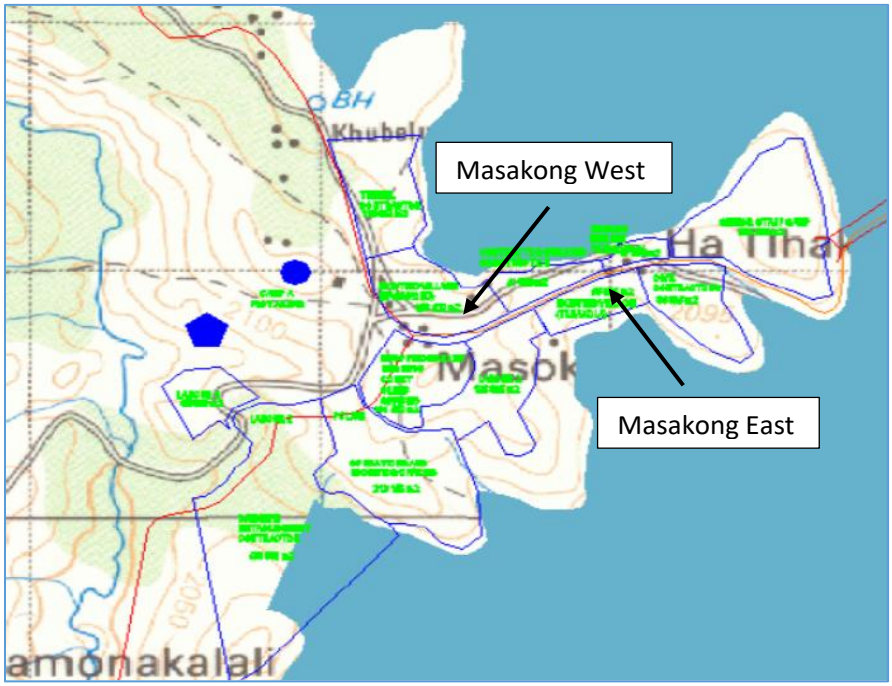

Figure 2: Proposed Infrastructure Developments in Masakong from Feasibility Study

After the appointment of the consultant responsible for the town planning, architectural design and construction supervision of infrastructure in the site establishment area (SEA) such as the Polihali Village and labour/management camps, efforts were made in consultation with the Resettlement Planning consultant to reposition and optimise infrastructure components such as the management/labour camps, the Polihali Village and the Operations Centre and associated access roads, to minimise potential impacts on the village. As the interface management between the consultants and LHDA continued, it was further observed that the resettlement of the village would delay the appointment of contractors for the construction of the Polihali Village and related civil works considering that resettlement must be completed prior to development, in terms of the law, international best practise and Project's Compensation Policy.

According to the Section 3.5.1.3 of LHWP Phase II Compensation Policy, there are three relocation options for those households that are eligible for resettlement as shown below:

- Local relocation - when affected households relocate within the vicinity of their village, allowing them continued use of unaffected assets.
- Relocation to Project-designated sites in the Project area selected by affected households in collaboration with LHDA, local authorities and, where applicable, host communities.
- Self-relocation - when affected households relocate to a place of their own choice because of social and/or economic factors, as opposed to a Project resettlement site. Where self-relocation entails a move to a location outside the Project area, all entitlements will be paid as a lump sum.

As a result, several options were discussed within LHDA to see how the emerging impacts on the village could be mitigated. Ultimately, in 2017, LHDA presented the following resettlement options to the Masakong community for consideration:

- Option 1 - Establishment of a permanent resettlement site at Masakong (local relocation).

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- Option 2 - Establishment of a permanent new resettlement site elsewhere (relocation to project designated site).
- Option 3 - Temporary resettlement and subsequent return to the existing village.
- Option 4 - Cash compensation for all losses.

4. CONSULTATION WITH THE COMMUNITIES

During consultation with the Masakong community, villagers unanimously selected Option 1 (establishment of a permanent relocation site at Masakong), which was also the LHDA's preferred option. The option included the reorganisation of the village to meet town planning standards, with formally surveyed and demarcated homestead sites, formalised internal village roads and additional demarcated homesteads sites for future village expansion, hereinafter referred to as formalisation.

Following the consultations and the selection of Option 1 by the community, the resettlement consultant proceeded to engage with the community on the available resettlement options, which included confirmation of the temporary acquisition (annual cash payments) of land assets required for the infrastructure developments (notably the Polihali Village access road) while the formalisation of the village layout was being finalised. This involved numerous community meetings, individual household consultations and focus group discussions with the different demographic categories (women, men, youth and the elderly).

The Masakong households supported in-situ resettlement via the formalisation option to avoid the undesirable resettlement of the village to a host community. Advertently the resettlement choice of the community also avoided delays in the appointment of construction contractors in the SEA. In compliance with local legislation and planning regulations, the Seate Community Council through the Physical Planners participated in the entire process, and the formalisation layout was subsequently approved by the Council. However, formalisation would affect several households in that the redrawing of homestead boundaries would require a number of households to be resettled elsewhere in the formalised village layout for the regularisation of homestead plot sizes and internal village roads. Through an iterative participatory process households displaced by the formalisation process were given the opportunity to choose new residential plots.

During the consultation process, it became evident that formalisation would add further inconvenience, namely that the development of their homesteads, which was informally restricted as a result of the gazetting of a Selected Development Area (SDA) in 2012 and the announcement that the village would be part of the Phase II resettlement programme, would be kept in abeyance until the implementation of the village formalisation. LHDA requested the community to be patient until the construction of the Polihali Village access road (one of the first Advance Infrastructure construction activities) is complete and undertook to revert with an incentive that would make their stay in Masakong worthwhile.

It is important to note that while the households of Masakong initially agreed to remain in the village, they endured the adverse construction impacts such as noise and dust pollution as well as influx of labour and 'newcomers' in their area, due to its proximity to the construction site, for more than five years. As a result, community complaints due to delayed resettlement evolved, leading to the disruption of construction works, work stoppages and consequential project delays and hike in project costs. Following a number of internal consultations and engagements with the community to abate the complaints, the LHDA acknowledged that expectations that they would be part of the resettlement programme had been raised and that households should, thus, be given the opportunity to select their resettlement option. To minimise physical displacement, LHDA proposed a development benefit in the form of a 20m^2 rectangular house or a lump sum equivalent (LSL 186,764.00) for households who would ultimately choose to remain in Masakong. Other households who chose to resettle to a Project designated relocation site or self-relocate, would only get their resettlement benefits, in line with the

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Compensation Policy. Following consultations with the community on the proposed development benefit and resettlement options, the outcome of resettlement is as presented in the table below:

Table 2: Household Resettlement Analysis

Compensation Option	Number of Households
Remain in Masakong	16
Relocate to Project relocation site	10
Self-relocation	21
Total	47

Although the development option was introduced with the intention of easing tensions, the community did not view it as a meaningful response to their concerns. Instead, the introduction of yet another "option" reinforced their perception that the original commitment to resettlement had been abandoned. Rather than resolving the conflict, this approach created space for renewed demands, undermined earlier agreements, and forced the process of confirming resettlement preferences to restart from the beginning for most households. This demonstrates that the longer communities are kept waiting, the more trust is eroded, people begin to say, "we don't believe you." As new options are repeatedly introduced, confusion deepens, and communities grow demoralised. Prolonged controversy weakens social cohesion: not everyone has the stamina for ongoing confrontation, and many withdraw to the margins of the debate. In the case of Masakong, for example, only a small group of younger people continued to push the issue forward, while most community members, apart from those who were confident their aspirations would be met, had become exhausted by the process.

5. KEY CHALLENGES

Several key challenges emerged from the Masakong case:

- **Conflicting communication:** Different project units and consultants engaged the community at different stages, which the community interpreted as contradictory information, often undocumented, thus giving rise to confusion.
- **Information Imbalance:** Lack of access to relevant project information by the community leading to growing mistrust.
- **Delayed decision-making:** The time taken to resolve Masakong resettlement created uncertainty and anxiety in the community. This has led to some households opting to resettle to locations of their own choice, outside Masakong and even outside the project area. These responses often made out of frustration with the project poses a risk to LHDA as the developer since they are responsible to ensure that households have arranged for replacement housing in the areas of their choice.
- **Documentation gaps:** Records of consultations and agreements were not always systematically kept, making it difficult to track what was promised and agreed over the years.
- **Raised expectations:** Prolonged engagement provided an opportunity for raised expectations about housing upgrades, service provision, and employment opportunities. For as long as the formalisation and any other benefit that had been offered was not delivered, aspirations would grow and demands increase.
- **Reputational risk:** Project delays and community protests attracted scrutiny from Community Based Organisations, lenders and development partners, affecting the project's image internationally.

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- **Interference by external stakeholders**: Delayed responses and deferred implementation of the resettlement programme provided opportunities for external stakeholders to influence the community in opposition to the project, thus undermining the legitimacy of the process.

6. LESSONS LEARNT

For a project of this nature to be successfully implemented, it is important for developers to implement an effective stakeholder engagement programme to ensure that community expectations are integrated from inception to the execution of the project. Below are key lessons learnt from the case of Masakong:

- **Management of community expectations**: Extensive interaction with communities to understand their expectations and mitigate perceived impacts is a fundamental principle.
- **Early and consistent engagement**: Consultations must begin early-on and be maintained consistently, with one coordinated communication channel.
- **Timely decision-making**: Delays in making and communicating critical decisions—such as eligibility for resettlement, compensation packages, relocation site selection, or timelines—can severely undermine the success of resettlement and relocation programmes.
- **Clear documentation**: Every consultation must be documented, with records shared internally to ensure consistent messaging.
- **Leveraging external stakeholders**: Community Based Organisations, lenders, and development partners can play constructive roles if engaged transparently and proactively rather than reactively.
- **Beyond compensation**: Development benefits can transform resettlement from a source of conflict into an opportunity for long-term community development.
- **Interventions that target vulnerable groups**: Design interventions to protect vulnerable groups such as women to be involved in decision-making processes and ensure equal access to benefits.

7. RECOMMENDATIONS

Based on the Masakong experience, the following recommendations are offered for future projects:

- Establish a single, dedicated stakeholder engagement office with trained staff to ensure consistent communication.
- Integrate a transparent and accessible grievance redress mechanism from the start, with clear timelines for response and resolution.
- Use participatory monitoring and evaluation interventions to track community satisfaction with engagement processes.
- Build flexibility into project design to allow adaptive management of resettlement expectations.
- Communicate proactively, even if the decision is pending, so that expectations are managed.
- Document all interactions and agreements systematically to reduce misunderstandings and maintain accountability.
- Promote third-party monitoring, for example, Community Based Organisations to manage misinformation and build trust in communities.

8. CONCLUSION

Large infrastructure projects inevitably bring disruption to local communities, and resettlement processes often become flashpoints for grievances. The Masakong case shows that even when a community is not directly affected by the project, secondary impacts such as land acquisition for

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construction facilities can create significant social challenges. Mismanaged expectations, inconsistent communication, and delayed decisions can erode trust and expose projects to reputational risk.

However, the case also demonstrates that adaptive solutions, such as in-situ resettlement combined with development benefits, can restore community relations and allow projects to proceed. The key lesson is that effective stakeholder engagement, backed by clear documentation and transparent communication, is not an optional add-on but a fundamental ingredient to project success.

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IS THE TOTAL SHUTDOWN OF THE LESOTHO HIGHLANDS WATER PROJECT HYDROPOWER AND WATER DELIVERY FOR MAINTENANCE SUSTAINABLE?

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PRESENTER: REENTSENG MOLAPO

ABSTRACT

The Lesotho Highlands Water Project (LHWP) 2024-25 tunnels steel linings corrosion protection reapplication and other previous outages for maintenance brought positive benefits and adverse impacts. The adverse impacts include lost local power generation, resulting in expensive and higher electricity imports, deferred water delivery, and loss of related royalties' payments. The positive benefits include increased assurance of continued delivery of water and electricity generation, structurally safe and unimpaired tunnels system for the next thirty years. The paper analyses whether these adverse impacts are sustainable in view of the continuing climatic conditions of prolonged droughts and economic changes. The authors make a compelling case for implementation of a parallel tunnels system and hydropower plant.

1. INTRODUCTION

South Africa is a water-scarce country and does not receive average rainfall as compared to other countries. The Lesotho Highlands Water Project (LHWP) established through the 1986 Treaty between the Kingdom of Lesotho and the Republic of South Africa, plays a key role in water delivery to Gauteng Province, the economic hub of South Africa, through the Vaal River. The Vaal River spans approximately about 1,458 kilometres in length and it runs through Mpumalanga, Gauteng, Free State, North-West and Northern Cape Provinces. Water users that are affected by the tunnel's outage are some of the RSA's key water boards and water companies such as Rand Water, Midvaal Water Company, Vaal Central Water and a number of municipalities (as some Municipalities are operating as both Water Service Authorities (WSA) and Water Services Providers (WSP). They abstract raw water, treat it into potable water standards, and then distribute it to consumers (Residential, Industrial, Commercial, etc).

The LHWP harnesses the water resources of the Senqu river and its tributaries through the construction of a series of dams and tunnels for the mutual benefit of the Kingdom of Lesotho and the Republic of South Africa. The Senqu is the largest catchment in Lesotho with an area of 24 485 km^2 and supplies the LHWP reservoirs.

The project is implemented in phases, and it is envisaged to have five major reservoirs on completion as shown in Figure 1. The first phase (Phase I) was completed in 2003. It consists of Katse (1950Mm^3) and Mohale (958Mm^3) dams, seventy-two (72 MW) 'Muela Hydropower Plant (MHP) and 'Muela tail pond (6Mm^3), and the tunnels in Table 1.

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Table 1. LHWP Katse - 'Muela - Ash River Outfall tunnels capacity

Tunnel	Length (km)	Capacity (m^3/sec)
Katse to 'Muela	44.95	36.5
'Muela to Ash River Outfall	37.06	38.2

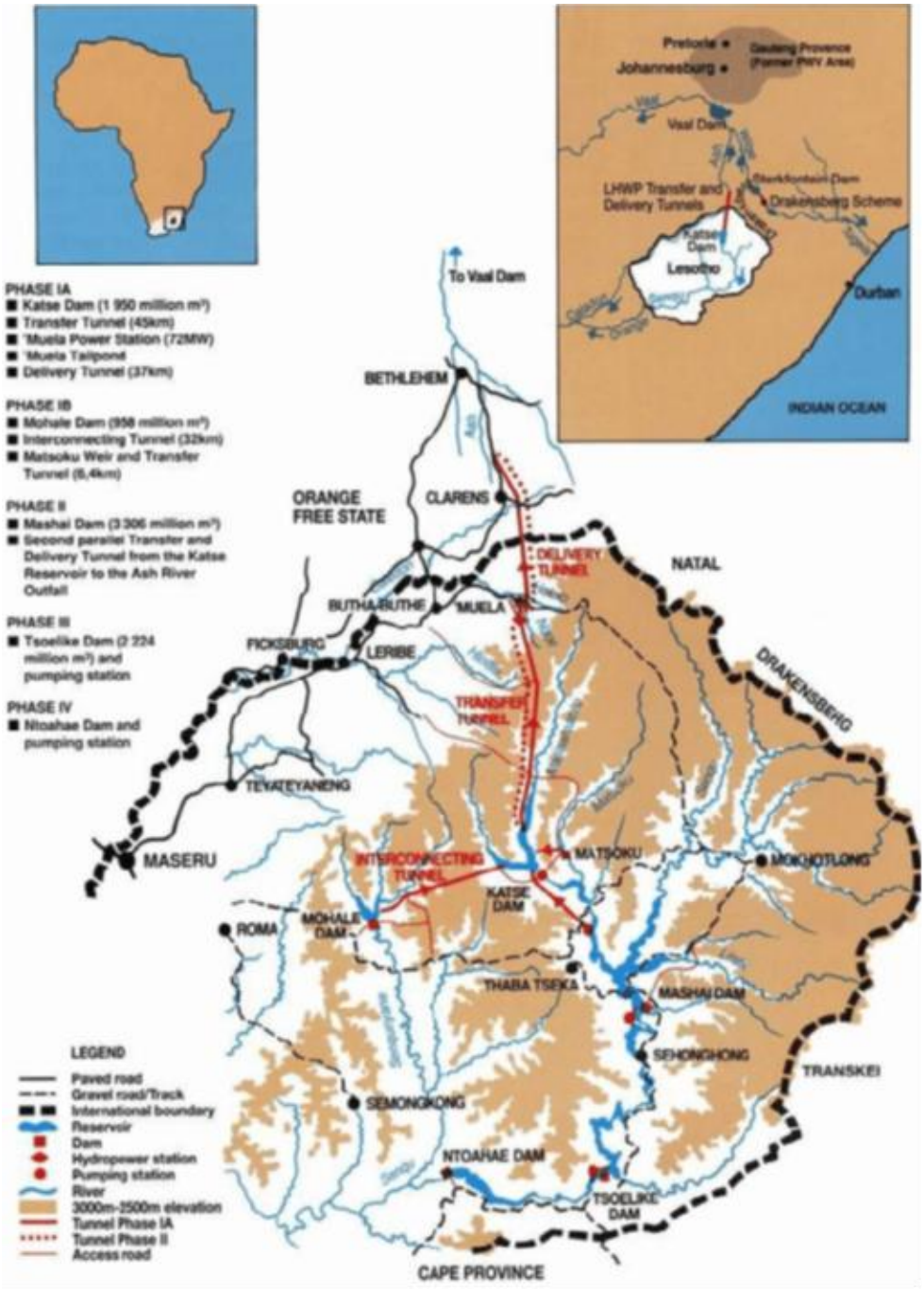

Figure 1. Lesotho Highlands Water Project

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As documented in Operations and Maintenance (O&M) Manuals for Phase I, supported by the Engineering Panel of Experts (EPoE) and the Original Equipment Manufacturers (OEM), as well as feedback from maintenance activities undertaken, major shutdowns for inspections and maintenance are necessary for prolonging the life cycle of the Lesotho Highlands Water Project (LHWP) Systems and Equipment.

This paper discusses the sustainability of the shutdown impact on both countries at various levels i.e. Kingdom of Lesotho's water delivery, royalties, power generation and Republic of South Africa water delivery) and possibility of shortages of raw water for critical areas including the economic heartland of South Africa, Gauteng. Sustainability is focused on meeting present-day needs without compromising the ability of future generations to meet their own needs, encompassing environmental, social, and economic dimensions, see Figure 2. The concept of sustainable access to water can be further linked to the United Nations Sustainable Development Goal number 6.

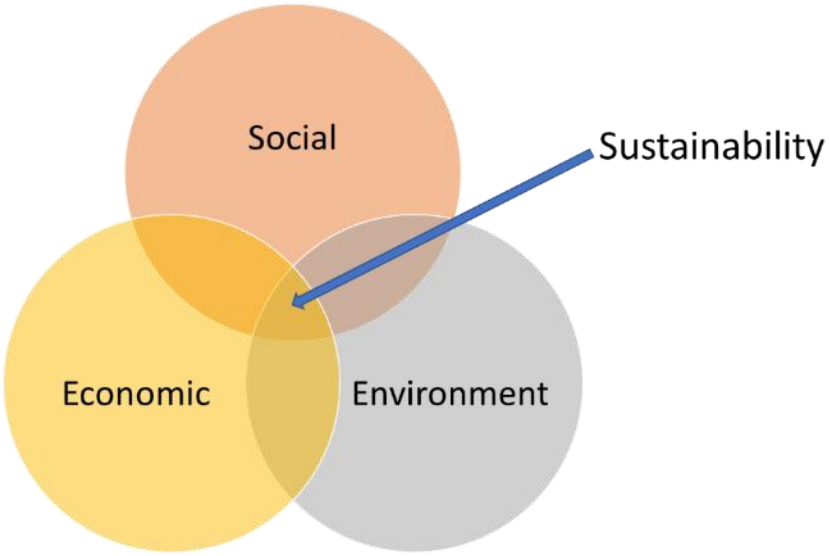

Figure 2. Three Pillars of Sustainability

When the shutdown is underway, many of these indicated factors are impacted negatively. The paper goes further to hypothesise that a second hydropower, interconnecting and delivery tunnels will enhance the sustainability of the project.

2. MAINTENANCE OF THE LHWP TUNNELS AND HYDROPOWER PLANT

All infrastructure assets are subject to gradual wear and aging. Regular and timely inspections and preventative maintenance, identify and mitigate the gradual wear and effects of aging and offset the loss of the asset before it happens. Studies have shown that neglecting maintenance decreases the efficiency of the infrastructure, increases the burden of repair for future generations, and more critically holds back the economic outcomes provided by the infrastructure concerned.

The operations and maintenance manuals of the LHWP tunnel prescribe that the tunnels should be dewatered for inspection at intervals between five (minimum) to ten (maximum) years.

Since commissioning, the delivery tunnel was closed for a short inspection and repairs on the RSA section in March 1998 and major inspection and maintenance repairs were undertaken in February

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2003. In March to April 2011 a forced outage was undertaken when it was discovered during normal annual outages that emergency repairs were needed at the 'Muela Intake.

In October to November 2012 a detailed inspection of the delivery and transfer tunnels from Katse Intake to Clarens outfall was undertaken. In 2019 another three months inspection and maintenance work was undertaken, followed by the recent 2024-25 six months outage.

All the outages required a total shutdown of the water delivery system and hydropower generation. There are limited options that can be employed to avoid service disruptions to occur. Maybe in the future advancement in technology may allow inspections without the need to dewater the system, i.e., use of underwater robotics. However, maintenance will still involve service disruptions.

The outages in turn brought positive benefits and adverse impacts to the Kingdon of Lesotho and the Republic of South Africa. The adverse impacts include lost local power generation, resulting in expensive and higher electricity imports, deferred RSA water delivery and related royalties' payments to Lesotho, and positive benefits include assurance of continued supply of water to RSA and electricity generation for Lesotho, structural safe and unimpaired tunnels system.

The LHWP similar to other major infrastructure is not typically designed in duplicates for maintenance but rather employs design principles focused on maintainability, such as the use of advanced materials, integrating operations and maintenance expertise into the design phase to ensure long term service and facilitate occasional maintenance that inevitably also require total shutdown of the entire system.

3. THE IMPORTANCE OF THE LHWP

The LHWP delivered nearly 19 580Mm³ of water between January 1998 and December 2024. These deliveries will increase to 1 270 Mm^3 annually in 2030 on completion of Phase II. The deliveries are managed through an approved annual schedule that indicates what amount has to be delivered each and every month. As the water is delivered it generates electricity for Lesotho, i.e., electricity generation is tied to water delivery and vice versa.

