Cost-effective spillway design/review for small dams in Victoria: avoiding dam failure emergencies

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Introduction

Australia has a large number of relatively small, privately owned dams (farm dams in particular): those which have failed number in the thousands (ANCOLD 1992). A large proportion of these dams are located in Victoria which has an estimated 170,000 farm dams, 800 of which are large enough to cause serious consequences downstream if they failed (ANCOLD 1992; Murley 1987). The growth of farm dams in Victoria (and Australia) is also increasing at a rapid rate. For example, in the Victorian Lal Lal Reservoir catchment alone (234 km^2), farm dams increased in number from 182 in 1970 to 534 in 1985, representing an increase of about 200% (GHD 1987). When these dams were constructed, the majority more than 20 years ago, their designs were based on rainfall frequencies and intensities, design methods and criteria and standards of risk available at that time. However, these aspects have changed over time, together with population distributions and the condition of the dams, raising serious doubts about dam adequacy.

In modern times, the major concern with dam safety world-wide is the provision of adequate spillway flood capability. This is mainly because significant advances made in the fields of meteorology and flood hydrology have updated both maximum probable rainfalls and design flood standards above those on which most existing dams were based. As a result of these revisions, many dams have insufficient spillway capacities.

In addition to this concern is the fact that most private owners hire contractors to construct their dams. These contractors are, typically, not properly trained or skilled in the design and construction of dams. Thus, many private dams are not built to an adequate standard. For example, the layers of soil that constitute the dams are not properly compacted and the structures are not provided with adequate outlet works. This is evidenced by a recent case study investigating private dam safety management practices in South Australia (Pisaniello 1997, see also Pisaniello and McKay 1998). The study identified many unsafe, hazardous private reservoirs and found that most owners are not taking the necessary action in terms of analysis and upgrading of their structures.

Consequently, the recognition of risks associated with the dams has increased greatly. A need has therefore developed for private dams and risk to co-exist and for owners to appropriately manage their dams in line with current standards in order to reduce the risks involved, reflect community standards and provide increased dam safety assurance to downstream communities.

In particular, owners should review the spillway flood capabilities of their dams, and upgrade if necessary, in order to avoid liability for possible failure consequences (McKay and Pisaniello 1995). Unfortunately, the engineering processes involved are highly rigorous and time-consuming in practice and therefore generate high consulting fees which in many cases are not affordable by private owners. For this reason, owners tend to overlook the need for reviewing their dams and instead develop a sense of complacency, believing that as the dams have not failed up to now, then they will never fail. In essence, owners lack an appreciation of the risk of failure to society and the costs. The result is that dams are deprived of necessary upgrading and downstream communities are placed at risk. Pisaniello & McKay (1998) demonstrate the potential seriousness of this problem.

A clear need has developed for a mechanism that:

- raises public awareness of this problem and improves the transparency of the risks
- promotes consistency and uniform standards
- simplifies the engineering desig/review processes involved while keeping in line with state-of-the-art practice
- minimises review/design costs to private owners and in turn encourages better dam safety management

The Department of Environment and Natural Resources, Victoria, recognising this need commissioned the University of South Australia to undertake a study based on Pisaniello (1997 PhD thesis, see also Pisaniello et al 1999), in order to establish such a mechanism for Victoria. This paper summarises the preliminary procedures involved in the study, presents the resulting cost-effective flood capability design/review procedure, and provides worked examples of how to apply the procedure.

The development process

The Pisaniello (1997) procedure primarily involves the development of regionalised flood capability prediction relationships for dams on small rural catchments based on the Reservoir Catchment Ratio (RCR):

$$RCR = \frac{SC}{PI_{PMF}} \cdot \sqrt{\frac{\sqrt{RA} \cdot SH}{1000 \cdot CA}} \cdot \frac{\log\left\{\dfrac{PI_{PMF}}{PI_{100}}\right\}}{\log\left\{\dfrac{PI_{100}}{PI_{50}}\right\}}$$

(Equation 1)

where:

SC = spillway overflow capacity (m^3/s)

PI_{PMF} = peak inflow for the PMP design flood event (m^3/s)

RA = reservoir area at Full Supply Level (km^2)

SH = maximum height of spillway overflow (m)

CA = catchment area (km^2)

PI_{100} = peak inflow for the 100 year ARI event (m^3/s)

PI_{50} = peak inflow for the 50 year ARI event (m^3/s)

For regions where no variation is observed in the Annual Exceedance Probability (AEP) of the Probable Maximum Precipitation (PMP), the RCR can take on the compact form:

$$RCR = \frac{SC}{PI_{PMF}} \cdot \sqrt{\frac{\sqrt{RA} \cdot SH}{1000 \cdot CA}}$$

(Equation 2)

Developing the RCR, based on the Pisaniello (1997) procedure, necessitates the collection and derivation of appropriate 'calibrated' catchment and reservoir data in the study region, and the formulation of a range of hypothetical dams (approximately 20) on each catchment representing all possible scenarios up to the PMP design flood event.

