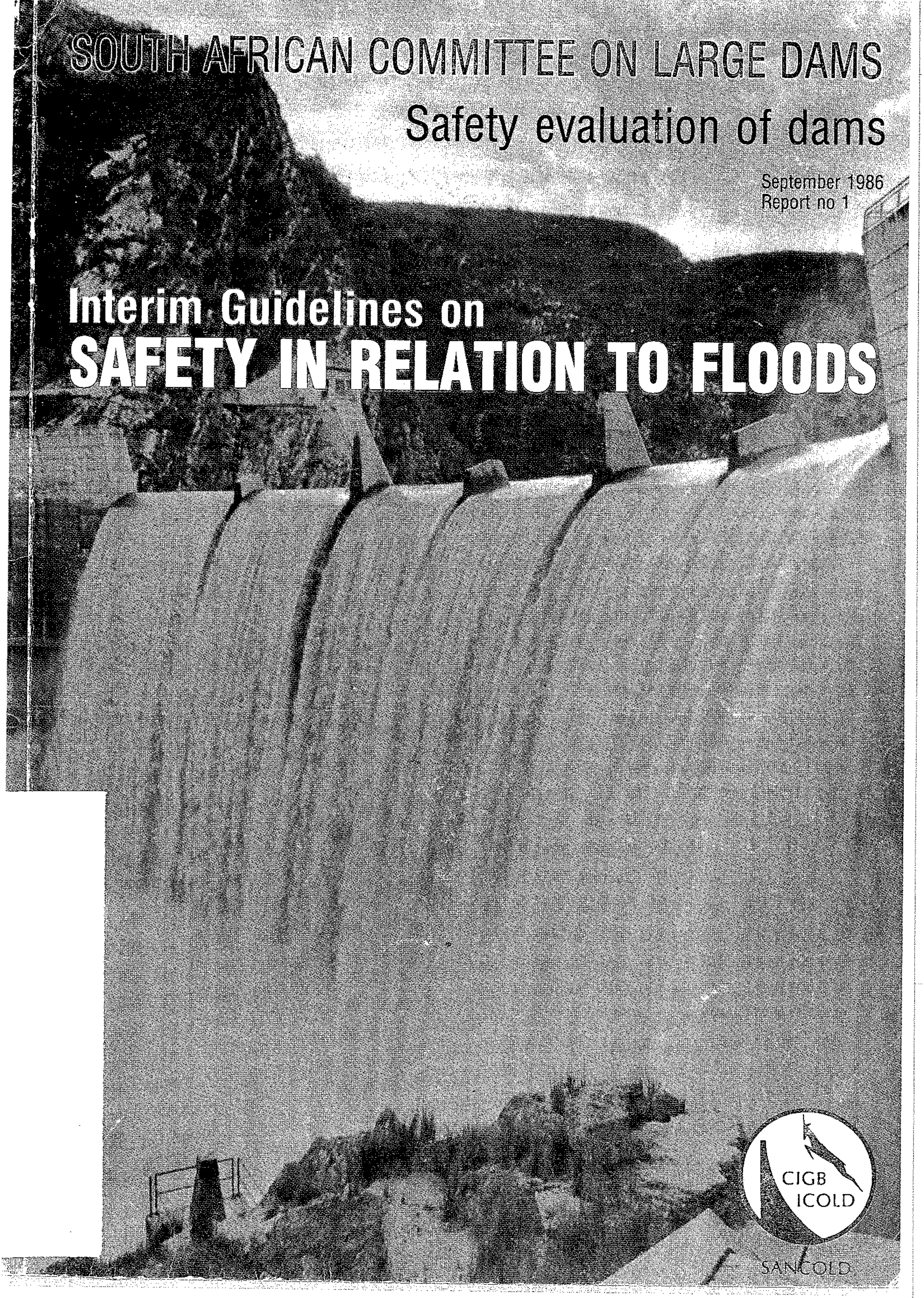
SOUTH AFRICAN COMMITTEE ON LARGE DAMS

Safety evaluation of dams

September 1986
Report no 1

Interim Guidelines on SAFETY IN RELATION TO FLOODS



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REPORT No. 1
SEPTEMBER 1986

PREFACE

South Africa, like many other countries, has recently introduced legislation to control the safety of dams. To conform with the requirements of the legislation, the authority administering the Act and professional engineers will be called upon to handle the registration, inspection and safety evaluation of a large number of dams throughout the country.

Engineering standards for dams are not prescribed in the legislation and there are no existing South African codes of practice for dams. However, it seems essential to provide a set of guidelines to assist not only the designers of new dams but also those charged with evaluating the safety of existing dams. The first of the set is concerned with safety in relation to flood. Other guidelines addressing structural and other safety aspects will be prepared in due course.

It will be noted that although the use of general standards is recognized as a rapid means of identifying those dams that require immediate further investigation, the tendency is to move towards decision-making based on site-specific analyses.

These guidelines have been prepared with great care, taking into account current practices followed in other countries. Since the guidelines include proposals for the use of modern techniques, it was considered appropriate to allow time for users to comment on possible unforeseen problems experienced in the application of these guidelines. Comments could be forwarded to: The Secretary, SANCOLD, P O Box 3404, Pretoria 0001, before September 30, 1987.

Embarking upon the compilation of the Guidelines on Floods prompted the preparation of a supplementary SANCOLD handbook on South African Flood hydrology which will fulfill a longfelt need.

G. G. de Plessis

Chairman

SANCOLD

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Cover: Paul Sauer Dam. Photograph by W.C.S. Legge.

INTERIM GUIDELINES ON SAFETY IN RELATION TO FLOODS

1. INTRODUCTION:

A dam is normally intended to meet a social need or to provide some benefit - but at a price, both in money and possible adverse environmental impact terms. The benefit may take the form of assurance of water supply for whatever purpose. The price is 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. The bigger the impoundment the smaller would be the risk of failure of water supply; 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 optimum size (e.g. of spillway capacity). It follows that sizing can be optimized by means of risk analyses. Where significant danger to human life is involved the inclination is to reduce the probability of dam failure to negligible levels. In a wide variety of circumstances, however, general standards can be adopted.

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, e.g. geologic, seismic and structural. The Guidelines aim to facilitate selection of that inflow design flood which will reduce to an acceptable level the risk of dam failure through inadequacy of the spillway system.

Legislation was introduced recently 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 a 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 operated and maintained in accordance with 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 established a sub-committee to prepare these Guidelines.

The Committee comprised:

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

In preparing these Guidelines, the Committee has concerned itself primarily with the sizing of the spillway systems of "safety risk" dams. Guidance is intended essentially 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. A proposed dam may in due course become an existing dam and accordingly these Guidelines must be applicable also to the design of new dams. Thus, a distinction is drawn between (a) the checking of an existing dam, for which it is necessary to determine the Safety Evaluation Flood (SEF), and (b) design of a new dam for which one seeks a Recommended Design Flood (RDF) which, after allowance for freeboard, must be checked against the SEF.