The delivery water from the LHWP is arguably one of the most important transfers into the Vaal River System, as it sustains Gauteng and surrounding provinces. Out of the nine provinces of South Africa, Gauteng Province has the smallest geographical area but the biggest urban area, where 14.8 million people, comprising 25.5% of the total national population, live. As the financial, commercial, industrial, and government hub of the nation, the province of Gauteng, accounts for 37% of the country's gross domestic product. Any restraint for whatever reasons on these water deliveries have and will, arguably, have a significant and negative impact on the RSA economy.

Table 2. LHWP Water deliveries to RSA & Royalties payments contribution to GDP

Year	Water delivery MCM	Royalties Contribution to Lesotho GDP
2019-20	639.15	2%
2020-21	779.99	3%
2021-22	901.87	4%
2022-23	799.05	4%
2023-24	779.04	4%

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Table 2 indicates the water deliveries to RSA in fiscal year 2019 to 2023 and the royalty's contribution to Lesotho GDP. The royalties contributed about three (3) percent into the total national revenue of the Kingdom of Lesotho. In 2024-25 the royalties' payments increased to M2.9billion and 8% of GDP. This was a 45% average increase between 2023 and 2024. This is an increase in real revenue because it exceeds the Lesotho inflation rate of 5%. And the royalty's payments are going to increase further when the deliveries from Phase II come online and the LHWP contribution to the fiscus of the Kingdom of Lesotho will undoubtedly become even more significant.

The 'Muela hydropower plant similarly is the critical component of the LHWP, and it has been supplying hydropower electricity since its commissioning in 1998. At the start of the operations the power station was supplying all the electrical power needed in the country and this has gradually decreased and to date the power plant produces almost fifty percent of the energy required, see Table 3.

Table 3. LHWP Electricity generation & contribution to national electricity demand

Year	Electricity Generated GWh	Contribution to National Energy Demand
2019-20	391	43%
2020-21	426	48%
2021-22	530	55%
2022-23	480	48%
2023-24	471	51%

The gradual decrease is due to increase in population and industrial activities requiring electricity. This will also increase by 30% when Phase II of the project is commissioned in 2030. The increase will come about due to more water being available to run the 'Muela turbines at full capacity all the time.

It is therefore arguable that there are grave consequences for any interruption on the water deliveries from the LHWP. Studies on the impact of water deliveries and hence on the electricity generation due to climate change indicate that LHWP water deliveries will mostly be met even under future drought conditions. The deliveries will be met provided the system is not under maintenance and the tunnel system is not shut down.

On equal scale, the electricity generation at the 'Muela hydropower fulfils a crucial role. Without this generation, Lesotho is dependent on exporting power from RSA and thus becomes susceptible to rolling blackouts (load shedding) due to insufficient electricity supply from Eskom. The power cuts affect daily life, impacting businesses and residents alike.

4. THE LHWP CLIMATE CHANGE ASSESSMENT

The LHWP is predominantly fed by rainfall, snow and recent studies on climate change impact assessment have established that in the coming years Lesotho hydropower generation and water deliveries to South Africa will increasingly be vulnerable to unmet conditions. Specifically, the analysis finds that the non-implementation of the additional phases envisaged exposes the system to higher situations of unmet hydropower and water deliveries. Figure 3 indicates a recent case where the LHWP storage was assessed against a drought that continued for over ten years starting in 2011. The storage did not recover during the summer rains year after year until it reached the lowest level of 29% in October 2020.

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The studies further indicate that delays in implementing the LHWP could undermine water security in South Africa it nearly happened in the year 2019-20 and limit the economic and development benefits that accrue to Lesotho.

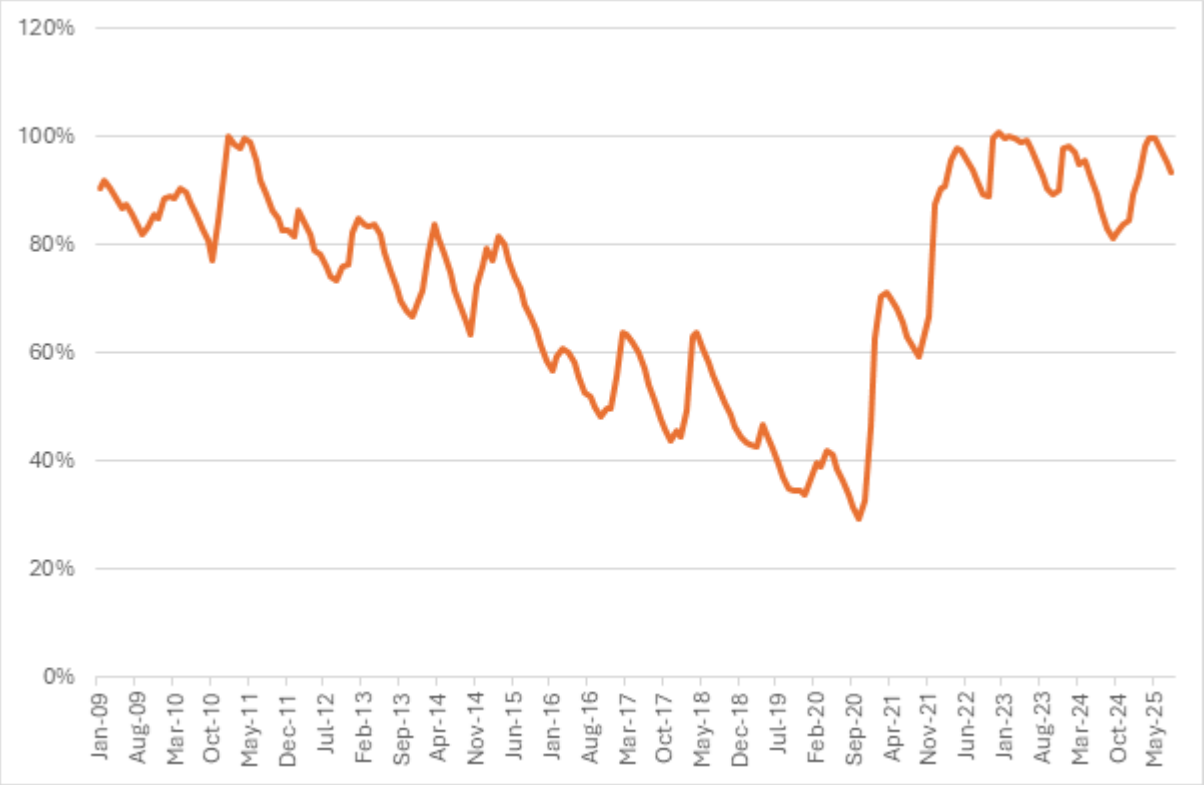

Figure 3. Drought impact on LHWP total storage

The analysis then finds that various adaptation strategies, including full construction of the proposed Polihali dam and the full construction of all five phases of the LHWP infrastructure, both increase the amount of water deliveries to South Africa and increase their reliability over a wider range of climatic conditions.

5. 2024-2025 HYDROPOWER AND TUNNELS OUTAGE-LESSONS LEARNED.

Over and above the positive benefits and adverse impacts already discussed above, the previous and recent LHWP hydropower and tunnels maintenance outage highlighted the following lessons,

- The critical importance of proactive and continuous maintenance of the hydropower, water transfer, and delivery system to ensure continued delivery of water to RSA, electricity generation for Lesotho and related benefits derived from the LHWP,
- The need for thorough stakeholder engagement for contingency planning in South Africa and Lesotho, effective water conservation, and responsible electricity usage measures during outages. Collaboration between the LHDA, the TCTA), RSA Department of Water and Sanitation and Lesotho Electricity Company (LEC) was crucial for arranging alternative electricity and water supplies during the shutdown. Clear communication with stakeholders and the public about the extended outage timeline and the necessity of the work was vital for public understanding and cooperation,

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- The outages have a diverse range of requirements from advanced materials and technologies, consultants and contractors' expertise, skills from LHDA employees, local labour recruitment, which demand advance planning and procurement.
- The challenges of extended timelines due to unforeseen challenges met during the implementation, highlighted the need for flexible outage schedules or implementation options.

6. SUSTAINING MAINTENANCE OF THE LHWP AT ALL TIMES

In order to ensure the sustainability of the LHWP, the project must also provide the services and benefits expected of it throughout its life cycle. Long-term maintenance and operations plan that include contingency plans for extreme events can help ensure service continuity to a certain extent. However, long term prediction of drought is still a challenge and difficult to forecast accurately. Planning for LHWP outages requires several years to undertake, this is because it sometimes requires special materials, specialist consultants and contractors that require procuring ahead of time.

The second phase (Phase II) currently underway is being implemented differently from the initial conception indicated in the Treaty. It consists of an additional reservoir but there is no second hydropower and interconnecting tunnel from Katse to 'Muela. However, subsequent phases are yet to be agreed upon. This second interconnecting tunnel was envisaged to accommodate the additional quantities of water deliveries to RSA and offer Lesotho an opportunity to expand the capacity of the 'Muela Hydropower Plant. As stated earlier, the current transfer and delivery tunnel having a capacity of 36.5 and 38.2 m^3/sec, respectively. This is below the ultimate delivery of seventy (70) m^3/sec when all the LHWP phases are implemented.

The additional benefit this tunnel will offer, is enabling maintenance of the system without a total shut down, and the negative impacts to the disruptions of services and benefits outcome would greatly be alleviated or reduced altogether.

7. CONCLUSION

Inspections and maintenance activities are essential to extend the life span of all LHWP infrastructure assets and suitable long-term maintenance and operations plan that includes contingency plans for extreme events can help ensure service continuity to a certain extent.

However, construction of further phases of the project including doubling of the tunnels from Katse to Ash River outfall and the second 'Muela hydropower plant will guarantee the availability of the services and benefits will continue to accrue even during the maintenance period. This will enable the LHDA to fulfil its mandate of implementing, operating and maintain the project to international standards as directed by the LHWP Treaty.

8. ACKNOWLEDGEMENTS

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THE 2024-25 LESOTHO HIGHLANDS WATER PROJECT HYDROPOWER, DELIVERY AND TRANSFER TUNNELS OUTAGE FOR MAINTENANCE - LESSONS LEARNT

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ABSTRACT

The Lesotho Highlands Water Project (LHWP) 2024-25 outage for tunnels steel linings corrosion protection reapplication brought positive benefits and adverse impacts to Lesotho and Republic of South Africa (RSA). The adverse impacts include lost local power generation, resulting expensive and higher electricity imports, deferred RSA water delivery and related royalties' payments to Lesotho, and positive benefits include assurance of continued supply of water to RSA and electricity generation for Lesotho, structural safe and unimpaired tunnels system for the next thirty years. The paper presents lessons learnt in outage planning, execution, risk and stakeholders' management.

1. BACKGROUND

The Phase 1A of the Lesotho Highlands Water Project, comprising of the Katse and 'Muela Dams, a 45km Transfer Tunnel (TT) to the 'Muela hydropower plant and 39 km Delivery Tunnel (DT) to the Ash River outfall in Clarens South Africa as shown in figure 1 was commissioned in 1998.

The Tunnel Boring Machines (TBMs) of the TT passed through long lengths of thick durable and in some cases non-durable basalt. Consequently, it was decided to line the entire tunnel with an in situ concrete lining. Steel linings were also required in discrete sections of the tunnel adits, bypass tunnel, hydropower penstock, and surge shaft. The steel lining was provided to prevent hydraulic jacking of the rock between the tunnel and zones of atmospheric pressure where ground cover was deemed insufficient to contain the pressurised water ways.

Similarly, the DT crosses the Hololo, Ngoajane, Caledon and Little Caledon Rivers and the tunnel alignment was maintained at a relatively high level between river crossings to facilitate construction access and drainage. And in these river crossings the tunnel is under relatively low cover and internal pressures greater than the containment afforded by the covering rock mass. In order to provide safety assurance from any uncontrolled leakage and structural support to the tunnel waterways, the tunnel is provided with watertight steel linings.

A further 220 m section located approximately 2 km from the northern end of the Caledon Tunnel is steel lined because there is insufficient cover relative to internal pressures. This steel lining is placed centrally inside the precast segmental lining provided by the TBM operation.

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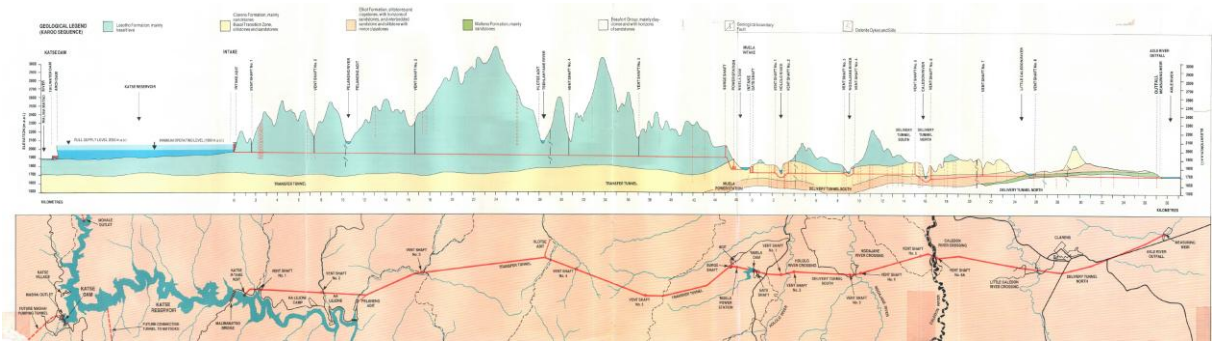

Figure 1. LHWP Phase 1 transfer and delivery tunnels layout

The Operations and Maintenance (O&M) manuals specify that detailed inspections and maintenance of the tunnels should be done at intervals not shorter than five years and at intervals not longer than ten years to reduce the deformations in the tunnel lining due to absence or presence of internal water pressure.

Since commissioning, the delivery tunnel was closed for a short inspection and repairs on the RSA section in March 1998 and a major inspection and maintenance repairs were undertaken in February 2003. In March to April 2011 a forced outage was undertaken when during normal annual outages, it was discovered that emergency repairs are needed at the 'Muela Intake.

In October to November 2012 a detailed inspection of the delivery and transfer tunnels from Katse Intake to Clarens outfall was undertaken.

However, repairs to the following components could not be performed at the time because they would have prolonged the outage period and the repairs were undertaken in 2019.

- Katse Intake – Leaking concrete plugged gate openings were sealed with geomembrane.
- 'Muela Station Bypass tunnel valve was replaced, and steel lining corrosion protection was spot repaired.
- 'Ngoajane Flow Measuring Equipment – Magnetic & Ultrasonic flow meters sensors reached end of their service life and were replaced.
- Muela HP Station Surge Shaft – more detailed inspection of the shaft and adjacent tunnels to find and repair the source of leakage into the Penstock Valve Chamber and Muela Access Adit.

During the 2019 outage, the inspection on the tunnels steel linings revealed increased extensive blistering, chips, and scrape marks compared to 2012 outage. The repairs were undertaken with on average one spot per linear meter of steel lining. Considering the above trend and the severity of the blistering in the delivery tunnel south and transfer tunnels it was concluded to grit blast every steel lined section around the entire circumference regardless of the apparent quality of the existing corrosion protection and re-apply. It was considered that this may be costly and time consuming but would at least mean that all the steel would be properly protected for another 30 years plus or minus.

2. OPTIMISED OUTAGE TIMING

As the outage required suspension of water delivery and electricity generation, a suitable starting time and duration that will have minimum impact on both countries needed to be decided upon taking the following into consideration.

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- The time chosen should be outside the winter period. During winter, electricity demand is at the highest and the non-generation of electricity at 'Muela hydropower Plant (MHP) implies Lesotho will import a lot more electricity at the time when the tariffs are highest which will increase the outage cost on the part of Government of Lesotho (GOL) on top of "losses" of royalties.
- The period should be during rainy season to reduce the impact of no transfers from LHWP system as the water requirements shall be covered by rainfall in alternative catchment areas.
- It should be during months normally requiring low water delivery quantities to reduce the deficit amount and recovery period.
- It should be warm during months, so that ambient temperatures in the tunnels are not extremely low so as to require heating the steel liners to enable proper application of corrosion coating and reduce outage cost.
- Less humidity.

Amongst many options considered October 2024 to March 2025 was found to be suitable. This option is during rainy season; outside the winter and therefore ambient temperature are higher. However, because it is during the rainy season and given the fact that there will be residual seepage in the tunnel, the humidity will be high and there will be requirement for ventilation to achieve suitable environment for corrosion protection application.

The outage period was approved in November 2020 and LHDA started planning for the works. The chosen period worked well. There was sufficient rainfall that provided adequate alternate water supply from the closed LHWP. The reservoir also filled as expected and only started to spill during the last month of the outage.

The works were in seven geographically separated areas, and it was decided to have dedicated team(s) work on each steel lining section simultaneously with others. From experience and on analysis of the works requirement, the programme for the contractor allowed for one year mobilisation period. This was to allow the contractor to recruit required number of skilled and unskilled local labour and provide them with required work and related safety training for confined space. In addition, the extended mobilisation period allowed the contractor to source any off the shelf equipment and material that was needed.

3. RISK AND STAKEHOLDER MANAGEMENT

The outage stopped delivery of water to RSA and electricity generation in Lesotho. Both commodities are critical topics in politics and public dialogues. Consultations were initiated as early as possible with several stakeholders. This extensive stakeholder engagement, while it was demanding, ensured more inclusive, socially responsible, and ultimately more successful outage by addressing the needs of all parties involved.

- Lesotho Highlands Water Commission (LHWC) for approval of the Project on behalf of the parties and their fiduciary duty for being responsible for implementation, operations, and maintenance of the LHWP.
- Trans-Caledon Tunnel Authority (TCTA) for selection of the appropriate outage time considering their stakeholders and carrying out joint outage planning.
- Lesotho Electricity Company (LEC) for advanced notification of outage timing and duration to enable them to arrange for alternative power supply for Lesotho.
- Lesotho Electricity and Water Authority (LEWA) regulation compliance on outage notification
- Local government councils, designated Ministries in Lesotho and RSA for resources mobilisation, review and confirmation of acceptable water delivery schedule and resulting deferred royalties' payment. and loss power generation impacts.
- General public and news media on the details of the outage and the resultant impacts.

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Particular lessons learnt from this are as follows,

3.1 Improved decision-making

- The different expectations and perspectives from the project, lead to more informed and well-rounded decisions.
- The project team avoided blind spots as the broad engagement ensured that various aspects of the project are considered, preventing missed opportunities or overlooked risks.

3.2 Enhanced collaboration and buy-in

- **Increased support**: Involving stakeholders in the implementation process gave them a voice and made them feel heard, fostering greater support and commitment to the project's success. The outage received ministerial support throughout and the Government of Lesotho: Minister of Natural Resources went into the tunnel and personally appreciated the working conditions and challenges brought about by the incessant seepage on controlling the environmental conditions.
- **Reduced conflicts**: Early and continuous engagement with the chiefs and local councils helped to understand and address stakeholder demands, which prevented conflicts from the public demanding employment and built stronger relationships.

3.3 Better risk and uncertainty management

- **Early risk identification:** The project team identified the risk of not meeting the water delivery requirement and loss of royalties to Lesotho. This was mitigated by increasing water delivery before and after an outage. The electricity generation disruption was mitigated by informing LEC well ahead of the outage and during the outage of encountered delays for them to secure alternative power supply to 'Muela hydropower plant.
- **Adaptability:** Understanding the challenges of the project enabled the project principals to allow the project schedule overrun within the prevailing complex social, economic, and political factors.

3.4 Project success and value adding

- **Increased project reputation:** Managing diverse stakeholder interests lead to better project outcomes and improved public perception and reputation.
- **Local labour mobilisation:** Chiefs, District administrators and local councils contributed towards needed local labour resources.
- **Social and Community Benefits:** Local communities benefited from skilled and unskilled jobs created by the outage' works.

4. IMPLEMENTATION OF THE CORROSION PROTECTION

The previous inspection of LHWP tunnels and pipes established that the corrosion protection epoxy material thickness was 600-1000 microns when originally applied in 1998. The coating material showed blistering after 14 years (2012) with extensive blistering observed after 21 years (2019).

Due to the limited outage time available for the October 2024 to March 2025 works, it was decided and agreed upon that a total Dry Film Thickness (DFT) for steel lining system of 40-50 MILS (1000-1200µm) DFT be applied in a single coat application.

All work was planned in accordance with the RSA Department of Water and Sanitation 9900 specification for corrosion protection of steel pipes and components in South Africa updated in March 2021. Previous inspections and maintenance of the LHWP pipes and tunnels indicated high humidity

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levels from 70 % to 95% and dew point temperature of the air relative to the steel had a difference of only between 1 to 2 degrees C. These are high condensing conditions and therefore corrosion protection application, is not normally permitted unless special precautions are taken. The following conditions were specified in the contract.

- The coating must have ability to tolerate high relative humidity (80%) during application and curing.
- The steel substrate temperature had to be at least 3º C above the dew point temperature of the air to prevent micro condensation which can negatively affect the adhesion of protective lining systems and negatively affect curing by slowing the polymerisation process.
- For cases where the coating application took place in conditions with relative humidity exceeding 80%, a dedicated ventilation plan had to be developed and approved by the quality control inspector.
- In addition, as a separate issue for Quality Control (QC) verification, the material supplier had to confirm in writing that the material can be applied at relative humidity between 70 and 85% as detailed above, to which they did.

The material opted for was a nano glass flake epoxy lining with 98% volume solids, Corrocoat Zip E, manufactured and supplied by the application contractor.

The tunnel relining was subdivided into nine (9) work fronts that were worked on simultaneously. The specifics of the areas are described below starting upstream at Katse dam end in Lesotho to downstream at the Ash River outfall in South Africa.

For quality compliance purposes, a suitably experienced third-party inspection consultant was employed to provide independent quality control assessment of the contractor's methodology and corrosion protection application.

4.1 Pelaneng

Located closest to Katse dam, the Pelaneng Under River Crossing (URC) comprising of thirty-six steel sections of 3.4m Internal Diameter (ID) x 3m long each. After the existing lining was removed by abrasive blasting, the bare steel was inspected. No significant pitting corrosion was found except for five areas that required some minor welding repairs.

4.2 Hlotse (Tsehlanyane)

The Hlotse access adit comprised of ten steel section of 3.0m ID x 3m long. The anticipated groundwater seepage between Pelaneng and Hlotse, proved to be a challenge. At the outset of the project, it was decided to use the Hlotse Adit to divert the water away from the 'Muela ridge. This therefore meant that Hlotse work was performed following completion of the 'Muela ridge.

After the existing lining was removed by abrasive blasting, the bare steel was inspected. No significant pitting corrosion was found, and the area was successfully relined without further challenges.