An initial search for appropriate 'calibrated' data for rural catchments up to 100km² proved unsuccessful. SMEC Victoria was then commissioned by the University to undertake a more detailed search: this revealed an absence of such data in the State. It was therefore necessary to generate the required calibrated data, but unfortunately, only three small gauged catchments with reasonable historical data were available for this purpose. Fortunately, these are reasonably well spread throughout the State and for the purposes of this study, can be considered to represent the three main regions of the State relative to the Great Dividing Range (GDR):

1. **Barringo Ck.** GS 230209
   (Area = 5.1 km², 20 yrs record):
   Central GDR (ie. mountainous region)
2. **Shepherds Ck.** GS 415244
   (Area = 6.4 km², 20 yrs record):
   Inland side of GDR
3. **LittleAire Ck.** GS 235204
   (Area = 11.2 km², 40 yrs record):
   Coastal side of GDR

It should be noted that the coastal region warrants further subdivision into East and West regions in order to include cases which represent the Gippsland zone: this will be undertaken in future studies in order to increase confidence in the developed prediction relationships applying to the whole of Victoria.

The RORB program (Laurenson and Mein 1990) was used for modelling; catchment and sub-area delineations were made using 1:25,000 scale topographic maps. All catchment calibration, reservoir flood capability and PMF studies were undertaken in accordance with Australian Rainfall and Runoff (AR&R) (IEAust 1987 and new edition) and Bulletin 53 (BoM 1994).

The calibration flood studies basically involved:

- collation of recorded streamflow, daily rainfall and pluviograph data
- RORB catchment modelling
- trial-and-error 'fitting' of modelled hydrographs with recorded hydrographs

SMEC Victoria was commissioned by the University of SA to perform the calibration study for the Barringo Creek catchment in order to provide a basis for independent comparison and check.

In order to create the flood capability prediction relationships, it was necessary to produce a wide range of flood capability outcomes relating to embankment dams placed at the outlets of the regional calibrated catchments. The aim of the process is to represent the hydraulic response of any size of reservoir and spillway(s) relative to the hydrological flood response of the selected 'catchment type' (Pisaniello 1997). In brief, this was achieved for Victoria by performing the following:

- Creating a number of hypothetical dam cases, 57 in total, at the outlets of the selected catchments, comprising of varying size reservoirs and spillways which will produce a wide range of flood capability outcomes up to the PMF. The spillways must be free flowing and weir-type in nature. A good variety of cases was obtained by either:
  - widening the spillway
  - raising the top of the crest which increases spillway height
  - deepening the spillway which increases spillway height and decreases reservoir surface area and storage capacity
  - raising the entire embankment and spillway which increases reservoir surface area and storage capacity.
- Including each of the hypothetical dams as 'special storages' in the already created RORB models of their respective catchments.
- Determining design rainfall information and design losses for the selected catchments for events between the 20 year ARI and the PMF using the procedures described in AR&R (1987 and new edition) and Hill et al (1996).
- Using the RORB program to route flood hydrographs through each of the hypothetical storages, assuming the most conservative 100% full 'start' storage level case, to determine peak inflow, peak outflow and water elevation for all events up to the PMF.
- Producing a design peak flow prediction equation for the PMF event, ie. scatter plot of catchment area (km²) versus peak flow (m³/s) in the logarithmic domain. This equation when substituted into the RCR establishes a Regionalised Reservoir Catchment Ratio (RRCR).
- Using the determined peak inflows and elevations to establish peak inflow-frequency and elevation-frequency relationships for each dam. With these relationships the Imminent Failure Flood (IFF) capability of each dam is determined as 1/AEP (years). The IFF is taken as the smallest flood which peaks at the lowest point of the non-overflow crest (ANCOLD 1986): this is in line with the ANCOLD (2000) definition of Dam Crest Flood (DCF) for embankment dams. It should be noted that ANCOLD (2000) defines IFF as 'the flood event that could be reasonably expected to cause failure of the dam', and hence, for the purposes of this paper IFF is 'reasonably' assumed to coincide with DCF.
- These flood capability outcomes are used to create scatter plots of RRCR versus IFF. Lines of best fit are then drawn through the scatter plots and the associated regression equations are determined, thus producing the required reservoir flood capability prediction relationships.

The flood capability relationships developed using the above procedure form the main part of the overall design/review mechanism presented later.

