SOUTH AFRICAN NATIONAL COMMITTEE ON LARGE DAMS

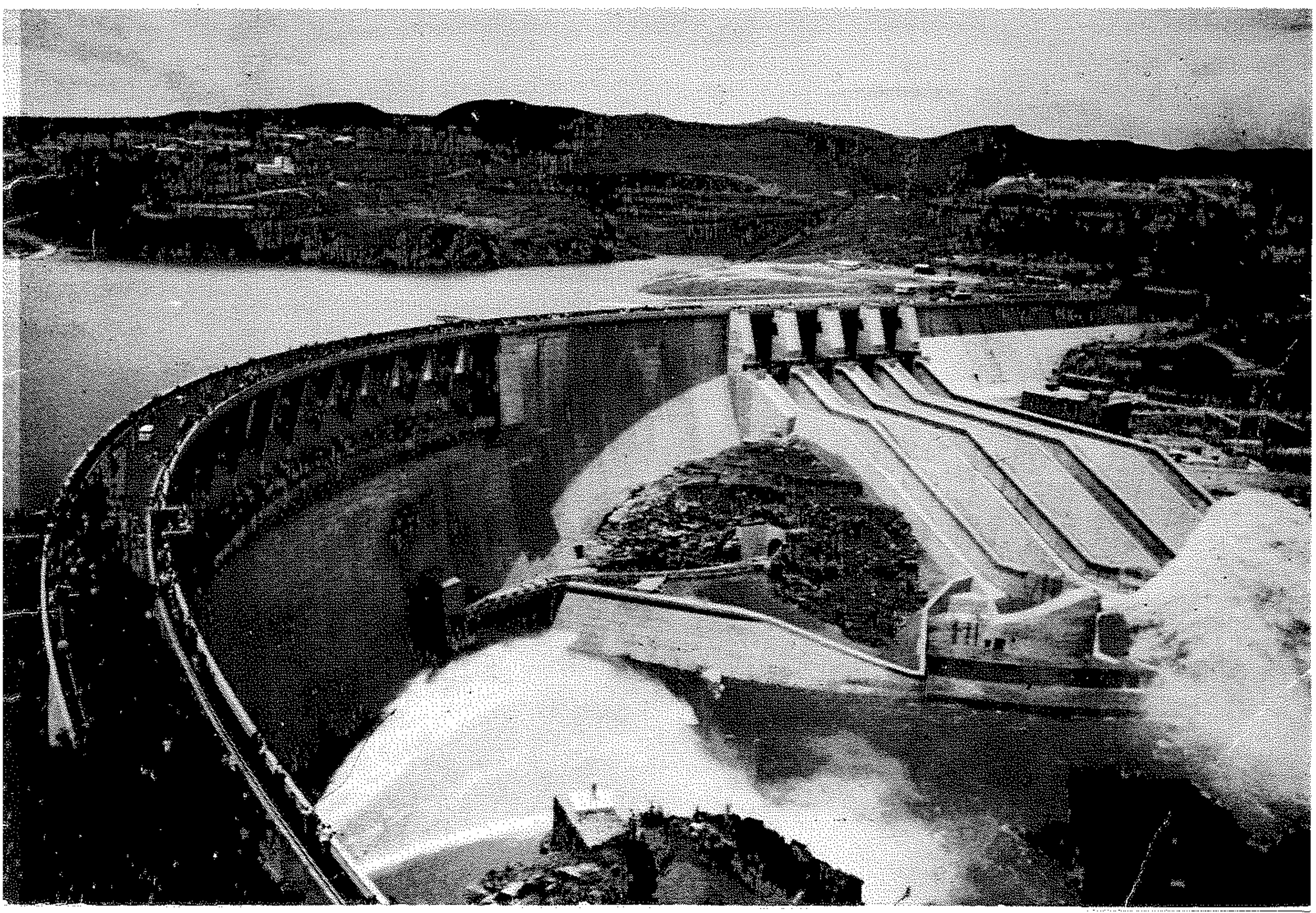
CIGB
ICOLD
SANCOLD

Safety Evaluation of Dams

Report No. 4

Guidelines on

Safety in Relation to Floods



PREFACE

Dam Safety legislation for South Africa was promulgated in 1986. Engineering design standards for dams, however, are not prescribed in the legislation and there exist no relevant South African codes of practice. Aware of the necessity to provide guidance not only to those charged with evaluating the safety of existing dams but to some extent also to the designers of new dams, SANCOLD issued Interim Guidelines in 1986.

That publication served to guide both the authority administering the legislation and the practising engineer towards selection of appropriate criteria for evaluation of spillway capacity for existing dams as well as for new dams. In a country having scarce financial resources it is reasonable to adopt generalised design criteria for the vast majority of cases. However, when dealing with important dams, it is essential to move towards decision-making based on site-specific analyses. This was indeed emphasised in the Interim Guidelines of 1986.

The 1986 document was not without shortcomings. In response to our invitation, several practitioners commented on a number of practical aspects and suggested many valuable improvements. The Interim Guidelines have now been revised, taking cognisance of the comments and the fact that other SANCOLD publications have since been released, viz. a Handbook on Flood Hydrology for southern Africa, Guidelines on Freeboard, Analysis of Dam Break Floods and Risk Analysis for Dams — A Review. The publications in the series are intended to complement one another.

Attention is drawn to the fact that the Safety Evaluation Flood criteria were revised subsequent to the publication of the Hydrology Handbook. The necessary alterations should therefore be effected to the tables in Chapter 12.

This revised edition of the Guidelines on Safety in relation to Floods takes into account not only current practices followed in other countries but also the latest available knowledge of hydrological conditions in southern Africa. No doubt as more information comes to light, however, further revisions will follow. It should be stressed that the Guidelines can in no way be regarded as a substitute for intelligent engineering judgement.

Chairman : SANCOLD

ACRONYMS

(with definitions in order of appearance in the text)

RDD : Recommended Design Discharge

The Recommended Design Discharge (RDD) is the level pool peak discharge which has the relevant value shown in Table 5.1 and provides the preliminary basis for checking the design of the spillway system for a new or an existing dam. (The spillway system must accommodate the RDD without damage. The requirement is conditional, however, in the case of an existing dam. See Section 3.8).

SED : Safety Evaluation Discharge

The Safety Evaluation Discharge (SED) is the level pool peak discharge which has the relevant value recommended in Table 5.2 and provides the initial screen for checking the adequacy of the spillway system of a new or an existing dam under extreme flood conditions. (Although substantial damage may result from occurrence of the SED, the design must be such that the dam will not fail).

RMF : Regional Maximum Flood

The RMF is defined in Report TR 137 Department of Water Affairs (1988) and provides the basis for calculating discharges appropriate to the application of generalized design criteria.

RDF : Recommended Design Flood

When site-specific hydrological calculations are required the term *flood* (implying a hydrograph) is used rather than *discharge* (implying unrouted level pool outflow possibly requiring to be routed). The RDF is a single flood hydrograph or a family of hydrographs having return periods suggested in Table 5.4 and must, after routing, be accommodated by the spillway system without damage. (Again, in the case of an existing dam the requirement is conditional. See Section 3.8).

SEF : Safety Evaluation Flood

Again, where site-specific hydrological calculations are required, the SEF is the flood hydrograph which after routing through the reservoir system may bring the dam to the point of failure but the resulting damage, although possibly substantial, must not be such as to cause failure of the dam. The RMF-based values given in Table 5.2 provide guidance to choice of the peak of the hydrograph for the relevant size and hazard rating but independent methods must be employed to verify whether or not the RMF basis leads to acceptably realistic flood values.

PMP : Probable Maximum Precipitation

The PMP is the precipitation having close-to-zero exceedance probability for given duration and provides the basis for estimation of the PMF.

PMF : Probable Maximum Flood

The PMF is a single-event flood having close-to-zero probability of being exceeded in either volume or peak, as determined by the methods explained in Report No. 1/72, Hydrological Research Unit (1979) or the SANCOLD handbook Flood Hydrology for southern Africa (Alexander, 1990).

IFF : Imminent Failure Flood

The IFF is the inflow flood which, when routed through the reservoir and spillway system, can be passed by the spillway with the reservoir level just threatening failure of the dam. See Section 5.3.

ZIF : Zero Incremental Impact Flood

The ZIF is the flood which would just cause failure of the dam by overtopping but would not be such as to cause significant increase in the downstream damages and potential loss of life, ie. zero incremental impact under present and foreseeable future conditions. See Section 5.3.

Note: *Size classification* of dams according to the Regulations to the Water Act is given in Table 2.1.

Hazard classification is given in Table 2.2.

Categorization of dams according to safety risk is given in Table 2.3.

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COVER: P K le Roux Dam
(Photo : WCS Legge)

1 INTRODUCTION

A dam is normally intended to meet a social need and to provide some benefit - but at a price, in terms of both money and possible adverse environmental impact. The benefit may take the form of assurance of water supply for whatever purpose while the price would be made up of (a) the prime cost of building the dam and (b) the risk cost associated with the damage that would be incurred should the dam fail in its function - whether through inadequacy of storage provision or of spillway capacity, or through structural failure. Environmental impact and associated compensation would be included in the prime cost. For a given water resources situation, the bigger the impoundment relative to the demand the smaller would be the risk of failure of supply; in relation to flood, the bigger the capacity of the spillway system the smaller the risk of failure of the dam by overtopping. Thus, with increasing size, there are rising prime costs accompanied by declining risk costs. Since the two components of cost are additive, there must be a minimum and therefore an economic optimum size (eg. of spillway capacity). It follows that sizing can be optimised by means of risk analysis. Where significant danger to human life is involved, however, the inclination is to reduce the probability of dam failure to negligible levels. In appropriate circumstances, generalised criteria of design can with advantage be adopted in lieu of economic or other risk analyses.

It is the risk associated with failure of a dam through inadequacy of the spillway system that is the concern of these Guidelines. It bears emphasizing, though, that inadequacy of spillway capacity is not the only potential cause of failure of a dam; there are others, eg. geologic, seismic and structural. The Guidelines aim to facilitate determination of flood values for purposes of dam design such that the risk of failure through inadequacy of the spillway system may be kept to acceptable levels.

Legislation was promulgated in 1986 to promote safety of dams in South Africa. Section 9C of the Water Act, as amended by Act 96 of 1984, grants wide powers to the Minister of Water Affairs to control the design, construction, operation, alteration or abandonment of "a dam with a safety risk" which, in general, is a dam of height over 5 m and impounding more than 50 000 m^3.

