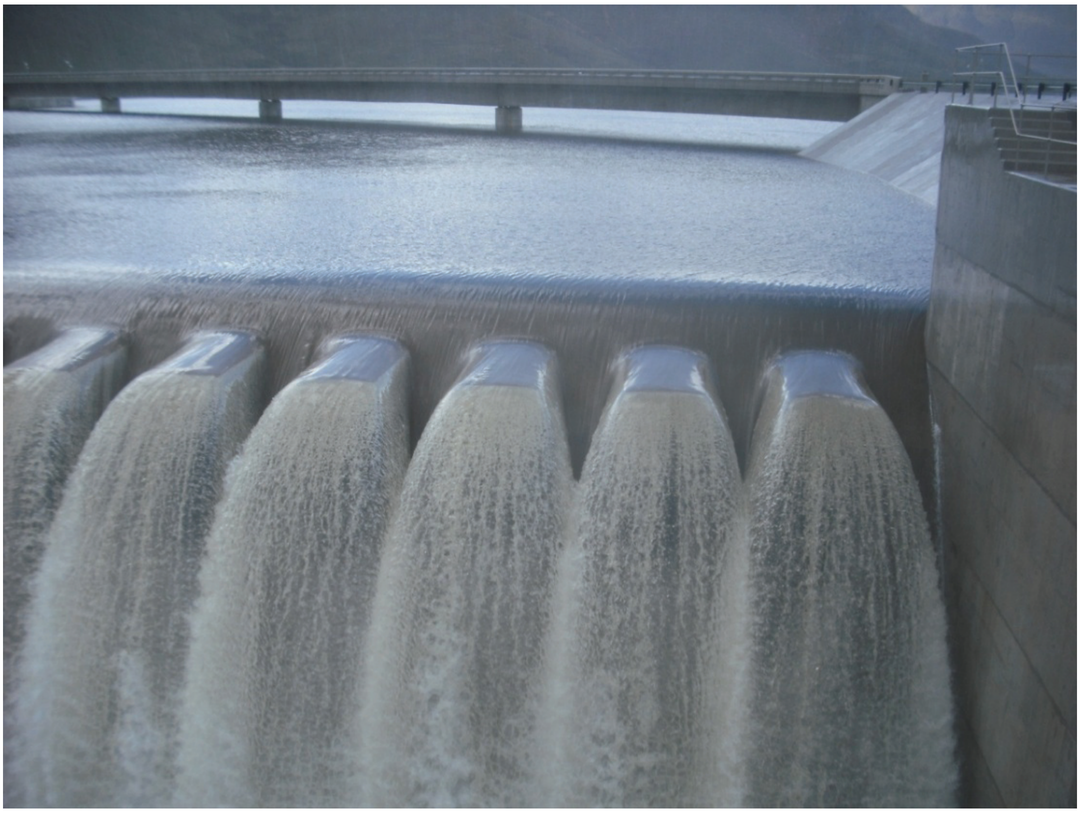
Guidelines on Freeboard for Dams

Volume I
Literature Review and Case Studies



Report to the

Water Research Commission

by

Eddie Bosman & Gerrit Basson (editors)
Institute for Water and Environmental Engineering
Department of Civil Engineering
University of Stellenbosch

WRC Report No. 1759/1/11
ISBN 978-1-4312-0150-1

AUGUST 2011

UNIVERSITEIT·STELLENBOSCH·UNIVERSITY
jou kennisvennoot • your knowledge partner

Acknowledgements

The research in this report emanated from a project funded by the SA Water Research Commission titled: Revision of the Sancold *Interim* Guidelines on Freeboard for Dams (2007-2009)

The Reference Group responsible for this project consisted of the following persons:

Mr Wandile Nomquphu	Water Research Commission (Chairman)
Prof C Oosthuizen	Department of Water Affairs
Mr AG Chemaly	Aurecon
Mr W Tunha	Department of Water Affairs
Dr P Roberts	Consultant
Dr M Shand	Aurecon Group
Prof A Rooseboom	University of Stellenbosch

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

Co-authors of this report are:

Mr J Basson
Mr G Cloete
Mr H Kriel
Prof JV Retief
Mr T Tente
Mr C Visser

This report was edited by Mr Eddie Bosman and Prof Gerrit Basson of the University of Stellenbosch.

Executive Summary

This WRC study on freeboard for dams commenced in 2007 and was carried out over a period of three years.

The deliverables of this research project are two reports:

- Volume I Literature Review and case studies (this report);
- Volume II Guidelines for the determination of freeboard for dams, which has also been reviewed by SANCOLD and is now the new South African guidelines document (2011), replacing the 1990 freeboard guidelines of SANCOLD.

A literature review was carried out to determine the state-of-the-art knowledge on the quantification of the secondary components to be taken into account in the determination of dam freeboard. Research carried out in the coastal engineering field of aspects which are relevant and applicable to dams was incorporated.

The use of mathematical models to predict freeboard components due to wind was evaluated against the analytical (simplified) methods. Software was incorporated and/or referenced where available for more detailed quantification of the relevant freeboard components.

The combination of the different freeboard components for different floods and for different dam types was evaluated and discussed at a stakeholder meeting. The probability of all the components occurring at the same time is low, and the assumptions made in previous guidelines were evaluated critically. Proposed updated guidelines were considered in evaluating the combined risk, with application at specific case study dams.

Guidelines on embankment erosion protection measures were also incorporated in the new guidelines. Only riprap (dumped rock) was considered and the riprap grading and required natural filter grading was evaluated.

The key findings of this study are:

a) The 1990 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 with the addition of cyclone data along the East Coast. Such regionalized maps could be used for planning purposes of category I and II dams, but for any detail design studies local wind data should be analysed.

c) For planning purposes and for the detail design of category I dams wind wave height, run-up and set-up calculation should be based on the Rock Manual. In all other cases the SWAN model should be used to determine wind wave height. 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 $H_{2\%}$.

d) 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 (1D) model could be used, but for category II and III dams 2D or 3D models are recommended.

e) The design of riprap at dams for protection against wind wave erosion should be based on the Rock Manual.

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. A risk analysis procedure using a revised DEFRA methodology to incorporate component scenarios could be used complementary to the deterministic approach and is recommended for detail design studies of category II and III dams.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

The deliverables of this research project on freeboard for dams are two reports:
- Volume I Literature Review and case studies (this report)
- Volume II Guidelines for the determination of freeboard for dams

The Volume II guidelines have been approved by SANCOLD (2011) and replace the previous Freeboard guidelines (SANCOLD, 1990).

This study was performed with due cognisance of the recent study performed under the auspices the Water Research Commission (WRC), i.e. "Review of the Selection of Acceptable Flood Capacity for Dams in South Africa in the context of Dam Safety" (WRC Report No 1420, 2007).

WRC Report No 1420 presents a comprehensive background to the status quo of dam safety in South Africa, and should be read in conjunction with this report on the current study. Report No 1420 also puts forward a Framework for Best Practice for Dam Safety Assessment which was well received at a workshop of Dam Safety Professionals held in October 2004. The framework comprises six hierarchical levels as follows:

- Level 1: Operating environment
- Level 2: Guiding principles
- Level 3: Regulatory system
- Level 4: Approaches
- Level 5: Procedures
- Level 6: Methods

The current study deals with Level 6, and focuses on updating the methods to be applied by dam safety professionals for determining the relevant **freeboard** for dams associated with the occurrence of the appropriate dam water level/design flood.

WRC Report No 1420 presents inter alia recommendations on issues including:
- Updating of the existing SANCOLD guidelines on dam safety by a representative technical committee.

The existing guidelines are:
- Report 1: Interim Guidelines on Safety in Relation to Floods (SANCOLD, 1986)
- Report 2: Interim Guidelines on Dam Break Floods (SANCOLD, 1990)
- Report 3: Interim Guidelines on Freeboard for Dams (SANCOLD, 1990)
- Report 4: Guidelines on Safety in Relation to Floods (SANCOLD, 1991)]
- Revised terminology related to dam safety evaluation floods
- Probabilistic flood determination methodologies in view of available longer data bases and recently developed probabilistic methodologies
- Risk Assessment and Analysis to keep up with international trends in this field

It should be noted however that WRC Report No 1420 has not been adopted by SANCOLD as official document.

The review of aspects of the *Interim Guidelines on Freeboard for Dams* which are recommended in SANCOLD Report 3, is the main aim of this project and is consistent with recommendations of WRC Report No 1420.

The definition of Freeboard presented in the *Interim Guidelines on Freeboard for Dams* (SANCOLD, 1990) is as follows:

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

Freeboard provides a margin of safety against overtopping failure of dams. Sufficient freeboard must be provided so that possible spillage over the non-overspill crest of a dam will not endanger the structure nor create hazards to human lives. Freeboard is normally divided into two components namely the flood surcharge rise above the full supply level, the primary component, and a secondary component allowing for wind, wave and surge effects. The focus of this research will be on the secondary component. The most important components for total freeboard calculation, which may not all occur simultaneously, are the following:

a) Flood surcharge
b) Set-up due to wind
c) Wind generated waves (with effects such as wave reflection and wave run-up additional to the wind set-up water level)
d) Seiches (resonance effects)
e) Earthquake-induced surges
f) Landslide-induced surges
g) Flood-induced surges
h) Gate adjustment surges

The lack of sufficient freeboard at dams is one of the main dam safety concerns at many small to large dams since there is a direct relationship between the rise in water level above spillway crest level during floods and available freeboard, i.e. the greater the freeboard the larger the available head on the spillway. The existing *Interim Guidelines on Freeboard for Dams (1990)* by SANCOLD are currently used for the dam safety assessment of existing dams, the design of new dams, and the dam safety rehabilitation of existing dams in South Africa, however a number of aspects of the document have become outdated:

a) Wind data is based on pre-1990 data.
b) Mathematical models are available now to determine wind setup, wave heights, etc. instead of the simplified analytical methods used in the existing guidelines.
c) The combination of different freeboard components with flood frequencies and risk should be reconsidered.
d) The existing guidelines do not provide design guidelines on erosion protection of embankment dams.

Dams are needed for the semi-arid climatic conditions in RSA to provide over-year storage. These dams must be designed to minimise the risk of dam failure so as to minimise the risk of possible loss of life and economic loss. WRC Report No 1420 indicates that the occurrence of dam failures in South Africa due to floods and insufficient spillway capacity was about 21% prior to 1987 and 39% between 1987 and 2002. The WRC report also quoted The International Commission of Large Dams (ICOLD) report on statistics of dam failures internationally up until 1986 (ICOLD 1995):

- For earth and rockfill dams the most common cause of failure was overtopping (31% as primary cause and 18% as secondary cause), followed by internal erosion of the body of the dam (15% as primary and 13% as secondary) and in the foundation (12% as primary and 5% as secondary).
- Overtopping was also the most common cause of failure of masonry dams (43%), followed by internal erosion of the foundation (29%).
- For concrete dams, however, internal erosion and insufficient shear strength in the foundation each accounted for 21%.

- From a global survey on dam safety by ICOLD (2003) the following conclusions reached, are also relevant here:
- The failure of embankment dams accounted for 80% and the remaining 20% were masonry and concrete dams.
- The most frequent cause of failure was overtopping (36%); 87% of these failures were caused by overtopping of embankment dams.

The most expensive structures that are constructed in the world are dams and therefore it is extremely important that they are well designed, designed to acceptable standards of safety, and well constructed.

This report attempts to document the present knowledge pool and methodologies for freeboard determination, as Volume I of the study. Volume II of the study concludes the investigation and provides updated methodologies in a revised SANCOLD *Guidelines on Freeboard for Dams (2011)* for the establishment of the secondary components for dam freeboard determination. Volume II of this study also addresses the combination of components to be taken into account in determining dam freeboard.

The following are addressed in this report (Volume I of this investigation):
- The overall objectives of this WRC research project are defined in Chapter 2.
- The methodology of the investigation is presented in Chapter 3.
- The literature review on current applied methods to quantify the components contributing to freeboard, excluding surcharge, is addressed in Chapter 4.
- Analytical methods to determine wind wave heights are described in Chapter 5.
- In Chapter 6 mathematical modelling using SWAN to determine wind wave heights is discussed.
- Unsteady flow patterns in reservoirs and methods to simulate the phenomena are discussed in Chapter 7.
- The combination of different freeboard components with the use of deterministic and risk analysis methods is discussed in Chapter 8.
- The literature review on current methods for the design of embankment dam upstream slope protection (limited in this review to rock armour protection of the upstream face of embankment dam walls) is discussed in Chapter 9.
- The report is concluded in Chapter 10.

2. AIMS OF THE PROJECT

The main aims of this investigation include:

- Review of the analytical methods used for the derivation of the different secondary freeboard components.
- Incorporate numerical models where possible.
- Evaluate the use of SA wind data and statistics to determine wind generated waves.
- Incorporate updated SA seismic statistics.
- Review of the combination of different freeboard components with flood frequency, for different dam types.
- Specify design guidelines for riprap rock protection of the upstream slopes of embankment dams.

Guidelines evolving from this investigation could contribute towards the updating of the existing SANCOLD "*Interim Guidelines on Freeboard for Dams" (1990)* for the establishment of the secondary components for dam freeboard determination.

3. METHODOLOGY

g) A literature review was carried out to determine the state-of-the-art knowledge on the quantification of the secondary components to be taken into account in the determination of dam freeboard. Research carried out in the coastal engineering field of aspects which are relevant and applicable to dams was incorporated.
h) The use of mathematical models to predict freeboard components due to wind was evaluated against the analytical (simplified) methods. Software was incorporated and/or referenced where available for more detailed quantification of the relevant freeboard components.
i) The combination of the different freeboard components for different floods and for different dam types was evaluated and discussed at a stakeholder meeting. The probability of all the components occurring at the same time is low, and the assumptions made in previous guidelines were evaluated critically. Proposed updated guidelines were considered in evaluating the combined risk, with application at specific case study dams.
d) Guidelines on embankment erosion protection measures were also incorporated in the new guidelines. Only riprap (dumped rock) was considered and the riprap grading and required natural filter grading was evaluated.

4. LITERATURE REVIEW

4.1 BACKGROUND PERSPECTIVE ON FREEBOARD IN DAM SAFETY EVALUATION OF DAMS

WRC Report No 1420 (2007) recommended updating of the SANCOLD dam safety guidelines specifically with respect to dam safety risk assessment in order to keep up with international trends, as there is extensive published literature on related subject areas, as well as ongoing research in related areas in countries including the UK, USA, Canada and Australia. ICOLD also recently (2003) promoted the further development of dam safety risk assessment procedures “because it provides a logical and comprehensive basis for assessment of dam safety” (ICOLD, 2003).

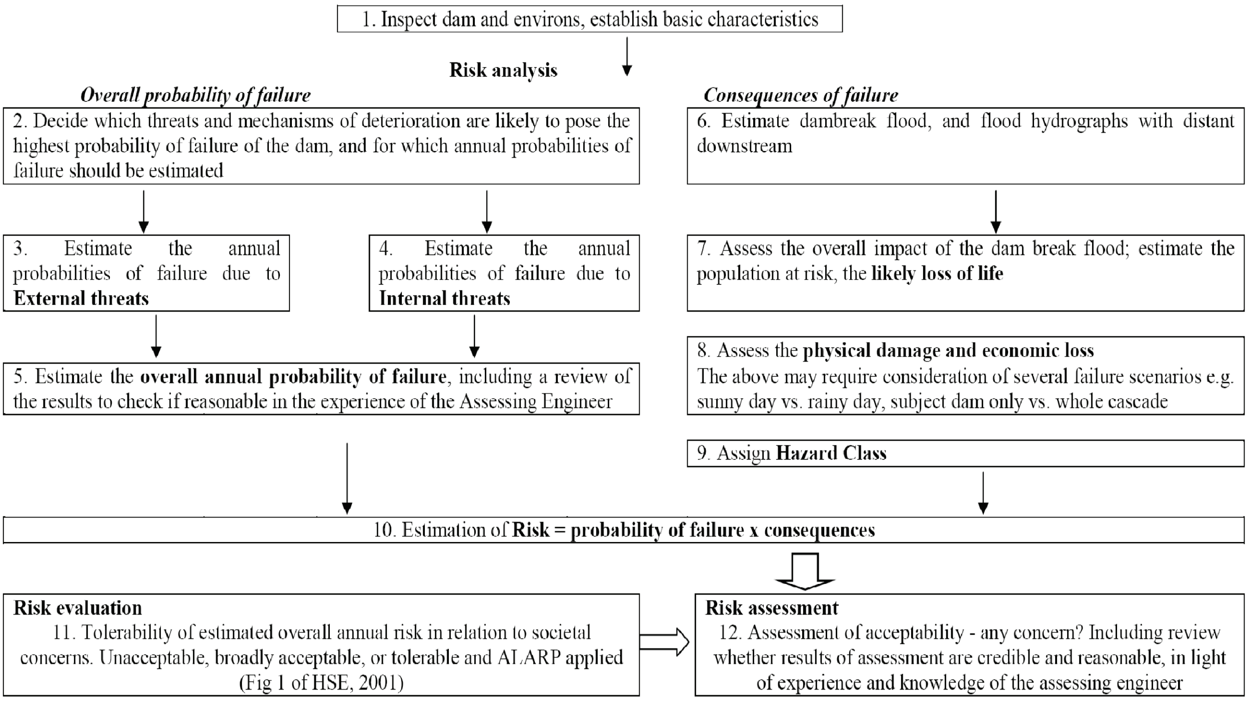
In view of this, it is considered appropriate to refer to the work done by the United Kingdom Department of Food and the Environment (DEFRA, 2002) in this area with the objective of providing background perspective on dam safety risk assessment and identifying where the subject of dam freeboard/overtopping plays a role.

The main aim of the DEFRA (2002) investigation as presented in their report, “Reservoir Safety – Floods and reservoir safety integration”, was to:

“Propose and demonstrate an Integrated System which provides a framework for decision making by Panel Engineers on the annual probabilities of occurrence, consequences and tolerability of all the various threats to reservoir safety”.

The proposed DEFRA integrated system is illustrated diagrammatically in Table 4.1.1 below.

Table 4.1.1 Process of application of integrated system. (The process comprises the activities in the numbered sequence). (DEFRA, 2002)



The “external” and “internal” threads as considered in the DEFRA 2002 report are presented in Table 4.1.2 below.