In the next chapter, categorization of dams in accordance with the Regulations is explained. There follows a major chapter (Chapter 3) on general considerations concerning the safe release of floods from dams, including legal aspects and philosophy of risk. The following chapter (Chapter 4) is devoted to providing an empirical means of estimating flood magnitudes specifically for the purposes of defining minimum values of the RDF and SEF in the matrix discussed in Chapter 5. The matrix in effect constitutes a coarse sieve through which many existing dams will pass, leaving only those caught on the sieve to be given detailed scrutiny. For such scrutiny, the Guidelines avoid prescribing specific techniques for estimating or deriving flood magnitudes but merely refer the reader to Flood Hydrology: A South African Handbook (SANCOLD, 1986) and stipulate that full use be made of all relevant data, along with modern hydrological techniques, in deriving flood magnitudes for design and safety evaluation of spillways.

In Chapter 5 recommended minimum design and safety evaluation values are discussed, as are allowances for freeboard, waves, wind set-up, and surges caused by rapid rise of the incoming flood to a reservoir.

2. DAM SAFETY LEGISLATION

2.1 CLASSIFICATION OF DAMS

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

a) "any dam having a storage capacity in excess of 50 000 m^3 and a vertical height in excess of 5 m, 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

b) 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 25/07/86) classify dams with a safety risk according to size as in Table 2.1.

Table 2.1. : Size Classification

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

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, 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 that 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 giving the highest rating is decisive. The hazard rating is thus a qualitative indication of the potential loss that would result from a "sunny day" failure, i.e. not necessarily associated with a natural flood entering the reservoir.

Table 2.2 : Hazard Classification

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

* For classification purposes potential economic losses less than R1 million and more than R10 million (1986) would be considered "minimal" and "great" respectively.

2.2 CATEGORIZATION 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 categorized in terms of the Regulations according to Table 2.3.

Table 2.3 : Categorization of dams having a safety risk

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

2.3 REQUIREMENTS IN RESPECT OF SPILLWAYS:

The Regulations require that 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 essentially geometric description in the case of a category I dam to 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 category II and category III dams.

Standards for deciding on the adequacy of spillways are not prescribed in the Regulations. The underlying principle embodied in the Regulations 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, eg. the spillway, the discharge carrier, the 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 prevent failure of the dam by overtopping. A properly functioning spillway protects the dam by passing excess flood waters downstream. An extraordinarily large natural incoming flood, when discharged by the spillway, may well cause downstream damage much the same as that which would in any event have occurred had there been no dam. Failure of the dam in such circumstances, however, would generally result in discharges and damages far greater than those already experienced. For failure under extreme flood conditions, therefore, it is the difference - the incremental damage - that is of concern. Malfunction and inept operation of spillway systems are dealt with in Section 3.5.

3.2 LEGAL CONSIDERATIONS:

From a legal point of view, the main question is whether the engineering judgements exercised reflect 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."

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.

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 on 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 upon 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 flood that could be considered as "an act of God" would be "so extraordinary and devoid of human agency that reasonable care would not avoid the consequences."

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 recognize 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 maximize 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).

<u>Table 3.1</u> : Mortality statistics for several Western countries

Cause of Death	Per million persons per year
All causes (including natural)	11 000
Accidents : All	460
Railway	4,3
Air	6,5
Water transport	5,3
Motor car	240
Poisoning by medical drugs, etc.	3,5
Poisoning by other substances	15
Falling	130
Fire	28
Natural and environmental factors	3,8
Lightning	0,87
Drowning	24

From the risks considered in Table 3.1, it is possible to identify several sub-groups as follows:

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 compensating 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 a vis involuntary aspect)
* 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) above, 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 not) of risk have been proposed or are being used, for example:

* Exposure, such as less than 10^{-6} fatalities/person/annum
* 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 1×10^{-6} per calender 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 and no doubt the subject will in due course be addressed in future guidelines.

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 will be equalled or exceeded at least once within the next L years (eg. the life L of a project) is given approximately by the expression:

$$P_T = 1 - (1 - {}^1/_T)^L$$

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:

$$P_T = 1 - (1 - {}^1/_T)^T$$

which, if T is large, approaches:

$1 - e^{-1}$, i.e. $1 - 0{,}37 = 0{,}63$.

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.

As is evident, lowering of the tolerable risk rapidly raises 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. (See Fig. 4.2).