4.3 'Muela Ridge

Muela Ridge comprised of ninety-nine steel sections of 3.4m ID x 3m long each. Similar to what was found at Hlotse, the anticipated groundwater seepage between Hlotse and the Muela Ridge, proved to be a challenge. This led to extensive water and environmental control management during the refurbishment of the tunnel liners.

Sizing and management of multiple de-humidification units were employed to control selected areas where surface preparation and re-lining were to take place. Multiple air blocking devices were installed to direct and control the air movement.

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One of the challenges observed was seasonal environmental changes from during the outage from October 2024 to mid-May 2025 that impacted the direction and severity of the airflow in the tunnels. The airflow in the tunnels would change direction from north to south multiple times during the 24-hours shifts worked, impacting the de-humidifier's efficiency and the blocking devices installed to control not only the dried air, but also the dust from the abrasive blasting taking place in the nearby working areas such as the 'Muela Hydropower Plant penstock, surge shaft or Station Bypass.

No significant pitting corrosion was found. However, thirteen areas required some welding repairs. Among these repairs were leaking grout plugs that only started leaking after the original lining was removed. These were located in the upper half (6 to 9 clock position) of the tunnel steel liner.

4.4 'Muela Station Bypass

Consisting of 186 steel sections 2,0m IDx3m long and the Bypass Guard valve. Significant pitting corrosion, deeper than 2mm, was found on the 20 steel sections leading up to the Guard valve (steel sections 163-183) in the bottom half of the tunnel liner (6 o'clock position).The location and concentration of the pitting close to the guard valve could be attributed to stagnant water, resulting in bacterial corrosion. As a result. Operations team now drain the bypass on a 3-monthly basis to introduce fresh water.

The Muela bypass also experienced a unique phenomenon where grout plugs started leaking with small droplets forming only after the original lining was removed. These were located in the top half (6 to 9 clock position) of the tunnel liner.

These areas required welding repairs prior to abrasive blast cleaning.

4.5 'Muela Y-Piece, Adit, and Surge Shaft

'Muela Y-piece consists of twenty-nine steel sections. Muela Adit and the Surge Shaft consist of seven and five cans respectively.

The surge shaft proved to be the area that required some innovative engineering controls to manage the groundwater and rain, for the lining application to take place. This was achieved through various environmental blocking devices.

The 'Muela Adit was also the only access to the Muela ridge. Kilometres of air hoses and water pipes were used to continuously manage air supply and water removal.

No significant pitting corrosion was found.

4.6 'Muela Hydropower Penstock and Upstream Manifold

'Muela vertical Penstock was challenging. Being a 197m vertical drop, a specially designed temporary man cage had to be installed to facilitate the blasting and lining. The Top bend was prepared, lined, and inspected by means of rope access.

There was no pitting corrosion found after the original lining was removed. The area was successfully relined.

4.7 Hololo

Hololo Under River Crossing (URC) consisted of 143 steel sections. No pitting corrosion was found.

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This area posed a particular problem due to the access tunnel being far (1000m) from the access point. Compressed air supply, dehumidification lines and the supply and removal of blasting grit had to be intensively managed to achieve the relining.

4.8 Ngoajane

Ngoajane URC consists of 132 steel sections and two access adits, north and south with four steel sections each. There was no pitting corrosion found after the original lining was removed. The area was successfully relined.

4.9 General observations for all areas

Despite the challenges faced, all areas met the specification requirements with the environmental parameters of less than 85% Relative Humidity (RH) and the difference between substrate surface temperature and dew point (DeltaT) of more than 3ºC during surface preparation and before application or rework took place.

Dust and debris amount and size were within acceptable levels in accordance with ISO 8502-3:2017.

Profile measurements were more than the specified 50µm. All measurements were carried out in accordance American Standard Testing Method (ASTM) D4417.

All the transfer and delivery tunnel steel sections complied with single coat application of the approved epoxy to the specified DFT of 1025µm minimum, with the exception of Hololo delivery tunnel under river crossing decline section, which had a short section that was lined with a product from a different supplier due to a shortfall/unavailability of the original specified material. All the required specification QC parameters were adhered to for the alternative material supplied.

5. IMPLEMENTATION OF ROBUST SAFETY PROTOCOLS AND EMERGENCY RESPONSES

Accidents in tunnels can quickly escalate, leading to severe consequences for workers, users, and the surrounding environment. The project management put in place a robust safety protocol and emergency response requirement to the contractor and regularly reviewed performance thereof.

The contractor was required to train all human resources employed in the tunnel with appropriate safety training for working in confined spaces. The training was also used as part of the selection process as only people who displayed adequate understanding of working in confined spaces were allowed in the tunnel.

This level of attention to safety ensured that no major safety incidents occurred during the outage except where people got injured because of improper risk assessment of the task at hand, lack of concentration or absent mindedness. Those that had minor incident were given on site first aid treatment.

6. CONCLUSION

Lessons from the recent LHWP tunnel maintenance include the critical importance of proactive and continuous maintenance of steel-lined tunnels to prevent corrosion, the need for thorough stakeholder engagement for contingency planning in South Africa and Lesotho, effective water conservation and electricity responsible usage measures during outages, and the challenge of extended timelines due to unforeseen unsuitable environmental conditions (excessive seepage & high humidity) for application of the corrosion protection, highlighting the need for flexible project management.

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6.1 Proactive and continuous maintenance is essential

Corrosion Risk: Inspections revealed the corrosion protection painting on steel-lined tunnels was nearing the end of its service life, posing a significant risk of corrosion if maintenance was delayed. This reinforces the need for regular, scheduled maintenance rather than waiting for visible signs of damage.

6.2 Stakeholder engagement and contingency planning

Ensuring service supply continuity: Successful collaboration between the LHDA, the TCTA), RSA Department of Water and Sanitation and Lesotho Electricity Company (LEC) was crucial for arranging alternative electricity and water supplies during the shutdown.

Communication: Clear communication with stakeholders, including the public, about the extended outage timeline and the necessity of the work was vital for public understanding and cooperation.

6.3 Water and energy management a collective responsibility

Water conservation: South African citizens along the Ash and Liebenberg rivers were urged to conserve water to manage the temporary reduction in supply from the LHWP.

Responsible electricity usage: In Lesotho, which relies on the LHWP for power, the public was encouraged to reduce their electricity consumption to ease the strain on the grid during the outage.

6.4 Flexible project management for unforeseen works

Extended timelines: The maintenance project experienced delays and an extended completion date due to the unsuitable weather conditions required at some specific locations.

Technical challenges: Tunnels were found to require more comprehensive control of the environmental condition prior coating than initially planned, necessitating additional time for the rehabilitation process.

6.5 The Importance of skills and Technology Transfer

Staff training and local labour: The project included elements of training LHDA staff for the corrosion protection of the tunnels steel linings, Local labour recruited for the work were trained on general safety for working in confined spaces and application of the corrosion protection which is a key aspect of the overall project's success and future employment.

Equipment and methodologies: The project used a range of advanced materials and technologies in the application of the corrosion protection, environmental control, air, and power supply to the work front. Despite this success, there is still a need to device means to improve efficiency of getting pressurised air, grit, and power to the work front.

7. ACKNOWLEDGEMENTS

The authors acknowledge the permission granted by the Lesotho Highlands Development Authority for authoring this paper for SANCOLD 2025.

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MITIGATION STRATEGIES FOR SILTATION ISSUES AT BOEGOEBERG DAM

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ABSTRACT

Siltation is a significant problem affecting the operational efficiency and lifespan of dams, particularly in arid and semi-arid regions. The Boegoeberg Dam, located in the Northern Cape Province of South Africa, has experienced substantial silt deposition over the years, reducing its storage capacity and impairing water management functions. This paper presents a study on mitigating the siltation problem at Boegoeberg Dam using the NAT-SILT tools, an innovative suite of technologies designed for sustainable sediment management. The NAT-SILT approach focuses on natural processes and low-impact interventions to control sediment deposition, restore dam capacity, and improve overall water quality.

Through an integrated methodology that includes numerical & hydrological modelling, and sediment transport analysis, and with reference to the NAT-SILT tools to facilitate the identification of critical sediment sources and pathways. This allows for the implementation of targeted mitigation measures such as catchment area management, vegetation restoration, and controlled sediment flushing. Results from the application of these tools demonstrate significant improvements in sediment reduction, with enhanced water flow and increased storage efficiency at the dam. The study underscores the importance of adopting sustainable, eco-friendly sediment management practices in water infrastructure management.

1. INTRODUCTION

Siltation is a widespread challenge facing water resource infrastructure, particularly in semi-arid regions where land degradation, steep slopes, and variable rainfall intensify erosion processes. In South Africa, many dams are losing their active storage capacity due to sediment deposition, with direct consequences for water supply, irrigation, and ecological functioning. The Boegoeberg Dam, located on the Orange River between Upington and Prieska in the Northern Cape Province, is one such example where sedimentation has emerged as a critical operational concern.

Constructed in 1931 as a mass gravity wall, Boegoeberg Dam plays a vital role in supporting irrigation schemes and regional water management. The dam has a Full Supply Level (FSL) of RL 884.57 m above sea level, corresponding to a gauge plate reading of 9.67 m. Its catchment covers an area of 343 048 km², with an effective runoff-contributing area of 270 022 km². Over the decades, sediment accumulation has progressively impaired its capacity, with current estimates indicating a sedimentation level of 16.8%. This equates to an average annual loss of 0.42% of storage, representing a significant threat to the dam's long-term functionality and the livelihoods that depend on it.

Addressing this problem requires a systematic understanding of sediment sources, pathways, and deposition zones within the catchment. The objective of this study is therefore to develop mitigation strategies for siltation at Boegoeberg Dam through the guidance of the National Siltation Management

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Strategy Programme (NatSilt) and the application of numerical modelling. By integrating land cover change analysis, slope evaluation, soil characterization, and hydraulic simulations, the study seeks to identify high-risk erosion areas, quantify potential sediment contributions, and recommend targeted interventions aligned with sustainable catchment management principles.

2. NAT-SILT PROGRAMME

The National Siltation Management Strategy Programme (NatSilt) was initiated by the Department of Water and Sanitation (DWS) in collaboration with the Water Research Commission (WRC) to address the growing problem of sedimentation in South Africa's dams. The NatSilt Programme was developed as a coordinated, national-level framework to provide practical tools, methods, and guidelines for managing this challenge.

At its core, NatSilt adopts a catchment-to-reservoir approach, recognising that sediment entering dams originates in upstream catchments where erosion processes are often exacerbated by land degradation, unsustainable agricultural practices, and steep terrain. The programme categorises landscapes into source, transfer, and deposition zones, thereby enabling targeted interventions that prevent or reduce sediment delivery. Through this framework, land use changes, soil erosion dynamics, and hydrological responses are analysed holistically rather than focusing solely on in-reservoir solutions.

The programme also produced the Dam Catchment Operations Model (DCOM), a decision-support tool that assists in identifying sediment-prone areas, prioritising catchment rehabilitation, and evaluating different management scenarios. This tool integrates remote sensing data, hydrological models, and sediment yield studies to provide a clear basis for intervention. The Dam Storage Sustainability Scoring System (SSS) Tool is used to assess the sustainability level of a dam in terms of siltation risk. It ranks dams on a scale from non-sustainable to sustainable. A non-sustainable dam is one that is filled with sediment and can no longer function, while fully sustainable dams can serve both current and future generations. This tool helps determine the priority ranking of dams and the level of intervention needed.

NatSilt emphasises low-impact, sustainable interventions such as re-vegetation, wetland restoration, gully control, and improved land use management. It also promotes stakeholder engagement, recognising that effective sediment management requires collaboration across local communities, farmers, government, and water managers.

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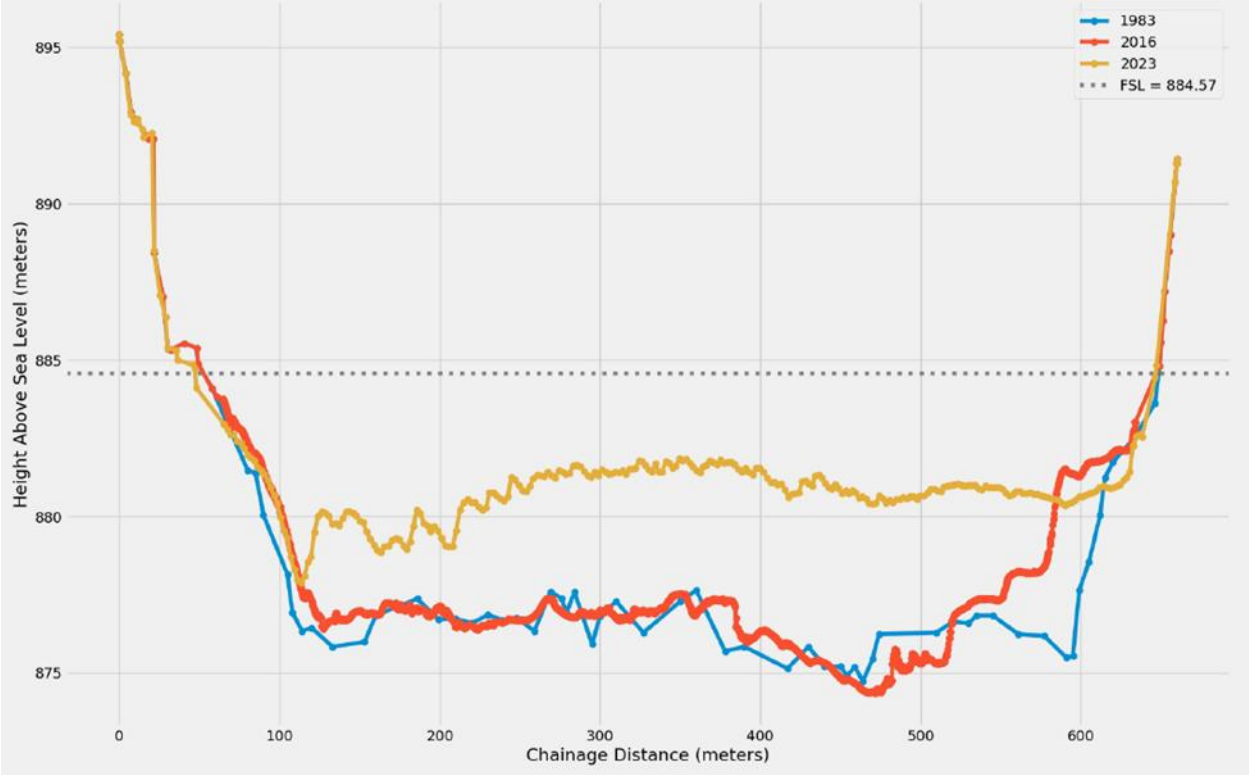

Figure 1: Sediment levels at the Dam over time (DWS)

3. METHODOLOGY

The methodology adopted in this study integrated geospatial analysis, soil and terrain data, and hydrological modelling to evaluate sediment dynamics within the Boegoeberg Dam catchment. The approach focused on identifying land degradation, quantifying erosion potential, and modelling overland flow velocities to assess sediment generation and delivery processes. All analyses were framed within the NatSilt Programme's conceptual structure of source, transfer, and deposition zones.

3.1 Land Cover Analyses

Multi-year national land cover datasets (2018 and 2022) were processed in QGIS to detect spatial and temporal patterns of land cover change. Raster datasets were clipped to the reduced catchment extent and reclassified into key categories relevant to sedimentation risk, including grassland, shrubland, eroded lands, wetlands, cultivated areas, and bush. Dissolve and reclassification operations were applied to simplify classes for analysis. A change detection matrix was then generated to quantify transitions between classes, with particular attention to conversions into eroded lands and degraded shrublands, which are critical sediment source zones.

3.2 Slope and Terrain Analysis

A Digital Elevation Model (DEM) was used to conduct slope analysis. Slopes were classified into gradient categories, and data were overlaid with land cover classes to determine the spatial overlap between degraded vegetation cover and steep slopes, thereby identifying high-risk zones for erosion.

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3.3 Hydraulic Modelling: Rain-on-Grid Simulation

The hydrological and hydraulic simulations were conducted using the Rain-on-Grid functionality within HEC-RAS 2D. This approach enabled the modelling of spatially distributed rainfall-runoff responses and overland flow velocities across the catchment. Uniform rainfall inputs were applied to the DEM-based computational grid, and infiltration characteristics informed by soil data were incorporated. The model outputs provided velocity maps for overland flow, which were compared against critical erosion thresholds derived from Hjulström-type relationships and South African soil studies.

4. THE CATCHMENT

Bogoeberg Dam boasts a catchment area of 343 048 km², with an effective runoff-contributing area of 270 022 km². The largest part of the catchment falls within the Grassland biome with the vegetation type of low shrubland, grassland and dense bush. The surface hydrology system consists of mainly the Orange River, with four major dams upstream of Boegoeberg such as Gariep dam and Van der Kloof dam (Orange River) and Vaal dam and Bloemhof dam (Vaal River), as seen in Figure 2 below.

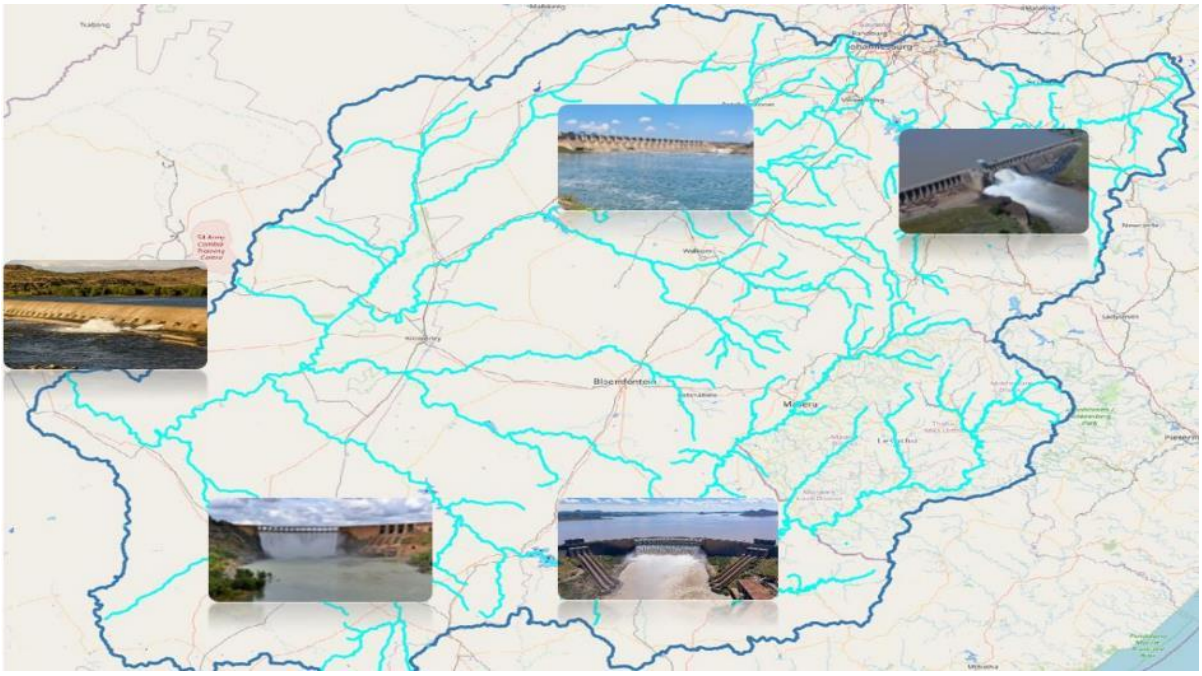

Figure 2: Boegoeberg Dam Catchment Area

It is noteworthy that the entire territory of Lesotho is situated within the broader catchment area. However, the scope of this study is confined to the sub-catchment located downstream of all major dams upstream of the Boegoeberg Dam. This approach assumes that the majority of sediment transported to Boegoeberg Dam originates from the intervening areas located upstream of Boegoeberg, but downstream of the four other major dams shown. The reduced catchment area extends across 37 Quaternary Catchments, namely C33C; C51K-M; C52K-L; C91B-E; C92A-C; D31B-C; D32F-J; D33A-K; D62A-H, J; and D71A-D; as well as D72A-C, see Figure 3.

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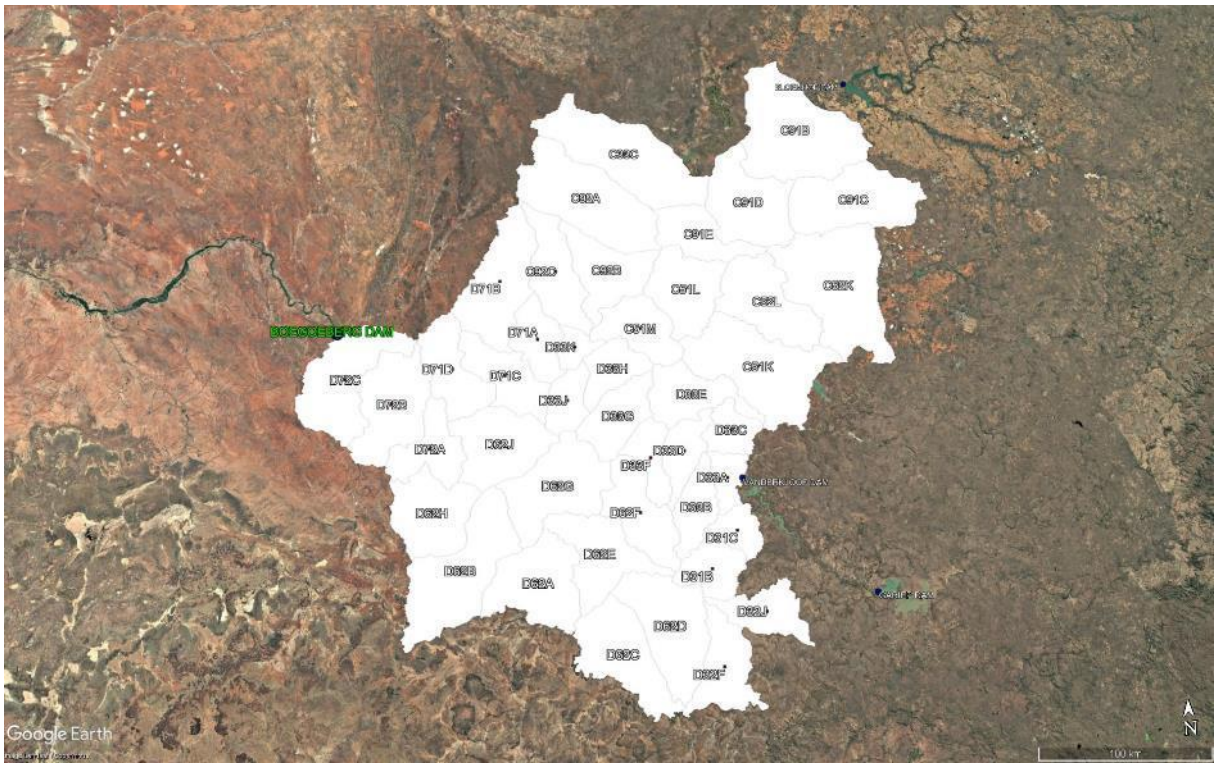

Figure 3: Relevant quaternary catchments downstream of the major dams

5. LANDCOVER AND TERRAIN

Land cover data formed the foundation of this study, as vegetation type and ground cover are primary controls on soil protection and erosion potential. For this study, multi-year national land cover datasets were obtained for 1990, 2018, 2020 and 2022. The South African National Land Cover (SANLC) classification system recognises 73 distinct land cover classes, capturing the full diversity of natural, agricultural, urban, and industrial landscapes across the country. These classes range from forests, shrublands, grasslands, wetlands, and bare surfaces to cultivated lands, settlements, and mining areas. This detailed framework provides a consistent national standard for assessing land cover change, ecosystem dynamics, and environmental impacts such as erosion and sedimentation.