Table 4.1.2 Matrix showing relationship between threats and failure modes (DEFRA, 2002)

Threats				**Modes of failure of the dam (causing uncontrolled sudden large release of water)**						*Annual probability of individual threat causing failure of dam = sum failure by mode M, given threat T*
Type	Individual	Sym. for threat T	*Annual probability of individual threat occurring*	External erosion: due to reservoir water flowing over dam body (overtopping)	External erosion: other	Internal erosion: 'through fill'	Internal erosion: 'along interface with structure'	Sliding	Appurtenant works incl. erosion across interface of dam and appurtenant works	
			Symbol for mode M	OT	EE	EF	ES	SL	EM	
External	Extreme rainfall/ Flood	FL	*APT_{FL}*	inflows exceed spillway capacity	local runoff down face/ abutments	increased head		high load/ increased pwp	high load	$APF_{FL}^{ALL} = \text{Sum } APF_{FL}^{OT} + APF_{FL}^{EE} + APF_{FL}^{EF} + APF_{FL}^{ES} + APF_{FL}^{SL} + APF_{FL}^{EM}$
	Failure of reservoir in cascade upstream	CA		as for extreme flood						*etc*
	Wind	W	*etc*	na	wave attack on upstream face of dam embankment	na		wave erosion steepening upstream slope	na	
	Snow/ ice	SN		blockage of spillway	na	na		na	various	
	Earthquake	EQ		seiches	na	disrupt filters	disrupt contact stresses	liquefaction; horizontal load	various	
	Terrorism/ sabotage/ accident	TR		blockage of spillway	na	na		blow up dam	various	
	Human error	HE		blockage of spillway	na	na		na	various	
	Aircraft strike	AS		physical disruption of crest	na	disrupt filters		impact load	unlikely	
Internal	Internal stability (embankment)	IE		settlement	na	various		various	na	
	Internal stability (structural)	IS		inability to open spillway gates/ outlets	na	na	broken pipe	na	various	
			Probability of incident	PI_{ALL}^{OT}	etc					
			Probability of failure	PF_{ALL}^{OT}	PF_{ALL}^{FF}	etc				*PF_{ALL}^{ALL}*

Notes

1 A linkage is only shown where the threat could cause a failure, as defined in Section 2.2 . Thus where the threat could only cause a serviceability problem, it is not included in the matrix.
2 This is part of the overall process of threat/ mechanism of deterioration/ indicators/ modes of failure; reference should be made to Section 2.2 for further details
3 In some instances it may require two or more threats to occur concurrently for a failure to occur

The DEFRA (2002) report indicated that the UK made significant investments in spillway upgrades in the early 1970's with the consequence that the frequency of dam failure incidents due to overtopping for the period from 1975 to 2000 has dramatically reduced compared to the period prior to 1975. This reduction was of such magnitude that dam failure incidents due to internal erosion became the most frequent type of incident.

An additional relevant finding of the DEFRA study revealed that for high hazard dams the greatest risk is from internal erosion, as for these dams the flood and earthquake guides specify reasonably conservative design requirements as shown in Figure 4.1-1.

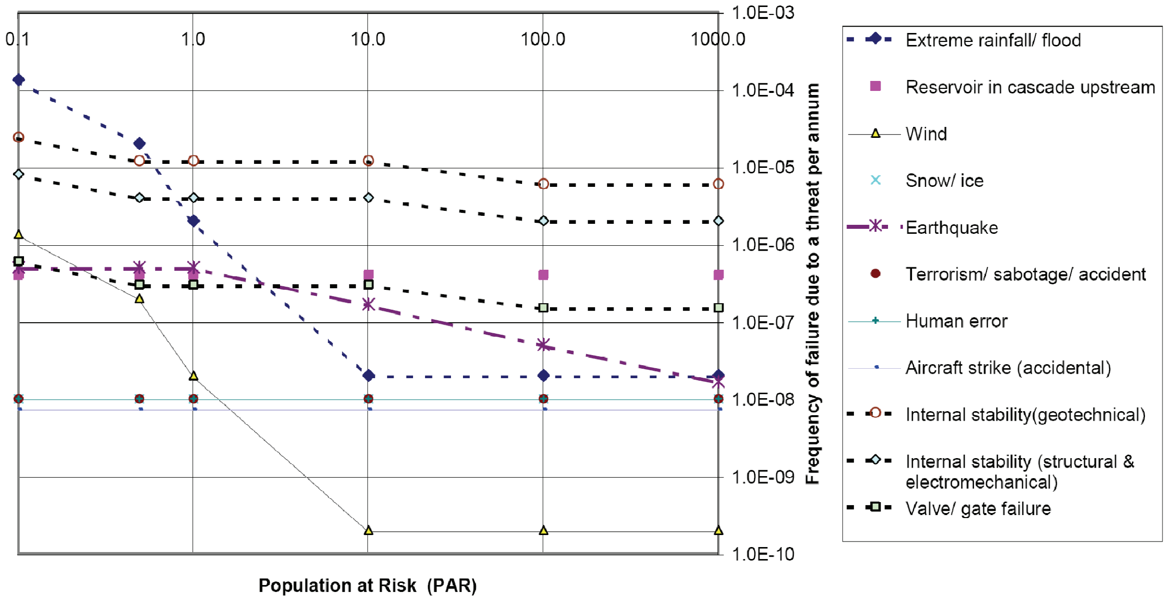


Figure 4.1-1 Annual probability of failure due to individual threat against population at risk (PAR) – DEFRA (2002)

While flood surcharge is considered in the DEFRA method, freeboard is dealt with separately from the risk assessment and only wind generated waves are considered.

One of the recommendations in the DEFRA (2002) report in the context of UK dams is inter alia that: "The prototype Integrated System devised under this research contract should be published in the form of a Preliminary Engineering Guide to Risk Assessment of dams (after minor improvements)".

4.2 GENERAL BACKGROUND ON DRIVING FORCES/MECHANISMS OF FREEBOARD CONTRIBUTING COMPONENTS IN THE SOUTH AFRICAN CONTEXT

4.2.1 General

It is useful to consider the main driving forces/mechanisms of the various processes that must be considered in the determination of freeboard for dams in the South African context, and the probability that they could occur simultaneously. Table 4.2.1 below presents the freeboard contributing components (as defined in SANCOLD, 1990) with their related driving forces/mechanisms.

Table 4.2.1 Freeboard contributing components and related driving mechanisms

	Freeboard contributing component	Driving mechanism
a	Flood surcharge	Rainfall 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 dam basin)	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.

The driving forces/mechanisms are briefly discussed under separate headings below.

4.2.2 South African main weather systems

The South African weather systems are the driving mechanisms of rainfall (with consequent runoff/floods), wind (with consequent wind set-up and waves on dam water bodies) and variations in barometric pressure (which cause a spatial and time variation in atmospheric pressure over a dam water body that can cause a long wave in the water body with consequent water level variations).

The three (3) main SA weather systems related to rainfall and wind are shown/illustrated in Figure 4.2-1.

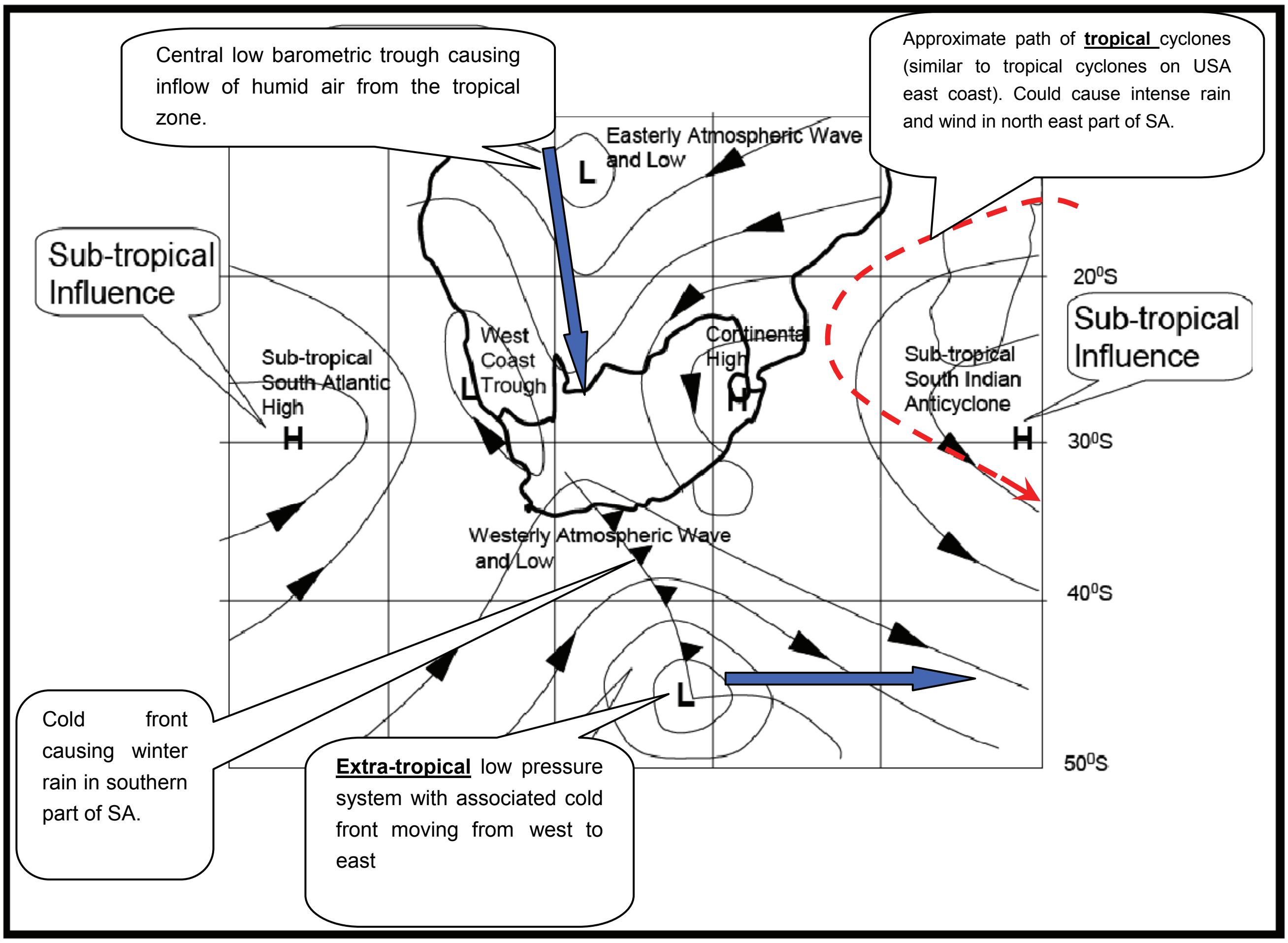
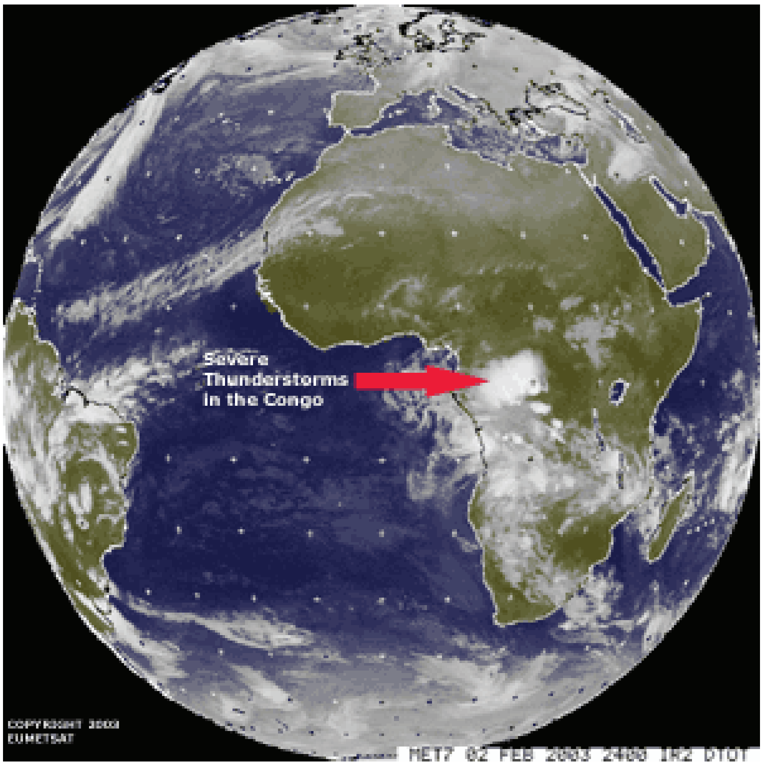


Figure 4.2-1 Composite diagram showing the important features of the surface atmospheric circulation over South Africa [Adapted from Tyson et al., 2000]

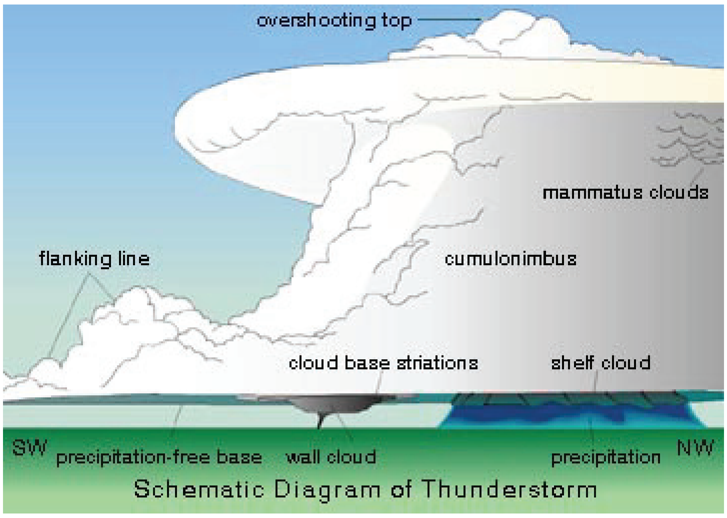
a) Summer rainfall system

The summer rainfall over the central and eastern part of SA is mainly caused by the central low pressure trough which allows the inflow of humid air/clouds (from the north-west direction) from the tropical zone. The rainfall from this system could be widespread with moderate to intense rainfall rates and wind speeds. In mountainous regions, the flow of the humid tropical air is influenced by topography as well as meteorological factors. These “orographic” wind effects, when augmented by critical meteorological patterns, may produce high wind velocities for relatively long periods of time. Therefore, they should be given special consideration when estimating wave action in reservoirs located in mountainous regions.

The humid tropical air could also form thunder storms which are more confined in area and are associated with high rainfall intensities and high local wind speeds but of shorter duration. Figure 4.2-2 shows the inflow of clouds from the tropical zone and demonstrates the composition of a thunderstorm system, which could comprise a group of thunder clouds.



a.

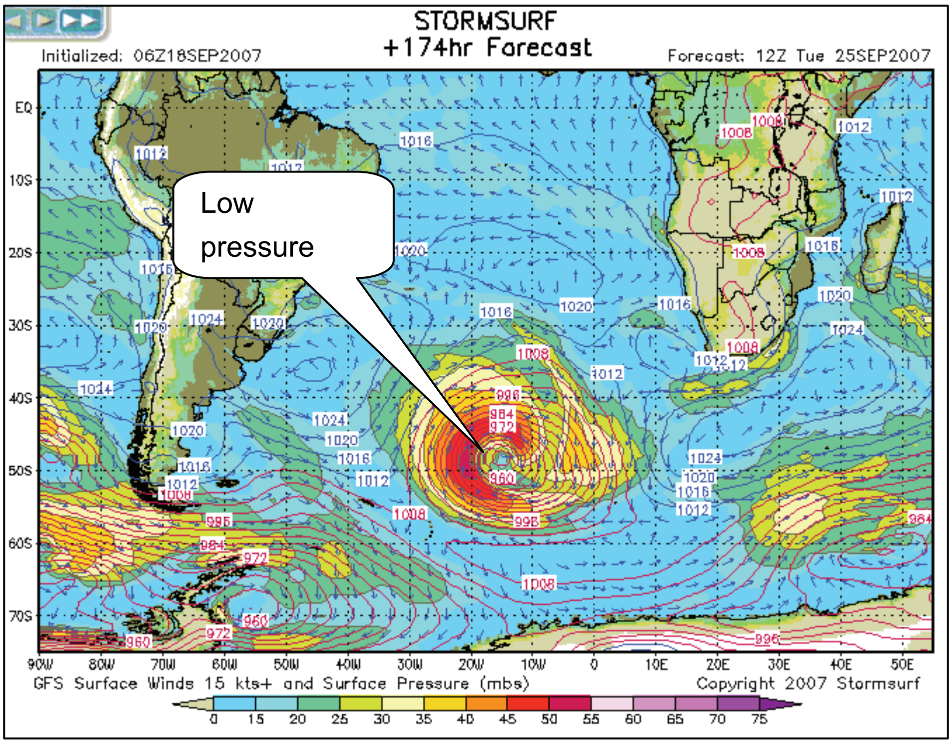


b.

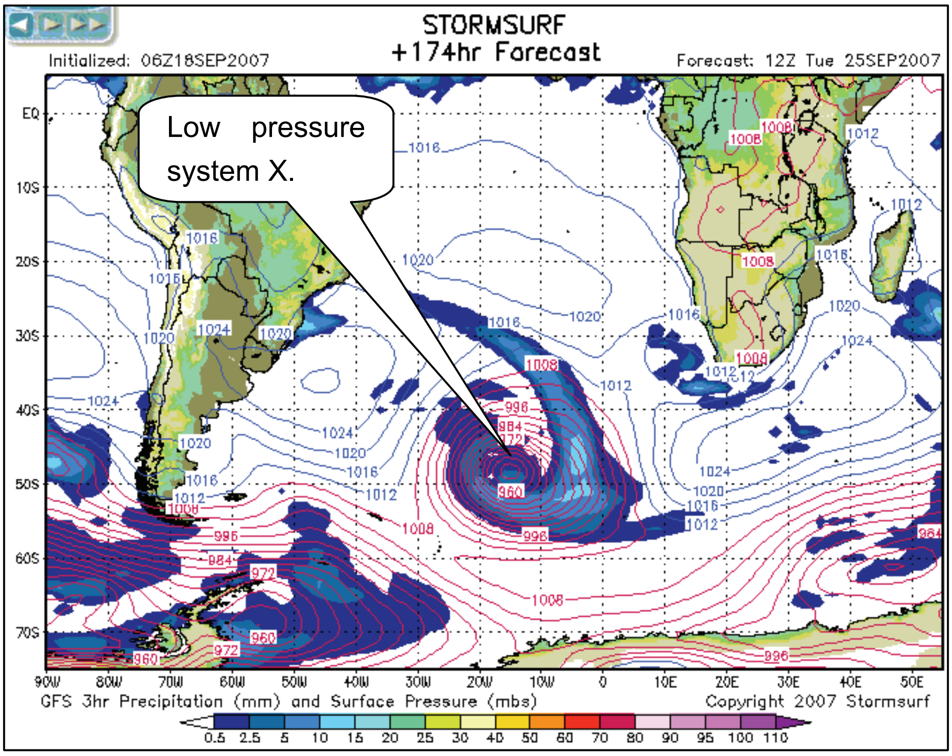
Figure 4.2-2 a) Humid tropical air (clouds) flowing into SA north east (Meteosat, 2003); b) Typical composition of a thunder storm

b) Extra-tropical storms/cold front weather system

The so-called cold front zone in the southern ocean is where the cold polar air meets with the humid and hot tropical air (the zone in the 40º to 50º latitude) and extra-tropical low pressure cells form, which move from west to east in the southern ocean, passing the African southern tip at a frequency of about one per week. These low pressure systems are associated with relatively high wind velocities (which normally change in direction sequentially from north-west, west and south as the system passed SA from west to east. The paths/tracks of these low pressure systems shift with the seasons (i.e. slightly southward during summer, and slightly northward during winter). Figure 4.2-3 shows an example of a typical low pressure system approaching SA from the west. The highest mean hourly wind speeds near the centre of these low pressure systems are typically about 25 m/s with gust wind velocities of approximately twice the mean hourly wind speed. The diameter of these low pressure systems could be up to 4 000 km.



a.



b.