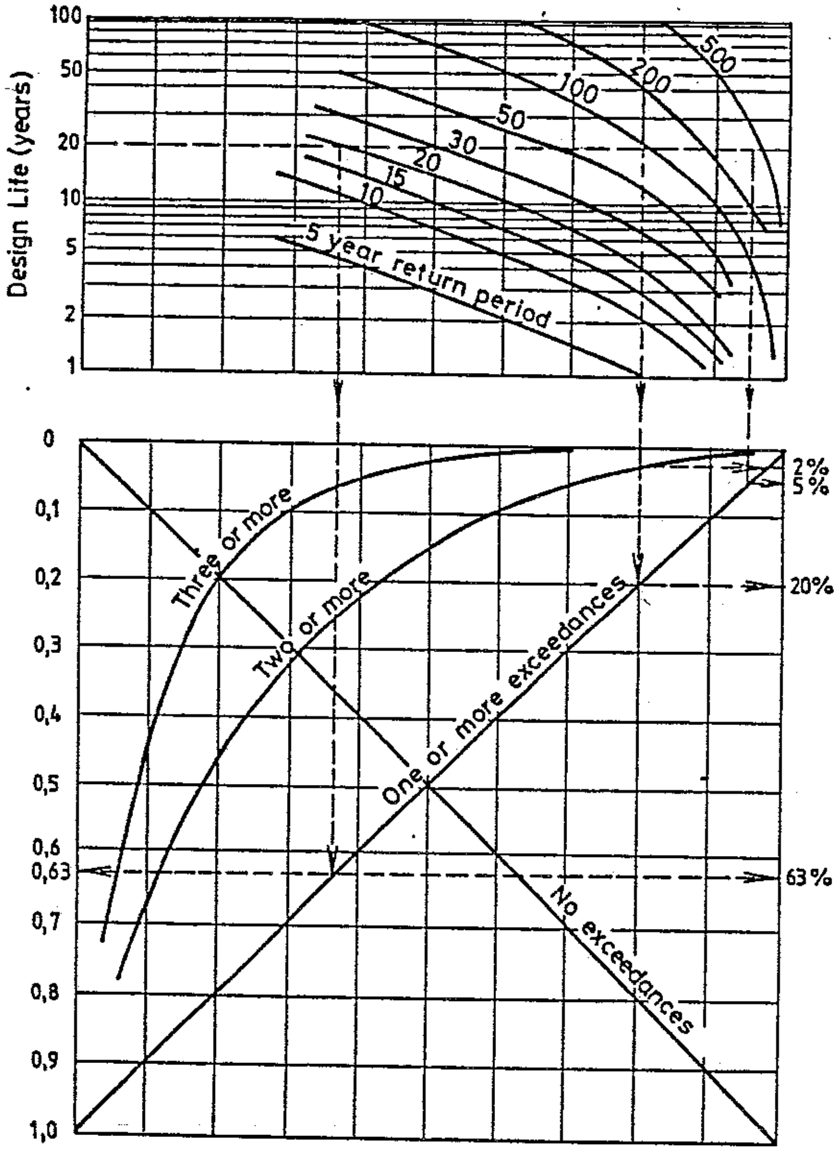


FIG. 3.1 RISKS OF EXCEEDANCE WITHIN A GIVEN DESIGN PERIOD

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 standards and partly by the quality of construction, inspection, operation and maintenance. Ideally acceptable levels of hazard, failure probability, and damage should be quantified for the site-specific conditions at each existing or proposed dam. Three current ways of dealing with these aspects can be identified: (1) use of generalized design standards, (2) the zero incremental impact approach and (3) optimization based on risk-based analysis.

3.4.1 Generalized design standards

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 standard, depending on height, storage capacity and hazard rating. Generalization of design standards is an attempt to define reasonable care by acknowledging that the level of protection provided should reflect consideration of the hazard potential, viz loss of human life or dam service, 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 standards in a generalized way with the result that the appropriateness of the design standard as applied to an individual dam becomes uncertain.

Although generalized standards are based largely on acceptable practices, their application demands careful engineering judgement. They stem primarily from the need for regulating bodies to exercise control uniformly over the many dams under their jurisdiction. Although they offer a practical way of deciding on a Safety Evaluation Flood (SEF) without expending too much effort, adoption of general standards is not the ideal way of accounting for site-specific conditions.

Further details of the procedure are given in Section 5.1.

3.4.2 Zero incremental impact approach

In assessing the damages 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 or incremental damage or loss of life compared with conditions during the same extreme flood without dam failure. Similarly for failure of even a large storage reservoir the incremental damage might well be small if all the downstream development were situated in low-lying areas that would in any event be inundated and severely damaged by a relatively small flood.

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 selection of spillway capacity is therefore determined neither on generalized design standards, 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 5.2. If, for a particular alternative under investigation, the zero incremental flood exceeds the imminent failure flood, or if the consequences of failure are unacceptable, further alternatives must be investigated or risk analyses performed to develop a further basis for decision.

This 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 there is no need to assign a monetary value to a human life.

3.4.3. 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 a dam failure. That option which yields the least total cost is thus identified. The method, which is elaborated in Section 5.3, requires that floods of various sizes and associated return periods be estimated well beyond the length of available records to permit flood frequency to be converted to damage frequency and thence to yield damage costs. A procedure for extending flood frequency distributions 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 consequential damages is generally difficult.

Where no loss of life is expected, as would be the case for low hazard dams, the design alternative that yields least total cost would be selected. The decision would thus be based purely 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 intention is to prepare a separate guideline on risk-based analysis.

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 both spillways and flood outlets but the contribution to flood release through flood outlets should be taken into account only when dependability of operation during flood can be assured.

Two general types of spillway are distinguished, viz. service spillways and auxiliary spillways:

- Service spillways, when complemented by an auxiliary spillway, should be designed to handle relatively frequent and sustained flood releases up to, say, the one-in-a-hundred-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.

  A gated spillway should always be complemented by an auxiliary spillway.

  Despite the considerable advantages of gated spillways under skilled supervision, it must be assumed in design that gates and automatic spillways will not always operate 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 engineering judgement.

- 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 inept operation or mechanical or electrical malfunction of gated spillways would have serious consequences. If the auxiliary spillways 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.

  It is incumbent upon the designer to adopt arrangements that will be sure to function when called upon to do so. It would be purposeless for a fuse plug to withstand the rising flood level while allowing an uncontrolled breach to occur elsewhere. The use of erosion-resistant materials in such structures is therefore to be avoided unless a proven failure section is provided.

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. On the other hand, inept 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 where, in anticipation of a major flood, water was pre-released with the intention of minimizing downstream damage but the hydrometeorological forecast proved 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 dambreak, may cause a flood wave in the form of a "wall of water". 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 rate of rise or the inflow hydrograph (Kovacs et al, 1984). On several occasions stage measurements and discharge calculations in South Africa have revealed major surges that caused higher spillway discharges than those of the contemporaneous inflows. 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 Section 5.1.2.

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 peak of a sharply peaked incoming flood but may hardly affect the peak of an incoming hydrograph that has resulted from prolonged rainfall in the catchment, although both hydrographs may have the same return period.

3.6 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 and the plans should reflect pre- and post-failure conditions.

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.

The dam-break discharge is largely a function of the height to which water is impounded above river bed level at the time of failure rather than of the volume impounded. Theoretical expressions for the dambreak peak discharge are normally based on sudden creation of a gap of given width in the dam of given height and crest width and given spillway breadth. A typical form (US Army Corps of Engineers, 1977) is:

$$Q_p = {}^{8}/_{27} \, [{}^{B_o}/_{b}]^{\frac{1}{4}} \; b \; g^{\frac{1}{2}} \; h_o^{\,3/2}$$

in which B_o is the downstream channel width, b the breach width and h_o the maximum reservoir depth before failure.

It is always difficult, however, to predict the width of breach and no account is taken of the rate of development of the breach. It is understandable therefore that engineers tend to rely on empirical data. The experience diagram, Fig. 3.2, is based on Guidelines for Evaluating Spillway Capacity (Kirkpatrick 1977). Oosthuizen (1986) derived the following expression for the envelope of 33 failure flood peaks:

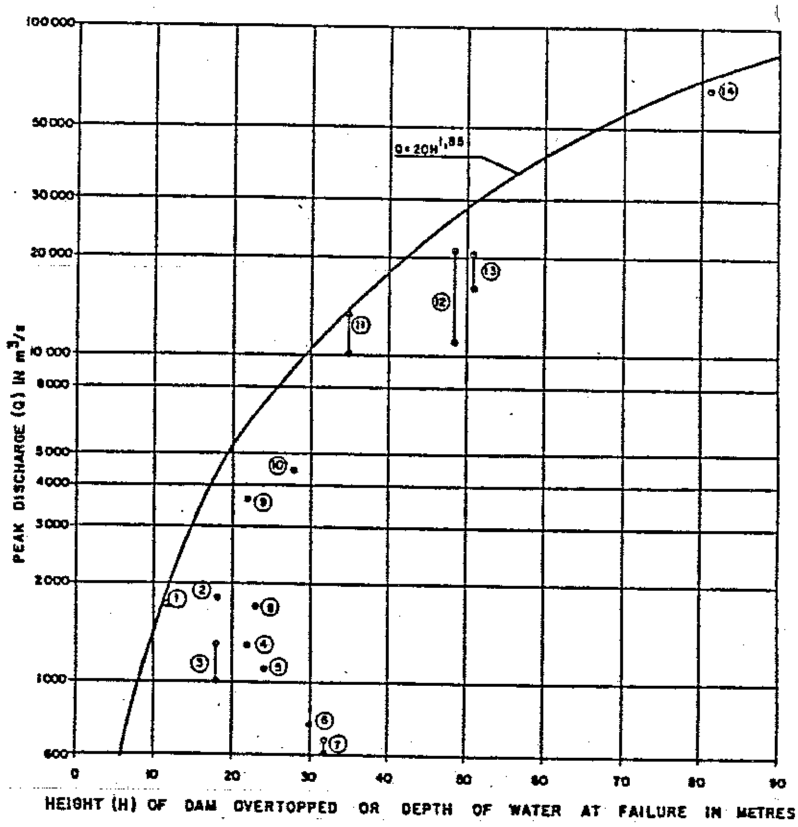
$$Q_{peak} = 20H^{1.85}$$

in which H = water depth (m) from bottom of break to reservoir level at the time of failure.

Oosthuizen's envelope is seen to embrace all of the failures noted by Kirkpatrick in Fig. 3.2.

As may be expected the dam-break discharge can be enormous, even when compared with the PMF inflow.

For a given inflow hydrograph, the rate at which the dam-break discharge would subside with cessation of the failure mechanism is a function of the volume of water impounded, while the attenuation with distance downstream



NUMBER	NAME OF DAM	LOCATION	YEAR OF FAILURE	TYPE OF DAM	REMARK
1	LOWER TWO MEDICINE	MONTANA	1964		
2	HATCHTOWN	UTAH	1914		
3	BALDWIN HILLS	CALIFORNIA	1963		
4	LITTLE DEER CREEK	UTAH	1963	EARTH	
5	DALE DYKE	ENGLAND			[illegible]
6	HELL HOLE	CALIFORNIA	1964	ROCK	
7	APISHAPA	COLORADO	1923	EARTH	
8	GRANITE CREEK	ALASKA	1971		[illegible]
9	CASTLEWOOD	COLORADO	1933	ROCK	
10	SCHAEFFER	COLORADO	1921		
11	OROS	BRAZIL	1960		
12	SWIFT	MONTANA	1964	ROCKFILL	
13	ST FRANCIS	CALIFORNIA	1928		
14	TETON	OHIO	1976	EARTH	

FIG. 3.2 DAM FAILURE FLOOD PEAK AS A FUNCTION OF DEPTH OF WATER

from the breach is a function of the valley shape and slope. These effects must be calculated by routing techniques in order to assess the extent of the downstream damage but, in general, it is well to appreciate that the dam-break flood wave can persist as a substantial water surface rise for a considerable distance downstream.

Several methods and models can be found in the literature for predicting the flood levels and arrival times of the flood wave at selected positions downstream of a breached dam. Oosthuizen (1986) concluded that, for damage assessment purposes, sufficiently accurate water profiles can be obtained by simplified calculations, such as those used in the program SMPBRK of the US National Weather Service. For more accurate analyses, sophisticated programs, such as DAMBRK, are available from the same organisation.

3.7 MODIFICATION OF DESIGN CRITERIA FOR EXISTING DAMS:

It is desirable that the criteria for determining the Safety Evaluation Flood (SEF) should be the same for existing dams as for new dams. There are, however, circumstances that require some relaxation of criteria for existing dams. Among these are the following:

a) 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;

b) Removal of the dam to remove the risk associated with a 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.

c) If it is known that a dam is to be abandoned within a short time, assessment of the risk of failure within a shorter life time would indicate that relaxed criteria could be applied to achieve the same level of safety (see Fig. 3.1); this would also be the case when assessing risks during construction of the dam or river diversion works.

d) 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.

If in evaluating the safety of a dam it becomes apparent that the appropriate criteria for a new dam would not be satisfied, further studies should be made on a site-specific basis to assess the risks involved and thus to decide what remedial measures would be essential.

4. DERIVATION OF FLOOD MAGNITUDES:

4.1 GENERAL:

For a given location and circumstance, clearly what are needed to enable one to reach a decision regarding the adequacy of the spillway of a proposed dam or of an existing dam under inspection are the following:

a) Ways of determining the frequency distribution of flood peaks

b) Ways of estimating the hydrographs of major floods to facilitate definition of not only peak discharges but also volumes of flood run-off

c) Techniques for routing flood hydrographs through reservoirs, with special consideration, where relevant, of possible surges associated with steeply-rising inflow hydrographs.

4.2 FREQUENCY DISTRIBUTION OF FLOODS:

Methods of deriving the frequency distribution of flood peaks from recorded or synthetic data are explained in hydrology textbooks and in the Flood Hydrology Handbook (SANCOLD, 1986).

Risk-based analysis, as indicated in Section 3.4.3, requires that annual probabilities or estimated return periods be assigned to extreme events. Since there are few places where flow records extend back as far as 100 years, it follows that statistical return periods cannot be accurately assigned much beyond about 50 years. The band of confidence embracing the frequency distribution curve widens rapidly for return periods longer than about half the length of the record.

Despite the fact that rainfall records are usually longer than those of river flow, much the same reasoning applies to flood hydrographs established by hydro-meteorological techniques in which flood response is deduced from the causative storm rainfall. Nevertheless, it is possible, by means of storm transposition and maximization techniques, to make a fairly good estimate of the Probable Maximum Precipitation (PMP) and, by unitgraph or other technique, to arrive at a reasonable estimate of the Probable Maximum Flood (PMF).

The PMF is the value to which the frequency distribution of flood peaks should become asymptotic at some extremely low probability or large value of return period which, for purposes of risk-based analysis, could be arbitrarily set at between 10^4 and 10^5 years. At a somewhat shorter return period, possibly in the range 500 to 1 500 years, would lie a value now referred to in South Africa as the Regional Maximum Flood (RMF) which can be estimated with acceptable reliability by methods developed by Zoltan Kovacs in Technical Report TR 105 (Department of Water Affairs, 1980). The work is a regional evaluation for Southern African flood peaks of the K values in the Francou-Rodier formula:

$$Q_{max} = 10^6 \; [A10^{-8}]^{1 - 0,1 K}$$

in which Q_{max} is the RMF (Regional Maximum Flood, m^3/s) and A the area of the problem catchment (km^2).

K values for the regions delineated on the map in Fig. 4.1 are as tabulated on Fig. 4.1 and in Table 4.1.

