THE LEGACY AND POTENTIAL OF JUNCTION REEFS DAM

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Junction reefs dam was designed in 1895 and constructed by 1897 as a multiple arch brick structure which was the first of its kind in Australia, and one of the earliest in the world. The dam was envisioned to provide mechanical and electrical power for gold mining. This paper provides an historical overview of the unique structure, and reassesses some of its engineering characteristics, such as the stress conditions in its unusual arches and reverse concrete gravity wing walls. The hydrology of the dam is re-assessed from the viewpoint of evaluating its potential as a mini hydro scheme. Commentary is also provided on the performance of its unlined spillway, which has been subject to regular spills for 120 years.

Keywords: *multiple arch dam, masonry dam, heritage dam, hydropower, unlined spillway*

INTRODUCTION

Junction reefs dam is located on the Belubula River about 7km North-West of Lyndhurst, NSW (33° 37.15'S, 148° 59.80'E). The dam structure holds international recognition as one of the earliest multi-arch dams, and the earliest of its kind in Australia (Chanson and James, 1998). The structure still stands today, 120 years since its construction in 1897. The historical development of this dam is described briefly below, and some the unique structural features of this dam are discussed. A hydrological study of the dam is also presented, and is used as a basis to assess the original and current hydropower potential of the site. Hydrological studies highlight the regularity of spillway operation and structure overtopping since its construction. The factors contributing to the demonstrated durability against scour are discussed.

THE HISTORY

The proposal for Junction Reefs dam was formed in the late 19th Century when the English company Lyndhurst Goldfields was establishing gold mining operations at Junction Reefs. The dam was envisaged to provide hydropower for the operations, and was specified to continuously provide 200 horse power (150 kW) (Schulze, 1897). Carl Oscar Schulze (1848-1919), or Oscar Schulze as he preferred to be known, was engaged as the designer for the dam. Schulze was a German born engineer who arrived in Sydney in 1879 as an agent for a printing press manufacturer. Historical records indicate that in the 1880's Schulze was involved in the development of aerial cableways for mining operations in the NSW Blue Mountains, and was later employed by the Union Bridge Company of Pennsylvania as the resident engineer for the Hawkesbury Railway Bridge (Mackie and Pells, 2011). Documentation of the design of Junction Reefs dam presented in Schulze (1897) demonstrated a wide range of dam engineering principles. Traditional earth / puddle core embankments were ruled out by Schulze on account of potential shrinking and cracking of the clay in the hot climate under variable streamflows, which was perceived to lead to piping failure (Schulze, 1897). Only concrete and masonry options were shortlisted, with masonry construction being selected for offering rapid construction. Under the supervision of Schulze, the main parts of the structure were completed within 9 months, and included 5500 cubic metres of concrete, and half a million bricks, all made on site. A photograph of the dam, taken shortly after construction, is shown in Figure 1.

A steel pipeline delivered water from the reservoir about 1.4 km downstream of the dam. Penstocks connected to four Pelton turbines. The three largest Pelton turbines were 6 foot, 4 foot and 3 foot in diameter and drove the following equipment:

- thirty five stamp batteries,
- mills,
- air compressor for rock drills,
- vanners (type of shaking table), and cyanide mixing equipment.

The fourth small Pelton turbine was used to generate electricity in a direct-drive configuration, as at the time no suitable governor had been invented. However, this was not successful - as electric lights and other equipment were turned off the Pelton wheel went faster and faster, the generated current went up, and the remaining lights were blown! A photograph of remnants of one of the original Pelton wheels as shown in Figure 2.

Figure 1 – After construction in 1897

Figure2 – Remnants of one of the original turbines (Steven Pells photo)

THE STRUCTURE

Original long section and cross sectional drawings of the dam are reproduced in Figure 3. The original plan drawing of the dam is shown overlain on an aerial photograph (October 2014) in Figure 4. The current extent of siltation is evident. The central section of the dam features 5 masonry elliptical arches which support a concrete face. Concrete gravity wing walls are found either side of the arched section, giving a total crest length of 131 m. The maximum height is reported in Schultze (1897) as 18.3m.