Study results

Calibration flood studies

As described, the calibration process involved generating the RORB parameters k_c and m by trial-and-error 'fitting' of modelled hydrographs (using catchment losses as determined from Hill et al 1996) with recorded hydrographs for the two largest historical events for each catchment. The calibration results are presented in *Table 1*. It is important to note that 'good quality' historical data were available for the Little Aire catchment only (i.e. around 40 years of record). The data for the two other stations, although workable, were rather poor (i.e. around 20 years of record): this deems the results for these catchments somewhat unreliable for use with less frequent events (i.e. 100 yrs to PMF) which is unfortunate as such events form the basis of this project. Nevertheless, and despite this, these results are based on the best available data and therefore their use would

GS 230209 Barringo Creek (A = 5.1 km^2)*					
Storm event	Peakflow	Calibrated parameters		AR&R (1987)	Andrews curves
	(m^3/s)	k_c	m	k_c (m=0.8)	k_c (m=0.8)
May 1974	1.76	13	0.8	5.3	3.6
July 1990	0.88	7	0.8		
GS 415244 Shepherds Creek (A = 6.4 km^2)					
Storm event	Peakflow	Calibrated parameters		AR&R (1987)	Andrews curves
	(m^3/s)	k_c	m	k_c (m=0.8)	k_c (m=0.8)
Sep 1984	5.13	11.2	0.8	1.6	1.4
Jan 1987	2.91	13.0	0.8		
GS 235204 Little Aire Creek (A = 11.2 km^2)					
Storm event	Peakflow	Calibrated parameters		AR&R (1987)	Andrews curves
	(m^3/s)	k_c	m	k_c (m=0.8)	k_c (m=0.8)
Jun 1978	24.5	7.0	0.7	7.6	9.8
Oct 1976	19.2	7.5	0.8		

The Calibration flood study for this catchment was undertaken by SMEC Victoria

Table 1: Comparison of RORB Parameters as Determined from Various Means

represent current acceptable practice.

In an attempt to substantiate the calibrated results, parameters were also determined using:

- regionalised prediction equations presented in AR&R (IEAust 1987), and
- Andrew's Fourier Plots (Dyer et al 1994). The results are also presented in *Table 1*, where they are compared to the calibrated results.

Table 1 demonstrates that, in general, there exists significant variation between the AR&R (IEAust 1987) and/or Andrews Curves results and the Calibrated results; Little Aire Creek being somewhat of an exception. The calibrated k_c values for both Barringo Ck. And Shepherds Ck. appeared abnormally high, which has the tendency to underestimate design floods, that is: they represent a non-conservative approach. At the same time, the lower values determined using Andrew's Curves would tend to overestimate design floods which represents a conservative approach. The erratic nature of these results was seen to have the potential to impact adversely on the final design/review curves. Therefore, following consultation with DNRE, it was decided to provide a sensitivity analysis and develop the design/review relationships at 'both ends of the spectrum', i.e. for both (1) a non-conservative approach (using the calibrated k_c values), and (2) a conservative approach (using the k_c values determined from Andrew's curves for the two smaller gauged catchments), as described below.

Developing the flood capability prediction relationships

Non-conservative relationships

A total of 57 hypothetical dam cases were created on the catchments, based on the Pisaniello (1997) procedure, so as to represent all the possible combinations of reservoir size and spillway capacity to pass the entire range of design floods up to the PMF. Flood capability studies were undertaken for each case in line with AR&R (IEAust 1987), and also keeping in mind the new edition of AR&R (1998), Book VI. All cases resulted in an AEP of PMF of 1 in 10^6 using the AR&R (1987) procedure (compared with 1 in 10^7 for all cases using the procedure of the new edition of AR&R): this therefore led to the Reservoir Catchment Ratio taking on the compact form, ie. *Equation 2*. Given the previous uncertainties surrounding Book VI of the new edition of AR&R, it was decided to adopt the more conservative AEP of PMF (1 in 10^6) for all works described here, while, if necessary, the less conservative case (1 in 10^7) would be considered in future works.

The magnitude of the Imminent Failure Flood (IFF) capability 1/AEP (years) was found to be a power function of the Reservoir Catchment Ratio for a single line of best fit over the entire range of AEPs. The sample data and line of best fit are presented in *Figures 1 and 2* respectively.

The coefficient of determination (R^2) for the relationship presented in *Figure 2* suggests a high level of predictive accuracy. However, to apply the above relationship also required the ability to accurately predict the peak PMF inflow associated with a dam for input to the RCR in order to establish the Regionalised Reservoir Catchment Ratio (RRCR). Therefore, the peak PMF inflows determined for the calibrated catchments were plotted against their areas and fitted with lines of best fit in the logarithmic domain. The peak PMF inflow (PI_{PMF}, m^3/s) was found to be a function of catchment area (CA, km^2) for the line of best fit as follows:

$$PI_{PMF} = 1.1723.CA^{2.5033} \qquad (R^2 = 0.9734)$$

(Equation 3)

The above equation was substituted into the RCR (*Equation 2*) to produce the Regionalised Reservoir Catchment Ratio (RRCR) applicable to the sample region as follows:

$$RRCR = \frac{SC}{1.1723 \cdot CA^{2.5033}} \cdot \sqrt{\frac{\sqrt{RA} \cdot SH}{1000 \cdot CA}}$$

(Equation 4)

A new flood capability prediction relationship was constructed using the same sample outcomes but based on the above RRCR. The resulting scatter plot and line of best fit are presented in *Figure 3*.