These datasets were clipped to the boundaries of the reduced Boegoeberg Dam catchment, see Figure 4, which encompasses 37 Quaternary Catchments contributing directly to the dam. To conduct the landcover change analyses, the clipped raster datasets were first reclassified into key land cover groups relevant to sediment dynamics: grassland, shrubland, eroded lands, wetlands, cultivated fields, bush and shrubland, forested areas, and water bodies. This reclassification simplified the original datasets and enabled direct comparison across time periods. To ensure consistency in the analyses, the datasets were harmonised by applying identical classification schemes, coordinate systems, and resolutions.

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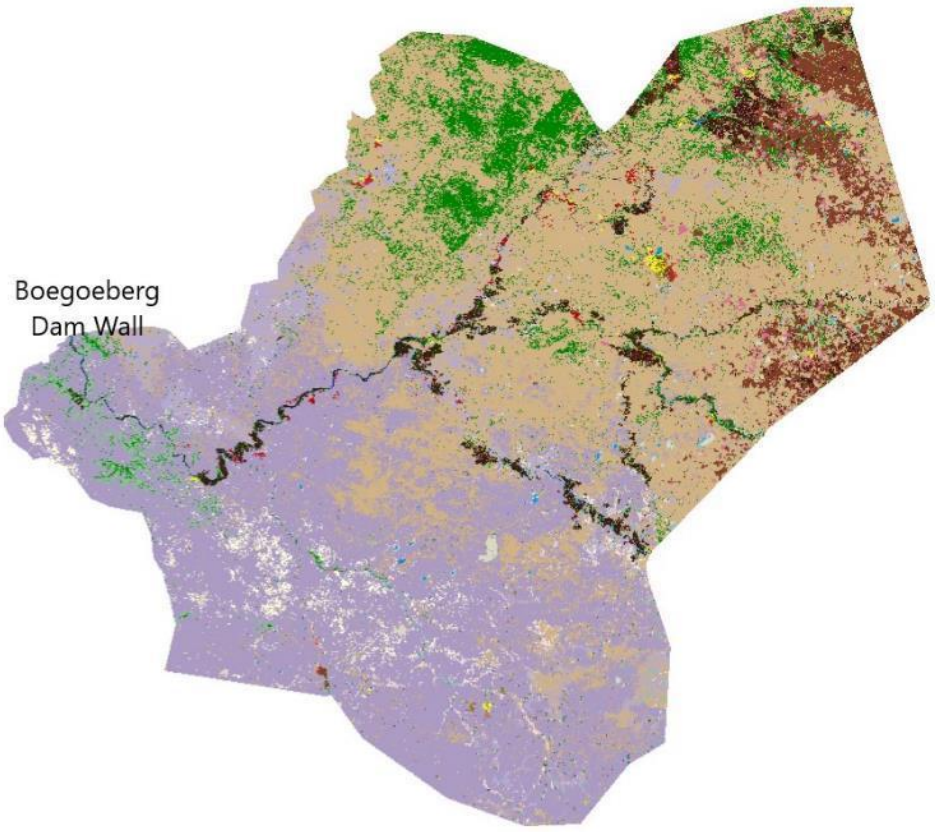

Figure 4: Clipped Landcover data

- To reduce the analyses time and also considering the type of land cover usually found in the Northern Cape, the land cover data was reclassified into the following categories, and the output can be visualised in Figure 5:
- Contiguous forest (combined very high, high and medium), low forest and thicket. - 1
- Grasslands - 2
- Wetlands - 3
- Eroded lands - 4.
- Shrubland. - 5.

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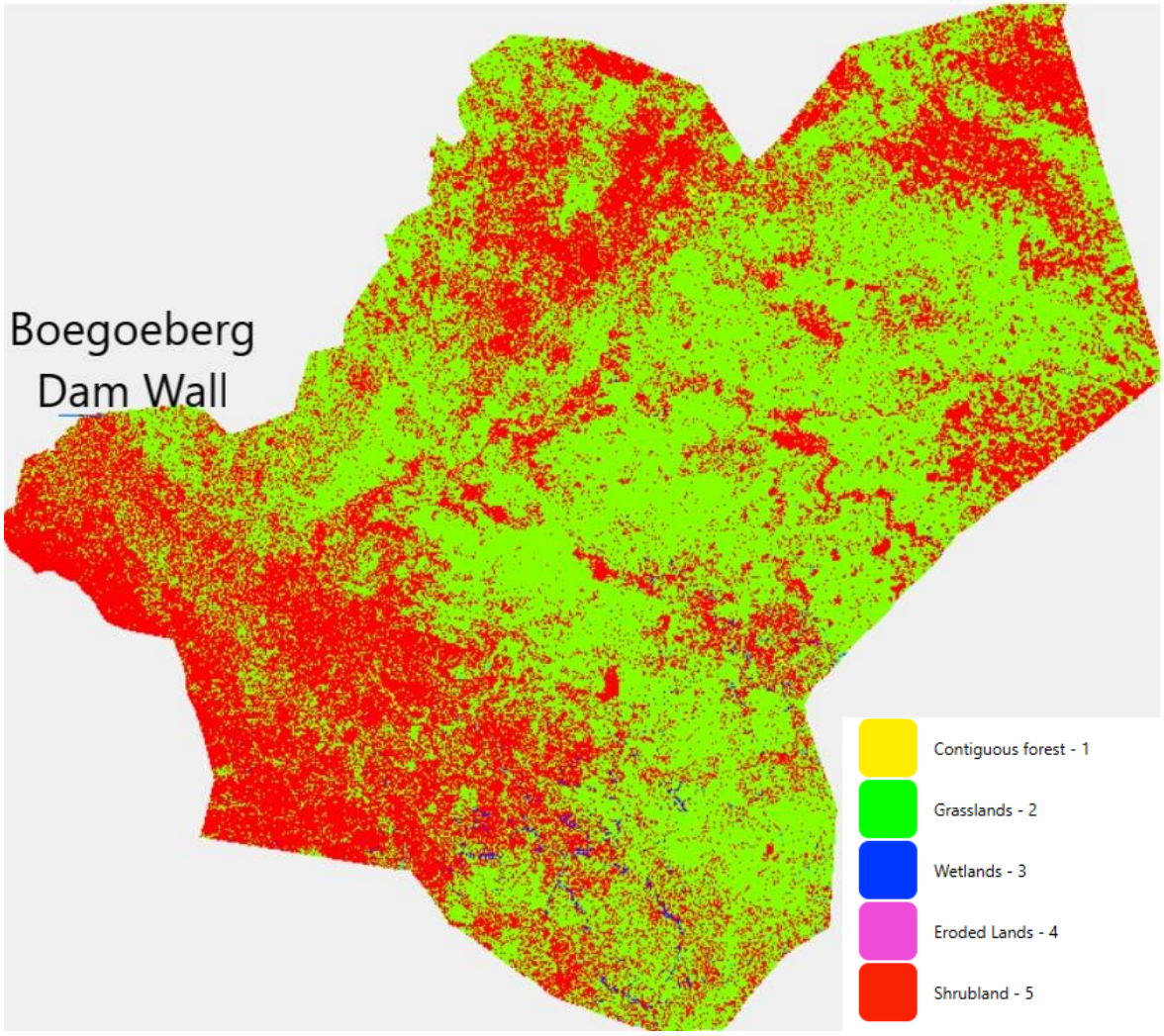

Figure 5: Reclassified data set

The land cover change detection analysis was conducted programmatically using Python. The primary libraries employed were Rasterio for geospatial raster data processing, NumPy for efficient numerical array computations, and Pandas for the generation of the quantitative transition matrix. A direct, pixel-by-pixel comparison was performed between the reclassified 2018 and 2022 raster. A prerequisite check confirmed that both datasets were spatially aligned, sharing the same coordinate reference system (CRS), resolution, and grid dimensions. The analysis produced three distinct outputs: an encoded change raster, a binary change mask, and a quantitative transition matrix.

- Encoded Change Raster: To map the specific nature of land cover transitions across the study area, an encoded change raster was generated. The value for each pixel in this raster was calculated using the formula:

$$C_{encoded} = (C_{2018} \times 10) + C_{2022}$$

Equation 1

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where C2018 and C2022 are the class values of the pixel in the 2018 and 2022 reclassified rasters, respectively. This encoding scheme assigns a unique integer value to every possible "from-to" change combination (e.g., a transition from Class 1 to Class 2 results in a pixel value of 12), allowing for detailed spatial analysis of specific land cover conversions.

- Binary Change Mask: To visualize the spatial extent of change, a binary change mask was created. In this raster, pixels were assigned a value of 1 if their class value differed between 2018 and 2022 ($C_{2018} \neq C_{2022}$), and a value of 0 if the class remained stable. This provides a clear and immediate representation of all locations where land cover change occurred, irrespective of the type of change.
- Transition Matrix: To quantify the overall changes, a transition matrix was computed via cross-tabulation. This matrix provides a statistical summary of the land cover dynamics, where the rows represent the five reclassified land cover classes in 2018 and the columns represent the same classes in 2022. Each cell (i,j) in the matrix contains the total count of pixels that transitioned from class i in 2018 to class j in 2022. The diagonal elements of the matrix represent the total number of pixels that remained unchanged for each class. The resultant change detection raster can be visualised in Figure 6 while Table 1 shows the change detection matrix.

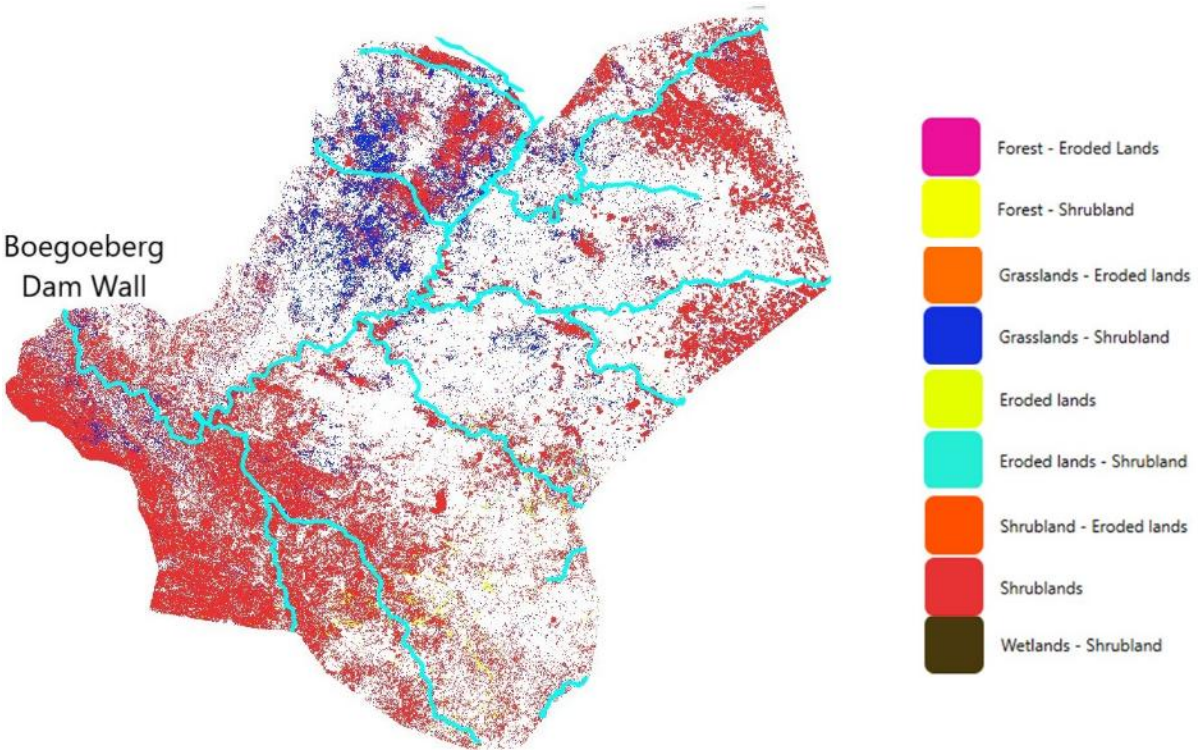

Figure 6: Change detection raster

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Table 1: Change detection matrix

		1	2	3	4	5
		Forests	**Grasslands**	**Wetlands**	**Eroded Lands**	**Shrubland**
1	**Forests**	591423	84173	1877	11	195962
2	**Grasslands**	266080	85976620	69267	9950	15755683
3	**Wetlands**	634	62950	496294	0	45324
4	**Eroded Lands**	637	492896	4	1034801	283921
5	**Shrubland**	598154	59215625	22667	119770	78601374

The key findings from the analysis indicate seven areas where land cover degraded to lower-quality classes, with grasslands being the most severe, potentially contributing to or indicating increased erosion in the catchment. Figure 7 highlights areas upstream of Boegoeberg Dam along the Orange River that have either remained shrubland (red) or have eroded further to eroded lands (yellow). These may be considered source zones.

Table 2: Key changes

From → To	x_Resolution	y_Resolution	Pixels
Forests - Shrubland	0.000179665	0.000179664	195 962
Forests - Grasslands	0.000179665	0.000179664	84 173
Grasslands - Eroded lands	0.000179665	0.000179664	9 950
Grasslands - shrubland	0.000179665	0.000179664	15 755 683
Wetlands - Shrubland	0.000179665	0.000179664	45 324
Eroded lands - Eroded lands	0.000179665	0.000179664	1 034 801
Shrubland - Shrubland	0.000179665	0.000179664	78 601 374

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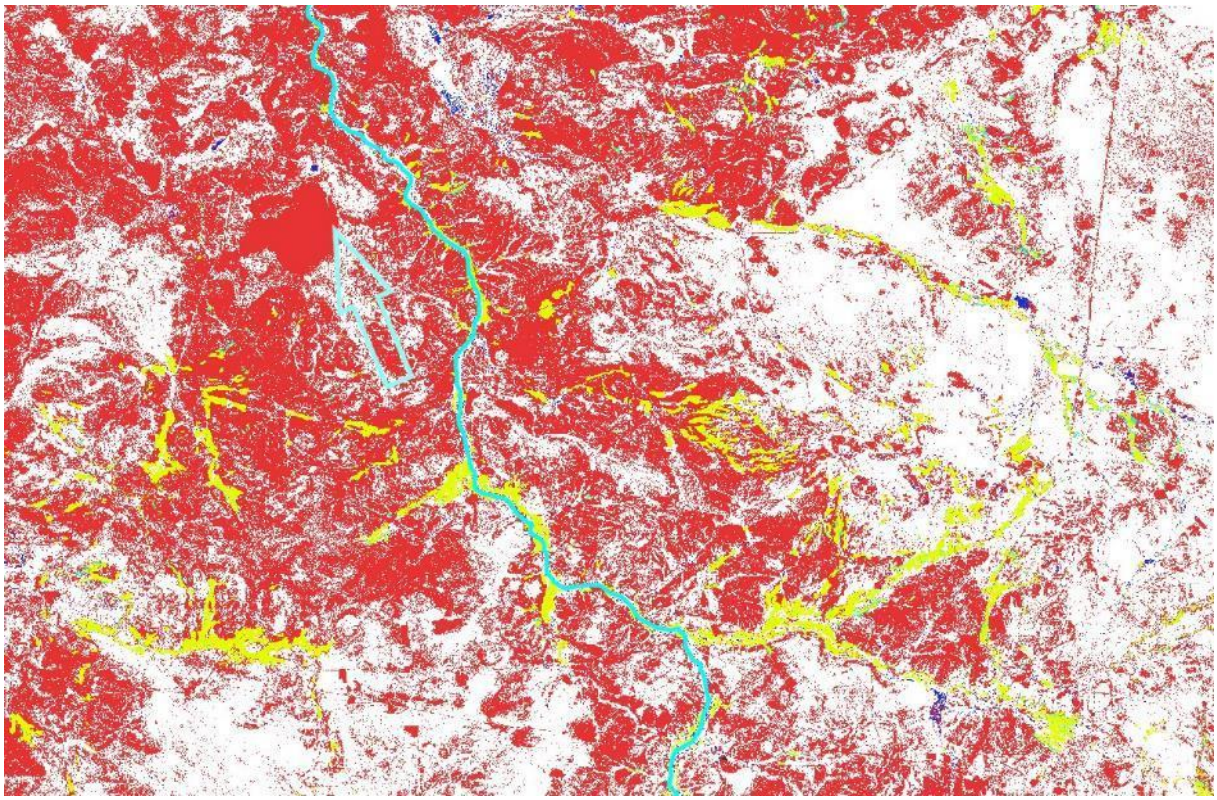

Figure 7: Areas near the river were eroded lands are predominant

The results of the change detection show that shrublands dominated the catchment and are increasing, while some of the source zones within the catchment are strongly associated with the conversion of grasslands and shrublands to eroded lands (increased yellow), primarily on mid to upper slopes and within the catchment and surrounding the river.

The relationship between slope and erosion is well-documented in soil science literature, most notably through the Universal Soil Loss Equation (USLE) and its modern successor, the Revised Universal Soil Loss Equation (RUSLE). These models use a topographic factor (LS-factor) to quantify the effect of slope on erosion. As the slope angle increases, the velocity of water flowing over the surface accelerates due to gravity. The erosive power of water is exponentially related to its velocity. Doubling the flow velocity can increase its erosive capacity by four times and its transport capacity by an even greater amount. This high-energy flow can easily dislodge soil particles and cut rills and gullies into the landscape. A longer slope allows runoff to accumulate, increasing in both volume and velocity as it moves downhill. This concentration of water results in a greater capacity to detach and carry away soil particles compared to a shorter slope of the same gradient.

As established by Wischmeier and Smith (1978) in their foundational work on the USLE, the combined LS-factor mathematically represents this process, showing that erosion risk escalates rapidly with even modest increases in slope steepness or length. On very gentle or nearly flat slopes (typically 0% to 2%), the risk of soil erosion by water is minimal. On such low gradients, water flows very slowly. This low velocity means the runoff has insufficient energy to detach and transport significant amounts of soil. The dominant hydrological process is often infiltration (water soaking into the ground) or, during heavy rainfall, ponding and deposition of sediment rather than erosion.

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As slopes become extremely steep, erosion rates become severe, and the dominant erosional process can shift from water erosion to mass wasting. On steep slopes (often considered >30%), runoff is rapid and highly destructive, when the angle of repose is exceeded. This is the steepest angle at which loose, unconsolidated material can remain stable, typically between 30° and 37° (equivalent to a 58% to 75% slope). When a slope exceeds this angle, mass wasting events like landslides, soil creep, and slumps become the primary mechanism of soil movement, often triggered by water saturation.

A high-level slope analysis was carried out using a Digital Elevation Model (DEM) created from the available 5 m contour data. The analysis focused on the terrain upstream of Boegoeberg Dam, specifically around the Orange River, as illustrated in Figure 7. This approach provided a representation of slope variations in the catchment, highlighting areas with gradients exceeding erosion thresholds and thereby indicating potential zones of accelerated runoff and sediment contribution to the dam.

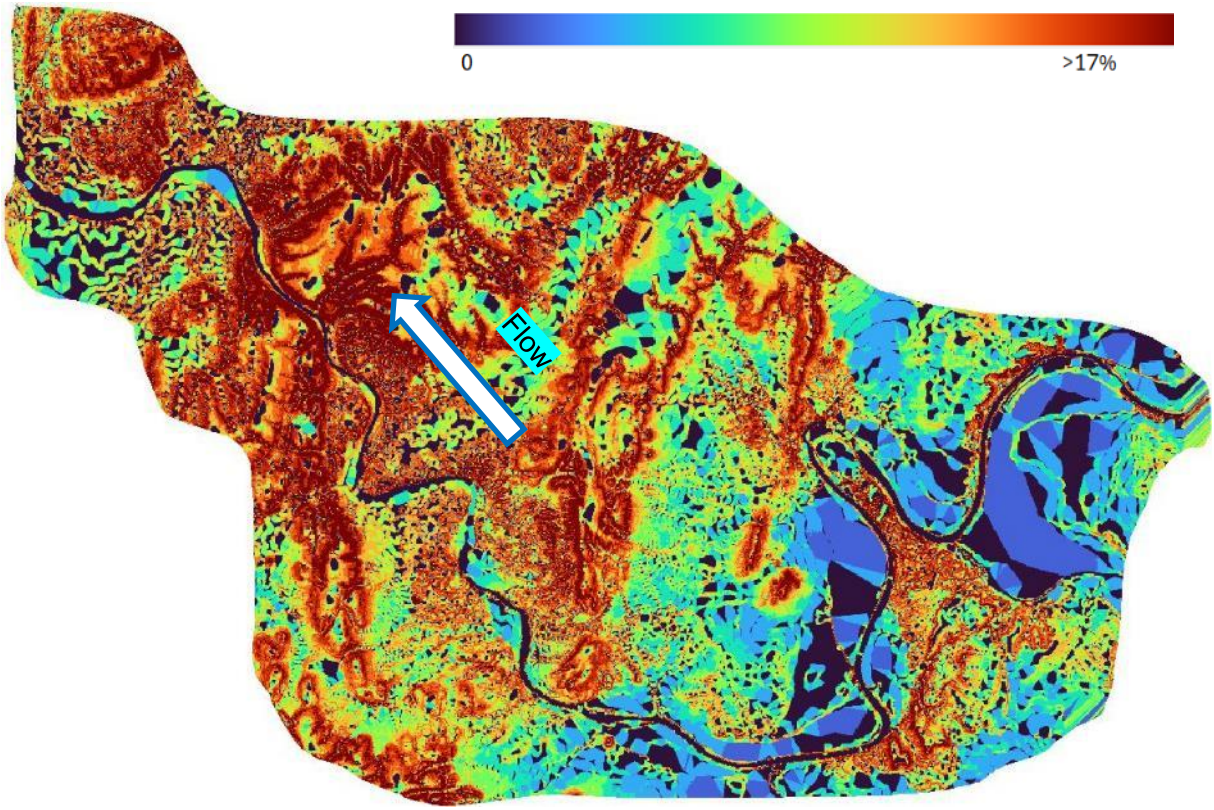

Figure 8: Slope raster

The slope analysis revealed that average slopes in the region range from 3% to 17%, excluding mountainous areas. When assessed against established erosion thresholds, these values indicate a high susceptibility to sediment generation in parts of the catchment. For eroded lands, where thresholds are as low as 5 to 8%, much of the terrain already exceeds the critical slope beyond which soil is highly prone to detachment and gully formation. Similarly, for shrubland, the threshold is around 12 to 15%, meaning that large portions of the catchment fall into the range where natural vegetation may no longer provide sufficient soil protection, particularly under intense rainfall. Even at the lower end, slopes of around 3% may contribute to sediment transport if coupled with degraded or bare soils. Thus, the results suggest that the catchment upstream of Boegoeberg Dam is dominated by slopes that exceed the critical erosion thresholds for the major land cover classes, highlighting the potential for sustained sediment delivery into the river system and ultimately into the dam.