In terms of the Regulations to the Act all dams having a "safety risk" have to be registered and those in Categories II and III must be checked for safety by an approved professional engineer. Construction, alteration or abandonment of a "safety risk" dam is subject to permit from the Minister and dams in certain categories must be designed by an approved professional engineer. Moreover Category II and III dams have to be regularly inspected by an approved professional engineer and must be operated and maintained in accordance with the conditions and requirements laid down in the relevant permit.

The South African Committee on Large Dams (SANCOLD) has considered it necessary to offer guidelines to engineers charged with the task of evaluating the safety hazards of existing dams. To this end, SANCOLD in 1985 established a subcommittee to prepare the Guidelines.

The Committee comprised five engineers

Chairman:	W S Croucamp
Members:	W C S Legge, D C Midgley, H N F Pells, A Rooseboom.

As a result of the Committee's work the SANCOLD Interim Guidelines on Floods in relation to Dams were published in September 1986.

During the past four years the SANCOLD handbook on Flood Hydrology for Southern Africa, authored by Alexander (1990), was finalised. Other Sub-Committees of SANCOLD have completed the tasks of preparing guidelines on other topics relevant to the selection of design floods, viz. freeboard, dam break analysis and risk analysis for dams. Some of the information included under the foregoing headings in the Interim

Guidelines of 1986 has now been dealt with extensively in publications based on the work of those Sub-Committees and has therefore been deleted from the revised Guidelines on Floods.

In the four years since promulgation of the dam safety legislation a clearer picture of the distribution and classification of dams in the RSA has emerged. An analysis of the features of dams falling within the ambit of the legislation has revealed that:

- 89% have wall heights less than 20 m and 55% have wall heights less than 10 m

- 46% are Category I dams (as defined in Section 2.2)

- 31% of the "small" dams are Category II

- 85% of all registered dams fall within the SMALL to MEDIUM size class with LOW or SIGNIFICANT potential hazard ratings.

The foregoing information has had an important bearing on the reframing of the Guidelines. On the strength of this analysis along with the invited comments on the Interim Guidelines, a revised draft was ready by September 1990 and distributed to a limited number of interested persons. There followed a well-attended symposium titled

Dam Safety Four Years On (SANCOLD 1990), after which several leading practitioners were invited to offer further comment. In the light of the widely canvassed opinions, parts of the text have been completely altered and parts that were duplicated in other SANCOLD publications have now been deleted.

The Committee is deeply indebted to those who have given valuable assistance and comments, in particular to Prof WJR Alexander, RGK Blyth, Dr A Görgens, Z Kovacs and Dr MJ Shand.

The Committee has concerned itself primarily with the sizing of the spillway systems of "safety risk" dams. Guidance is intended mainly to help engineers charged with evaluating the safety hazards of existing dams and determining the extent to which betterment measures, if any, are desirable or essential. In view of the possibility that a proposed dam will in due course become an existing dam, the Guidelines must be applicable also for checking the design of a new dam.

The procedure recommended for safety evaluation is illustrated in the Flow Diagram, Fig 1.1. A primary feature of the checking philosophy, stems from the results of the previously referred to analysis, namely, that generalised standards could probably serve to deal with safety evaluation of most dams, and it followed that a streamlined procedure could probably be designed to screen out those requiring detailed treatment.

In brief, as may be noted, the first step is to check whether the Recommended Design Discharge (RDD) can be accommodated by the spillway with adequate freeboard. If not, a distinction is drawn between an existing and a new dam. Checking follows different paths for existing and new dams to the point where the spillway has to be checked for coping with the Safety Evaluation Discharge (SED). If it can cope and if it is an existing dam, approval can be recommended without further ado. A new dam of significant size and risk, however, requires intensified study. In either event, if the spillway fails the SED test site-specific hydrological calculations are needed, of which there are three alternative types. If none can be satisfied the recommendation would be that the spillway should be redesigned. As will be seen, when the problem becomes site specific, attention is usually widened from peak discharge to flood hydrograph and reference is then to RDF and SEF (F referring to flood).

The procedure is detailed in Chapter 5 in which the terms used in Fig 1.1 are defined and evaluated.

In Chapter 2, categorisation of dams in accordance with the Regulations is explained. There follows a major chapter (Chapter 3) on general considerations concerning the safe passage of floods, including legal aspects and philosophy of risk. In Chapter 4 appropriate methods of deriving flood peak-frequency relationships and flood hydrographs are outlined as well as the routing of floods.

In Chapter 5, ways of selecting the flood values for safety evaluation are explained. The procedure starts from generalised design criteria, with recommended discharge values for spillway design and safety evaluation. The discharge values are derived from Regional Maximum Flood (RMF) values given in Report No. TR 137 (Department of Water Affairs, 1988). The recommended values for RDD and SED given in Tables 5.1 and 5.2 are readily repeatable and constitute conservative but relatively coarse screens through which many existing dams will pass, leaving only those caught on the screen to be given detailed scrutiny. As explained in Chapter 5, and illustrated by the Flow Diagram Fig 1.1, detailed scrutiny involves site-specific hydrological calculations to establish a series of design hydrographs which would be routed through the reservoir/spillway system.

The SANCOLD handbook (Alexander 1990) provides guidance for the performance of hydrological calculations. The methods presented in the handbook reflect sound and desirable hydrological practice. Where conditions at a specific site require departure from these methods, reasons for departure should be detailed. It should be noted that changes to the criteria for safety evaluation were made subsequent to the publication of the SANCOLD Handbook of Hydrology by Alexander. The reader should therefore effect the necessary changes in Chapter 12 of the Handbook to conform with Chapter 5 of this Guide.

Chapter 5 deals also with the site-specific approaches, zero incremental impact and risk-based or hydro-economic analyses.

> *For the design of new dams having significant or high potential hazard rating, it is essential that practitioners do not rely solely on generalised design criteria but apply site-specific methods with the best available hydrological techniques.*

Chapter 6 contains concluding remarks.

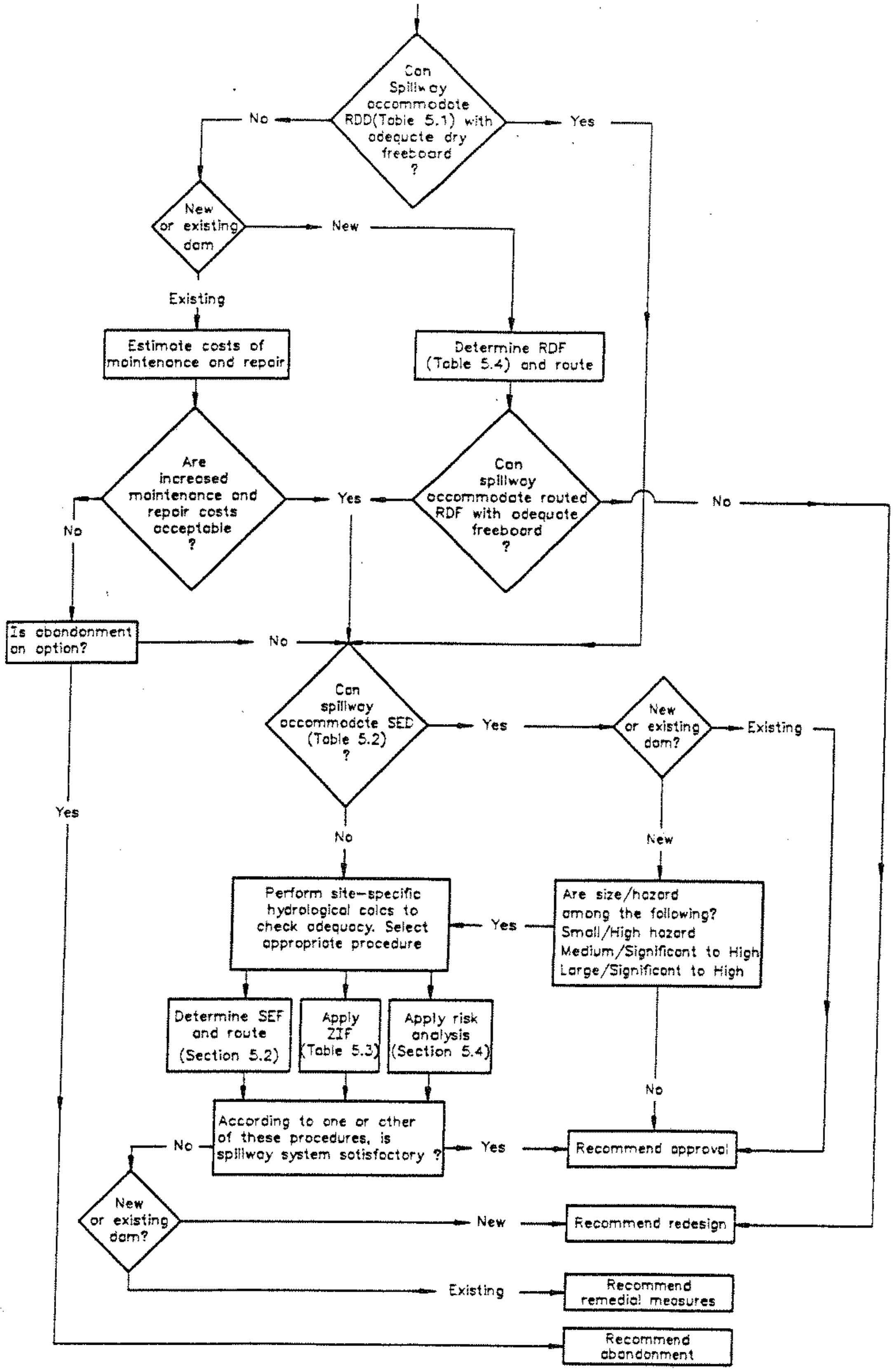

Fig 1.1 Flow diagram for dam safety evaluation

2 DAM SAFETY LEGISLATION

2.1 Classification of dams

Section 9C of the Water Act defines a "dam with a safety risk" as being:

> *"any dam having a storage capacity in excess of 50 000m^3 and a vertical height in excess of 5m, measured, in the case of a dam consisting of a structure situated across a water course, from the natural level of the bed of the water course on the downstream face of the structure, and, in the case of a dam consisting of any other structure, from the lowest elevation of the outside limit of the structure to the top of the structure which is the level of the roadway or walkway, or, in the case of a structure consisting of a spillway only, is the crest level of the spillway;* and
>
> *any other dam, or any other dam belonging to a category of dams, ..."*

(Here the Act refers to dams that the Minister may, by notice in the Government Gazette, declare to pose a threat to life or public safety.)