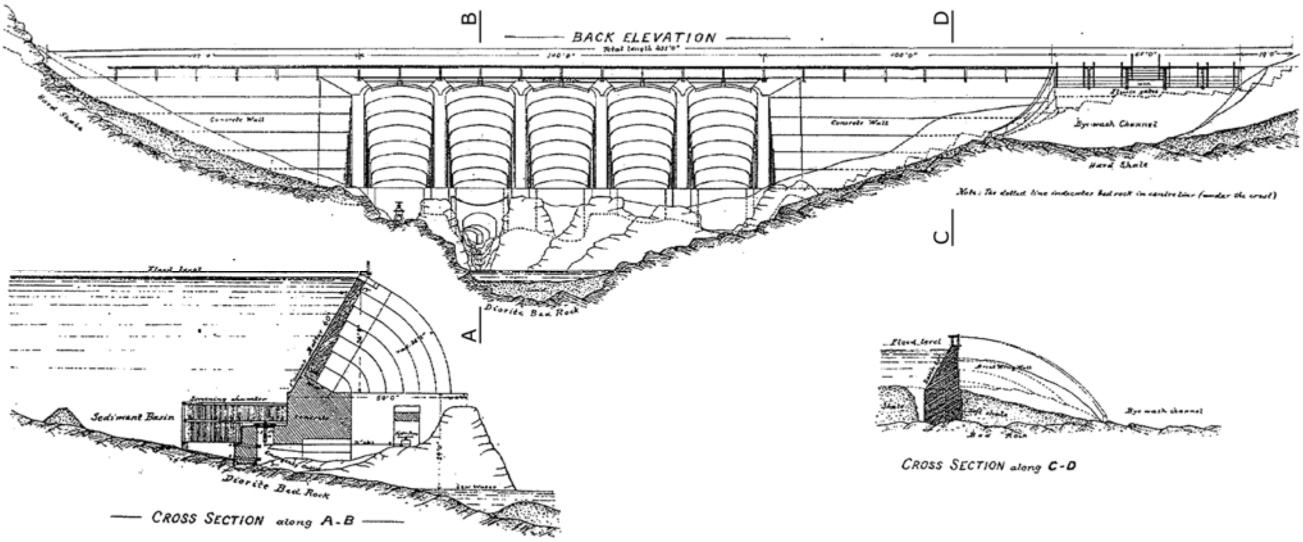

Figure 3 – Elevation and sections through the dam

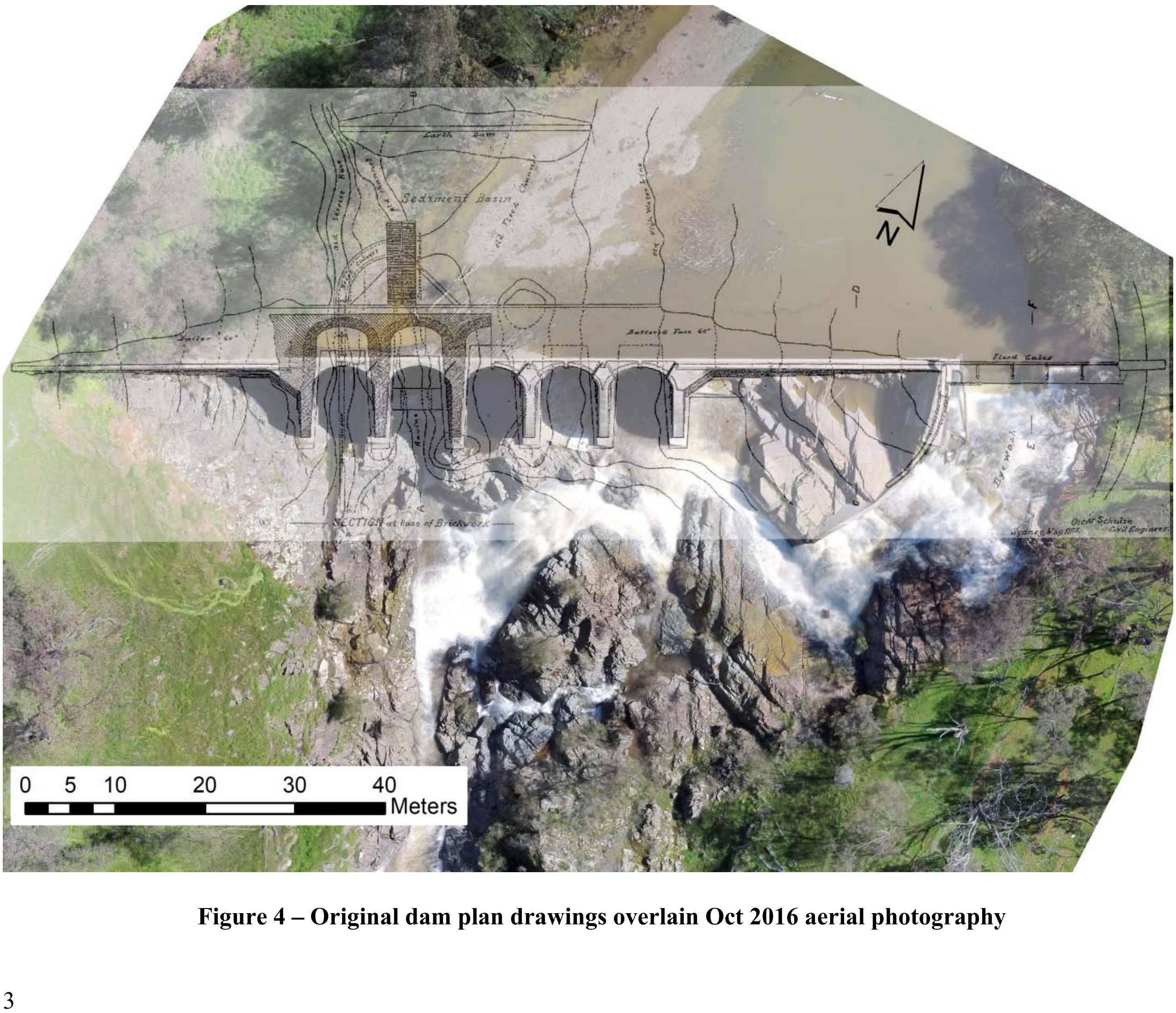

Figure 4 – Original dam plan drawings overlain Oct 2016 aerial photography

An unusual and interesting feature of this dam, is the fact that the gravity wing walls are “the wrong way round” compared with conventional concrete gravity dams (Figure 5).

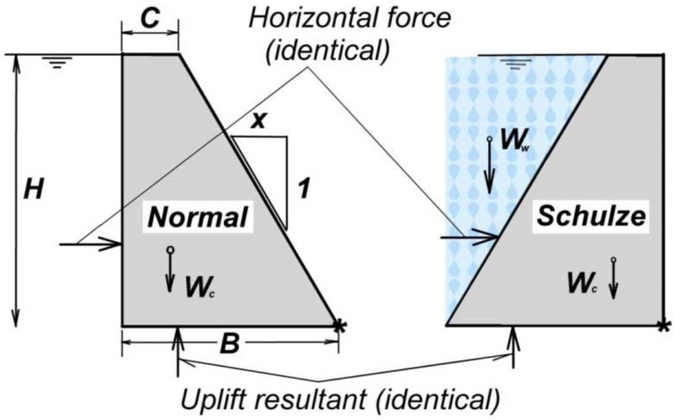

Figure 5 – Unusual ‘reverse’ gravity structures employed by Schulze

Consideration of stability against overturing around the downstream toe of the normal and reverse gravity structures shows that both have the same overturning moments due to horizontal water loading and whatever uplift assumption is adopted. A difference lies in the restoring moments, where the normal section is all governed by the concrete weight, but the ‘Schulze’ section also has a vertical water loading on the upstream face. Consider a case where the crest with is zero (so we have a simple triangular cross-sections). Taking moments around the downstream toe, for each case, gives:

‘Normal section’ (left hand of Figure 4): restoring moment = $W_C \frac{2}{3} B$

‘Schulze section’ (right hand of Figure 4): restoring moment = $W_C \frac{1}{3} B + W_w \frac{2}{3} B$

Therefore the ratio of ‘Schulze section’ factor of safety (**FOS**) against overturning to ‘normal section’ FOS is:

‘Schulze’ / ‘Normal $= \frac{W_C \frac{1}{3} B + W_W \frac{2}{3} B}{W_C \frac{2}{3} B} \quad = \frac{1}{2} + \frac{W_W}{W_C}$

Therefore the ratio is independent of height and the angle of the sloping face, and depends only on the ratio of structure unit weight to water unit weight. If the structure is built of concrete with a unit weight of 22kN/m^3, then the ratio is always 0.5 + 0.45 = 0.95; so the normal section is safer that the Schulze section. However, if the structure is built of brickwork which at Junction Reefs has a unit weight of 17kN/m^{3,} then the Ratio is 0.5 + 0.59 = 1.09; meaning that the Schulze section is safer than the Normal section.

