SOUTH AFRICAN NATIONAL COMMITTEE ON LARGE DAMS (SANCOLD)

Volume II
Guidelines on
Freeboard for Dams

Report to the

Water Research Commission

by

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Preface

The revision of the original 1990 SANCOLD Interim Guidelines on Freeboard for Dams coincides aptly with the draft revision of the Dam Safety Regulations in terms of the National Water Act (Act 36 of 1998). It is pleasing to note that the 1990 Guidelines have stood the test of time and only one major change has been advocated which is in respect of wave heights to be used in the calculations of freeboard. The revised guidelines also have updated wind, earthquake and landslide information for South Africa. The guidelines are also updated in respect of analytical tools and the use of computer software.

SANCOLD wishes to acknowledge the collaboration with the Water Research Commission and the researchers of the Institute for Water and Environmental Engineering, Department of Civil Engineering of the University of Stellenbosch.

South Africa, like many other countries, has legislation to control the safety of dams. To conform to the requirements of the legislation, the authority administering the Act and approved professional persons 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. 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. These guidelines have been prepared with great care, taking into account current practices followed in other countries.

Comments on these guidelines should be forwarded to: The Secretary, SANCOLD, P O Box 3404, Pretoria, 0001, or by e-mail to secretary@sancold.org.za.

DB Badenhorst
CHAIRMAN
SANCOLD

Executive Summary

This document replaces the 1990 SANCOLD Interim Guidelines on Freeboard for Dams. South Africa has legislation to control the safety of dams. Engineering standards for dams are not prescribed in the legislation. This guideline document is therefore essential to assist not only the designers of new dams but also those charged with evaluating the safety of existing dams.

There is a Volume I report in this series on freeboard which deal with a literature review and case studies. Volume I also provides more information on a risk analysis approach and on the design of riprap for erosion protection against wind generated waves at embankment dams.

The key revisions made in this document are:

a) The Milford wind map was replotted for 1:25, 1:50 and 1:100 year 1 hour duration wind speeds.
b) Wind generated wave heights; run-up and set-up should be based on the Rock Manual for analytical calculations, while the SWAN software is proposed for detailed assessment of wind wave height.
c) The $H_{2\%}$ wind wave (the wave height exceeded by 2% of the waves in an irregular wave train) is proposed in the revised guidelines, which is 1.4 times higher than H_s (the average of the upper third of the wave heights in a wave train) calculated with the 1990 guidelines. This is probably the main difference with the old guideline. (It should be noted that H_{max} (the maximum wave in a wave train) is still 1.4 times higher than $H_{2\%}$).
d) Unsteady flow patterns in reservoirs such as seiches, oscillations, flood surges, land slide waves, etc. should be simulated by mathematical hydrodynamic models.
e) Freeboard combination scenarios based on hazard rating and dam size were revised.

Acknowledgements

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The Reference Group members of this WRC project were:

Mr Wandile Nomquphu	Water Research Commission (chairman)
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This document was also reviewed by the South African National Committee on Large Dams (SANCOLD) and officially replaces the 1990 document for use in South Africa. The SANCOLD reviewers included:

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The financing of the project by the Water Research Commission and the contribution of the members of the Reference Group and reviewers are gratefully acknowledged.

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LIST OF ABBREVIATIONS

AEP	Annual Exceedance Probability
CEM	Coastal Engineering Manual
FSL	Full Supply Level
HFL	High Flood Level
NOC	Non-Overspill Crest
RDD	Recommended Design Discharge (routed)
RDF	Recommended Design Flood (unrouted)
SANCOLD	South African National Committee on Large dams
SED	Safety Evaluation Discharge (routed)
SEF	Safety Evaluation Flood (unrouted)
SWAN	Software: Simulating Waves Nearshore
WRC	Water Research Commission

1. INTRODUCTION

The SA Water Research Commission initiated this project in 2008 to propose revisions to the 1990 Interim Guidelines on Freeboard for Dams of the South African National Committee on Large Dams (SANCOLD). Following a peer review process by the WRC Reference group and SANCOLD, this document now officially replaces the 1990 document. There are two documents in the series and this Volume II report provides guidelines on the use of updated methodologies to determine freeboard for dams and also addresses the combination of freeboard components to be taken into account.

Volume I of this report series documents the present knowledge pool and methodologies for freeboard determination, with case studies (WRC, 2011).

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 (NOC) 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. The SANCOLD guidelines are dynamic documents and these guidelines replace the previous 1990 document.

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 (NOC) of the dam, excluding camber (allowance for consolidation), 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 calculations of total freeboard adequate provision must be made for the reasonable combination of conditions which may play a role. The most important components for total freeboard calculation are indicated in Table 1.1, all of which are not cumulative.

Table 1.1 Freeboard contributing components and related driving mechanisms

	Freeboard contributing component	Driving mechanism
(a)	Flood surcharge (with allowance for possible gated spillway or bottom outlet malfunctioning)	Rainfall and run-off of relevant weather system
(b)	Wind set-up	Wind of relevant weather system
(c)	Wave run-up/wave reflection	Wind generated wave
(d)	Seiche/resonance (oscillating water body in reservoir)	Disturbance of water body in dam basin by: -Spatial variation of barometric pressure over dam basin by relevant weather system -Earthquake
(e)	Earthquake induced surge	Earthquake shock/acceleration of dam water body and earth crust
(f)	Landslide induced surge	Volume of landslide displacing part of water body in dam basin and consequent long wave Landslide could be caused by: -High rainfall and subsequent excessive saturated subsoil -Earthquake
(g)	Flood induced surge	-Flood flowing into dam basin water body, causing water body disturbance and consequent long wave -Sudden outflow adjustment
(h)	Gate adjustment surges	-Rapid gate opening or closure

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.

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

2. FREEBOARD CALCULATION PROCEDURE

2.1 FLOODS

The capability of safely passing floods such as the Recommended Design Discharge (RDD) and Safety Evaluation Discharge (SED) is paramount in any dam's design. For guidance on the determination of the incoming design flood, and the safety evaluation flood, references SANCOLD (1991), Alexander and Kovacs (1988), Alexander (1990), Kovacs and Brink (1987), Kovacs (1988), and WRC (2007) could be consulted. Flood absorption takes place due to the temporary storage space above Full Supply Level (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 should be based on level pool routing or hydrodynamic mathematical modelling.

2.2 FLOOD SURCHARGE

The flood surcharge component contributing to the freeboard for dams is outside the scope of this investigation since it has been treated in detail in SANCOLD (1986). When the decision has been made on the inflow flood hydrograph(s) to be applied in the dam safety evaluation exercise, the best current practice to determine the maximum flood surcharge at the dam wall would be to employ an appropriate hydrodynamic model which takes into account the characteristics of the selected inflow hydrograph (including possible superposition of dam break(s) upstream of the dam under consideration), flood absorption of the dam basin, surge due to the inflowing flood hydrograph, spillway characteristics, etc. Level pool routing or mathematical hydrodynamic modelling should be carried out. The flood-absorption calculation procedure is based on:

a) A relationship between storage volume and level above FSL;

b) 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. Contraction losses of abutments and possible piers should be taken into account when determining the effective length of the spillway. Submergence of especially a side channel spillway should be taken into account;

c) Operating rules for handling floods where floods gates and/or bottom outlets are involved;

d) The initial water level condition (normally taken at FSL);

e) Future sedimentation in the reservoir for a minimum period of 50 years. As much as 30% of the sediment could be deposited above FSL which will affect the flood attenuation;

f) The calculation method, i.e. horizontal water surface or backwater; and

g) The shape of the hydrograph.

The Recommended Design Flood (RDF), the Safety Evaluation Flood (SEF) or other floods with given return periods are chosen, as estimated from several methods given in the SANCOLD Guidelines on Safety in Relation to Floods (SANCOLD, 1991) and the Flood Hydrology Handbook (Alexander, 1990).

Once the RDF and SEF have been routed through the reservoir, the peak discharges obtained at the dam becomes the Recommended Design Discharge (RDD) and Safety Evaluation Discharge (SED) respectively. Other freeboard contributing components as listed in Table 1.1 should be added to the RDD surcharge calculated at the dam. The total freeboard between FSL and NOC should not be exceeded during the RDD. The SED only takes into account the surcharge at the dam and no other freeboard components. The SED is based on a major extreme flood and no major structural damage to the dam is allowed.

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

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

a) Where the attenuation for RDF is less than 10 per cent. 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.

b) Where the time to outflow peak is short as in the case of controlled outlet structures where 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.

2.2.1 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:

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

b) Combinations of uncontrolled + gated spillways. Here surcharges 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.

c) Fully gate-controlled spillway. Maximum releases, where outflow equals inflow and pool level remains constant, are possible. Gates are not to be overtopped unless there is specific design provision for this eventuality. 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.

d) 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.

e) 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.

2.3 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.

2.3.1 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.

If wind data is available for meteorological 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. wind blowing along valleys onto an exposed water surface can also cause higher local wind speeds.

For a given atmospheric pressure gradient and meteorological conditions, wind speeds over water are higher than over land. The adjustment of overland wind speed is treated later in this Section.

A tropical cyclone is similar to an extra-tropical cyclone but smaller in diameter (i.e. approximately 40 km to 80 km in diameter), with much higher wind velocities and rainfall intensities than in an extra-tropical cyclone. The mean hourly average wind speeds in a tropical cyclone frequently exceeds 50 m/s. The statistics of wind velocity and occurrence of tropical cyclones on the east coast of South Africa are presented in Figure 2.3-1 (Rossouw, 1999).

South Africa is not as severely exposed to tropical cyclones as the USA east coast. An historic example of tropical cyclones, which caused significant storm damage in KwaZulu-Natal, was Domoina in 1984.

The spatial distribution of wind speed statistics (one hourly average at 10 m above recorded surface) in SA prepared by Milford (1987) and referred to in the SANCOLD Interim Guideline Report (1990), has not been updated since no new long-term hourly data are available for South Africa. The original data used by Milford was however replotted with the cyclone data as shown in Figures 2.3-2 to 2.3-4.

The 25-year, 50-year and 100-year design isopleths for hourly mean wind speed at 10 m above surface as given in Figures 2.3-2 to 2.3-4 can be used as a basis for selecting a design wind speed. The record lengths varied between 10 to 30 years. Some 200 wind stations are currently available, but no statistical analysis has been carried out by the SA Weather Service. The database in electronic form only goes back to 1992.

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

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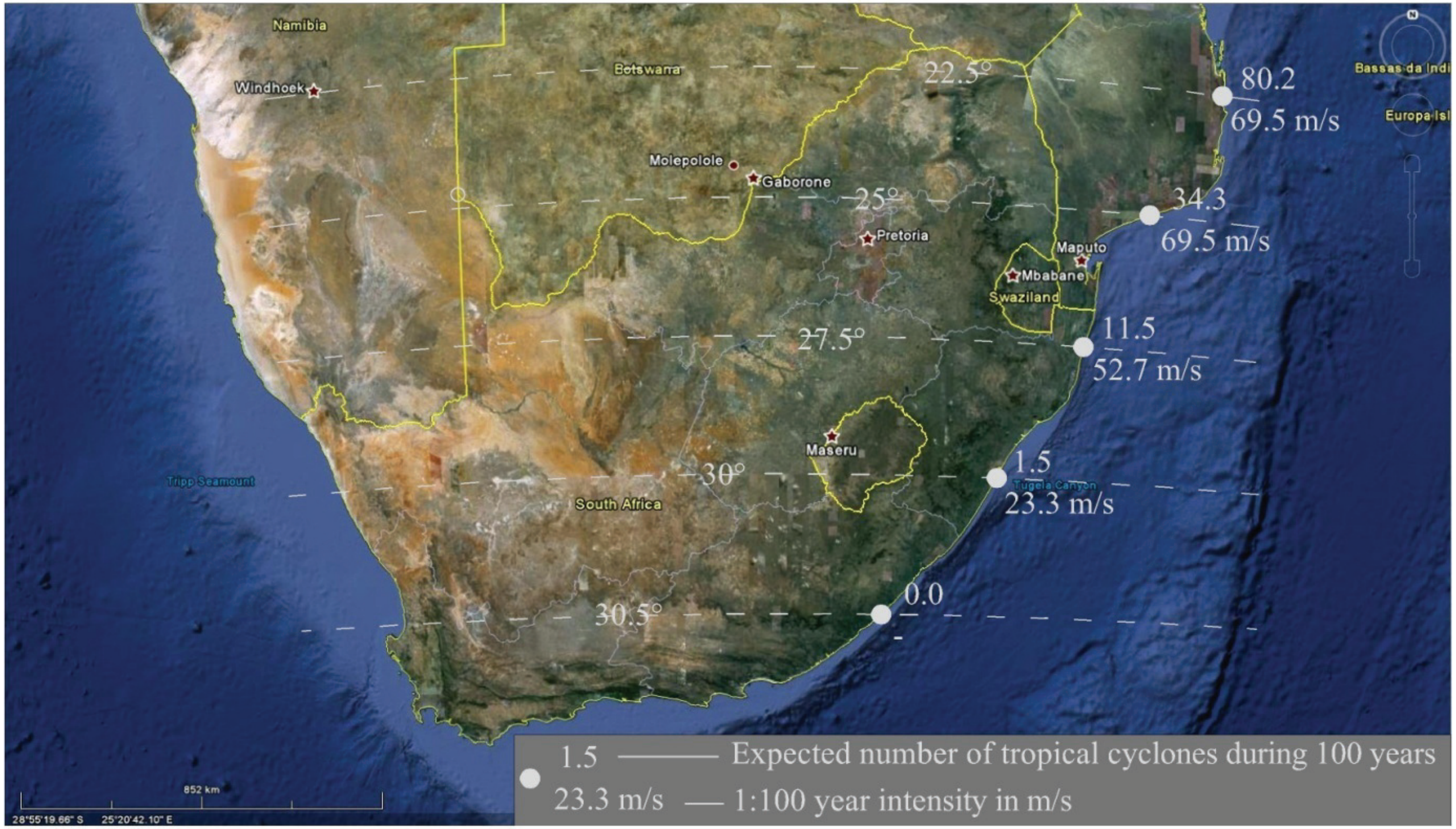


Figure 2.3-1 Tropical cyclone occurrence frequency and wind speed (1 knot = 0.514 m/s) on SA East coast. {White dots and related occurrence frequencies and wind speeds relate to dotted latitude line}. [Adapted from Rossouw, 1999]