The Regulations published in terms of Section 9C (6) of the Act (Government Gazette No. 10366 of 25/7/86) classify dams with a safety risk according to size as in Table 2.1.

Size class	Maximum wall height (m)
Small	More than 5 and less than 12
Medium	Equal to or more than 12 but less than 30
Large	Equal to or more than 30

Table 2.1 Size classification

It should be noted that the Act refers also to dams other than those impounding flow in streams or rivers. This category would comprise such structures as off-channel storage reservoirs and slimes or tailings dams, which could be overtopped as a result of extreme direct precipitation, as well as large stormwater detention dams and any other impounding structures catastrophic failure of which might be deemed to present a hazard.

The Regulations also provide for classification of dams according to the potential loss of life and property that might result from failure. The hazard rating of a dam is determined by separate consideration of potential loss of life and potential economic loss, as given in Table 2.2, and the factor given the highest rating is decisive. The hazard rating is a qualitative indication of the potential loss that would result from a "sunny day" failure, ie. not necessarily associated with a natural flood entering the reservoir.

Hazard rating	Potential loss of life	Potential economic loss*
Low	None	Minimal
Significant	Not more than 10 lives	Significant
High	More than 10 lives	Great

Table 2.2 Hazard classification

* For classification purposes potential economic losses less than R2 million and more than R20 million (1991) would be considered "minimal" and "great" respectively.

2.2 Categorisation of dams

For purposes of prescribing the degree of attention required as regards design, construction, commissioning, operation and maintenance of dams having a "safety risk", dams are categorised in terms of the Regulations according to Table 2.3.

Size class	Hazard rating		
	Low	Significant	High
Small	I	II	II
Medium	II	II	III
Large	III	III	III

Table 2.3 Categorisation of dams having a safety risk

2.3 Requirements in respect of spillways

The Regulations require that the information in respect of spillways be submitted as part of the application for a permit to construct or to impound. The required information varies from an essentially geometric description in the case of a Category I dam to a disclosure of the calculation methods and criteria adopted as well as the results of calculations in the case of a Category III dam. It is further required that the adequacy of the spillway should be evaluated in each safety inspection report on a Category II or Category III dam.

As indicated earlier, criteria for deciding on the spillway adequacy are not prescribed in the Regulations. The underlying principle embodied in the Regulations to the Act is that the engineer should apply accepted current practice, taking into account site-specific conditions. In these Guidelines, the endeavour is to assist the practising engineer in the process of deciding upon the appropriate flood magnitude to adopt for sizing the spillway of a new dam or for evaluating the adequacy of the spillway system of an existing dam.

3 SAFE PASSAGE OF FLOODS: GENERAL CONSIDERATIONS

3.1 Function of the spillway system

In these Guidelines the term "spillway system" is adopted so as to include all components of the dam that are designed to pass flood waters downstream, viz. spillway, discharge carrier, energy dissipators, outlets, crest gates as well as auxiliary spillway, fuse-plug or other emergency works.

The primary purpose of the spillway system is to discharge excess inflow downstream in such a way as to prevent failure of the dam. An extraordinarily large natural incoming flood, when discharged by the spillway, may well cause downstream damage to much the same extent as would have occurred had there been no dam. Failure of the dam in such circumstances, however, would generally cause inundation and damage far more extensive than would have been experienced in the absence of the dam. On the other hand, there may well be circumstances in which failure of a dam under extreme flood may cause no more damage than would in any event have occurred. This provides an approach to safety evaluation dealt with in Section 5.3. Malfunction and non-optimal operation of spillway systems are dealt with in Sections 3.5 and 3.6.

3.2 Legal considerations

From a legal point of view, the main question following damage is whether the engineering judgement exercised reflects reasonable care and prudence. Terms that frequently crop up in the courts are: "reasonable precautions to prevent injury", "a reasonable exercise of power given (to prevent damage)", "neglecting to take adequate precaution" and "reasonable care to see that unnecessary damage (is not caused)".

Two factors that are decisive in considering reasonableness are cost and the level of safety provided.

With reference to the cost factor a South African Appeal Court judge remarked (Breede River Irrigation Board v Brink 1936 AD 359 and 366):

> *"In considering whether a measure is reasonable or practicable, regard may be had to local requirements, and to the financial resources of the public body, and to the cost of taking such a measure of precaution against injury, and when it is in the public interest*

that works should be constructed, they should not be made impossible by prohibitively expensive protective measures".

3.2.1 Reasonableness of cost

It thus follows that a public body is legally obliged to provide a structure to suit the circumstances, to do this within its financial resources and thus to reduce costs plus possible damages to reasonable proportions.

3.2.2 Reasonableness of risk

With reference to the required level of safety during floods the outcome of the following court case provides guidance. In New Heriot Gold Mining Co. v Union Government (1916 AD, 415 to 438) Justice Innes remarked:

"In a country where rainfalls of great volume and severe intensity are common, and where meteorological data are scanty, I think that those on whom a duty in favour of others is cast to deal with flood water should be expected to provide a considerable margin of safety. A defence of vis major (or 'act of God') should not be upheld save on the clearest evidence".

The question to be addressed by every spillway designer or safety evaluator remains: "What constitutes reasonable care and prudence in selecting the magnitude of the flood for which a dam should be designed or checked for safety?"

3.3 What level of risk is reasonable?

3.3.1 A discussion of risk

The problem faced by designers of dams, and by the public who use, pay for, and are affected by these structures, is to decide just how much protection should be provided.

It is not feasible nor even possible to provide absolute safety against all natural hazards. The objective should be to balance the benefits of providing a dam to meet a given need against the cost of increasing its safety beyond that associated with a reasonable level of risk. One must recognise that this does not mean trying to eliminate all risks but rather reducing them to acceptable levels.

Objectives either for dam design or for safety evaluation (from the point of view of extreme inflow) can be considered under two broad categories, namely (1) those relating to cost effectiveness and (2) those relating to equity. Cost effectiveness objectives seek to maximise the excess of project benefits over project costs. Equity objectives on the other hand seek an appropriate balance among the competing interests of such parties as: the dam owner, those who benefit from it and those who would be harmed should it fail. Since the timing and the magnitude of future floods are unpredictable, direct determination of optimum measures to attain the economic objectives is not possible and one must fall back on probabilistic methods. For the same reason simple answers to problems of equity among those affected by a dam are not readily attainable.

Application of the equity principle can lead, however, to an approach whereby acceptable levels of risk associated with the presence of a dam can be prescribed.

Bogey (1981) suggests that risks imposed on individuals can be related to the background levels of risk experienced as part of everyday life. For a particular community this background or ambient level of risk can be estimated from mortality statistics. Table 3.1 presents mortality statistics (originally produced by Cox) for accidents in the USA, UK, France, Belgium and the Netherlands (Bogey, 1981).

Cause of death	**Per million persons per year**
All causes (including natural)	11 000
Accidents: All	460
Motor car	240
Falling	130
Fire	28
Drowning	24
Poisoning by other substances	15
Air	6,5
Water transport	5,3
Railway	4,3
Natural and environmental factors	3,8
Poisoning by medical drugs, etc	3,5
Lightning	0,9

Table 3.1 Mortality statistics for several Western countries

From the risks considered in Table 3.1, it is possible to identify several subgroups.

a) Risks of an everyday kind without any compensating benefits, which are acceptable principally because they are fundamentally unavoidable and immutable, such as falling, fire and natural and environmental factors (including lightning strikes) and poisoning.

b) Risks which are in principle avoidable but provide direct compensation benefits and in practice are unavoidable for people who wish to be part of modern society, eg. all transport accidents and poisoning by medication.

c) Risks which are truly avoidable in the sense that people who expose themselves do so of their free will in order to gain some other benefit, eg. most of the drowning cases and some of the motor accident cases.

From his study of risk, Oosthuizen (1986) concluded that the reasonableness of risk depends on several factors such as:

- a combination of the likelihood and magnitude of the potential hazard (economic, human, socio-economic, political etc.)
- perception of the hazard
- involvement (apart from the voluntary vis-à-vis involuntary aspects)
- exposure to the hazard
- comparability with other risks
- benefits associated with the risk

It is thus clear that all these factors have to be taken into account to determine the reasonableness of risk under specific conditions.

3.3.2 Criteria for the acceptability of risk

For the population living below a dam, the risk is perceived as one which is essentially unavoidable (involuntary) and from which they do not necessarily receive a direct benefit. It is therefore comparable with the risk in category (a) in Section 3.3.1, except that the dam is a man-made hazard rather than a natural one and it is therefore reasonable to expect the man-made risk to be sufficiently low that it does not make a significant difference to the pre-existing comparable natural risk.

Several criteria for the acceptability (or otherwise) of risk have been proposed or are in use, for example:

- exposure, such as lower than one fatality per annum per million persons exposed
- cost per life saved (compared with similar public sector programmes)
- graphs combining probability of occurrence and potential hazard.

For example, in the nuclear context, the expected probability of death for any individual throughout the population shall not exceed a value of 1 x 10^{-6} per calendar year.

It must be stressed, however, that views on the acceptability of risk differ among various communities and what may be acceptable for one may be totally rejected by another.

It seems that acceptability criteria for risks associated with dam failure still require considerable thought. The SANCOLD (1990) review of Risk Analysis for Dams addresses this issue.

3.3.3 Occurrence of an event within a time span

It is well to recall here the levels of probability that an undesirable event may occur within the design life of a project.

The probability that a flood of return period T years will be equalled or exceeded at least once within the next L years (eg. the life L of a project or the loan period) is given approximately by the expression:

$$p_T = 1 - \left(1 - \frac{1}{T}\right)^L \tag{3.1}$$

It is important to appreciate that the probability p_T that a T-year flood will be equalled or exceeded within the first T years is given by:

$$p_T = 1 - \left(1 - \frac{1}{T}\right)^T \tag{3.2}$$

Which, if T is large, approaches:

$$1 - e^{-1}, \quad i.e.\ 1 - 0{,}37 = 0{,}63 \tag{3.3}$$

Thus, if the construction cost of a dam is expected to be paid off over a period of say 20 years and if the spillway were to be designed to cope with only the 20-year flood, there would be a more than even chance (63 per cent) that it would be surcharged before the dam had been paid for. Indeed, if the spillway were designed to accommodate the 100-year flood without freeboard, there would still be an appreciable probability (20 per cent) that the dam would be overtopped before the 20-year payment period had expired; in fact, more than a 2 per cent chance that this would happen twice!

