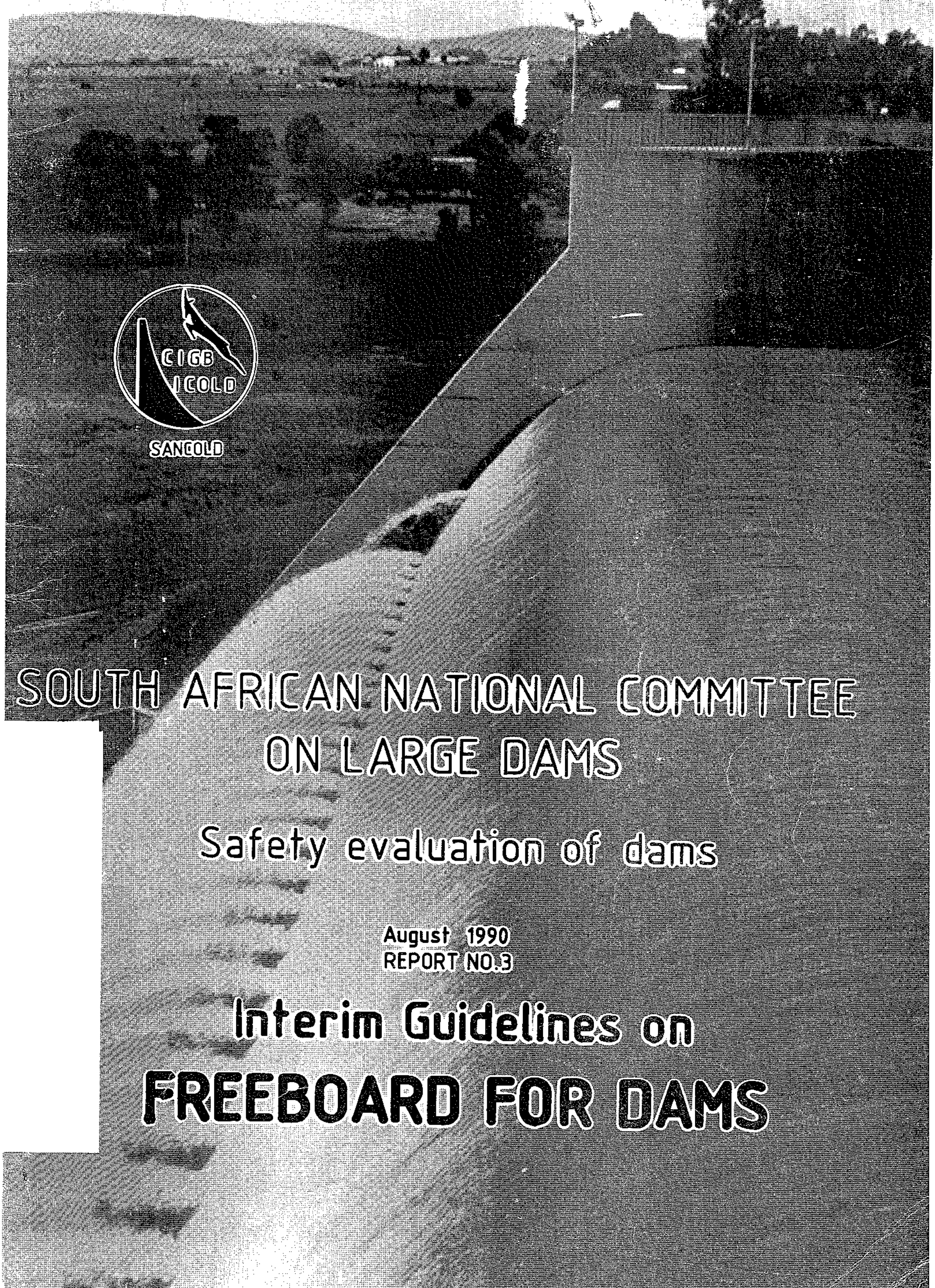


SANCOLD

# SOUTH AFRICAN NATIONAL COMMITTEE ON LARGE DAMS

## Safety evaluation of dams

August 1990
REPORT NO.3

# Interim Guidelines on FREEBOARD FOR DAMS

SANCOLD

CIGB ICOLD

# Safety Evaluation of Dams

## Report No. 3

# Guidelines on Freeboard for Dams

*published by*

The South African National Committee on Large Dams
P.O. Box 3404
Pretoria 0001

*September 1990*

# 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 approved professional engineers will be called upon to handle the design, 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 was concerned with safety in relation to floods while this guideline addresses the freeboard requirements for dams.

These guidelines have been prepared with great care, taking into account current practices followed in other countries. It was considered appropriate to allow time for users to comment on possible unforeseen problems experienced in the application of these interim guidelines. Comments should be forwarded to: The Secretary, SANCOLD, P.O. Box 3404, Pretoria 0001, before November 30, 1991.

It should be stressed that the aim of the guideline is to highlight the philosophy and approaches taken in determining freeboard requirements for dams. Calculation methods are not detailed and the user should refer to the appropriate literature references. Some calculation aids are provided for convenience in the Appendix.

CHAIRMAN

SANCOLD

jmj/1f/1915F/0700A

# INTERIM GUIDELINES ON FREEBOARD FOR DAMS

## INDEX

# 1. INTRODUCTION

Freeboard provides a margin of safety against overtopping failure of dams. Sufficient freeboard has to be provided so that possible spillage over the non-overspill crest of a dam will not endanger the structure and/or human lives. Freeboard allowances for concrete dams can in general be less conservative than those for embankment dams because of the greater resistance of concrete dams and, in most cases, their foundations to erosion damage resulting from overtopping. Nominal 'extra' freeboard or safety margin is often provided in addition where hydrological uncertainties do not permit accurate calculations. These Guidelines are drawn up as an aid to designers and dam safety evaluators. It should be noted those the definition of freeboard and guideline values in this document differs from that in Ref. 1. The SANCOLD guidelines are dynamic documents and these guidelines replace the previous ones.

## DEFINITION OF FREEBOARD

| |
|---|
| The total freeboard for a dam is defined as the vertical distance between the normal full supply level (FSL) and the nominal non-overspill crest of the dam, excluding camber, but including adequately designed parapets and wave barriers proud of the crest. Freeboard is usually divided into two components namely the flood surcharge rise above the FSL, the primary component, and a secondary component allowing for wind, wave and surge effects. |

In the calculation of total freeboard adequate provision must be made for the worst reasonable combination of disturbances which may play a role. The most important components for total freeboard calculation are the following all of which are not cumulative:

(a) Flood surcharge (with allowance for possible gated spillway or bottom outlet malfunctioning).

(b) Wind-generated waves (with allowance for effects such as wave-reflection, wave run-up).

(c) Set-up due to wind.

(d) Seiches (resonance effects).

(e) Earthquake-induced surges.

(f) Landslide-induced surges.

(g) Flood-induced surges.

Some dams are equipped with large capacity bottom outlets and these must also be taken into account when considering the freeboard.

Upstream dam failures (cascade effect) should be taken into account in estimating the Safety Evaluation Flood (SEF) and are therefore not listed here. Vide Ref I: Interim Guidelines on safety in relation to floods, SANCOLD, 1986.

Figures 1, 2 and 3 illustrate the above concepts for the non-overtopping cases of an embankment and a concrete dam.

## 2. FREEBOARD CALCULATION PROCEDURE

### 2.1 FLOOD AND FLOOD SURCHARGE CALCULATIONS

The capability of safely passing floods (RDF* and SEF**) is paramount in any dam's design. For guidance on the determination of the incoming design flood, and the safety evaluation flood Refs. 1, 2, 6, 20 and 21 should be consulted. Flood absorption takes place due to the temporary storage space above FSL*** which is mobilized during the passage of the flood. The spilling flow rate is uniquely determined by the head over the crest and the spillway crest dimensions and gate openings, if applicable. The maximum rise, or surcharge is thus calculable and the procedure for determining its magnitude is given in Refs.2, 3 and also 2.1.1 below.

#### 2.1.1 Flood surcharge

Flood surcharge to be provided is calculated as follows:

The recommended design flood, the safety evaluation flood or other floods with given return periods are chosen, as estimated from several methods given in the SANCOLD interim guidelines (Ref.1) and the Flood Hydrology Handbook (Ref. 2, Alexander).

Flood surcharge calculations (Ref. 7, USBR) are based on certain required assumptions and utilise a flood-absorption calculation program. The relationships are:

(i) A relationship between storage volume and level above FSL,

(ii) A discharge rating curve, i.e. discharge versus water level above FSL for uncontrolled spillways and similar relationships for various gate settings for controlled spillways.

(iii) Operating rules for handling floods where flood gates and/or bottom outlets are involved.

(iv) The initial water level condition (normally taken at FSL).

(v) The calculation method i.e. horizontal water surface or backwater.

(vi) The shape of the hydrograph.

It is realistic and normal practice to make use of the available temporary storage space above full supply level necessary for surcharge for flood attenuation.

The following are cases where this benefit should not be taken advantage of:

(i) where the attenuation for RDF is less than 10 per cent or in the case of category I dams. In this case it is suggested that for simplicity the outflow peak be regarded as equal to the inflow peak and adequate freeboard be provided to cope with the unattenuated peak.

---

* RDF = Recommended Design Flood

** SEF = Safety Evaluation Flood

*** FSL = Full supply level

(ii) where the time to outflow peak is short as in the case of controlled outlet structures. The gate operator may be prevented from operating the gates in time. Adequate freeboard to cope with various scenarios of this kind should therefore be provided.

2.1.2 Practical aspects in relation to flood surcharges and freeboard

Some aspects to be taken into consideration in the determination of flood surcharges and freeboard and uncertainties thereof are:

(i) Uncontrolled spillway. Here surcharge and freeboard are accurately calculable and uncertainties are minimal.

(ii) Combinations of uncontrolled + gated spillways. Here surcharge and freeboard are less predictable relating to the reliability of operation. An advantage is that pre-releases are possible which permit a reduction in total water level rise. In the case of maloperation of some of the gates, adequate freeboard must be provided for this flood discharge by way of the uncontrolled spillway.