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6. OVERLAND FLOW

This component of the study focused on simulating overland flow dynamics using the Rain-on-Grid approach in HEC-RAS. The analysis is considered high-level, as it was based on a Digital Elevation Model (DEM) derived from 5 m contour data, which provides a generalised rather than highly detailed representation of the terrain. The primary intention was not to generate exact flow hydrographs but rather to assess the typical velocities experienced on the land surface during rainfall events. These overland flows, originating from the broader catchment, eventually converge into the main Orange River upstream of Boegoeberg Dam, where they contribute to sediment mobilisation and transport. By identifying zones where velocities exceed known erosion thresholds, the model provides valuable insight into potential sediment source areas and the mechanisms driving siltation within the dam.

The catchment receives a Mean Annual Rainfall (MAR) of 543 mm. Historical rainfall data spanning the period 1920 to 2010 was compiled and processed to develop a representative hyetograph, as shown in Figure 9. This hyetograph was applied as the rainfall input for the Rain-on-Grid model, enabling the simulation of overland flow responses under conditions that reflect long-term climatic variability in the catchment.

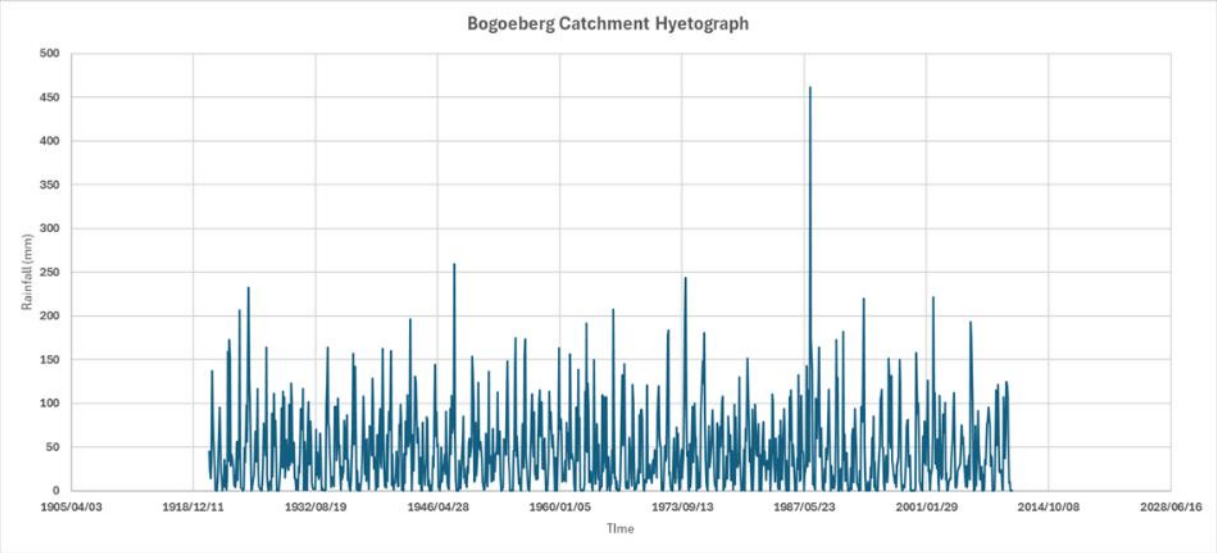

Figure 9: Rainfall Hyetograph

This simulation allows for the identification of primary drainage paths as they lead to the orange river. The model was ran such that the flows occur within a concentrated time frame as opposed to over the 90 years. Figure 10 and Figure 11 show the identified minor streams feeding into the main channel, and the corresponding maximum velocity map, the results of the velocities generated from the model are summarized in Table 3.

Table 3: Average Velocities

Location	Velocity (m/s)
Main Channel (Orange River)	3 - 15
Minor Streams	10 - 15
Overland	2 - 8

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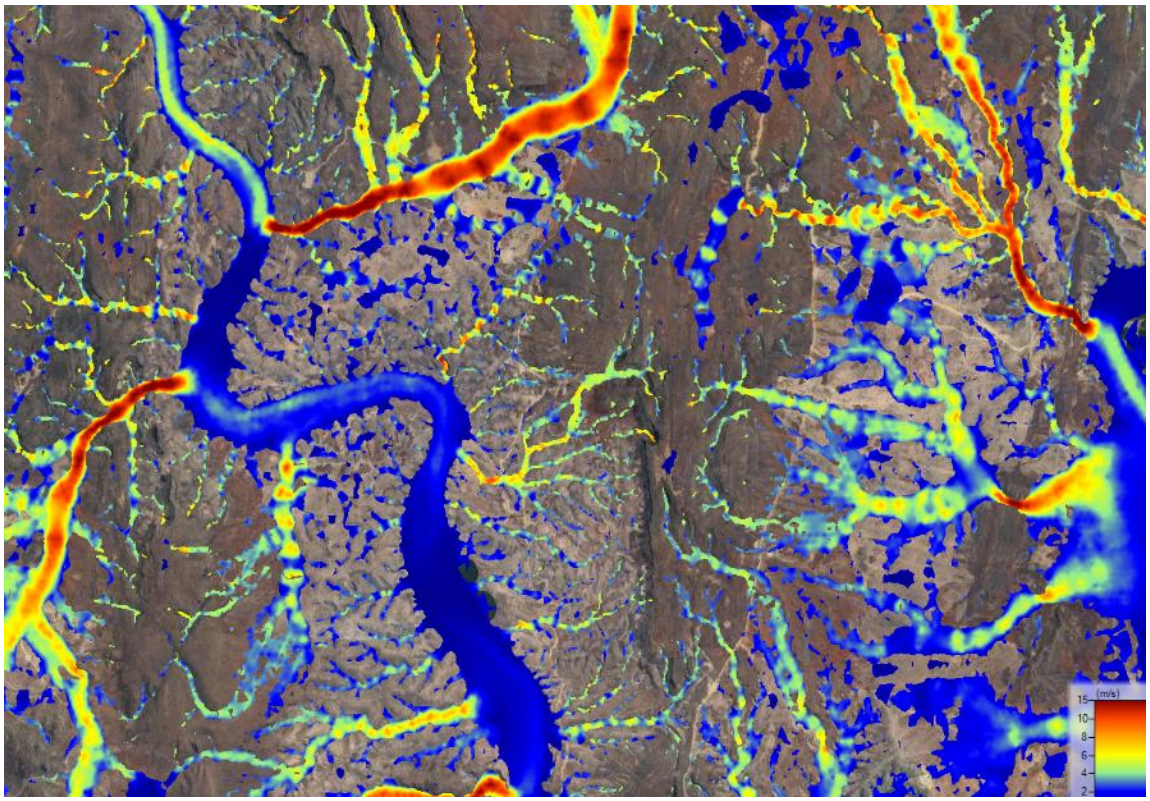

Figure 10: Flow paths that connect to the Orange River

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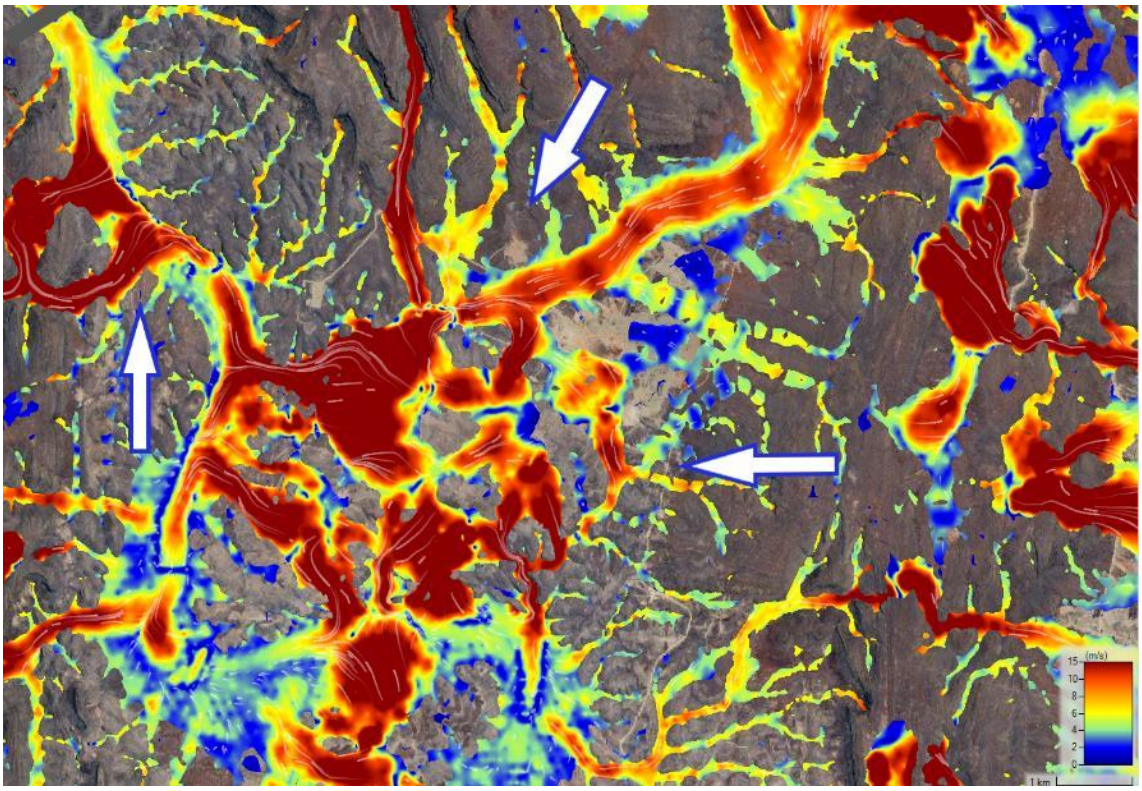

Figure 11: Maximum Velocity Map

Table 4: Erosive velocities according to land type

Land Type	Dominant Soil Texture	Bedload/Sediment Size (mm)	Erosion Index (RUSLE K-factor)	Min. Erosive Velocity (m/s)	Critical Shear Stress (Pa)
Grassland	Sandy Loam	0.01-0.25 (Silt-Fine Sand)	0.10-0.25	0.25-0.40	0.5-1.2
Shrubland	Loamy Sand/Clay Loam	0.05-0.50 (Fine-Medium Sand)	0.15-0.30	0.30-0.55	0.8-1.8
Erosive Lands	Sodic Clay/Silty Clay	<0.01 (Clay) to 0.10 (Silt)	0.30-0.45	0.15-0.30	0.2-0.6

Table 4 above was deduced from available literature including national soil databases (e.g., ISRIC SoilGrids, ARC-ISCW), academic studies, and erosion models (e.g., RUSLE, USLE). It defines the theoretical thresholds at which sediment transport is initiated, specifying a minimum erosive velocity ranging from as low as 0.15 m/s for highly erodible sodic clays to a maximum of 0.55 m/s for the more stable loamy sands found in shrubland areas. These values represent the point at which the shear stress exerted by flowing water overcomes the soil's cohesive and gravitational forces, setting particles in motion.

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When compared against the simulated flow velocities, the significance of the model's output becomes clear. The simulation indicates that even the slowest overland flows, which occur as shallow sheet flow across the landscape, reach velocities between 2 and 8 m/s. This is an order of magnitude greater than the highest required erosive velocity. This substantial difference between the required and actual velocities confirms that during a representative rainfall event, the hydraulic energy is more than sufficient to trigger widespread erosion across all major land types within the catchment. The "Erosive Lands," characterized by dispersive clays, are particularly vulnerable; with a critical velocity of just 0.15 m/s, they are readily mobilised by even the weakest simulated flows. Furthermore, the high-velocity flows occurring in minor streams (10-15 m/s) possess immense transport capacity, ensuring that any sediment detached from the surrounding landscape is efficiently conveyed into the main Orange River channel.

7. DISCUSSION

Targeting the source zones provides the most effective strategy for reducing sediment delivery into Boegoeberg Dam. The modelling results showed that areas of degraded land cover, steep slopes, and erodible soils upstream of the dam are experiencing overland flow velocities that exceed erosion thresholds. By implementing interventions directly in these source zones, both runoff and flow velocity can be reduced before water reaches the main Orange River.

Measures such as vegetation restoration (grass and shrub re-establishment), contour farming, riparian buffer strips, and gully rehabilitation help to slow down runoff, promote infiltration, and stabilise soils. In turn, this decreases the erosive force of overland flow, thereby reducing sediment detachment and transport. The principle is that addressing erosion where it originates, rather than only in transfer or deposition zones, ensures long-term effectiveness and sustainability. This approach aligns with the NAT-SILT programme, which emphasises catchment management and natural interventions as central to reducing sediment inflows into strategic water infrastructure. Studies have shown that even in dry arid conditions, certain plants such as Spekboom thickets can greatly reduce runoff and thus reduce sediment yield in an area. Some of the other recommended vegetation includes deep-rooted shrubs (increase concentration thereof), and succulents.

8. LESSONS LEARNT

Several technical lessons were learnt from this study, providing valuable insights for future sedimentation or siltation studies. First, the quality of land cover data proved to be a key factor. While the South African National Land Cover (SANLC) datasets are comprehensive, their resolution and classification accuracy limited fine-scale detection of land degradation. Furthermore, the frequency of national land cover updates, currently every few years, is insufficient for dynamic monitoring of erosion and land use pressures in sediment-sensitive catchments. More frequent or targeted updates, particularly in erosion-prone regions, would greatly enhance predictive modelling.

The quality of the DEM was another limitation. The analysis relied on a DEM created from 5 m contour data, which restricted the level of terrain detail captured. This impacted the accuracy of slope and flow velocity modelling, especially for small-scale gullies and micro-topographic features that strongly influence erosion. Future work would benefit from access to higher-resolution elevation data, such as LiDAR or photogrammetric DEMs, to improve terrain representation.

Additionally, while soils data from the SOTER database provided useful baseline information, it would be beneficial to conduct geotechnical investigations in the Boegoeberg catchment to quantify soil erodibility, infiltration rates, and sediment bed material characteristics. Such studies would strengthen the parameterisation of hydrological and sediment transport models, moving from generalised assumptions toward site-specific calibration. To improve this study, future modelling efforts should focus on integrating multi-temporal land cover datasets with slope and soils information to track and predict land use changes. Collaboration with the Department of Forestry, Fisheries and the Environment (DFFE)

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could ensure consistent data access and the development of early-warning systems for land degradation hotspots. Building capacity for more frequent land cover monitoring, combined with high-quality terrain and soils data, will significantly enhance the ability to design and implement effective sediment mitigation strategies for Boegoeberg Dam and other strategic reservoirs in South Africa.

9. ACKNOWLEDGEMENTS

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DYNAMIC CALIBRATION OF THE FINITE ELEMENT MODEL OF KATSE DAM

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PRESENTER: RYAN CASSELLS

ABSTRACT

The last Panel of Expert review of Katse Dam in 2018 (Dungar et al: 2019) concluded that the original design calculations although appropriate at the time, do not satisfy all the current international accepted criteria and procedures. This specifically pertains to the abutment seismic stability as well as the overall long-term behaviour of the dam wall during normal operation and extreme conditions. In addition, advancements in numerical modelling techniques since the 1990s—when the dam was designed—warranted a state-of-the-art structural evaluation. As a result, the LHDA initiated a project to perform a finite element analysis of the dam as well as evaluate the monitoring system and monitoring results and determine threshold values for the monitoring system.

A key component of the structural evaluation involved developing a finite element model of the dam and foundation for undertaking state-of-the-art simulations of the dam behaviour under a suite of loading conditions. The first step in the calibration process of the finite element model of the dam was to perform an eigenvalue back-analysis of the model for comparison with the in-situ measured ambient vibration measurements (AVM measurement). This was achieved by validating the model mass and stiffness parameters through the process of aligning the computed modal characteristics with those measured on site.

This paper will discuss the dynamic calibration approach as well as report on the challenges encountered, and lessons learnt during this dynamic calibration using the ambient vibration measurement results.

1. BACKGROUND

Katse Dam, located on the Malibamatšo River in Lesotho, forms part of Phase 1A of the Lesotho Highlands Water Project. The dam is a 185 m high, wide-valley, double-curvature arch dam with a developed crest length of 710 m giving it a crest length to height ratio of 3.8. The dam is founded in a U-shaped valley and has a relatively large surface area to height ratio (canyon shape factor). It consists of 47 blocks, with each block 15 m wide. The maximum thickness of the dam at the base is 60 m, and the thickness at the crest is 9 m. Construction of the dam started in January 1991 and was completed in April 1997. A comprehensive account of the various design considerations and characteristics of the dam can be found in the conference proceedings of the ICOLD Congress held in Durban, South Africa in 1994 (Tardieu, 1994).

According to Tardieu (1994), the foundation of the dam consists mainly of basaltic lavas of the Drakensberg formation. The lava flows have been deposited nearly sub-horizontally and vary in thickness – from 1 m to 45 m with a median thickness of 3 m. The dam abutments consists of more that

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60 flows. The main discontinuities are composed of the sub-horizontal flow contacts; a sub-horizontal shear zone two horizontal layers of auto-brecciated basalt and some vertical stress relief joints. The extent of these different components are given in Figure 2.

Furthermore, the dam contains a number of special design features which could have an influence on the dynamic behaviour of the dam. These include the following:

- Shear keys was installed underneath the dam on both flanks to bridge the auto-breccia rock layer downstream of the central foundation;
- A preformed joint was provided on the upstream side of the lowest part of the arch to force tension cracking in the concrete instead of the weaker foundation;
- A seismic belt (heavily reinforced hidden beams) was included at the central portion of the crest of the arch around Gallery A stretching across the uncontrolled spillway from Block 15 on the right flank to Block 16 on the left flank. The purpose of this belt is to tie the blocks to avoid out-of-phase movement between the blocks in the event of an earthquake and thus prevent damage at crest level.

2. RATIONAL OF DYNAMIC CALIBRATION

The structural evaluation of Katse Dam was undertaken considering the current conditions and state of the dam and foundation. To ensure the model characteristics and properties correctly represent the current conditions on site, the finite element model required validation of the assumptions on which it is based.

With this in mind, the dynamic characteristics of a dam, i.e., natural frequencies and mode shapes, are related to its structural properties, i.e., mass and stiffness. These parameters are system parameters, and they are directly related to the structural behaviour of the dam (stiffness, mass and boundary conditions). The mode shapes are largely dependent on the geometry of the dam, while the natural frequencies are largely dependent on the mass and stiffness of the dam.

By fine-tuning the theoretical finite element model to match the measured dynamic characteristics obtained from an in-situ ambient vibration measurement (AVM) (Moyo et al, 2025) as closely as possible, the resulting theoretical finite element model would be a good replica of the dynamic characteristics as-built dam. Extensive research has been conducted at the University of Cape Town into the dynamic behaviour of large concrete arch dams (based on the methodology used for the theoretical analysis of bridges) since 2007 (Moyo & Oosthuizen, 2013, Moyo et al, 2016, Moyo & Nyoni, 2020, Bukenya & Moyo, 2023, Nyoni & Moyo, 2023). This research highlighted the use of the dynamic calibration as an essential first step in the calibration of the theoretical model in the proper behavioural analysis of existing dams.

The calibration steps therefore adopted for this project at Katse Dam involved the following:

- Calibration of the dynamic properties using the results of the AVM testing (Moyo et al, 2025) and available material properties obtained during design and construction (this paper);
- Thermal calibration of the concrete structure (Cassells et al, 2024). Observed thermal behaviour was available from the static monitoring results; and
- Lastly, deformation calibration/validation using the results of the static monitoring (Cassells et al, 2025b).

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3. CALIBRATION METHODOLOGY

A numerical finite element model of the Katse Dam and foundation was created. The constitutive material parameters of the model were initially assumed in accordance with the material properties presented in the final design reports of Katse Dam and updated to be representative of the structure by way of this calibration study.

The dynamic calibration process entailed developing a theoretical structural finite element model of the dam-foundation-reservoir system, to ensure its dynamic properties match the measured dynamic properties (natural frequencies and mode shapes) of the system as closely as possible. This comprised of an iterative process whereby a sequence of modal analyses was performed, from which the eigenvalue mode shapes and associated frequencies were determined and compared to the actual observed results from AVM testing results (Moyo et al, 2025). Differences between the natural frequencies were observed, and the dam/foundation elastic modulus input parameters adjusted to bring the difference closer. The modal analysis was then repeated for this configuration, and the results re-evaluated. This process was continued in an iterative manner until the theoretical finite element model modal analysis results provided a close match to those measured.

It is important to note that this iterative methodology adopted considered the fundamental laws of structural dynamics, according to which a stiffer dam exudes higher natural frequencies, while a more flexible dam will have lower natural frequencies. The iterative analyses commenced with an assumed concrete modulus of elasticity of 40 GPa, as per original dam design reports. The natural frequencies using this initial concrete stiffness were significantly higher than the observed natural frequencies. The modulus of elasticity was systematically reduced. Five different scenarios were subsequently considered – 38 GPa, 36 GPa, 35 GPa, 34 GPa and 33 GPa. From the results it was evident that a modulus of elasticity of 34 GPa provided the closed results compared to the observed natural frequencies.

The mode shapes from the finite element model were also visually compared with mode shapes observed from the AVM measurement. This comparison highlighted some differences in the different mode shapes on the upper right abutment. As a consequence, the modulus of elasticity of the upper right abutment was increased from 35 GPa to 38 GPa to achieve better calibration.