Fig.3.1 illustrates the probability that events of various return periods will be exceeded once, twice of three times within given design periods.

It is evident from the diagram that lowering of the tolerable risk rapidly increases the return period of the design flood and it is for this reason that one endeavours to establish approximate values for the low probability tail of the frequency distribution curve.

3.4 Ways of accounting for risk

The goal of dam safety practice is to limit the risks of dam failure to acceptable levels. Probability of failure is controlled partly by design criteria and partly by quality of construction, inspection, operation and maintenance. Ideally, acceptable levels of hazard, of damage and of failure probability should be quantified for the conditions prevailing at each existing or proposed dam. The ideal, however, cannot always be readily achieved and the required computational effort may not always be warranted. We deal here with four alternative ways of evaluating the safety of dams:

- use of generalised design criteria
- site-specific hydrological calculations
- the zero incremental impact approach and
- optimisation employing risk-based analysis.

The procedures are outlined in general terms in what follows and explained in more detail in Chapter 5.

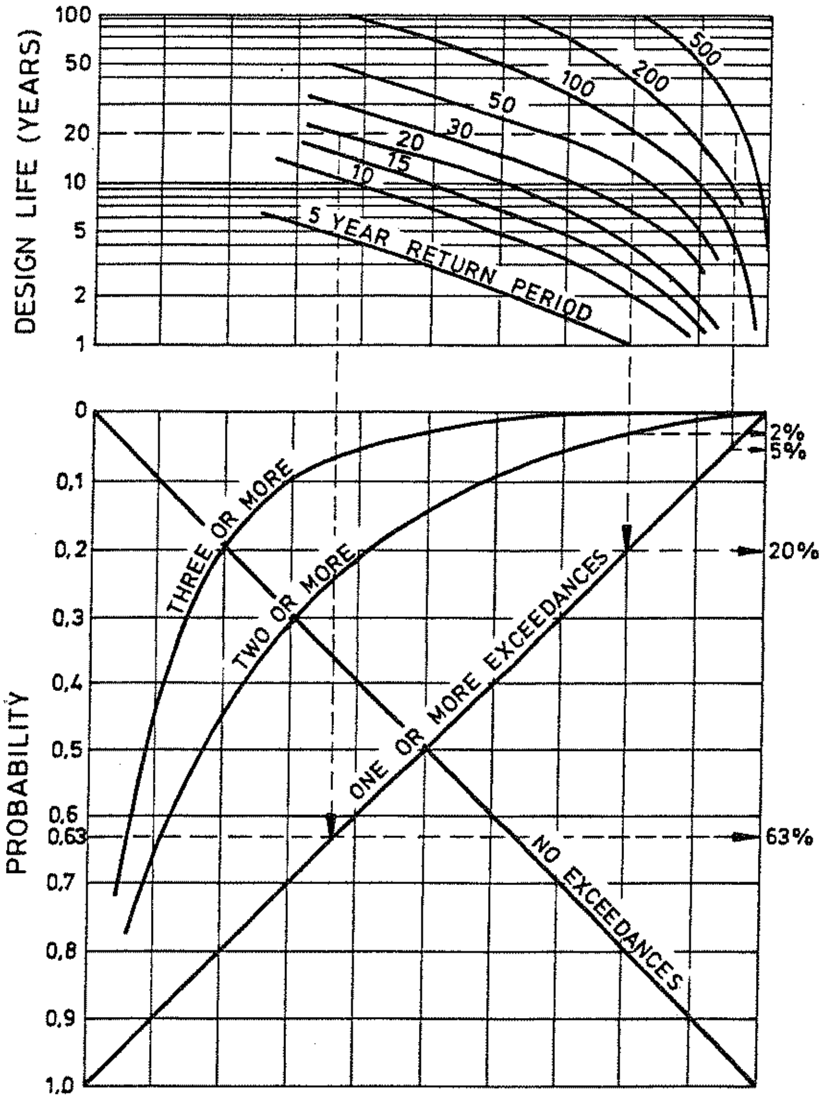

Figure 3.1 Probability of exceedance within a given design period

3.4.1 Use of generalised design criteria

The most widespread current practice is to classify dams according to the potential hazards they present and to assign one of a number of grades or ranges of design criteria depending on height, storage capacity and hazard rating. Generalisation of design criteria is an attempt to ensure reasonable care by acknowledging that the level of protection provided should reflect consideration of the hazard potential, eg. loss of human life, loss of service of the dam, consequential property damage, and future benefits foregone in the event of failure of the dam. However, the procedure treats all the elements needed for selecting design criteria in a generalised way with the result that the appropriateness of the design criteria to a particular dam may become uncertain.

In these Guidelines the generalised criteria chosen are specified as flood peak *discharges* rather than *floods* (implying hydrographs). Thus, instead of recommended design flood (RDF) and safety evaluation flood (SEF), the generalised criteria refer to recommended design discharge (RDD) and safety evaluation discharge (SED). The discharge values are derived in the first instance from experience envelopes, namely Regional Maximu m Flood (RMF) peaks which can be readily evaluated from Technical Report No. TR 137 (Department of Water Affairs, 1988). Use of generalised criteria stems primarily from the need for regulating bodies to exercise control as far as possible uniformly over the many dams under their jurisdiction.

3.4.2 Site-specific hydrological calculations

For checking the designs of new dams in Category III and most new dams in Category II, as well as existing dams caught on the safety evaluation screen referred to in the Introduction, it is necessary to perform hydrological calculations appropriate to the site. The main difference from the procedures based on generalised criteria referred to in Section 3.4.1 is the need to determine more accurately the levels to which the water in the reservoir will rise at the spillway when the incoming flood is routed. This involves generation of hydrographs, not merely discharges. Methods are explained in the SANCOLD handbook (Alexander, 1990) and enlarged upon in Chapter 4.

3.4.3 Zero incremental impact approach

In assessing the damage likely to result during an extreme flood, the location of development relative to the expected flood lines is decisive. For a dam impounding a small volume of water, it is possible that failure during an extreme flood would cause little or no additional damage or loss of life compared with conditions during the same extreme flood without dam failure. Similarly, in a situation where all downstream development has taken place in low-lying areas that would in any event be inundated and damaged by a natural flood, failure of even a large impoundment may well cause minimal additional downstream damage.

In the zero incremental impact approach, repetitive computations have to be performed to find the minimum spillway capacity such that all significant downstream flood damages from flood releases and other sources would in any event have occurred even had the dam not failed by overtopping. The sizing of spillway capacity is therefore determined neither on the basis of generalised design criteria nor on purely economic grounds but by assessment of the incremental impact associated with different floods. The approach requires the evaluation of the imminent failure flood (IFF), imminent failure inundation and resulting damage, post-failure inundation, incremental impact and finally the zero incremental impact flood, all as explained in Section 5.3. If, for a particular alternative under investigation, the zero incremental flood exceeds the imminent failure flood, or if the consequences of failure are unacceptable, other alternatives must be investigated to develop a further basis for decision.

The zero incremental impact approach is allowed by the US Bureau of Reclamation (1981) and by several states of the USA as an alternative way of selecting the design flood. It is also recommended in a guideline prepared by an Inter-agency Committee on Dam Safety in the USA (ICODS, 1983). The Committee for Safety Criteria for Dams (1985) recommended its use for the safety evaluation of high hazard dams in the USA. An advantage of the method is that it circumvents the necessity in the analysis to assign a monetary value to human life.

3.4.4 Risk-based analysis

The object of risk-based analysis is to determine for several alternative designs the total cost, which includes the initial capital cost as well as the consequential cost arising from dam failure, and thus to identify the option which yields the least total cost. The method, which is

elaborated in Section 5.4, requires floods of various sizes to be estimated, having return periods well beyond the length of available records so as to permit flood frequency to be converted to damage frequency in order to yield damage costs. A procedure for extending flood frequency relationships is explained in Section 4.2. Estimates are required, too, of the consequences of dam failure, such as expected damages, estimated value of future benefits foregone and loss of life. Because of uncertainties in regard to failure modes, flood behaviour, future developments and economic parameters, evaluation of total flood damage is generally difficult.

Where no loss of life is expected, as would be the case for low hazard dams, the design alternative that yields the least total cost would be selected. The decision would thus be based on economic considerations. However, when there is possible loss of life, it becomes difficult to establish criteria for decision making. Several authors have proposed ways of assigning a monetary value to human life so as to allow decisions to be based on economic considerations but the approach is generally viewed with disfavour. According to Oosthuizen (1986) the current attitude is to separate the risks associated with human losses from those incurring monetary losses. Separation of risks would require adoption of acceptable risk levels for dams in the significant and high hazard categories. The SANCOLD (1990) publication on Risk Analysis for Dams reviews current practices in this regard.

3.5 Selection of type of spillway and flood outlets

Spillways and flood outlets are selected on the basis of both dam safety requirements and project economics. Selection is influenced by the topography, geology and hydrology at the dam site as well as by type of dam and other project factors, such as expected reliability of maintenance and operation. The performance standards demanded should be higher for spillways expected to operate frequently than for those expected to come into use only on rare occasions. Flood releases can be accomplished by spillways as well as flood outlets but the contribution to flood release through flood outlets and turbines should be taken into account only when dependability of operation during flood can be assured. Two general types of spillways are distinguished, viz. service spillways and auxiliary spillways.

- **Service spillways**, when complemented by auxiliary spillway, should be designed to handle relatively frequent and sustained flood releases up to, say, the 1:100 year event without incurring significant erosion and related damage. However, in many cases, the service spillway is the sole spillway of the dam. Ungated service spillways are generally favoured over gated spillways because of greater reliability. Because of their operational flexibility and large discharge potential, gated spillways are often essential to satisfy project requirements. Gated spillways can be used to advantage to attenuate downstream flood peaks, but only where there is sufficient warning of incoming floods to allow significant pre-releases to be effected. For effective attenuation the minimum warning needed is of the order of 24 hours.