(iii) Fully gate-controlled spillway. Maximum releases, where outflow equals inflow and pool level remains constant, are possible. If gates are not to be overtopped, which is likely to happen when only some gates are raised, all gates should rather be raised by equal amounts. The surcharge level must be below or at dam crest level. Advantage can be taken of the raised gate-leaf top when the gates are opened for the achievement of flood absorption. This type of operation increases, however, the water level and increases risk levels.

(iv) Fully gate-controlled but without additional flood freeboard (surcharge). Total reliance on effective gate operation must be made. All gates must therefore be able to be raised together, which it is not suitable for barrages where constant pool levels must be maintained. Special rules apply to barrages which have to pass floods in an unhindered way and in the process may even pass floods earlier and at higher levels than would have occurred naturally.

(v) The cascade effect or the sequential breaking of dams, all on the same river course, may aggravate the process. In the freeboard allowance for each dam consideration should be given to the possible inclusion of allowance for the eventuality of a dam break higher up in the catchment, which may be beyond the jurisdiction of the authority administering the particular dam under consideration. In such cases a site-specific approach should be used. A risk analysis which takes into account the shortcomings of the upstream dam would be a useful means whereby to assess the implied risks, legal aspects and hence guide the decision makers.

## 2.2 WIND WAVES AND RUN-UP

Wind generated waves and the corresponding wave run-up on the dam wall must be taken into account in the freeboard calculation. The factors which govern these effects are discussed below.

### 2.2.1 Selection of design wind speed

A prerequisite for calculation of wave height and wave run-up is the selection of a design wind speed. This selection must take into account the category of dam and hazard potential as well as the severity of the flood under consideration as dealt with in Section 3.

The 50-year design isopleths for hourly mean wind speeds as prepared by Milford (Ref. 8) and given in Figure A1 of Appendix A can be used as a basis for selecting a design wind speed. If wind data is available for meteorologic stations in the vicinity of the terrain, it should be analysed to provide a more accurate selection of the design wind speed for the specific site. Topography plays a major role in the transposition of wind data and could either increase or decrease the wind speed depending on whether the data is transposed to a relatively flat or irregular terrain. It should also be noted that major floods in the northern and eastern low lying areas of the country are caused by cyclonic storms with very high wind speeds close to the cores of the cyclones. These cyclonic cores are not stationary and can thus result in the most critical wind direction towards the dam wall occurring during the passage of the flood. Funnelling effects i.e. winds blowing along valleys onto an exposed water surface can cause higher local wind speeds.

For a given atmospheric pressure gradient and meteorological conditions, wind speeds over water are higher than over land and Saville et al (Ref. 10) determined the following approximate correction factors :

WIND SPEED RELATIONSHIP - WATER TO LAND

| Effective fetch (km) | 1 | 2 | 4 | 6 | 8 (or more) |
|---|---|---|---|---|---|
| Wind speed ratio $\frac{\text{Over water}}{\text{Over land}}$ | 1,10 | 1,16 | 1,23 | 1,28 | 1,30 |

The analysed observed surface wind (over land) data from the Weather Bureau publication WB38 (Weather Bureau, 1975) given below can also be used as a basis for selecting a design wind speed. (Ref. 9.)

HIGHEST OVER LAND HOURLY WIND SPEED (m/s) TO BE EXPECTED IN 25, 50 AND 100 YEARS

| STATION | 25 years | 50 years | 100 years |
|---|---|---|---|
| ALEXANDER BAY | 25,3 | 25,9 | 26,4 |
| BEAUFORT WEST | 29,8 | 31,6 | 33,5 |
| BLOEMFONTEIN | 22,4 | 23,8 | 25,4 |
| D.F. MALAN | 20,0 | 21,1 | 22,1 |
| DURBAN | 24,9 | 26,6 | 28,4 |
| EAST LONDON | 21,3 | 22,2 | 23,2 |
| ESTCOURT | 16,7 | 17,7 | 18,7 |
| GEORGE | 20,8 | 22,2 | 23,6 |
| JAN SMUTS | 21,2 | 22,7 | 24,3 |
| KEETMANSHOOP | 21,7 | 22,8 | 23,9 |
| KIMBERLEY | 19,6 | 20,7 | 21,8 |
| MIDDELBURG | 16,6 | 17,4 | 18,2 |
| PIETERSBURG | 20,3 | 22,4 | 23,4 |
| PORT ELIZABETH | 21,1 | 22,3 | 23,0 |
| PRETORIA | 15,6 | 16,4 | 17,2 |
| UPINGTON | 21,4 | 22,4 | 23,4 |
| WINDHOEK | 17,1 | 18,6 | 20,2 |

### 2.2.2 Effective fetch

The distance over which the wind acts to generate waves (referred to as the fetch) is affected by the varying width of the body of water. Calculation of the effective fetch was developed by Saville et al (Ref. 10) as the average reach over a 90° arc across the water from a point of concern at the dam wall. A typical example of computation of effective fetch is shown in Figure A2. The water level in the dam should correspond to the associated flood level under consideration when the effective fetch is determined.

### 2.2.3 Design wave height

The design wave heights to be selected for determination of wave run-up for wind freeboard allowances, are based on the so-called significant wave height, defined as the average wave height of the highest one-third of the waves in a wave spectrum or in a regular wave train. This significant wave height is equalled or exceeded by 13% of the waves generated by a particular (design) wind speed.

Wave heights are governed by the reservoir water depth, effective fetch, wind speed, and the duration of the wind acting at the design speed along the fetch towards the dam wall.

The characteristics of waves in deep water differ from those for waves in shallow water with shallow water being defined as a water depth less than about one-third to one-half the wave length. In general, wind waves on dam reservoir surfaces fall within the category of deep water waves and this aspect should be confirmed by estimating the significant wave period as provided for by Saville et al (Ref. 10) and shown in Figure A3 and then calculating the wave length for deep water in metres as $L = 1{,}56T^2$ with T being the significant wave period in seconds.

The minimum wind duration is the length of time required for the wind (at design speed) to blow in essentially the same direction over the fetch towards the dam wall, for fully developed wave conditions at the dam wall. Figure A4 is a diagram reproduced from the UK Institution of Civil Engineers' Engineering Guide on Floods and Reservoir Safety (Ref. 11) for determining the significant wave height for a given fetch and wind speed, as well as the corresponding minimum wind durations as transposed from the original diagram by Saville et al (Ref. 10).

The selection of a design wind speed is associated with a wind duration of one hour which should be appreciably longer than the minimum durations required for generating the design wave spectrum. However, a wind lasting for the minimum duration only will not be critical with regard to wave damage potential and generally speaking the one hour duration wind speeds as given in Figure A1 should be used.

The extent to which waves can be permitted to splash over the crest of a dam depends on the type of dam. For this reason the following design wave heights expressed in terms of significant wave height have been proposed for various types of dams in the UK Institution of Civil Engineers' Engineering Guide on Floods and Reservoir Safety (Ref. 11):

| TYPE OF DAM | DESIGN WAVE HEIGHT IN TERMS OF SIGNIFICANT WAVE HEIGHT |
|---|---|
| Concrete dam | 0,75 |
| Rockfill dam with road on crest | 1,0 |
| Earthfill dam with road on crest and selected grass on downstream slope | 1,1 |

The above multiplication factors should also be seen in relation to the probability of exceedance of a particular wave height. A design wave height with a multiplication factor of 1,1 could on average be equalled or exceeded by 9% of the waves in the wave spectrum as against a corresponding exceedance of 32% where a factor of 0,75 is used.

Site-specific conditions

The ability of wind to generate waves, wave run-up and wind set-up is sensitive to site-specific conditions at the reservoir which might give rise to results that differ from the theoretically calculated values.

These site-specific conditions are:

(i) reservoir depth;

(ii) direction of prevailing winds with respect to orientation of dam relative to the effective fetch;

(iii) funnelling effect of topography adjacent to the reservoir;

(iv) shape of the shorelines; and

(v) vegetation surrounding the reservoir.

### 2.2.4 Wave run-up

Wave run-up is defined as the difference in vertical height from stillwater level (that would prevail without waves) to the maximum level attained by run-up of the design wave against the dam wall.

The wave run-up ratio (wave run-up/design wave height) is a function of slope and texture (roughness and permeability) of the upstream face of the dam wall. The wave run-up ratio diagram in the UK Institution of Civil Engineers' Engineering Guide on Floods and Reservoir Safety (Ref. 11) as reproduced in Figure A5 can be used to determine the wave run-up. Distinction is made between three different surfaces, namely:

(i) a smooth surface representing a concrete lined, soil-cement lined or grass-covered surface;

(ii) a rough open-jointed stone pitching or shallow rubble surface

(iii) a thick permeable rip-rap.

Interpolation is required if the design conditions do not relate to one of the above surfaces.

It should be noted that if the wave propagation direction is not normal to the dam wall, the slope of the wall face to be used in that case should be the slope in line with the wave propagation direction. The elevation of the wave run-up can thus be calculated in the wave propagation direction.

As shown in Figure A5 the wave run-up ratio for a vertical upstream dam wall face in deep water is 1,0, but it is to be noted that this ratio could approach 2,0 in some circumstances in shallow water. This could be the case when a standing wave, is formed at the wall face due to the reflection of a wave spectrum from the vertical or nearly vertical face as described in the US Shore Protection Manual (Ref. 12). It would, however, appear that a ratio higher than 1,33, or 1,5 at the most, need not be considered for dam design except for rather special cases where an upstream vertical face in shallow water is exposed to very high wind waves. (See also Venter, Ref. 13, for experimental data on wave run-up and Van Rooyen, Ref. 5, for embankment protection.)

## 2.3 WIND SET-UP

Saville et al (Ref. 10) defines wind set-up as the result of surface water being driven in the downwind direction with the wind blowing over a water surface exerting a horizontal stress on the water. This results in a build-up of water at the leeward end of an enclosed body of water, and a lowering of the water level at the windward end. It should be assumed for design purposes that the wind (at design speed) will be blowing towards the dam and causing maximum wind set-up at the wall.