With a finite crest width matters are more complicated and the FOS and effective stress distribution along the base depend on height, crest width and face slope angle. However, as shown in Figure 6 the key issue is the unit weight of the structure. With concrete the Normal design is safer; with brickwork the Schulze ‘wrong-way-round’ design is safer. So Schulze’s design at Junction Reefs dam is not as silly as first appears! In this regard it is worth noting that there is another old brick gravity dam built for mines near Mandurama which is also “the wrong way round”. Whether this was designed by Schulze is not known.

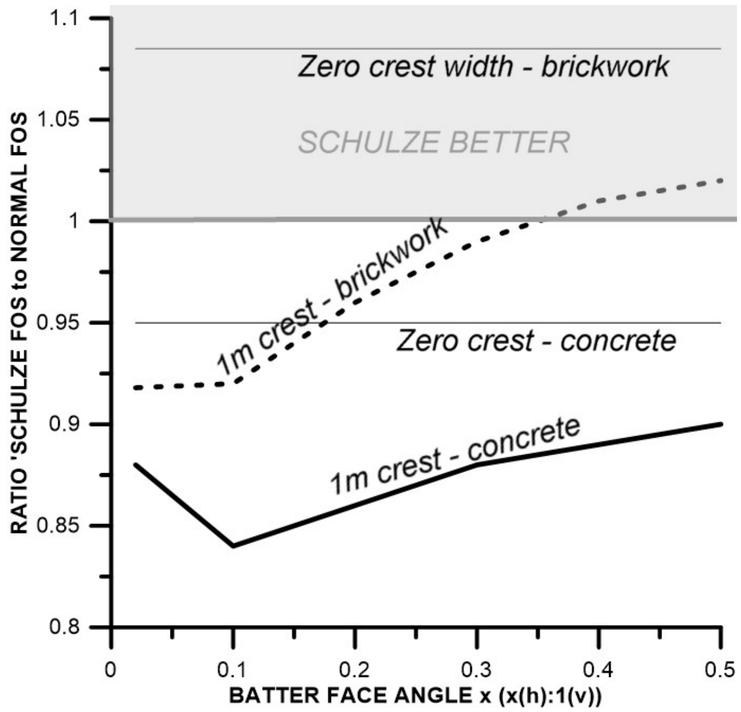

Figure 6: Comparison between overturning safety factor for 'Schulze' and 'Normal' designs.

THE HYDROPOWER POTENTIAL

At the dam site, Schulze (1879) assessed the reservoir storage volume would be 87120000 cubic feet (2465 ML), based on an assessed storage area of 125 acres (50.6 ha) and an average depth of 16ft (4.88m). The elevation drop, for an average reservoir level, was assessed to be 200 feet (61 metres). Schulze (1879) therefore considered, in the case of a period of no flow, that the reservoir could reliably provide a constant 200 horse power (150 kW) for 77 days, assuming a turbine efficiency of 75%, and allowing 10% loss of storage to evaporation. The 1897 paper by Schulze did not present calculations, but they are established below, where the power in a water flow is given as:

$$P = \rho g Q H e \tag{1}$$

Where:			
	P	=	Power produced (W)
	ρ	=	density of water (taken as 1000 kgm^{-3})
	g	=	acceleration due to gravity (9.81 ms^{-2})
	Q	=	discharge ($m^3.s^{-1}$)
	H	=	head differential (m)
	e	=	conversion efficiency

From rearrangement of Equation (1), using Schulze's values, the design discharge is given as:

$$Q = \frac{P}{\rho g H e} = \frac{150000}{1000 \times 9.81 \times 61 \times 0.75} = 0.334 \text{ m}^3.\text{s}^{-1} = 334 \text{ L.s}^{-1}$$

At a discharge of 334 L.s, a reservoir of 2465 ML (minus 10% evaporation) would be emptied in $0.9 \times 2465 \times 10^6/334$ seconds, or 77 days.

Schulze's design to provide a constant 200 horse power was based on anecdotal information regarding flow frequency, stating "according to the proverbial oldest inhabitant of the district, the river has never been dry for longer than three months" (Schulze, 1897).

Since that time, various gauging stations have been established in the surrounding catchments as shown in Figure 7. Gauging at Junction Reefs dam (sometimes referred to as 'Belubula dam', as annotated in Figure 7) has not been reported. In this present paper two separate hydrological models were established to simulate long term streamflows through Junction Reefs dam. Firstly, a hydrologic / hydraulic model was established using the model SWMM 5.

Suitable long-term simulations of rainfall runoff over rural regional can be achieved in SWMM by invoking the 'aquifer' routine for representation of baseflow, which was undertaken using the methodology presented in Pells and Pells (2015). Rainfall records from Bureau of Meteorology stations in Orange (63018), Blayney (63010) and Carcoar (63216) were utilised and model parameters were adjusted until the modelled runoff and flow-frequency from gauges 412165, 412056 and 412080 approximated the gauged runoff and flow frequency diagrams over the period 24th November 1970 to 8th July 2011. This represented a period when most of the gauging data was available, and the key storages in the catchment (ie Lake Rowlands and Lake Carcoar) were operational. These lakes were represented in the model (including the storage volume, evaporation from the storage and spillway overflows), although operational data was only approximately represented. Plots of gauged and simulated runoff depth and flow frequency are shown in Figure 8, and indicate that the model approximated flow-frequency characteristics of the period of observation. The mass balance reported from SWMM is summarised in Table 1, and indicates that the numerical model is based upon reasonable values.