Figure 4.2-3 a) Low pressure system with associated isobars, wind intensity and direction; b) Same low pressure system with associated isobars and rainfall intensity in cold front spiral shape. Website: http://www.stormsurf.com/mdls/menu_wx.html

c) Tropical cyclone systems

Tropical cyclones originate from solar heating of the ocean surface, releasing large amounts of water vapour. They occur globally and most prominent are the tropical cyclones to which the USA east cost is exposed (called hurricanes by the Americans of which Katrina was one of many). The tropical cyclones affecting SA originate north-east of Madagascar and follow tracks approximately as shown in Figure 4.2-4.

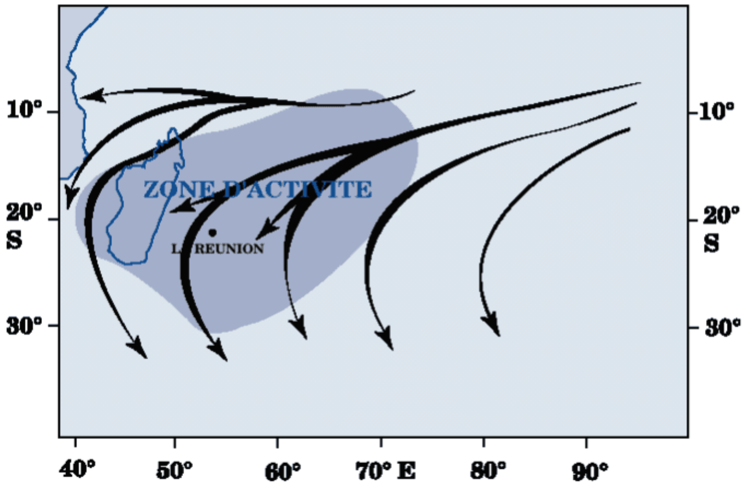


Figure 4.2-4 Approximate tracks of tropical cyclones affecting South Africa from time to time (Website http://www.prh.noaa.gov/cphc/pages/FAQ/Forecasting.php)

Figure 4.2-5 shows a satellite image of a tropical cyclone east of Madagascar.

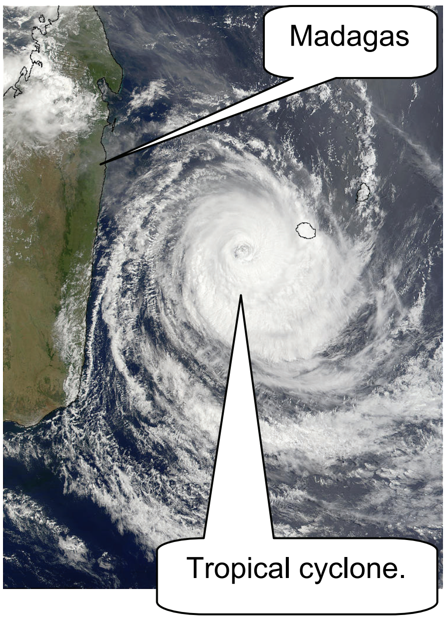


Figure 4.2-5 Example of a tropical cyclone in the Madagascar area – moving south-westward (Website http://www.prh.noaa.gov/cphc/pages/FAQ/Forecasting.php).

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 SA are presented in Figure 4.2-6 (C Rossouw, 1999).

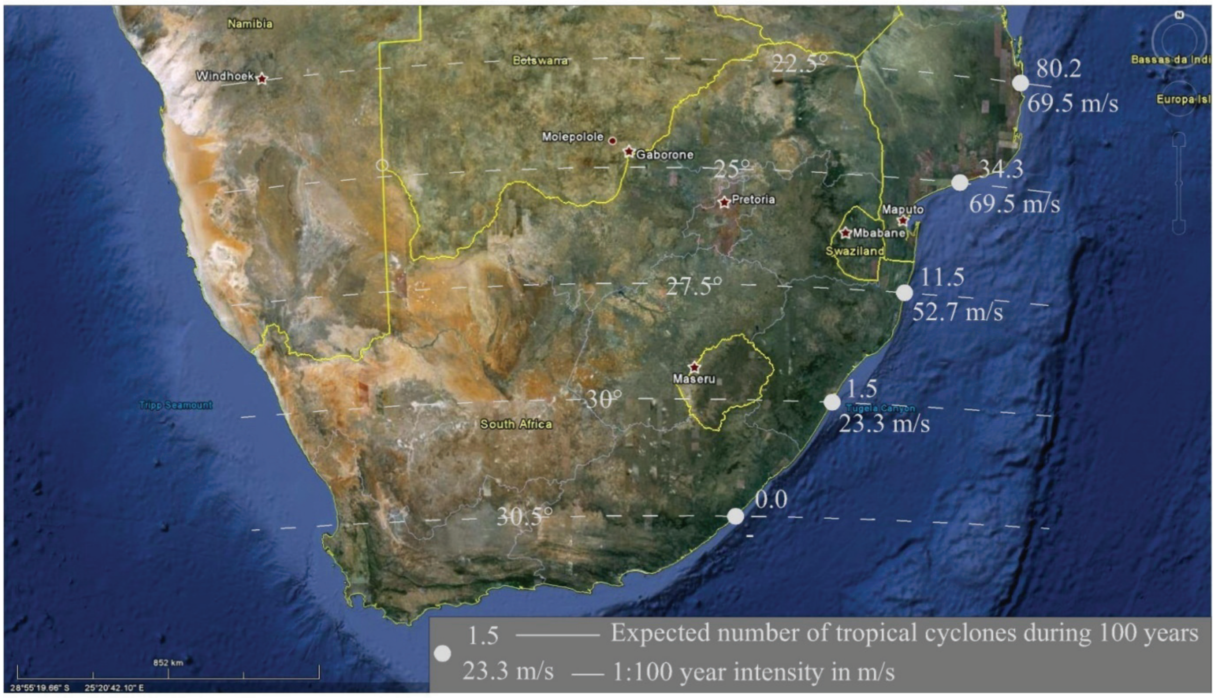


Figure 4.2-6 Tropical cyclone occurrence frequency and wind speed (1 knot = 0.514 m/s) on SA East coast. {White dots and related occurrence frequencies and wind speeds relate to dotted latitude line}. [adapted from Rossouw, 1999]

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

Indication of character and scale of different types of weather systems/mechanisms leading to high wind speeds

Table 4.2.2 after DEFRA (2002) obtained from the Engineering Sciences Data Unit (ESDU, 1990) summarizes the various mechanisms leading to high wind speeds globally.

Table 4.2.2 Mechanisms leading to high wind speeds (from ESDU, 1990)

Mechanism		Size	Wind speed
Name	Remarks		
Fronts, depressions, extratropical cyclones	Tropical storms that meander into more temperate regions and lose their identity can often be regenerated as extartropical storms when they combine with a growing disturbance along a front to form a frontal storm of extreme ferocity	Up to about 4000km in dia	Mean at 10 height of 25m/s inland; October 1987 storm had winds gusting to 45m/s
Thunderstorms	Develop along cold fronts, or when a mass of warm air is displaced upwards by mountains or solar heating. In very severe thunderstorms tornadoes may develop from vortices within the storm.	Small scale, unpredictable, typically 10km wide and 10km high	Squall gusts can be very strong, although of short duration (5 to 30 minutes at one location). In temperate latitudes winds usually less strong than those associated with deep depressions
Tropical storms (hurricanes, typhoons, cyclones)	Comparatively rare at any location, Sketch 4.3 of ESDU showing location and annual occurrence (0.1 to 5/ year). Although restricted to between –40 and + 50 latitude, the sketch suggest N. Atlantic storms from east coast of US could track into SE UK. Originate from solar heating of ocean surface, which releases vast quantities of water vapour into atmosphere.	400 to 800km in dia.	Most destructive events with very high wind speeds- surface gusts to 100m/s and mean wind speeds up to 65ms/, but typically about 50m/s, and usually very heavy rainfall. When a tropical storm attains mean wind speeds at 10m height in excess of 33m/s (74 mph) identified as hurricane. These apply where over the ocean; speeds will decay it moves to a region of cold ocean current, or if storm makes landfall.
Tornadoes and dust devils.	May occur in any area of world where strong thunderstorms likely. In temperate latitude (40 to 60°) become dominant source of extreme winds for mean recurrence intervals > 500 years (Wen & Chu, 1973) Considered unlikely in hilly terrain. Quotes devastating tornado in London in 1954 (AMS, 1982).	Typically 200m dia at ground level; path length 15km	Typically 40 to 50m/s, although in most severe cases up to 100m/s
Locally generated winds in hilly or mountainous region	Similar to tornadoes, but different in origin and much smaller in size		

Wind and rainfall statistics in South Africa

Rainfall and consequent flood statistics and methods of estimating extreme floods for dam safety evaluations in SA are reviewed in WRC Report 1420 (2007).

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 SANCOLD Guideline Report 3 (1990) has not been updated in literature. From Milford's wind data mean hourly wind speeds in South Africa can be estimated for different occurrence frequencies (from 1:50 years to 1:500 years). Until Milford's wind statistics have been updated these will continue to be the most reliable source of information for selecting wind speed (unless local wind statistics are available) for the determination of freeboard contributing components such as wind set-up and wind wave characteristics of the surface of a reservoir. In this study the Milford map was however combined with the data of Figure 4.2-6 to obtain new maps (Refer to the Volume II report). For the application of the wind speeds obtained from Milford's work (or locally recorded data) some adjustment of the wind speed is required. Milford's spatial distribution of wind speed statistics are presented below in Figure 4.2-7a.

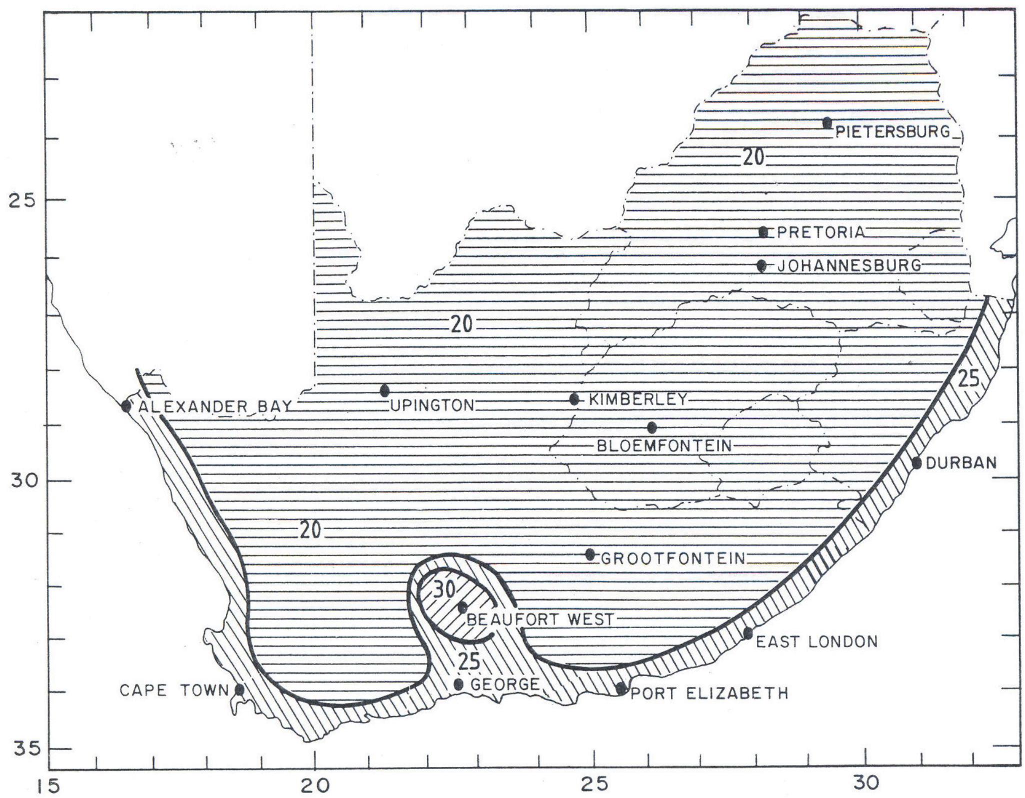


Figure 4.2-7a Mean hourly wind speed distribution in South Africa with a 1:50 year occurrence (Milford, 1987)

Milford also proposed relationships between the 1:50 year hourly average wind speed and other occurrence frequencies as presented in Figure 4.2-7b below.

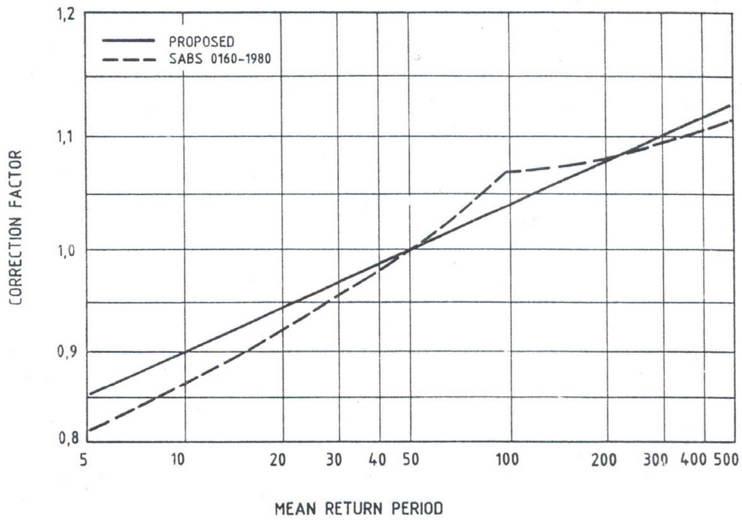


Figure 4.2-7b Milford's proposed (and SABS 0160-1980) relationship between 1:50 year hourly mean wind speed and other return periods (Milford, 1987)

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

- To adjust the selected 1 hourly wind speed (selected from Milford (1987) for a site or determined from local recordings) for a *particular wind fetch length* of a reservoir, the procedure of the logic diagram as presented in Figure II-2-20 of the Coastal Engineering Manual (CEM, 2006) is recommended. The CEM (2006) can be downloaded from the internet at: Website http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=ARTICLES;104
- Further adjustment is necessary for wave prediction if the wind duration to reach wave generation equilibrium over a given fetch length is longer or shorter than I hour (as discussed later in Section 4.4.3). This adjustment is normally done using the relationship between the 1 hour mean (U3600) and the longer or shorter (Ut) duration wind speeds in accordance with Figure II-2-1 of CEM (2006) as presented in Figure 4.2-8 below.

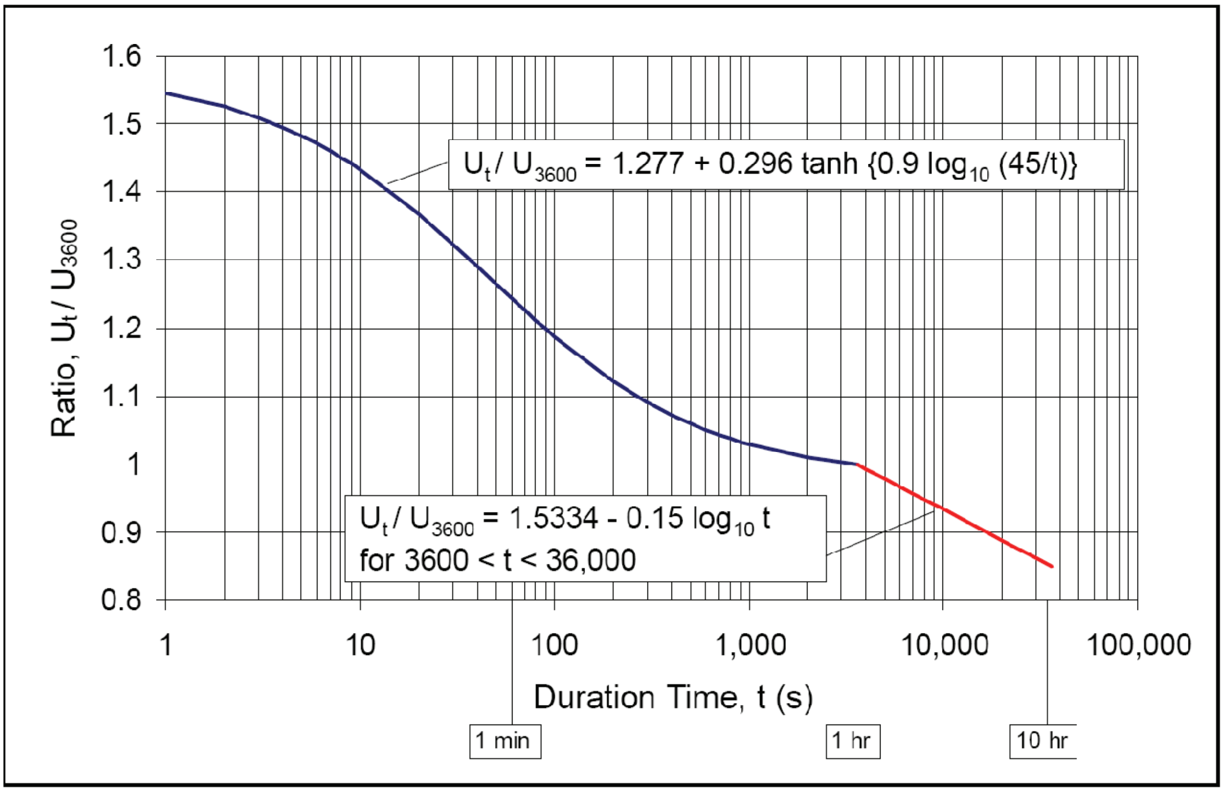


Figure 4.2-8 Ratio of wind speed of any duration, *Ut,* to the 1-hr wind speed, *U*3600 CEM, 2006).

4.2.3 Seismic statistics of South Africa

The existing SABS 0160 (1989) seismic hazard map has been updated and it is recommended that this updated map should replace the seismic hazard contained in the SABS code. The updated seismic hazard map is presented in Figure 4.2-9 below.