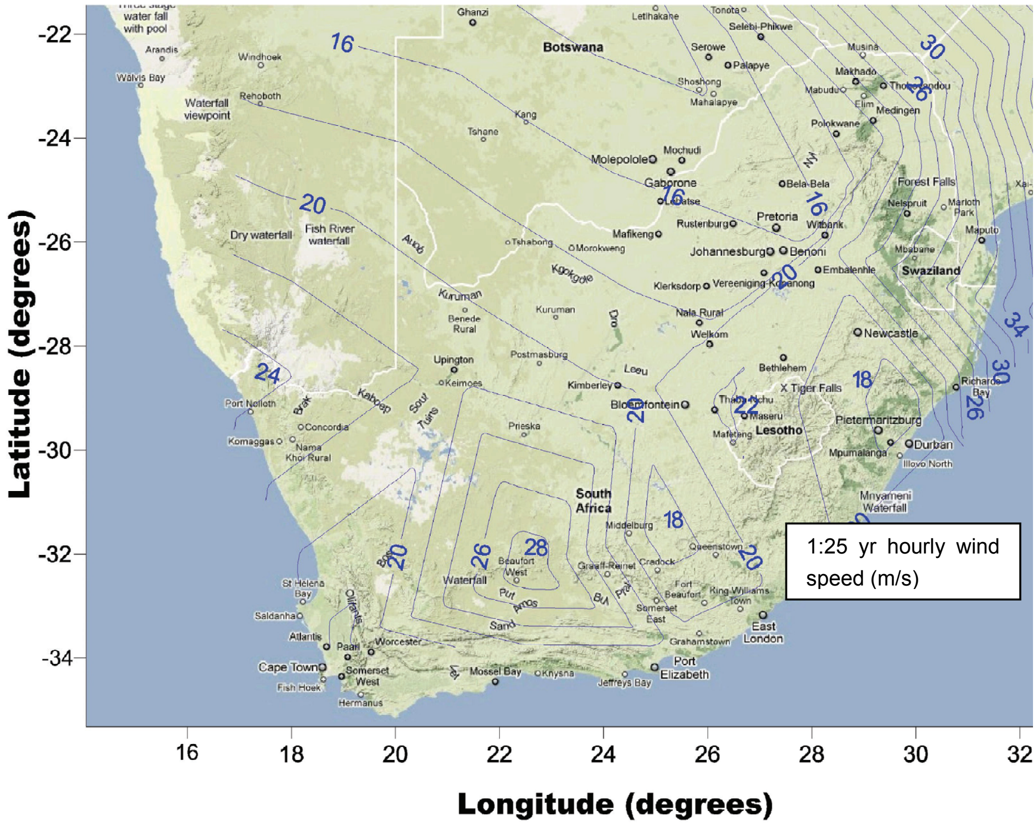


Figure 2.3-2 1:25 year hourly wind speed for South Africa

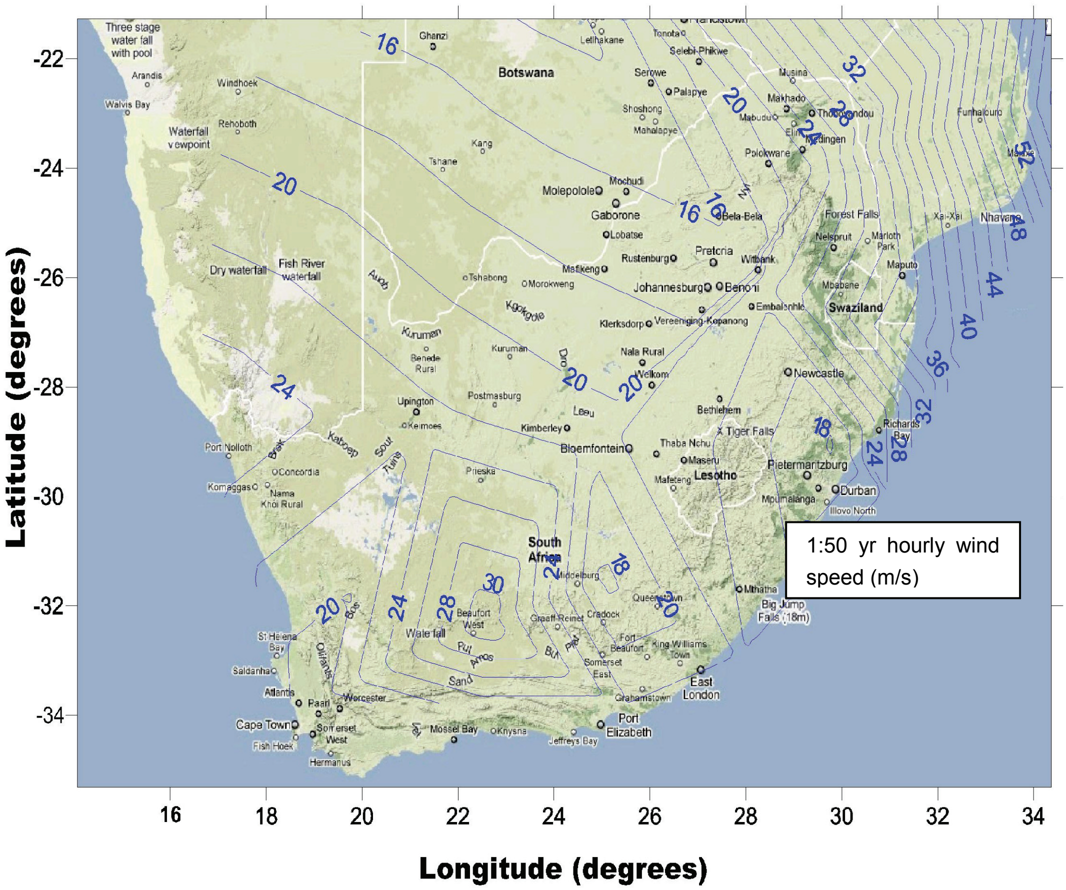


Figure 2.3-3 1:50 year hourly wind speed for South Africa

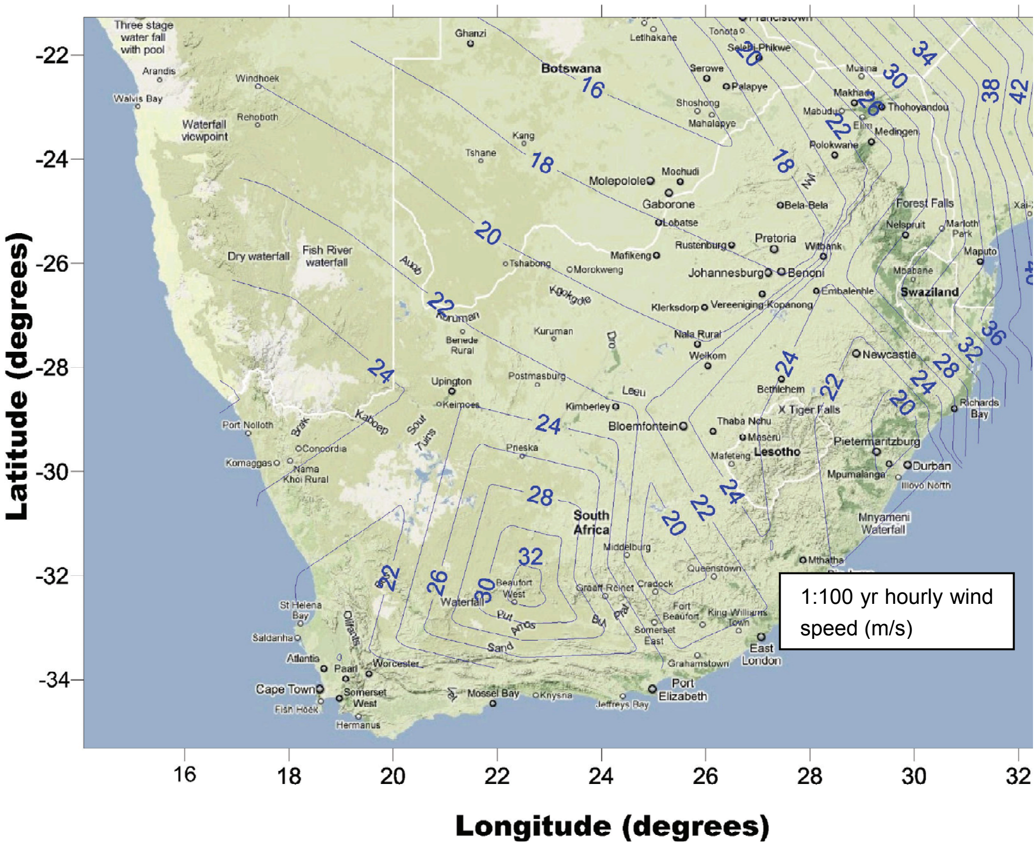


Figure 2.3-4 1:100 year hourly wind speed for South Africa

Milford (1987) also proposed relationships between the 1:50 year hourly average wind speed and other occurrence frequencies as presented in Figure 2.3-5 below.

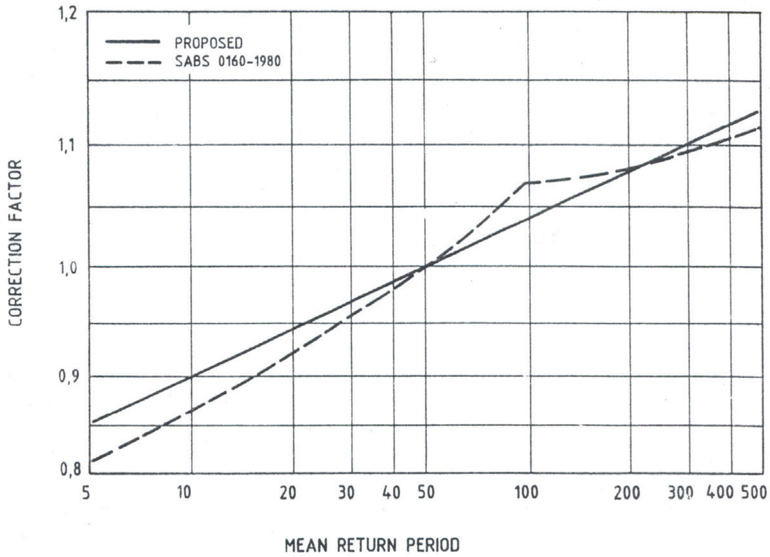


Figure 2.3-5 Milford's proposed (and SABS 0160-1980) relationship between 1:50 year hourly mean wind speed and other return periods (Milford, 1987)

The following guidelines can be followed for determining wind speeds for wave and wind set-up predictions:

a) To adjust the selected 1 hourly wind speed (selected from Figures 2.3-2 to 2.3-4, or Appendix B), for a site or determined from local recordings) for a *particular wind fetch length* of a reservoir, the procedure summarised in the logic diagram as presented in Figure II-2-20 of the Coastal Engineering Manual should be followed (CEM, 2006 – a public domain document – refer to References for web download address).

b) The above referred CEM procedure includes adjustment of the wind speed for a particular wind fetch length of a reservoir in accordance with the wind duration, time (min) required to reach wave generation equilibrium (refer Figure II-2-3 of CEM, 2006) or t_{min} calculated according to the empirical formulae discussed in Section 2.5.3 below. If this duration is longer or shorter than one hour, the adjustment of the selected one hour wind speed is done according to the relationship between the 1 hour mean (U3600) and the longer or shorter (U_t) duration wind speeds in accordance with Figure II-2-1 of CEM (2006) as presented in Figure 2.3-6 below.

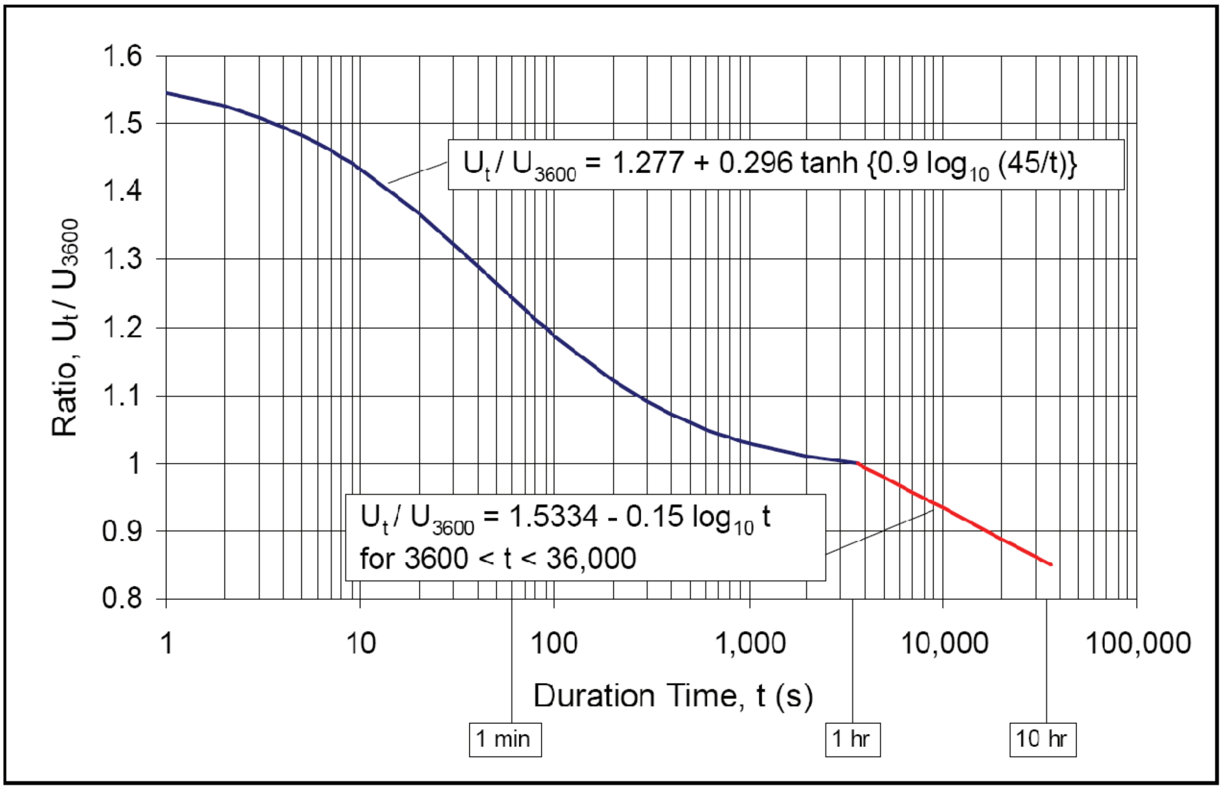


Figure 2.3-6 Ratio of wind speed of any duration, U_t, to the 1-hr wind speed, $U3600$ (CEM, 2006).

2.3.2 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 over which the wind blows. It is common practice to assume that waves are generated from a sector (normally 45º either side of the main wind direction) radiating from the point at which the wave condition is to be determined. In the case of applying empirical methods to derive wave conditions in a dam basin (with normally an irregular plan shape), an effective fetch is usually derived by means of some type of weighting average. Empirical wind wave prediction methods which are recommended to derive wave conditions for dam freeboard purposes, are the three methods treated in The Rock Manual (2007 – a public domain document – refer to References for web download address). One of these methods includes the method as described in the SANCOLD Interim Guidelines (1990) with a slight variation. The recommended three methods as contained in The Rock Manual (2007) are dealt with in more detail in Section 2.5.3 below.

Instead of applying the rather cumbersome methods to derive an effective fetch length for the different empirical wave prediction methods, it is recommended that the maximum straight line fetch length be used in the empirical wave prediction methods proposed in these guidelines for the following reasons:

a) Using the maximum straight line fetch instead of the effective fetch will result in a larger wave height. This will compensate for probable higher design wind speeds (due to wind funnel effects in the dam basin valley) than derived under Section 2.3.1.

b) Since the publishing of the SANCOLD Interim Guidelines (1990) third generation mathematical models became available (refer Section 2.5.1) capable to derive wave parameters from a wind field more accurate than the above referred empirical methods.

It is also recommended that the main wind direction of the design wind be assumed to be the same as the maximum straight line fetch direction towards the dam wall. This is recommended since the available design wind speed data are omni-directional and since a conservative approach is considered appropriate here.

2.4 WIND SET-UP

Wind set-up is 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. Normally wind setup is relatively small compared to other dam freeboard components.

Wind set-up could be determined approximately by means of simplified analytical methods (e.g. Rock Manual, 2007 and Kamphuis, 2002 as presented below) or it could be derived more accurately by means of hydrodynamic mathematical models (e.g. MIKE 11, MIKE 21, MIKE 3 and DELFT3D).

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 is taken as <u>2 times</u> the effective fetch used for wave height computations.

The Rock Manual (2007) recommends the following analytical method for an approximation of wind set-up (n_w) for a simplified case (i.e. a closed water domain with constant water depth):

$$n_w = \frac{1}{2} \cdot \frac{\rho_{air}}{\rho_w} \cdot C_D . \frac{U_{10}^2}{gh} . F$$

Where:

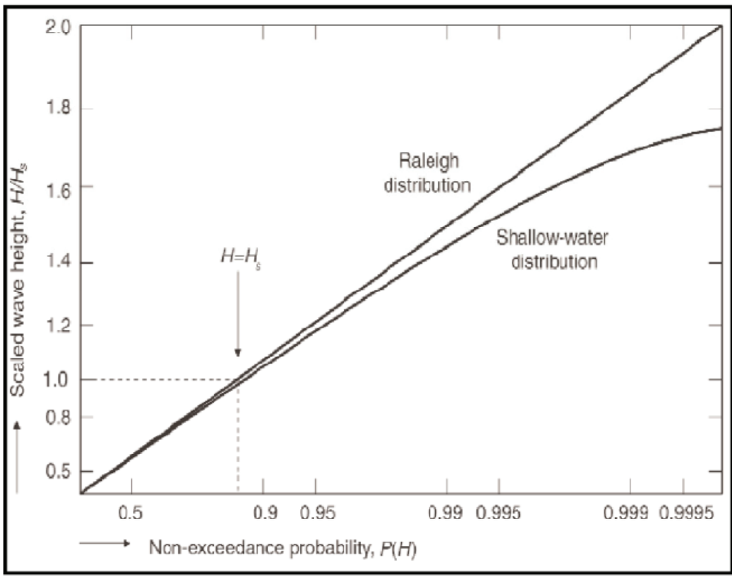
ρ_{air}	=	density of air = 1.2 kg/m³
ρ_w	=	density of water = 1000 kg/m³ for fresh water
C_D	=	air/water drag coefficient (0.008 to 0.003); assume 0.005 after Kamphuis (2002)
U_{10}	=	mean one hourly wind speed as derived in Section 2.3.1 at 10 m above surface
H	=	average water depth = (average volume at FSL)/(average surface area at FSL)
F	=	2 x effective fetch used for wave height computations (Recommendation in this guideline for F = 2 x maximum straight line fetch length)
g	=	gravitational acceleration

2.5 DESIGN WAVE HEIGHT

2.5.1 Wave parameters

It is considered appropriate to briefly describe the wave parameters obtained from both the numerical models and empirical formulae:

- Wave height – usually the significant wave height (H_s), which is the mean of the top third of the wave heights in an irregular wave train. The energy based significant wave height Hm0 (which is obtained from the energy spectrum) is approximately equal to H_s. The characteristic wave height ratios for an irregular wave state with a Rayleigh distribution of wave heights are presented in Table 4.8 (page 357) of The Rock Manual and in more detail in Table 2.5-1 below: e.g. the ratio between $H_{2\%}/H_s \approx 1.4$ and $H_{max}/H_s \approx 2$. $H_{2\%}$ is the wave height in the irregular wave train, which is exceeded 2% of the time and H_{max} is the maximum wave in a reasonably long irregular wave train. These wave heights are relevant in the determination of wave run-up.



H/Hs	Exceedance probability in % according to the Raleigh probability distribution
0.75	32
1	13.5
1.02	12.5
1.07	10
1.1	9
1.22	5
1.4	2
1.51	1
1.63	0.5
1.77	0.2
1.86	0.1
1.95	0.05
2.06	0.02

Figure 2.5-1 Exceedance probability of wave heights in an irregular wave train, according to the Rayleigh distribution (The Rock Manual, 2007)

- Wave period – usually the peak period is used (this is the period in the irregular wave spectrum where the maximum wave energy occurs). T_m-1,0 which is a spectrum related period is more recently used in wave run-up calculations and T_p/T_m-1,0 ≈ 1.1. Tm (mean wave period) is between $0.71T_p$ to $0.87T_p$. T_p/T_m usually lies between 1.1 and 1.25. T_s (significant wave period) is between 0.90Tp and 0.96Tp.
- Waves in deep water (where the wave length is shorter than the water depth) which steepness (wave height/wave length) are greater than 1/7 will become unstable and break (the simplified formula for a deep sea wave length ≈ $1.56xT^2$). When the water depth is less than half the deep sea wave length, the wave is deformed by the depth and can break due to limited depth.
- More detailed information on wave kinematics in deep and shallower water (such as wave shoaling, refraction and breaking) can be obtained from CEM Part II (2006) and The Rock Manual (2007) Paragraph 4.2.4 and more specifically Paragraph 4.2.4.3.

2.5.2 Design waves consideration

The more recently and generally accepted method to determine wave run-up for the design of hydraulic structures is to use the 2% wave run-up ($R_{2\%}$) which is the wave run-up caused by the 2% wave height ($H_{2\%}$). $H_{2\%}$ is the wave height which is equalled or exceeded by 2% of the waves in an irregular wave train. The $R_{2\%}$ is recommended for the wave run-up contributing component to the design dam freeboard. (This is different from the 1990 guidelines which used the so-called significant wave height and the resulting wave run-up. The significant wave height is 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 13.5% of the waves generated by a particular (design) wind speed – refer Section 2.5.1).

For determination of approximate wave conditions, empirical methods (as described in The Rock Manual (2007) and referred to below) are recommended. For more accurate derivations (e.g. for Category II and III dams) the mathematical model SWAN, which are dealt with in more detail below, is recommended.

Wind generated wave conditions on the upstream side of a dam wall are governed by:

a) Design wind speed and duration (dealt with in Section 2.3.1);
b) Effective fetch (discussed in Section 2.3.2 for used in empirical wave prediction formulae);
c) Reservoir depth including bed roughness. This phenomenon is included in both the recommended empirical formulae and the recommended mathematical model, SWAN; and
d) Water depth in the wave approach zone upstream of the toe of the dam wall (dam wall foreshore). In general, wind waves on dam reservoir surfaces fall within the category of deep water waves (i.e. water depth is larger than half the deep water wave length with the deep water wave length = 1.56 x (wave period)^2. However, if the dam wall foreshore depth is less than half the deep water wave length, the wave height can be modified by wave breaking – also by wave shoaling and wave refraction. Using the recommended empirical wave prediction formulae, the shallow water wave adjustment method, presented in The Rock Manual (Paragraph 4.2.2.4) is recommended. The recommended SWAN model includes the effects of wave shoaling, wave refraction and wave breaking.

As indicated earlier in this section, $R_{2\%}$ (resulting from the $H_{2\%}$ wave height) is recommended for the wave run-up contributing component to the design dam freeboard – this implies that 2% of the waves will overtop the dam wall. If more wave overtopping can be tolerated *without endangering the structural stability of the dam wall* (such as in the case of a protected downstream slope of an embankment dam or stable foundation conditions on the downstream side of a vertical concrete dam wall), it could be considered to reduce the recommended design $R_{2\%}$. A generalised guideline in this regard (Ref. UK ICE, 1978) is presented in Table 2.1 below.

Table 2.1 Dam type, design wave height and run-up

Type of dam	Design wave height in terms of H_s wave height (UK ICE,1978)	Wave run-up component to design freeboard in terms of the recommended design $R_{2\%}$ (assuming Rayleigh wave height distribution and linear relation between wave height and run-up)
Concrete dam	0.75	0.55 $R_{2\%}$
Rockfill dam with road on crest	1.0	0.71 $R_{2\%}$
Earthfill dam with road on crest and selected grass on downstream slope	1.1	0.79 $R_{2\%}$

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.

The above UK ICE guideline is very generalised and does not consider the stability against wave overtopping of specific dam wall designs – this guideline should therefore be treated with caution. A more reliable basis for considering the reduction of the recommended design wave run-up ($R_{2\%}$.), is to determine the rate of overtopping for different dam wall crest levels and compare it with available data on tolerable overtopping for different types of dam walls and types of protection.