Figure 3 demonstrates increased scatter and, hence, some loss of accuracy in moving from the RCR to the RRCR; this is a direct result of using the derived PMF prediction equation. Nevertheless, the level of accuracy displayed is still considered acceptable for predicting the flood capability of reservoirs on small catchments in the region.

Conservative Relationships

As indicated, the sensitivity analysis involved reconstructing the IFF prediction curves (i.e. *Figures 1, 2 and 3*) based on the lower k_c values determined using Andrew's Curves for the two smaller gauged catchments (see *Table 1*). A total of 19 additional points covering a range of AEPs from the 20 year ARI to the PMF were derived for this purpose: these are illustrated for the RCR in *Figure 4*.

Figure 4 demonstrates minimal overall scatter despite the large range of k_c values which the entire data set represents (ie. 1.4 to 13): this is a positive result as it suggests that the RCR absorbs much of

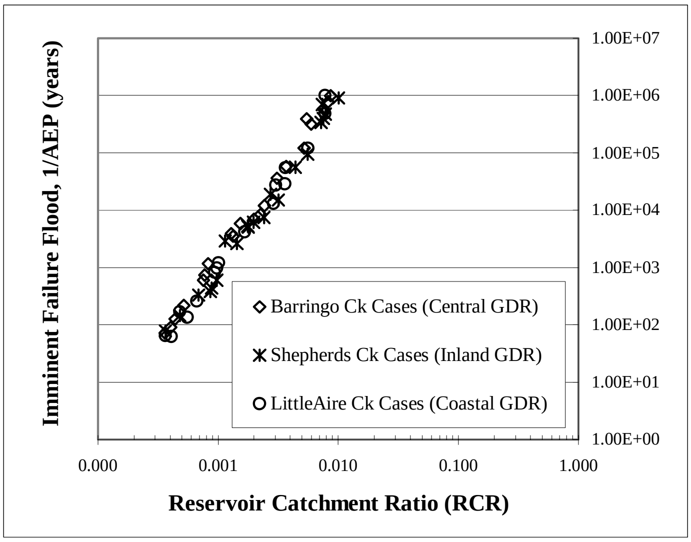


Figure 1: sample data according to each of the 'calibrated' regions

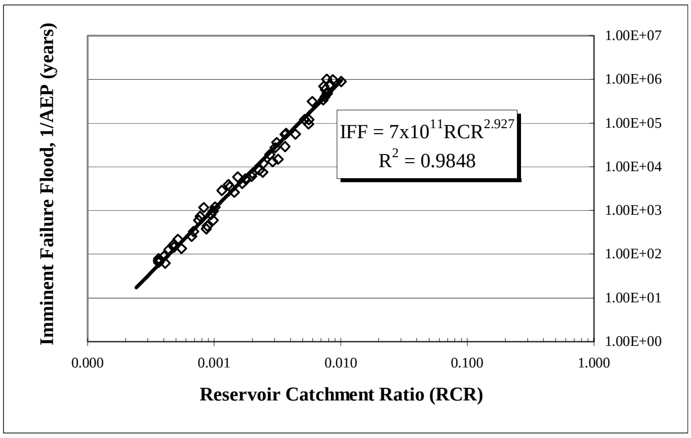


Figure 2: relationship between RCR and IFF capability for entire sample space

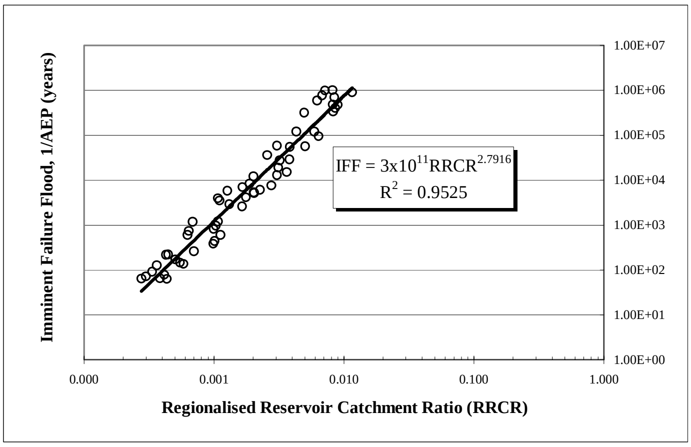


Figure 3: sample data and line of best fit for IFF prediction based on the RRCR

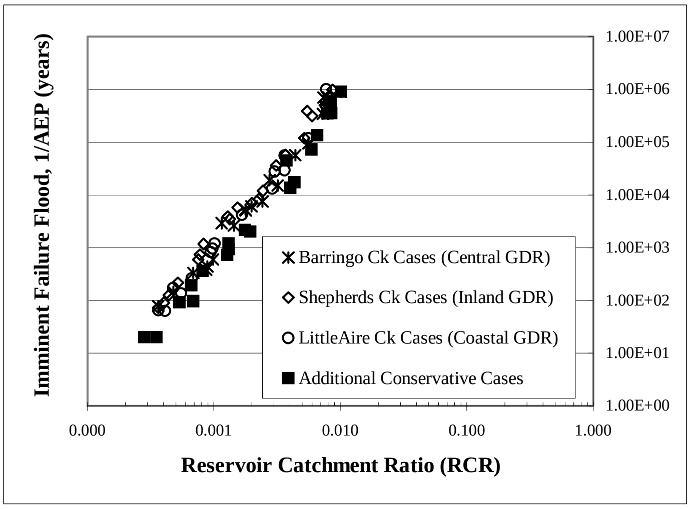


Figure 4: additional RCR sample data derived for the sensitivity analysis

the k_c influence in IFF prediction.