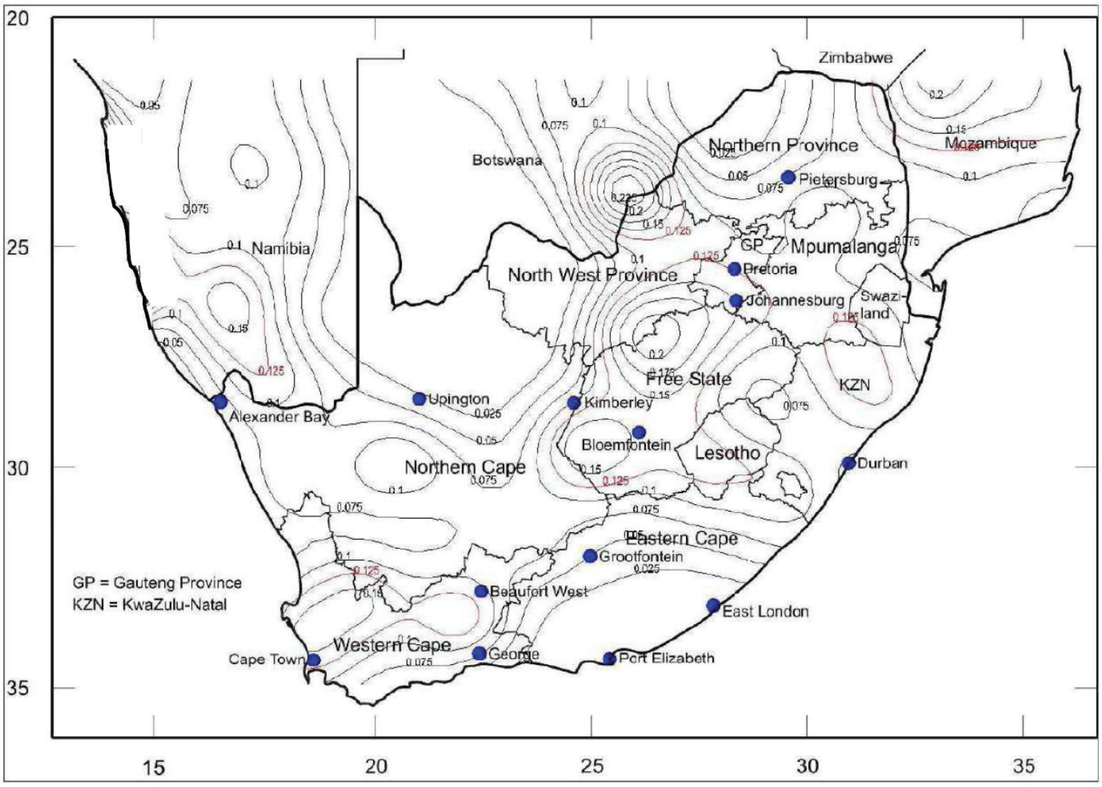


Figure 4.2-9 Seismic hazard map from Council for Geoscience (2003) data showing peak ground acceleration as a percentage of g (gravity acceleration) with 10% exceedance in 50 years

As indicated in Table 4.2.1 earthquakes (seismic events) can cause landslides as well as oscillation of a water body in a reservoir with consequent surge or water level oscillation at the dam wall. An example of a landslide that slid into a lake in Alaska is presented in Figure 4.2-10 below.

Figure 4.2-10 Landslide in a lake in Alaska

4.3 EXISTING SANCOLD *INTERIM* GUIDELINES ON FREEBOARD FOR DAMS

The existing SANCOLD Interim Guidelines on Freeboard for Dams (1990) was prepared with great care by a very competent team of professionals, taking into account current practices followed by other countries at the time. It was recommended by the authors of the 1990 Guidelines that this should be a dynamic document to be updated from time to time as more data becomes available and to keep up with the development of new technologies.

WRC Report No 1420 (2007) recommended the update of all SANCOLD Guidelines relating to dam safety, including the SANCOLD *Interim* Guidelines on Freeboard for Dams (1990). Therefore in this report the methodologies reported in the literature for quantifying the components contributing to dam freeboard (excluding flood surcharge) are reviewed in Section 4.4 below. The combination of components (weakly or strongly related) that could occur simultaneous will also be reported on as found in the literature.

4.4 LITERATURE SURVEY OF LATEST DEVELOPED METHODS AND TOOLS TO QUANTIFY FREEBOARD CONTRIBUTING COMPONENTS

For clarity an adapted schematic diagram of the freeboard contributing components (DEFRA, 2002) is presented in Figure 4.4-1 below.

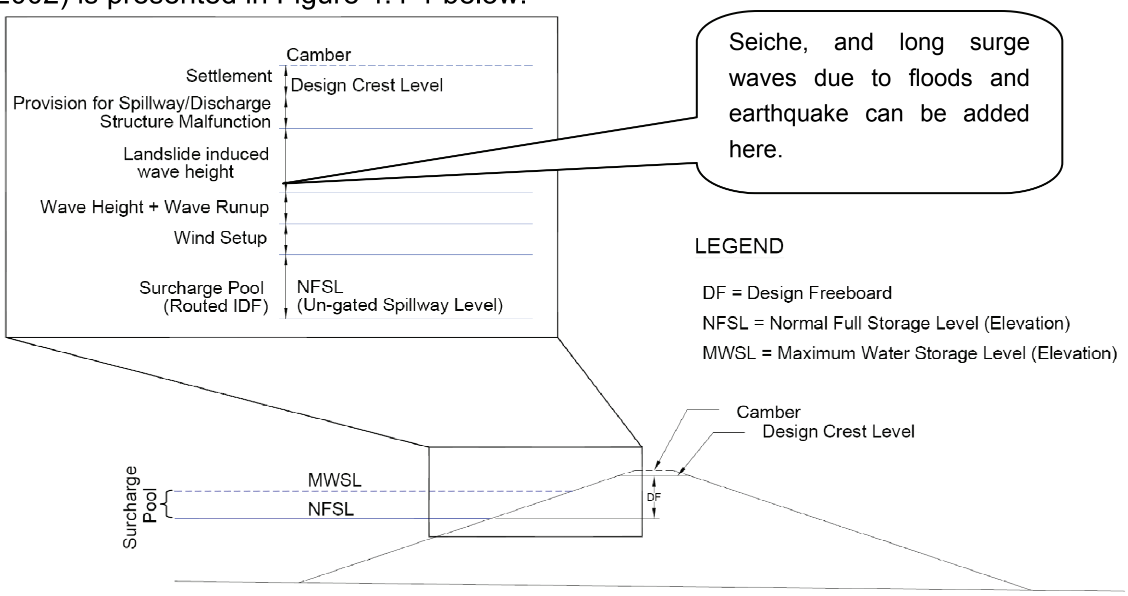


Figure 4.4-1 Components contributing to freeboard (demonstrated for an embankment dam (DEFRA, 2002)

As indicated earlier in this report, the following literature survey presents current practice regarding the quantification of secondary dam freeboard contributing components (i.e. excluding flood discharge). Current practice in coastal engineering, relevant to the study is also included in the literature survey. Both applicable sophisticated numerical methods as well as simplified analytical desk top methods are presented.

4.4.1 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 WRC Report No 1420 (2007). 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, etc. Appropriate hydrodynamic numerical models which could be employed include:

DELFT 3D [Website: http://www.delftcluster.nl/website/nl/page107.asp]
MIKE11, MIKE 21 and MIKE 3
[Website: http://www.dhigroup.com/Software/Marine/MIKE21.aspx]

4.4.2 Wind set-up

As described in SANCOLD (1990), wind set-up can be defined as an increase in water level resulting from wind shear stress arising from wind blowing over the water body, or the surface of the reservoir formed by the dam wall. This is illustrated in Figure 4.4-2 below.

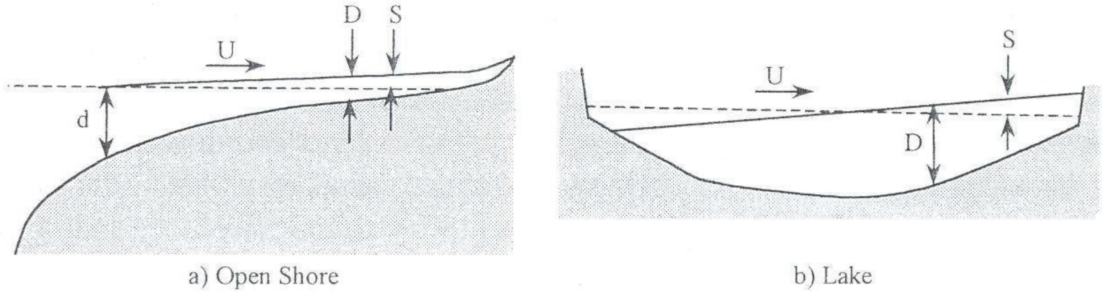


Figure 4.4-2 Definition sketch for storm surge (Kamphuis, 2002)

This temporary water level increase occurs at the same time as the concurrent wave action and is the cause of coastal flooding in many countries including the low laying deltaic areas on the Dutch coast, in the Mississippi delta, in Bangladesh, etc. (with wind set-ups in the order of 5 m to 10 m). However, in dams with limited wind fetch lengths, the wind set-up is significantly less but should be calculated and taken into account when determining the *secondary component* of the total freeboard for a dam.

During storm surges, the water level at the down-wind shore is raised by the shear stress from the wind and is counteracted by gravity (the slope of the water surface).

The wind set-up in a dam basin can be determined using hydrodynamic models similar to the ones indicated under Section 4.4.1, i.e.:
DELFT 3D
MIKE 21

Simplified analytical methods are also available of which those of Dean et al. (1992) and Kamphuis (2002) are relevant. The method proposed by Kamphuis is a practical method which takes water depth into account. The method is described in Box 4.4.1, which is an excerpt from Kamphuis (2002).

Box 4.4.1 Excerpt from Kamphuis (2002) describing the relevant formula for one-dimensional surge calculation (refer to Figure 4.4.1).

In this case wind-generated shear stress is the main driving force. For simple problems, the equations can be reduced to a one-dimensional computation

$$\frac{dS}{dx} = \frac{\zeta\,(U\cos\phi)^2}{g\,D} \tag{6.4}$$

where S is the storm surge (the setup of the water level by the wind), x is the distance over which the storm surge is calculated, ζ is a constant ($=3.2\cdot10^{-6}$), U is the wind speed, ϕ is the angle between the wind direction and the x-axis and D is the new depth of water (=d+S). Equation 6.4 shows that storm surge is greatest in shallow water; that is why Bangladesh on the delta of the Ganges, Brahmaputra and Meghna rivers and the Netherlands on the delta of the Rhine, Meuse and Scheldt are very susceptible to storm surge.

Kamphuis provided an example on a spreadsheet showing how incremental surge levels over a dam basin can be calculated. The spreadsheet example has been made available by Kampuis in the public domain (listed as the last item, "Software", of the book content) and can be downloaded at:
Website http://www.worldscibooks.com/engineering/4064.html

Kamphuis (2002) pointed out that Equation 6.4 in Box 4.4.1 above assumes steady state conditions with the wind blowing in the same direction at the applied wind velocity. Thus it computes the maximum wind set-up and should be used for feasibility studies and conservative desk-top design.

4.4.3 Wind generated waves on the surface of a reservoir

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 eg:

- **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 and mean wave 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) and where it is undergoing further enhancements. It is specified as the new standard for nearshore wave modelling and coastal protection studies. Deltares has integrated the SWAN model in several models and is applying SWAN in its consultancy projects.

- **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;9

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), 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.

For the purpose of a desk-top study, simplified formulae could be applied to determine the above wave parameters. Numerous formulae sets are available for application to the open ocean situation but only three formulae are recommended by the comprehensive recently updated manual, The Rock Manual (2007) [Electronic copy can be downloaded from Website www.kennisbank-waterbouw.nl/rockmanual] and are described under Paragraph 4.2.4.6 (page 367 to 370). The Rock Manual explicitly indicates that the open coast type formulae cannot be applied to inland lakes and dam basins. It recommends and defines the following three methods for application to reservoirs and lakes (pages 371 to 374):

- Saville method (or SBM method with effective fetch)
- Donelan method
- Young and Verhagen method

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 (Hs), 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 Hs. The characteristic wave height ratios for an irregular wave state with a Raleigh distribution of wave heights are presented in Table 8 (page 357) of The Rock Manual: eg the ratio between $H_{2\%}$/Hs ≈ 1.4 and Hmax/Hs ≈ 2. $H_{2\%}$ is the wave height in the irregular wave train, which is exceeded 2% of the time and Hmax is the maximum wave in a reasonably long irregular wave train. These wave heights are relevant in the determination of wave run-up. [An irregular wave train generated by wind over a dam is illustrated in **Appendix B** – other relevant wave parameters including a JONSWAP wave energy spectrum are also presented].

- Wave period – usually the peak period is used (this is the period in the irregular wave spectrum where the maximum wave energy occurs). Tm-1,0 is also a spectrum related period often used in wave run-up calculations and Tp/ Tm-1,0 ≈ 1.1. Tm (mean wave period) is between 0.71Tp to 0.87Tp. Tp/Tm usually lies between 1.1 and 1.25.
- Waves in deep water (where the wave length is shorter than the water depth) which are steeper (wave height/wave length) 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 specific Paragraph 4.2.4.3.

4.4.4 Wave run-up

Wave run-up on a sloped embankment is defined in Figure 4.4-3 below for a $H_{2\%}$ wave height with wave height Hm0≈ Hs at the embankment toe.

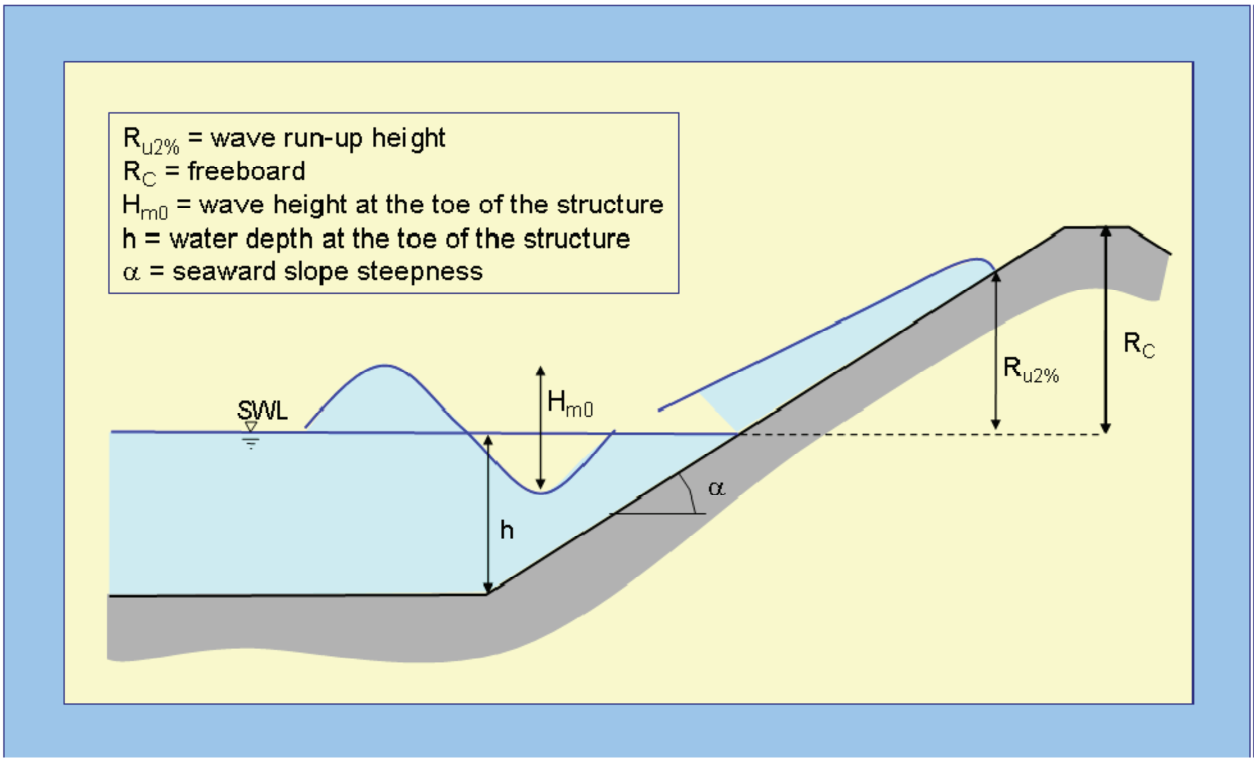


Figure 4.4-3 Wave run-up for a $H_{2\%}$ wave height on a smooth embankment slope (Pullen et al., 2007)

The time history of wave run-up on a slope is demonstrated in Figure 4.4-4 below.

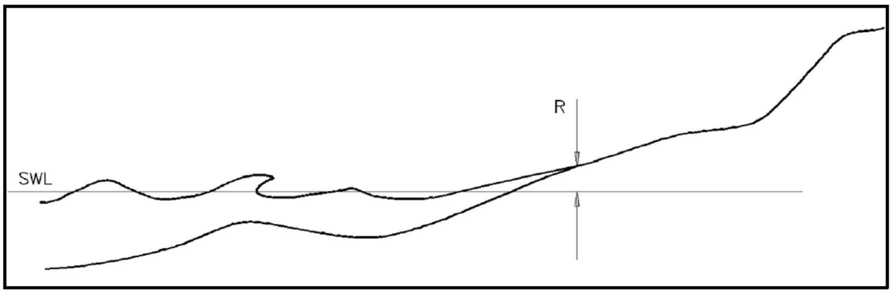


Figure II-4-11. Definition sketch for wave runup

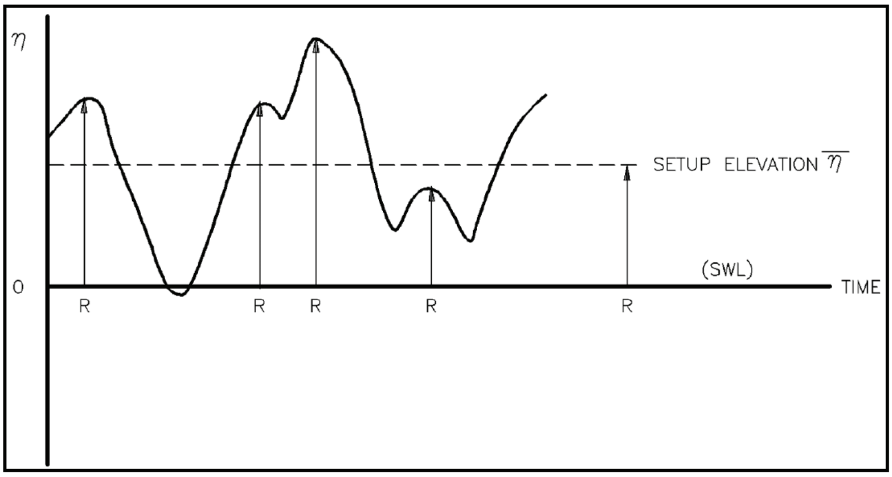


Figure 4.4-4 Illustration from CEM (2006) of an irregular wave train's run-up time history (includes wave set-up)

Wave run-up is treated in detail in The Rock Manual (2007) under paragraph 5.1 (page 487) and in the CEM Part VI Chapter 5. Run-up of an irregular wave train is comprised of two components i.e. wave set-up and wave-run-up as demonstrated in Figure 4.4-4.

[Note: The Rock Manual (2007) is available in English and French as both a book and a CD-Rom. The material can also be downloaded from the CIRIA and CETMEF websites (www.ciria.org and www.cetmef.equipement.gouv.fr). A large number of equations from this manual are included in the software package CRESS, which is free to download from the website www.cress.nl . More than 100 experts from Europe and across the world were involved in the project to update *The Rock Manual*, ensuring that the updated edition will retain its place as the primary reference guide worldwide for the use of rock in hydraulic engineering.

The present generally accepted wave run-up formulae are presented in Paragraph 5.1.1.2 of The Rock Manual (2007). Calculation methods for run-up probabilities on rough slopes of 0.1%, 1%, 2% and 10% are presented on page 495.