The method to calculate wave overtopping as presented in Paragraph 5.1.1.3 of The Rock Manual (2007) is recommended together with tolerable overtopping rates as presented in Table 5.4 of The Rock Manual (2007), Table VI-5-6 of the CEM (2006) and in Paragraph 3 of the EurOtop Manual (2007 – a public domain document – refer to References for web download address). [A link to a useful online tool to calculate wave overtopping for a large variation of dam wall configurations are also available in EurOtop Manual (2007)].

2.5.3 Empirical methods for wind wave prediction

For the purpose of a desk-top study, simplified empirical formulae could be applied to determine wave parameters. It is recommended that the following three methods, as described in Paragraph 4.2.4.6 (page 371 to 374) of The Rock Manual (2007), be used to determine wave conditions:

a) Saville method (or SBM method with effective fetch – refer also to SPM (1987) for the version of the SBM formulae which include water depth);
b) Donelan method; and
c) Young and Verhagen method.

The input required by these formulae include wind speed (refer Section 2.3.1), fetch length (Section 2.3.2) and mean water depth. The relevant output data include wave height (H_s) and wave period (either Tp or Ts – these can be converted to Tm-1,0 with the ratios presented in Section 2.5.1 to apply in the run-up formulae).

It is recommended that all three formulae be used for correlation purposes and that the output of the formula which gives the maximum wave height be used. A spreadsheet to calculate the relevant wave parameters with the above referred three formulae is attached in Appendix E.

2.5.4 Mathematical model for wind wave prediction

To enable the calculation of wave run-up, relevant wave parameters (caused by the selected wind velocity) must be determined. The current more sophisticated (and more accurate) method is to apply numerical wave models e.g.:

a) SWAN (Simulating waves nearshore) developed by the Technical University of Delft in The Netherlands by Booij *et al.*, 2006). The model has been made available in the public domain and can be downloaded at: website http://vlm089.citg.tudelft.nl/swan/index.htm. SWAN provides many output quantities including two-dimensional spectra, significant wave height, mean wave and peak period, average wave direction and directional spreading, root-mean-square of the orbital near-bottom motion and wave-induced force (based on the radiation-stress gradient). The SWAN model has successfully been validated and verified in several laboratory and (complex) field cases (see, e.g., Ris, 1997). The SWAN model was developed at Delft University of Technology, Delft (the Netherlands), where it is undergoing further enhancements. It is specified as the new standard for nearshore wave modelling and coastal protection studies.

b) STWAVE. This model was developed by the Coastal and Hydraulics Laboratory – Engineer Research and Development Center Waterways Experiment Station – Vicksburg, Mississippi – US Army Corps of Engineers. The website where more information of this model can be obtained is:
Website http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Software

The main input data required for the above numerical models are dam basin water surface configuration, basin bottom topography, and wind speed and direction over the reservoir (all in grid formation). The main results obtained from the above models include significant wave height (Hs), peak wave period (Tp), mean wave period (Tm), and average wave direction spatially over the surface of the reservoir.

The above numerical models account for all the main factors relevant to wind wave generation including, wave refraction, wave diffraction, wave reflection and wave energy loss due to wave breaking (white capping), bottom friction and wave-wave interaction.

The SWAN model was tested on three case studies: Berg River Dam, Voëlvlei Dam and Bloemhof Dam (See Volume I of this report series).

A short user manual for SWAN has been developed and is attached in **Appendix C**. The software used with SWAN to generate contours and to represent results is SURFER (see **Appendix D**). A spreadsheet to set up the input file for SWAN is enclosed on the CD in **Appendix E**.

Typical SWAN simulated wave height and direction results of Bloemhof Dam for a wind speed of 36 m/s is indicated in Figure 2.5-2. In Bloemhof Dam all the analytical methods used gave wave heights higher than those estimated by SWAN. Bloemhof Dam is a very long and narrow dam and the wave energy is dissipated along the shores of the dam in the narrow reaches. SWAN includes this energy loss in its estimation while the analytical methods do not.

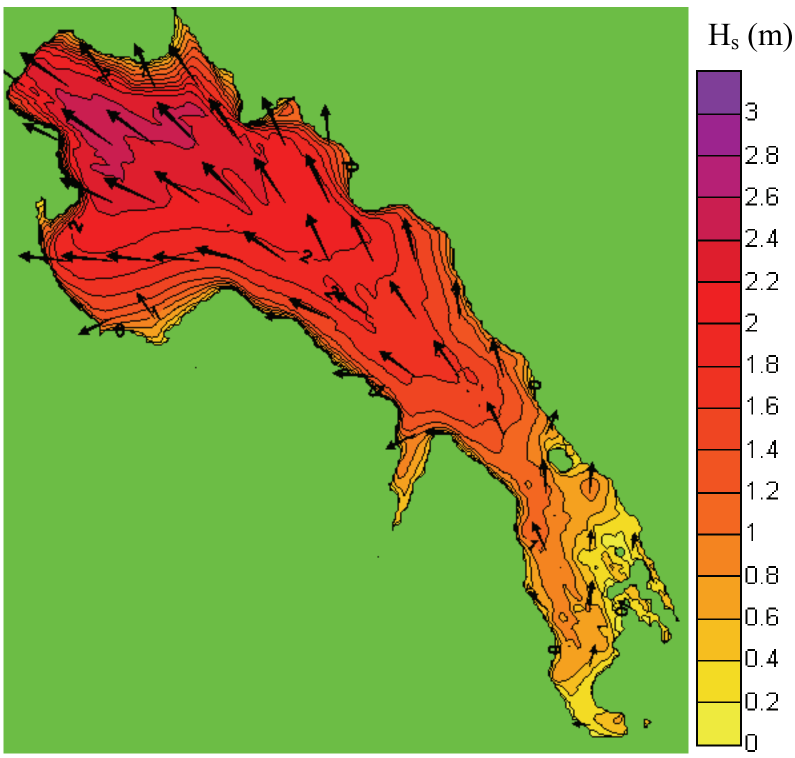


Figure 2.5-2 Simulated wind wave conditions (H_s in m and dominant wave directions) at Bloemhof Dam using SWAN (Wind speed 36 m/s in direction of longest straight line fetch)

2.6 WAVE RUN-UP

Wave run-up is defined as the difference in vertical height from still water level (that would prevail without waves) to the maximum level attained by run-up of the design wave against the dam wall, as defined for an embankment type dam in Figure 2.6-1 below for the $H_{2\%}$ wave height. Wave run-up for both sloped structures (such as embankment dams) and vertical walls are addressed in this section.

2.6.1 Run-up on sloped structures

The empirical formulae as presented in The Rock Manual (2007) are recommended for sloped structures. These formulae are summarised in Box 2.6-1 below. The basis of these formulae is the fictitious surf similarity parameter (ξ) as defined in Figure 2.6-2 below. It is important to note that to calculate the fictitious wave steepness (Hs/L), the wave height at the *toe* of the wall and the *deep water* wave length are used. The formulae in Box 2.6-1 use different wave periods (i.e. Tp, Tm-1,0 and Tm – refer Section 2.5.1) to calculate the deep water wave length and these also reflects in the symbols for the surf similarity parameters (i.e. ξp, ξm-1,0 and ξm). Reduction factors (γ) are applied in the formulae to provide for slope roughness, oblique waves, shallow foreshores and bermed structures.

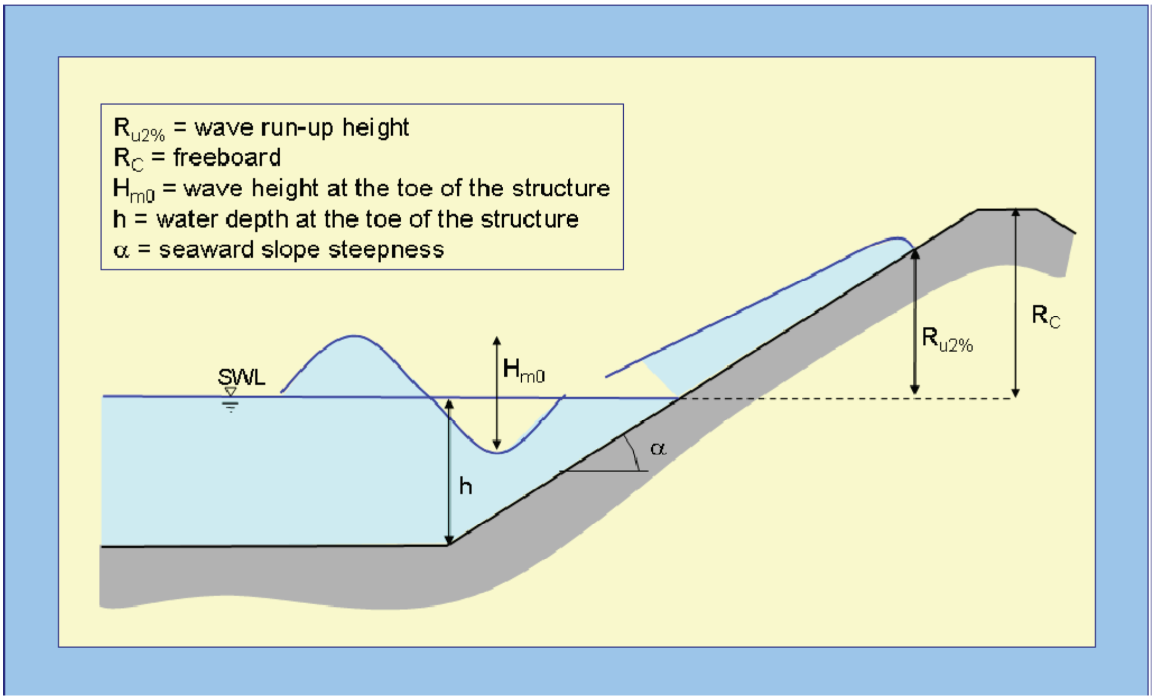
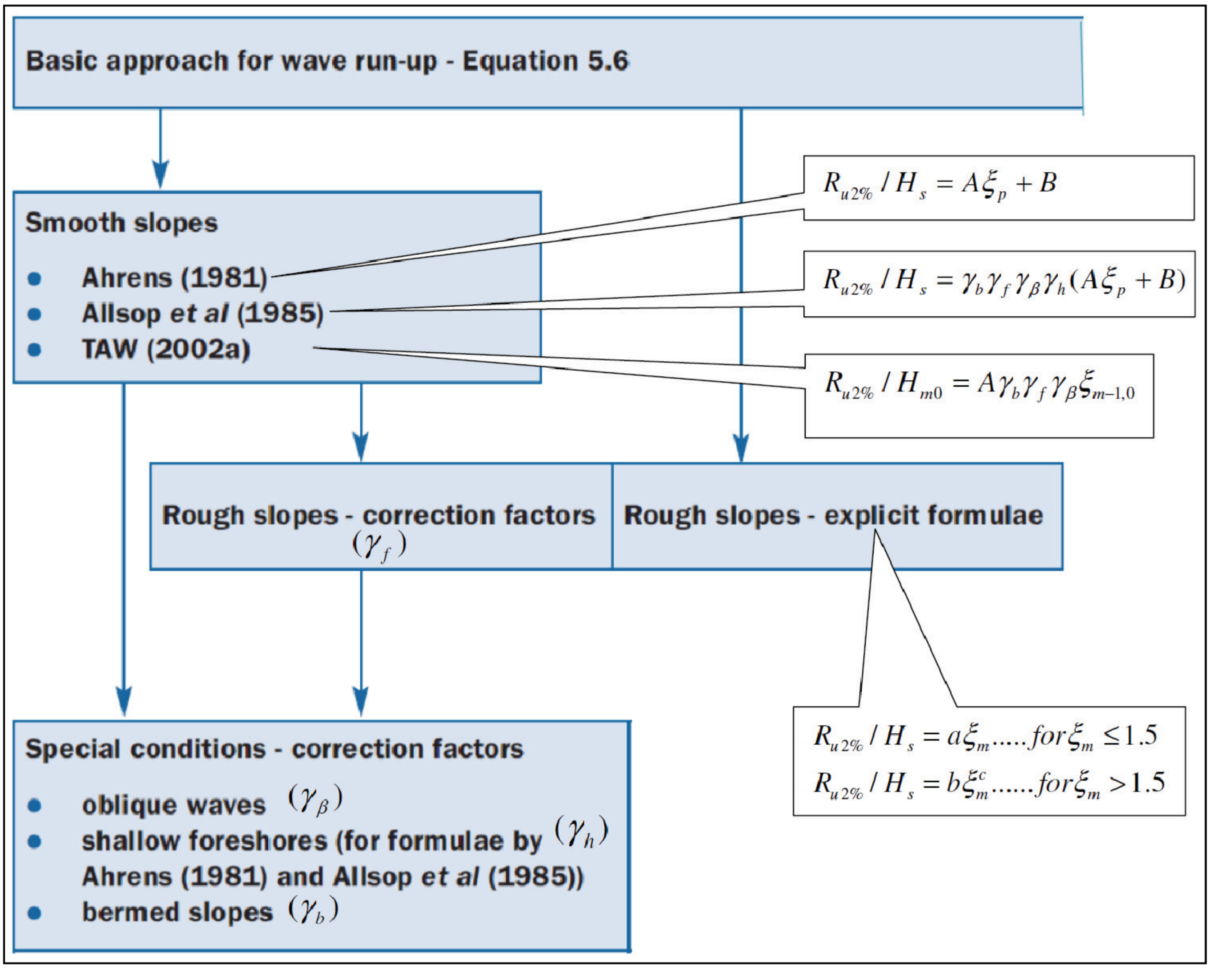


Figure 2.6-1 Wave run-up ($R_{2\%}$) caused by a $H_{2\%}$ wave height on a smooth embankment slope (Pullen et al., 2007)

Box 2.6-1 Recommended wave run-up ($R_{2\%}$) formulae for sloped structures (The Rock Manual, 2007 – Paragraph 5.1.1.2)



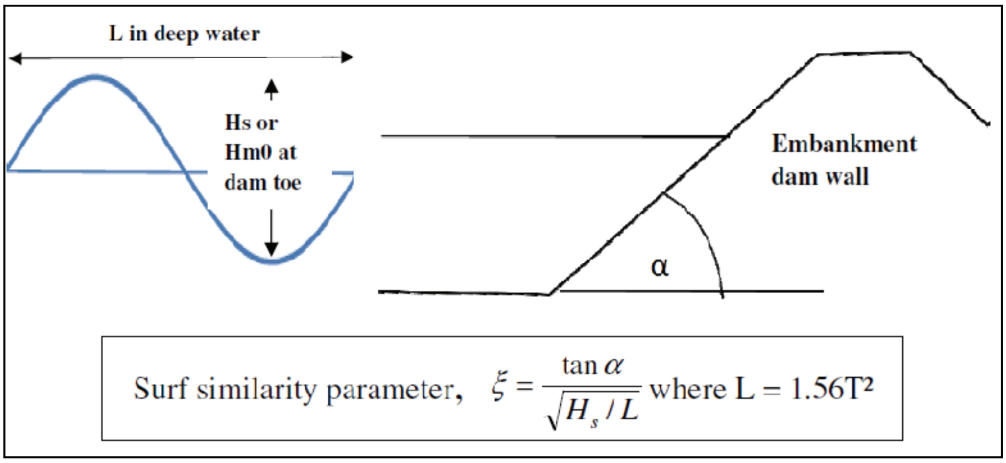


Figure 2.6-2 Definition of fictitious surf similarity parameter (ξ)

2.6.2 Vertical walls

The wave run-up ratio (run-up/design wave) for a vertical upstream dam wall face in deep water is 1.00 (the design wave being the $H_{2\%}$), 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 (United States Army Corps of Engineers, 1984 – refer Figure 7-14). 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.

2.6.3 Other considerations

As indicated earlier in Section 2.5.2, if a tolerable overtopping rate (which would not cause the dam to fail) for a specific design is known, wave overtopping rates could be derived according to the formulae presented in Paragraph 5.1.1.3 of The Rock Manual (2007). The wall crest level which would result in a tolerable overtopping rate for the design wave height could then be determined by trial and error. The EurOtop Manual (2007) contains the same overtopping formulae for sloped structures as in The Rock Manual (2007) but also contains formulae for vertical walls (it also has a link to a useful online overtopping calculation tool).

2.7 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. Observations by Kovacs *et al.* (1984) 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 Gariep Dams. An allowance of 0.5 to 1 m could be made in freeboard calculations in these cases. Alternatively the effect of a low pressure zone could be built into a fully hydrodynamic mathematical model (1D or 2D (depth averaged)), with the initial water level at the dam say 0.5 to 1.0 m lower than the upstream end of the reservoir.

A long-period oscillation or "seiche" might persist long after the waves have died down. According to (Rainchlen, 1983) 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 a subsequent flood.

It is proposed that for Category II and III dams, mathematical models 1D, 2D and/or 3D are used to analyse unsteady flow patterns considering:

- Local wind data;
- Gate operation;
- Inflow flood hydrograph characteristics;
- The reservoir and river bathymetry; and
- Local low pressure effects.

2.8 WAVES AND SURGES DUE TO EARTHQUAKES

2.8.1 Background and Seismic hazard map

Earthquake excitation could cause rapid oscillatory motions of the body of water resulting in standing waves or seiches.

The existing SABS 0160 (1989) seismic hazard map has been updated and it is recommended by (Kijko *et al.*, 2003) that this updated map should replace the seismic hazard contained in the SABS code. The updated seismic hazard map is presented in Figure 2.8-1 below.