As before, applying the above relationship required the ability to predict the peak PMF inflow associated with a dam for input to the RCR in order to establish the RRCR. The PMF predictor derived for this purpose using the conservative k_c values is:

$$PI_{PMF} = 78.411.CA^{0.8123} \qquad (R^2 = 0.74)$$

(Equation 5)

Equation 5 was substituted into the RCR to produce a 'conservative' RRCR applicable to the sample region as follows:

$$RRCR = \frac{SC}{78.411 \cdot CA^{0.8123}} \cdot \sqrt{\frac{\sqrt{RA \cdot SH}}{1000 \cdot CA}}$$

(Equation 6)

A new flood capability prediction relationship was constructed using the same sample outcomes but based on the above RRCR. The resulting scatter plot and line of best fit are presented in *Figures 5 and 6* respectively, together with the non-conservative cases for comparison.

As before, *Figures 5 and 6* demonstrate increased scatter and, hence, some loss of accuracy in moving from the RCR to the RRCR. Despite this, however, the R^2 value displayed in *Figure 6* for the conservative curve still suggests a high level of predictive accuracy: this is a positive result. Figure 6 also demonstrates minimal separation between the conservative and non-conservative curves, with the curves actually converging towards the PMF. This is very encouraging considering the wide range of k_c values that the curves represent: similar to the RCR, the RRCR also absorbs much of the impact of k_c variance.

Application of the developed flood capability design/review

Procedure

The relationships presented in the above Section (ie. *Figures 3 and 6*) provide a procedure for engineers and dam owners to readily and effectively review and/or design the spillway flood capability of reservoirs on small catchments (area up to, say, 12 km^2) in Victoria. ANCOLD (1986) criteria on design floods for dams, which for the most-part coincide with ANCOLD (2000) 'fallback' acceptable flood capacity criteria, can be incorporated into both *Figures 3 and 6* to create *Figure 7*: the principal design/review tool.

The procedure can be used in either review or design mode. However, the following three main conditions are associated with the mechanism:

- the catchment must be free of any significant flow attenuating storages upstream of the principal reservoir as these contribute to non-systematic, case-specific type flood response
- the spillway(s) must be free flowing and weir-type in nature
- the IFF must be taken as the smallest flood which peaks at the lowest point of the non-overflow crest. Providing this conservative condition is acceptable, the mechanism can be applied to any dam-type structure. ANCOLD (1986 and 2000) suggest that this

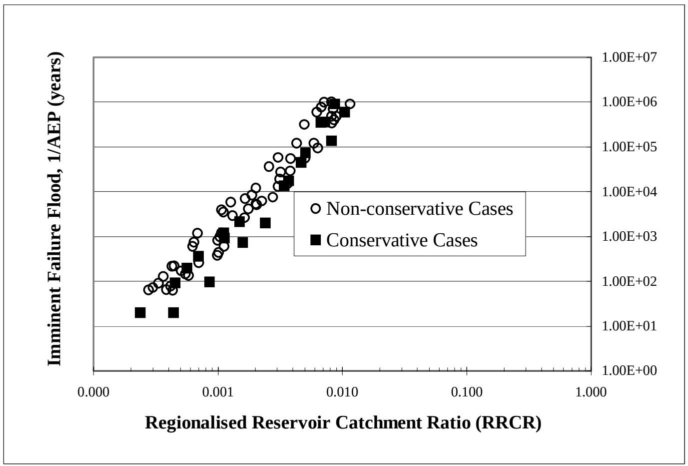


Figure 5: additional RRCR sample data derived for the aensitivity analysis

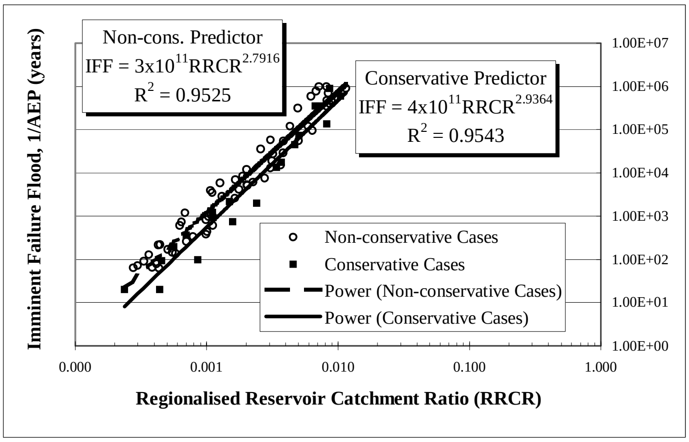


Figure 6: additional sample data and line of best fit for IFF prediction based on the conservative RRCR and comparison with non-conservative curve