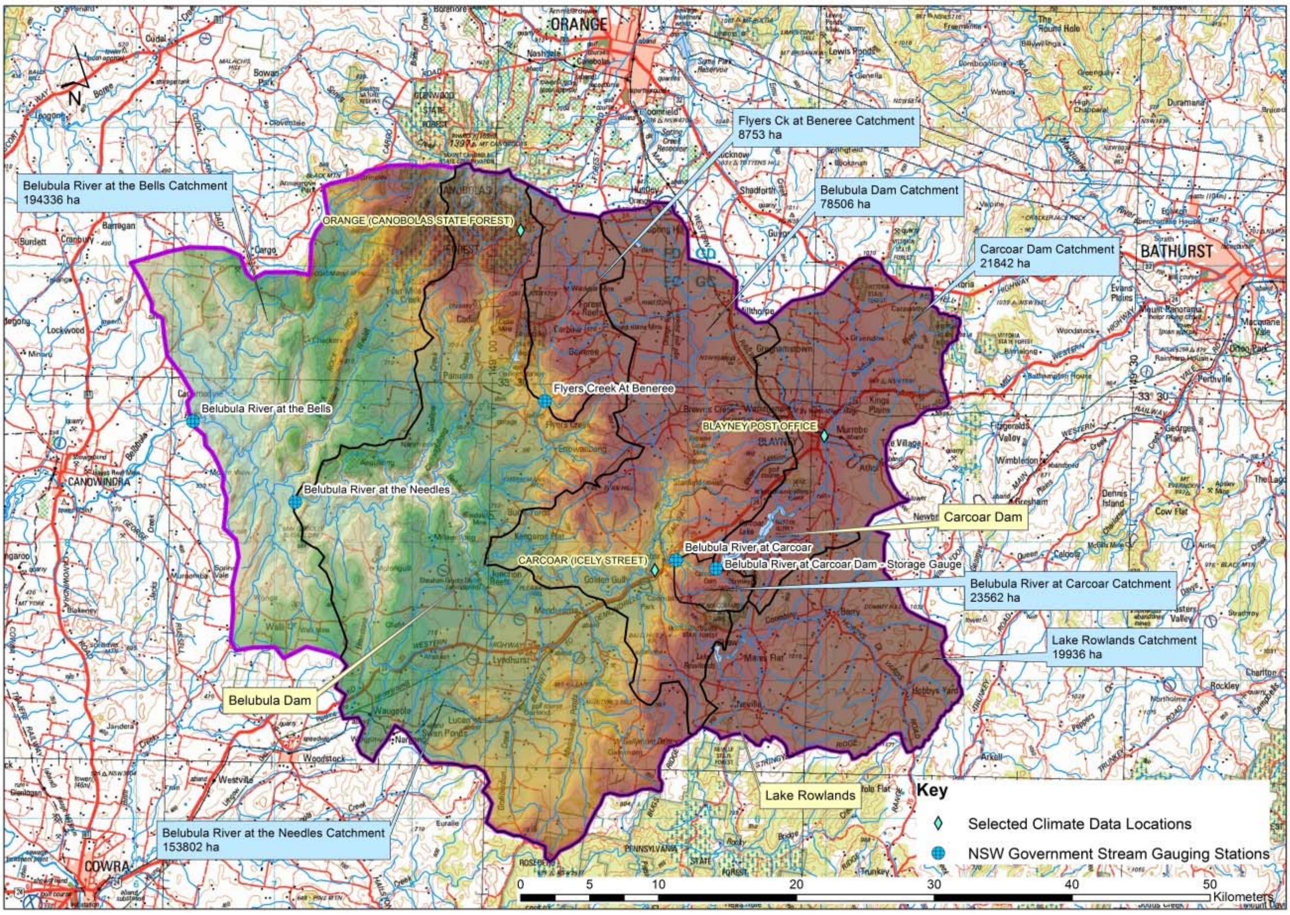

Figure 7 – Gauged catchments in the locality to Junction Reefs dam

Table 1 – Mass Balance of SWMM Model

Item	**Tag**	**Average Annual Volume** (mm/a)	**% of Total Rainfall**
Precipitation	P	774	100%
Total Actual Evapotranspiration	E_{TA}	738	95%
Surface Evaporation	Es	109	14%
Upper Zone Evapotranspiration	E_{TS}	314	41%
Lower Zone Evapotranspiration	E_{TD}	315	41%
Infiltration	I	683	88%
Runoff , Throughflow and Baseflow	F + T + B	49	6%
Recharge to Shallow Aquifers	R	48	6%
Deep Recharge	D	19	2%