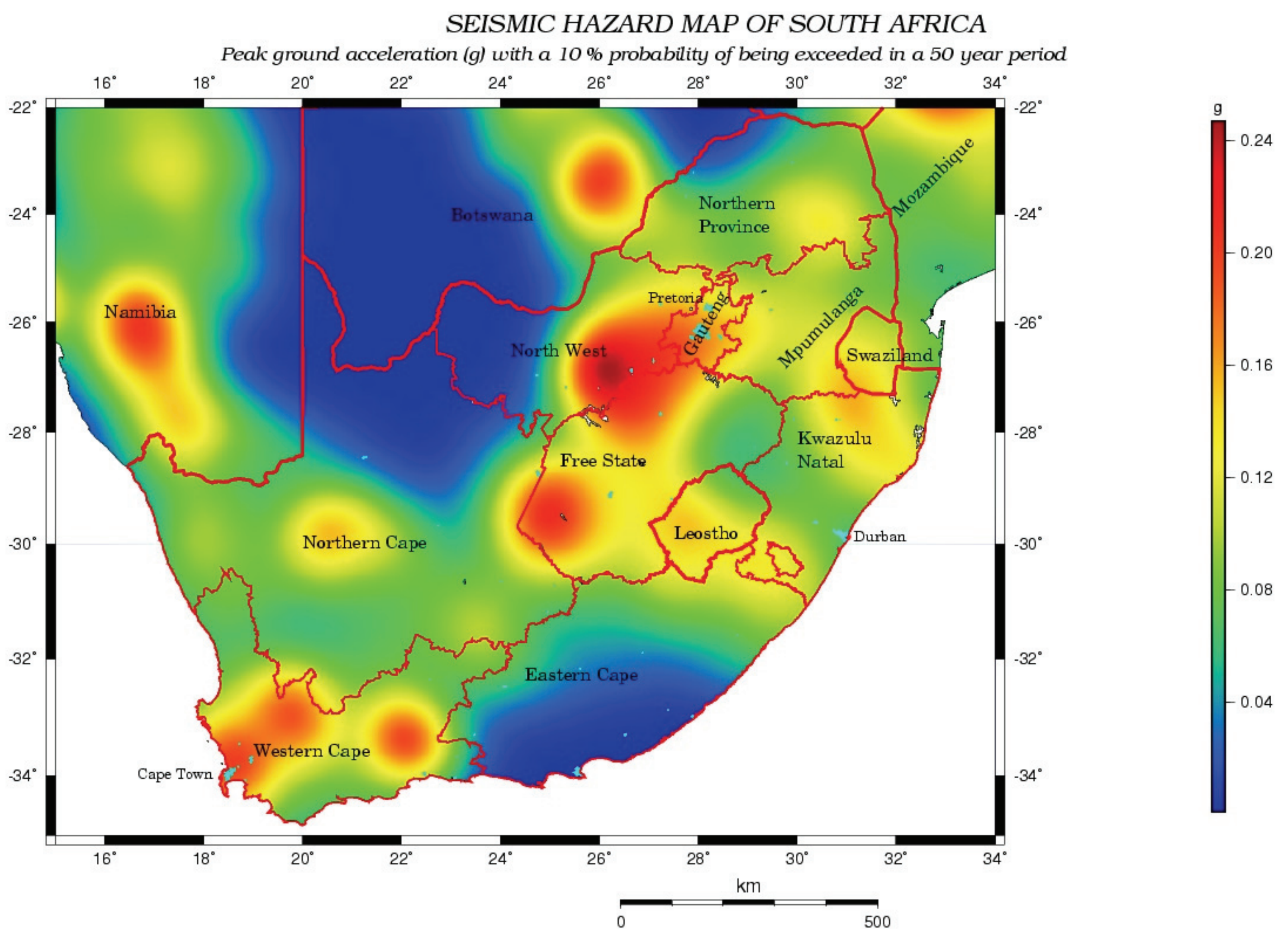


Figure 2.8-1 Seismic hazard map from Council for Geoscience (2003) data showing peak ground acceleration as a ratio of g (gravity acceleration) with a 10 % probability of being exceeded in a 1:50 year return period (Kijko *et al.*, 2003)

2.8.2 Estimation of wave heights in at a vertical concrete dam caused by the horizontal movement of a dam wall due to earthquakes

a) Background on earthquake oscillation periods

The study "Empirical response spectral attenuation relations for shallow crustal earthquakes" by Abrahamson *et al.* (1997) is relevant under this subject. The latter study is based on worldwide data which consists of strong ground motions from shallow crustal events in active tectonic regions, excluding sub-duction events. The number of recordings used was 853 from 98 main-shocks and aftershocks with magnitudes greater than 4.5. Figure 2.8-2 shows the distribution of shock oscillation periods versus number of recordings as obtained from the analysis of the data.

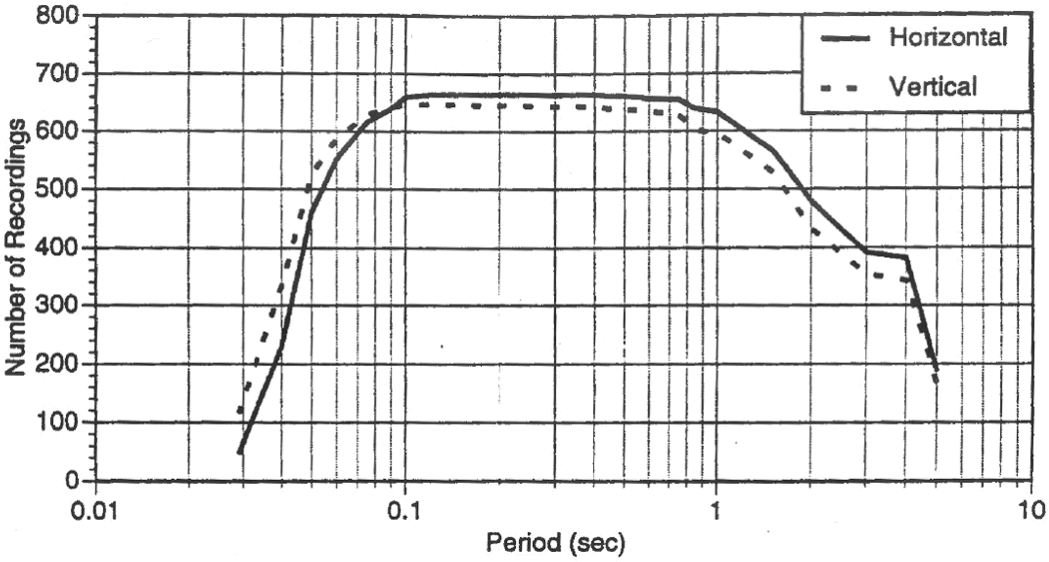


Figure 2.8-2 Distribution of shock oscillation periods versus number of recordings (Abrahamson *et al.*, 1997)

Figure 2.8-3 presents the derived horizontal accelerations versus shock oscillation periods for different shock magnitudes 10 km away from shock source for different shock magnitudes (M).

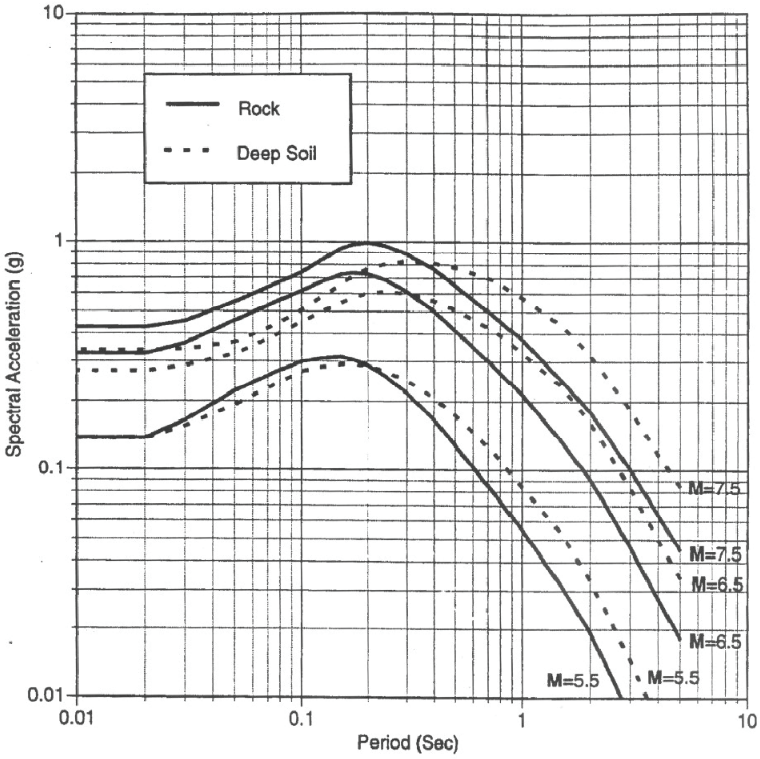


Figure 2.8-3 Derived horizontal accelerations versus shock oscillation periods fordifferent shock magnitudes 10 km away from shock source (Abrahamson *et al.*, 1997)

b) Estimation of seismic generated water wave heights on the upstream vertical face of a dam wall that moves horizontally at different periods and accelerations due to earthquakes

The following steps could be followed to estimate the order of magnitude of seismic generated water wave heights on the upstream vertical face of a dam wall that moves horizontally at different periods and accelerations due to earthquakes (assuming the dam wall as a horizontal moving wave generating paddle):

(i) Estimate a horizontal acceleration from a seismic hazard map (e.g. Figure 2.8-1). (say 0.1 g and 0.2 g);
(ii) Select different shock oscillation periods in the dominant oscillation period range – refer Figure 2.8-3.
(e.g. 0.5, 1, 1.5 and 2s);
(iii) Calculate the amplitude of the horizontal oscillation for the parameters assumed under (i) and (ii) above, by assuming the shock wave orbital motion as circular (this is a reasonable assumption since observed vertical and horizontal seismic accelerations are near equal – refer Figure 2.8-3). The formula for calculating this amplitude is then:

$$Amp = \left(Acc/4\right) \times \left(T/\pi\right)^2$$

Where:

Amp = amplitude of horizontal oscillation = horizontal stroke length

		of wall (S)
Acc	=	horizontal acceleration due to shock (e.g. 0.1 g and 0.2 g)
T	=	period of seismic oscillation = water wave period generated by it
π	=	3.1415

(iv) Calculate the water wave amplitude, assuming the vertical wall face moving as a horizontally moving wave paddle. To calculate the wave height caused by a horizontal movement, the linear wave maker theory can be applied as given in Chapter 6 of Dean and Dalrymple (1992). The online tool of Dalrymple could also be used for this purpose

(website: http://www.coastal.udel.edu/faculty/rad/wavemaker.html).

However, based on Figure 2.8-4 below (obtained from the latter referred reference), it can be shown that the water wave amplitude (H/2) generated by a dam height higher than 15 m is approximately equal to the horizontal seismic stroke length of the vertical wall with a period of 0.5s and longer (point A in Figure 2.8-4).

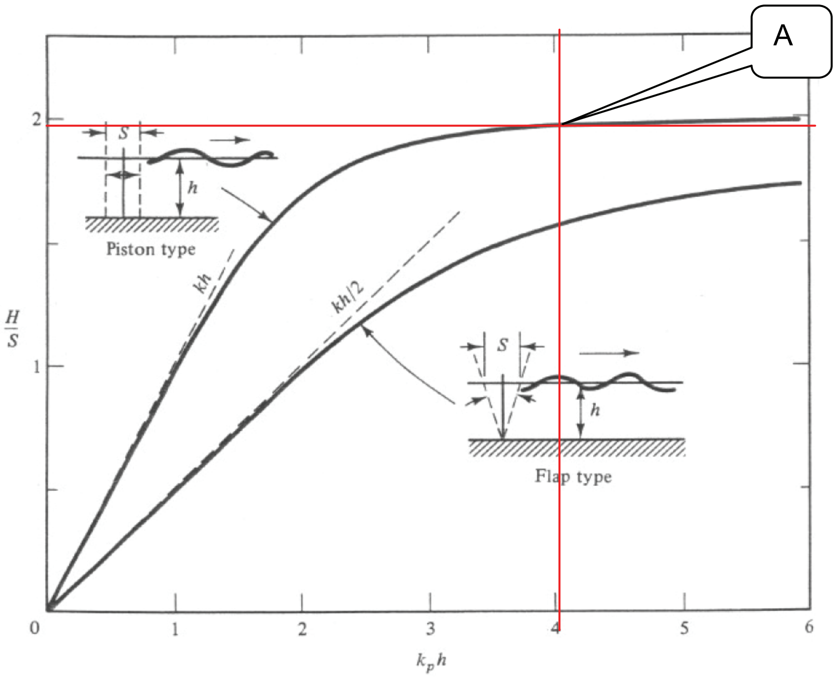


Figure 2.8-4 Plane wave maker theory. Wave height (H) to Stroke (S) ratios versus relative depths (2π/L)*h. Piston and flap type wave maker motions. [kp = 2π/L = 2π/(1.56*T^2) for a deep water wave as would be the case since the oscillation period is short and the water level at a vertical concrete wall is relatively deep]. (Dean and Dalrymple, 1992)

Figure 2.8-5 below presents an example calculation for concrete dam walls higher than 15 m for 0.1 g and 0.2 g horizontal seismic accelerations. The consequent dam wall stroke length and water wave amplitude generated by it on the upstream of the wall are presented.

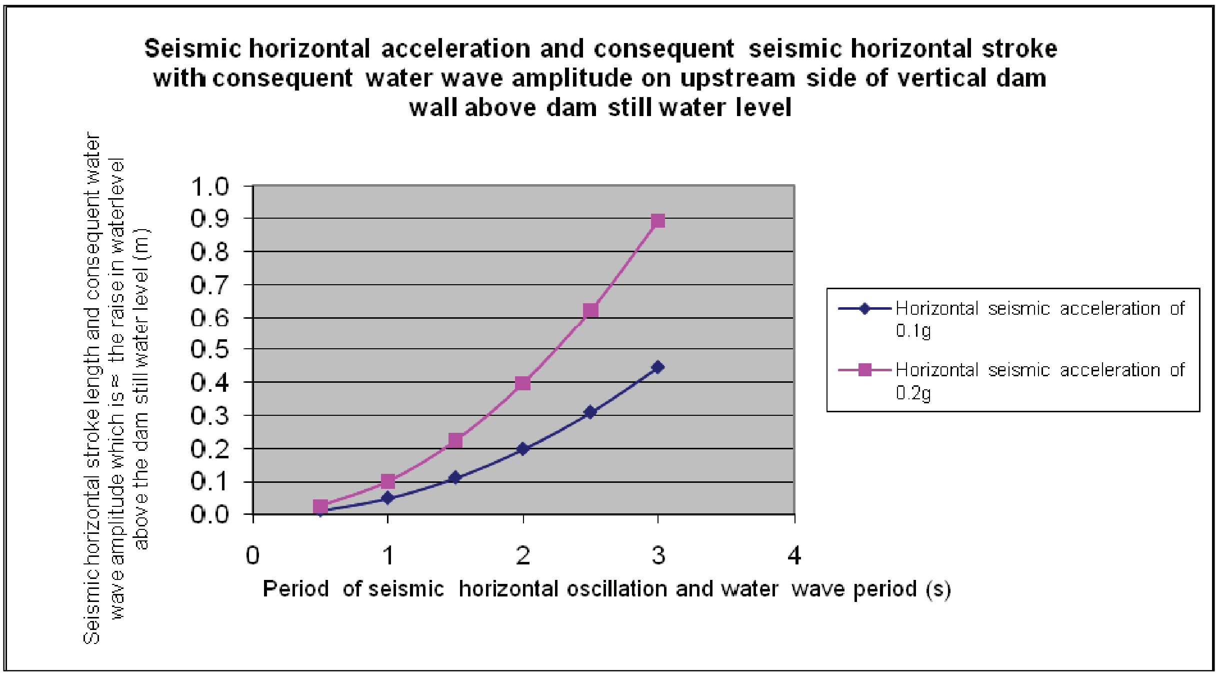


Figure 2.8-5 An example of water wave amplitude calculation for concrete dam walls higher than 15 m for 0.1 g and 0.2 g horizontal seismic accelerations

2.9 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, (1986) namely (i) falls, such as rock masses of 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 severe. Surges or long period waves can give rise to extreme seiching oscillation, run-up and overtopping.

There are various methods that could be used to predict the landslide generated wave height:

- General equations developed from model tests;
- Prototype-specific model tests;
- Numerical simulations;
- Empirical equations derived from field data; and
- Analytical methods.

Noda (1970) developed an analytical method to estimate the wave height, while Davidson (1975) developed model tests to analyze and verify the models by the use of historical data. Huber and Hager (1997) developed simplified formulas based on 2-D physical modelling to calculate the initial impulse wave height. Their simplified method allows the assessment of the initial impulse wave height caused by a rock fall or mass landslide. The impulse wave generated is dependent on the parameters of the falling mass. These include the slide volume, density of falling material, depth of still water, and slope angle at the impact site.

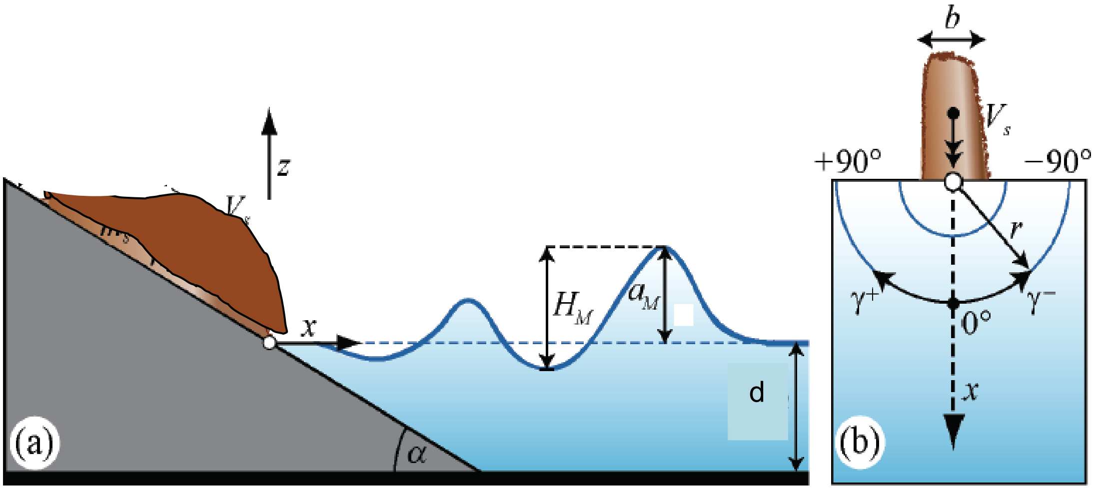


Figure 2.9-1 Landslide surge wave definition sketch (Huber and Hager, 1997)

In flumes and long reservoirs the dimensionless wave height relation is:

$$\frac{H}{d} = 0.88 \cdot \left(\frac{\rho_s}{\rho_w}\right)^{\frac{1}{4}} \cdot \left(\frac{V_s}{bd^2}\right)^{\frac{1}{2}} \cdot \left(\frac{d}{x}\right)^{\frac{1}{4}} \cdot \sin\alpha$$

Where:

H/d	=	Dimensionless wave height
d	=	Water depth
V_s	=	Slide volume falling into the reservoir (i.e. the volume of water displaced, in m^3)
b	=	Slide width (m)
$\frac{\rho_s}{\rho_w}$	=	Density ratio of the slide material to water
α	=	Impact angle
d/x	=	Dimensionless travel distance

In pools, propagation direction γ is an additional parameter. The propagation distance is replaced by the radius r measured from the slide impact centre and the equation becomes:

$$\frac{H}{d} = 1.76 \cdot \left(\frac{\rho_s}{\rho_w}\right)^{\frac{1}{4}} \cdot \left(\frac{V_s}{bd^2}\right)^{\frac{1}{2}} \cdot \left(\frac{d}{r}\right)^{\frac{2}{3}} \cdot \sin\alpha \cdot \cos^2\left(\frac{2\gamma}{3}\right)$$

Where:

r	=	Radius measured from the centre of slide impact
γ	=	Propagation direction

Huber and Hager (1997) suggest that the formulas could predict the wave height with an accuracy of between ±15% ~ ±20%. The limit of validity for these formulas is for x/d<100 for 2-D situation and r/d<30 for 3-D cases.

It should be noted these simplified formulas should be used with physical and/or numerical model studies. In recent years with the advances in computer science, it is now possible for practicing engineers to analyse the wave heights more efficiently. Computer software if applied correctly could predict the wave height accurately and prove to be a useful tool.

In a comparison of the computer models and physical models it was found that the relative mean square differences between measured and computed first wave peaks never exceeded 20% if the probes near the impact point and the wave absorber are excluded. From an engineering point of view, this may be considered a satisfactory performance of the presently validated computational model (Zweifel *et al.*, 2007).

The propagation of surge waves in a reservoir could be carried out by hydrodynamic mathematical modelling, by using 1-D models, or in more complex reservoir layouts 2-DH or even 3-D models.