Table 4.1 : K values for regions shown in Fig. 4.1

Region No	A_{min} (km^2)	K value
1a	100	5,6
1b	10	5,4
1	10	5,25
2	10	5,0
3	20	4,6
4	100	3,6
5	500	2,5

Note: For areas smaller than A_{min} the formula over-estimates the RMF and other methods outlined in the Flood Hydrology Handbook (SANCOLD, 1986) should then be used.

The family of Francou-Rodier lines are envelopes of recorded flood peaks in the relevant regions. As indicated earlier the PMF would be higher than the RMF.

Max. flood peak region No	Ia	Ib	I	2	3	4	5
Rodier envelope "K"	5,6	5,4	5,25	5,0	4,6	3,6	2,6
Amin (km^2)	100	10	10	10	20	100	500

Note: if A < Amin estimate max. flood peak from PMP

Francou-Rodier equation

$$Q = 10^6 \left(\frac{A}{10^8}\right)^{1-0,1K}$$

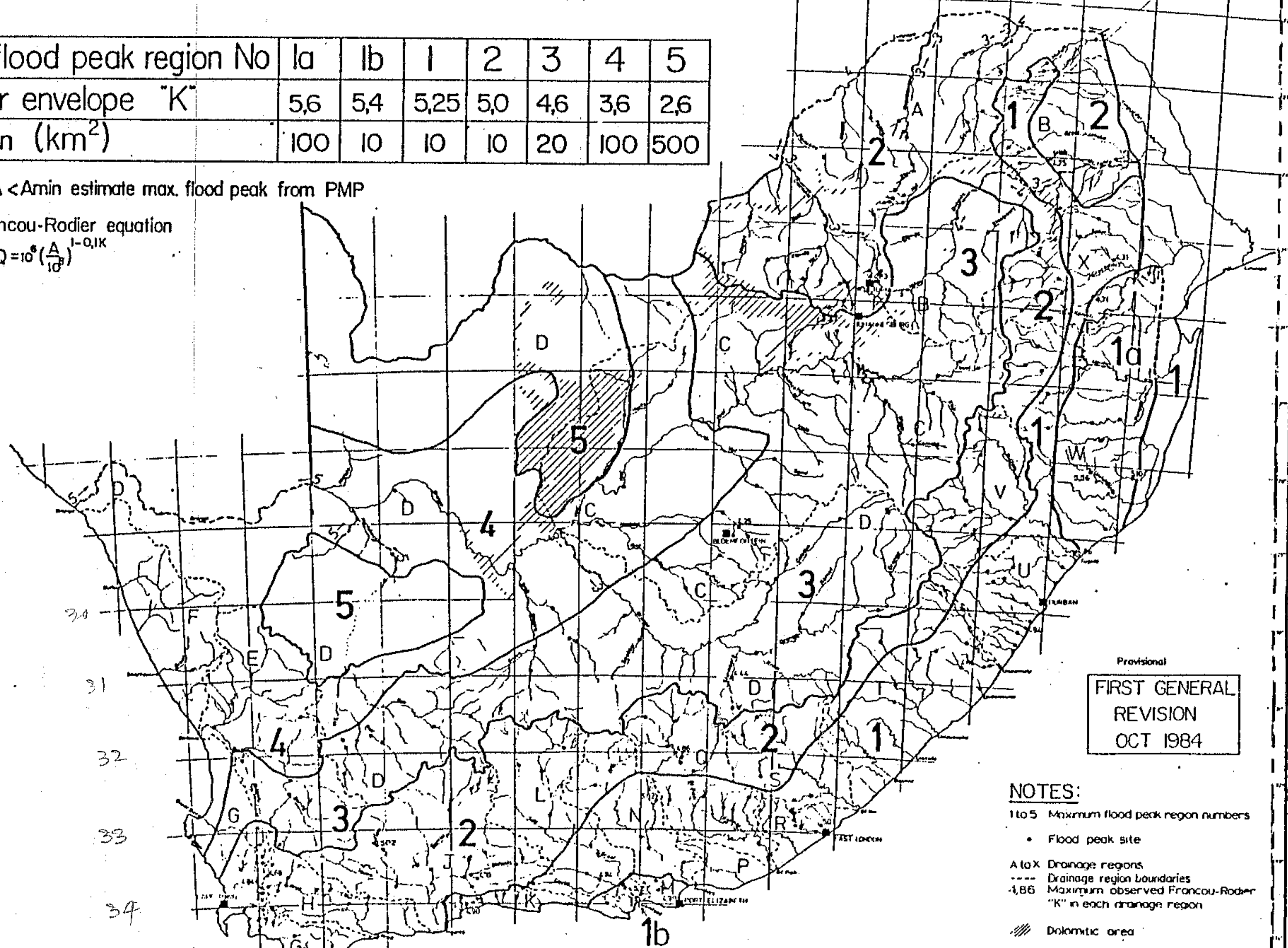


FIG. 4.1 MAXIMUM-FLOOD PEAK REGIONS IN SOUTH AFRICA

As will be noted in Chapter 5 both RMF and PMF have been adopted as the basis for recommending minimum values for the SEF for the various categories of dam.

Whatever the statistical frequency distribution that may have been fitted to an array of recorded or calculated flood peaks, one recognises that flood magnitude cannot continue to increase indefinitely with lengthening return period (or decreasing probability) but rather must reach the PMF asymptotically at some very large value of return period. In 1973 the ASCE Task Committee on Re-evaluation of the Adequacy of Spillways of Existing Dams (ASCE, 1973) recommended the arbitrary assignment of a return period of 10 000 years to the PMF. In the United Kingdom, however, a 10 000-year return period is normally assigned to the half-PMF.

Referring to Fig. 4.2 as an example one therefore may have a frequency distribution of flood peaks, derived either from an actual record or by hydrometeorological technique, up to say the 100-year event as well as a ceiling value - the PMF. For purposes of economic analyses one can then sketch in the frequency distribution curve between the 10^2-year and 10^4-year to 10^5-year return periods.