condition is appropriate for embankment type dams

When using the procedure in review mode, the simple parameters required in the associated dimensionless ratio must be first determined for an existing reservoir. These parameters are then put into the prediction relationship to read off the corresponding flood capability, which is automatically checked against the displayed ANCOLD criteria. When used in design mode, the same basic parameters are related to a proposed reservoir, or upgrade of an existing reservoir. The parameters must be varied iteratively in the associated dimensionless ratio until the ANCOLD safety criteria together with the owner's storage needs are satisfied. Any proven method for estimating the storage capacity of a reservoir can be a useful tool in the iteration process, but is not a critical one as it does not affect the predicted flood capability used for design (this is illustrated in Appendix A). Pisaniello (1997) developed a model for this purpose based on two equations:

$$V = 0.415\,A.H$$

(Equation 7)

$$V(h) = \tfrac{1}{2}\left[\frac{A.h^2}{2H} + \frac{A.h^3}{3H^2}\right]$$

(Equation 8)

where:
V = total storage volume
A = top surface area
H = maximum height of storage
h = any height less than the maximum height (H)
V(h) = storage volume at height (h)

This model was verified by Pisaniello (1997) against real storage-height relationships, but unfortunately, these were of South Australian farm dams only. In order for the model to be used with confidence here, it should be verified against a Victorian data set. Nevertheless, and despite this, the model can still be used as a 'rough' predictor of storage capacity for farm dams in the State.

As to which relationship should be adopted in any particular case, this depends on the level of risk that an owner is prepared to take and/or the judgement and discretion of the design engineer. As a general rule, it is recommended that for design, the limiting ANCOLD criteria should always be satisfied with the conservative curve. However, when reviewing existing dams, particularly Low and Significant hazard ones, if the limiting ANCOLD criterion is not satisfied by a small margin via the conservative approach, but is satisfied with the non-conservative approach, then the overall flood capability can be based on the latter (at least until the former is refined in future works as described below). Both review and design worked examples are presented in *Appendix A*.

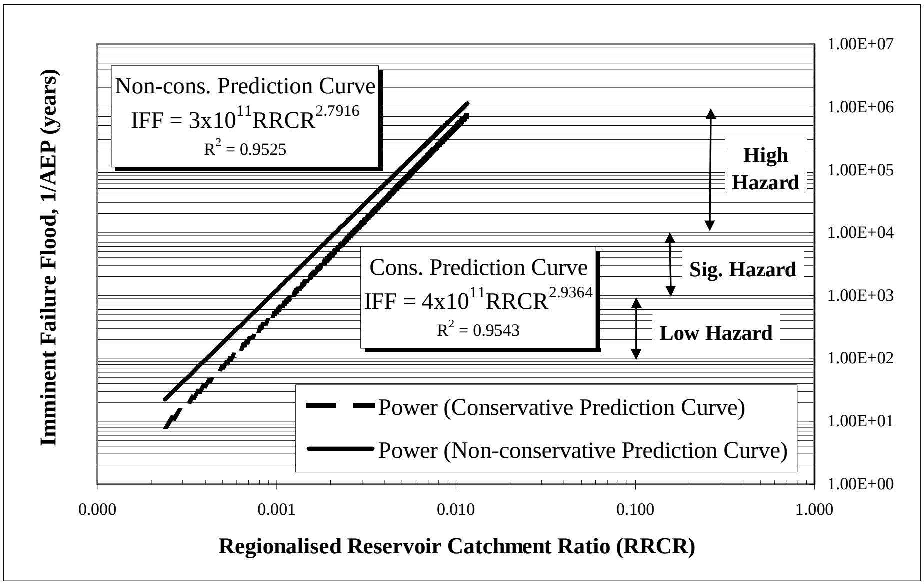
Overview, discussion and future research

At this stage the credibility of the relationships presented in *Figure 7* may be questionable due to the lack of representation of varying calibrated catchment sizes and subsequent uncertainty surrounding the RORB k_c parameters. Nevertheless, they do demonstrate worthiness of further works to increase credibility and genuine potential to provide a beneficial design/review tool to farm dam owners.