3. COMBINING FREEBOARD COMPONENTS

3.1 DETERMINISTIC APPROACH

The recommended design flood (RDF) routed through the dam 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 (SANCOLD, 1986) with flood surcharge freeboard only, is an extreme flood considered appropriate for the specific structure which may cause substantial damage to structures 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 SED at non-overspill crest, i.e. where overtopping is not allowed. However, the other case (RDD surcharge and other freeboard components) must also be checked to ascertain the most critical condition.

In the "Guidelines on Safety in relation to Floods" (SANCOLD, 1986)", spillage under any of the generally accepted criteria must not endanger the safety of the structure. These criteria are not listed in that document, but are proposed here.

The important conditions are grouped in combination numbers (Table 3.1) that should be tested as shown in Table 3.2. All conditions in the combinations mentioned and indicated in Table 3.1 (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 should be duly taken into consideration. Adjustments for direction of wind and possible wind funnelling effects should be made when specific data are available to substantiate such adjustments.

Adjustments for uncertainties in the hydrology are to be allowed for on the basis of site specific conditions as well as sensitivity analyses, e.g. the difference between the flood surcharge for a particular flood and that for possible smaller/larger flood(s).
The possibility of simultaneous occurrence or not of the flood peak and maximum wind speed should be considered based on local data.

Table 3.1 Proposed Design Combinations of Freeboard Conditions to be considered with the RDD surcharge*

Combination number	RDD Surcharge	Wind wave and run-up 100-year event (a)	Wind set-up	Surges and seiches	Earth-quake wave	Land-slide wave (b)	Flood outlets (c)
1	X	X					
2	X	X	X	X			
3					X		
4	X					X	
5	X	X	X	X			X

Notes: * Assumed starting level is FSL

a) For cyclonic conditions extra allowance for wind waves is to be made based on local data.
b) Landslides are considered with storm events.
c) At least twenty five percent (25%) of crest gates or flood outlets considered inoperable (Also refer to Section 4.5).

Table 3.2 Recommended minimum values for applicable freeboard (FB) criteria in terms of the design combination numbers (Table 3.1) as well as related Dam Category and RDD

Dam size	Dam category, Floods and Freeboard criteria	Hazard rating		
		Low	**Significant**	**High**
Small (H < 12 m)	Dam Category RDD **FB Criteria**	I Q_{20}-Q_{50} **1**	II Q_{100} **1**	III Q_{100} **2, 5**
Medium (H = 12-30 m)	Dam Category RDD **FB Criteria**	II Q_{100} **1; 5**	II Q_{100} **2; 5**	III Q_{200} **2; 3; 4; 5**
Large (H > 30 m)	Dam Category RDD **FB Criteria**	III Q_{200} **2; 3; 5**	III Q_{200} **2; 3; 4; 5**	III Q_{200} **2; 3; 4; 5**

NOTES:

a) The RDD criteria as reflected in the Guidelines on Safety in Relation to Floods (1991) have for the sake of convenience been included in Table 3.2. The Recommended Design Discharge Flood (RDD) is given in terms of Annual Exceedance Probability (AEP). Q_T is defined as the peak flood discharge with an AEP of 1/T.

b) Unforeseen events described in Section 4.1.1 and practical aspects of Section 2.2.1 should be considered when selecting the freeboard criteria.

Should a change be made to the RDD values in a revision to SANCOLD, (1991), then the new values will supersede 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 3.3. These practical rules of thumb are often applicable to small dams and medium sized dams with a low hazard rating and also provide a check on freeboard calculations. These practical guidelines are also discussed in further paragraphs dealing with different types of dam.

Table 3.3 Simplified practical freeboard guidelines

Type of dam	Minimum total freeboard (m)	Minimum difference in level between stillwater RDD surcharge level and non-overspill crest (m)	Remarks
Earthfill (Category I)	0.8	0.5	-
Earthfill (Categories II & III)	-	1.5	For RDD
Rockfill (Categories II & III)	-	1.5	For RDD
Concrete (Categories II & III)	1.5	1.0	-

Note: Calculated freeboard for Category I and small Category II dams can be replaced from the values in this table.

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**.

3.2 RISK ANALYSIS APPROACH

3.2.1 Objectives of assessment

The motivation for considering a risk basis for the treatment of freeboard for dams derives from the general trend in the application of risk assessment as a rational basis for safety assessment of public facilities. Risk assessment is also allowed specifically for dam safety assessment in South Africa. Risk based guidelines for the various elements of dam safety, such as the provision of freeboard, will enhance such practice and its continuous development. Risk assessment provides a useful integral view of the effect of various hazards (threats) and failure mechanisms, which can be compared to independently set risk criteria. It is however not sufficiently operational to serve as the basis for design, and is therefore used in a complementary manner to deterministic freeboard calculation procedures.

4. DAM SPECIFIC FREEBOARD REQUIREMENTS

4.1 GENERAL REMARKS

4.1.1 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:

a) 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.

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

c) 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.

d) Over-sophistication: 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) Second line defence: 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, emergency auxiliary spillways (fuseplugs).

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.

4.1.2 General conditions

Other conditions having implications on freeboard are:

a) Controlled versus uncontrolled spillways

Uncontrolled spillways lend themselves to much more definite calculations 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, except at well controlled major dams such as Vaal, Bloemhof, Grootdraai and other dams.

b) 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 or catering for the SED. 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 SED from overtopping the main embankment. Breaching sections should be designed to limit sudden increases in outflow.

c) Shape of hydrograph

For each flood a number of hydrograph shapes should be investigated, 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.

d) 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.

e) Use of wave or splash walls

The use of wave or splash walls 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 RDD 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.

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.

4.2 SPECIAL REQUIREMENTS FOR EARTHFILL DAMS

4.2.1 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 principal consideration 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 on large rivers the non-overtopping requirement from a safety point of view is normally dominant in the provision of freeboard (SED 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 RDD 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 FSL and good flood absorption characteristics.

4.2.2 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 construction 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 may vary from 1 to 2.5% depending on site conditions.

4.2.3 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 RDD. The adequacy of the top portion of the zoned embankment including the filters to prevent piping during the surcharge event should be evaluated.

4.2.4 Parapet walls

The use of wave walls to provide freeboard allowances for embankment dams may be considered on a case-by-case basis (see also 4.1.2 (e)). The following criteria are proposed to be met:

- Normally the parapet or wave deflection wall may only replace the portion of the freeboard needed to prevent overtopping from wave run-up. 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.

- Foundation and embankment settlement that would affect 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.
- Wave walls should be continuous and level. All joints should be watertight to prevent concentration of flow.

4.2.5 Minimum requirements: Category I Dams

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 RDD surcharge level for a low hazard dam, while for significant and high hazard ratings it is **1.0 m**.

4.2.6 Overtopping during extreme floods (SED conditions)

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

- Earthfill is erodible;

- 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; and

- 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.

4.2.7 Fuse plugs

Fuse plugs or emergency spillways are specially designed embankment sections with the purpose of breaching during extreme flood conditions or upstream dam failure in order to protect the main wall from failing. The breaching level depends on economic factors in each case. Fuse plugs are often designed to breach for floods between the RDD + the 25 year wind wave condition and the SED. 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.

4.2.8 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 and riprap. Care must be taken with such applications and might require the use of large scale models or mathematical modelling to prove effectiveness.

4.3 SPECIAL REQUIREMENTS FOR ROCKFILL DAMS

4.3.1 Characteristics of rockfill dams

Rockfill embankments generally have considerably steeper slopes than those of earthfill. Rockfill also has the characteristic that the roughness of the upstream face ranges between either very smooth (in the case of a concrete faced rockfill dam) 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.

a) <u>Smooth upstream face</u>

The smooth upstream face of a 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.

b) <u>Rough upstream face, exposed rock surfaces</u>

The upstream face of a central impervious core rockfill dam is usually composed of the coarsest rock material in the upstream rockfill zone 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.

4.3.2 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 possible damage that might occur. Most modern rockfill dams are built subject to very careful quality control, with good compaction techniques 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 super elevation 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 abutments 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 at 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 super elevation against settlement should be specified. Settlement, when eventually occurring will then result in a lower central portion.

4.3.3 Other precautions against overtopping failure associated with minimum freeboard

a) Reinforcement of the downstream face

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

b) 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 Figure 3, Appendix A).

4.3.4 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.

4.4 SPECIAL REQUIREMENTS FOR CONCRETE DAMS

In the calculation of freeboard for Categories II and III concrete dams use the Q_{100} and Q_{200} floods respectively to determine the recommended design discharge. The total freeboard will depend on the fetch and estimated wave height but a minimum of **1 m** between RDD surcharge level and non-overspill crest should be maintained unless otherwise calculated. Keep, wherever possible, the SED 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 SED 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 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 (RDD) can be passed with the gates open, and at or below the normal FSL. The gates themselves must also have a marginal wind freeboard to counteract them being overtopped by wind waves. In some cases gates are designed to be overtopped allowing to some extent for inoperable possibilities. <u>Due allowance should be made for each particular case for a proportion of the gated outlets that might not work</u>, 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 few gates, or 25% of the gates if there are a large number, should be considered inoperable.

4.5 SPECIAL REQUIREMENTS FOR COMPOSITE DAMS

<u>A composite dam</u> is a combination of some of the above types of dams, for example an embankment dam with concrete spillway or 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.

5. CONCLUSIONS AND RECOMMENDATIONS

Chapters 1, 2, 3 and 4 above discuss the concept of freeboard, the various quantitative components thereof, how to calculate them and combine 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 3.1 and 3.2. Some of the components may be given less emphasis, depending on the size and importance of the dam, others may not, due to the uncertainly of hydrology. Practical guidelines are given in Table 3.3.

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 discussed and relates to cost and risk.

Some of the salient aspects of these Guidelines in comparison with the previous SANCOLD (1990) guidelines are:

a) The 1990 SANCOLD Interim Guidelines on Freeboard for Dams had a good scientific basis and only minor changes are proposed in the methodology;

b) The wind data and Milford map has been plotted for 1:25, 1:50 and 1:100 year 1 hour duration wind speeds. Such regionalized maps could be used for Category I and II dams, but for any detail design studies local wind data should be analysed;

c) Wind wave height, run-up and set-up calculation could be based on the Rock Manual. In all other cases the SWAN model should be used to determine wind wave height;

d) The main difference with the 1990 guidelines method is that $H_{2\%}$ is calculated with the new methods, which is 1.4 times higher than H_s calculated with the 1990 methodology. It should be noted that H_{max} is still 1.4 times higher than $H_{2\%}$ and 2 times H_s;

e) Unsteady flow patterns in reservoirs such as seiches, oscillations, flood surges, etc. should be simulated by mathematical hydrodynamic models. For planning purposes and for Category I dams a one dimensional (1-D) model could be used, but for Category II and III dams 2-D or 3-D models are recommended;

f) The combination of freeboard components could follow a deterministic approach similar to the method used in the 1990 guidelines, with some revisions to the combination of scenarios proposed;

g) A risk analysis procedure that incorporate component scenarios could be used complementary to the proposed deterministic approach, and could be considered especially for detail design studies of Category II and III dams; and

h) Minimum dam specific freeboard guidelines have been proposed similar to the 1990 guidelines, but with more scenarios considering the hazard class of a dam.

The following recommendations are made:

a) Adopt freeboard figures developed from calculations above and test their adequacy under a variety of hydrological scenarios and spillway designs;

b) Calculate the volume and cost of the extra height of dam needed;

c) Investigate whether a less expensive solution in the form of fuse plug, parapet, wave wall or gated outlet could reduce the bulk of the dam required to establish the necessary freeboard crest level;

d) Evaluate the greater or lesser risk of failure of the scheme thus devised against the original calculation in (b) above;

e) Check the design again against all occurrences considered in (a) above, individually and collectively where appropriate; and

f) Do final modifications in crest level upwards to increase safety margin, or downwards to reduce cost to arrive at an optimally balanced freeboard value and hence crest level.

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APPENDIX A

FREEBOARD DRAWINGS

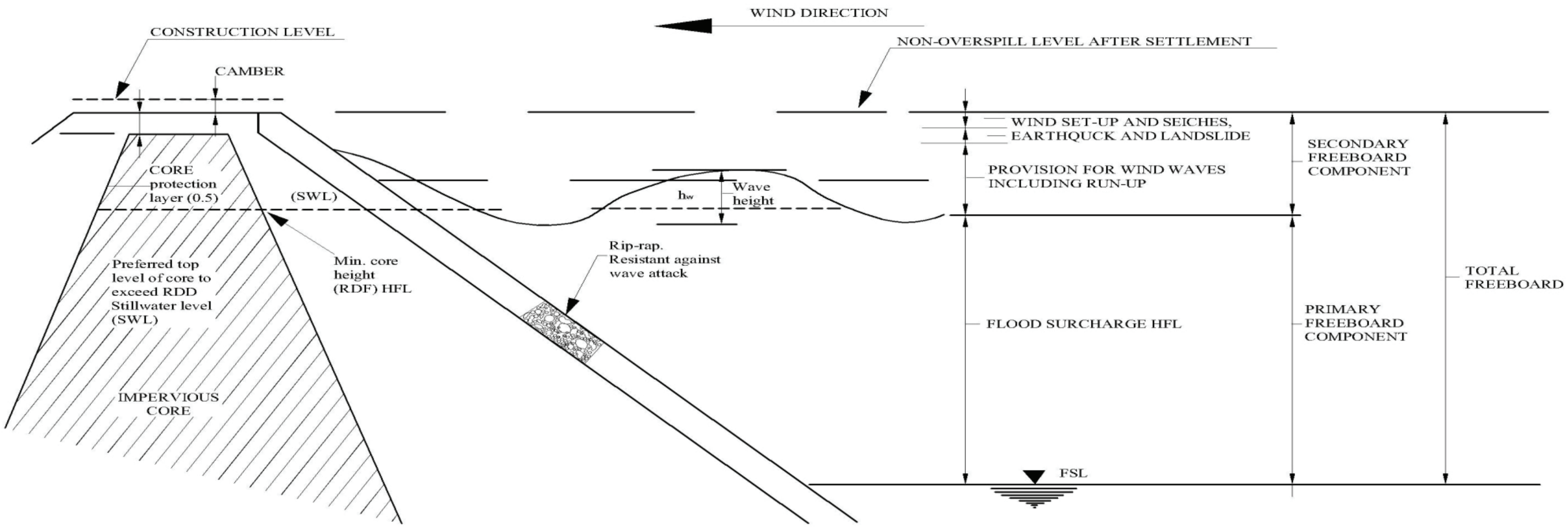


Figure A-1 Definition of Freeboard (for the case of an embankment dam)

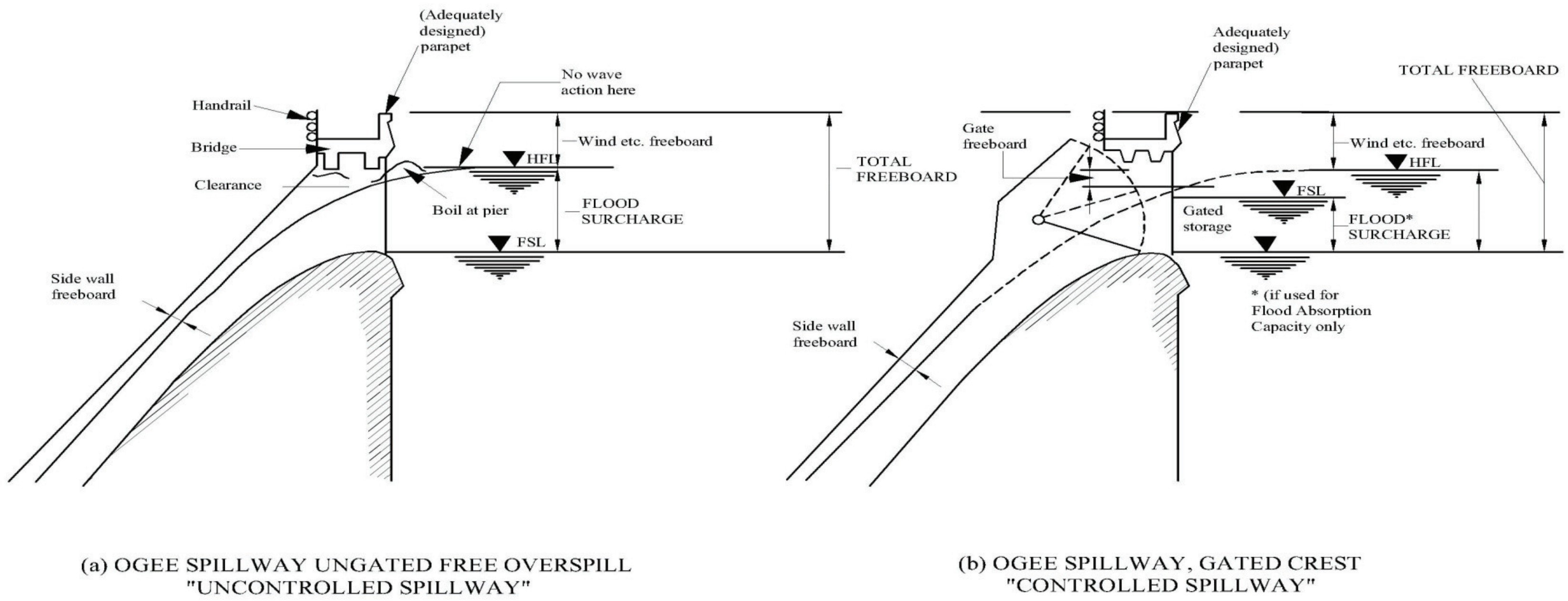


Figure A-2 Detail of crest treatments for uncontrolled and controlled spillways

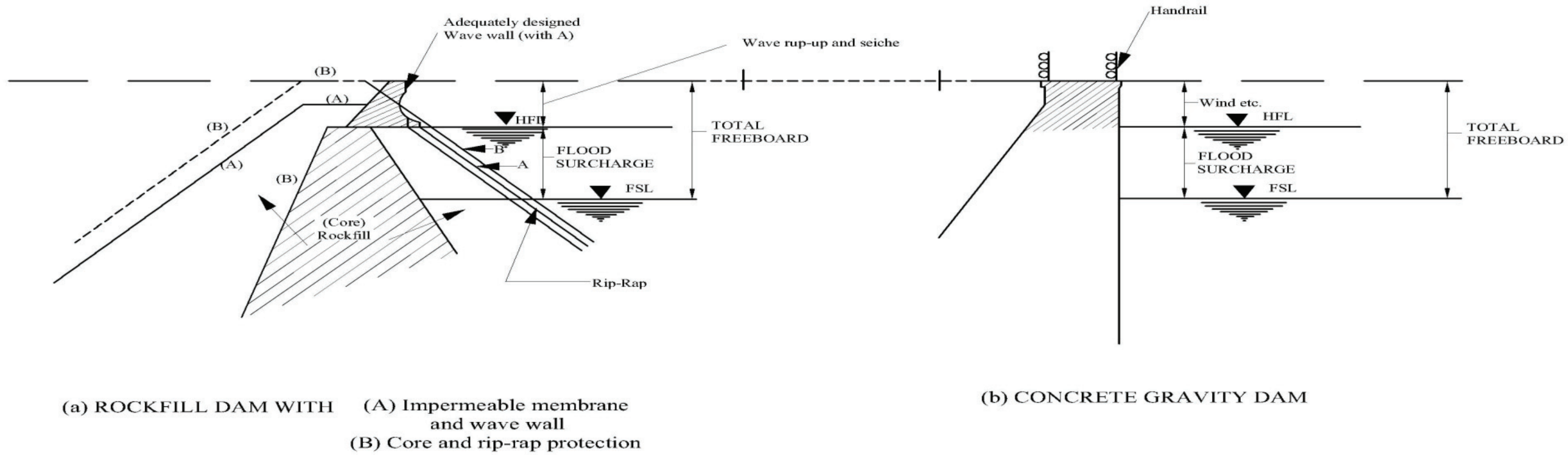


Figure A-3 Crest treatments for rockfill and concrete gravity dams

APPENDIX B
Wind data for South Africa and Namibia

HIGHEST OVER LAND (10 m ABOVE SURFACE) 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
EASTCOURT	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
PIETERMARITZBURG	2.03	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

APPENDIX C

Short user manual of SWAN

1. Introduction

SWAN is an acronym for **S**imulating **WA**ves **N**earshore.