In the case of wind set-up the effects may be transferred around substantial bends in a reservoir and the fetch length in wind set-up computations could be substantially longer than the effective fetch used for computing wave heights. Usually the fetch length in wind set-up computations is taken as 2 times the effective fetch used for wave height computations.

The rise in water level above the stillwater level at the dam wall due to wind set-up can be determined from the following formula given by Saville et al (Ref. 10):

$$S = \frac{V^2 F}{4850\ D}$$

in which: S is the rise above stillwater level (m)
V is the design wind speed (m/s)
F is the fetch (km), normally equal to 2 times the effective fetch used for computing wave heights
D is the average water depth in the basin (m) along the fetch, F

## 2.4 SEICHES AND SURGES

These are periodic oscillations or unique rises in reservoir level due to atmospheric pressure variations or sudden inflows.

These phenomena have been known to exist for a long time but have been difficult to quantify. Recent observations by Kovacs, Roberts and Jordaan (Ref. 18) indicate that there may be oscillations of the order of 0,5 to 1 m in height in moderate size reservoirs (Floriskraal Dam) and more than 1 m in larger ones like Pongolapoort and Hendrik Verwoerd Dams. An allowance of 0,5 to 1 m should be made in freeboard calculations in these cases.

A long-period oscillation or "seiche" might persist long after the waves have died down. According to Raichlen (Ref. 16) a reservoir basin is set into oscillation in one or more of its natural modes by externally arriving long-period waves. Such oscillations die down by being absorbed by the reservoir sides. Surges could also be caused by debris blockages being washed away by a subsequent flood (e.g. such as might occur at bridges). The method of dealing with surges in reservoirs has been analysed by Kovacs et al, (Ref. 18), with a simplified analysis of flood surges given by Shand (Ref. 19) in a subsequent discussion.

## 2.5 WAVES AND SURGES DUE TO EARTHQUAKES

Earthquake excitation could cause rapid oscillatory motions of the body of water resulting in standing waves or seiches, calculation of the amplitudes of which for any given set of conditions would be an extremely involved procedure. In mildly seismic areas such calculations do not seem warranted. Tremors would have to be of long period and persisting over several minutes to create substantial surface oscillations in the body of water.

Under severe earthquakes the potential damage suffered by the dam's structure itself or that caused by landslides into the reservoir could far outweigh the overtopping hazard of the water being set into a wave-like motion. Sustained accelerations of 0,1 or 0,2g could theoretically set up water surface slope variations of 0,1 and 0,2 at the surface. At the periods of earthquake "ground" waves of the order from 1 to several seconds only short wavelength water waves would be created: $L = 1{,}56\ T^2$, where L = wavelength and T = wave period in seconds. The heights of these waves could not be larger than 1/7th of this wavelength, i.e., for example for a 3 sec. period wave a wavelength of 15 m and a maximum wave height of 2 m is conceivable.

Dispersion and reflection could be assumed to be counteractive to each other, hence a 2 m wave and surge freeboard should be sufficient to allow for earthquake water waves. Allowance for earthquake seiches or body oscillations of the water mass could be considered included in other allowances for seiching such as for meteorological disturbances, low pressure system transit over larger reservoirs, or surge oscillations caused by sudden flooding events; since the two unconnected situations would be unlikely to coincide.

## 2.6 WAVES AND SURGES DUE TO LANDSLIDES

Reservoirs surrounded by steep unstable slopes are subject to landslides which can displace material into the reservoir causing volumetric displacement of water over the dam and setting up surges and waves in the water body. Volumetric displacement by material can be dealt with as an incoming volume and subsequently leading to a capacity reduction. Calculation of the slip volume possibly threatening a dam can be made from a geological analysis of the surrounds of a basin. Three types of slips occur according to Vischer, Ref. 15 namely (i) falls, such as rock masses off a cliff, with a low volume and high energy intensity, (ii) slides such as slip-circle type slides, also known as debris-flow and (iii) more gradual flows which are associated with long time intervals.

The surges and waves caused by landslides in enclosed bodies of water can be very severe, Davidson et al (Ref. 17). Surges or long period waves can give rise to extreme seiching oscillation, run-up and overtopping. It is difficult to calculate the wave heights and periods exactly but Vischer (Ref. 15) has given for a two-dimensional canal of constant width a relationship between wave height, water depth ratio and the displacement number which may be utilized for freeboard calculations.

# 3. FREEBOARD ALLOWANCES AND SPECIAL REQUIREMENTS FOR DIFFERENT TYPES OF DAM

## 3.1 GENERAL REMARKS

### 3.1.1 Different sizes and categories of dams and hazard potentials.

Since there are many more dams in category I, i.e. small, low risk dams than in Category II and III (medium, large to high risk dams), the probability of a failure of any one of them in any given period is far greater than the risk of failure of one of the dams in the higher categories. The careful consideration of risks of all modes of failure for category I dams are often overlooked since controls are less stringent.

The category II and III dams should be designed in a more circumspect way, because there is more scope for uncertainty factors to enter and if any of these should be overlooked and a failure should result it is potentially far more damaging to life and property.

The factors relating to unforeseen events (i.e. hydrological and geological conditions) affecting freeboard and the suggested counteractive steps are:

(i) Lack of reliable hydrological data: Design with greater safety margin, e.g. freeboard and auxiliary spillway capacity, and with upper-catchment dam-breaks in mind.

(ii) Unknown geological or seismic conditions: Design also with slips, slides and rock-falls in mind.

(iii) Unknown human factors (operator, political, future ownership): Design with simplicity in mind, uncontrolled spillway, fuse plug; overtoppable gates, if gates are to be used at all.

(iv) Oversophistication: Do not rely heavily or solely on flood-warning systems, telemetry, automation, but provide a simple back-up system that will work, should all else fail, e.g. design a safely overtoppable dam.

(v) Design risks: Incorporate second-line defences e.g. a downstream slope resistant to erosion, an upstream slope with wave reflecting or absorbing characteristics and ample wing walls and sufficient camber on embankments.

(vi) Hazard potentials: The above remarks apply equally well to dams located in areas with greater hazard potentials, in that more thorough checks on the adequacy of safety measures should be taken in cases where high risks to life and property exist.

### 3.1.2 General considerations

Other conditions having implications on freeboard, are:

(i) Controlled versus uncontrolled spillways

Uncontrolled spillways lend themselves to a much more definite calculation of their behaviour and no element of uncertainty exists due to time of flood arrival or necessity for operation. Controlled spillways, on the other hand, introduce various risks such as operator error or malfunction of automatic systems. Therefore, where controlled spillways are incorporated, a larger margin of safety should be allowed in the freeboard and related aspects. Pre-release to draw down the water surface in advance is often mooted as a virtue of gated dams but seldom implemented locally.

(ii) Accuracy of hydrological data

In the case of short-duration records or inaccurate observations ample redundancies must be built in, i.e. sufficient uncontrolled spillway capacity to cater for the safety evaluation flood (SEF). Where gated spillways are to be used for other reasons, an auxiliary spillway capable of taking a portion of the gated flow discharge at HFL should be provided for the eventuality of inoperability of the gates.

In the case of an embankment dam, an auxiliary spillway should be provided to take sufficient flow, adequate to keep the SEF from overtopping the main embankment. Breaching sections should be designed to limit sudden increases in outflow.

(iii) Shape of the hydrograph

For each flood recurrence interval a number of hydrograph shapes should be investigated, (Ref. 4) with the historic maximum flood on record as a first example. These floods should be individually routed through the reservoir applying flood absorption if significant, otherwise neglecting it, and the freeboard determined from the maximum water level rise obtained with the various hydrographs.

Multiple-peaked hydrographs often occur. In any dam with gated spillways, and especially in fill dams the occurrence of multiple peaks in the hydrograph must be carefully analysed. A near-full dam experiencing a second hydrograph peak especially during the closing-down stage of gate operation can present a dangerous situation.

Flood warning with pre- or post-release options should be utilized where possible but should not be relied upon for safe dam operation. Sufficient reserve capacity and freeboard should be maintained to absorb the effects of unheralded floods or during conditions of malfunction of gates or services.

(iv) Type of dam

Adequate freeboard allowance is more critical for fill dams than for concrete dams due to failure dangers associated with potential overtopping. This also applies to composite dams having an earthfill component. Since the objective of freeboard is to provide assurance against possible overtopping due to various causes, each cause needs to be more carefully considered in the case of fill dams.

(v) Use of wave or splash walls

The use of a wave - or splash - wall along the upstream edge of the non-overspill crest of an embankment dam may often be an economical way to prevent overtopping by wind-wave action, particularly when considering recommended design flood (RDF) conditions. A wave wall can be shaped to deflect the run-up water and model studies may be the only means of accurately establishing the effectiveness of a wave wall on more sensitive designs. (Ref. 14.)

The effectiveness of parapet walls to contribute to freeboard should be discounted, however, if they are merely ornamental, open or structurally incapable of resisting the shear and bending moment due to the full static and dynamic water pressure to their tops. If intended to be considered as contributing to freeboard, i.e. capable of resisting water pressure, they should be designed accordingly. If only intended to serve as a wave deflector they should be designed to cope with the dynamic forces of breaking and standing waves.

## 3.2 FREEBOARD ALLOWANCES

The recommended design flood (RDF) routed through the dam (Ref. 1) with appropriate freeboard provides the basis for design of the dam and spillway system. No damage is to be caused during these circumstances.

The safety evaluation flood (SEF) routed through the dam (Ref. 1) with flood surcharge freeboard only, is the flood which may cause substantial damage to the structure and surroundings but must not be such as to cause the dam to fail catastrophically causing loss of human life and economic loss. It will be found that in many cases the determining condition for freeboard will be that for the SEF at non-overspill crest i.e. where overtopping is not allowed. However, the other cases must also be checked to ascertain the most critical condition.