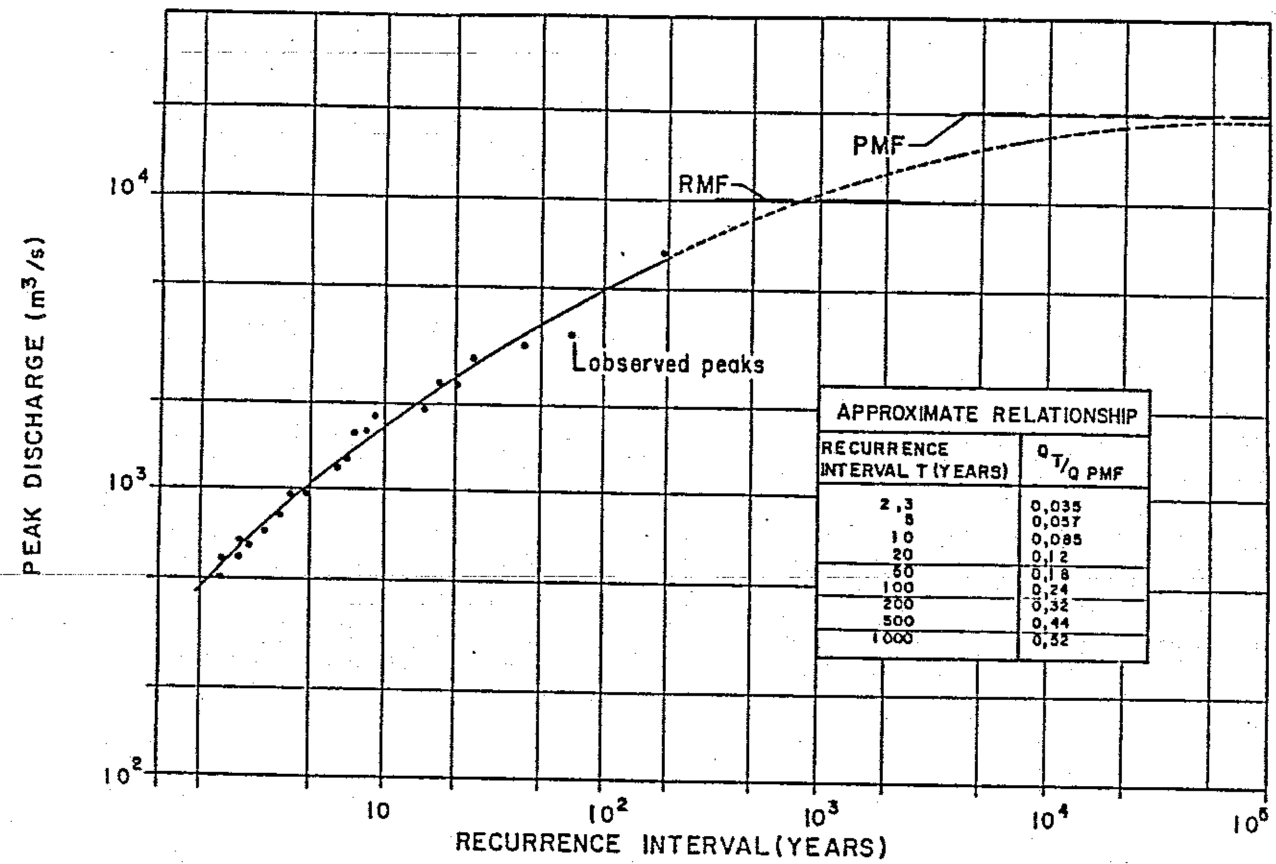


FIG. 4.2 EXTRAPOLATION OF A TYPICAL FLOOD FREQUENCY DISTRIBUTION

A rough guide to the frequency distribution of flood peaks in terms of PMF is tabulated on Fig 4.2 for purposes of preliminary economic optimization, as dealt with on Fig 5.2

4.3 FLOOD HYDROGRAPHS:

As indicated earlier, from a particular catchment there can be a wide variety of floods all having the same return period. Probably the only South African work on analysis of the joint probability distribution of flood peaks and flood volumes is that by Hiemstra and his assistants at the University of Natal. The diagrams in their Runhydrograph - Theory and Application for Flood Predictions (Hiemstra et al, 1979) suggest a way whereby hydrographs of given return period having a range of volumes and peaks can be generated.

Hiemstra's joint probability diagrams indicate that, particularly in the domain of rare floods, a hydrograph representing maximum volume for a given return period will have a peak significantly less than that of the hydrograph exhibiting maximum peak for the same return period. Conversely, the flood having the highest peak for a given return period will have a flood volume smaller than that of the maximum volume flood of like return period. Given the hydrograph of either the maximum peak or the maximum volume for a specified return period, therefore, one can reconstruct a range of inflow hydrographs to be routed through the reservoir for purposes of spillway design and downstream routing for damage assessment.

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 Flood Hydrology Handbook (SANCOLD, 1986). 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.

Unless the reservoir operating rules dictate otherwise, the reservoir must be assumed to be at not less than full supply level upon arrival of the flood hydrographs under consideration.

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 A DESIGN FLOOD AND SAFETY EVALUATION FLOOD:**

There is no single universally satisfactory approach for selecting an appropriate design flood or safety evaluation flood. In some instances the use of generalized design standards would suffice, while in other cases more elaborate site-specific analyses are essential. In every case due consideration should be given to aspects such as the hydrological characteristics of the catchment, the resistance of the dam to overtopping, the extent of the services dependent on the existence of the dam, the area that would be inundated in the event of a dam failure and the development in that area. In this section practical guidelines are given to assist the engineer to select appropriate values for designing or checking a spillway system.

5.1 GENERALIZED DESIGN STANDARDS:

From the structural 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, i.e. 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 anticipated. For such conditions narrower safety margins would be allowed, but it would nevertheless be an essential requirements that the structure should not fail catastrophically. In general, the two sets of loading conditions are defined on the basis of experience and sound engineering judgement. This same philosophy can well be adopted where loading relates to flooding and use of the RDF and SEF is in line with this thinking.

5.1.1 Definition of RDF, SEF and Freeboard

a) The recommended design flood (RDF) is the flood event which has the recommended return period and which places the highest surcharge on the dam. The RDF with appropriate freeboard provides the basis for design of the dam and spillway system.

b) The safety evaluation flood (SEF) is that incoming flood, which, when routed through the reservoir and spillway system, may cause substantial damage to the structure and surroundings but must not be such as to cause the dam to fail catastrophically.

Recommended minimum RDF and SEF values are tabulated in Table 5.1.

Table 5.1 : Recommended minimum values for design flood (RDF) and safety evaluation flood (SEF).*

	Dam size class	Hazard rating		
		Low	Significant	High
RDF	Small	20-50 yr	100 yr	100 yr
SEF		0,4 RMF 0,2 PMF	0,7 RMF 0,5 PMF	1,0 RMF 0,7 PMF
RDF	Medium	100 yr	100 yr	200 yr
SEF		0,7 RMF 0,5 PMF	1,0 RMF 0,7 PMF	1,5 RMF 1,0 PMF
RDF	Large	200 yr	200 yr	200 yr
SEF		1,0 RMF 0,7 PMF	1,5 RMF 1,0 PMF	1,7 RMF 1,1 PMF

* RMF from TR105 (DWA, 1980) and PMF from HRU 1/72 (HRU, 1979); both values to be calculated and judgement to be exercised in the selection of the final value.

c) __Freeboard__ is the vertical distance from the normal reservoir still water surface or full supply level (FSL) to the non-spill crest of the dam, excluding parapets and wave barriers proud of the crest.

d) __Minimum freeboard__ is defined as the difference in elevation between the non-spill crest and the maximum still water reservoir surface that would result should the RDF occur with the outlets and spillway functioning as planned but without wind set-up, waves and surges. This minimum freeboard is often referred to as the dry freeboard and the rise of water surface from FSL to RDF level as the wet freeboard. See definition sketch, Fig. 5.1.