The purpose of this Customised User Guide is to provide a basic procedure on how to use SWAN to obtain realistic estimates of wave parameters (especially wind generated wave heights) on inland reservoirs. This document contains information only relevant to the procedure followed by the author.

It is strongly recommended that users who are keen in making use of SWAN for more complex problems read the SWAN User Manual which can be downloaded free of charge from http://130.161.13.149/swan/download/info.htm. At the time of preparing this document SWAN 40.51 was the latest version. Furthermore, the functionalities and limitation of SWAN can be obtained from the SWAN User Manual.

2. Software required for modelling

The following software packages were utilised by the author.

(a)	AUTOCAD	- For tracing contours
(b)	DXF2XYZ	- For converting coordinates into XYZ
(c)	Microsoft Excel	- For managing large data sets
(d)	SWAN	- For computation of Wave Heights
(e)	Surfer 8	- For digitising and presentations

3. Units and Coordinate system

In SWAN all quantities, input and output parameters are expressed in S. I. units. SWAN operates in a Cartesian coordinate system or in a Spherical coordinate system, i.e. flat plane or on spherical earth. In the case of inland reservoir design the Cartesian coordinate system is more relevant.

In the Cartesian system, all geographical locations and orientations are defined in terms of one common origin (0,0). This origin may be chosen totally arbitrarily by the user. A computation grid is defined in relation to the chosen origin. The user defines the geographic location, size, resolution and orientation of the computational grid. Regular (rectangular) grids are most often used and are preferable for inland reservoir. The computational grid parameters to be specified are illustrated in the Figure 1 below.

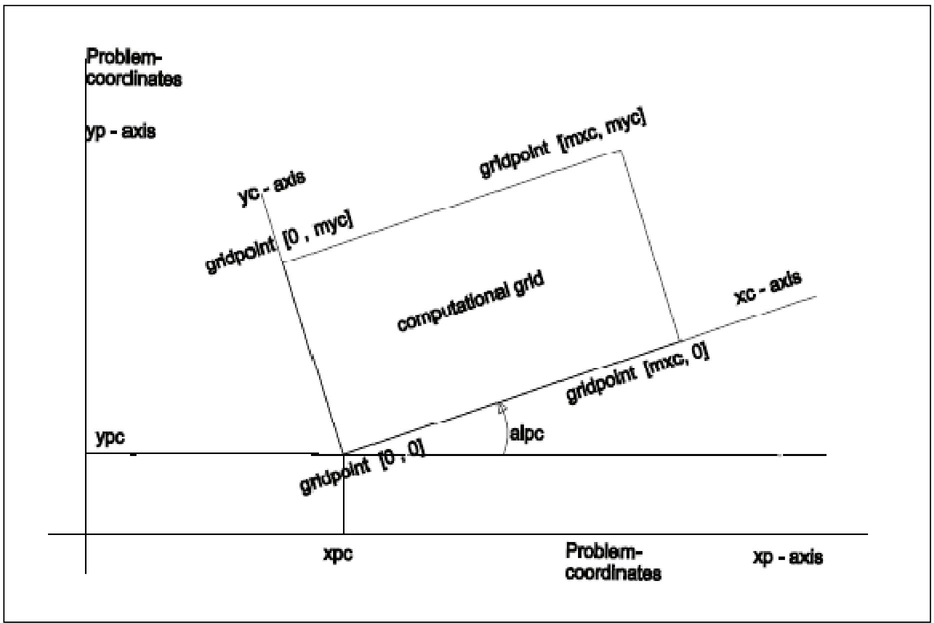


Figure 1: Generalised illustration of problem coordinate system (e.g. available coordinate system of bed topography of problem case investigated) in relation to chosen grid system best suited for SWAN.

4. PROCEDURE

Step 1 – Data Acquisition

The data can be obtained in several forms which include but are not limited to the following:

1. Raw topographic survey data (x,y,z) of the dam basin
2. A satellite image of the dam basin from Google Earth
3. A CAD drawing of the dam basin showing contour lines from which (x.y.z) values can be obtained (Recommended).

If raw survey data is available – Save the XYZ file and **Go to Step 3**

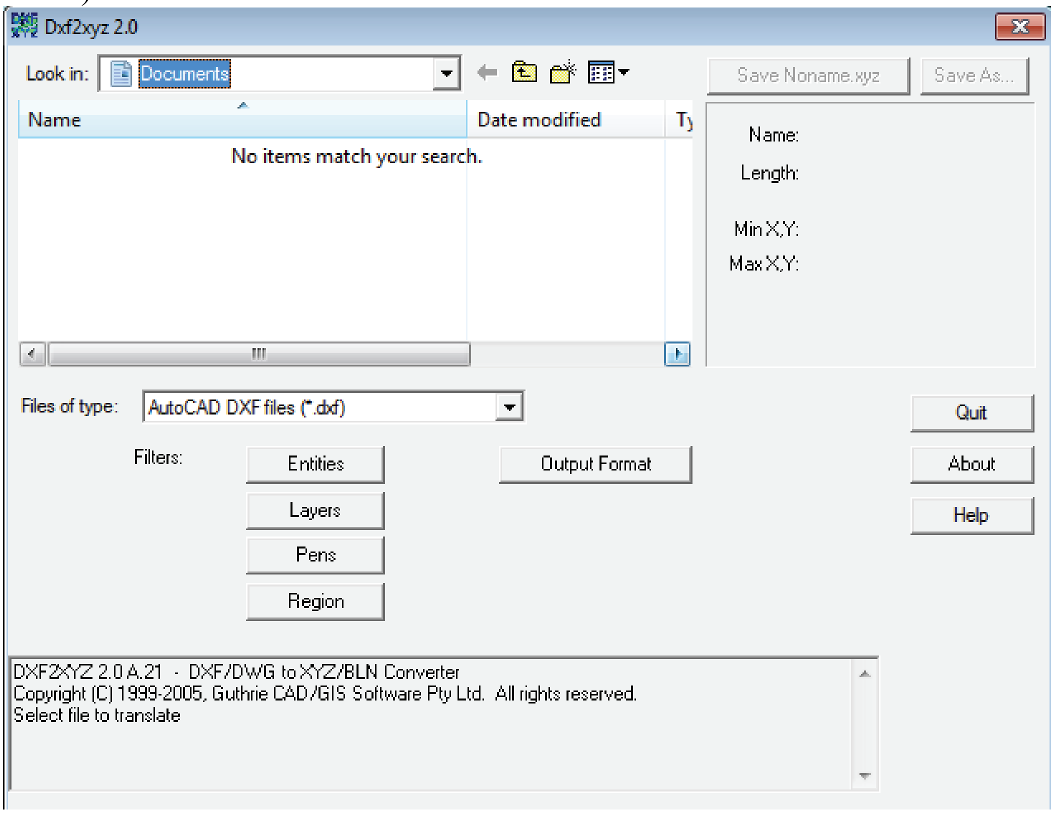
If an image of the contour map of the dam basin is available, the user must follow these instructions:

- Import the image into AutoCAD,
- Draw polylines to obtain x and y coordinates by tracing the contours on the image.
- Extend the polyline from one bank to the other along the dam structure to form a continual line for each level
- Save the traced drawing as *.dxf and **Go to Step 2**

If a topographic contour CAD drawing of the dam basin is available without the topographic drawing of the dam wall, import the dam wall cad drawing into the dam basin drawing to enable the connection of the contour lines from one bank to the other along the dam structure. Save the drawing as *.dxf and **Go to Step 2**

Step 2 – Conversion of Raw Data into XYZ

Use DXF2 Program to generate a text file with XYZ values (see Screen-display 1 below).



Screen-display 1

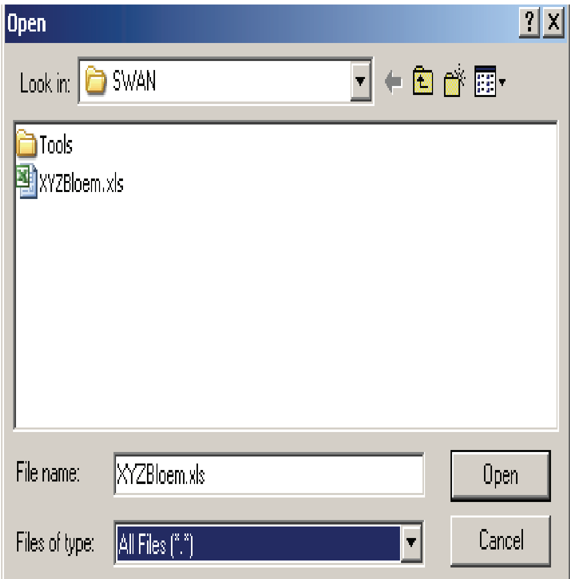
Step 3 – Computational Grid

The computational grid is required in a specific format because of the way the SWAN programme is designed to read the Z value each point.

The computational grid can be created manually but this is not advised because depending on the size of the reservoir one can end up with a large number of points. It is therefore recommended that users utilise an available digitising software program. The program, Surfer 8, could be used for this process.

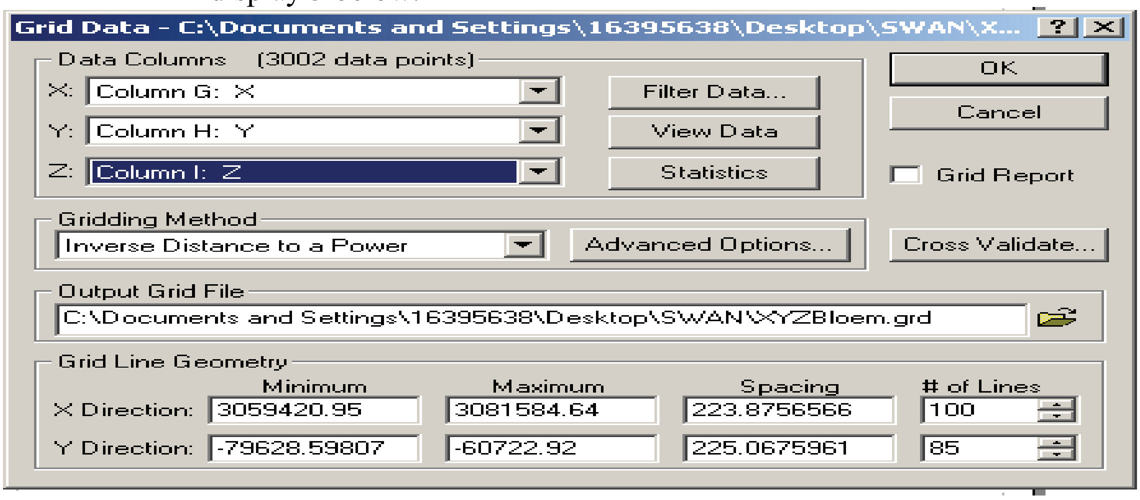
If Surfer is used, these steps must be followed:

1. Open Surfer.
2. Click on **grid**, then **data**. A pop out menu will appear.
3. Change the section on **Files of type** to All Files [*.*] and mark the created xyz-file. See Screen-display 2 below



Screen display 2

4. Press open, a pop out menu will appear. Where it says **Data columns** on the pop out menu, select the location of the file under which the XYZ file from Step 1 or 2 above was saved. Create the desired grid by typing the appropriate numbers in the space provided for **Spacing**. See Screen-display 3 below.



Screen display 3

5. Click on **OK** and the data/map will be created *and saved in the chosen Output Grid File.*
6. *Open the new file with Surfer (File – Open... - newout.dat – press Open) and s*ave the data as type ASCIIXYZ (*.dat), (change also the name of the file so that the original will not be lost!)

   Note

   The user must ensure that the grid line spacing in both X and Y direction are whole numbers. This can be achieved by adjusting the **Minimum** *and* **Maximum** *values slightly. The X and Y spacing does not need to be equal but this is preferable.*

   Some data may be lost if the data points are more than 65000 because Microsoft Excel has provision for maximum of 65,000 rows. Transfer the *data in batches or reduce the number of nodes in the grid by increasing the spacing between the grid lines.*

7. Create in a separate Excel-sheet another column by subtracting the Z values from the dam Full Supply Level (FSL) or a selected flood level. In this way the chosen level, e.g. FSL will be zero, negative figures will represent land and positive value water depth. Open Surfer – File – Open... – choose the ASCIIXYZ (*.dat) – press Open. Copy the Z-column to a new Excel-sheet and subtract the FSL or the selected flood level.
8. Copy the created column (FSL-Z) from the Excel spreadsheet and paste it on a separate Surfer-worksheet .
9. Save the new worksheet as a ASCIIXYZ (*.txt) file
10. Rename the ASCIIXYZ (*.txt) to a (*.dat) file. Go **to Step 4** –SWAN Code.

<u>Step 4 – SWAN Code</u>

The programming code of SWAN can be found in Appendix C of the SWAN user manual. This code has extensive functionality, but this guideline will only focus on applications relevant to inland reservoirs. Certain aspects of the SWAN code will now be extracted from Appendix C and discussed.

A SWAN file is an ordinary text file saved with a .swn extension, use the Notepad for writing the SWAN code.

The essential and most applicable commends are described below. **The underlined portion of the command is all that is needed for SWAN to identify the command**. For example, when referring to PROJECT only PROJ is used in the code.

Command **<u>PROJ</u>**

<u>PROJ</u>ECT 'name' 'nr'
'title 1'
'title 2'
'title 3'

PROJ - Each .swn file is given a **PROJECT** name and this command
'name' - The user is required to specify the project **name**
'nr' - is to distinguish this **run** amongst other runs for the same project
'title' - can be used for addition information to appear on the output file if necessary.

Command **<u>COORD</u>**

<u>COORD</u>INATES <u>CART</u>ESIAN

COORD – The user is required to choose the coordinate system
CART – Tells SWAN that the user has chosen the Cartesian coordinate system

Model Description

Command **CGRID**

With this required command the user defines the geographic location, size, resolution and orientation of the computational grid. Regular grids are most often used. The parameters to be specified are illustrated in the Figure 2 below.

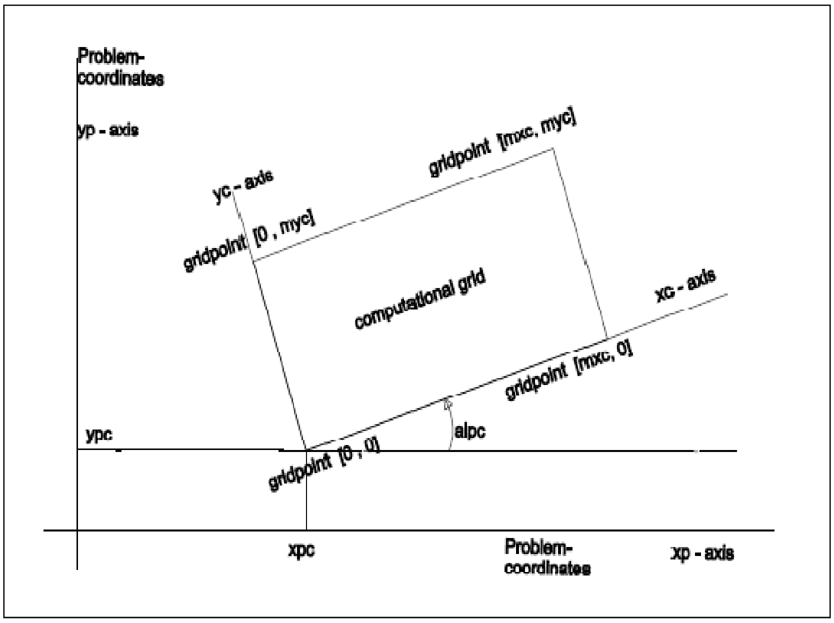


Figure 2: Parameters to be specified to define the computational grad in relation to the coordinates of the case investigated (i.e. the Problem coordinates).

CGRID REG [xpc] [ypc] [alpc] [xlenc] [ylenc] [mxc] [myc]

CIR [mdc] [flow] [fhigh] [msc]

CGRID -The user is required to define the Computational Grid
REG -Indicates that the computational grid is to be taken as uniform and rectangular
[xpc] - The x-coordinate (in, m) of the origin of the computational grid
[ypc] - The y-coordinate (in, m) of the origin of the computational grid ([xpc], [ypc] can be found in the grid report of the first grid (xyz), open the file and click on Options and then Grid info…)
[alpc] - Is the direction (in, degrees) of the positive x- axis
[xlenc] - Length (in, m) of the computational grid in the x-direction
[ylenc] - Length (in, m) of the computational grid in the y-direction
[mxc] - number of mesh spacings in the x-direction for a uniform recti-linear grid
[myc] - number of mesh spacings in the y-direction for a uniform recti-linear grid
(The number for mxc & myc is one less than the number of grid points)

Command **INP**

This command specifies certain parameters of the input grid.

INPGRID BOTTOM REG [xpinp] [ypinp] [alpinp] [mxinp] [myinp] [dxip] [dyinp]

INPGRID - Defines the geographical location, size and orientation of the input grid
BOTTOM - Defines the input grid of the bottom level (i.e. the dam basin bottom bathymetry).
REG - Indicates that the computational grid is to be taken as uniform and rectangular
[xpinp] - The x-coordinate (in, m) of the origin of the input grid
[ypinp] - The y-coordinate (in, m) of the origin of the input grid
[alpinp] - Is the direction (in, degrees) of the positive x- axis
[mxinp] - number of meshes in the x-direction of the input grid
[myinp] - number of meshes in the y-direction of the input grid
[dxinp] - mesh line spacing in the x-direction of the input grid
[dyinp] **-** mesh line spacing in the y-direction of the input grid

Command **READ**

With this command the program is informed how to read the input bathymetry grid

---

READINP BOTTOM [fac] 'fname' [idla] [nhedf] FREE

READINP - Controls the reading of values of the indicated variables from file
BOTTOM - Indicates that the bottom levels (m) are to be read from file
[fac] - Is the factor used to multiply values read from the file
'fname' - The name of the input file in which the bathymetry (z values) are kept
[idla] - Defines the order in which the bathymetry values are read (see page 55 of the SWAN user manual for various orders)
[nhedf] - Is the number of header lines in the input file to be ignored during simulation, Default: [nhedf] =0
FREE - With this the user indicates that the values are to be read in free format

Command **WIND**

Another input parameter that is required to be specified is the dominant wind velocity and direction.

WIND [vel] [dir]

WIND - With this command the user indicates that wind is constant.
[vel] - Wind velocity in (m/s) .
[dir] - Wind direction in degrees.

Command **SETUP**

With this command (optional) the wave induced setup is computed and accounted for in the wave computations.

SETUP [supcor]

SETUP - If given, SWAN will include wave induced setup.
[supcor] – A user defined setup constant. The user can modify the setup constant at one point in the computational grid. If the user does not require defining this constant, the default, which is zero, will be used.