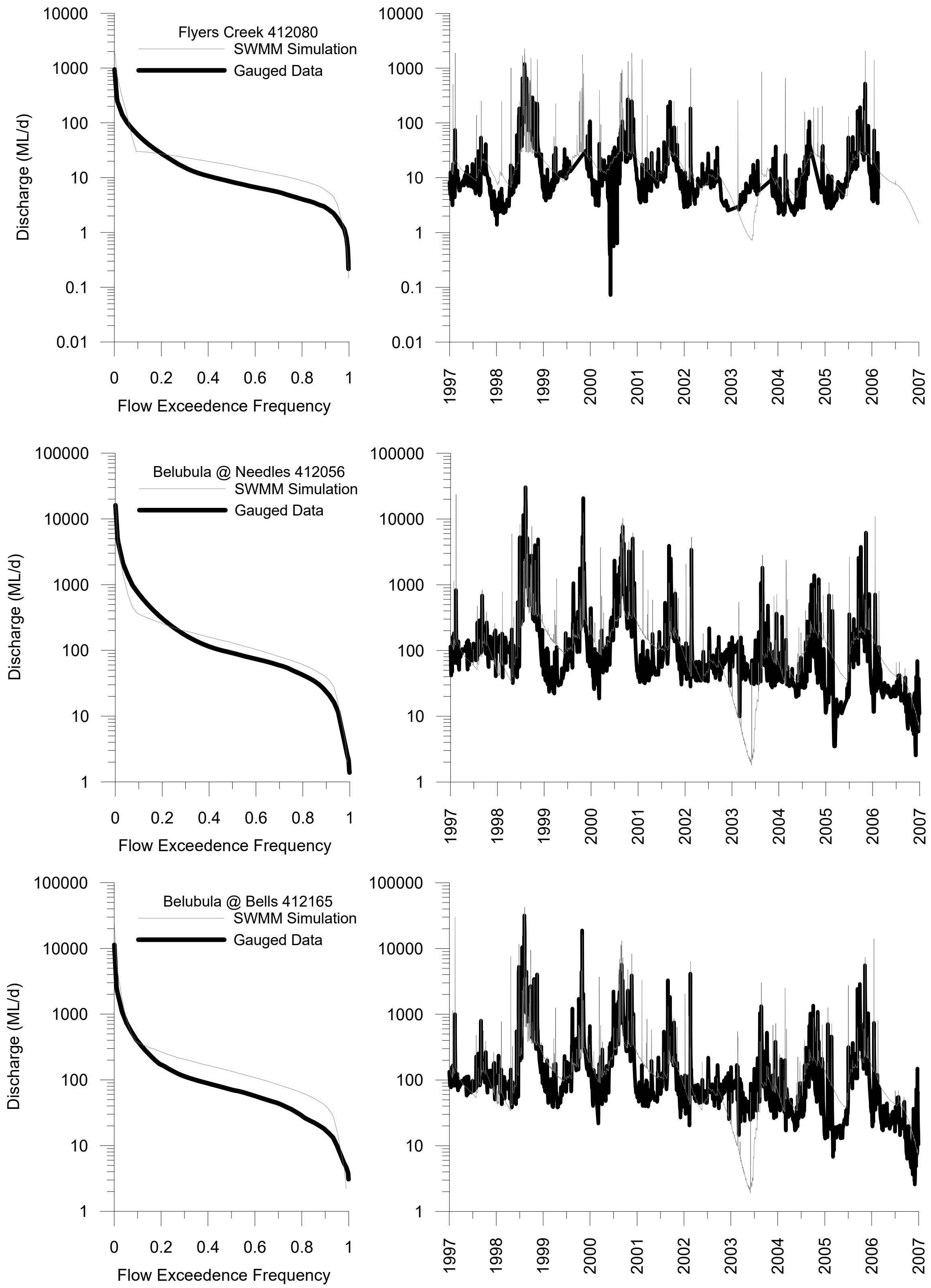

Figure 8 – Calibration of SWMM Model

An alternative and simpler model was also run using SimHyd. The lumped catchment model was run to represent the gauged flows at the Needles at Junction Reefs (Gauge 412056). This model does not directly account for influences from the operation of Lake Rowlands and Lake Carcoar upstream of Junction Reefs Dam.

Both models were then run to simulate flow frequency at Junction Reefs dam using daily rainfall records over a 110 year period, and using catchment conditions that represent current day land use. The predicted flow frequency characteristics at Junction Reefs dams from both models is shown in Figure 9. Average annual runoff from the SimHyd model of

80 mm (10%) was larger than the 49 mm for the SWMM model. A baseflow separation undertaken on the simulated timeseries indicated indices of 0.37 and 0.33 for the SWMM and SimHyd simulations respectively. The flows from the SWMM model were therefore consistently smaller, which may be partly due to the representation of Lakes Carcoar and Rowland. Nonetheless, the flow predictions are considered to be appropriately representative of the uncertainty in the runoff analysis and are used simply to approximate the frequency of historical flows through the dam.

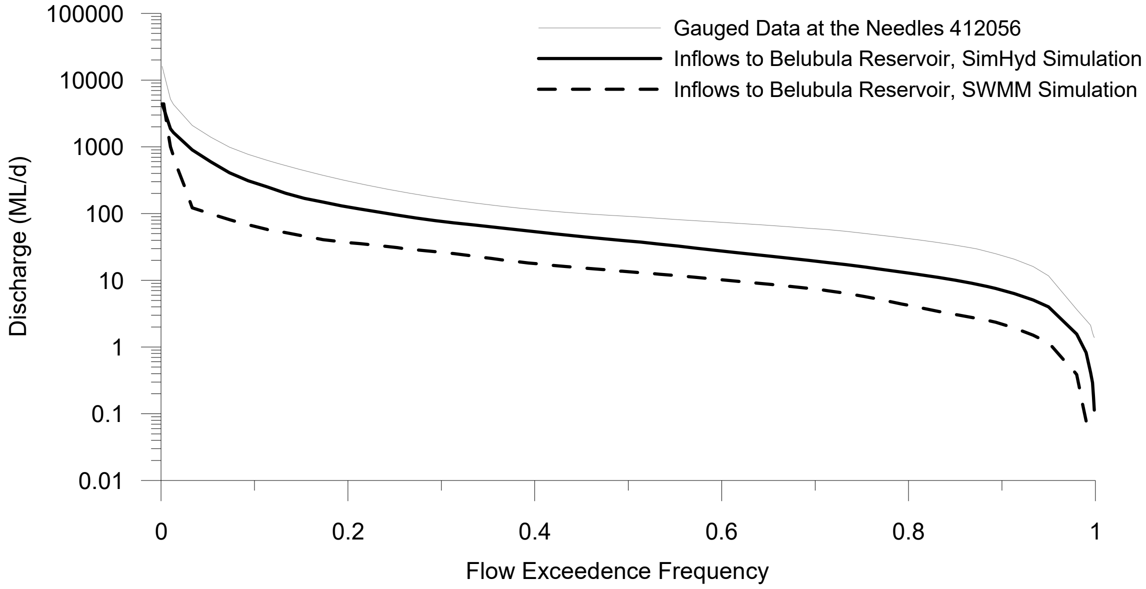

Figure 9 –Simulated Flow-Frequency Distributions for Junction Reefs dam

Theoretical hydro schemes were trialled using the simulated flow frequency timeseries presented above, and for the cases of three different reservoir storage sizes:

1. 2465 ML, representing Schulze's original estimated reservoir volume
2. 160 ML, representing the estimated current reservoir, which has been subject to significant sedimentation
3. 1000 ML, assuming some dredging of sediment is undertaken.