Something else to consider is that the modulus of elasticity of concrete is strain rate dependent (Shkolnik, 2007) and usually increases under dynamic loading with high strain rates. It is however noted that the calibrated elastic modulus of the dam was derived from dynamic mode shape responses, under operational conditions, where strain rate and the level of strain development in the dam are very low. Under these low strain conditions, the dynamic and static elastic modulus of the dam are approximately equal. As a result, the calibrated model is suitable for static loading conditions.

4. FINITE ELEMENT MODEL

The as-built geometrical configuration of the dam site was determined by a state-of-the-art three-dimensional terrestrial and bathymetrical survey of the dam and foundation topographies during this project. The relative accuracy achieved by the terrestrial survey was 5 mm, whilst that of the bathymetrical (hydrographical) survey accuracy is 30 mm (Schalekamp et al, 2023). This was used to develop the finite element model.

4.1 Finite element mesh

The finite element model of the dam and foundation was created using the Midas FEA NX software (see Figure 1). The dam and foundation were modelled as a continuum mesh body discretised with a hybrid of hexahedral, tetrahedral, pyramid and wedge 3D solid elements. The model is hexahedral (8 Gaussian integration points) dominant with other elements only used in the vicinity of sharp edges and faces. The

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dam and foundation finite element model comprises 240 000 nodes and 460 000 elements. This complex model used a sound meshing strategy with very high resolution of elements within the dam body and the foundation zone in close proximity of dam body and low-resolution elements on the outer extents of the foundation.

4.2 Boundary Conditions

The outer extents of the foundation model were constrained against translation accordingly, with the bottom face constrained against translation in all three directions of the cartesian plane, X, Y and Z. The side faces of the foundation mesh were constrained against horizontal lateral translation.

4.3 Dam reservoir interaction

The dam water reservoir has a significant impact on the stiffness and mass of the dam-foundation-reservoir structural system. The dynamic characteristics of the dam, as indicated by the operational modal analysis results from the ambient vibration testing, implicitly account for the mass, stiffness and damping impact of the reservoir and foundation on the dam.

For purposes of dynamic calibration of the dam, the dam-reservoir interaction was modelled using the generalized Westergaard added mass approach. Although the Westergaard added mass approach is generally the accepted practice when undertaking seismic analyses of dams (ICOLD, 2013; ICOLD, 2018), it is known to be conservative. Considering the above and as one is trying to replicate the actual behaviour of the dam, the Westergaard added masses were reduced by applying a ratio of 0.25 to nodal added mass values (Moyo & Oosthuizen, 2013; Makha, 2012; Lemos et al, 2008)

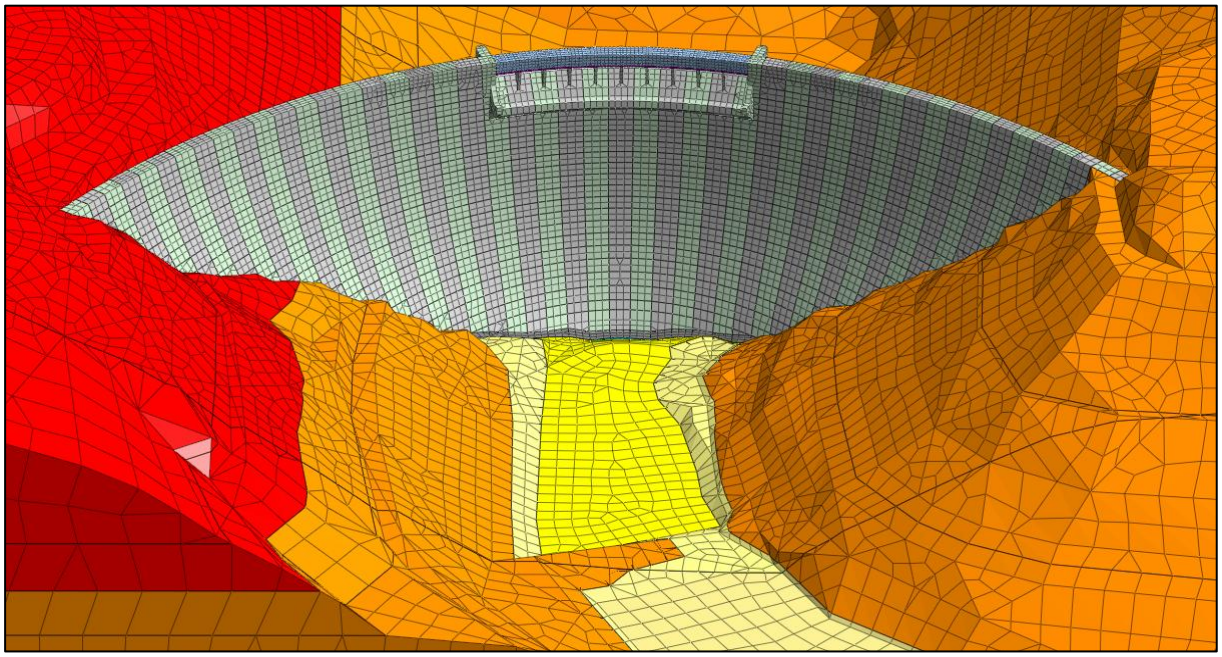

Figure 1. Finite element model of Katse Dam

4.4 Initial baseline material assumptions

The mass of the dam is directly related to its volume (geometry) and unit weight. The dam concrete unit weight can be measured within a reasonable level of accuracy, and this was done extensively during the construction of the dam. The unit weight is not expected to vary much over time, and the original measured unit weight will indicate current unit weight of the concrete.

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The modulus of elasticity of concrete is inherently time dependent and is the physical characteristic that is most likely to change over time since construction phase. The structural properties of concrete are dependent on sample size and testing conditions, and it has been shown that the properties obtained in the laboratory normally vary from the true properties on site. (ICOLD, 2009).

The material parameters for the dam concrete, as assumed at design stage, were obtained from laboratory testing of specimens cast and cured in the on-site laboratory under controlled conditions. The modulus of elasticity and Poisson ratio of the dam concrete were obtained from laboratory deformation testing of concrete specimens. Table 1 provides the initial baseline characteristics for the concrete.

No in-situ concrete tests have been carried out on the dam since the completion of construction to evaluate the actual modulus of elasticity of the concrete under existing site conditions. The dam concrete stiffness value is therefore the structural modelling variable considered to have the most significant variation from that obtained in the laboratory, having the largest impact on the reliability of the finite element model. The dam modulus of elasticity therefore was the first variable that was adjusted during this dynamic calibration process.

Table 1. Katse Dam baseline linear elastic concrete parameters

Concrete type	Density (kg/m3)	Modulus of elasticity (GPa)	Poisson ratio
Mass concrete	24.5	40	0.2
Reinforced concrete	25.0	45	0.2

Foundation material parameters of the Katse Dam site, such as elastic modulus and Poisson ratio, were derived from a combination of laboratory and in-situ geotechnical tests done before and during construction of the dam. The combination of laboratory and in-situ testing provided a level of confidence in the reliability of the results in reflecting actual foundation conditions by minimizing the variability caused by a difference between site and laboratory test conditions.

The results of which were used to determine baseline foundation material parameters (see Table 2). It is important to note that these values were significantly different from the values used during the design. The corresponding geological model incorporated in the finite element model is shown in Figure 2.

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Table 2. Katse Dam baseline foundation material parameters

Foundation layer description	Location	Density (kN/m^3)	Elasticity Modulus (GPa)	Poisson ratio
Lower Basalt layer below auto brecciated layer (A)	Below 1 866 mASL	0	40	0.21
Lower auto-brecciated layer (B)	Between 1 866 & 1 869 mASL	0	20	0.30
Basalt layer between auto-brecciated layers (C)	Between 1 869 & 1882 mASL	0	25	0.27
Upper auto-brecciated layer (D)	Between 1 882 & 1 885 mASL	0	10	0.35
Basaltic layer (D)	Above 1 885 and below flow contact at 1 892.5 mASL	0	30	0.25
Upper basaltic layer (E)	Above flow contact at 1 892.5 mASL	0	30	0.25
Upper right bank basaltic layer (F)	Above 1 975 mASL	0	35	0.23

5. CALIBRATION RESULTS

A summary of the natural frequencies for the first six modes of vibration of the different iterative analysis using different concrete moduli of elasticity are given in Table 3.

Table 3. Comparison of natural frequencies of AVM measurement and finite element analysis

Mode of vibration	Natural frequency (Hz)						
	Observed from AVM measurement	Finite element analysis					
		33 GPa	34 GPa	35 GPa	36 GPa	38 GPa	40 GPa
1	1.81	1.75	1.78	1.80	1.82	1.86	1.91
2	1.95	1.89	1.92	1.94	1.97	2.01	2.06
3	2.38	2.34	2.37	2.41	2.44	2.50	2.55
4	2.94	2.93	2.97	3.01	3.05	3.12	3.20
5	3.56	3.56	3.61	3.66	3.71	3.80	3.89
6	4.27	4.25	4.30	4.36	4.41	4.52	4.62
Mean error between FEA and AVM (%)		-1.4%	-0.1%	1.2%	2.5%	5.0%	7.4%

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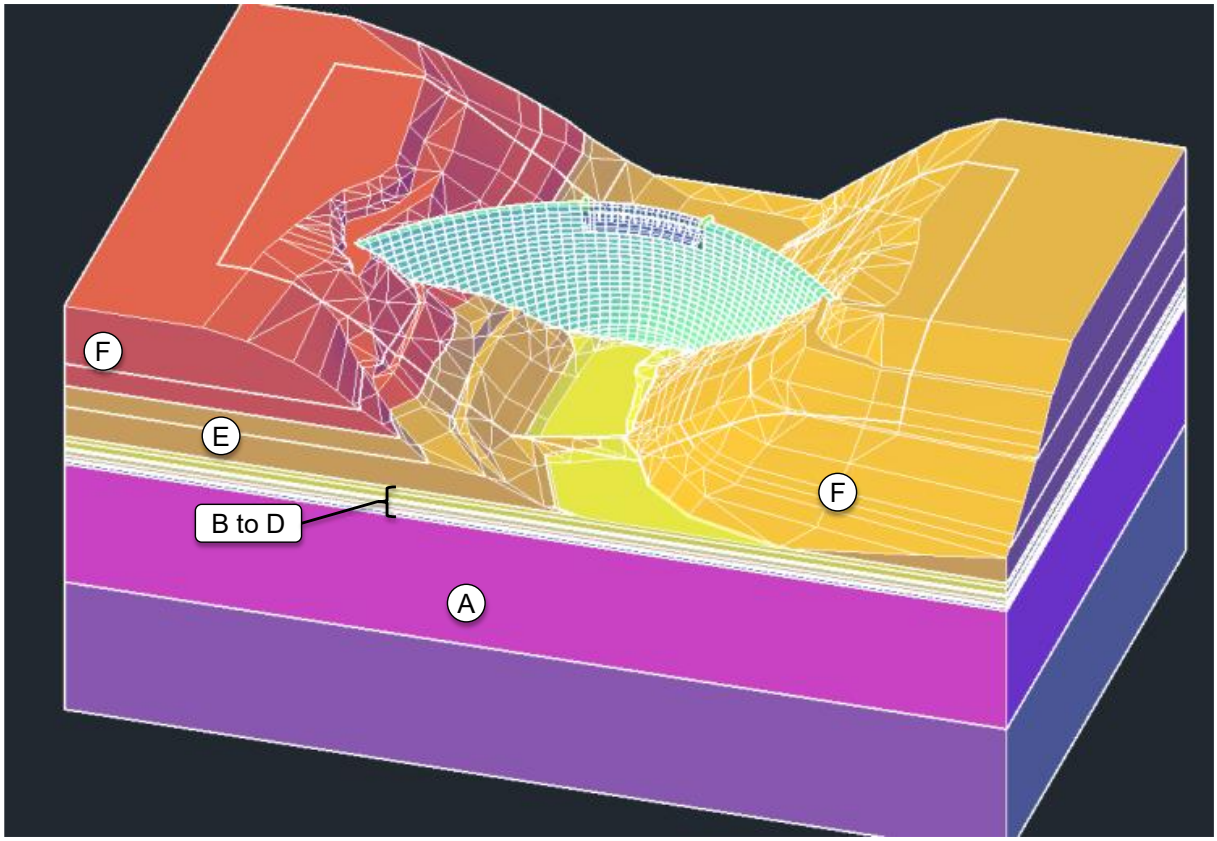

Figure 2. Katse Dam 3D finite element model with foundation detail

When comparing the difference between observed AVM natural frequencies and the different finite element model iterations (see Figure 3), it is evident that the mean error is the lowest with a concrete modulus of elasticity of 34 GPa. In other words, the best convergence between modelled and measured modes was achieved with a dam concrete elastic modulus of 34 GPa.

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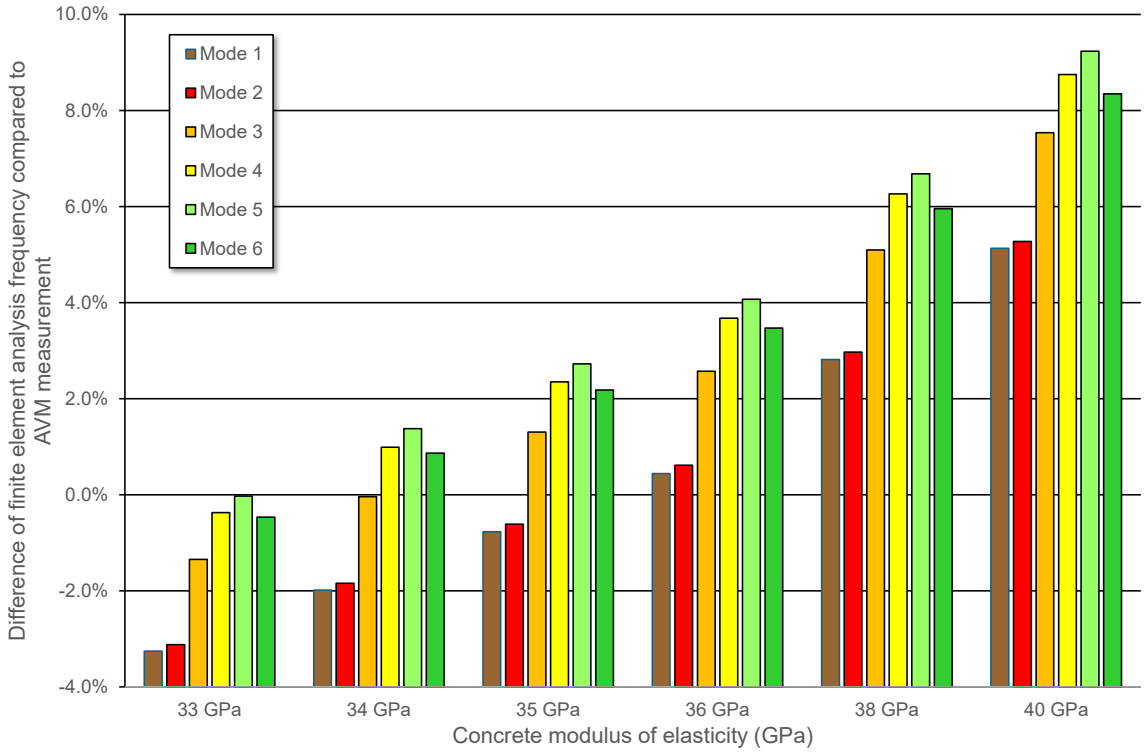
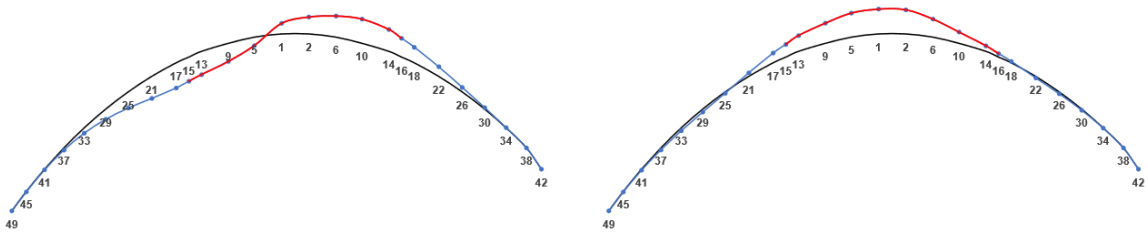


Figure 3. Comparison between AVM and finite element analysis natural frequencies

It is, however, interesting to note that for all the iterations, the frequencies of the first two structural are lower than the observed frequencies, while for modes 4 to 6 the frequencies are higher. The differences between modelled and measured natural frequencies for each of the first six modes are all within an absolute error margin of 5 %. The overall frequency scatter percentage method, commonly used to correlate between modelled and measured natural frequencies, provides a value of 2.4 % (Rad, 1997) which is considered low, indicating a very good data fit.

In general, it was found that the modal properties of the dam were not significantly sensitive to changes in foundation stiffness. This phenomenon was reported by other authors for cases where the ratio of the foundation elastic modulus to dam elastic modulus is greater than 0.67 (Nzuza, 2013; Proux & Paultre, 1997), which is the case for Katse Dam. As a final calibration iteration, the foundation elastic modulus of the upper right abutment was increased to a value of 38 GPa which is marginally higher than the original design value to achieve better calibration of the mode shapes.



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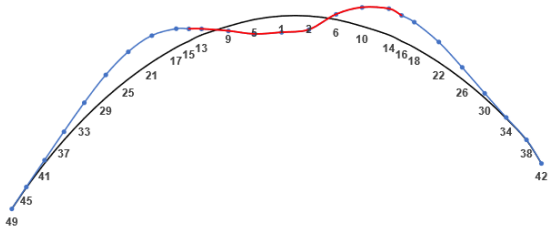
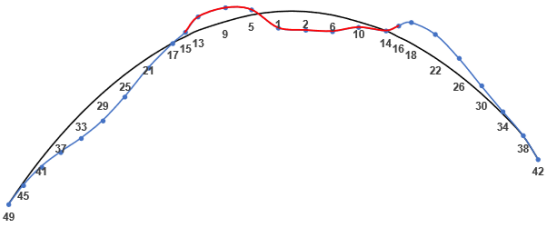
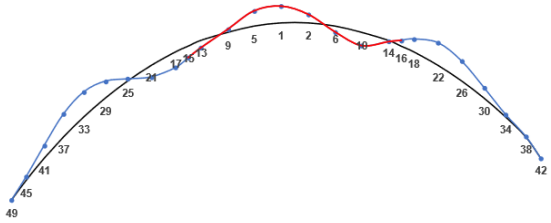
Mode 1: asymmetrical mode, 1.81Hz



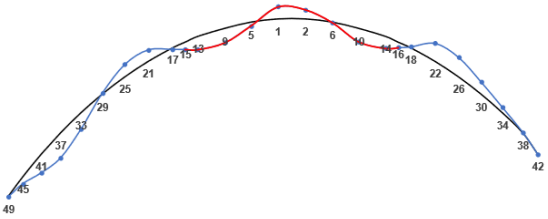
Mode 2: symmetrical mode 1.95Hz



Mode 3: symmetrical mode, 2.38Hz



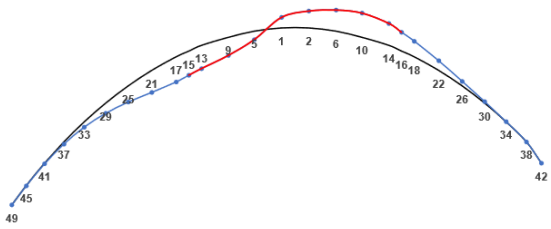
Mode 4: asymmetrical mode: 2.94Hz



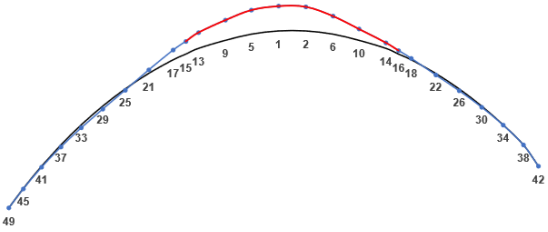
Mode 5: symmetrical mode: 3.56Hz

Mode 6: asymmetrical mode: 4.27Hz

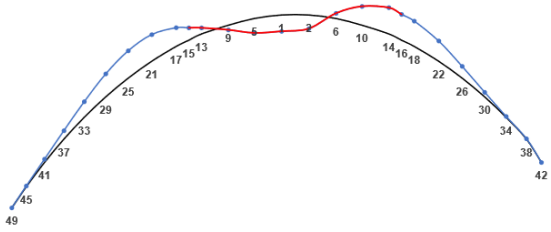
Figure 4 shows the first six observed natural frequencies and the corresponding mode shapes extracted from ambient vibration measurements taken in Gallery A (Moyo et al, 2025). Figure 5 shows the first six natural frequencies and the corresponding mode shapes from the calibrated finite element model using a concrete modulus of elasticity of 34 GPa. Not only is the frequencies within an acceptable range, but the theoretical mode shapes also closely follow the observed mode shapes.