In the "Interim Guidelines on Safety in relation to Floods" (Ref. 1) spillage under any of generally accepted criteria must not endanger the safety of the structure. These criteria are not listed but are proposed here.

The important conditions are grouped in combination numbers (Table I) that should be tested as shown in Table II. All conditions in the combinations mentioned and indicated in Table I (for a specifically numbered combination of criteria) are to be met simultaneously.

Site specific conditions at the reservoir which may influence wave run-up and wind set-up are given under 2.2.3 and should be duly taken into consideration.

Adjustments for direction of winds should be made when specific data are available to substantiate such adjustments.

Adjustments for uncertainties in the hydrology are to be allowed for.

The possibility of simultaneous occurrence or not of the flood peak and maximum wind speed should be considered.

TABLE I: PROPOSED DESIGN COMBINATIONS OF FREEBOARD CONDITIONS*

| Combination number | RDF | 20-year flood | Wind wave and run-up | | Wind set-up | Flood surges and seiches | Earthquake wave | Landslide wave (b) | Flood outlets (c) |
|---|---|---|---|---|---|---|---|---|---|
| | | | 25-year event (a) | 100-year event | | | | | |
| 1 | X | | X | | X | | | | |
| 2 | X | | X | | X | X | | | |
| 3 | | X | | X | X | X | | | |
| 4 | | | | | | | X | | |
| 5 | X | | | | | | | X | |
| 6 | X | | | | | | | | X |

NOTES: (a) For Lowveld or coastal cyclonic conditions extra allowance for wind waves is to be made.

(b) Landslides are to be taken into account only if caused by flood water conditions.

(c) A certain proportion of crest gates or flood outlets considered inoperable.

---

* Assumed starting level is FSL.

TABLE II: RECOMMENDED MINIMUM VALUES FOR:

- APPLICABLE FREEBOARD (FB) CRITERIA IN TERMS OF THE COMBINATION NUMBERS (TABLE I);
- RECOMMENDED DESIGN FLOOD (RDF) IN TERMS OF RECURRENCE INTERVAL; AND
- SAFETY EVALUATION FLOOD (SEF) IN TERMS OF A FACTOR OF THE REGIONAL MAXIMUM FLOOD AND THE PROBABLE MAXIMUM FLOOD

| Dam size | Freeboard criteria and floods | Hazard rating (Category of dam in brackets) | | |
|---|---|---|---|---|
| | | Low | Significant | High |
| | | (I) | (II) | (II) |
| Small (H = 5-12 m) | FB Criteria<br>RDF<br>SEF | 1<br>20-50 year<br>0,4*RMF<br>0,2*PMF | 1<br>100 year<br>0,7*RMF<br>0,5*PMF | 2<br>100 year<br>1,0*RMF<br>0,7*PMF |
| | | (II) | (II) | (III) |
| Medium (H = 12-30 m) | FB Criteria<br>RDF<br>SEF | 2;6<br>100 year<br>0,7*RMF<br>0,5*PMF | 2;3;6<br>100 year<br>1,0*RMF<br>0,7*PMF | 2;3;4;5;6<br>200 year<br>1,5*RMF<br>1,0*PMF |
| | | (III) | (III) | (III) |
| Large (H > 30 m) | FB Criteria<br>RDF<br>SEF | 2;3;6<br>200 year<br>1,0*RMF<br>0,7*PMF | 2;3;4;5;6<br>200 year<br>1,5*RMF<br>1,0*PMF | 2;3;4;5;6<br>200 year<br>1,7*RMF<br>1,1*PMF |

Note: The RDF and SEF criteria as reflected in the Interim Guidelines on Safety in Relation to Floods Ref. (1) have for the sake of convenience been included in Table II. Should a change be made to the RDF and SEF values in a revised Ref. 1, then the new values will supercede those given above.

Certain practical guidelines for the determination of freeboard have been developed by various organisations over the years and are given in Table III. These practical rules of thumb are often of application to small dams and medium sized dams with a low hazard rating and provide also a check on freeboard calculations. These practical guidelines are also discussed in further paragraphs dealing with different types of dam.

TABLE III: SIMPLIFIED PRACTICAL FREEBOARD GUIDELINES

| Type of dam | Minimum total freeboard (m) | Minimum difference in level between stillwater HFL and non-overspill crest (m) | Remarks |
|---|---|---|---|
| Earthfill (Category I) | 0,8 | 0,5 | - |
| Earthfill (Categories II & III) | - | 1,5 | For RDF |
| Rockfill (Categories II & III) | - | 1,5 | For RDF |
| Concrete (Categories II & III) * | 1,5 | 1 | - |

* Freeboard for Category I and small Category II dams can be replaced from these values.

Another useful rule of thumb is that the minimum acceptable wave heights for Category II and III dams used in the wave run-up calculation is 0,75 m.

## 3.3 SPECIAL REQUIREMENTS FOR EARTHFILL DAMS

### 3.3.1 Introduction

Earthfill dams as opposed to concrete dams are built with erodible material. Furthermore, most earthfill dams settle in time and often in a differential way. The principle for safety is thus to prevent excessive overtopping resulting in erosion of embankments where this may lead to the possible loss of the dam.

For large dams located in large rivers the non-overtopping requirement from a safety point of view is normally dominant in the provision of freeboard (SEF condition). This should not, however, be taken as a fixed rule because a dam could have flood and basin shape characteristics resulting in the freeboard allowance determined on the recommended design flood (RDF) as the higher one. Examples of the last mentioned are:

- a large off-channel storage dam in a small catchment, and

- a dam with a large storage capacity above full supply level and good flood absorption characteristics.

### 3.3.2 Settlement of embankment and foundation

Normally settlement of the embankment and foundation due to consolidation is expected and compensated for by adding camber to the design crest elevation of the dam and also to the top of the impervious zone. Soft foundation and inadequate control during contruction may necessitate additional freeboard allowance.

For well-compacted embankments and dense foundations most of the consolidation occurs during construction. Any additional settlement in the form of secondary consolidation is normally allowed for by "camber". Normally, a camber of 1% of local dam height along the axis is allowed for settlement but it may vary from 1-2,5% depending on site conditions.

### 3.3.3 Top of impermeable zone

To control leakage in the case of a zoned embankment, the top of the impervious zone must be constructed higher than the safe elevation i.e. at least up to the surcharge level for the RDF. The adequacy of the top portion of the zoned embankment including the filters to prevent piping during the surcharge event should be evaluated.

### 3.3.4 Parapet walls

The use of wave walls to provide freeboard allowances for <u>embankment dams</u> may be considered on a case-by-case basis (see also 3.1.2(iv)). The following criteria are proposed to be met:

(i) Normally, the parapet or wave deflection wall may only replace the portion of the freeboard needed to <u>prevent overtopping from wave run-up</u>. If it is to prevent leakage from surcharge water or other components of freeboard, it should be tied into an adequately impervious zone for example the core, and it should be conservatively stable against overturning or other erosional forces.

(ii) Foundation and embankment settlement that would effect the top level or stability of the wall, should be allowed to occur prior to construction of the wall, or the design should allow for future settlement.

(iii) Wave walls should be continuous and level. All joints should be watertight to prevent concentration of flow.

3.3.5 Minimum requirements: Category I Dam

In order to provide safeguards against various uncertainties, the minimum acceptable non-overspill crest elevation for a category I fill dam is 0,5 m above the surcharge level.

3.3.6 Overtopping during extreme floods (SEF conditions)

Overtopping of fill dams is to be avoided for the following reasons:

(i) earthfill is erodible;

(ii) the crest of an embankment dam is not always 100% level due to settlement or other reasons. Water overflowing the lower parts would have erosional effects there;

(iii) overflow water concentrated by concrete walls at the junction between the spillway and the embankment will have erosive effects, and adequate erosion protection should be provided in this area should overtopping be likely.

For existing dams it may also be necessary to determine the risk of the dam being washed away due to overtopping. Reference is made to the 1986 SANCOLD publication Design floods, (Ref. 1) and Proceedings of the SANCOLD Symposium 1986 (Knoesen and Oosthuizen, Ref.22).

3.3.7 Fuse plugs

Fuse plugs or emergency spillways are specially designed embankment portions with the purpose of breaching during extreme flood conditions or upstream dam failure in order to protect the main wall from breaking. The breaching level depends on economic factors in each case. Fuse plugs are often designed to breach for floods between the RDF + the 25-year wave condition and the SEF (Knoesen, Ref. 23). Special care should be taken in design, so that the resulting outgoing flood is smaller in size compared to the incoming flood. Where a fuse plug is provided the adequacy of the freeboard over the rest of the embankment is dependent on its proper functioning during a flood.

3.3.8 Erosion resistance increase

Ways and means have been developed to increase erosion resistance of downstream slopes of embankments, especially in smaller structures, e.g. grass planting, paving blocks, mastic asphalt. Care must be taken with such applications and might require the use of large scale models to prove effectiveness.

## 3.4 SPECIAL REQUIREMENTS FOR ROCKFILL DAMS

### 3.4.1 Characteristics of rockfill dams

Rockfill embankments are generally have considerably steeper slopes than those of earthfill. Rockfill also has the characteristic that the roughness of the upstream face can range between either very smooth (in the case of a bitumen or concrete upstream impervious layer) or very rough, similar to the riprap on an earthfill dam. The effect of these two characteristics, slope and roughness, on freeboard allowance will be dealt with below.

Smooth upstream face:

The smooth upstream face of a bitumen or concrete faced rockfill dam has almost no wave energy absorption. Wave run-up on the face can therefore be severe and could cause overtopping even under relatively mild wind conditions. It may be necessary to provide some form of wave barrier on the top of a smooth faced rockfill dam instead of raising the level of the non-overspill crest and the choice will be governed by economics. This barrier must be designed not only for the dynamic impact of the waves but also for hydro-static pressure conditions if these could arise. The stability of the wave wall is vital to the safety against overtopping of a rockfill dam and forms a very important component of the overall design with respect to freeboard. If correctly designed it will contribute to the value of the freeboard.