  Despite the considerable advantages of gated spillways when maintained and operated under skilled supervision, it must be assumed in design that gates and automatic spillways will not always function or be operated as intended. Partial blockages of controlled spillways can, and are indeed likely to, occur. The degree of allowance for functional failure demands sound experience-based engineering judgement.

  A spillway system embodying floodgates should in normal circumstances be complemented by an auxiliary spillway. Despite provision of adequate back-up to cope with power outage or mechanical breakdown, the structure should be capable of satisfactorily accommodating the safety evaluation flood when at least one gate or 15% of the gates, whichever means the higher proportion of the total, cannot for one or other reason be opened.

- An auxiliary spillway is used in combination with the service spillway and sometimes in combination with flood outlets without a service spillway. Auxiliary spillways may be designed to lower standards than those for service spillways. They offer protection to the structure where there is a chance, however remote, that non-optimum operation or mechanical or electrical malfunction of gated spillways would have serious consequences. If the auxiliary spillway takes the form of a fuse plug, care should be exercised in the design to ensure timeous and controlled release without creating downstream flow conditions more severe than would have occurred without the dam. Fuse plug spillways should not come into operation prematurely nor with catastrophic suddenness.

It is incumbent upon the designer to adopt arrangements that will be sure to function when called upon to do so. It would be dangerous, for instance, for a fuse plug to withstand the rising flood level sufficiently long to allow an uncontrolled breach to occur elsewhere. Careful choice of erodible materials for such a structure is therefore essential.

3.6 Reservoir operation

It is important to appreciate that the discharge downstream of a dam differs from that entering the impoundment. There may be attenuation to varying degrees depending upon the reservoir operation and upon the characteristics of the reservoir basin and of the inflow hydrograph. In contrast, non-optimal operation of a gated spillway can result in a downstream flood more severe or more damaging than that entering the reservoir. This could in fact happen, for instance, where water was pre-released in anticipation of a major flood with the intention of minimising downstream damage but where the hydrometeorological forecast happened to be inaccurate.

It is well to appreciate that sudden changes of discharge — both upwards and downwards — can be extremely damaging. For example, sudden collapse of a fuse plug spillway during an already high discharge, although not as severe as dam break, may cause a steep-fronted flood wave. Conversely, too rapid shut-down of flood gates may cause sloughing of river banks with consequent loss of valuable riparian lands. Such factors must be taken into account when considering reservoir operation rules in relation to the sizing of gated spillways.

Another aspect requiring careful consideration is related to the steepness of the rising limb of inflow hydrograph (Kovacs et al, 1984). On several occasions stage measurements coupled with discharge calculations in South Africa have revealed major surges that caused greater discharge at the spillway than the contemporaneous inflow. Such surges are unrelated to wind set-up or ordinary wave action and so must be included in consideration of freeboard allowances. These are dealt with in the Interim Guidelines on Freeboard for Dams, SANCOLD (1990).

From the foregoing it follows that it is not necessarily the inflow flood having the highest peak that will impose the greatest load on the spillway capacity. A reservoir with substantial surcharge capacity can greatly reduce the outgoing discharge of a flood. The more sharply peaked the hydrograph, the greater the degree of attenuation. On the other hand, the outgoing peak of a flood that has resulted from intense precipitation following prolonged rainfall in the catchment may differ little from the inflow peak; yet both hydrographs may have the same return period.

3.7 Inundation plans

For all dams with a high hazard rating, as well as for both medium and high dams having significant hazard ratings, inundation plans should be prepared so as to facilitate the work of civil defence authorities.

The plans should reflect pre- and post-failure conditions. A Department of Water Affairs (1990) report on Guidelines for the preparation of flood contingency plans for dams in South Africa contains valuable information on this subject.

The hazard rating of a dam can in general be reduced only by acquiring, and controlling, the development of the affected downstream properties.

Preparation of inundation plans entails a dam-break analysis with estimates of the resulting flood wave and its effects in areas downstream. As the state-of-the-art is continually improving, the most up-to-date techniques should be employed. In the Guidelines on Dam Break Floods, SANCOLD (1990), various dam break models are discussed and practical calculation procedures recommended.

3.8 Modification of design criteria for existing dams

It is desirable that the safety evaluation criteria should be the same for existing dams as for new dams. When dealing with existing dams, however, there are circumstances that may require some relaxation of the criteria. Some of these are listed here.

- The option of not building the dam no longer exists and the persons and developments in the downstream area are already exposed to the risk associated with the existence of the dam.

- Removal of the dam to allay the risk associated with failure may have other disbenefits such as:

  - an increase in the frequency of flooding

  - loss of an investment for many who may have contributed to the building of the dam, and

  - deprivation of benefits, such as recreation, irrigation and water supply on which many may have become dependent.

- If it is known that a dam is soon to be abandoned assessment of the risk of failure within the shorter life span would imply that relaxed criteria would achieve the same level of safety (see Fig 3.1); the same reasoning would apply when assessing risks involved during construction of a dam or the associated river diversion works.

- To modify an existing spillway to satisfy the same safety criteria as those for a new dam would generally be much more expensive than to build the same spillway for a new dam. Therefore if, in evaluating the safety of an existing dam, it becomes apparent that the appropriate criteria for a new dam would not be satisfied, site-specific studies should be undertaken to reassess the risks involved and thus to decide on the remedial measures that would be essential for safety.

4 DERIVATION OF FLOOD MAGNITUDES

4.1 General

Where adoption of generalised design criteria would not be appropriate (see Sections 3.4.1 and 3.4.2) site-specific hydrological calculations become necessary.

To enable one to reach a decision regarding the adequacy of the spillway of a proposed dam, or of an existing dam under inspection, one needs the following:

- methods of determining the frequency distribution of flood peaks
- methods of estimating the hydrographs of major floods to facilitate determination of not only peak discharges but also volumes of flood runoff
- techniques for routing flood hydrographs through reservoirs with special consideration, where relevant, of possible surges associated with steeply rising inflow hydrographs.

As pointed out in Chapter 1, however, a wide range of designs of both new and existing dams can readily be checked for spillway adequacy without the need either to generate or to route flood hydrographs through the reservoir and spillway system.

4.2 Frequency distribution of floods

It is primarily for risk-based analyses that one requires the full spectrum of potential flood events. Methods of deriving the probability distribution of flood peaks are explained in hydrology textbooks and extensively elaborated for southern African conditions by Alexander (1990). In his Section 12.7.1, Alexander quotes fairly fully from Section 4.2 of the Interim Guidelines on Safety in relation to Floods published in September 1986. There is therefore no need to repeat the relevant parts of that section here.

Attention is drawn to a useful means of compiling the frequency distribution of flood peaks from the Regional Maximum Flood (RMF) in TR 137, viz. Section 8: Estimation of the 50-year to 200-year flood peaks from RMF. Kovacs analysed large numbers of flood events in each of his K-regions and, in Figs. 14(a), (b) & (c), he has compiled for each region graphs of the ratio Q_T/RMF versus Return Period with Area of Catchment as parameter. The results are also tabulated in his Appendices 6 and 7. These offer a quick means of compiling a first-estimate frequency distribution of flood peaks in a problem catchment and provide the means of determining the Recommended Design Discharge (RDD) values for Table 5.1.

It is possible that the resulting values may lie well above those derived by statistical or other methods. On the other hand, it is highly probable that in parts of the country extreme floods result from at least two distinct families of meteorological events having completely different frequency distributions. For example, the rivers draining the eastern escarpment are vulnerable to invasion by major cyclones (such as Domoina) the frequency distribution curves of which are much steeper than those reflecting the behaviour of ordinary seasonal floods. Although the records of major cyclonic events are sparse it can well be appreciated that two- and three-family frequency distributions will reflect high values for flood events in the 50-to 200-year range.

It must be emphasised, however, that the primary purpose of introducing generalised criteria is to provide a means of deriving repeatable values. For the same reason it has been found convenient in Chapter 5 to stipulate minimum discharge values derivable from the RMF for the Safety Evaluation Discharge (SED) screen referred to in the Introduction and compiled in Table 5.2. In both respects conservatism is therefore essential.

Where it is necessary to establish a reasonable upper limit to the peak discharge at a site, that is, a value having a probability of exceedance close to zero (what is usually referred to as the probable maximum flood, PMF) such value, where determined by methods explained in the SANCOLD handbook (Alexander, 1990), should nowhere exceed twice the RMF nor the value derived by assigning to the catchment a K-value of 6,0 in the Francou-Rodier equation. Higher values than these would exceed all reasonable expectations for Southern African conditions. If an existing dam fails to pass the SED screen, Table 5.2, full site-specific calculations must be performed and these may or may not demonstrate the stipulated generalised criteria to have been too harsh.

4.3 Flood hydrographs

When generalised criteria are found to be inappropriate, the time distribution of flood discharge of given return period (ie. the hydrograph shape) becomes of importance to the design or checking of a spillway system. It is not necessarily the inflow flood hydrograph having the highest peak that causes the greatest rise above full supply in the reservoir, thus determining the spillway design capacity for the given return period. A given volume of surcharge storage, ie. volume between full supply level (FSL) and design high flood level (HFL), can attenuate a sharp-peaked flood to a greater extent than a flood having a broad-bodied or double-peaked hydrograph.

By unitgraph methods it is possible to generate several hydrographs from storms of given return period but of increasing duration - not merely that which produces the highest peak discharge - and to route these through reservoir storage to ensure encountering the critical one that determines the required spillway capacity.

Estimation of hydrograph shape by routing of the time-area graph has also proved useful. Indeed, with minimal observed information, the instantaneous unit hydrograph (IUH) can be derived by the Clark method, explained by example in Wilson (1975). From the IUH the hydrograph resulting from any net storm input can readily be derived.

Wherever possible, it is wise to check the shape of the design hydrograph, by whatever means derived, against the shapes of several locally observed hydrographs. Valuable information is to be found in DWA documentation of major floods (DWA Reports Nos TR 116, 120, 122, 132, 139 & 142). Typical flood hydrographs, runoff as percentage of rainfall, flood volumes etc. for a large number of flood events in various parts of the country are documented in these publications.

4.4 Flood routing

Textbooks on open channel hydraulics explain techniques for both reservoir routing and channel routing. The subject matter is also dealt with in the Hydrology Handbook (Alexander 1990).