A useful software tool to correlate hand calculation results using the Rock Manual's run-up and over topping formulae can be checked with a software tool CRESS which is downloadable from website www.cress.nl .

Since the SANCOLD Flood Guidelines of 1990 indicates that for the safety evaluation discharge (SED), extensive damage to the dam by overtopping may be allowed (however the

dam must not fail under these conditions), it is useful to refer to the "EurOtop Wave Overtopping of Sea Defenses and Related Structures: Assessment Manual" (2007) since the entire wave spectrum is considered and one can establish overtopping rates for different freeboards as well as at which crest level overtopping will be zero. The manual can be downloaded from:
Website http://www.overtopping-manual.com/index.html

Also available on the latter website (managed by HR Wallingford in collaboration with their Dutch and German partners on this ongoing project) is an on-line calculation tool which could be used in the public domain to calculate overtopping rates for different types of structures (eg embankments with and without crest structures).

[This project is funded in the UK by the Environmental Agency, in Germany by the German Coastal Engineering Research Council (KFKI), in the Netherlands by Rijkswaterstaat, Netherlands Expertise Network on Flood Protection].

4.4.5 Example of calculation procedure to determine wind related freeboard contributing components

Since the process of calculating wave run-up comprises a relatively large number of steps and alternatives, **Appendix A** (DMC -2, 2006) is included in this report as an example which could be useful in guiding the calculation procedure. This example is taken from the extensive Everglades Wetland Restoration Programme on the northern section of Florida, USA, where large embankment dams with overflows are created as part of the restoration plan. The Everglades wetlands are located in a large area at the Southern tip of Florida peninsula, which is in the path of tropical cyclones (termed hurricanes by Americans). The embankment dams are relatively large with extensive fetch lengths, which, with the passing hurricanes with their high intensity waves, could cause significant waves in the impoundment dams. Therefore adequate freeboard has to be provided to the embankment dams to prevent damage to them.

Although mostly American references are used in the calculation procedure of wind set-up and wave run-up, the basic procedure (comprising 33 steps) is considered generally applicable even with different formulae and models being applied. The number of steps seems excessive, however, it guides the user who might not be so familiar with the relevant field of expertise to ensure that all relevant factors, which play a role, are addressed. These steps are considered to be generally applicable even with different formulae and models being applied. The basic procedure covers the following main areas:

- Determine design wind condition.
- Determine wind set-up.
- Determine wave height and wave run-up.

The example methodology (presented in **Appendix A**) which covers the above three areas, follows thirty three (33) steps, each with the formula(e) applied and relevant source reference(s).

4.4.6 Surges induced by barometric variations, floods and landslides

Surges caused by barometric pressure variation over a dam basin, floods flowing into a dam basin, landslides displacing a significant volume of the dam basin water and earthquakes all cause so-called long waves in the dam basin. The behaviour of these long waves in the dam basin can only be estimated by means of 2D (depth averaged) or 3D hydrodynamic models such as MIKE 21 and DELFT 3D with the objective to quantify the water level variation at the wall of a dam.

4.4.7 Estimation of magnitude of wave heights that can be generated in the water in a dam basin (on the upstream side of a vertical concrete dam wall face) due to horizontal movement of a dam wall due to earth quakes.

4.4.7.1. Background on earth quake 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. Figures 4.4-5 shows the distribution of shock oscillation periods versus number of recordings as obtained from the analysis of the data.

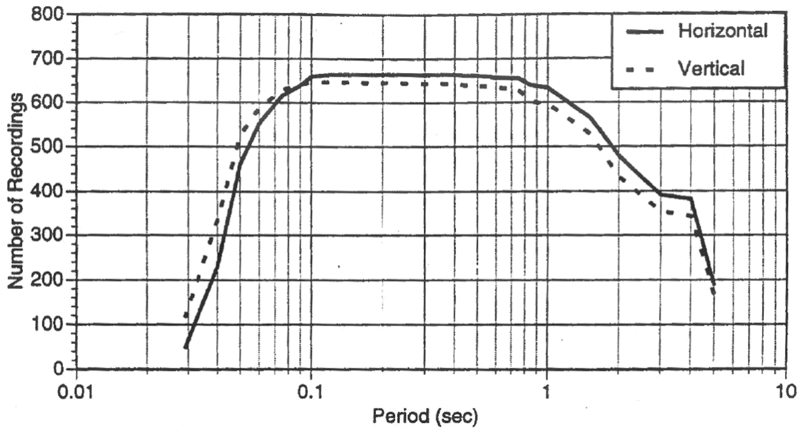


Figure 4.4-5 Distribution of shock oscillation periods versus number of recordings (Abrahamson et al., 1997).

Figure 4.4-6 presents the derived horizontal accelerations versus shock oscillation periods for different shock magnitudes 10 km away from shock source for different shock magnitudes (M).

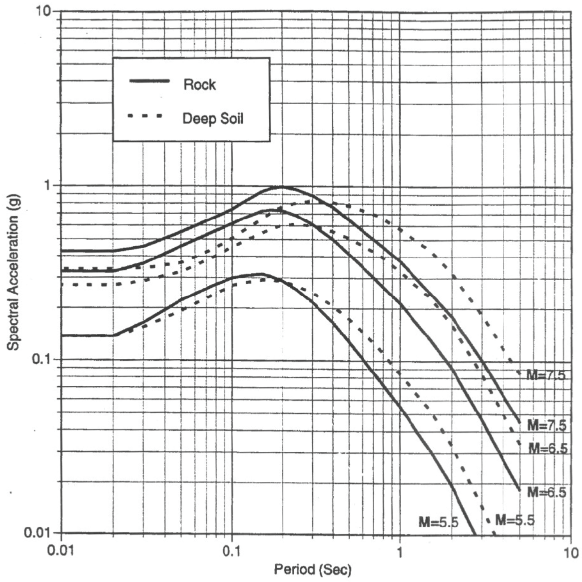


Figure 4.4-6 Derived horizontal accelerations versus shock oscillation periods for different shock magnitudes 10 km away from shock source (Abrahamson et al., 1997)

4.4.7.2. Estimation of seismic generated wave 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 wave water wave heights on the upstream vertical face of a dam wall that moves horizontally at different periods and accelerations due to earth quakes (assuming the dam wall as a horizontal moving wave generating paddle):

(i) Estimate a horizontal acceleration from a seismic hazard map (e.g. Figure 4.2-9 of literature review part of the main report).
(say, 0.1 g and 0.2 g).

(ii) Select different shock oscillation periods in the dominant oscillation period range – refer Figure 4.4-5.
(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 4.4-6). The formula for calculating this amplitude is then:

$$Amp = \left(\frac{acc}{4}\right) \times \left(\frac{T}{\pi}\right)^2$$

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, **Water Wave Mechanics for Engineers and Scientists (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 4.4-7 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 4.4-7).

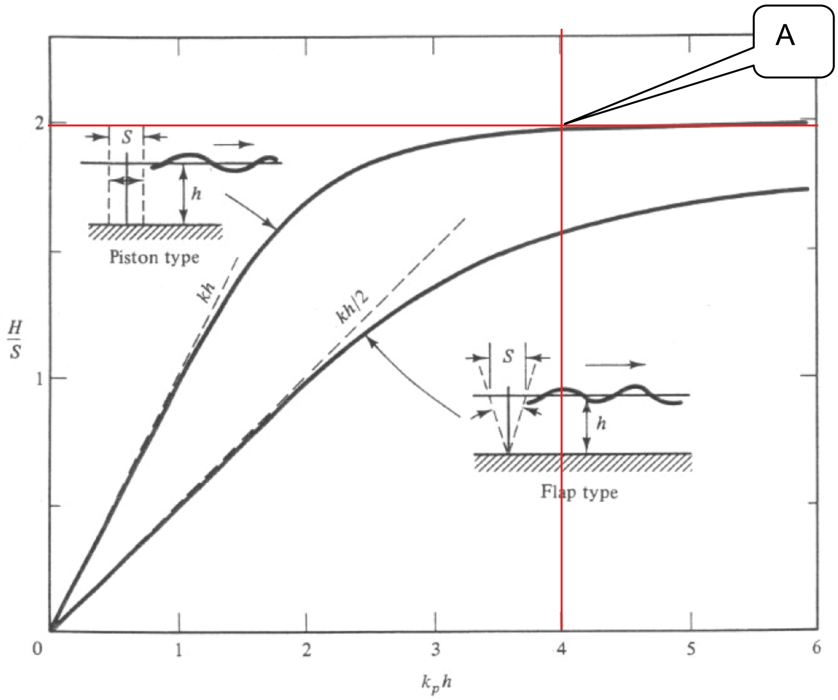


Figure 4.4-7 Plane wave maker theory. Wave height (H) to Stroke (S) ratios versus relative depths (2π/L)*h. Piston and flap type wave maker motions. [k_p = 2π/L = 2π/(1.56*T^2) for a deep water wave as would be the case since the oscillation period is short and the water level at a vertical concrete wall is relatively deep]. Dean and Dalrymple (1997).

Figure 4.4-8 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.

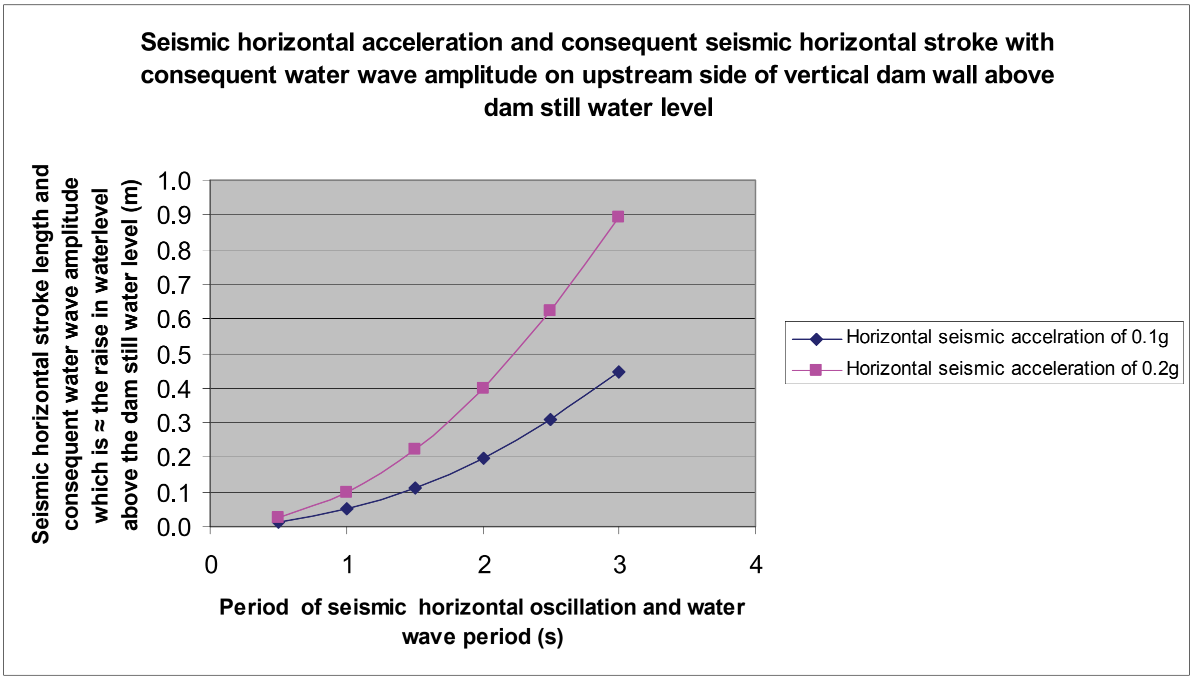


Figure 4.4-8 An example of water wave amplitude calculation for concrete dam walls higher than 15 m for 0.1 g and 0.2 g horizontal seismic accelerations.

4.5 CONCLUSIONS

The literature survey has shown that in recent years (approximately 2002 to 2007) a number of methodologies have been developed to determine the magnitudes of the secondary dam freeboard contributing components. Selected references are summarized in this report and most are public domain and downloadable at no charge.

5. ANALYTICAL METHODS TO DETERMINE WIND WAVE HEIGHT

5.1 INTRODUCTION

5.1.1 Objective

The objective of this specific component of the study was to compare and evaluate some of the different analytical methods with regard to Wind Wave Height that were identified in the Literature Review as described in Chapter 4.

The Bloemhoek Dam was selected to serve as case study for these comparisons. Bloemhoek Dam is an earthfill embankment dam located some 4 km east of Kroonstad in the Free State on the Jordaan Spruit, which is a tributary of the Vals River. The dam provides domestic water to the town of Kroonstad.

5.1.2 Scope

From the Literature Review (Chapter 4), three publications describing different freeboard calculation was selected as comparison to the "Interim Guidelines on Freeboard for Dams" (SANCOLD, 1990). These three publications were as follows; the South Florida Water Management District – Design Criteria Memorandum (DCM-2), 2006; the Coastal Engineering Manual (CEM), 2006; and the Rock Manual, 2007.

Each of the following freeboard design parameters; fetch, wind speed, wind set-up, wave height and wave period as well as wave run-up were dealt with individually in order to provide a detailed comparison of each of the design methods with regard to the different freeboard design parameters.

5.2 FREEBOARD DESIGN PARAMETERS

5.2.1 Fetch and Effective Fetch

The calculation of effective fetch as described in the SANCOLD guidelines (1990) was developed by Saville et al., as the average reach over a 90° arc across the water from a point of concern at the dam wall. It is assumed that the effectiveness of the wind is proportional to the cosine of the angle from the average wind direction. The total effectiveness of each fetch segment is proportional to the product of these two values. An effective fetch of 1.66 km was calculated. The fetch is determined for the corresponding flood surcharge as specified in the SANCOLD guidelines.

The DCM-2 does not specifically deal with effective fetch and mention is only made that the effective fetch should be selected. In the attachment A of the DCM-2 the fetch length of the example reservoir was determined by constructing 9 radials from the point of interest at 3° intervals. The average length of the radials is used to represent the fetch length at the point of interest. Should the same number of nine radials from the point of interest at 3° intervals be used on Bloemhoek Dam, the effective fetch would be 2.41 km.

Fetch is defined by the CEM as a region in which the wind speed and direction are reasonable constant. The CEM also states that fetch should be defined so that wind direction variations do not exceed 15° and wind speed variations do not exceed 2,5 *m/s* from the mean. The determination of effective fetch is however not addressed in the CEM.

The Rock Manual mentions three methods with regard to reservoir fetch; *Saville Method*, the *Donelan method* and the *Young and Verhagen Method*.

The Saville method was described earlier as part of the SANCOLD Guidelines (1990). It is however important to note that the example in the Rock Manual makes use of radials of 7.5° intervals. The result of using fewer radials within the 90° arc is that the effective fetch is reduced.

According to the Rock Manual, the Donelan Method is based on the idea that the fetch length should be measured along the wave direction (θ), rather than the wind direction (Φ_w). The effective fetch can be determined by maximising the product of $\cos(\Phi_w - \theta)F^{0.426}$, where θ is the wave direction and Φ_w the wind direction. Using this method the effective fetch for Bloemhoek Dam was calculated as 1.68 km.

5.2.2 Design Wind Speed

In the SANCOLD Guidelines (1990) it is stipulated that the selection of wind speed must take into account the category of dam and the hazard potential as well as the severity of the flood under consideration.

The nearest updated wind velocity and duration information available was for Bloemfontein. The highest over land hourly wind speeds (*m/s*) for 50 year recurrence interval was taken as 23.8 *m/s*. The effective fetch is used in the conversion of the over land wind speed to over water wind speed. From the effective fetch of 1.67 *km* the wind speed ratio was determined as 1.14, yielding a 50 year recurrence interval over water wind speed of 27.1 m/s. Using the correction factor for other return periods contained in the SANCOLD Guidelines (1990), of 1.04 for 100 year recurrence interval, the design wind speed was calculated as 28,2 m/s.

The DCM-2 recommends that the 50 year, 3 second wind speeds be selected and that conversion factors given in the DCM-2 are used to convert it to the required hourly recurrence interval. The DCM-2 refers to the CEM for the adjustment of over land wind speed to an over water wind speed. Without any information with regard to the 3 second wind speed, the 50 year hourly wind speed was taken from the SANCOLD Guidelines (1990). With a recurrence interval conversion factor of 1.07 and a 1.2 adjustment to over water wind speed, a design wind speed of 30,6 m/s was calculated using the DCM-2.

The ratio of wind speed of any duration to the hourly wind speed is given in the CEM, (Figure II-2-1). The CEM states that several different averaging times should be considered for wave prediction to ensure that the maximum wave growth scenario has been identified. A conversion factor of 1.2 is given for over land wind to over water wind speed for a fetch length less than 16 km.

The Rock Manual does not refer to a specific time series of wind speed and direction and provides a table for wind speed conversion factors related to duration of wind speeds with the hourly wind speed as the base.

5.2.3 Wind Set-Up

The SANCOLD Guidelines (1990) recommends using the following formula by Saville for calculating the rise in water level above the Stillwater level at the dam wall due to wind set-up:

$$S = \frac{V^2 \text{ x } F}{4\,850 \text{ x } D}$$

where	S	=	rise above stillwater level (m)
	V	=	design wind speed (m/s)
	F	=	fetch (km)
	D	=	average water depth in the basin along the fetch (m)

Usually the length in wind set-up computations is taken as twice the effective fetch used for wave height computations.

The average water depth is calculated as follows:

$$D = \frac{\text{Storage Volume at FSL}}{\text{Area at FSL}}$$

Using the formula recommended by the SANCOLD Guidelines (1990) the wind set-up was calculated to be 0.08 m.

In the DCM-2 mention is made of three methods for determining wind set-up. The most widely used wind set-up equation as applied to reservoirs is the Zeider Zee equation, which according to the DCM-2 normally results in the highest wind set-up determination of the three methods. For deeper impoundments, with average depths greater than 4.9 m, the Zeider Zee equation is recommended because the equation provides a continues solution without excessive over estimation of wind set-up.

$$S = \frac{U^2 \text{ x } F}{1\,400 \text{ x } d}$$

where	S	=	rise above stillwater level (feet)
	U	=	average wind velocity over the fetch (miles per hour)
	F	=	fetch length (miles)
	D	=	average water depth in the basin along the fetch (feet)

The Zeider Zee equation yields a wind set-up of 0.04 *m*, 50% less than what was calculated using the SANCOLD Guidelines (1990). Should one use twice the fetch as in the SANCOLD Guidelines (1990) the Zeider Zee equation gives the same answer as in the SANCOLD Guidelines (1990).

No mention is made of wind set-up in the CEM.

The Rock Manual recommends the following equation to calculate wind set-up:

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

For a closed water domain of length, F (m), with a constant water depth, h (m), and a constant wind speed, U_{10} (m/s), blowing over the water domain, the resulting maximum wind set-up, n_w is calculated. The air/water drag coefficient C_D increases with wind speed with typical values of 0.8E-3 to 3.0E-3. Using a value of 3.0E-3 for C_D, the wind set-up was calculated to be 0.04 m.

It is important to note that the wind set-up is especially sensitive to the wind speed and water depth. Without any survey information for the reservoir basin, it was not possible to calculate the average depth over the specific fetch length and therefore the average water depth for the total reservoir was used.