In each case, a daily mass balance at the reservoir was undertaken accounting for simulated inflows, surface evaporation, spillway flows and the operation of a turbine of an assumed capacity. The turbine was operated only when the reservoir storage and inflow was sufficient to deliver a full day of the design capacity. When insufficient storage was available, the turbine was not operated. This was repeated for various turbine sizes, for the three assumed reservoir sizes and for each of the simulated runoff models. The results of this analysis are presented in Figure 10. These analyses indicate that the original power generation aspirations of Schulze - 200 horsepower (150 kW), could have been provided on a 80% to 99% reliability (depending on which hydrological model is adopted), producing approximately 1GWh per annum. In the current condition of the reservoir, being almost completely silted, power generation would be almost on a run-of-river basis, with lower reliability. If some dredging was undertaken, such as to restore reservoir capacity to 950 ML, perhaps 3 GWh could be produced at Junction Reefs dam, using a turbine of 0.5 to 1 MW capacity, and operated on a 40% to 75% utilisation. If the average household power usage is taken as 5 to 10 MWh/annum, this represents powering between 100 to 200 households – a relatively small regional power source.

Removal of sediment from the reservoir would be a significant operation, but does result in greater hydropower potential. Opportunities for sediment removal could consider disposal of the sediment in the old mine workings adjacent to the dam, and/or usage of sediment in regional agriculture, depending on the characteristics of the sediment.

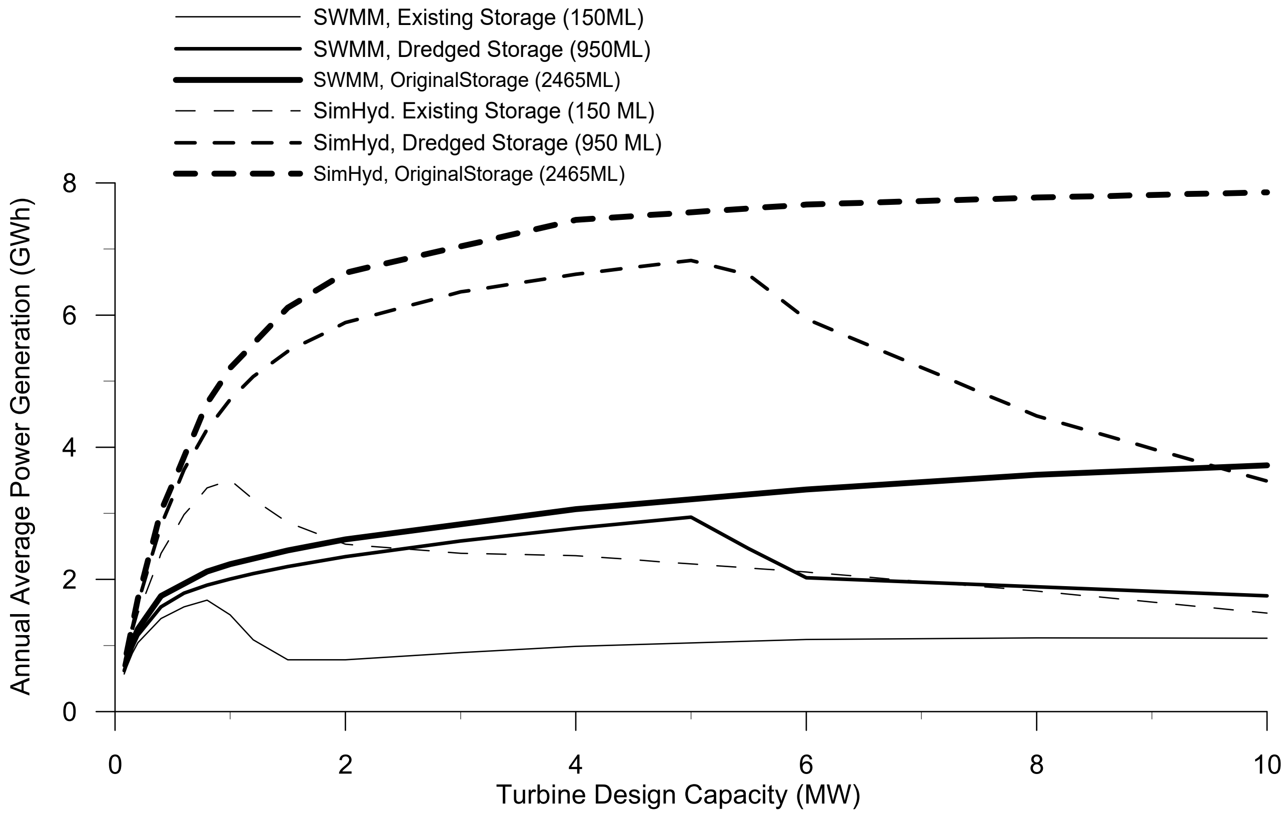

Figure 10 – Simulated power generation potential at Junction Reefs dam

THE SPILLWAY

The flow frequency analysis presented above indicates that the Junction Reefs dam spillway would have been subject to regular spills over the last 120 years. It can be seen from comparison of photographs of the spillway in 1897 and 2014 (Figure 11) that the spillway has been subject to some reshaping through scour. Recent photographs of the spillway are shown in Figures 12 and 13.

Dissipation of stream power has been utilised as an indicator of the erosive capacity of a flow of water (eg Bull, 1979). The original spillway had a well-formed slope, probably around 5 to 7 degrees. A dam crest flood was assessed in the present paper to invoke a spillway discharge of around 10 to 15 m^3/s, and under uniform flows down the original slope, would dissipate approximately 0.6 to 1 kiloWatts per m^2. Significantly higher rates of hydromechanical energy dissipation occur for similar flows plunging over the current rock benches / cascades. The classical solution for hydraulic drop structures (ie as presented in Henderson, 1966) is relatively representative of the current spillway, and for a drop of 4 metres yields an estimate of 70 to 100 kW per m^2. These calculations illustrate that the observed erosion has created increased erosive potential. However, erosion has revealed more competent rock, as discussed below.

The rock mass forming the present day spillway appears to be reasonably fresh exposures of Ordovician sandstones and siltstones, perhaps partly metamorphosed, with a persistent bedding structure dipping 10 degrees upstream. Rock structure mapping undertaken by the authors in 2014 is presented in Figure 14, showing three dominant joint sets. Regular flows have removed a more highly weathered and fractured overburden, which was seen to form the present day left-hand wall of the spillway chute. Regular spillway flows over the years have likely intermittently dislodged wedges of the fresh rock material, formed along the bedding plane, and created by two sub-vertical sets. This intermittent dislodging has formed stepped faces in the spillway, in a head-cutting fashion. Due to the upstream-dip of the bedding, flows plunging from the steps strike perpendicular upon the bedding layers, and fail to create a mechanism for removal of rock units. A relatively stable configuration appears to exist, which proves to be durable against regular flows over the cascades. The demonstrated durability arising from bedding structures dipping upstream was a key feature characteristic of rock mass resistance to scour identified in Pells (2016).