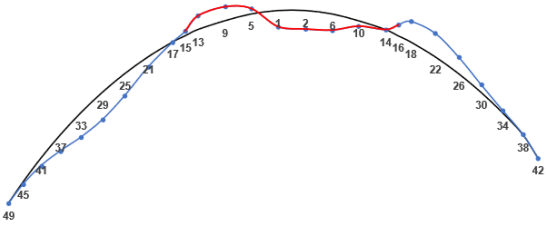
Mode 1: asymmetrical mode, 1.81Hz



Mode 2: symmetrical mode 1.95Hz



Mode 3: symmetrical mode, 2.38Hz



Mode 4: asymmetrical mode: 2.94Hz

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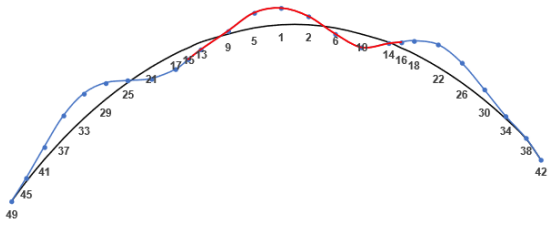
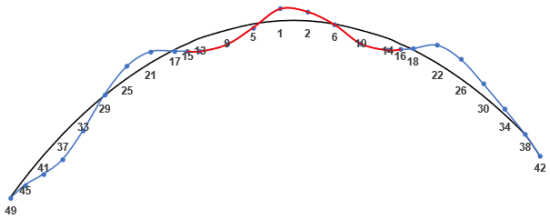


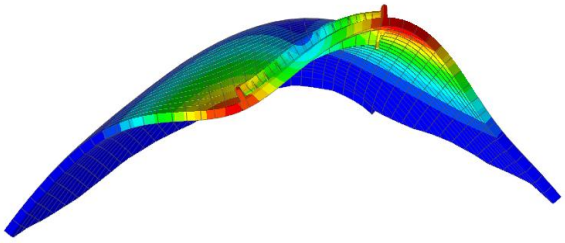
Mode 5: symmetrical mode: 3.56Hz



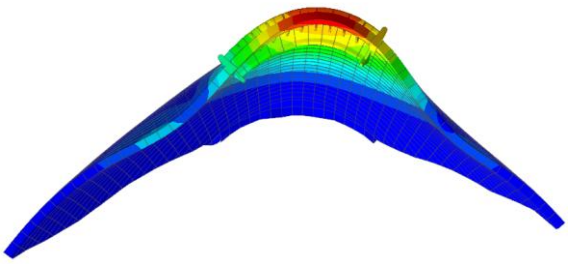
Mode 6: asymmetrical mode: 4.27Hz

Figure 4. Natural frequencies and mode shapes measured in Gallery A by AVM measurement (seismic belt is shown in red) (Moyo et al, 2025)

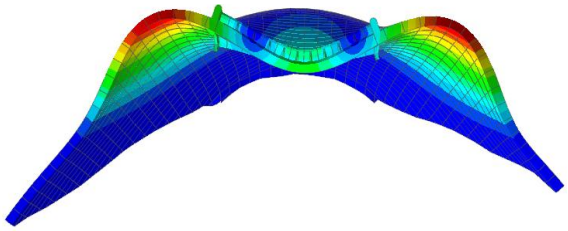
If one considers the percentage change in the natural frequency per GPa compared to the final calibrated setup using 34 GPa (see Figure 6), it is clearly evident the one can expect between 1.2% and 1.4% increase in the natural frequency for each 1 GPa increase in stiffness. The increases for modes 1 and 2 are on the lower range while modes 3 to 5 in general are on the higher range.



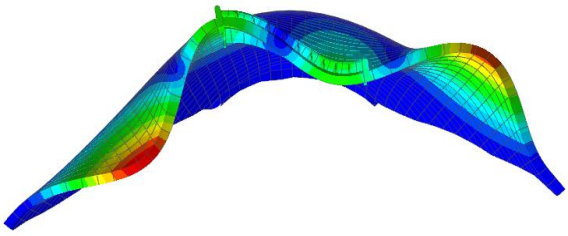
Mode 1: asymmetrical mode, 1.78Hz



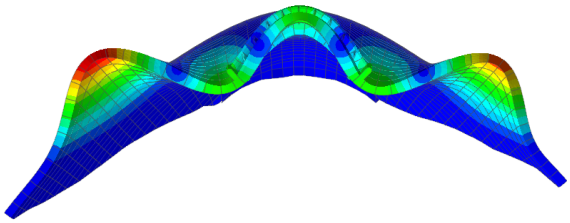
Mode 2: symmetrical mode 1.92Hz



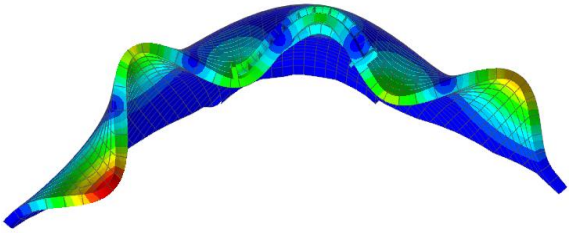
Mode 3: symmetrical mode, 2.37Hz



Mode 4: asymmetrical mode: 2.97Hz



Mode 5: symmetrical mode: 3.61Hz



Mode 6: asymmetrical mode: 4.30Hz

Figure 5. Natural frequencies and mode shapes from calibrated finite element analysis using E = 34 GPa

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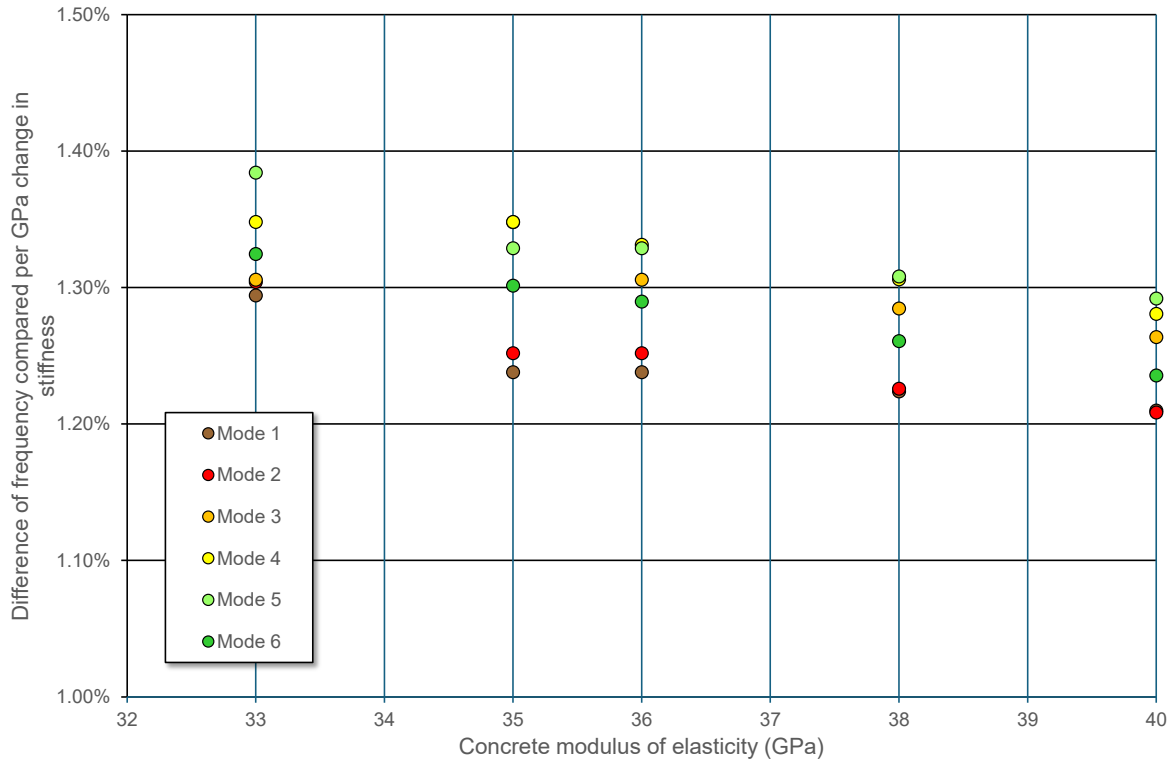

Figure 6. Frequency difference per GPa change in stiffness with 34 GPa as baseline

6. CONCLUDING REMARKS

From the dynamic calibration of the finite element model of Katse Dam, the following conclusions were possible:

- It was possible to develop a finite element model that closely mimics the measured dynamic properties of the dam and its foundation. The final calibrated modulus of elasticity for the concrete of 34 GPa was significantly lower than the design value of 40 GPa. This will have a significant effect on both the stress as well as deformation results from the finite element analysis. The final validated elastic modulus for the rock mass foundation material on the upper right abutment was validated as 38 GPa.
- For this particular case history, the frequencies of the first two structural are relatively lower than the observed frequencies from the AVM measurement, while for modes 4 to 6 the frequencies are relatively higher. Also, the natural frequency difference per unit stiffness change decreases with an increase in stiffness.
- Considering the time dependency of the strength as well as stiffness properties of concrete, it is important to note that this dynamic calibration is as a result is also time dependent. Consideration should be given to repeat this calibration in future by taken take into account the observed increase in natural frequencies.
- For similar future analysis, consideration should also be given to perform tests on the concrete of the dam to determine the physical characteristics, especially the modulus of elasticity. Should such testing be done careful consideration should be given the sample size and testing conditions.

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IMPACT OF SURFACE ROUGHNESS ON FLOW DYNAMICS OVER OGEE SPILLWAYS

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PRESENTER: JANNEKE GROENEWALD

ABSTRACT

Ogee spillways are widely used for their hydraulic efficiency and structural stability, yet long-term performance may decline as ageing and surface wear increase surface roughness. Such changes can alter flow behaviour, introducing risks of cavitation, air entrainment, and flow separation. This research investigates the influence of uniform roughness heights on the hydraulic performance of an ungated ogee spillway with a USBR crest profile. A combined approach was adopted using a laboratory-scale physical model and a numerical model developed in FLOW-3D HYDRO. Five discharges (ranging from 0.072 m^3/s to 0.503 m^3/s) and five roughness heights (ranging from 0 mm to 1.18 mm) were tested. Experimental repeatability tests demonstrated low error margins (velocity: 2% – 9%; pressure: ≤0.04%). The numerical model was calibrated with high-discharge pressure data and validated under intermediate conditions, with differences generally below 0.7%. Numerical results showed that increasing roughness reduced velocities by up to 17% and slightly increased pressures, particularly near the toe. Roughness generally increased cavitation indices, reducing cavitation risk, and consistently dampened flow separation risk, though both phenomena remained above the risk threshold limits. Air entrainment responses varied between models, highlighting the need for further research. Overall, while surface roughness introduced measurable but minor changes in hydraulic behaviour, it remains a valuable proxy for assessing long-term surface degradation, it remains important for assessing ageing spillway safety and may even offer secondary hydraulic benefits.

1. INTRODUCTION

Spillways are among the most critical safety components of dams. Despite well-developed design theory for new, smooth ogee surfaces, the behaviour of flows over aged and roughened ogee surfaces is less comprehensively understood and quantified. Ageing, environmental exposure, and operational wear gradually erode the surface, exposing coarse aggregate and increasing roughness. This degradation can alter velocity and pressure fields, and with it the potential for damaging hydraulic phenomena such as cavitation, air entrainment, and flow separation. Since these processes compromise structural integrity over time, investigating surface roughness provides a practical means of assessing how natural deterioration may influence spillway safety.

After years in operation, spillways exhibit a surface roughness that differs from the original design, which significantly impacts the spillway discharge coefficient and overall discharge capacity (Kamanbedast et al., 2014). Surface roughness amplifies wall pressure fluctuations and extends their influence beyond the boundary layer, indicating that pressure and velocity respond differently to roughness effects (Bhaganagar et al., 2007). Furthermore, roughness-induced cracks and irregularities can intensify cavitation damage through these amplified fluctuations, yet in some cases, strategic or uniform roughness placement can mitigate pressure drops and delay cavitation inception by promoting boundary turbulence (Nie, 2001, and Falvey, 1990). Similarly, roughened surfaces may enhance aeration, providing a protective cushion against cavitation damage (Chanson, 1992). These findings suggest that the hydraulic role of surface roughness is complex and context-dependent, with varying outcomes based

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on type, magnitude, and location. This highlights the need for further systematic analysis to clarify whether roughness primarily poses a deterioration hazard or, under certain conditions, offers hydraulic benefits. This research treats roughness as a proxy for surface degradation. A sensitivity analysis was conducted to examine the effects of varying roughness heights on flow behaviour. A FLOW-3D HYDRO Computational Fluid Dynamics (CFD) numerical model was calibrated and compared with a physical laboratory model.

2. LABORATORY MODEL

The physical model consisted of an ogee spillway with a height (model) of 0.7 m. The model was installed at the end of a 1.0 m wide, 1.2 m high, 40 m long hydraulic glass flume in the Hydraulics Laboratory at Stellenbosch University. A longitudinal section of the glass flume and the ogee model is shown in Figure 1. A prototype crest height of 14 m was assumed, resulting in a selected model scale of 1:20 as the model was designed according to the Froude similitude. This scale minimises scale effects while remaining compatible with the practical constraints of laboratory instrumentation.

This research evaluated low to moderate flow discharges testing five flow rates: 0.072 m^3/s, 0.105 m^3/s, 0.222 m^3/s, 0.447 m^3/s and 0.503 m^3/s. Surface roughness effects were translated using sand as a representative material. The sand was sifted through sieves to achieve consistent grading, resulting in selected grain sizes of 0.15 mm, 0.6 mm, 1.18 mm and 2 mm. A water level needle was used to measure the water levels at five upstream sections of the spillway within the glass flume, namely 0.5 m, 1 m, 4 m, 10 m and 20 m, to determine the stabilised water levels in the flume. The average flow depths at each location were compared with theoretically calculated values based on the ogee spillway head-discharge relationship. The average percentage differences between observed and theoretical water levels were -3%, -3%, -2%, 0%, and 0% with increasing discharge. The water velocity was measured using a Nixon Streamflo Flowmeter probe at four points along the centreline of the ogee spillway's downstream face (Figure 2). Air entrainment was measured using an intrusive conductive needle probe along the first three points where the velocity data were recorded. Ten Wika S-20 pressure transmitters were strategically positioned along the downstream end of the spillway (Figure 2), where Placement 1 is closest to the crest and Placement 4 is closest to the toe.

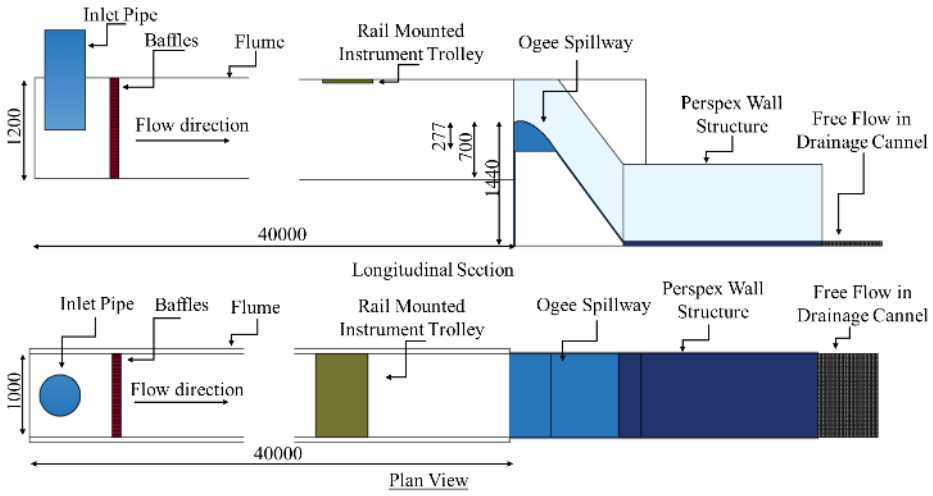

Figure 1. Longitudinal and plan view of the flume and ogee setup

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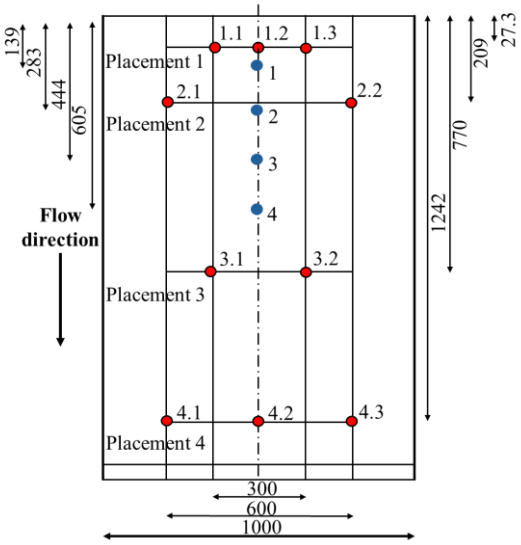

Figure 2. Spillway pressure (red) and velocity and air entrainment (blue) measurement points

This research employed linear regression to examine discharge and roughness effects on pressure, velocity, air concentration, cavitation index, and flow separation (using the Reynolds number) over an ogee spillway. This method identifies the relationship and significance of the parameters, providing global trends from upstream to downstream and localised trends at different points.

2.1 Experimental Flow Velocity Results

Velocity measurements were conducted under the three highest discharge scenarios (0.222 m³/s, 0.447 m³/s, and 0.503 m³/s) across five roughness conditions. The regression results indicate that the R^2 values range from 0.57 to 0.60 globally, indicating that approximately 57% to 60% of the variation in velocity is explained by the model at each point (blue dots on Figure 2). Discharge consistently had a positive and statistically significant effect at all points ($p > 0.05$). The discharge regression coefficient increased from 2.53 (Point 1) to 4.51 (Point 4). This trend suggests that flow velocity becomes increasingly sensitive to discharge further downstream. The slight dip at Point 2 (1.58) likely corresponds to local flow adjustments near the transition from the curved ogee crest to the straight tangential chute. Roughness exhibited a globally negative effect on flow velocity at all points, with roughness regression coefficients ranging from -0.16 at Point 1 to -0.57 at Point 4. This indicates that increasing roughness causes a decrease in velocity, especially further downstream, where roughness becomes statistically significant at Point 4 ($p = 0.03$) close to the tangential chute section. These results are in alignment with literature expectations, stating that surface texture influences velocity profiles and turbulence. Increased surface roughness causes frictional resistance, reducing velocities (Bhaganagar et al., 2007).

2.2 Experimental Air Entrainment Results

Air concentration measurements were conducted for three surface conditions (smooth, 0.15 mm, and 0.6 mm) at the two high discharges. The strongest regression performance was observed at Point 1 ($R^2 = 0.80$ and Adjusted $R^2 = 0.67$), indicating that the model explained a large amount of variability closest to the crest and that flow conditions are more sensitive to roughness and discharge changes. The discharge coefficient was consistently positive across all measurement points, indicating that higher discharges lead to increased air entrainment. However, this relationship was not statistically significant at any point ($p > 0.05$). Globally, the magnitude of the discharge coefficient peaked at Point 2 (47.62) compared to 29.76 and 17.86 at Points 1 and 3. This non-monotonic trend may reflect transitional flow behaviour from the crest to the tangential chute section. Increased turbulence and velocity at higher

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discharges increase air entrainment once aeration starts, consistent with the literature (Chanson, 1992). Unexpectedly, smooth surfaces showed higher air concentrations than roughened ones. Roughness consistently showed a negative influence on air entrainment at all points, meaning that increased surface roughness tends to reduce the amount of entrained air. Roughness effects diminished downstream, with the regression coefficients decreasing from -11.80 (Point 1) to -2.05 (Point 3). This may reflect the probe being positioned at a safe distance from the surface to prevent damage, which limited near-boundary sampling, localised crest effects, where small irregularities intensified turbulence, or scale and flow development limits, with aeration forming beyond the chute length. The anomaly highlights the need for further laboratory investigations with additional data points.

2.3 Experimental Pressure Results

Pressure was measured across all discharge-roughness conditions, with averages and 1% exceedance probabilities calculated (1988, as cited in Castillo et al., 2018). The unprocessed data show a favourable pressure gradient near the crest at low discharges, followed by localised adverse gradients downstream, which become more pronounced at high discharges. Regression analysis shows the strongest predictive performance near the crest (Points 1.1 to 1.3, $R^2 > 0.83$), weak performance in the mid-region (Points 2.1 to 2.2, Adjusted $R^2 < 0.12$ and $p > 0.05$), and regained strength and significance downstream (Points 4.1 to 4.3, Adjusted $R^2 > 0.88$). The discharge regression coefficients show a dual effect on pressure, where coefficients are negative near the crest and turn positive from the tangential section onward. The direction of the roughness effect is not consistent across measurement locations, with coefficients alternating between slightly positive and negative values; therefore, its influence is better described as localised and variable rather than uniformly increasing or decreasing pressure. Roughness has minimal influence upstream and in the mid-region but becomes statistically significant in the lower section (Points 4.1 and 4.3). Although the absolute magnitude of the roughness coefficients is small, the effect indicates that roughness contributes to localised pressure increases once the boundary layer has fully developed. Discharge is the dominant factor, while roughness produces measurable, localised increases downstream. This downstream influence aligns with the literature stating that pressure and velocity respond differently to roughness, while the upstream and mid-region insensitivity reflects the limited boundary layer development (Bhaganagar et al., 2007).

2.4 Experimental Cavitation Risk

Cavitation risk, using the cavitation index (σ), was qualitatively assessed using the recorded pressure and velocity data from the physical model. The cavitation index is highly sensitive to velocity, since it appears squared in the denominator. Pressure (numerator) only influences the index linearly, so when both rise, velocity typically dominates. A lower σ value indicates increased cavitation risk, with cavitation onset occurring at $\sigma \leq 0.2$. The cavitation index formula is described as (Falvey, 1990):

$$\sigma = \frac{P_0 - P_v}{\frac{\rho {u_0}^2}{2}}$$

where, P_0 is the local reference absolute pressure (Pa), P_v is the water vaporisation pressure (Pa), ρ is water density (1000 kg/m^3) and u_0 is the average cross-section velocity (m/s). Velocity regression results indicate a velocity-driven effect for the cavitation index, especially at higher discharges, with flow accelerating downstream and the largest discharge coefficient of 4.51 at Point 4, furthest downstream. Although pressure also increases downstream, its change is smaller and often not statistically significant, so the quadratic effect of velocity dominates. This leads to a general downstream decrease in cavitation index and an increased cavitation risk from crest to toe. Estimated region-averaged cavitation indices, calculated by pairing velocity and nearest pressure recordings, decreased downstream for all discharges: 43.8 to 27.9 for 0.222 m^3/s, 27.6 to 20.8 for 0.447 m^3/s, and 24.1 to 20.3 for 0.503 m^3/s, confirming increased downstream cavitation risk. Roughness produces competing effects on σ. It reduces local pressure (increasing cavitation risk) but also reduces velocity (reducing risk). Because σ depends on velocity squared, the net effect is location specific. Roughness may either

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mitigate or exacerbate cavitation depending on which change is larger locally. Increased roughness significantly reduces both pressure (notably at Points 4.1 and 4.3) and velocity downstream. Falvey (1990) recommends extremely smooth surfaces at high velocities to avoid cavitation risk and states that uniform roughness does not necessarily cause cavitation and may help prevent it by enhancing boundary turbulence.

2.5 Repeatability Tests

To ensure the reliability of the experimental results, repeatability tests were conducted for velocity and pressure measurements. Velocity tests were performed with 1.18 mm and 2 mm roughness configurations, comparing original and repeated results by calculating percentage differences at three discharge rates (0.222 m^3/s, 0.447 m^3/s, and 0.503 m^3/s). For 1.18 mm roughness, the maximum percentage repeatability errors were 6%, 5%, and 9%. For 2 mm roughness, they were 6%, 5%, and 2%. For pressure measurements, tests were repeated for the smooth surface and both roughness conditions across all discharges. The results demonstrated excellent repeatability, with maximum errors of 0.02%, 0.02%, and 0.04% for the smooth surface, 1.18 mm, and 2 mm roughness, respectively.