Command **NUM**

This command deals with iterative process of SWAN and termination of the computation process.

NUM ACCUR [drel] [dhoval] [dtoval] [npnts]

NUM – Used to influence numerical properties of SWAN
ACCUR – Specifies the criterion for terminating the iterative procedure during computations
[drel] – Sets allowable difference between wave heights & period from two consecutive iterations
[dhoval] – Sets the fraction difference between the average significant wave heights over all wet grid points
[dtoval] – Sets the fraction difference between the average mean wave period over all wet grid points
[npnts] – The user is not required to specify this parameter, SWAN uses a default value.

Output Requirements

Command **GROUP**

GROUP 'sname' [ix1] [ix2] [iy1] [iy2]

GROUP - Defines a set of output location on a regular grid
'sname' - Name of the frame defined by the GROUP command
[ix1] - Origin of the index in terms of the computation grid in x-direction
[ix2] - The highest index in term the computational grid in the x-direction
[iy1] - Origin of the index in terms of the computation grid in y-direction
[iy2] - The highest index in term the computational grid in the y-direction

Command **TABLE**

TABLE 'sname' HEADER 'fname' [XP] [YP] [DEP] [HS] [PDIR] [RTP]

TABLE - Indicated to SWAN that the output should be in table format
'sname' - Refers to the group data
HEADER – Specified the heading of the table in the output file
'fname' – Name of the data file where the output is to be written.
XP – User instructs SWAN to write the x-coordinate of the output location
YP – User instructs SWAN to write the y-coordinate of the output location
HS – Significant wave height
PDIR – Peak direction
RTP – Peak period in (s)

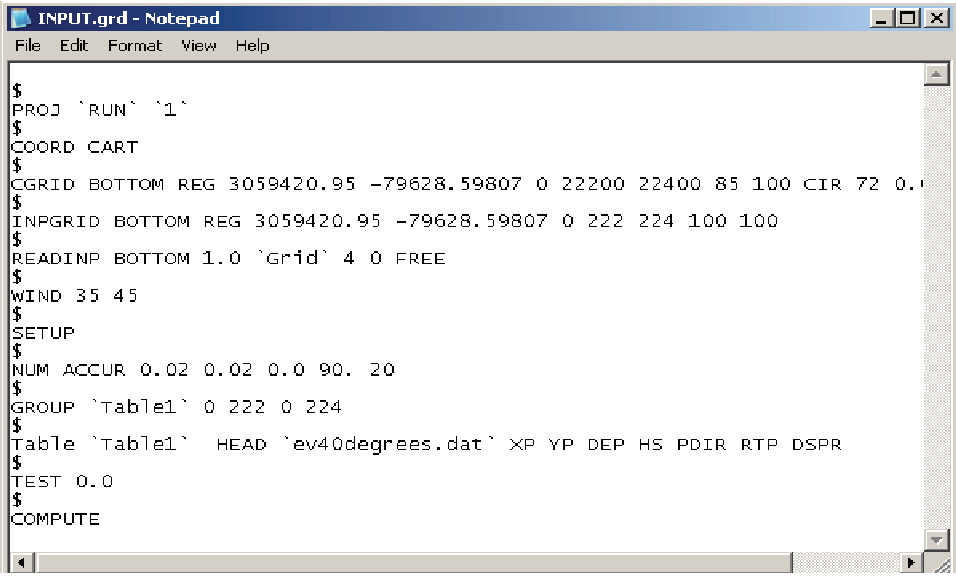
Command **COMPUTE**

Orders SWAN to start computing

Command **STOP**

Indicates to SWAN to ignore any other command after this one.

Save the text file (SWAN Code) with an extension (*.swn) by typing the file name and extension in the space provided for **File name** (refer Screen-display 4).



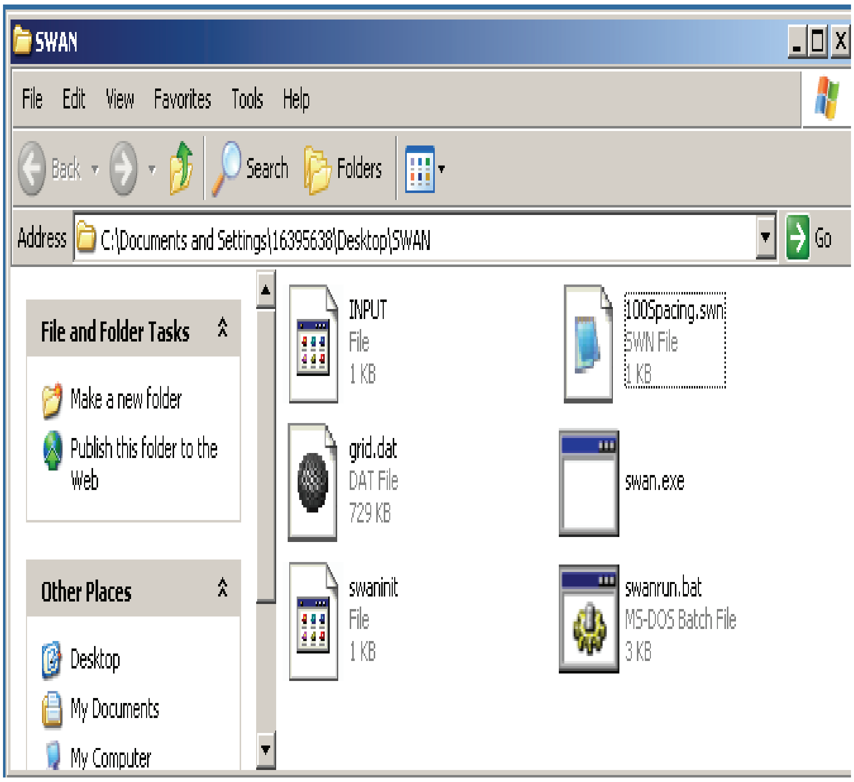
INPUT.grd - Notepad

File Edit Format View Help

```
$
PROJ `RUN` `1`
$
COORD CART
$
CGRID BOTTOM REG 3059420.95 -79628.59807 0 22200 22400 85 100 CIR 72 0.
$
INPGRID BOTTOM REG 3059420.95 -79628.59807 0 222 224 100 100
$
READINP BOTTOM 1.0 `Grid` 4 0 FREE
$
WIND 35 45
$
SETUP
$
NUM ACCUR 0.02 0.02 0.0 90. 20
$
GROUP `Table1` 0 222 0 224
$
Table `Table1`  HEAD `ev40degrees.dat` XP YP DEP HS PDIR RTP DSPR
$
TEST 0.0
$
COMPUTE
```

Screen-display 4: Input data as described under Step 4 above

1. The above (swan-file) must be saved two times (in the example in Screen-display: 1. INPUT, 2. 100Spacing.swn) in the folder which contains the following files as illustrated below (refer Screen-display 5 below):
    a. Swan.exe
    b. Swaninit
    c. Swanrun.bat
    d. And the grid file (with z-values) which was created under **Step 3** (23) above



Screen-display 5

2. SWAN runs in dos mode. To access the dos mode, click **Start menu** then **Run**. **Type** “cmd” on the pop-up menu and press **OK**.

3. To access the folder under which the relevant swan files are saved, **Type** “cd” followed each time by the folder name as per the file path.
4. **Type** “dir” followed by enter to confirm that you are in the correct folder (see Screen-display 6 below).

```
C:\WINDOWS\system32\cmd.exe
Microsoft Windows XP [Version 5.1.2600]
(C) Copyright 1985-2001 Microsoft Corp.

C:\>cd Documents and Settings\16395638\Desktop\SWAN

C:\Documents and Settings\16395638\Desktop\SWAN>dir
 Volume in drive C is 80GB C-drv
 Volume Serial Number is 385F-2952

 Directory of C:\Documents and Settings\16395638\Desktop\SWAN

2010/05/19  04:03 PM    <DIR>          .
2010/05/19  04:03 PM    <DIR>          ..
2010/05/19  03:13 PM               464 100Spacing.swn
2010/05/19  10:01 AM           746,018 grid.dat
2010/05/19  10:01 AM               446 INPUT
2006/08/11  02:53 PM         1,511,764 swan.exe
2010/04/14  09:29 AM               950 swaninit
2006/08/11  11:00 AM             2,267 swanrun.bat
               6 File(s)      2,261,909 bytes
               2 Dir(s)  13,274,845,184 bytes free

C:\Documents and Settings\16395638\Desktop\SWAN>swanrun 100spacing
```

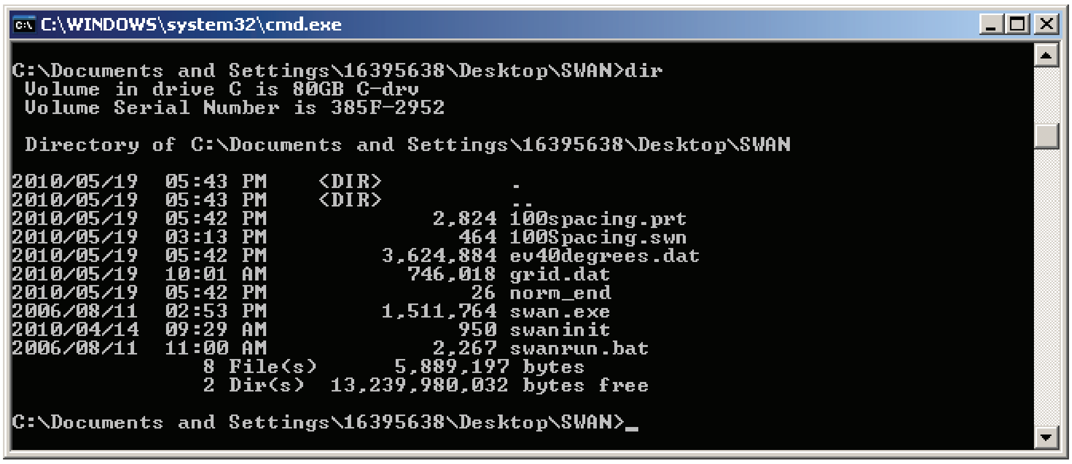
Screen-display 6

5. To run swan, **Type** “swanrun” followed by the filename of the file with the extension (*.swn) and then **Press** Enter.
Swan should commence the computation as shown in Screen-display 7 below. If nothing happens, SWAN will create an error file in the folder from which SWAN is running. Read the error to establish where the error may have occurred. The common error is specifying the wrong mesh.

```
C:\WINDOWS\system32\cmd.exe
C:\Documents and Settings\16395638\Desktop\SWAN>swanrun 100spacing
 SWAN is preparing computation

 iteration     1; sweep 1
+iteration     1; sweep 2
+iteration     1; sweep 3
+iteration     1; sweep 4
 accuracy OK in    0.46 % of wet grid points ( 90.00 % required)

 iteration     2; sweep 1
+iteration     2; sweep 2
+iteration     2; sweep 3
+iteration     2; sweep 4
 accuracy OK in   54.99 % of wet grid points ( 90.00 % required)

 iteration     3; sweep 1
+iteration     3; sweep 2
+iteration     3; sweep 3
+iteration     3; sweep 4
 accuracy OK in   91.67 % of wet grid points ( 90.00 % required)

+SWAN is processing output request     1
  Normal end of run 1

C:\Documents and Settings\16395638\Desktop\SWAN>
```

Screen-display 7

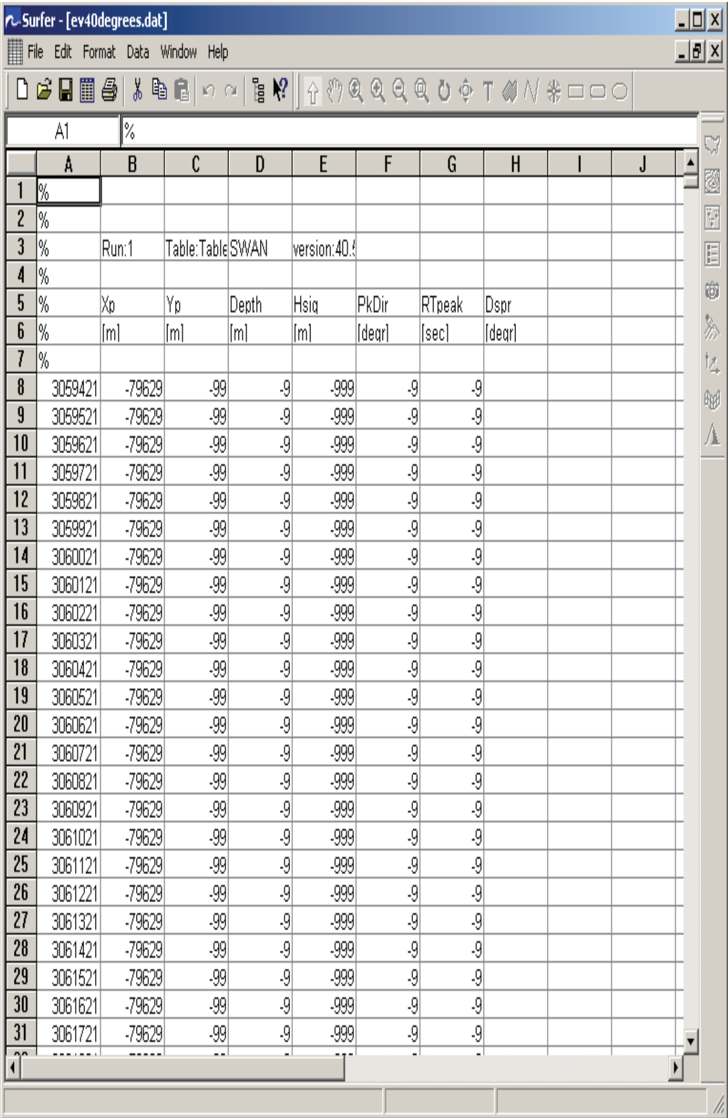
When SWAN has completed the iteration process it will save the output file in the folder from which SWAN is running.



```
C:\WINDOWS\system32\cmd.exe

C:\Documents and Settings\16395638\Desktop\SWAN>dir
 Volume in drive C is 80GB C-drv
 Volume Serial Number is 385F-2952

 Directory of C:\Documents and Settings\16395638\Desktop\SWAN

2010/05/19  05:43 PM    <DIR>          .
2010/05/19  05:43 PM    <DIR>          ..
2010/05/19  05:42 PM             2,824 100spacing.prt
2010/05/19  03:13 PM               464 100Spacing.swn
2010/05/19  05:42 PM         3,624,884 ev40degrees.dat
2010/05/19  10:01 AM           746,018 grid.dat
2010/05/19  05:42 PM                26 norm_end
2006/08/11  02:53 PM         1,511,764 swan.exe
2010/04/14  09:29 AM               950 swaninit
2006/08/11  11:00 AM             2,267 swanrun.bat
               8 File(s)      5,889,197 bytes
               2 Dir(s)  13,239,980,032 bytes free

C:\Documents and Settings\16395638\Desktop\SWAN>_
```

Screen-display 8

6. Open the output file using Surfer. A typical output file is as shown in Screen-display 9 below depending on the parameter specified by the user in the swan code.



	A	B	C	D	E	F	G	H	I	J
1	%									
2	%									
3	%	Run:1	Table:Table	SWAN	version:40.					
4	%									
5	%	Xp	Yp	Depth	Hsig	PkDir	RTpeak	Dspr		
6	%	[m]	[m]	[m]	[m]	[degr]	[sec]	[degr]		
7	%									
8	3059421	-79629	-99	-9	-999	-9	-9			
9	3059521	-79629	-99	-9	-999	-9	-9			
10	3059621	-79629	-99	-9	-999	-9	-9			
11	3059721	-79629	-99	-9	-999	-9	-9			
12	3059821	-79629	-99	-9	-999	-9	-9			
13	3059921	-79629	-99	-9	-999	-9	-9			
14	3060021	-79629	-99	-9	-999	-9	-9			
15	3060121	-79629	-99	-9	-999	-9	-9			
16	3060221	-79629	-99	-9	-999	-9	-9			
17	3060321	-79629	-99	-9	-999	-9	-9			
18	3060421	-79629	-99	-9	-999	-9	-9			
19	3060521	-79629	-99	-9	-999	-9	-9			
20	3060621	-79629	-99	-9	-999	-9	-9			
21	3060721	-79629	-99	-9	-999	-9	-9			
22	3060821	-79629	-99	-9	-999	-9	-9			
23	3060921	-79629	-99	-9	-999	-9	-9			
24	3061021	-79629	-99	-9	-999	-9	-9			
25	3061121	-79629	-99	-9	-999	-9	-9			
26	3061221	-79629	-99	-9	-999	-9	-9			
27	3061321	-79629	-99	-9	-999	-9	-9			
28	3061421	-79629	-99	-9	-999	-9	-9			
29	3061521	-79629	-99	-9	-999	-9	-9			
30	3061621	-79629	-99	-9	-999	-9	-9			
31	3061721	-79629	-99	-9	-999	-9	-9			

Screen-display 9

For graphical representation of the results, the user is encouraged to read surfer manual or to use any available software. A typical plot of wind wave height distribution in a dam basin as obtained from SWAN and plotted by Surfer is presented in Figure 3 below.

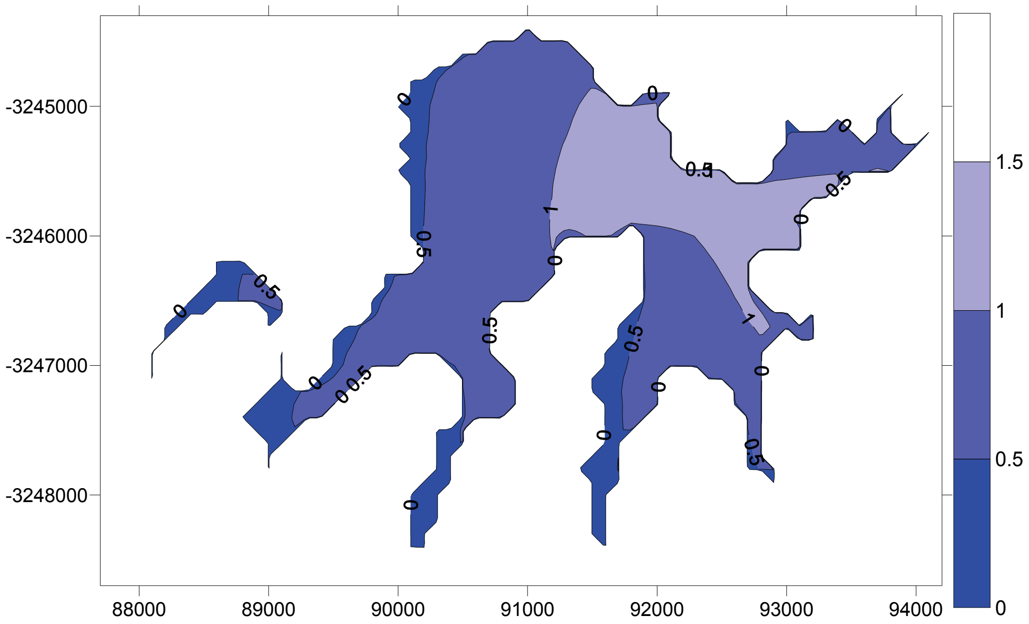


Figure 3: Typical plot of wave height distribution in a dam basin as obtained from SWAN and plotted by Surfer (wave heights, H_s are in metres)

5. REFERENCES

1. SWAN computer program, Cycle version 40.41 User Manual and software downloadable as freeware at: http://vlm089.citg.tudelft.nl/swan/index.htm
2. DXF2XYZ computer program. Freeware downloadable at: http://www.softpedia.com/get/Science-CAD/Dxf2xyz.shtml
3. SURFER computer program. Downloadable at: http://www.goldensoftware.com/products/surfer/surfer.shtml

APPENDIX D

Short user manual of SURFER

D.1 Creating grid files, contour- and wireframe maps

Surfer has very good basic tutorials. Access the tutorials from the help menu. Do Lessons: 1-4. The lessons are easy to follow and will give you a good basic feel for working with Surfer.