When routing extreme flood hydrographs for safety evaluation, it is normal to assume, unless reservoir operating rules dictate otherwise, that the reservoir is full upon arrival of the flood under consideration. It may be argued, however, that the reservoir could well be spilling strongly at the time of incidence of the probable maximum precipitation (PMP) or other cause of the probable maximum flood (PMF).

This aspect was dealt with extensively under Question 63 during the ICOLD San Francisco Congress (ICOLD, 1988). Most authors suggested that the reservoir should be assumed to be at normal operating level at the start of the routing but Wang (page 658) had the following to say:

"A cyclonic storm may precede or follow another cyclonic storm within a 3- to 5- day period, depending on the season of the year and the meteorological characteristics of the region. Sequential thunderstorms may occur within even shorter periods between the storms. Consideration of antecedent and subsequent storms, therefore, may be important

in estimating the PMF. The importance, however, depends on the discharge capacity of the proposed reservoir at normal maximum pool.

In general, if the discharge capacity at the normal maximum pool is larger than the peak flow caused by the antecedent storm, it would not be necessary to consider the antecedent storm.

Consideration of a subsequent storm also would not be necessary if the discharge capacity is large enough to return the high reservoir level caused by the PMP back to the normal maximum pool before the subsequent storm. On the other hand, if the discharge capacity of the proposed reservoir is so small that the flood caused by the antecedent storm cannot be discharged before the PMP occurs, or if the subsequent storm would add to the maximum flood surcharge caused by the PMP, the antecedent and/or subsequent storms will have to be considered in determining the PMF".

South African experience with large dams has been that, on average, spillway discharge antecedent to a major flood has been about 5 percent of the peak of the flood.

Dam break flood routing is dealt with in the SANCOLD Guidelines report No.2 (SANCOLD, 1990). With the aid of texts the engineer can establish the stage hydrograph, both in the reservoir and at appropriate sections downstream, for inflow floods having a range of return periods, with various configurations of spillway system. From stage hydrographs routed downstream, first without failure and then with failure, one can establish the frequency distribution of the additional areas inundated as the result of dam break.

Where the downstream area takes the form of a complex flood plain it may be necessary to refer to the flood plain management models developed by Weiss in HRU reports No. 7/75, 3/76, 6/76 and 79/1 (Hydrological Research Unit, 1975, 1976, 1979).

5 SELECTING FLOODS FOR DAM DESIGN AND EVALUATION OF SAFETY

From the *structural engineering* point of view, dams are normally designed to satisfy certain generally accepted criteria. Criteria for conditions of "normal" loading differ from those for "abnormal or extreme" loading. Under "normal" loading conditions, ie. such as might reasonably be expected to be imposed during its useful life, the structure is required to function satisfactorily without appreciable deterioration. "Abnormal" loading would constitute the most severe conditions that could reasonably be expected to occur. In such circumstances even substantial damage can be countenanced but it would nevertheless be an essential requirement that the structure should not fail catastrophically. The philosophy is much the same in *hydraulic engineering* where the loading relates to the flood to be accommodated by the spillway system except that instead of normal and extreme conditions the terms "design flood conditions" and "extreme flood conditions" are used; the dam must not suffer damage under design flood conditions and must not fail should an extreme flood occur.

Whether a spillway system is being examined in relation to generalised criteria (Section 5.1) or site-specific hydrological calculations (Section 5.2), the two scenarios in which acceptable levels of performance must be achieved are:

- "design flood conditions" during which, provided normal maintenance work is executed on a regular basis, the spillway will operate without damage to any of its components or to the associated dam structure, and

- "extreme flood conditions" under which spillway operation may result in substantial damage to its components and/or to parts of the dam structure but would not result in catastrophic failure of the dam.

Criteria for both scenarios are laid down in Sections 5.1 and 5.2. When applying the alternative site-specific procedures outlined in Sections 5.3 and 5.4 other scenarios are envisaged.

Recommendations are made in the following paragraphs to aid selection of appropriate values for the design of new (or upgraded) spillways as well as for the evaluation of the adequacy of existing spillways.

For the "design" condition the Recommended Design Flood (RDF) must be specified and, for the "extreme" condition, the Safety Evaluation Flood (SEF). For the reasons outlined in Section 3.4.1, when applying generalised criteria, these are specified as discharges to be accommodated by the spillway system, ie. Recommended Design Discharge (RDD) and Safety Evaluation Discharge (SED).

The reader should refer to the Flow Diagram in Fig 1.1 to appreciate and follow the steps in the recommended evaluation procedure.

5.1 Generalised criteria

From a practical point of view, application of generalised criteria should demand minimal time and effort and should indicate immediately whether or not more detailed studies, taking into account site-specific conditions, have to be undertaken.

In essence, generalised criteria disregard many aspects that may be applicable to a particular site. As there are unknown parameters that could significantly affect the correctness of a particular choice, it follows that generalised criteria must reflect conservatism.

5.1.1 Criteria for design flood discharge conditions

The design flood condition is that for which the Recommended Design Discharge (RDD) is relevant with, of course, the appropriate freeboard requirement. For freeboard design values, reference should be had to SANCOLD (1990) Report No.3 Interim Guidelines on Freeboard for Dams.

For reasons mentioned in Chapter 1, suggested RDD values are expressed as level pool unrouted discharges derived from the relevant RMF peaks given in Report TR 137 (Dept of Water Affairs, 1988).

The recommended design discharge (RDD) is that shown in Table 5.1 and provides the preliminary basis for checking the design of the spillway system for new or existing dams.

If the practitioner has reason to question the RDD value derived from the RMF peaks according to Appendices 6 & 7 of Report No. TR137 he should refer to recommended methods of estimating flood frequency distributions in the SANCOLD handbook (Alexander, 1990).

Dam size class	Hazard rating		
	Low	Significant	High
Small	$0{,}5Q_{50} - Q_{50}$*	Q_{100}	Q_{100}
Medium	Q_{100}	Q_{100}	Q_{200}
Large	Q_{200}	Q_{200}	Q_{200}

Table 5.1 Recommended RDD values

(the subscripts to Q indicate the return period in years)

* Half Q_{50} is usually an acceptable approximation for Q_{20}

The degree of flood attenuation in the reservoir, and therefore the potential benefit to be derived from flood routing, can be assessed by estimating the flood volume likely to be associated with a single flood event. If the flood volume is large relative to the volume of available surcharge storage, ie. the volume between spillway crest and non-spill crest levels, flood routing would probably not have any significant effect on the spillway dimensions. Görgens et al (1990), seeking a quick guide to the necessity or otherwise of routing, examined a couple of dozen or more cases for which level pool routing had been performed. The regression of the peak discharge attenuation ratio (Q_{out}/Q_{in}) on the area ratio in the following equation was found to have a correlation coefficient of 0,82:

$$\frac{Q_{out}}{Q_{in}} = 0{,}99 - 5{,}56\ \frac{A_r}{A_c} \qquad (5.1)$$

where: A_r is area of reservoir at full supply level (FSL)
A_c is area of catchment commanded by the dam.

Thus, if the area of reservoir at FSL is as great as 10% of the area of the catchment, attenuation, ie. $(1 - Q_{out}/Q_{in})$, could be as high as 57%.

Freeboard is provided to allow for water surface rises that may be caused by a number of disturbances, possibly superposed on the flood discharge. The most important of those which may influence determination of appropriate freeboard are:

- wind generated waves
- wind set-up
- seiches (resonance)

- flood surges
- landslide-induced waves
- earthquake-induced waves

Various combinations of conditions should be considered in establishing the desired elevation of the non-spill crest. The SANCOLD (1990) Interim Guidelines on Freeboard for Dams deals extensively with the subject.

The general rule is that spillage under any of the "normal flood" conditions up to the RDF (in this case the RDD) should not cause damage and certainly not endanger the structure. For an embankment dam, settlement allowance should compensate for consolidation of foundation and embankment materials. The longitudinal crest profile of an existing dam should be checked for possible low spots that could reduce the effective freeboard.

To satisfy the requirement that no damage should be caused during normal flood conditions, the RDD values in Table 5.1, with appropriate freeboard allowances, may be adopted for preliminary checking of the design of a new spillway as well as for the redesign, if necessary, of an existing spillway.

It is standard practice to impose the hydraulic conditions associated with normal flood conditions for determining the geometry of the hydraulic components of a new dam. The same criteria should therefore be used to evaluate the expected performance of an existing spillway. If an existing spillway does not satisfy the recommended criteria for normal flood conditions, then the consequences, in terms of increased maintenance and repair cost, should be evaluated. If the consequences are unacceptable consideration should be given to remedial measures or upgrading the spillway system.

For small dams of low hazard rating, financial considerations rather than potential hazard may dictate the appropriate criteria. The minimum return periods recommended for the RDD of these dams in Table 5.1 are based on experience.

If the spillway passes the RDD test the next step is to test performance under extreme flood conditions. The design must be checked to ensure that the recommended safety evaluation discharge (SED) referred to in the next section can be accommodated without resulting in dam failure.

5.1.2 Criteria for extreme flood conditions

The scenario of extreme flood conditions is that for which occurrence of the Safety Evaluation Discharge (SED) is relevant.

The magnitude of an extreme event cannot be determined with great accuracy due to the lack of sufficiently long hydrological records. Furthermore, "reasonable care" requires that the potential hazard associated with the mere existence of the dam should be taken fully into account.

Again, for practical reasons it is desirable that the method chosen for determining the SED should yield consistent results without necessarily involving excessive computational effort. Accordingly, the SED is expressed in terms of a peak discharge rather than a flood (implying a hydrograph). In other words, like the RDD, it is expressed as a level pool unrouted discharge derived from the relevant RMF value given in Report No. TR 137 (Dept of Water Affairs, 1988):

The Safety Evaluation Discharge (SED) is the discharge which has the value recommended in Table 5.2 and provides the initial screen for checking the adequacy of the spillway system of a new or an existing dam under extreme flood conditions.

The capacity of the spillway system under examination must be adequate to pass the unattenuated SED in such a way that, although substantial damage may be suffered by the structure and surroundings, the dam will not fail. Failure in this context means unplanned catastrophic uncontrolled release of water from the reservoir.