Field assessments by the authors in 2014 assigned a Geological Strength Index (GSI) (Hoek and Marinos) of 75 to 80 for the fresh exposures of rock mass forming the spillway, and of 30 to 35 for the weathered overburden, now eroded from the slope, but still found in the left hand spillway walls. At that time, no correlation between erodibility and GSI had been considered, but subsequent extensive field investigations of spillway erosion presented in Pells (2016) found that GSI could be usefully correlated to observed erosion. An additional factor, to account for structure orientation, was developed and was found, when added to GSI (yielding the erodibility index 'eGSI') to provide improved representation of rock mass erodibility (Pells et al, 2016). The reduction factor applicable to the fresh exposures of rock mass forming the spillway at Junction Reefs dam is nil, in recognition to this favourable shape and orientation, giving an eGSI value of 75 to 80. The more cubic structure of the weathered, closely fractured, material gives a reduction of at least 15, resulting in an eGSI value less than 20.

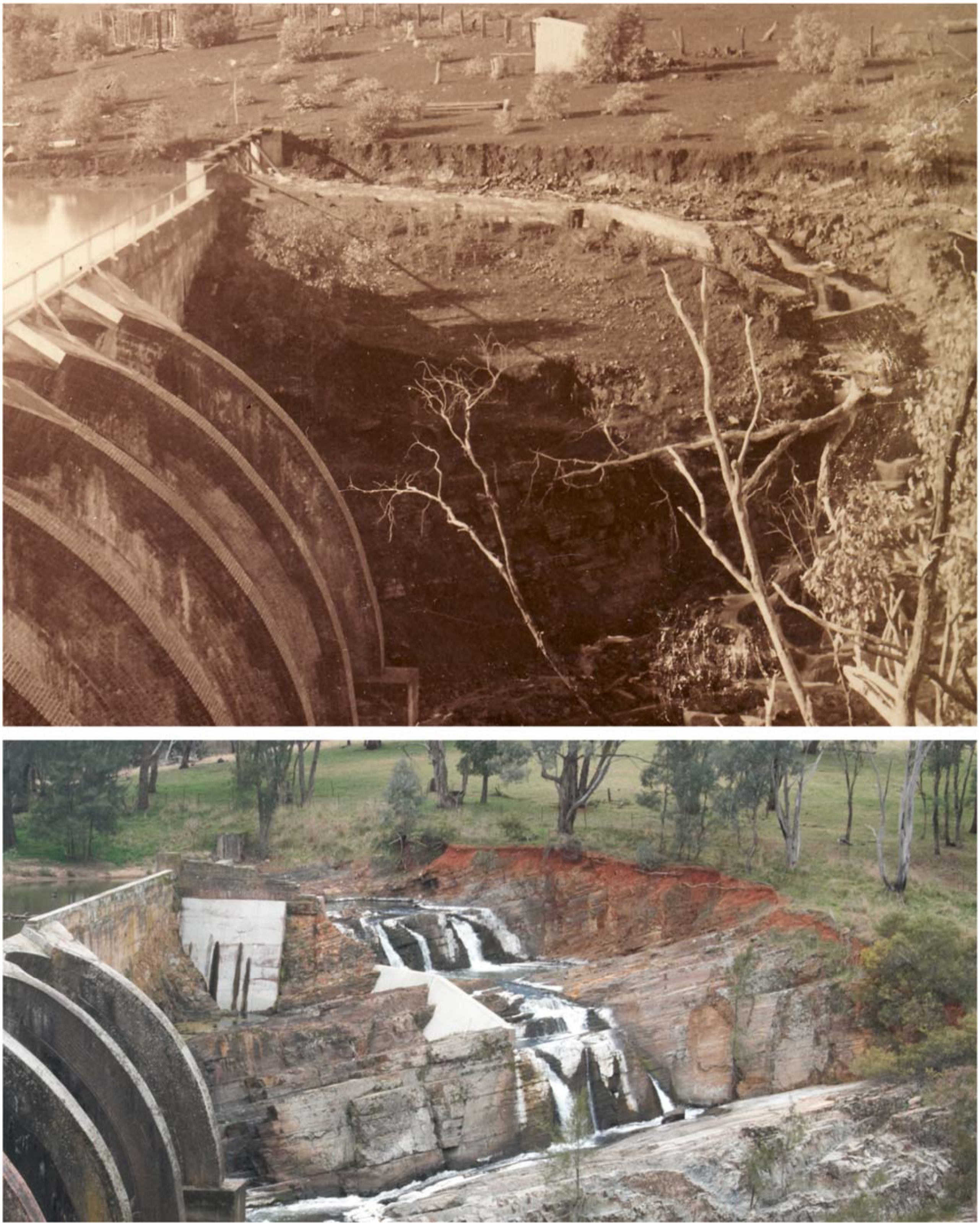

Figure 11 – Junction Reefs dam spillway, 1897 (top) and August 2011 (bottom)

Figure 12 – Junction Reefs dam spillway, January 2014

Figure 13 – First cascade of Junction Reefs dam spillway, January 2014

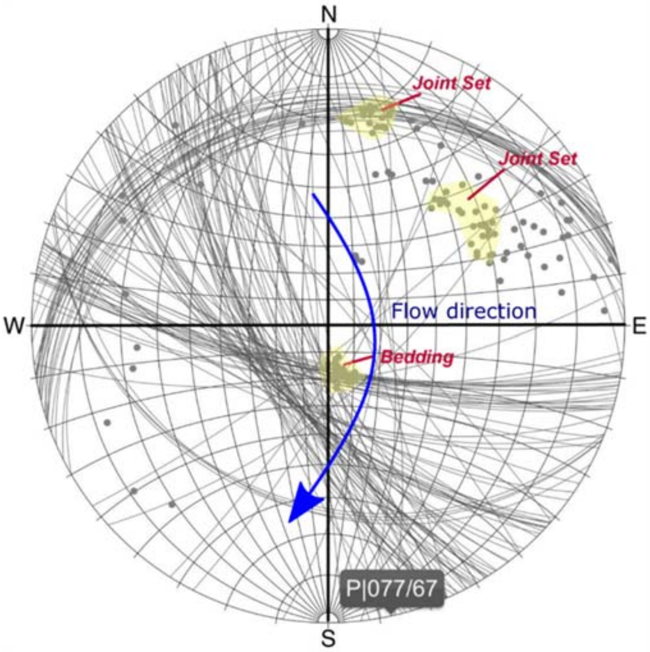

Figure 14 – Geological mapping of defects in the unlined spillway, Junction Reefs dam