Rough upstream face, exposed rock surfaces:

The upstream face of a rockfill dam which has an impervious centre core is usually composed of the coarsest rock material on site and therefore has maximum potential for wave energy absorption. It is therefore not usually necessary to provide wave walls or any other form of wave energy dissipation device in these cases. It must, however, be borne in mind that some additional freeboard can be obtained at fairly modest cost by increasing the slope of the rockfill from above the level of normal full supply level to the crest.

### 3.4.2 Safety against overtopping

In all dam designs, some element of risk of overtopping exists. In reviewing old dams which were designed with less than adequate flood information, the possibility of overtopping could be very significant indeed. This possibility necessitates consideration of the mechanism of overtopping and the possible damage that might occur. Most modern rockfill dams are built subject to very careful quality control, with good compaction technique being exercised and hence only minor post-construction settlement is likely to occur. Most rockfill dams, particularly the larger ones, do have quite a considerable longitudinal superelevation proportional to the local height or "camber" on the crest which is desirable against settlement. The implication of a cambered crest, i.e. with varying top elevation, however, is that overflow will occur initially at the abutment and that it will be the toe (i.e. the valley between the embankment and the abutment) that will be most subject to the erosive forces of water. Lack of attention to the toe-valleys could therefore result in failure of a rockfill embankment at relatively modest overtopping levels. Attention must therefore be given to the prevention of erosion on the shoulders of the abutment and of the embankment itself at the contact with the abutment, or alternatively no "camber", but a constant superelevation against settlement should be specified. Settlement, when eventually occurring will then result in a lower central portion.

### 3.4.3 Other precautions against overtopping failure associated with minimum freeboard

Reinforcement of the downstream face:

Rockfill embankments readily lend themselves to reinforcement of the downstream face by means of rip-rap or paving at comparatively modest costs. This additional feature which might in any case be required during construction, should be considered as an additional safety feature available in the design of rockfill embankments against overtopping.

Crest treatment:

The treatment of the crest of a rockfill embankment can have a major effect on its erosion resistance. Normally rockfill crests are finished with a layer of gravel, particularly where a central earth core requires protection from desiccation, however, in many cases the crest of a rockfill dam can serve the additional purpose of an access road, and if provided with a bitumen surface such a crest can increase the erosion resistance of the embankment. The choice of suitable wave walls, parapets or handrails should be investigated and their effect on reducing dam height, while maintaining adequate freeboard, utilized to the full (see Fig.3).

### 3.4.4 Type of dam

In selecting the type of dam freeboard considerations could become important as an increment of height on a very high dam could affect the cost markedly differently for a concrete gravity dam, a rockfill dam or an earthfill dam. The valley shape, broad and flat versus U- or V-shaped also enters into the marginal cost.

## 3.5 SPECIAL REQUIREMENTS FOR CONCRETE DAMS

In the calculation of freeboard for Categories II and III concrete dams use the 1:100 and 1:200 year flood respectively to determine the recommended design flood. The total freeboard will depend on the fetch and estimated wave height but a minimum of 1 m between high flood stillwater level and non-overspill crest should be maintained. Keep, wherever possible, the SEF within the confines of the spillway but for this flood allow zero secondary freeboard. This condition is generally the determining factor. Where this leads to excessive freeboard or where the spillway width is limited, consider the use of parapet walls suitably designed as water retaining structures. On a non-composite concrete dam the SEF could be allowed to overtop the non-overspill crest, with some damage being accepted as long as the dam is safe under these conditions. In such cases special attention should be given to the erosion resistance of the foundation.

All dams require special considerations peculiar to the site and application, but this is especially true for Category I and small Category II dams where freeboard requirements can be relaxed depending on the hazard potential and the consequences of failure. If the failure of the dam under extreme flood conditions would not make any difference to the downstream effect of the flood, and if the cost of making provision for handling this particular flood through the dam is excessive, then a compromise solution might be adopted, and the dam designed for a smaller flood.

Controlled spillways can be readily incorporated into concrete dams and for these the flood surcharge could become zero, provided the design flood (RDF) can be passed with the gates open, and at or below the normal full supply level. The gates themselves must also have a marginal wind freeboard to counteract their being overtopped by wind waves. In some cases gates are designed to be overtopped allowing to some extent for inoperation possibilities. Due allowance should be made for each particular case for a proportion of the gated outlets that might not work, in other words a redundancy of spillway capacity or of freeboard allowance should exist for large dams. A practical guide which is recommended is that at least one gate in a set of a few gates or 10% of the gates if there are a large number, should be considered inoperable.

## 3.6 SPECIAL REQUIREMENTS FOR COMPOSITE DAMS

A Composite dam is a combination of some of the above types of dam, for example an embankment dam with concrete spillway or a concrete main dam with embankment saddles or flanks. The freeboard for each component should be commensurate with the type of wall e.g. the wave run-up factor should agree with the value for that particular upstream slope and roughness. For a concrete/embankment composite dam apply the appropriate freeboard standards for each component.

## 4. SUMMARY AND RECOMMENDATIONS

Chapters 1, 2 and 3 above discuss the concept of freeboard, the various quantitative components thereof, how to calculate them and the application thereof to various types of dam.

While these are guidelines they are to be considered flexible and subject to engineering judgment also involving costs and risks.

The way of arriving at a combination for determining the total freeboard has been indicated in Tables I and II. Some of the components may be given less emphasis, depending on the size and importance of the dam; others may not, due to the uncertainty of hydrology. Practical guidelines are given in Table III.

The problem varies from section to section in a composite dam and each component is to be dealt with both separately and collectively. The provision of fuse plugs, wave walls and parapets is included and relates to cost and risk.

The advisability of having both uncontrolled and gated spillways in a dam has been noted. This ties in intimately with hydrological and operational considerations and cannot be categorically dealt with in the scope of this guideline.

The following recommendations are made:

1. Adopt freeboard figures developed from calculations above and test their adequacy under a variety of hydrological scenarios and spillway designs.
2. Calculate the volume and cost of the extra height of dam needed.
3. Investigate whether a less expensive solution in the form of fuse plug, parapet, wave wall, gated outlet could reduce the bulk of the dam required to establish the necessary freeboard crest level.
4. Evaluate the greater or lesser risk of failure of the scheme thus devised against the original calculation in 2 above.
5. Check the design again against all occurrences considered in 1 above, individually and collectively where appropriate.
6. Do final modifications in crest level upwards to increase safety margin, downwards to reduce cost and arrive at an optimally balanced freeboard value and hence crest level.

## 5. REFERENCES


1. SANCOLD: Interim guidelines on safety in relation to floods; Safety evaluation of dams, South African Committee on Large dams. Report no.1. (September 1986).

2. Alexander, W.J.R., Flood Hydrology for Southern African SANCOLD, Pretoria (1990).

3. U.S. Bureau of Reclamation; United States Department of Interior, DESIGN STANDARDS NO.13-EMBANKMENT DAMS, Chapter 6 Freeboard (1984).

4. Hiemstra, L.A.V. and Francis, D.M. The Run Hydrograph Theory and application for flood predictions. Department of Civil Engineering, University of Natal, Water Research Commission Report. (1979).

5. Van Rooyen, J.A., Damoewerbeskerming: B. Ing. Skripsie, Departement van Siviele Ingenieurswese; Universiteit van Pretoria (1978).

6. Kovacs, Z., TR137, Regional maximum flood peak discharges in Southern Africa. Department of Water Affairs, Pretoria (1988).

7. Bureau of Reclamation: United States Department of the Interior, Freeboard criteria and guidelines for computing freeboard allowances for storage dams; ACER Technical memorandum no.2, Assistant Commissioner - Engineering and Research Denver, Colorado. (1981).

8. Milford, R. V. Maximum wind speeds for South Africa The Civil Engineer in South Africa, (January 1987).

9. WEATHER BUREAU. Climate of South Africa, Part 12. Surface Winds. WB 38 (1975).

10. Saville, T., McClendon, E.W. and Cochran, A.L., Freeboard allowances for waves in inland reservoirs. Journal of the Waterways and Harbours Division, ASCE, Vol. WW2, (1962).

11. UK Institution of Civil Engineers, Floods and Reservoir Safety : An Engineering Guide. Institution of Civil Engineers, London (1978).

12. United States Army Corps of Engineers, Shore Protection Manual, Vol. I, II and III, Coastal Engineering Research Center (1977).

13. Venter, A.J., Golfaksie op gronddamme, B.S. Skripsie, Dept. Siviele Ing., Univ. v. Stellenbosch. (1987).

14. NRIO (CSIR) Stellenbosch. Vaaldam splash wall model study, CSIR Report No. C/SEA 7936 (1979).

15. Vischer, D.L., Rockfall induced waves in reservoirs, Water Power and Dam Construction, (September 1986).

16. Raichlen, F., The excitation of harbours by Tsunamis, Tsunmamis their science and engineering, Terra Sc. Publ. Co., (TERRAPUB) Tokyo (1983).

17. Davidson, D.D. and McCartney, B.L., Water waves generated by landslides in reservoirs, Journal of Hydraulics Division HY12, ASCE, pp 1489-1501, (December 1975).

18. Kovacs, Z., Roberts, C.P.R., and Jordaan, J.M., Overtopping of dams by surging flow, Trans. SAICE, (1984).

19. Shand, M.J., Simplified analysis of flood surges in reservoirs, Discussion on Ref. 18, Trans. SAICE.

20. Kovacs, Z., and Du Plessis, B. Extreme floods in Southern Africa caused by Tropical Cyclone Domoina. International Water Power and Dam Construction", (July 1987).

21. Alexander, W.J.R. and Kovacs, Z. Lessons learnt from exceptional floods in Southern Africa, ICOLD Congress, San Francisco, 1988.

22. Knoesen, J.S. en Oosthuizen, C., Weerstand van grondwalle teen oorstroming. Symposium on Dam Safety, SANCOLD and SAICE, (Paper No.12), Pretoria, (23-25 September, 1986).