In the Literature Review in Chapter 4, reference is made to a simplified analytical method proposed by Kamphuis. Due to fact that no information was available with regard to the reservoir floor elevations, assumptions were made with regard to the water depth at different distances from the dam wall. The wind set-up was calculated as 0.7 m.

This method yields a wind set-up an order of magnitude higher than the other methods. However, if one takes a single increment as calculated by the Kamphuis spreadsheet, then the wind set-up for such a segment with depth of 7.1 m and a length of 3.3 km is only slightly higher than what was calculated for the same depth and length using the SANCOLD Guidelines (1990). The difference comes in by adding each of the segments together; the cumulative wind set-up becomes significantly higher. The method proposed by Kamphuis also illustrates the effect of water depth on wind set-up.

5.2.4 Wave Height and Wave Period

Wave heights are governed by the reservoir water depth, effective fetch, wind speed and the duration of the wind. Diagrams are given in the SANCOLD Guidelines (1990) for the determination of the significant wave height and wave period. Making use if these diagrams resulted in a wave height of 0.83 m and a wave period of 3.0 s.

Mention is made in the DCM-2 of computer programs as well as calculations that can be used in the determination of wave height. The DCM-2 refers to the CEM and the Shore Protection Manual (SPM) for the calculation of wave height and wave period. The following two deep-water equations from the SPM were used to calculate the wave height and wave period:

Box 5.2.4.1 Excerpt from the Shore Protection Manual for equations 3-33 and 3-34.

$$\frac{gH_{m_o}}{U_A^2} = 1.6\text{x}10^{-3} \left(\frac{gF}{U_A^2}\right)^{1/2}$$

$$\frac{gT_m}{U_A} = 2.857\text{x}10^{-1} \left(\frac{gF}{U_A^2}\right)^{1/3}$$

The SPM recommends using the relative depth to determine whether the computations of the wave height and period should be based on the shallow-water equations or deep-water equations. In the case of Bloemhoek Dam the relative depth was greater than 0.5 and therefore the deep-water equations were used. The wave height was determined to be 0.77 m and the wave period 2.61 s.

The CEM makes mention of nomograms that have played an important role of providing wave information in the past. This task has now been taken over by computer programs such as the ACES Program. Conventional prediction methods are also discussed in the CEM. The CEM identifies three situations in which simplified wave predictions can provide accurate estimates of wave conditions, but only two are commonly met in nature. The first is fetch limited which occurs when the wind blows, with essential constant direction over a fetch for sufficient time to achieve steady state. The second situation is that of a fully developed wave height. Formulae as well as nomograms are provided in the CEM for the determination of the wave height. It is recommended that a straight-line fetch be used to define fetch length for the CEM applications. The CEM also recommends that deepwater wave growth formulae be used for all depths, with the constraint that no wave period can grow past a limiting value as shown by Vincent (1985). The limiting wave period was calculated as 8.3 s from the following relationship, $T_p = 9.78*(D/g)^{0.5}$. From the nomograms provided in the CEM and using the fetch and wind speed determine by the DCM-2, the wave height was determined to be 0.94 m and the wave period 2.2 s.

The Rock Manual uses two methods to calculate the wave height, the Donelan method and the Young and Verhagen method. The Donelan Method is based on the idea that the fetch length should be measured along the wave direction (θ) rather than the wind direction (Φ_w).

The effective fetch can be determined by maximising the product of $\cos(\Phi_w - \theta)F^{0.426}$, where θ is the wave direction and Φ_w the wind direction. Once the wave direction is determined, the significant wave height, peak period and minimum wind duration are derived from equation 4.86-4.88 (p372).

Box 5.2.4.2 Excerpt from the Rock Manual (2007) for equation 4.86 to 4.88.

$$\frac{g H_s}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}=0.00366\left(\frac{g F_\theta}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}\right)^{0.38}$$

$$\frac{g T_p}{U_{10}\cos(\theta-\phi_w)}=0.542\left(\frac{g F_\theta}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}\right)^{0.23}$$

$$\frac{g t_{min}}{U_{10}\cos(\theta-\phi_w)}=30.1\left(\frac{g F_\theta}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}\right)^{0.77}$$

From these formulae the wave height was calculated to be 0.93 m and the wave period 3.1 s.

The Young and Verhagen method uses the formulae given in box 5.2.4.3.

Box 5.2.4.3 Excerpt from the Rock Manual (2007) for equation 4.92 and 4.93.

$$\frac{gH_s}{U_{10}^2}=0.241\left(\tanh A_1 \tanh\left(\frac{B_1}{\tanh A_1}\right)\right)^{0.87}$$

where: $A_1=0.493\left(\frac{gh}{U_{10}^2}\right)^{0.75}$ and $B_1=0.00313\left(\frac{gF}{U_{10}^2}\right)^{0.57}$.

$$\frac{gT_p}{U_{10}^2}=7.519\left(\tanh A_2 \tanh\left(\frac{B_2}{\tanh A_2}\right)\right)^{0.37}$$

where: $A_2=0.331\left(\frac{gh}{U_{10}^2}\right)^{1.01}$ and $B_2=0.0005215\left(\frac{gF}{U_{10}^2}\right)^{0.73}$.

From these formulae the wave height was calculated to be 0.57 m and the wave period 8.8 s. The wave height calculated by the Young and Verhagen method was somewhat low compared to the other answers and the wave period appeared to be incorrect.

5.2.5 Wave Run-Up Height

Wave run-up is defined in the SANCOLD Guidelines (1990) as the difference in vertical height from stillwater level to the maximum level attained by run-up of the design wave against the dam wall. The SANCOLD Guidelines (1990) provides a wave run-up ratio diagram to determine the wave run-up. The wave run-up ratio is a function of slope and texture of the upstream face of the dam. The wave run-up was determined as 1.11 m.

As in the previous section, the DCM-2 refers to the CEM and the Shore Protection Manual (SPM) for the calculation of wave run-up. The SPM takes the wave height, slope and roughness of the run-up surface as well as the incident angle of the wind to the slope surface into account when determining the wave run-up. Figures 7-8 through 7-13 in the SPM provides relationships between the ratio of H_0 / g * T_{02} and the embankment slope as well as a run-up correction factor for scale effects. All these figures are based on a smooth, impermeable run-up surface and therefore a surface roughness reduction factor of 0.55 was selected (Table 7-2, SPM). From these figures the wave run-up was determined as 0.83 m.

In the approach followed in the Rock Manual for the calculation of wave run-up, the wave run-up is expressed as a function of the breaker parameter, ξ;

$\xi = \tan\alpha / \sqrt{s_0}$

$$s_0 = \frac{2\pi}{g} \frac{H}{T^2}$$

The formula to calculate wave run-up provided in the Rock Manual, is a linear function of the breaker parameter. It gives the general relationship between the run-up level and the slope angle as well as the wave height and period;

$R_{u2\%} / H_s = A\xi + B$

where A and B are fitting coefficients.

The wave run-up for a smooth slope was calculated and multiplied with a roughness reduction factor of 0.7 (Armourstone – single layer on impermeable base, Table 5.2, Rock Manual 2007) to allow for the rough rip-rap surface. With wave height of 0.93 m and the wave period of 3.1 s, the wave run-up was calculated as 2.09 m.

The CEM gives a comprehensive description of wave run-up. The wave run-up depends on the height and steepness of the incident wave and its interaction with the preceding reflected wave, as well as the slope angle, the surface roughness and the permeability and porosity of the slope. As is the case with the Rock Manual, the wave run-up calculated in the CEM is also depended on the type of wave breaking and therefore the breaker parameter, ξ. The CEM also recommends the same formula as given in the Rock Manual, to calculate the wave run-up.

$R_{u2\%} / H_s = (A\xi + C)\gamma_r \gamma_b \gamma_h \gamma_\beta$

where $R_{u2\%}$ = run-up level be 2% of the incident waves
ξ = wave parameter
A,C = coefficients dependent on ξ
γ_r = reduction factor for influence of surface roughness
γ_b = reduction factor for influence of a berm
γ_h = reduction factor for influence of shallow water conditions
γ_β = factor for influence of angle on incidence β of the waves

The surface roughness reduction factors provided in the CEM is more comprehensive (Table VI-5-3, CEM 2006) and with a correction factor of 0.55, the wave run-up was calculated as 1.17 m.

The Overtopping Manual focuses on wave run-up and overtopping with emphasis on the overtopping. It is not a design manual but comprehensive references are made to the main manuals and guidelines such as the Rock Manual and CEM. The European Overtopping website provides an on-line calculation tool to calculate the overtopping rate. This tool was used to calculate the wave run-up, using the wave height and period for each of the different methods described under 2.5 in this chapter. The results calculated using the calculation tool, were significantly higher than those calculated using the other methods. A comparison table is provided below.

Table 5.2.1 Wave Run-Up Parameters

Description		SANCOLD (1990)	DCM-2	CEM	Rock Manual
Component	Unit	Value			
Wave Height	m	0.84	0.77	0.94	0.93
Wave Period	s	3.0	2.61	2.2	3.1
Wave Run-Up	m	1.11	0.83	1.10	1.97
Wave Run-Up “Overtopping calculation tool”	m	2.73	2.05	1.99	3.03

5.3 CONCLUSION

The different freeboard design parameters as determined in section 2 above are summarised in Table 5.3.1.

Table 5.3.1 Freeboard Design Parameters

Description		SANCOLD (1990)	DCM-2	CEM	Rock Manual	Kamphuis	Overtopping Manual
Component	Unit	Value					
Effective Fetch	km	1.66	2.41	2.41*	1.68	-	-
Design Wind Speed	m/s	28.2	30.6	30.6*	28.2*	28.2*	-
Wind Set-Up	m	0.08	0.04	0.04*	0.04	0.70	0.04*
Wave Height	m	0.84	0.77	0.94	0.93	-	0.94*
Wave Period	s	3.0	2.61	2.2	3.1	-	2.2*
Wave Run-Up	m	1.11	0.83	1.17	2.09	-	1.99
Required Freeboard	m	**1.19**	**0.87**	**1.21**	**2.13**	**-**	**2.03**

* Substituted Values – Not Calculated

There is a large variety in the methods used to calculate the effective fetch.

With design wind speed, it is important to use the most comprehensive data available when determining the design wind speed. By making use of updated weather information the accuracy of the freeboard determination can be significantly improved.

The different publications all use more or less the same method in calculating the wind set-up, however, the SANCOLD Guidelines (1990) recommends using a safety factor by doubling the fetch. The spreadsheet provided by Kamphuis for the calculation of incremental wind set-up, results in an extremely high value. It is a product of adding the incremental wind set-up of each of the segments together that makes the final wind set-up becomes significantly higher. The method proposed by Kamphuis also illustrates the effect of water depth on wind set-up.

Table 5.3.2 Comparison between required Freeboard

Description		SANCOLD (1990)	DCM-2	CEM	Rock Manual	Overtopping Manual
Component	Unit	Values				
Required Freeboard as calculated in Table 3.1	m	1.19	0.87	1.21	2.13	2.03
Required Freeboard with wind set-up of Kamphuis	m	1.81	1.52	1.87	2.79	2.69

Table 5.3.2 shows the impact on the required freeboard when using the method of Kamphuis to calculate the wind set-up. It is therefore important to take special note of the significant effect of water depth on wind set-up, to not underestimate the total freeboard required.

Comprehensive information is available with regard to wave height and wave period as well as wave run-up. The CEM and Rock Manual consider an exceedance level when calculating the maximum wave run-up. The most common irregular wave run-up parameter is calculated for the 2% exceedance wave height. The calculation tool provided as part of the European Overtopping Manual calculates significantly higher wave run-up, than any of the other methods.

It is recommended that if an analytical approach is followed, the methodology of the Rock Manual should be followed.

6. MATHEMATICAL MODELLING OF WIND WAVE HEIGHT USING SWAN

6.1 BACKGROUND

Determination of required freeboard for a dam can be performed using traditional methods, published engineering manual or guidelines/standards from appropriate institutions. In recent years computer programmes have been developed to allow for faster computations. In this study a numerical wave model **S**imulating **WA**ves **N**ear **S**hore (**SWAN**), from Delft University of Technology (Booij et.al, 2004) is adopted for computation of wave parameters on inland reservoirs. The Berg River Dam was used as one of the case studies.

Wind Characteristic over Reservoirs

Freeboard protects dams and embankments from overflow caused by other means including wind induced tides and waves. For dam engineering applications and estimation of required freeboard, the following basic considerations are important for wind induced waves:

- In mountainous regions, the flow of air is influenced by topography and meteorological factors. Hence, estimation of wave action in mountainous regions requires specific attention.

- If wind velocity over a particular fetch remains constant, wave height will progressively increase until a limiting maximum value is attained. The height is dependent on fetch distance, wind velocity and duration.

- Wind Velocities over water are higher than over land surfaces because of smoother and more uniform surface conditions, hence the adjustment of the wind speed.

- When wind blow over water surface, it exerts a horizontal stress on water, driving it in the direction of wind. In an enclosed body of water, this wind effect results in a piling up of water at the leeward end, and a lowering of water level at the windward end. This effect is called “wind set-up”.

- The characteristic wind-generated waves are influenced by the distance that wind moves over the water surface in the fetch direction.

- Wave characteristics are considered to be shallow when depths in the wave-generating area are generally less than half the theoretical deep-water wavelength (L_o) corresponding to the same period (T).

6.2 LINEAR WAVE THEORY

When ocean waves enter coastal waters, their amplitude and direction will be affected by the limited water depth. A parallel can be drawn between ocean waves approaching the coast and waves in an inland reservoir approaching the dam wall or the banks.

In this section, some of the scientific principles governing the transformation of waves from deep water to shallow will be briefly explained. The processes that can affect the wave characteristics as it propagates from deep into shallow water include but are not limited to the following:

a) Shoaling
b) Refraction
c) Diffraction
d) Reflection

a) Shoaling

Shoaling is the variation of waves in their direction of propagation due to depth-induced changes of the group velocity in that direction. These changes in group velocity generally increase the wave amplitude as the wave propagates into shallower water (the propagation of wave energy slows down, resulting in energy bunching).

b) Refraction

Refraction is the turning of waves towards shallower water due to depth or current-induced changes of the phase. The crest moves faster in deeper water than it does in shallow water so that in a given time interval, the crest moves over larger distance in deeper water than it does in shallower water. The effect is that the waves turn towards the region with shallower water, i.e. towards the coast.

c) Diffraction

To introduce the phenomenon of diffraction, consider a wave travelling in water of constant depth, around a headland or break water. In the absence of refraction (since the bottom is

horizontal), the waves will travel into the shallow obstacle in a circular pattern of crests with diminishing amplitudes.

d) Reflection

Waves that propagate into a solid object such as breakwater, a dam-wall, a cliff or a sloping beach may reflect. A vertical cliff may well reflect 100% of the incoming wave energy, whereas a gentle beach will barely reflect energy. Reflection is often ignored, particularly near sandy beach like coast where it is often deemed insignificant. However, in the case of inland dam design, this cannot be ignored.

One of the most notable phenomenon of reflecting waves is the standing waves. To introduce this phenomenon, consider a one-dimensional situation with long-crested wave at normal incidence, reflecting off a vertical wall. The resulting profile is the summation of two waves: the incident wave, propagating towards the wall and the reflected wave propagating away from the wall.

6.3 THE SWAN WAVE MODEL

Simulating WAves Nearshore (SWAN) is a numerical model used to obtain realistic estimates of wave parameters for a given wind, current and bottom conditions. In order to accurately predict the propagation of ocean waves a bathymetry grid is required of the area of interest. The basic equation of SWAN is the spectral action balance equation. The equation is as shown below:

$$\underset{A}{\frac{\partial}{\partial t} N} + \underset{B}{\frac{\partial}{\partial x} C_x N} + \underset{C}{\frac{\partial}{\partial y} C_y N} + \underset{D}{\frac{\partial}{\partial \sigma} C_\sigma N} + \underset{E}{\frac{\partial}{\partial \Theta} C_\Theta N} = \underset{F}{\frac{S}{\sigma}}$$

Where:

A Represents the local rate of change of action density with time

B & A Represent propagation of action (or energy) density in geographical space, (with propagation velocity $C_{g,x}$ and $C_{g,y}$ in x- and y-space respectively, thus accounting for shoaling

D Represents the change/shifting in relative frequency due to variation in depth & current (with propagation velocity C_σ in σ-space)

E Represents depth induced and current induced refraction (with propagation velocity C_Θ in Θ space; diffraction is optionally included)

F Is the source term and it represents the effects of generation, non-linear wave-wave interaction and dissipation.

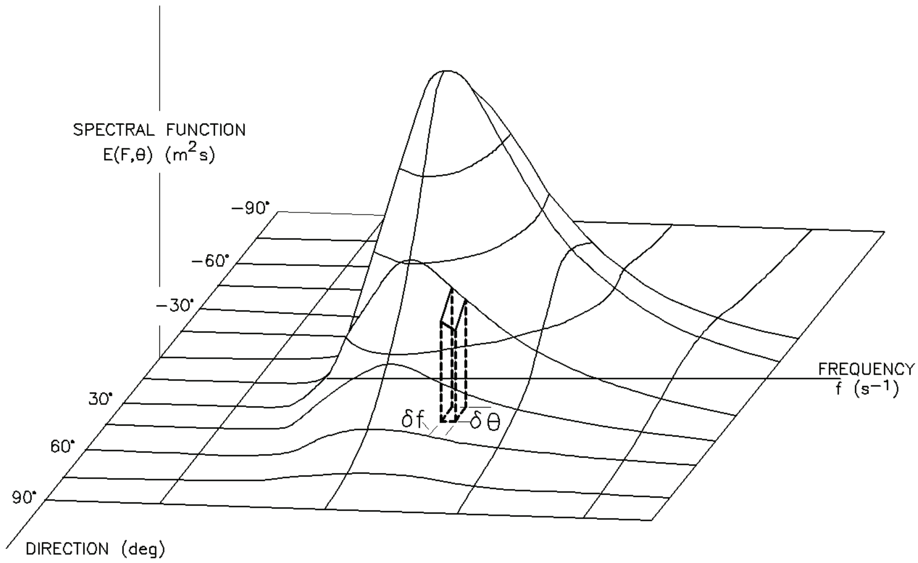


Fig 6.3-1 The two-dimensional spectrum of wind generated waves

In SWAN energy is added to the system by means of wind action and is removed from the system in three forms of dissipation. White Capping, bottom friction and depth induced breaking.

Generation by wind

When wind starts to blow over still water: the first waves to appear a small and very short, slowly getting longer and higher. When initial waves have been generated, they modify the airflow and hence the wind induces pressure at the water surface in such a way that they enhance their own growth. Therefore waves are generated by induced surface *pressure* not by wind induced surface *friction*.

In his theoretical model, Miles (1957) finds that the air pressure at the surface attains a maximum on the windward side of wave crest and a minimum on the leeward side of the wave crest. This implies that wind effectively pushes the water surface down where the wave surface is moving down and pulls the water surface up where the surface is moving up. This out of phase coupling between pressure and surface motion transfers energy to the waves. This transfer depends on the amplitude of the water wave, it becomes more effective as the wave evolves.