3. NUMERICAL MODEL

The impact of surface roughness on the flow field over an ogee spillway was investigated using both physical and numerical modelling. A scaled physical model was used to collect data with which to calibrate a CFD model. The numerical model allowed the individual flow processes to be examined in much greater detail than was possible in the physical model. The complex three-dimensional dynamics of the flow field over the ogee spillway were simulated in a CFD numerical model built using FLOW-3D HYDRO. The calibration process involved comparing the simulated pressure results with the laboratory model's data at the ten predefined points (Figure 2) along the spillway's downstream face using the 0.503 m^3/s discharge. The comparison focused on two statistical parameters: the average absolute pressure and the 1% exceedance pressure. Both statistical parameters were assessed with the 1% exceedance pressure serving as the primary factor during calibration. The velocity data from the numerical and physical models were compared to validate the pressure-calibrated numerical model. The post-calibration simulations for velocity, pressure, air entrainment, and cavitation and flow separation risk were all done along the ten measurement points in Figure 2.

3.1 Numerical Model Set-up and Calibration

The three-dimensional (3D) numerical model comprised three main components: the water entering the flume and reaching the upstream face of the spillway, the ogee spillway where the roughness was measured and the downstream river section. The design of the spillway was achieved by combining straight lines, arcs and a mathematical equation, calculated using Excel and imported as a spline curve to Inventor. A transient, two-phase, volume of fluid (VOF) simulation mimicking the physical model was compiled.

The numerical model was constructed using Cartesian meshes with rectangular grid elements. Geometries interact with and are embedded into this mesh using the Fractional Area Volume Obstacle Representation (FAVOR) method. The geometry is represented as a plane within each cell by calculating six area and volume fractions. These area and volume fractions are then included in the governing equations (Flow Science, 2023). The mesh (Figure 3) entailed multi-block meshing, a method that allows the domain to be divided into multiple mesh blocks, with rectangular prisms (non-conforming). This research used a nested block (finer mesh) over the spillway region inside a containing block (coarser mesh). A mesh refinement exercise was performed to ensure that the simulation is grid independent and capable of emulating the amplitude and frequency of the measured pressure and velocity fluctuations. Cell sizes ranged from 0.0095 m in the nested block to 0.018 m in the containing block, with 0.036 m used in the restart source simulation. The mesh consisted approximately of 3.3 million cells with run times in the order of 10 hours and 37 minutes for 10 seconds of simulation time.

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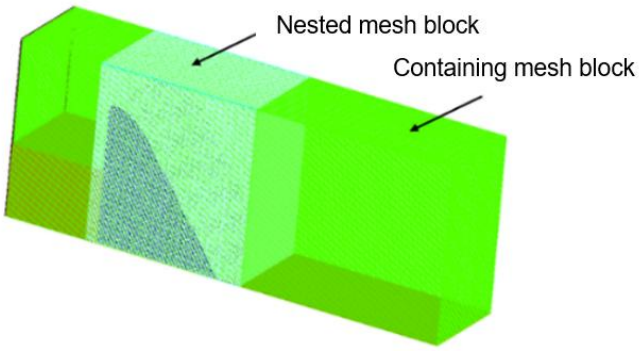

Figure 3. Final nested and containing mesh block

A boundary condition must be specified for each of the six faces of all mesh blocks. A separate restart source simulation was conducted to model the effects of the 40 m flume upstream of the spillway simulation (the restart simulation). The spillway simulation was initiated using the grid overlay boundary at the crest inlet, allowing the flow conditions from the flume simulation to transfer to the spillway model. This allowed the flume section to be excluded from the simulations, but included its effects and saved computational time. The selected boundaries are (Flow Science, 2023):

- Symmetry: Applies zero gradient boundary for both variables and zero velocity perpendicular to the boundary. Applied to the Y_{min}, Y_{max} and Z_{min} boundaries in the restart source and restart simulations (containing block) and all but Z_{max} for the nested block in the restart simulation.
- Volume flow rate: Specified a flow rate at the X_{min} boundary in the restart source simulation.
- Specified pressure: Sets the boundary pressure. The fluid fraction is set to zero, representing fluid exiting the domain. Applied to the X_{max} boundary in the restart source and restart simulations (containing block) and Z_{max} boundary of all blocks (treats top as a free surface).
- Grid overlay: Applies the restart source simulation solution (flume section) as the boundary condition for the restart simulation (spillway section). Applied to the restart simulation's containing block X_{min} boundary.

3.1.1 Turbulence Model and Maximum Turbulent Mixing Length Calibration

Different turbulence models were tested to determine which best replicated the experimental results. The models that were evaluated, based on the literature, are the Two-Equation $k-\varepsilon$ Model (RANS), Renormalised Group (RNG) Model and the Two-Equation $k-\omega$ Model (RANS). The $k-\omega$ turbulence model showed the best match with experimental results, achieving the lowest root mean squared error (RMSE) values for average (376 Pa) and 1% exceedance pressure (345 Pa). The differences between experimental and simulated results ranged from 0.02% to 0.65% for the 1% exceedance pressure. The most significant discrepancies were observed at Points 2.1 and 2.2, which are located near the transition from the ogee crest to the tangential downstream chute face.

The maximum turbulent mixing length is a safety limit in the $k-\omega$ turbulence model to prevent unrealistic turbulence levels. It can be dynamically computed, adapting across the domain, or set as a constant. Higher values reduce turbulence dissipation, exaggerating flow energy, while lower values increase dissipation, potentially underestimating turbulence. The recommended value for a constant computed mixing length is 7% of the flow depth over the spillway (Flow Science, 2023). Four mixing lengths were tested under the $k-\omega$ turbulence model: dynamic and constant values of 0.005 m, 0.0075 m, and 0.01 m. The dynamic model achieved the smallest RMSE of 345 Pa for 1% exceedance and the lowest percentage differences. While the 0.01 m case occasionally matched average pressures, its RMSE was higher, while the other cases underperformed. Dynamic was therefore selected for its consistent accuracy.

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3.1.2 Roughness Calibration

Roughness representation on the physical model was achieved using sieved sand with particle size ranges. Three values per range were tested for the CFD calibration simulations. Across all ranges, selected values aligned closely with measured pressures, with variations attributed to physical sand preparation and application methods:

- Smooth (0 mm): 0 mm produced the lowest RMSE (345 Pa) and the smallest percentage differences; selected as representative for smooth tests.
- 0.15 mm: 0.15 mm performed the best overall in its range (RMSE = 298 Pa); selected.
- 0.6 mm: 0.9 mm matched the experimental 0.6 mm roughness the best with RMSE (343 Pa), but 0.6 mm performed better across most points and matched the physical grain size; selected.
- 1.18 mm: 0.9 mm produced the best match (RMSE = 349 Pa), outperforming the nominal 1.18 mm; discrepancy likely due to sieving or application variations.
- 2 mm: 2 mm and 1.6 mm performed equally (RMSE = 377 Pa); 2 mm chosen for exact match with physical grain size.

3.1.3 Calibration Validation Tests

Five verification tests were conducted post-calibration to assess reliability. The 0.447 m^3/s discharge was paired with 0.6 mm, 1.18 mm, and 2 mm, while 0.105 m^3/s was paired with 0 mm and 0.15 mm. Differences were minimal, mostly under 0.7%, with slightly higher values at mid-region points (e.g., Points 2.1 and 2.2) for 0.447 m^3/s, peaking at $\approx 0.6\%$. The 0.105 m^3/s simulations showed variations consistently under 0.2%, and RMSE remained low (97 Pa and 99 Pa for 0.15 mm and smooth surfaces, respectively). Increased variation at higher discharges and roughness likely reflects heightened turbulence, causing local pressure fluctuations. Velocity verification tests at 0.222 m^3/s, 0.447 m^3/s, and 0.503 m^3/s across all roughness values showed larger average differences at mid-region points (Points 3: 15%, Point 4: 16%) than near the crest (Points 1: 8%, Point 2: 11%). Overall, the comparisons confirm that the numerical model reliably replicates measured pressure distributions and velocity fields across discharges and roughness conditions, providing confidence in its predictive accuracy.

3.2 Simulated Velocity Results

Roughness and discharge consistently show a moderate to strong effect on changes in flow velocity. The regression model fit is particularly strong in the upstream region with R^2 and Adjusted R^2 values of ≈ 0.98. The fit becomes less accurate at Points 2.1 and 2.2 ($R^2 = 0.06$) but improves again further downstream, with R^2 ranging from 0.46 to 0.84. Increasing discharge consistently and significantly increased velocity, with the magnitude of change becoming progressively larger downstream. This confirms that increasing discharge increases velocity across the spillway. Roughness has a small but statistically significant slowing effect. The negative roughness regression coefficients (e.g., -0.05 at Points 1.1 to 1.3) indicate that increasing roughness causes a decrease in flow velocity and is especially statistically significant from the mid-section downward. Notably, while Points 2.1 and 2.2 have a roughness coefficient of -0.023, the model fit here is poor ($R^2 = 0.06$) and the p-values, ranging from 0.804 to 0.807, suggest the effects in this region are not statistically reliable, possibly due to the transitional flow characteristics in the ogee crest to tangential chute transition. These results are broadly consistent with the physical model and literature, which showed that velocity increased with discharge and roughness introduced local flow reductions. Both approaches therefore indicate that discharge controls overall velocity patterns, while roughness exerts a weaker, site-specific slowing effect.

3.3 Simulated Air Entrainment Results

The simulations detected negligible values at the crest (Points 1.1 to 1.3). These points showed perfect linearity with $R^2 = 1$ due to the absence of detectable air entrainment. At the midway section (Points 2.1 and 2.2), where air entrainment originates, the R^2 (0.21) and Adjusted R^2 (0.14) values begin to increase. Both the discharge and roughness regression coefficients are close to zero in this midway

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region, with relatively high p-values (0.095 and 0.11), suggesting statistical insignificance. The regression models improve downstream (Points 4.1 to 4.3), with Adjusted $R^2 = 0.66$ and $p < 0.05$. Globally, from upstream to downstream, increasing discharge consistently reduces air entrainment. The regression discharge coefficients range from -0.0002 at Point 2.1 to approximately -0.8 at Point 4.3, with $p < 0.001$ at Points 3.1 to 4.3. It is particularly pronounced at Points 3.1 to 4.3, where the results are statistically significant (p<0.001). These results suggest that higher discharges limit air entrainment. However, the analysis becomes more complex. According to the literature, air entrainment starts near the crest at low discharges but is weaker; at higher discharges, it begins farther downstream, is more vigorous, and extends over a longer chute length, enhancing air-water mixing (Chanson, 1992). This phenomenon was confirmed by the numerical results. Increasing discharge shifted the inception point downstream, causing less overall air entrainment across the spillway. This means that increasing discharge did not necessarily, directly decrease the air entrainment, but merely shifted the inception point down, decreasing the area where entrained air was detected. The increased area provides more time for air development and overall greater entrainment. Roughness showed a consistently positive and statistically significant ($p < 0.05$) relationship with air entrainment. Roughness coefficients range from 0.014 at Point 2.1 to 0.062 at Point 4.3. This confirms that increased roughness enhances air entrainment, particularly downstream, again aligning with literature stating that increased turbulence increases air entrainment. This trend is observed across all discharges and roughness.

3.4 Simulated Pressure Results

The stacked bar chart (Figure 4) shows the sum of squared residuals (SSR) between observed and simulated pressures for different roughness heights, subdivided by discharge. The lowest SSR occurs at 2 mm, followed by 0 mm, while the highest appears at 0.6 mm. This indicates that roughness influences model accuracy, though no clear trend emerges. Residuals are dominated by the 0.503 m^3/s case, suggesting the model either struggles at higher discharges or roughness inaccuracies become more pronounced under such conditions.

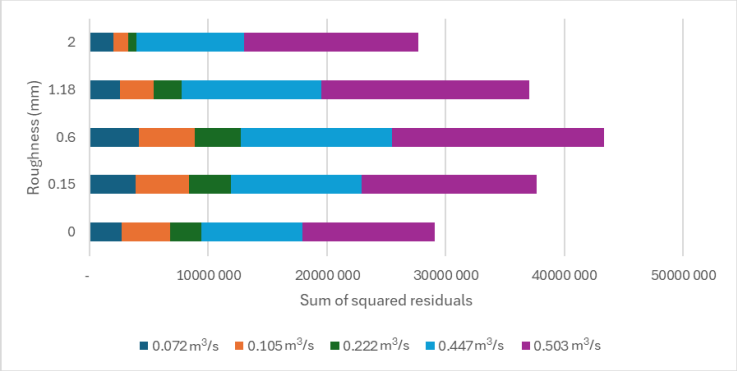

Figure 4. Sum of squared residuals between observed and simulated absolute pressures for varying roughness heights

The unprocessed numerical model data show the same general pattern compared to the physical model (favourable near the crest, adverse downstream), but with much weaker gradients. The crest region is consistently stable across all tests, while the chute is where the greatest variations occur. Pressures recover and converge across roughness levels at the toe, though higher discharges consistently maintain elevated absolute values. The regression models for natural logarithm (Ln)-transformed absolute pressure reveal a clear improvement in performance from upstream to downstream. Upstream (Points 1.1 to 1.3), the regression model explains a relatively small proportion of the simulated pressure variation, with $R^2 \approx 0.39$, while the downstream points show strong fits with values exceeding 0.95. Discharge significantly affects pressures at all points ($p < 0.001$), with negative regression coefficients upstream, indicating pressure reduction with increased discharge, and positive coefficients from Point

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3.1 onward, showing increasing pressure downstream. The magnitude of both negative (upstream) and positive (downstream) discharge regression coefficients grows along the spillway, indicating heightened sensitivity of pressure to discharge. Pressure changes are modest at lower discharges, but upstream drops and downstream recovery intensify at higher flows. Across the entire spillway, roughness slightly increases pressure. Roughness regression coefficients are small (1×10^{-4} to 5×10^{-5}) and statistically insignificant upstream, but become statistically significant downstream (Points 4.1 to 4.3). Pressure recovery toward the toe increases with discharge in both the laboratory and numerical model, and roughness alters pressure patterns locally (Bhaganagar et al., 2007). This suggests that roughness affects local pressure patterns but does not significantly alter global structural loads.

3.5 Simulated Flow Separation Results

Flow separation is associated with a low Reynolds number (Re) and occurs more readily in laminar flow conditions than in turbulent flow (Kundu et al., 2012). In open channel flow, laminar flow occurs at $Re < 500$ and turbulent flow at $Re > 1000$. The Reynolds number depends on the local flow velocity and the depth. An increase in either quantity tends to increase the Reynolds number, thereby reducing the flow separation risk. The formula for the Reynolds number is (Chadwick et al., 2013):

$$Re = \frac{DV}{\nu}$$

where D is the pipe diameter or hydraulic radius (m), V is the flow velocity (m/s) and ν is the kinematic viscosity (m^2/s). The Ln-transformed Reynolds number regression analysis showed that discharge has a statistically significant and positive effect across all measurement points. The discharge regression coefficients range from 4.42 to 5.96, with $p < 0.001$ throughout. The regression model averaged strong R^2 and Adjusted R^2 values of 0.9 across the entire spillway. Increasing discharge increases Re across all points, indicating a progressive improvement in flow separation risk mitigation. Roughness effects show a clear spatial dependency, although it is not statistically significant ($p > 0.1$). In the upstream regions, roughness negatively affects Re (regression coefficients from -0.01 to -0.03), suggesting a minor increase in flow separation risk. While roughness effects are minimal or slightly negative upstream, it sharply increases local Re values downstream, indicating reduced separation risk. As the flow progresses downstream (3.1 to 4.3), the roughness regression coefficients become increasingly positive, ranging from 0.05 to 0.18. This indicates that roughness enhances local turbulence and increases Re (decreasing flow separation risk) downstream. Roughness shows a progressively amplifying effect downstream, meaning that higher roughness at downstream locations generates larger incremental gains in Re than upstream. The downstream enhancement of Re, decreased separation risk, with roughness and discharge correspond to research showing separation in turbulent flow is delayed, and the higher-speed fluid remains closer to the surface than laminar flows (Kundu et al., 2012).

3.6 Simulated Cavitation Results

The cavitation regression model revealed the best fit upstream (Points 1.1 to 1.3, $R^2 \approx 0.92$) and midway (Points 3.1 and 3.2, $R^2 \approx 0.78$) and drops significantly downstream, especially at Point 4.3 ($R^2 \approx 0.17$). Figure 5 shows the variation of the cavitation index (σ) along the chute at different roughness conditions at 0.447 m^3/s. The cavitation index is relatively high upstream, indicating a low risk of cavitation, and peaks around Points 2.1 and 2.2 for all roughness values. Downstream (Points 3.1 to 4.3), the cavitation index drops sharply, showing the greatest cavitation susceptibility. The same trend was observed for all other discharges, with the overall cavitation index decreasing with increasing discharge. The regression results confirmed this, indicating increased cavitation risk with increasing discharge. The sensitivity is strongest and most significant upstream, where the discharge regression coefficients fall as low as -249 and reduce downstream to -10.9 midway (3.1 and 3.2) and -11.2 at the toe (Point 4.1). This shows that the cavitation index is less sensitive to discharge changes downstream.

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Figure 5 shows that increased roughness generally raises the cavitation index at most points, which aligns with the regression analysis. Roughness regression coefficients are especially statistically significant and positive at the midsection, indicating a protective effect. Here, at Points 3.1 and 3.2, the regression coefficients are +1 and +1.1, and $p = 0.005$ and 0.002, respectively. Roughness regression coefficients turn negative at Points 4.2 to 4.3 (-0.07 and -1.77), with statistically insignificant p-values, suggesting that roughness may offer no benefit or even pose a slight risk in these regions. These numerical results are generally in alignment with the physical model observations. Both show minimal cavitation risk upstream and in the midsection, with slightly increased susceptibility downstream, particularly under low roughness conditions.

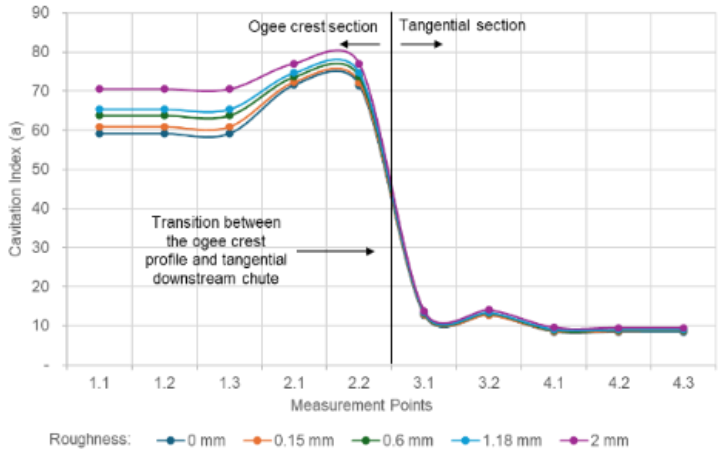

Figure 5. Discharge grouped simulated cavitation index results (0.447 m^3/s)

4. CONCLUSION

Both numerical and laboratory models showed that increasing roughness consistently reduced velocity, particularly downstream. Regression analysis confirmed roughness as a significant factor ($p < 0.05$), though discharge remained dominant. The numerical model showed mean velocity increasing from 1.44 m/s at the spillway crest to 4.34 m/s at the toe, while the four laboratory point measurements increased from 2.61 m/s to 3.69 m/s. Roughness decreased velocities by 3% to 7%, reducing local stresses that contribute to erosion and potentially extending structural life.

Both models show pressure decreases upstream and recovers near the toe with increasing roughness. Numerical changes were statistically significant but small in absolute terms. Laboratory averages indicated crest-to-toe increases of ≈1.1% rising to 2.7% at the highest discharge. These trends align with Bhaganagar et al. (2007), who linked roughness to increased wall pressure fluctuations due to form drag and local turbulence. Results suggest roughness affects local gradients but not global loads across the spillway from upstream to downstream, indicating that monitoring is more appropriate than a full redesign.

Air entrainment trends differed between models. The numerical model showed negligible entrainment at the crest and roughness enhanced it downstream (Points 3.1 to 4.3). Entrainment only occurred further downstream as discharge increased. This is consistent with the literature, stating that entrainment occurs after the inception point and moves downstream with increased discharge (Chanson, 1992). Interestingly, laboratory tests showed roughness dampened upstream entrainment, but entrainment increased with discharge, with the most pronounced entrainment near the crest. These differences suggest that laboratory measurements may be influenced by flow development or probe placement, highlighting the need to interpret results carefully when comparing the two models.

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Numerical results show σ values dropping from ± 110 upstream to ± 11 downstream (90% decrease) and the regression indicates that discharge is the primary driver of cavitation risk, particularly near the crest, with the cavitation index (σ) decreasing as flow increases. Laboratory measurements, though not computing σ directly, show a consistent directional trend: higher discharge elevates cavitation susceptibility downstream. Surface roughness effects are spatially variable. Roughness generally increases cavitation risk, though a local improvement in σ appeared in the mid-chute region, suggesting uniform roughness may occasionally mitigate cavitation risk (Falvey, 1990). While σ never fell below the typical threshold of $\sigma \approx 0.2$ (Li, 2021), the trend highlights the cavitation risk trend.

The Re values rose from approximately $138\,000$ (crest) to about $256\,000$ (toe), representing an 84% reduction in separation risk. The higher-speed fluid remains closer to the surface in turbulent flows than in laminar flows (Kundu et al., 2012), which helps reduce separation risk as the flow accelerates. Roughness had a dual effect on the Reynolds number. In upstream regions, it reduced Re and increased separation risk, while downstream it promoted turbulence and flow attachment. Discharge remained the primary stabilising factor, with roughness providing secondary, spatially dependent benefits.

5. RECOMMENDATIONS

Discharge is the dominant driver of hydraulic behaviour (significant regression coefficients, $p \approx 0$), while roughness effects, though smaller, remain statistically significant in certain regions. The results reveal that roughness increases risks near the crest but can reduce it downstream. Laboratory and numerical findings consistently show that discharge accelerates flow and elevates cavitation and turbulence risk, whereas roughness modifies these effects locally. Roughness should therefore be evaluated as a location-specific maintenance prioritisation rather than assumed to be universally detrimental. However, based on the findings of this research, it is recommended that discharge resilience should be prioritised, since discharge-driven velocity increases are far greater than the influence of roughness. It is recommended to maintain smooth crests, as cavitation and separation risks are most sensitive in this region. However, increased downstream roughness caused a localised decrease in cavitation risk, increased turbulence to mitigate flow separation, and enhanced air entrainment in the numerical model, which can further act as a cushion to mitigate local cavitation damage. Therefore, roughness may be considered downstream, where uniform roughness can promote turbulence and energy dissipation, but irregular patches should be avoided (Falvey, 1990).

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