Dam size class	**Hazard rating**		
	Low	**Significant**	**High**
Small	$RMF_{-\Delta}$	$RMF_{-\Delta}$	RMF
Medium	$RMF_{-\Delta}$	RMF	$RMF_{+\Delta}$
Large	RMF	$RMF_{+\Delta}$	$RMF_{+\Delta}$

Table 5.2 Recommended SED values

The Regional Maximum Flood (RMF) expressed as the peak discharge must be determined from the formula for the appropriate Region and Zone in Table 5.3 (taken from Table 6 in DWA report TR 137). In Table 5.2 the subscript ($-\Delta$) means: choose the region one step lower numerically than that in which the problem catchment lies. Likewise, the subscript ($+\Delta$) implies: adopt the adjacent region numerically one step higher.

	Transition zone		Flood zone	
Region	RMF (m^3/s)	Area range (km^2)	RMF (m^3/s)	Area range* (km^2)
2,8	$30\ A^{0,262}$	1 - 500	$1{,}74\ A^{0,72}$	500 - 500 000
3,4	$50\ A^{0,265}$	1 - 300	$5{,}25\ A^{0,66}$	300 - 500 000
4	$70\ A^{0,34}$	1 - 300	$15{,}9\ A^{0,60}$	300 - 300 000
4,6	$100\ A^{0,38}$	1 - 100	$47{,}9\ A^{0,54}$	100 - 100 000
5	$100\ A^{0,50}$	1 - 100	$100\ A^{0,50}$	100 - 100 000
5,2	$100\ A^{0,56}$	1 - 100	$145\ A^{0,48}$	100 - 30 000
5,4	$100\ A^{0,62}$	1 - 100	$209\ A^{0,46}$	100 - 20 000
5,6	$100\ A^{0,68}$	1 - 100	$302\ A^{0,44}$	100 - 10 000

Table 5.3 Formulae of the envelopes of regional maxima
A is the effective area of catchment (km^2)

* The upper limit is relevant only to South Africa.

It must be emphasised that the peak discharge values recommended in Table 5.2 for preliminary safety evaluation are intended to reflect reasonable care; in some instances they may in fact be too strict.

The explanatory and supplementary notes that follow deserve attention.

- In small catchments (e.g. less than 10 km^2 in area) and in urban catchments, RMF does not offer a satisfactory basis for screening; site-specific methods should be employed.

- Because of variations in magnitude of the discharge increments from one region to another, it may appear at first sight that the degree of conservatism varies from an increase of 20%, which seems reasonable for the regions of higher annual rainfall, to an increase of 100%, which may seem high for the drier regions. It should be noted, however, that the smaller increments relate to regions where flood data are relatively adequate. The recommended values are thus consistent with the availability of data.

- Zero dry freeboard is generally considered acceptable with the SED, except that, for an embankment dam, it would be desirable to check the adequacy of the top portion of a zoned embankment, including the filters, for possible piping during a surcharge event.

- Experience has proved that a well established grassed downstream slope of an embankment dam can withstand overtopping to depths of almost a metre and lasting up to an hour. Although one would not build such a circumstance into a new design, careful examination of all the site-specific factors may nevertheless lead one to recommend approval of an existing spillway system where an embankment would be subjected to substantial overspill in the event of extreme conditions.

- Overtopping of the non-overspill crest (NOC) of a concrete dam by the SED would be permissible provided stability conditions are not violated and provided the foundations are not endangered by scour.

- Flood surges due to failure of upstream dams (cascade effect) are generally not included directly in freeboard calculations. The possibility of such occurrences must nevertheless be taken into account and the SED adjusted if necessary.

- It is necessary to alert owners of dams to the implications of changing downstream land use, particularly urbanisation, which may alter the categorisation of a dam. Increasing hazard may preclude further downstream development.

- As indicated in Chapter 1, RDD and SED values in Tables 5.1 and 5.2 respectively are intended as screens to identify existing dams that require further investigation of spillway adequacy. Spillways of existing dams that can accommodate the indicated RDD without causing damage and the SED without causing catastrophic damage need no further investigation.

- The likelihood that a dam that is caught on the RDD screen would pass the SED screen is fairly rare; an example would be a reservoir having a long fetch but small catchment of low flood runoff. The reason for initiating the dam safety tests with the RDD (which is not strictly related to safety) is for convenience in that it follows the design sequence.

- As mentioned in Chapter 1, approximately 85 per cent of all registered dams in the RSA fall within the SMALL and MEDIUM size classes with LOW or SIGNIFICANT potential hazard ratings. The figure should be roughly the same for new dams and therefore, by implication, the values in Table 5.2 may also be adopted for checking the majority of new dams.

- Although small dams having low hazard ratings are not subject to comprehensive safety evaluation by professional engineers, values of the RDD and SED are included in Tables 5.1 and 5.2 as a general guide. Sizing of spillways for such dams is usually decided on purely financial grounds.

Most of the foregoing explanatory and supplementary notes are relevant also to site-specific conditions in what follows.

For all new HIGH hazard dams, as well as for medium and large dams having SIGNIFICANT hazard ratings, it is obligatory that site-specific analyses shall be the basis for evaluation of the safety status under extreme flood conditions.

As indicated in the Flow Diagram Fig 1.1 and Section 3.4, there are three alternative approaches, described in Sections 5.2, 5.3 and 5.4; circumstances will dictate which is the most appropriate to follow.

5.2 Site-specific hydrological calculations

If a spillway system fails to meet the safety criteria outlined in Section 5.1.2, ie. a discharge equal to the level pool unrouted SED value stipulated in Table 5.2 would be likely to cause failure of the dam, the engineer is left with three alternatives, all of which entail site-specific hydrological calculations as discussed in Chapter 4. The first approach is discussed in what follows.

When significant attenuation of an incoming flood is expected either under design flood or under extreme flood conditions, hydrological calculations involving generation of hydrographs are necessary for determination of water surface elevations at the spillway.

5.2.1 Design flood conditions

Acceptance criteria for this procedure require that, for the design flood condition, the spillway system should accommodate the routed recommended design flood (RDF) with adequate dry freeboard, ie. including allowance for waves, wind set-up, surges and seiches but not for earthquake or landslide waves. (See SANCOLD Report No.3, 1990).

The suggested return periods of the incoming RDF are given in Table 5.4 and are the same as those adopted for the RDD but, instead of a single discharge value, a family of hydrographs of the appropriate return period should be compiled and routed through the reservoir/spillway system to find that which determines the capacity of the spillway under design flood conditions.

Methods of generating hydrographs are explained in the Handbook (Alexander, 1990) and in Report 1/72 (Hydrological Research Unit, 1979).

Dam size class	Hazard rating		
	Low	Significant	High
Small	20 — 50	100	100
Medium	100	100	200
Large	200	200	200

Table 5.4 Suggested RDF return periods (years)

5.2.2 Extreme flood conditions

For the extreme flood condition the safety evaluation flood (SEF) is intended to quantify the incoming flood hydrograph which, after routing through the reservoir/spillway system, has to be handled in such a way that although the freeboard may be fully absorbed, indeed even exceeded, the resulting damage will not be such as to cause failure of the dam.

For MEDIUM and LARGE HIGH hazard dams and LARGE dams of SIGNIFICANT hazard, the incoming hydrograph shall be the PMF. For dams of smaller size and lesser hazard the PMF may be downrated proportionately according to the interrelationship of corresponding SED values in Table 5.2.

The incoming PMF hydrograph may be derived by unitgraph procedure from catchment storm PMP adjusted for "storm loss" but must be checked against suitably adjusted observed extreme flood hydrographs associated with the specific or a hydrologically similar river system. In this context reference must be had to historic flood information (e.g. documented floods, Dept. of Water Affairs, 1982, 1984, 1985, 1987, 1988 and 1989) as well as reliable paleoflood peaks or peaks estimated from considerations of valley cross-section and morphology. Hydrographs having a feasible range of shapes should be generated and routed, preferably those resulting from storms of duration half to four times the response time of the catchment but generally not longer than 7 to 10 days.

As there is a high degree of correlation between hydrograph peak and hydrograph volume, caution should be exercised before adjusting the hydrograph peak value downwards for a large-volume flood. There is ample evidence in South Africa and elsewhere to show that extreme flood peaks are caused by long duration storms, even in small catchments. Therefore, as a matter of principle, SEF design hydrographs should envisage the probable simultaneous occurrence of highest peak with largest volume.

The routed PMF peak may well be found to be substantially lower than the corresponding recommended SED value in Table 5.2. If so, it is essential that all the steps in the derivation procedure be carefully re-checked. At the same time, attention is drawn to the last paragraph of Section 4.2 concerning the need to avoid over-estimation of the PMF peak. To check the volume of flow represented by the flood hydrograph, the experience envelope of precipitation maxima, Fig C4 of Report 1/72 (Hydrograph Research Unit, 1979) provides a useful basis for estimating regional point PMP as a function of duration of storm. Application of appropriate areal reduction and storm loss factors can ensure that a generated hydrograph does not represent an unrealistic volume of flood runoff. The documented flood studies of DWA referred to above provide better information on large area storms in the eastern sector of the country than that in Figures D of Report HRU 1/72.

5.3 Zero incremental impact approach

The cornerstone of this method is that spillway capacity is determined from an evaluation of the incremental impact likely to be experienced in the area downstream of the dam. The minima set by the generalised design criteria may be considered as the starting point of the analysis and the endeavour is then to assess whether the risks associated with the project justify relaxing or tightening the generally accepted criteria. The approach is a recent development and has been recommended for the safety evaluation of high hazard dams in the USA by the Committee on Safety Criteria for Dams (1985). It is now also recommended in the RSA for design and safety evaluation of high hazard dams as well as for medium and large dams having significant hazard ratings.

It is envisaged that this approach could well be used also to determine the acceptable spillway capacity for dams that have been classified Category II solely because of the presence of one or more downstream road crossings. In such cases one should determine the flood (peak and/or volume) that would render the road crossings impassable to the extent that lives would be threatened. This flood, with an appropriate factor of safety to allow for uncertainties, could be used as the basis for determining the zero incremental impact flood (ZIF). Clearly, there is a need at present to establish practical guidelines for assessing damage functions for river crossings.

The features that follow have to be evaluated.

The imminent failure flood (IFF). This is the inflow flood which, when routed through the reservoir, can be passed by the spillway with reservoir level just threatening failure of the dam. For an embankment dam, the IFF could be taken to be the incoming flow that just causes overtopping of the embankment. For concrete dams the IFF would be derived after consideration of structural stability and downstream threat to foundations under overtopping conditions.