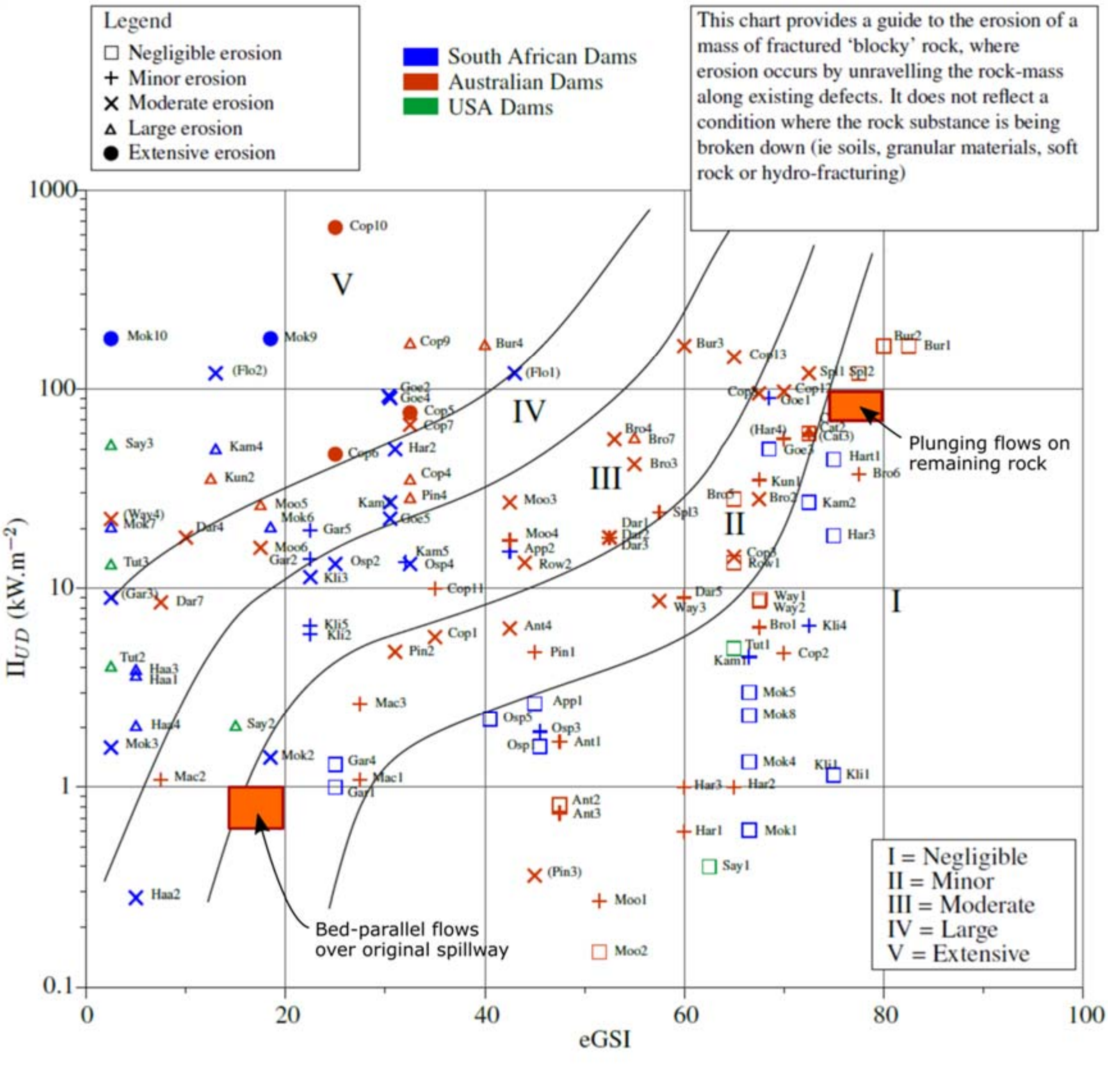

Figure 15 – Spillway erosion vulnerability at Junction Reefs dam, according to the eGSI method

The assessed indexes of erosive power (stream power dissipation) and erosive resistance (eGSI), are plotted on Figure 15, which is the eGSI assessment chart, taken from Pells et al 2016. The highly weathered and fractured rock material that was interpreted to form the bed of the original spillway falls amongst case studies where minor to moderate erosion occurred. Despite higher stream power dissipation, the freshly exposed material forming the current cascade arrangement is associated with a much lower expected risk or erosion due to its structure and orientation.

CONCLUDING REMARKS

The brief review of the design of Junction Reefs dam in light of current data , as presented above, supports the original design objectives set out by Carl Oscar Schulze. The structural design principles were novel, yet sound, and the original hydrological assumptions and assessments of power potential are borne out by recent analysis. Little was published in terms of spillway erosion at the time of design, but current methods indicate that erosion of the designed spillway would now be expected. The assessed durability of the current spillway arrangement provides valuable validation of current assessment methods.

In the social and political context at time of writing, hydropower schemes are considered favourably. Although it is an aged piece of infrastructure, Junction Reefs dam is nonetheless subject to ongoing maintenance for heritage reasons. If it is to be maintained, the question is then asked as to whether it can also be utilised beneficially. Hydraulic calculations indicate that the hydropower potential of the dam is modest, but is nonetheless potentially significant to local power users. Hydrologic modelling indicates that the full hydropower potential of the reservoir could only be achieved if large removal of sedimentation was undertaken. Such an operation could include filling of old mine workings, or usage of sediment in local agricultural industry.

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