When creating a grid file make sure that the grid line spacing in both the X and Y direction are whole numbers. See Figure D.1. This will make the SWAN programming easier. The X and Y line spacing does not need to be equal to each other. Also adjust the axis of given data so that North on the grid is up (the same as SWAN). Doing this will make working with direction easier when making vector maps.

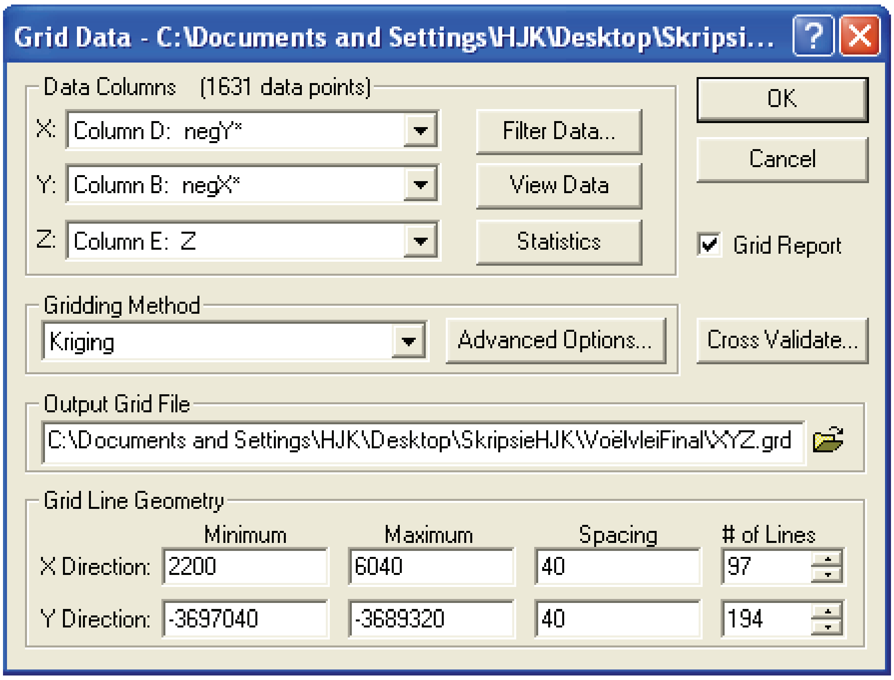


Figure D.1: Grid data in Surfer

D.2 Creating a *.dat file for SWAN

When creating a grid file for SWAN keep in mind that the mean water level should be equal to zero with water depths being positive and land being any negative number. After Surfer created the grid file save the file as an ASCII XYZ (*.dat) file. See Figure D.2. Open the new file from whichever folder you saved it in and delete the X and Y column. You should now have a grid file with only the Z coordinates in the first column.

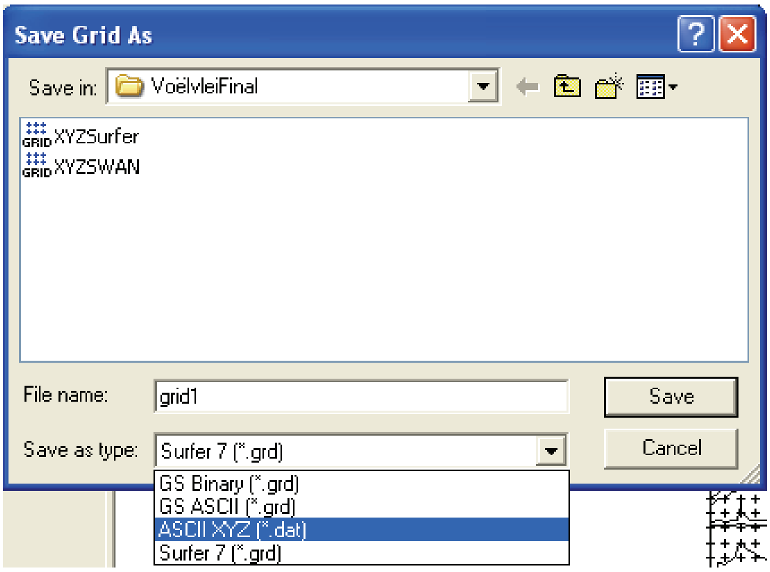


Figure D.2: Saving grid file for SWAN

D.3 Creating vector maps

Do not attempt before you've done the first 4 tutorial lessons. This step-by-step guide will be done on the wave peak directions of the Voëlvlei dam at a 36.6 m/s wind speed in the direction of the longest straight line fetch.

Open the *.dat file that SWAN produced as output. Make sure which columns contain the X-, Y coordinates, significant wave height and the Peak direction. Create 2 grid files: one with the significant wave height as the Z-axis and one with the peak direction as the Z-axis (X=X-axis, Y=Y-axis). Make sure that the two grid files have the same grid line spacing.
Open a new plot document. Go to the grid's math function. See Figure D.3.

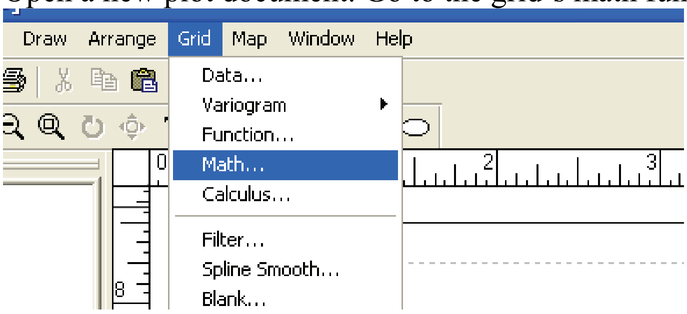


Figure D.3: Grid Math's location

Choose the grid file you created with significant wave height as the Z-axis. Type in the "Enter a function …" textbox the formula: C=max(A,0). This will substitute negative values of H_s with 0. (SWAN gives significant wave height as negative on land). This is done because the vector map will use the H_s grid to draw the length of the vectors (can't have negative length vectors). After clicking ok Surfer will create a grid file named "out.grd" with 0 values in the place of negative values for H_s.

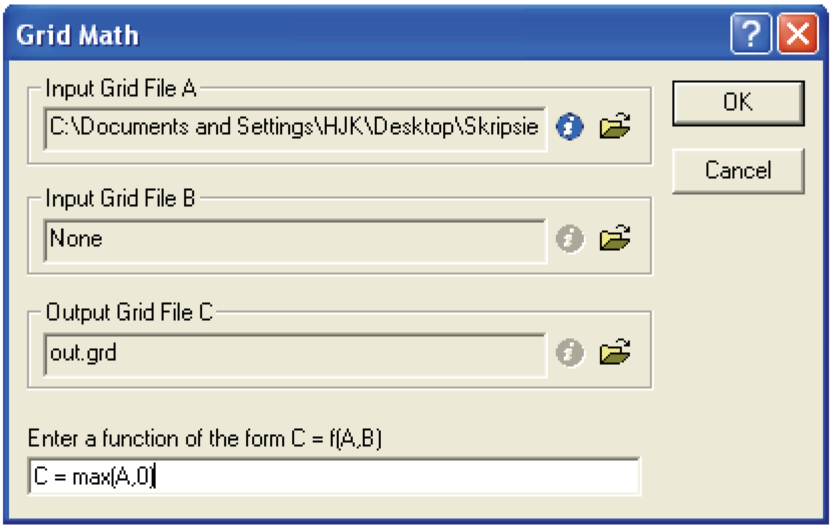


Figure D.4: Grid Math function

Create a new 2-Grid Vector Map. See Figure D.5.

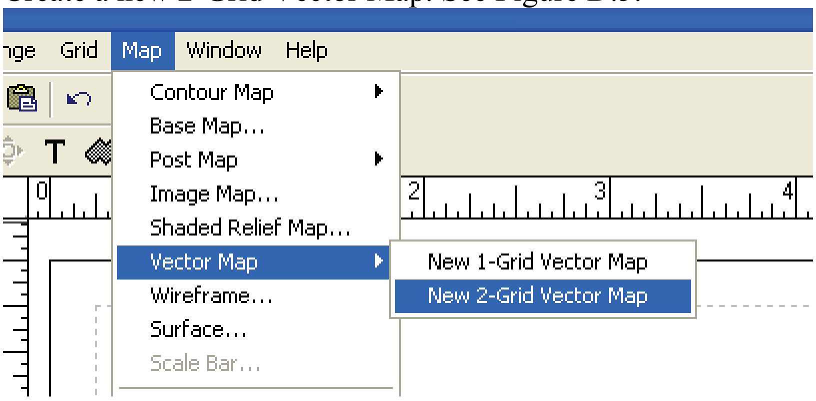


Figure D.5: 2-grid Vector map location

When asked for the X-component (angle) grid select the grid file you created with the peak direction as the Z-axis. See Figure D.6.

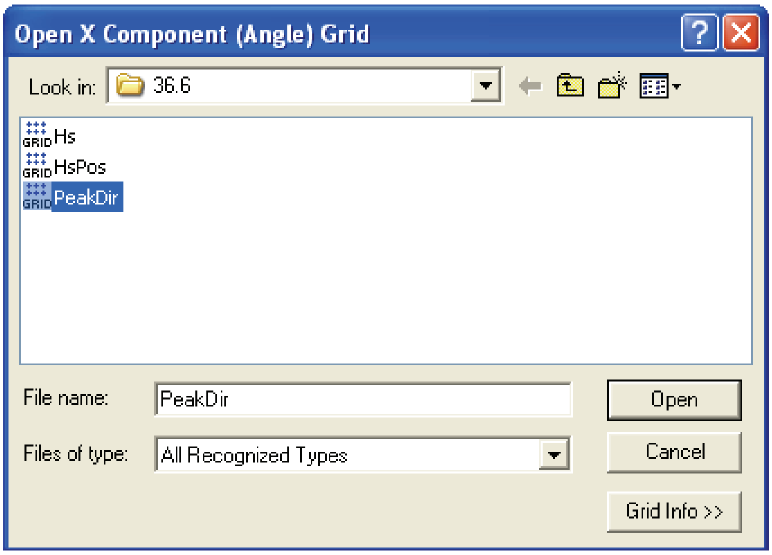


Figure D.6: angle-component

After you clicked ok you will be asked for the Y component (length) grid. Select the grid you created with the grid math function. See Figure D.7.

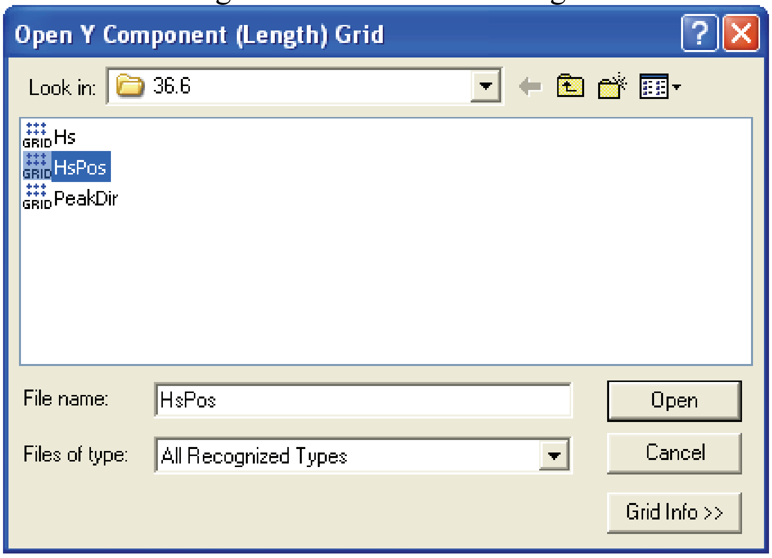


Figure D.7: length-component

After clicking ok Surfer will create a vector map that will look something like Figure D.8. Double click on the vector map to open the Map: Vector properties. Change the Coordinate System to Polar and the Angle to 0 = East. See Figure D.9 and D.10.

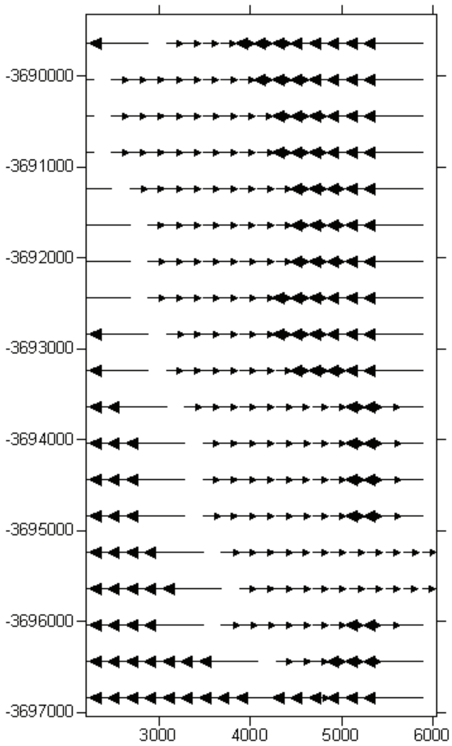


Figure D.8: Vector map 1

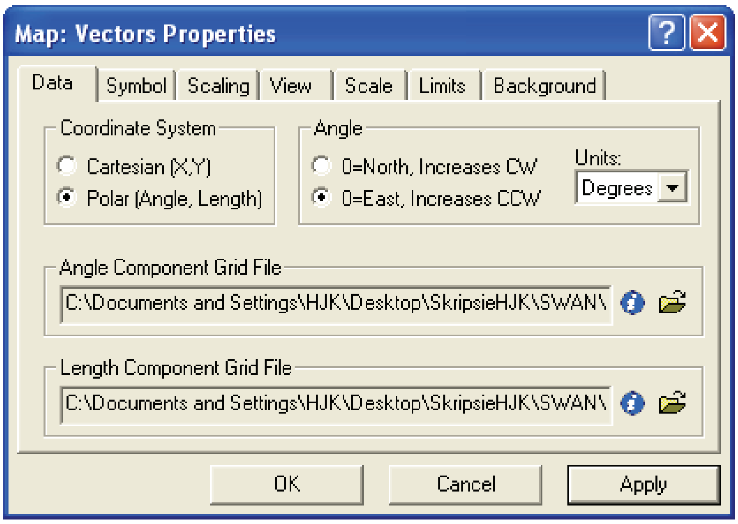


Figure D.9: Vector properties

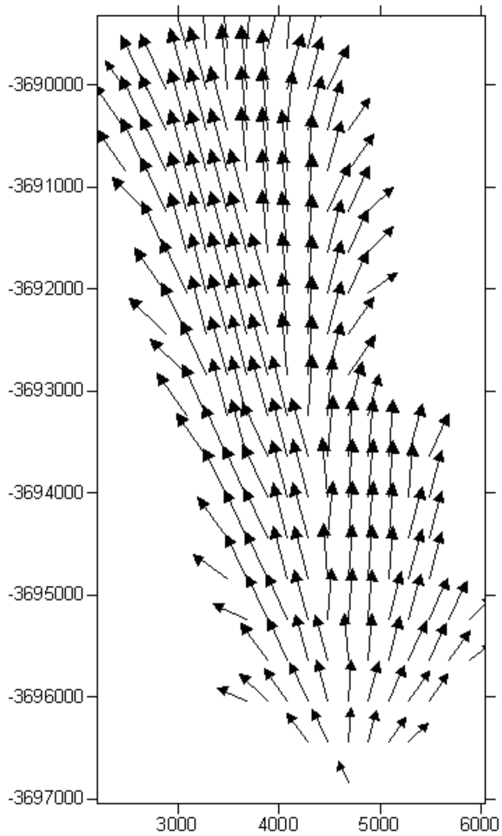


Figure D.10: Vector map after properties was changed

After you created the vector map you can put it over the contour map you created with H_s as the Z-axis. Change the colours and levels till you end up with a map that looks like Figure B.11.

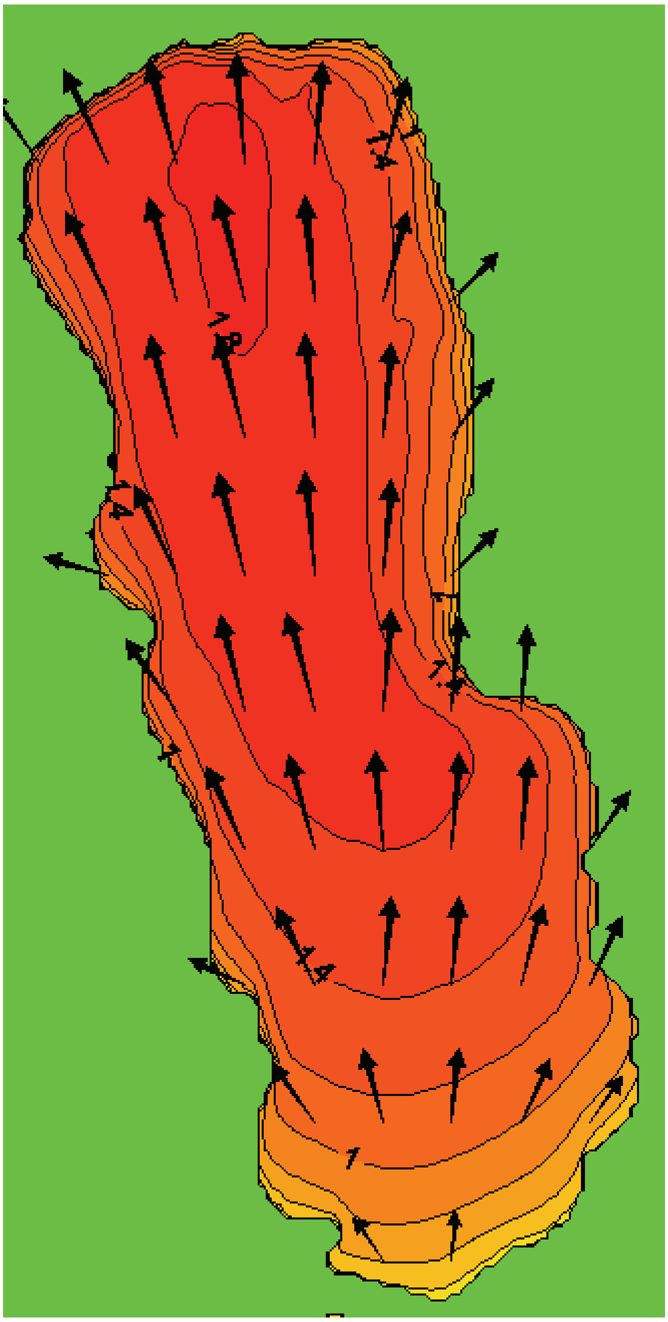


Figure D.11: Vector map over a contour map

APPENDIX E

CD with SWAN case study example files
and
with Excel spreadsheet to calculate wave height and period (based on empirical formulae) for selected wind speed and consequent wave run-up on a sloped (embankment) wave wall.