Imminent Failure Inundation. A plan view of the downstream area that would be inundated by the IFF routed through the reservoir and the downstream reach.

Post-failure Inundation. The plan view on which is demarcated the area likely to be inundated by the dam break wave moving downstream, superposed on the IFF outflow.

Incremental Impact. The incremental impact is the difference between the losses (of life and property) associated with the IFF and those attributable to the dam break flood superposed on the IFF outflow. Loss of life need not be converted to monetary terms.

Zero Incremental Impact Flood (ZIF). The ZIF is the flood which would just cause failure of the dam by overtopping but would not be such as to cause significant increase in the downstream damages and potential loss of life, ie. zero incremental impact under present and foreseeable future conditions. The ZIF has to be determined by trial.

There are six steps in following the ZIF procedure.

(a) Determine the SEF recommended for the category of dam in Table 5.2.

(b) Determine the IFF.

(c) If the IFF is less than the recommended SEF, assess the incremental impact due to a failure during the SEF, using hydrograph generation and routing techniques referred to in Section 4.3 and dam break according to the Guidelines on Dam Break Floods (SANCOLD, 1990). If the incremental impact is clearly so severe that overtopping and failure of the dam simply cannot be countenanced, design adequate measures to avert failure of the dam. If, however, the incremental impact is not significant, proceed to step (d).

(d) Determine the ZIF by stepwise trials.

(e) If the ZIF is less than the IFF, consider the consequences of dam failure and loss of project services at the probable frequency associated with the IFF. If it is judged that such risks can be tolerated, no additional work to provide further safety against extreme flood would be indicated.

(f) If the ZIF is greater than the IFF, or if it is considered that the consequences of dam failure are unacceptable, proceed with a risk-based analysis to develop a further basis for a decision on alternatives.

Several aspects relevant to assessment of the IFF have been listed by the Australian National Committee on Large Dams (SANCOLD), 1984.

(i) The imminent failure flood (IFF) should be based on a reasonable but not optimistic assessment of the conditions that could lead to a major failure. Some damage of an embankment dam, such as scouring of the crest and areas downstream of the spillway energy dissipator, can be accepted if unlikely to cause dam failure, even though substantial repairs may be required.

(ii) For embankment dams, it may be practicable to select the IFF as the flood giving a stillwater level at crest level (excluding camber) allowing wave splash onto the crest, provided other factors such as stability are satisfactory. Leakage through the crest road foundation above the core can also be significant. The existing dam crest profile should be checked for spots which could concentrate flow, but which could be rectified by minor remodelling works.

(iii) Parapet walls should be checked for structural stability and defects, such as formed gaps and shrinkage and expansion cracks; the walls should provide freeboard merely against wave run-up, unless specifically designed against stillwater loading. In some cases it may be advisable either to ignore the walls or to have them reconstructed.

(iv) For concrete dams positive overflow as well as wind set-up and wave run-up could be allowed over the crest proper (negative freeboard) depending on the abutment/dam interface and the stability.

(v) For gated spillways, the arrangement, number and reliability of the gates, level of underside of gates when fully raised and effects of water flow against the underside of the gates need to be considered.

5.4 Risk-based analysis

As will be recalled from the introductory remarks in Section 3.4.3, prerequisites to risk analyses are probability distributions of both flood peak and flood volume, ie. the appropriate flood hydrographs. By routing the hydrographs downstream one can establish the probability distribution of downstream damage *without dam*. The hydrographs can then be routed through the reservoir and downstream river reaches for various capacities of spillway system, with (if appropriate) reservoir operation rules, to establish the probability distribution of *with dam* damage as a function of the capacity of the spillway system. The dam must be presumed to fail as the capacity of each spillway system tested is exceeded. There will thus be a probability distribution of failure and therefore of damage due to failure. Now if the probability distributions are converted to curves showing damage as a function of probability of exceedance (a) with and (b) without dam failure, these can be integrated to yield for each the total damage, or the average annual damage. The net or incremental damage is the difference between the two integrations.

It stands to reason that the greater the capacity of spillway system that has been provided the lower will be the frequency of assumed failure. It may be noted, however, that the annual damage likely to result from dam failure is not closely related to the capacity of the spillway system. The dam-break damage will be much the same whatever the size of the spillway. It is its frequency that diminishes with increasing capacity. Once the dam-break damage has been established, therefore, one can merely multiply the damage cost by its annual probability of exceedance in order to yield the annual damage costs to be used in the risk-based analysis.

To the annual damage cost must be added the annual cost of providing the spillway system, so as to yield the total cost, as illustrated in Fig 5.1. The minimum total cost indicates the optimum design.

Unfortunately, however, conversion of stage hydrograph or areas inundated to the corresponding damage costs can be difficult as damage is not a direct function of either depth or area of inundation. Apart from duration of inundation and hydraulic force (which is a function of the velocity of the inundating waters), there are many factors that influence the resulting damage, eg. sediment and debris content of the waters, time of the year (as far as crops and orchards are concerned), the alignment of streets and crop or tree rows (whether parallel or transverse to the main direction of flow) and so on. Despite several studies of flood damage sponsored by the Water Research Commission there have emerged few multidimensional damage functions that can be used by the engineer to convert depth-area-duration-frequency relationships (which are what the hydrological/hydraulic analysis can yield) to damage-frequency relationships (which are what the risk analyses require as input). Fortunately, however, the decision as to optimum spillway capacity and freeboard is seldom highly sensitive to inaccuracies in the cost curves, as Fig 5.1 illustrates.

The SANCOLD (1990) Guidelines on Dam Break Floods give some guidance on procedures to be followed to assess damage.

Where property losses likely to result from dam break are intolerably high, the damage costs curve in Fig 5.1 will decline slowly forcing the minimum, ie. the optimised flood to be accommodated, so far to the right that it will turn out to be the flood having a practically zero probability of occurrence (ie. the PMF).

Similar evaluation techniques can be used for the other aspects such as loss of life, socio-economic losses and environmental damage. (See Oosthuizen, 1986). The SANCOLD (1990) publication on Risk Analysis for Dams and the ASCE (1988) report on the Evaluation Procedures for Hydrological Safety of Dams should also be studied.

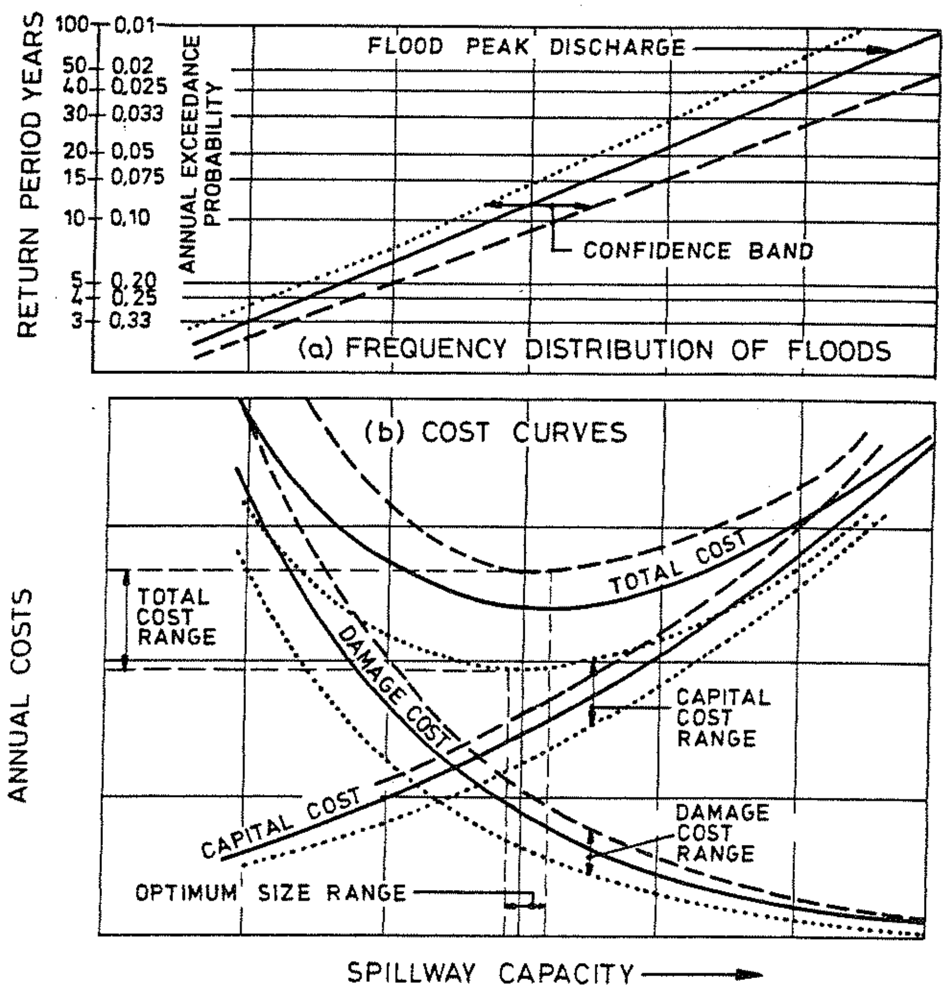

Figure 5.1 Economic optimisation of spillway capacity

6 CONCLUDING REMARKS

It is well to emphasise that the Guidelines do not constitute a Code of Practice and that there in no intention to prescribe the use of any specific hydrological technique. The recommendations made in these guidelines, however, particularly in regard to determination of the safety evaluation flood, are deemed to be the most satisfactory at the present state of knowledge, which admittedly is in many respects deficient. It is appreciated that special circumstances may call for departure from the recommendations. Any departure must be supported by careful reasoning based on the use of modern methods and the exercise of sound engineering judgement.

Judicious consideration of economic as well as safety aspects must temper decisions as to whether an existing spillway may be considered satisfactory or should be upgraded.

It goes without saying that site-specific investigations may well lead to acceptance of higher or lower flood values than have been indicated as recommended values in Tables 5.1 and 5.2. It follows, however, that adoption of lower values must be strongly motivated by carefully checked site-specific analyses.

Although utmost care must be exercised to ensure appropriate levels of safety in design, maintenance and operation of dams, it is equally important to avoid incurring fruitless expenditure through overdesign as the result of unrealistic hydrological estimates.

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