5.1.2 Freeboard allowances

The dry 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 these are:

Wind generated waves
Wind set-up
Seiches (resonance)
Earthquake-induced waves
Landslide-induced waves
Flood surges

Flood surges due to upstream dam failures (cascade effect) are not included here. Such occurrences must be taken into account in estimating the SEF.

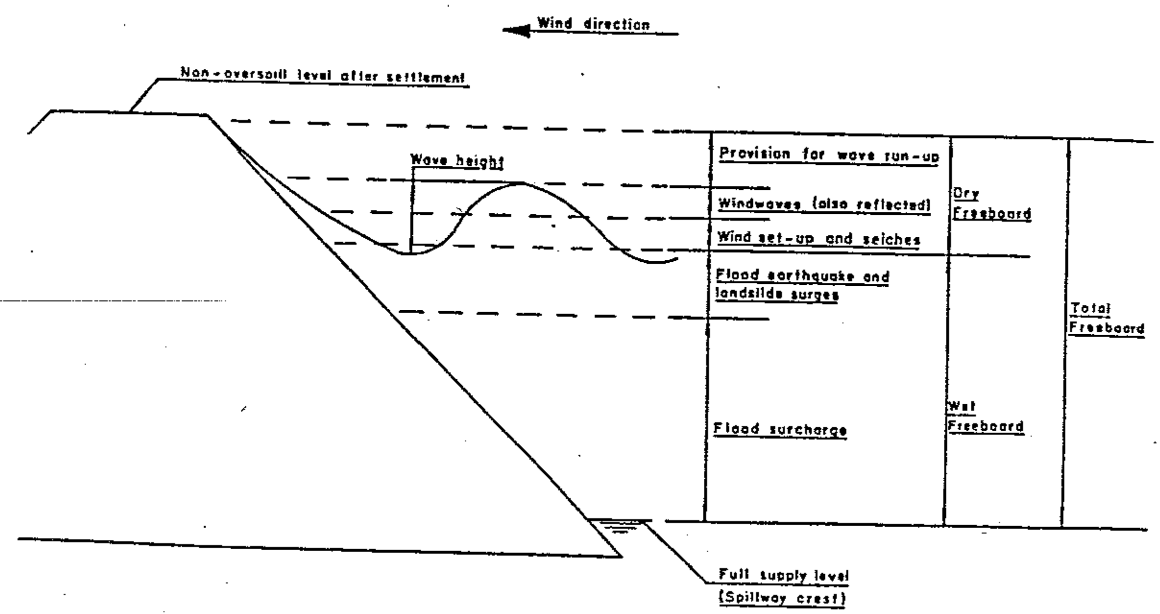


FIG. 5.1 DEFINITION SKETCH ILLUSTRATING FREEBOARD

Wind waves superposed on wind set-up normally play the dominant role in determining the required dry freeboard.

Wind data can be acquired in the form of 25-, 50- and 100-year maxima for 1-hour periods from Weather Bureau publication WB 38 (Weather Bureau, 1975).

Wave heights and wave run-up can be calculated from wind data. Useful references are the Institution of Civil Engineers' Floods Guide (1980), Saville et al (1962), US Shore Protection Manual (1978).

According to Kovacs et al (1984), corroborated by Shand (1985), the incidence and behaviour of surges in reservoirs can be estimated with acceptable accuracy. During extreme flood, for instance, the outflow peak may increase momentarily by 20 - 25 per cent as the result of rapid water surface rise due to a surge. At some South African dams it has been deduced from analysis of the autographic stage hydrographs that sudden rises of nearly a metre may have occurred as steep-fronted flood waves entered the reservoir.

Wind set-up and seiches

Here, references are sparse for inland water bodies and tend to be contradictory. In any event the values are generally small compared with wave heights. Values normally considered to be adequate to allow for wind set-up and weather-induced seiches follow:

Category I	dams :	0	-	0,3 m
Category II	dams :	0,3	-	0,45 m
Category III	dams :	0,45	-	0,6 m

Minimum freeboard allowances

Various combinations of conditions should be considered in establishing the desired elevation of the non-spill crest.

Typical test combinations are:

i) RDF + wind set-up + 25-year wave conditions + flood surges

ii) 20-year flood level + wind set-up + maximum wave height + flood surges

iii) FSL + earthquake wave

iv) RDF + landslide (if landslides are likely to be caused by excessively high water level).

Spillage under any of these conditions must not endanger the structure. For an embankment dam, if no camber is provided, allowance should be made for settlement due to consolidation of foundation and embankment materials and, for existing dams, the level of the crest should be checked for possible low spots that could effectively reduce the available freeboard.

Zero dry freeboard is generally considered acceptable for the SEF except that, for an embankment dam, the maximum still water level should not exceed that of the top of the impermeable core.

Overtopping of concrete dams by the SEF is permissible provided stability conditions are not violated and provided the foundations are not endangered by scour.

5.1.3 Recommendations for the application of general design standards

To satisfy the requirement that no significant damage be caused during normal operating conditions the RDF values in Table 5.1, with the recommended freeboard allowances, may be adopted for the design of a new spillway or for the re-design of an existing spillway. The design must then be checked to ensure that the recommended SEF can be accommodated without causing catastrophic failure by overtopping.

The SEF values in Table 5.1 are also intended as a sieve to identify existing dams that require further investigation of spillway adequacy. Spillways that can accommodate the indicated SEF, suitable routed, without causing catastrophic damage need no further investigation. For spillways that fail to meet the criteria, site-specific analyses are recommended to ascertain the degree of inadequacy and required remedial work.

The SEF values may also be used to evaluate the adequacy of the spillway of a less important new dam but site-specific analysis is strongly recommended for evaluation of the safety status under extreme flood conditions for all high hazard dams as well as for medium and large dams having significant hazard ratings.

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

5.2 ZERO INCREMENTAL IMPACT APPROACH:

The cornerstone of this method is that spillway capacity is determined from an evaluation of the incremental impact experienced in the area downstream of the dam. The minimum criteria set by the general design standards may be considered as the starting point of the analysis and the endeavour is to assess whether the risks associated with the project would justify relaxing or tightening of 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 the design and safety evaluation of high hazard dams as well as for medium and large dams having significant hazard ratings.

The following require to be evaluated:

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 which just causes overtopping of the embankment. For concrete dams the IFF would be derived after consideration of strucutral stability and downstream threat to foundations under overtopping conditions.