23. Knoesen, J.S., Breekseksie as 'n hulpoorloop vir damme, M. Ing. skripsie, Universiteit van Stellenbosch, Desember 1987, (bl.6.2).

## 6. ACKNOWLEDGEMENT

The collaboration and assistance of the following members of the Committee on Freeboard for dams is acknowledged with thanks.

Mr D.B. Badenhorst
Mr W.S. Croucamp
Dr J.M. Jordaan
Mr N. Pells
Dr C.P.R. Roberts (Chairman)
Prof. A. Rooseboom
Mr W. van Wyk

Dr J.M. Jordaan undertook the bulk of the editorial work.

The following engineers also assisted: F. Hollingworth, H.F. Elges and Z. Kovacs. Professor W.J.R. Alexander provided reference material from the Flood Hydrology Handbook.

CPR Roberts

C.P.R. ROBERTS

CHAIRMAN: SANCOLD: COMMITTEE ON FREEBOARD FOR DAMS

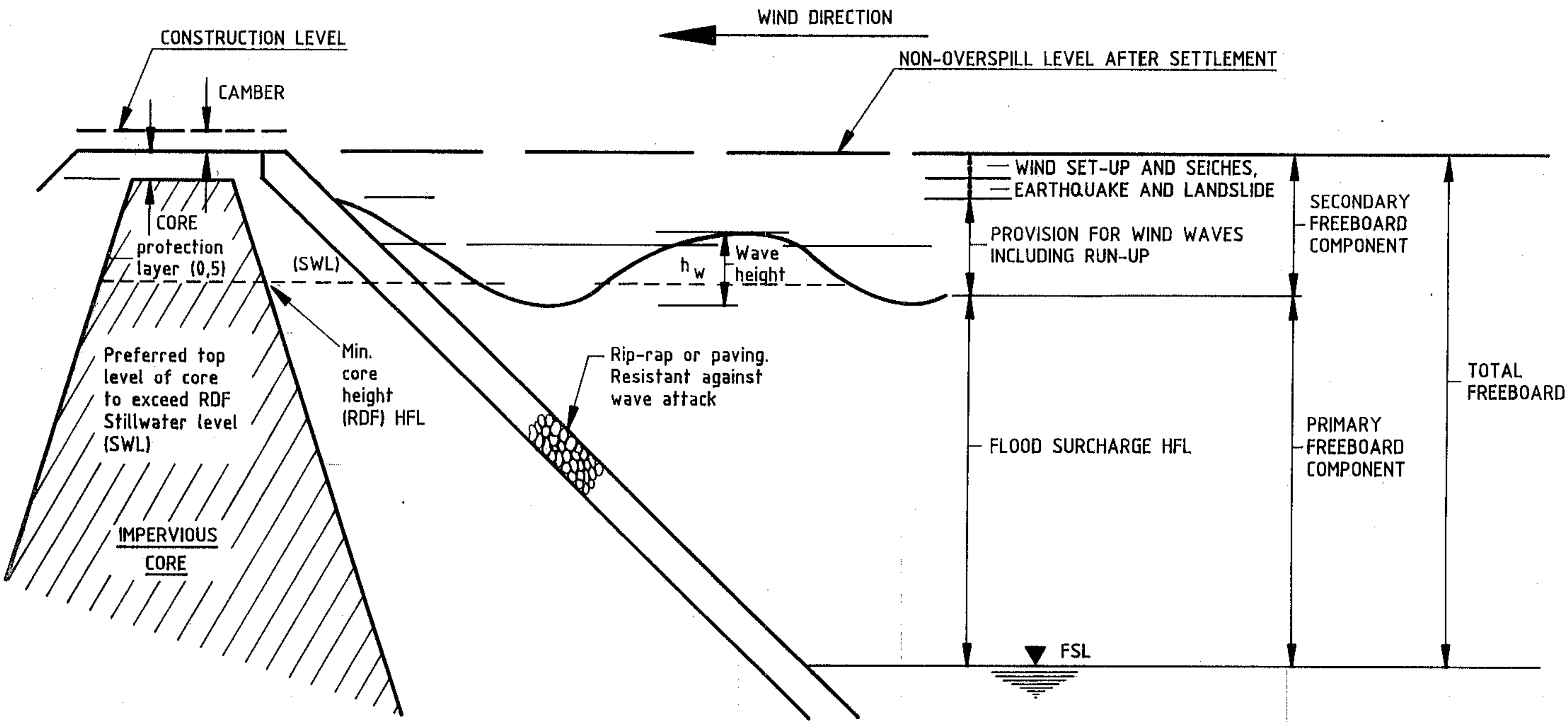
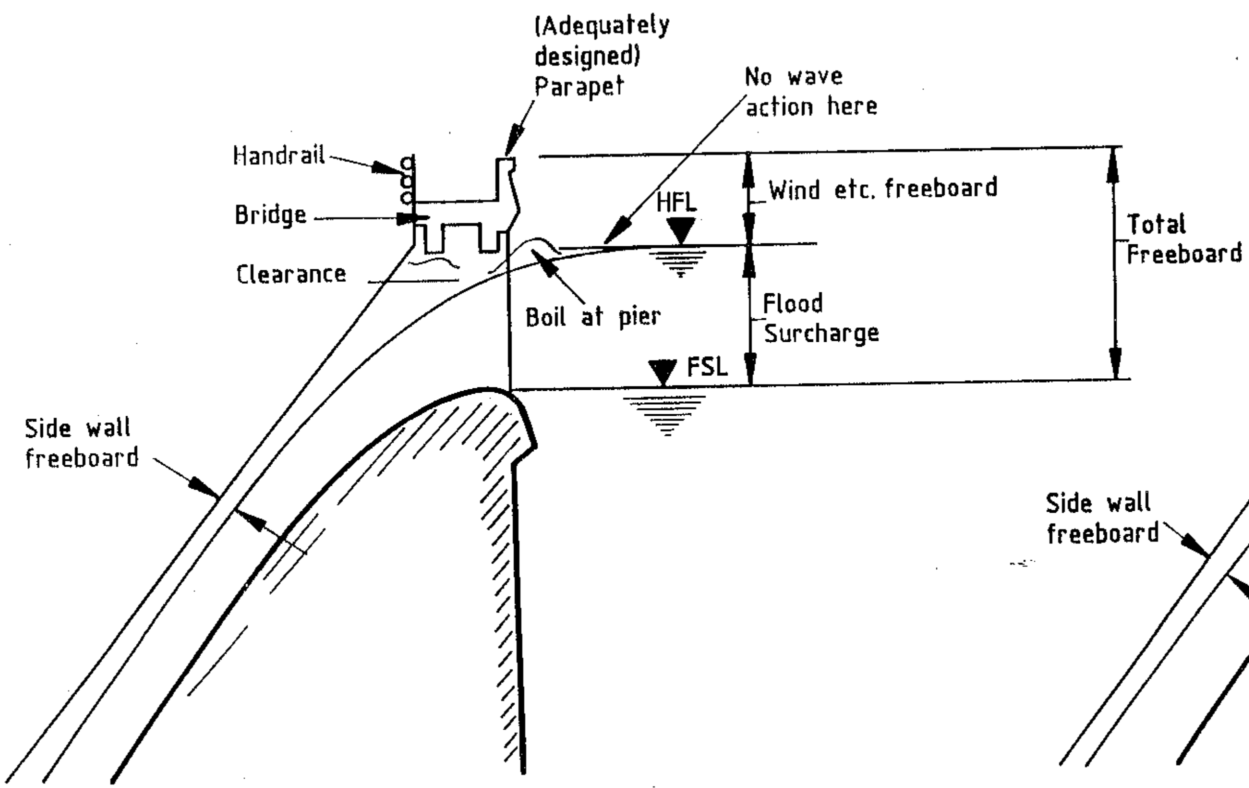
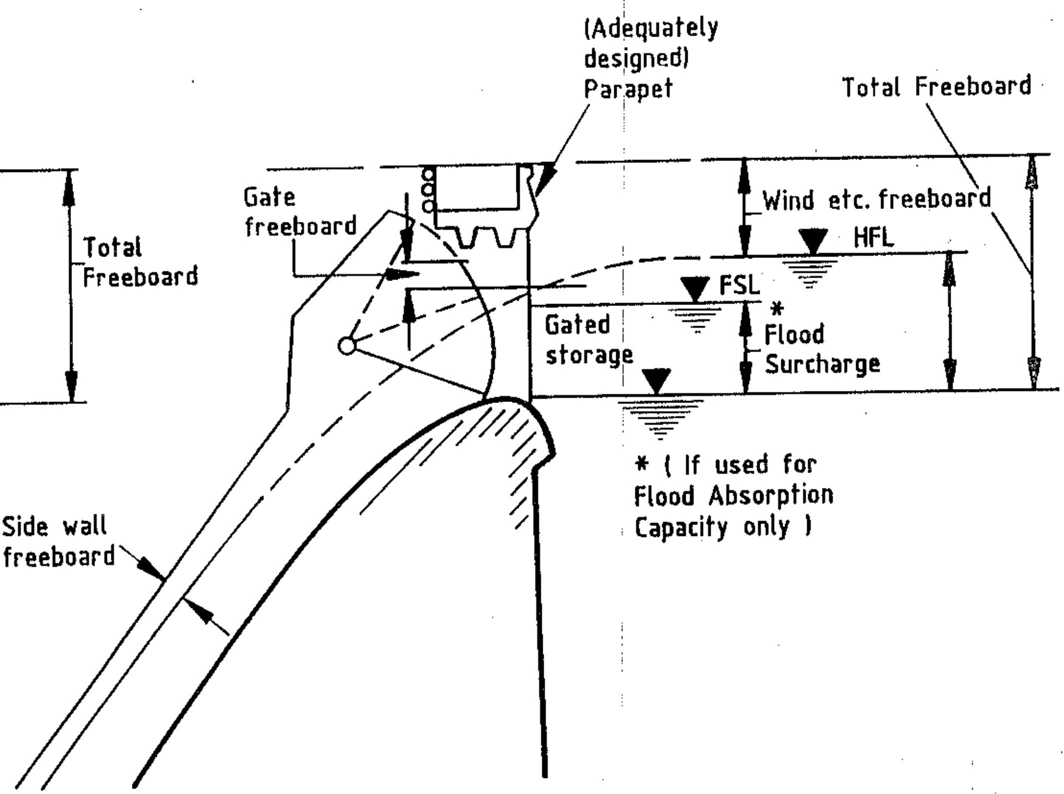