In summary, the source term for the generation of waves by wind can be expressed as follows:

$$S_{in}\ (f,\Theta) = \alpha + \beta\ E\ (f,\Theta)$$

Where α and β depend on wave frequency, wind speed and their respective direction. α is the linear growth term and $\beta\ E$ represents exponential growth term.

Dissipation

Dissipation is represented in SWAN by White-capping, bottom friction and bottom induced breaking.

White-capping

Wave breaking in deep water (called white-capping) is a very complicated phenomenon. Some speculation as to what controls wave breaking have been made and it seems reasonable to assume that it is the wave steepness. According to Hasselmann theory, the weight of white-cap acts against the rising water surface, thus draining the energy from the wave.

The theory of Hasselmann (1974) gives only a general form for the white-capping source term as:

$$S_{wc}\ (f,\Theta) = -\mu\ k\ E(f,\Theta)$$

Where k is the wave number and μ is a coefficient representing statistical property of white-caps.

Bottom Friction

Bottom friction is the dominant bottom dissipation mechanism for continental shelf seas with sandy bottoms. The corresponding source term may generally be represented as

$$S_{bfr}\ (\sigma,\ \Theta) = \frac{-\ C_{bfr}}{g} * [\ \sigma/\mathrm{Sinh}\ (kd)]^2 * E\ (\sigma\ ,\Theta)\ u_{rms,bottom}$$

Depth-induced (surf-) breaking

Wave is limited by both depth and wavelength. For a given water depth and wave period, there is a maximum height above which the wave become unstable and breaks. The maximum possible wave height in the local water depth H_{max} is determined from

$H_{max} = \gamma * D$,

Where D is the total water depth including the wave-induced set-up, and is the breaker parameter, with a default value of 0.73 in SWAN.

Reflection, transmission and absorption

To accommodate situations with line structures such as breakwaters, SWAN can reflect wave energy off and transmit wave energy through or over such structure. Such obstacles can affect the wave field in two ways, first it will reduce the wave height locally along its length and second it will cause diffraction around its ends.

Wave Induced Set-up

The gradients in the wave-induced radiation stresses, which generally cause currents and set-up are standard output of SWAN. The results of such computations can be returned to SWAN as input to achieve the feedback between wave on one hand and set-up and currents on the other. In one dimensional situations, the computation of the wave induced set-up in SWAN is based on the vertically integrated momentum balance equation, which represents a balance between the wave induced force and the vertically integrated hydrodynamic pressure gradient.

$$\frac{dS_{xx}}{dx} + \rho g h * \frac{d\acute{\eta}}{dx} = 0$$

Where H = d+ ή is the total water depth (including wave induced set-up), d is the bottom level and ή is the mean surface elevation.

6.4 CASE STUDY – BERG RIVER DAM (BRD)

In this report only wind related components contributing to determine dam freeboard has been addressed. The wind generated waves, and the corresponding wave run-up and wind set-up were calculated in accordance with SANCOLD Report No 3, Guidelines on Freeboard for Dams (SANCOLD, 1990). These parameters were also computed using the SWAN wave model and the preliminary results of the computer generated parameters were compared with the traditional hand calculation methods.

Calculation of BRD Wave Parameters using Traditional Methods

Design Wind Speed

Fig. A1 in SANCOLD (1990) , gives the hourly mean design wind speed for a 50 year return period = 25 m/s

In the BWP Tender Design Report, the effective fetch (F) was given as 2.15 km

The wind speed was increased by a factor of 1.16 to cater for higher speed over water than over land.
Therefore,

The Design Wind Speed (V) = 25 x 1.16
= 29 m/s

Design Wave Height

From SANCOLD (1990); Figures A3, A4 and A5 (smooth slope) for a wind speed of 29 m/s over water and effective fetch of 2.15 km

Wind Wave Period (T) = 3.0 s

Significant wave height (Hs) = 1.0 m

Wave Run-up ratio = 2.4 for dams max ratio is take as 1.5

Therefore, for rockfill dam with road on crest the design wave height in terms of significant wave height is multiplied by a factor of 1.0.

The Design Wave Height = 1.0 m

The wave length for deep water (L) **= $1.56T^2$**
= 14 m

Wind Set-up

From Saville (SANCOLD, 1990),
Wind Set-up (S) $= \frac{V^2F}{4850D}$
= 0.041

Where S is the rise above still water level (m)
V is the design wind speed (m/s)
F is effective fetch (km), normally equal to 2 times effective fetch
D is the average water depth in the basin (m) along the fetch, F

Site Specific Conditions

It is well known that in the south-easterly winds in the Western Cape accelerates significantly as they fall over the mountain. The funnelling effect of the topography will increase the 34 m/s velocities (mean hourly average) recorded at Theewaterskloof Dam significantly.

Winds speed as high as 40 m/s have been record at Berg River Dam site weather station. This specific case can lead to a wind set-up = **0.**12 m at the dam wall.

Calculation of Berg River Dam Wave Parameters using SWAN

A 50 mx50 m grid file with elevation at the nodes was prepared. The software “Surfer” was used to convert the information into a contour map as shown below.

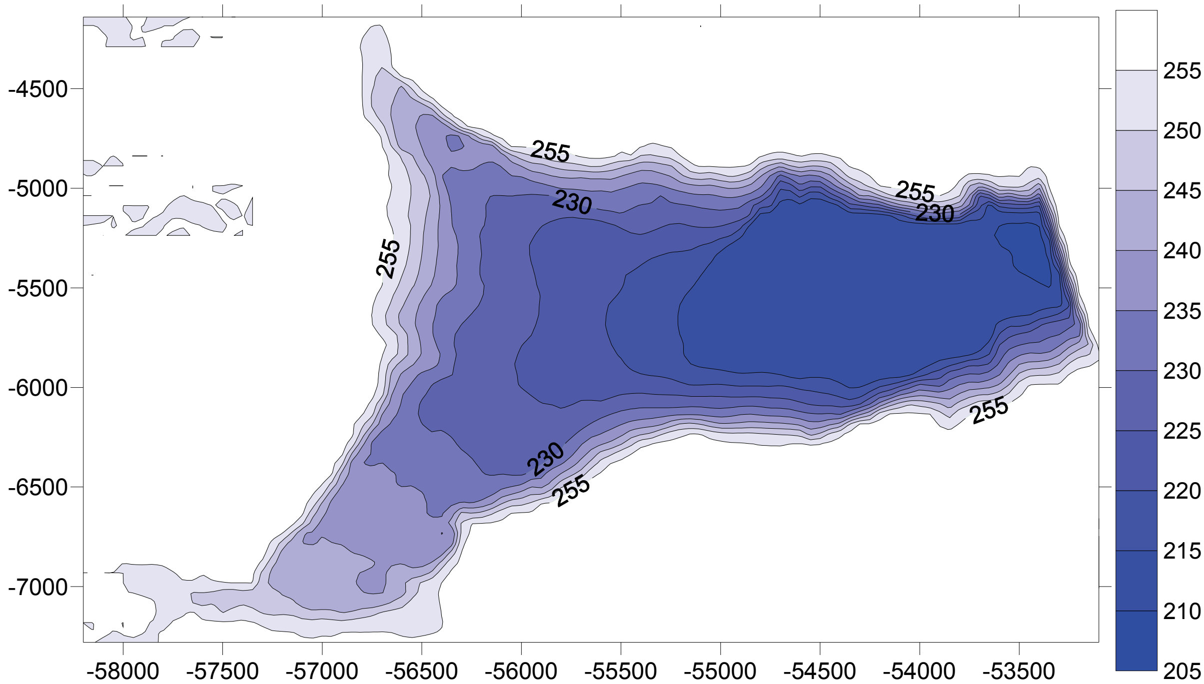


Figure 6.4-1 Berg River Dam bathymetry

A SWAN code file was prepared and several input conditions were changed to check the effects on the output data. [elevation contours are presented in metre above mean sea level]

Typical SWAN Code File

The Programming Code of SWAN can be found in Appendix C of the SWAN user manual. The code has extensive functionality, the customised guideline will only focus on basic instructions applicable to inland reservoir designs.

```
$*************HEADING****************************************
PROJ 'BergRun' '2'
$

COORD CART
$***********MODEL INPUT********************************** ***
$
CGRID REG -58100 -7300 0.0 5100.0 3200.0 102 64 CIR 72 0.033 1.0 40

$
INPGRID BOTTOM REG -58100 -7300 0.0 102 64 50.0 50.0
READINP BOTTOM 1.0 'Grid3.dat'  4 0 FREE

$WIND
WIND 40 0

$WAVES
INI ZERO

$OBSTACLES
OBST DAM 5.0 2.6 0.15

$WIND SETUP
SETUP

$DIFFRAC 1
NUM ACCUR 0.02 0.02 0.0  90. 20
$
$OFF BREA
$OFF WCAP
$OFF QUAD
$
$************ OUTPUT REQUESTS **************************

GROUP 'Table1' 0 102 0 64
$OUT
Table 'Table1' HEAD 'BergOut.dat' XP YP DEP HS PDIR RTP DSPR SETUP

$
TEST 0,0
COMPUTE
STOP
```

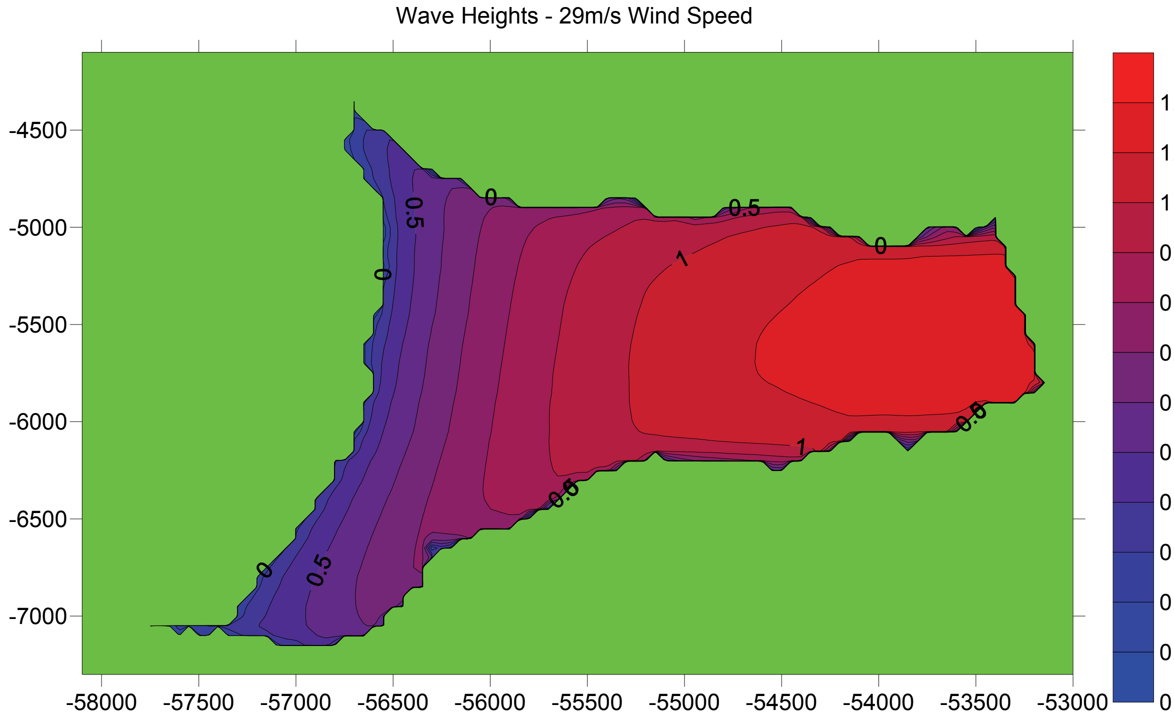


Figure 6.4-2 Berg River Dam simulated wave heights at wind speed 29 m/s

Output data: Wave Period (Tp) 1.08-3.59 Sec, Wave Height (Hs) = 1.18 m max,
Wave Setup = 0.003 m at the dam wall

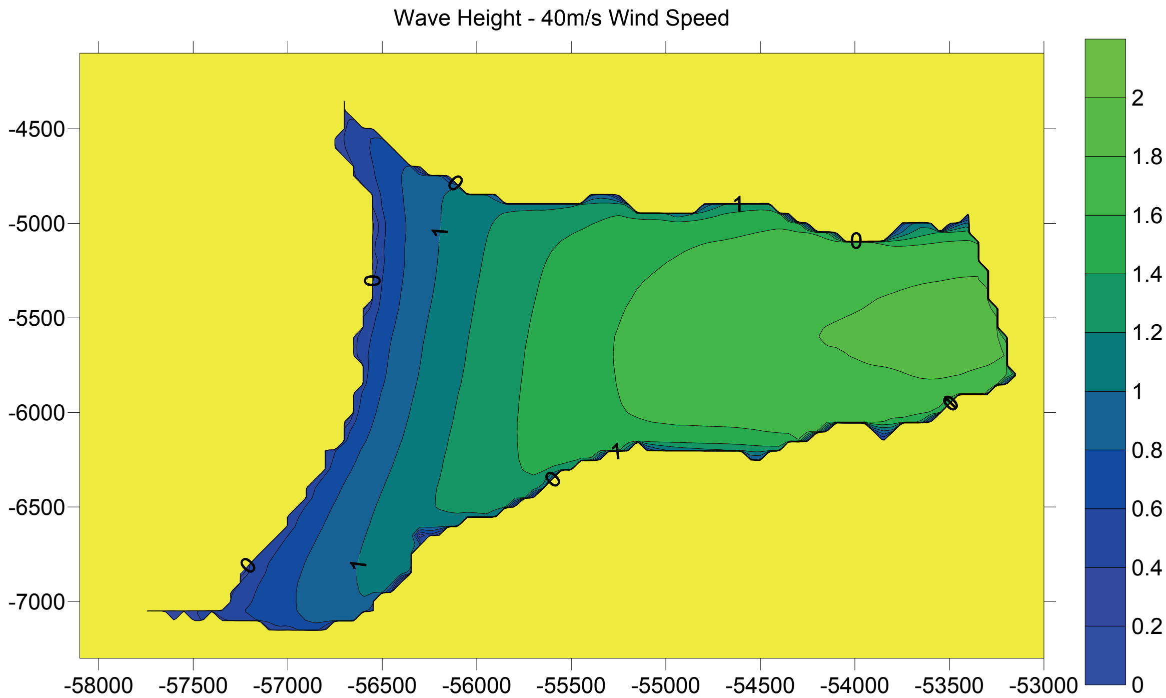


Figure 6.4-3 Berg River Dam simulated wave heights at wind speed 40 m/s

Output data: Period 2.15 – 4.64 Sec, Wave Height = 2.0 m max, Setup = 0.01

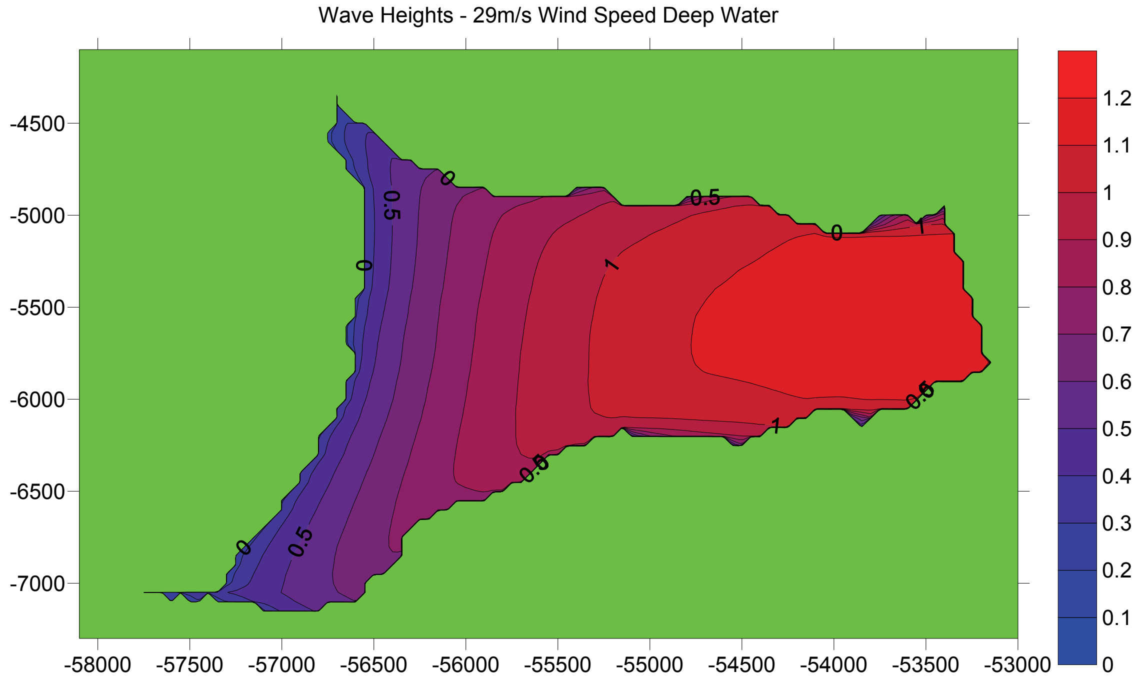


Figure 6.4-4 Berg River Dam simulated wave heights at wind speed 29 m/s

Output data: Period 2.15 – 3.59 Sec, Wave Height = 1.20 m max, Setup = 0.0003

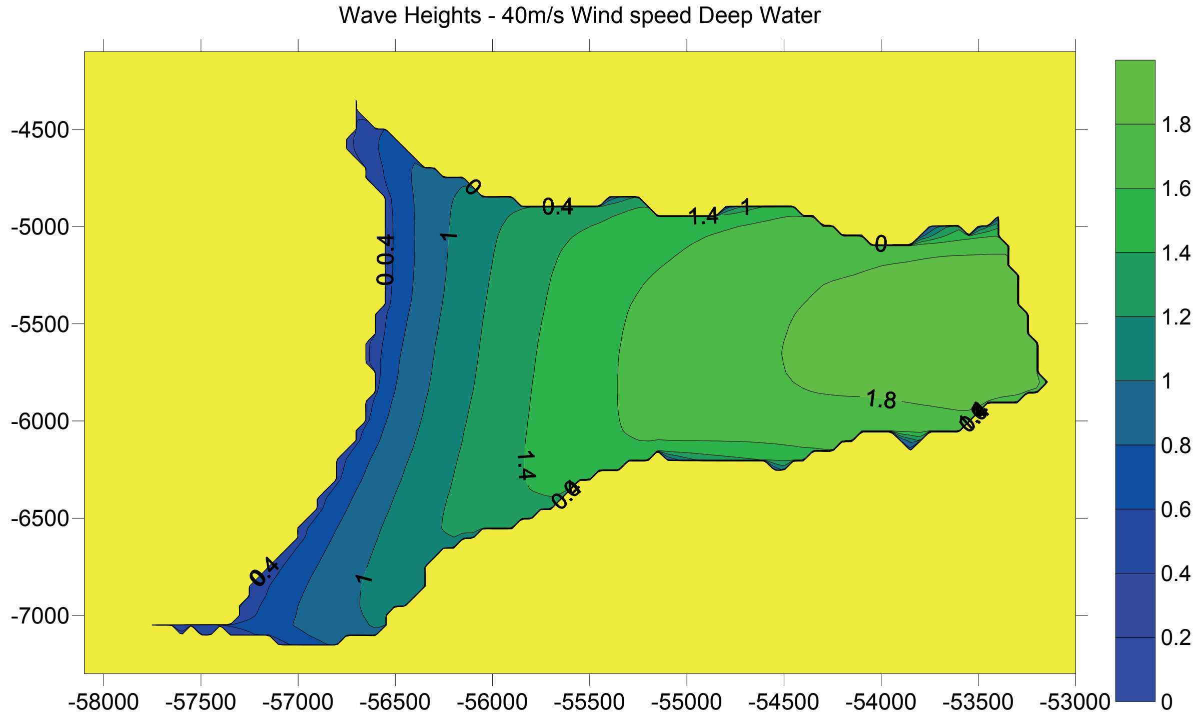


Figure 6.4-5 Berg River Dam (deep water) at 40 m/s wind speed

Output data: Period 2.15 – 4.64 Sec, Wave Height = 1.90 m max, Setup = 0.0006 m

6.5 BERG RIVER DAM SWAN SIMULATION RESULTS AND OBSERVATIONS

The following findings have been deduced from the output data.