In essence, there exists an underlying need to better establish appropriate k_c values for small rural catchments in Victoria. Given the lack of small gauged catchments in Victoria, the Dyer et al (1994) procedure would be ideal for readily determining k_c values on small ungauged catchments in the State: this being the sort of catchments on which small farm dams are commonly located. However, in its calibration study of Barringo Creek, SMEC noted that the catchment did not identify particularly well with any of the groups of Andrew Curves and that this is not unusual in their experience. In contrast, when applied to the Shepherds Ck. and Little Aire catchments, this procedure provided for remarkable coincidence with the Type 2 Andrew Curve, thus providing some support for its use on other catchment cases for further refining the flood capability design/review curves. It is reasonable to adopt either the Dyer et al procedure or the AR&R (IEAust 1987) prediction equations in place of the abnormally high calibrated values as these were derived from very limited historical data (i.e. 20 years) which can not be related with confidence to extreme events. This notion is also supported by the analysis of the Little Aire catchment which contained much better historical data (i.e. 40 years) and in turn provided 'more expected' outcomes which better coincided with both the Dyer et al and AR&R values.

As such, future works will be undertaken so as to refine the conservative design/review relationship by:

- Establishing a 'well spread' range of additional catchments, say 4 to 6, of varying morphometry (particularly of the smaller scale size and including at least one in the Gippsland region)—determining k_c using either the Dyer et al (1994) procedure and/or AR&R (IEAust 1987) prediction equations—



For Non-Conservative approach:

$$RRCR = \frac{SC}{1.1723 \cdot CA^{2.5033}} \cdot \sqrt{\frac{\sqrt{RA} \cdot SH}{1000 \cdot CA}}$$

For Conservative approach:

$$RRCR = \frac{SC}{78.411 \cdot CA^{0.8123}} \cdot \sqrt{\frac{\sqrt{RA} \cdot SH}{1000 \cdot CA}}$$

where: SC = spillway overflow capacity (m^3/s)
RA = reservoir area at Full Supply Level (km^2)
SH = maximum height of spillway overflow (m)
CA = catchment area (km^2)

Figure 7: reservoir flood capability design/review relationship incorporating ANCOLD (1986) criteria

and including these in the refinement process so as to increase credibility and confidence in the developed relationships applying to the whole of Victoria.

- Given the final publication of Book VI of the new edition of AR&R (1998)—fully developing the alternative, less conservative relationship based on 1 in 10^7 AEP of PMF as determined using Book VI: this will merely produce a similar relationship to that presented here but with different slope.

The above works are currently being undertaken and will be reported in a future article.

Conclusion

There is a clear need to encourage private owners to review the spillway flood capabilities of their dams in line with current acceptable practice and to take appropriate remedial action where necessary. The regionalised procedure developed here can be used to provide such encouragement. The procedure is applicable to dams on small catchments up to 12 km^2 in size: this will cater for most private dam cases in the State.

The main benefit of the procedure is its simplicity which dramatically reduces the effort and resources required for conducting a 'state of the art' reservoir flood capability study. The procedure provides a basis for quick yet accurate review and/or design of private dam spillways against any design flood standards, and is in line with modern acceptable practice which is of critical importance in a court of law.

However, at present the relationships upon which the procedure is based may be seen to lack credibility primarily because of the lack of representation of varying catchment sizes throughout the State. This is due to the absence of appropriate small gauged catchments throughout the State and the uncertainty associated with the calibration results: this will be rectified in future works. At the same time, the sensitivity analysis has done well to demonstrate:

- the narrow bound within which more refined relationships will lie, therefore making the relationships presented here 'useable' in their current form
- the worthiness of further works to increase credibility
- genuine potential to provide both a reliable and beneficial design/review tool to farm dam owners, which will undoubtedly encourage better private dam design and safety management in Victoria.

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Conference Announcement

Twelfth world congress on Disaster and Emergency medicine

The twelfth world congress on Disaster and Emergency medicine welcomes you in an exceptional congress setting, where for three days, we will share our experience, present our research and discover the work of other international teams regarding emergency, catastrophe and humanitarian medicine.

It is to be held in Lyon from the ninth to the twelfth of May, 2001. The congress will deal with the practical aspects of catastrophe situations such as setting up a triage center, handling radio communications, studying identification procedures.

Two specialized trilingual one-day symposiums will also be held: the first is for emergency nursing staff; the second for paramedics, emergency medical teams and ambulance staff. Their aim will be to compare and share practical experience, guidelines and projects on an international level.

There will also be a discussion on how to evaluate medical practice. The organisers encourage you to bring to the debate methods and results and finally, the congress aims to highlight new pedagogical tools.

For further information

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www.wcdem2001.org, or http://pdm.medicine.wisc.edu/pdmcalendar.html.)

Appendix A : worked examples demonstrating the application of the developed flood capability design/review procedure

Review Mode Worked Example

Case Description: Farmer Jones owns an embankment dam at Mount Macedon in Victoria with a catchment area of 8 km^2 (as measured from 1:25,000 scale topographic map) and a well populated valley downstream in what would be a dam-break inundation area. The reservoir has a maximum still-water depth of approx. 10m and a surface area at Full Supply Level (FSL) of 0.048 km^2 (as measured from 1:10,000 scale aerial photo). The dam has a free flowing, broad crested weir-type spillway which is 10m wide and 2m high (max.) to the lowest point on the non-overflow crest. Mr Jones would like to know if the flood capability of his dam is of adequate standard in relation to ANCOLD(1986) guidelines?