FIG 1 : DEFINITION OF FREEBOARD (FOR THE CASE OF AN EMBANKMENT DAM)




(a) OGEE SPILLWAY, UNGATED FREE OVERSPILL
"UNCONTROLLED SPILLWAY"




(b) OGEE SPILLWAY, GATED CREST
"CONTROLLED SPILLWAY"

FIG 2 : DETAIL OF CREST TREATMENTS FOR UNCONTROLLED AND CONTROLLED SPILLWAYS

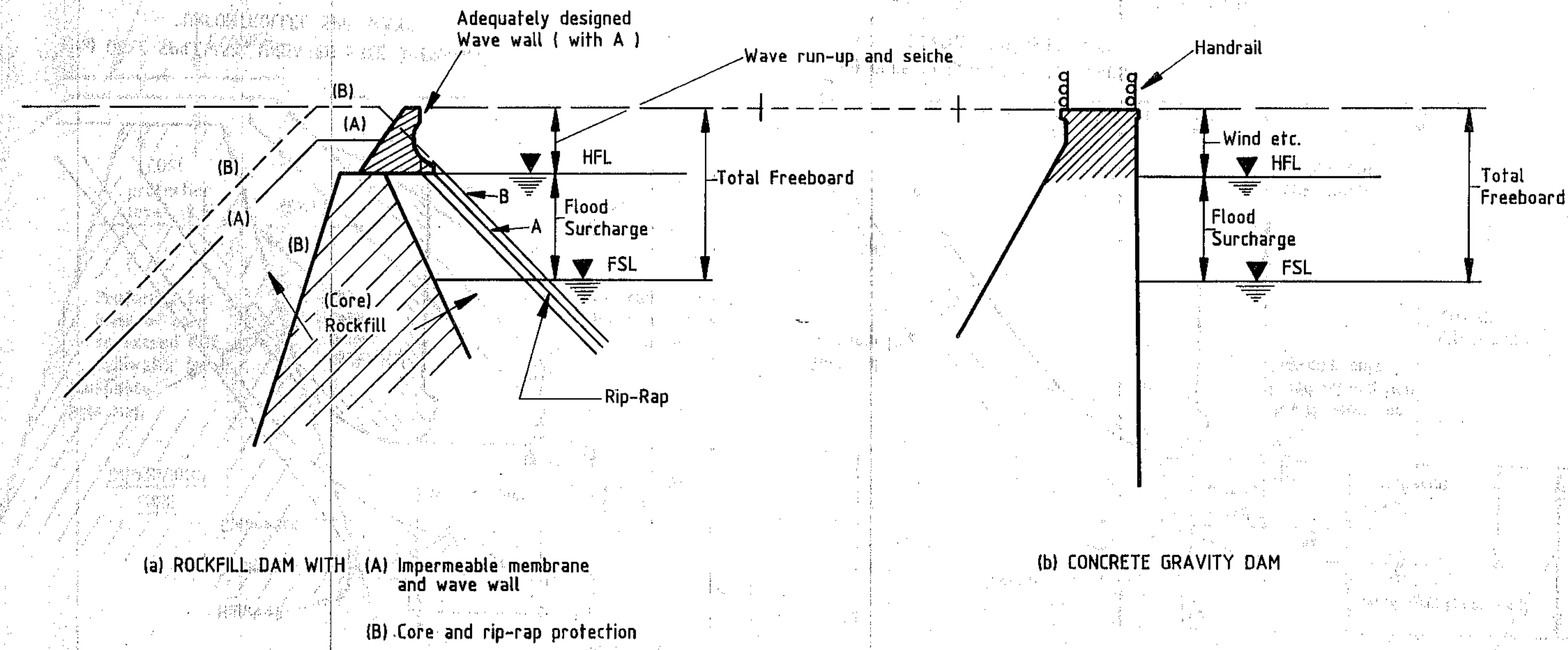
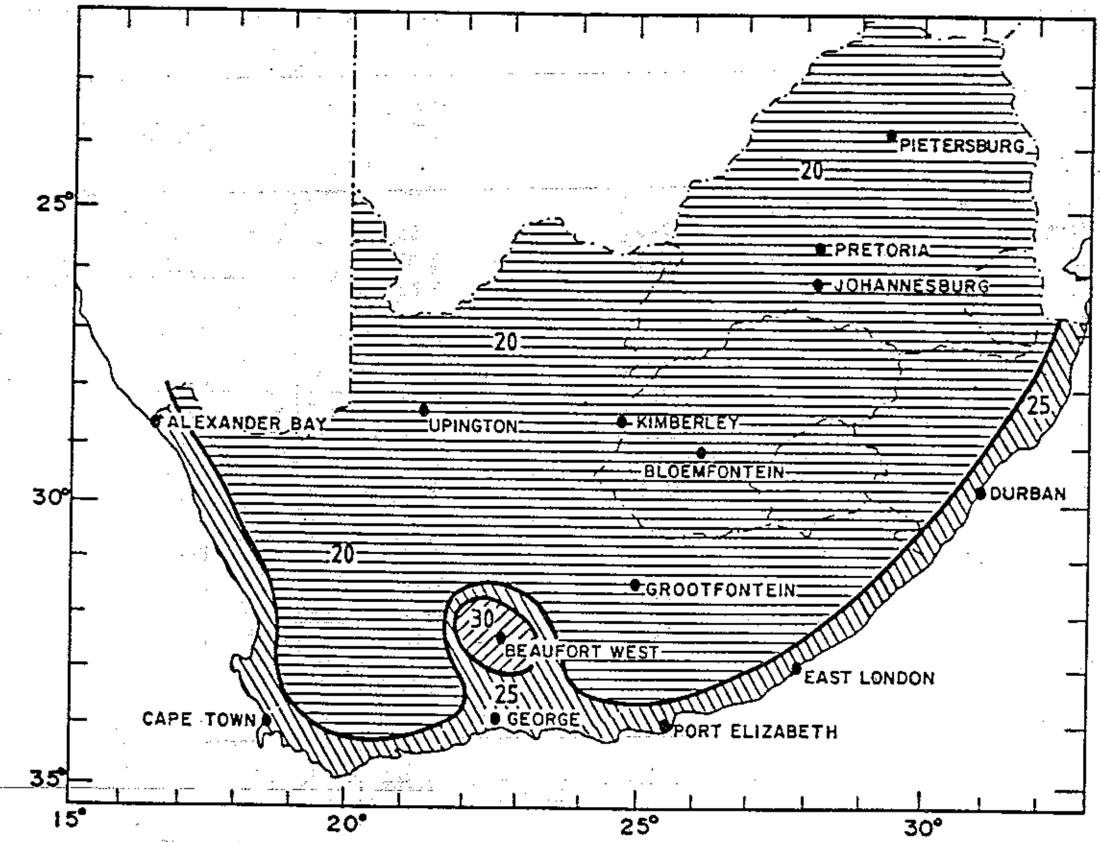



FIG 3 : CREST TREATMENTS FOR ROCKFILL AND CONCRETE GRAVITY DAMS




**CORRECTION FACTORS FOR CONVERTION TO OTHER RETURN PERIODS**

| RETURN PERIOD (YEARS) | CORRECTION FACTOR |
|---|---|
| 10 | 0,90 |
| 20 | 0,95 |
| 50 | 1,00 |
| 100 | 1,04 |
| 200 | 1,08 |
| 500 | 1,12 |

**Reference** : Hilford RV - Annual Maximum Wind Speeds for South Africa. Civ Eng S Afr, January 1987

FIGURE A1 : HOURLY MEAN DESIGN WIND SPEEDS ( m/s ) FOR A 50-YEAR RETURN PERIOD

| Angle from central radial $\alpha$ | Cos $\alpha$ | Length of radial Xi | $\Sigma$XiCos$\alpha$ |
|---|---|---|---|
| 42 | 0,743 | 2,65 | 1,97 |
| 36 | 0,809 | 2,35 | 1,90 |
| 30 | 0,866 | 2,05 | 1,78 |
| 24 | 0,914 | 1,85 | 1,69 |
| 18 | 0,951 | 2,00 | 1,90 |
| 12 | 0,978 | 6,35 | 6,21 |
| 6 | 0,995 | 5,15 | 5,12 |
| 0 | 1,000 | 5,00 | 5,00 |
| 6 | 0,995 | 4,95 | 4,93 |
| 12 | 0,978 | 6,30 | 6,16 |
| 18 | 0,951 | 3,60 | 3,42 |
| 24 | 0,914 | 3,05 | 2,79 |
| 30 | 0,866 | 2,90 | 2,51 |
| 36 | 0,809 | 1,50 | 1,21 |
| 42 | 0,743 | 1,50 | 1,11 |
| Totals | 13,512 | | 47,70 |

$$\text{EFFECTIVE FETCH} = \frac{\Sigma \text{XiCos}\,\alpha}{\Sigma \text{Cos}\,\alpha} = 3{,}5\text{km}$$