Imminent Failure Inundations: A plan view of the downstream areas inundated by the IFF routed through the reservoir.

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

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. Obviously included is the effect of the dam-break flood wave on areas that would already be inundated by the IFF. Loss of life need not be converted to a monetary value.

Zero Incremental Impact Flood (ZIF): The ZIF is the minimum flood for which failure of the dam would cause no significant increase in the downstream damages and potential loss of life, i.e. zero incremental impact under present and forseeable future conditions. This ZIF has to be determined by trial.

In the case of an existing dam the following procedure should be followed:

a) Determine the SEF recommended for the category of dam in Table 5.1.

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. 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 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.

The following aspects are relevant to the assessment of the IFF as listed by ANCOLD (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 downstream of the spillway energy dissipator, can be accepted provided they will not cause 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 low 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 reconstruct them.

iv) For concrete dams a 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.

5.3 RISK-BASED ANALYSIS

As will be recalled from the introductory remarks in Section 3.4.3, prerequisites to risk analyses are frequency distributions of both flood peak and flood volume, i.e. the appropriate flood hydrographs. By routing the hydrographs downstream one can establish the frequency distribution of downstream damage <u>without dam</u>. The hydrographs can then be routed through the reservoir and downstream river reaches with various capacities of spillway system, with (if appropriate) reservoir operation rules, to establish the frequency distribution of <u>with dam</u> damages 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 frequency distribution of failure and therefore of damages due to failure. By integrating the frequency distributions (that of damages with and without dam failure) one can establish for each the total damage, or the average annual damages. 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 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 to yield the annual damage cost.

To the annual damage cost must be added the annual cost of providing the spillway system to yield the total cost, as illustrated in Fig. 5.2. The minimum total cost indicates the optimum design.

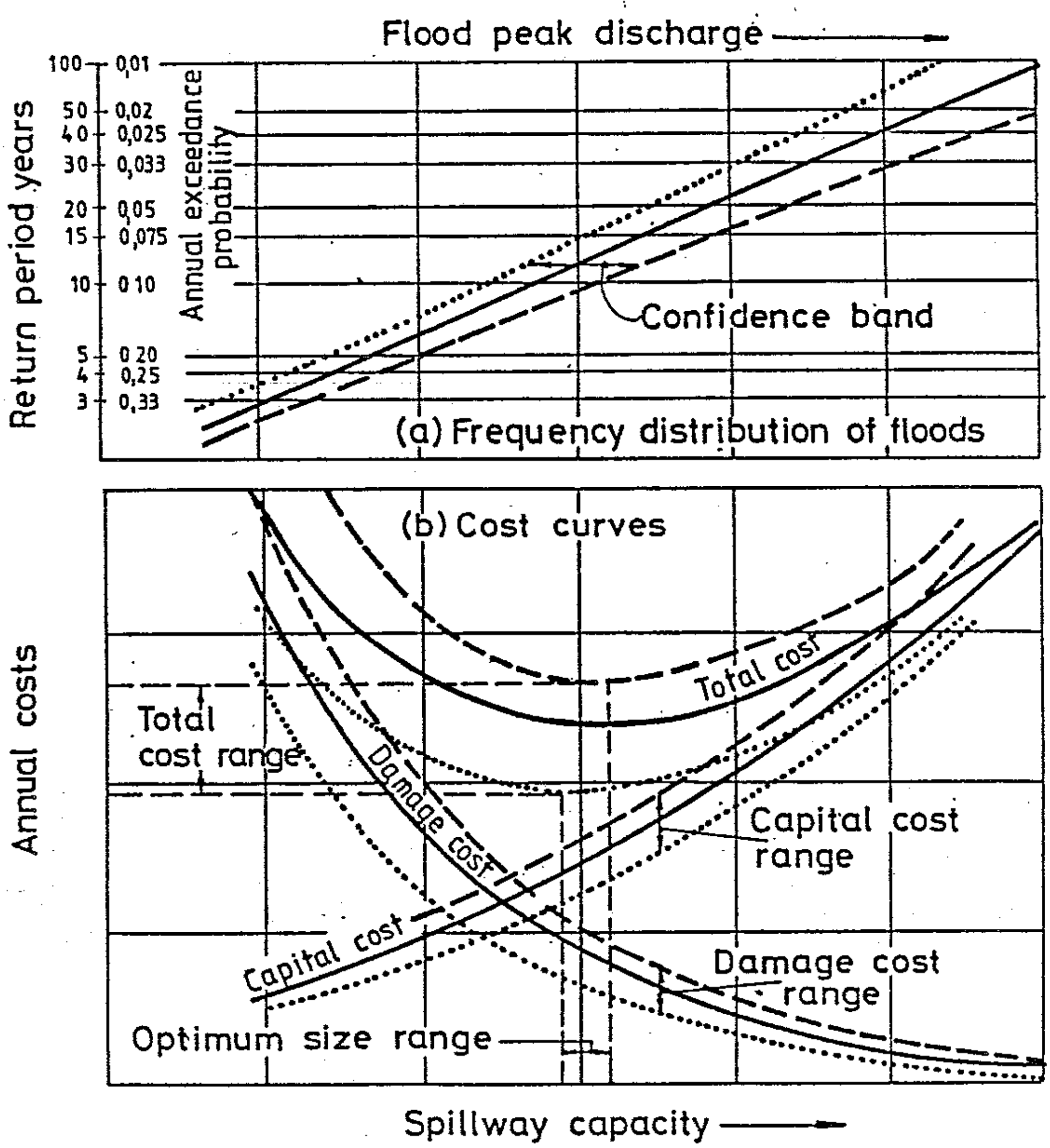


FIG. 5.2 ECONOMIC OPTIMIZATION OF SPILLWAY CAPACITY

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, e.g. 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 multi-dimensional damage functions that can be used by the engineer to convert depth (or area)-duration-frequency relationships (which are what the hydrology/hydraulic analysis can yield) to damage-frequency relationships which are what the risk-analyses demand 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. 5.2 illustrates.

Where the property losses likely to result from dam-break are intolerably high, the damage costs curve in Fig. 5.2 will decline slowly forcing the minimum, that is the optimized 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 (i.e. the PMF).

Similar evaluation techniques could be used for the other aspects such as loss of life, socio-economic losses and environmental damage. (See Oosthuizen, 1986).

6. CONCLUDING REMARKS:

It is well to emphasize that the Guidelines do not constitute a Code of Practice and that there is no intention to prescribe the use of any specific hydrologic technique. It is nevertheless expected that modern techniques be adopted and that sound engineering judgement be exercised in deciding upon design flood and safety evaluation flood values, as well as in reaching decisions as to whether an existing spillway is to be considered satisfactory or requiring to 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 minimum values in Table 5.1. It follows, however, that adoption of lower values must be strongly motivated by carefully checked site-specific analyses.

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