Case Solution: In accordance with ANCOLD guidelines, the dam warrants a 'High' hazard rating given the populated valley downstream. It must therefore have an IFF capability of at least 1 in 10,000 AEP (see *Figure 7*) in order to be of adequate 'ANCOLD' standard. This can be checked as follows:

1. First check via Non-conservative curve (*Figure 7*):

- Determine Non-cons. RRCR:
  - RA = 0.048 km^2, CA = 8 km^2, SH = 2m,
  - need to determine spillway capacity (SC) which for a rectangular weir with flow width, SW (m), and weir coefficient, C_w, is given by **SC = C_w.SW.$SH^{1.5}$**, where C_w = 1.69 for free flowing, broad crested weir-type spillway (IEAust, 1987; Pisaniello, 1997). Hence, with SW = 10m, SC = 1.69x10x$2^{1.5}$ = 47.8 m^3/s.
  - Substituting into Non-cons. RRCR;

$$RRCR = \frac{47.8}{1.1723 \cdot 8^{2.5033}} \cdot \sqrt{\frac{\sqrt{0.048} \cdot 2}{1000 \cdot 8}}$$
$$= 0.00166$$

- Using the non-conservative prediction equation in Figure 7;

IFF = 3x10^{11}x$0.00166^{2.7916}$
= **5200 years** (1/AEP)

2. As the non-conservative approach does not meet the standard of 1 in 10,000 AEP, then there is no point in checking for the conservative approach, as the flood capability will only be worse.

3. Overall Assessment: As 5200 < 10,000, the dam is in need of remedial action!

Design Mode Worked Example

Case Description: Mr Jones, the owner of the dam in the above case, would like to know the amount by which he must increase the size of his spillway in order to make the dam of adequate flood capability standard? However, he must be left with a full storage capacity of at least 190 ML in order to meet his annual farming needs, and he would also like to avoid the option of raising the entire non-overflow crest.

Case Solution: A new spillway can be designed as follows:

1. In the review of this dam, 1 in 10,000 AEP for the Recommended Design Flood (RDF) was used as the minimum standard and was compared to the non-cons. predicted flood capability. For design, as lives are at risk downstream, best to adopt the conservative approach:

- Thus, can determine the 'required' cons. RRCR to meet this standard by using the appropriate equation in *Figure 7* in reverse;

for 10,000 = 4x$10^{11}$$RRCR^{2.9364}$,
RRCR = 0.00259

2. Can now design the new spillway for RRCR = 0.00259 by using the cons. RRCR equation in reverse:

- As the height of the spillway (SH) cannot be increased by raising the embankment, extra spillway capacity can only be obtained by widening the spillway (either the existing one or a new secondary one) and/or deepening its base. However, the amount by which the bottom of the spillway can be deepened is restricted by the farmer's storage capacity requirement. Therefore, need to determine the maximum depth that the spillway can be dug out without loosing excessive storage capacity. Equation 8 can be used for this purpose by 'trial and error' as follows:
  - Try increasing spillway depth by 0.2m, ie: SH = 2.2m. Therefore, the maximum reservoir depth reduces from 10m to 9.8m.
  - Substituting necessary parameters into *Equation 8*, the new storage capacity of the reservoir (RC) would become;

$$RC = \tfrac{1}{2}\left[\frac{0.048\times10^6\times9.8^2}{2\times10}+\frac{0.048\times10^6\times9.8^3}{3\times10^2}\right]$$
$$= 190.5 \text{ ML}$$

  - As 190.5 > 190, increasing the depth of the spillway by 0.2m is just acceptable. Therefore, can work with SH = 2.2m new maximum spillway depth.
- Also require a new reservoir area at FSL (RA). This can be determined using *Equation 7* in reverse with a new maximum reservoir depth of 9.8m;

for 190.5x10^3 = 0.415x(RA)x9.8
RA = 0.0468 km^2

- Therefore, substituting all necessary parameters into the cons. RRCR equation and applying it in reverse;

for
$$0.00259 = \frac{SC}{78.411 \cdot 8^{0.8123}} \cdot \sqrt{\frac{\sqrt{0.0468} \cdot 2.2}{1000 \cdot 8}}$$

SC = 142.6 m^3/s

- The spillway width (SW) required to provide this spillway capacity for a 2.2m maximum depth is determined using the broad-crested rectangular weir equation (presented above under Review Mode Worked Example) in reverse;

for 142.6 = 1.69xSWx$2.2^{1.5}$
SW = 25.9m

- Therefore, as the spillway width is already 10m, it must increased by 15.9m, for a 0.2m increase in depth.

3. Overall Assessment: The size of the spillway must be increased from 2m deep x 10m wide to **2.2m deep x 25.9m wide** in order to satisfy the ANCOLD (1986) flood capability standard.

Note: If the use of Equations 7 and 8 is to be avoided, then alternatively the dam owner can maintain the original storage capacity of the reservoir, and an increase in spillway width can be determined for the original 2m high spillway.