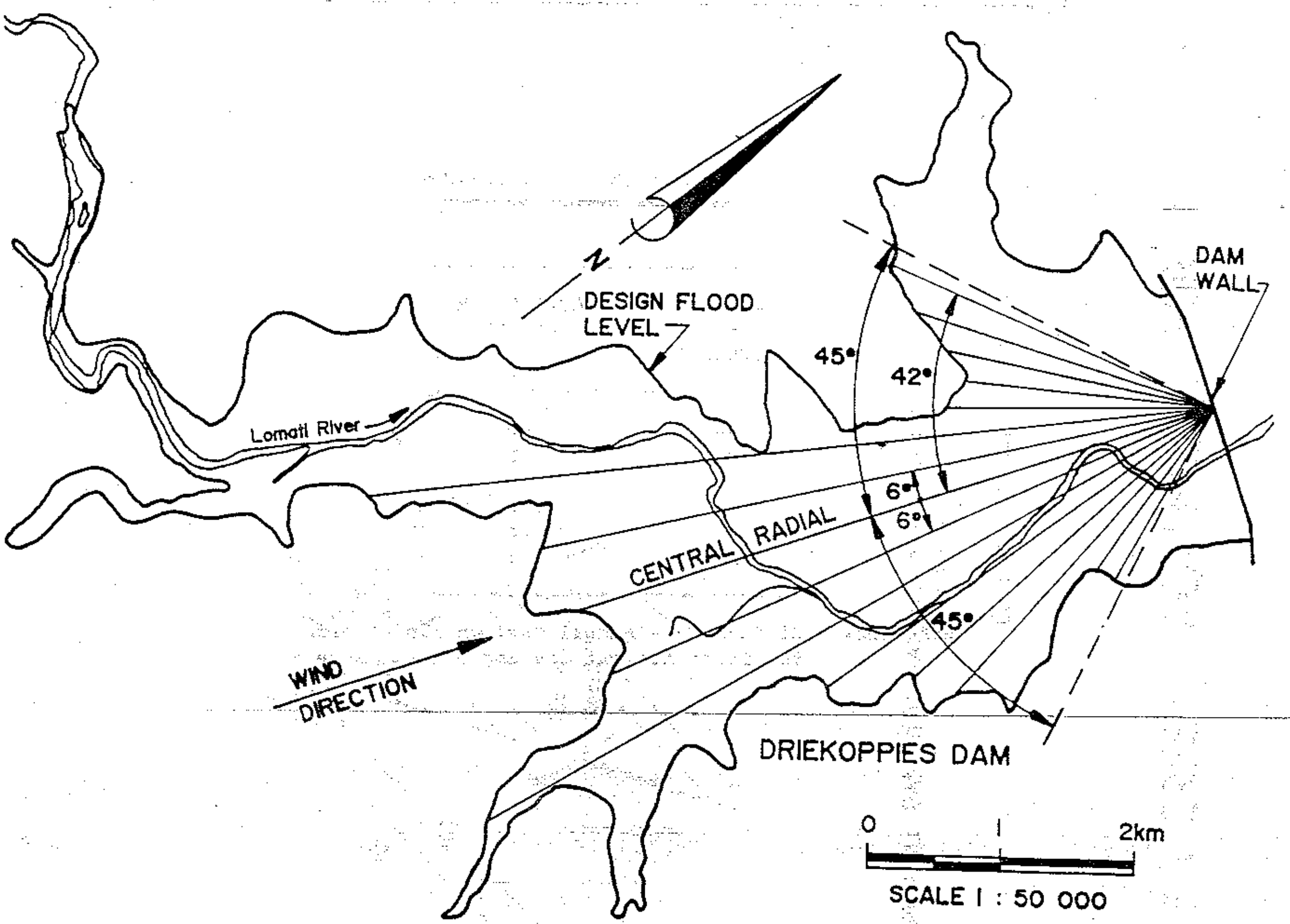



FIGURE A2 : TYPICAL EXAMPLE OF COMPUTATION OF EFFECTIVE FETCH FOR WIND WAVES

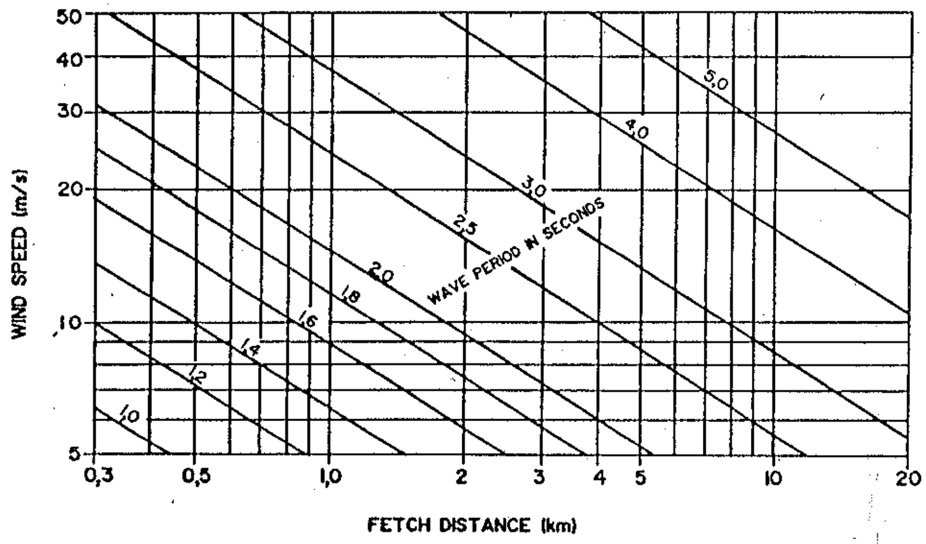



FIGURE A3 : WIND WAVE PERIODS

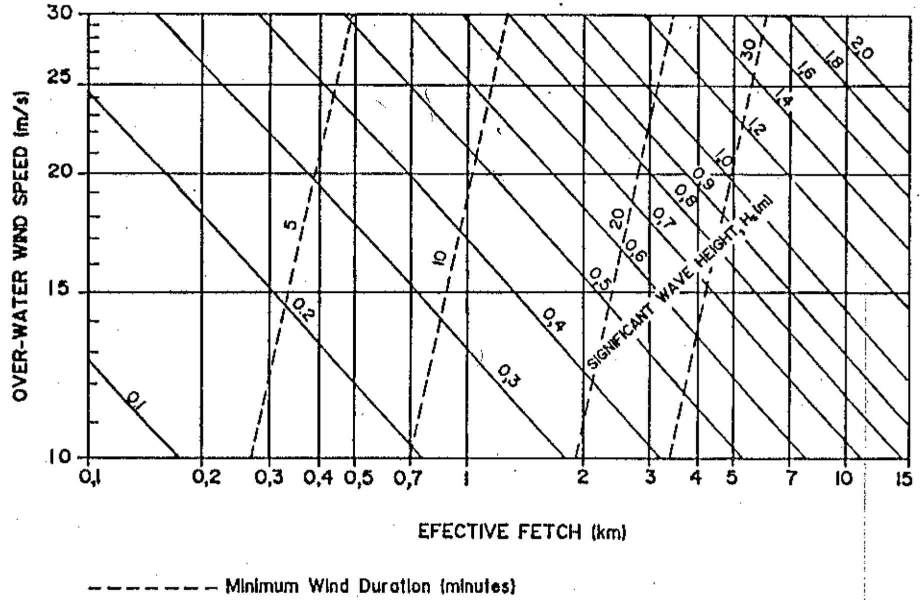
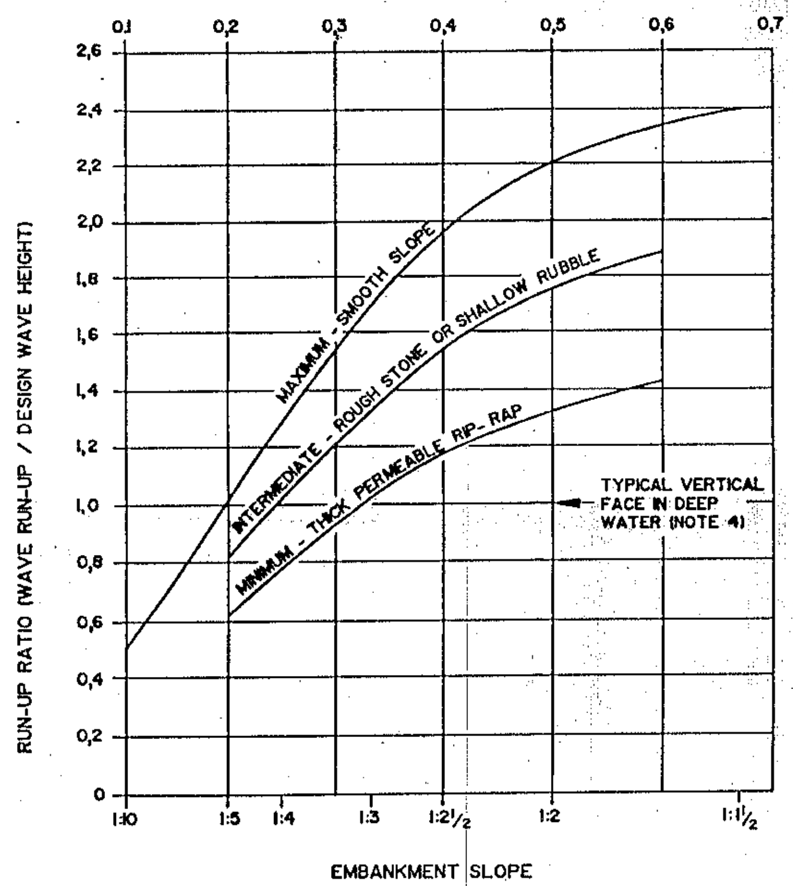



FIGURE A4 : SIGNIFICANT WAVE HEIGHTS




NOTES:

1. MAXIMUM LINE FROM SAVILLE ET AL (1962) FOR TYPICAL WAVE STEEPNESS (SIGNIFICANT WAVE HEIGHT/LENGTH) = 0,05.
2. INTERMEDIATE LINE IS 0,8 x MAXIMUM.
3. MINIMUM LINE IS 0,6 x MAXIMUM.
4. FOR FACES OFF-VERTICAL THE RUN-UP RATIO RISES ABOVE UNITY AND CAN APPROACH 2 IN SOME CIRCUMSTANCES WHERE THE DEEP WATER CONDITION IS NOT FULFILLED.

REPRODUCED FROM UK GUIDELINES (1978)

FIGURE A5 : WAVE RUN-UP