- The wave heights (significant wave height, Hs) and wave periods (peak wave period, Tp) computed from the model were of the same order as those calculated using analytical methods. Thus confirming that SWAN can be used for computation of waves generated by wind in the dam freeboard design.

- Increase in water depth did not have significant impact on the wave height or period. This shows that the wind induced waves are limited to the upper layers of the water surface.

- Even though the computed wave setup at the dam wall was out by a factor of 10 from the values computed using analytical methods, the general trend was such that water was piling up at the leeward end and lowering at the wind ward end.

- It was also noted that at high wind speed (40 m/s) the extent of higher waves was reduced on the abutment for shallow depths due to the wave breaking effect. This phenomenon was not that apparent for deep water (150 m) cases.

- The wave setup was significantly reduced with increases water depth.

6.6 VOËLVLEI AND BLOEMHOF DAM CASE STUDIES

Two additional case studies have been included in this report: Voëlvlei and Bloemhof Dams. Voëlvlei Dam has a short fetch and is relatively deep. Bloemhof Dam has a long fetch and is relatively shallow. This makes these two dams ideal to indicate under what conditions certain methods work. The bathymetries of the two reservoir are shown in Figures 6.6-1 and 6.6-2.

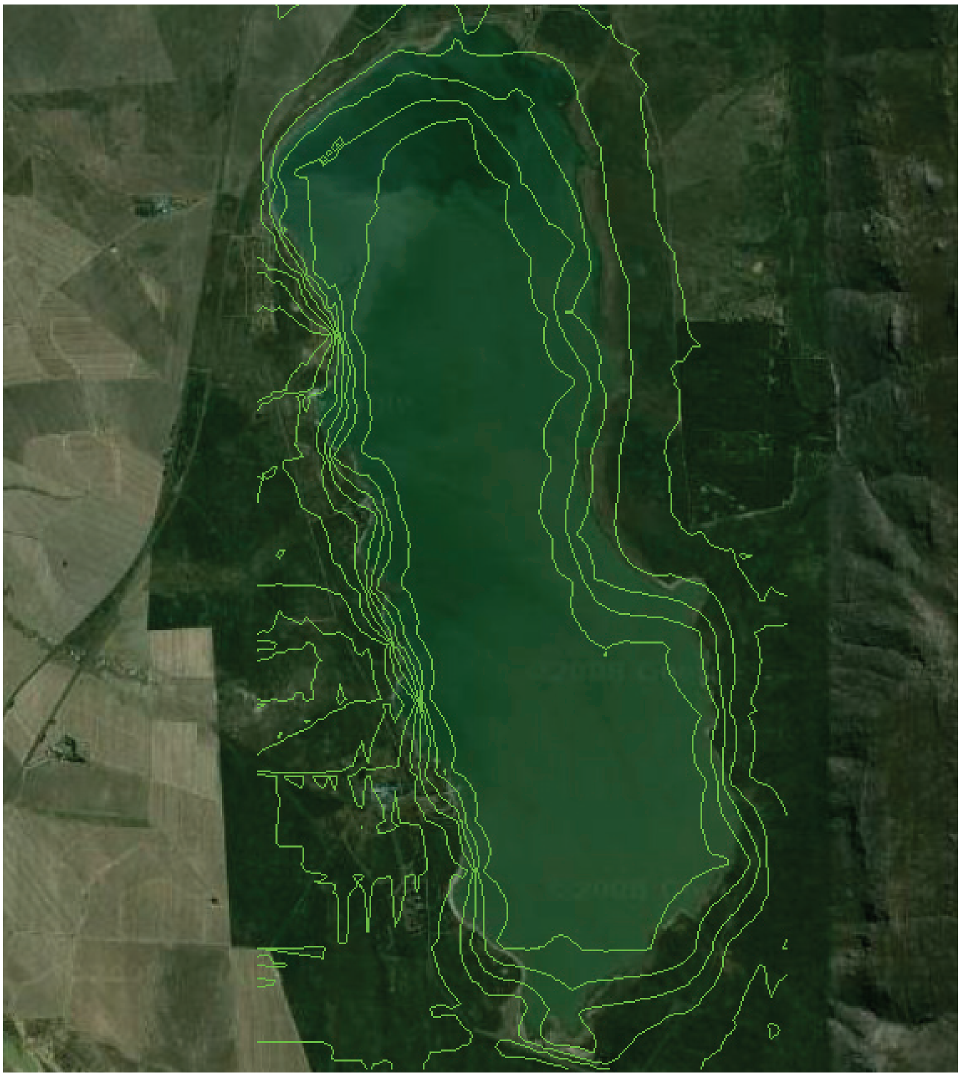


Figure 6.6-1 Contour map over satellite image of Voëlvlei Dam (Google maps, 13-08-08, [maps.google.com])

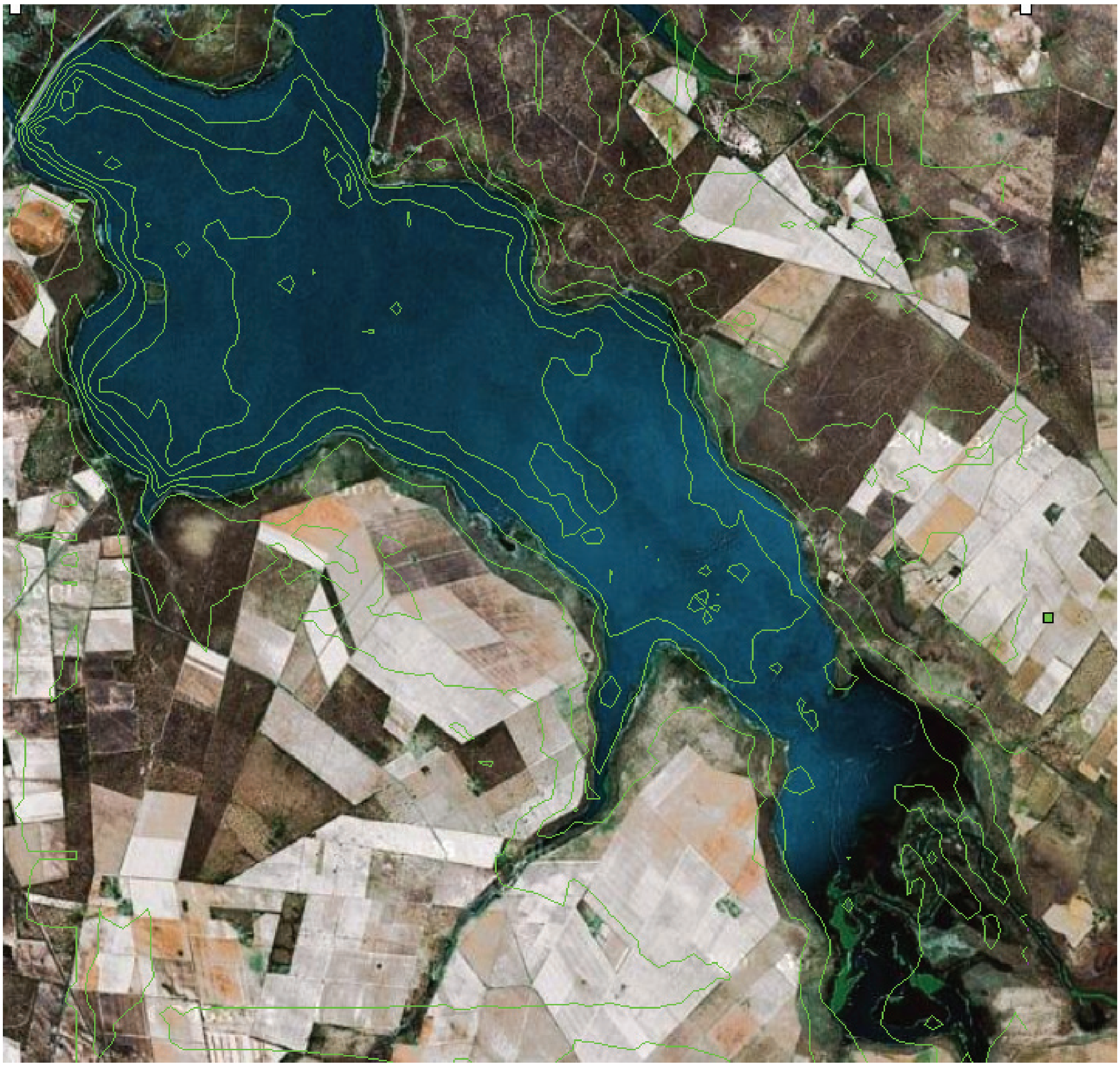


Figure 6.6-2 Contour map over satellite image of Bloemhof Dam (Google maps, 9-10-08, [maps.google.com])

A short user manual for SWAN has been developed and is attached in Appendices C of Volume II. The software used with SWAN to generate contours and to represent results was SURFER (see Appendix D of Volume II). A spreadsheet to set up the input file for SWAN with case studies is enclosed on the CD in Appendix E of Volume II. Appendix D (Volume II) also contains an Excel sheet with empirical methods to determine wave height and period.

SWAN simulated wave height and direction results of Voëlvlei Dam and Bloemhof Dam for different wind speeds are indicated in Figures 6.6-3 and 6.6-4 respectively.

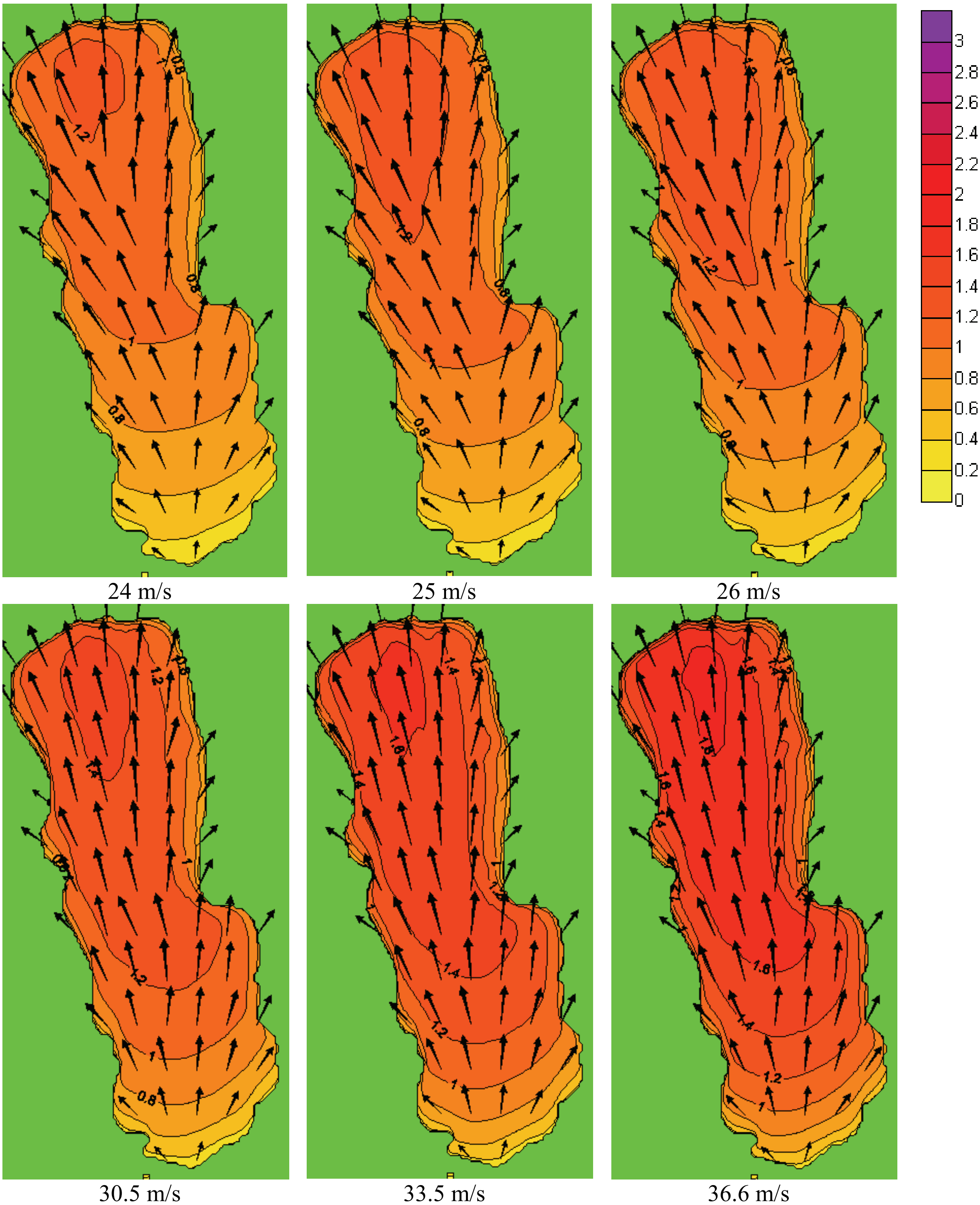
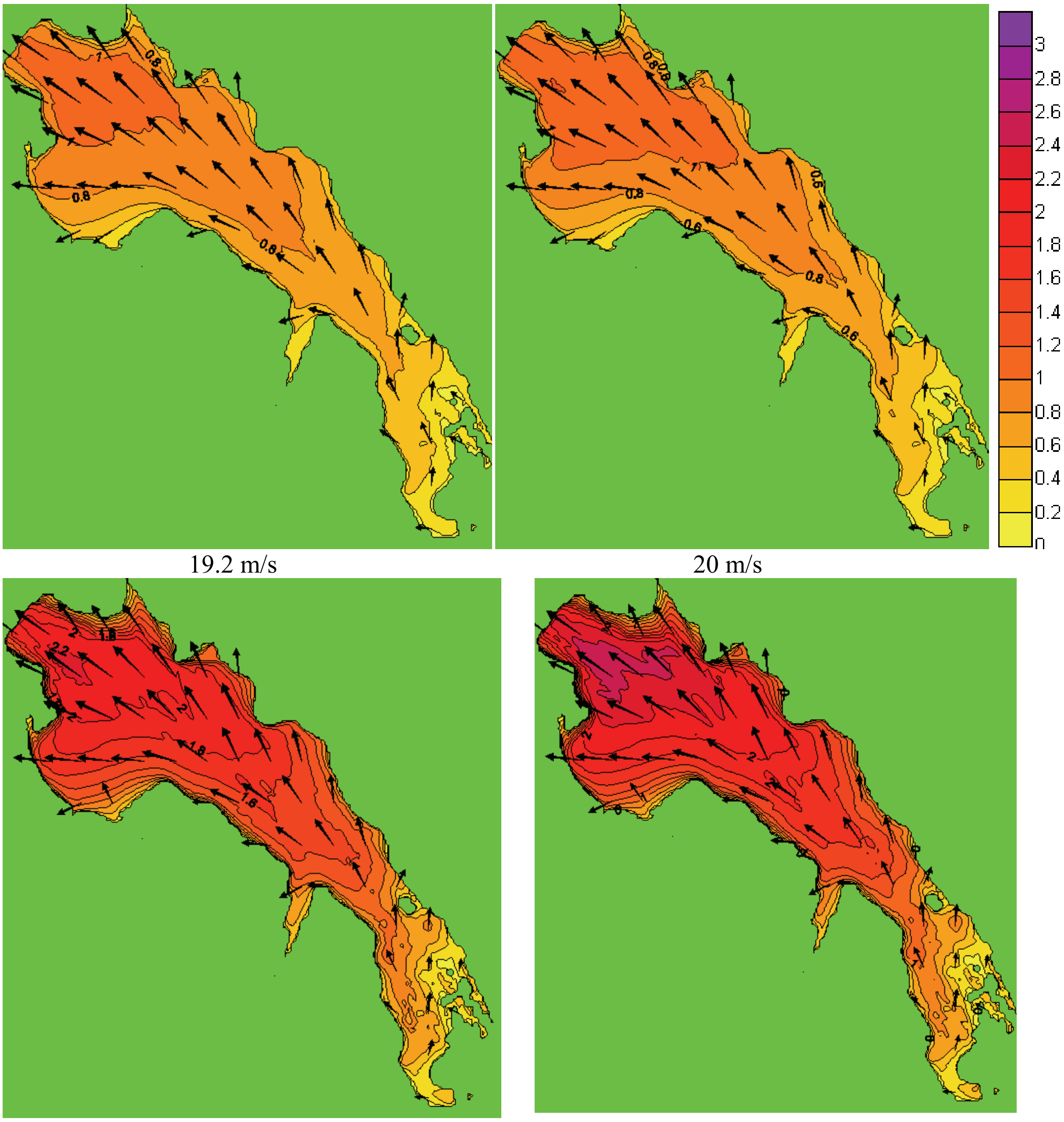


Figure 6.6-3 Voëlvlei Dam: Simulated wave heights with wind in direction of longest straight line fetch



19.2 m/s　　20 m/s

33.5 m/s　　36.6 m/s

Figure 6.6-4 Bloemhof Dam: Simulated wave heights with wind in direction of longest straight line fetch

Discussion of Results for Voëlvlei and Bloemhof Dams

The SWAN simulation results were compared with analytical calculations (Figures 6.6-5 and 6.6-6). The analytical methods gave estimated wave heights very similar to that produced by SWAN in the Voëlvlei Dam. Voëlvlei Dam has a very straightforward layout. There are no obstructions in the reservoir that will hinder wind induced wave height calculations.

SANCOLD's method (1990) gave the highest significant wave height at all wind speeds and fetch lengths. SANCOLD (1990) seems to be too conservative (too high) in its estimations. The CEM's method estimated wave heights that did not differ that much from that of SWAN in Voëlvlei Dam.

In Bloemhof Dam all the analytical methods used gave wave heights far exceeding that estimated by SWAN. Bloemhof Dam is a very long and narrow dam. The wave energy is dissipated along the shores of the dam in the narrow stretches. SWAN includes that energy loss in its estimation while the analytical methods do not.

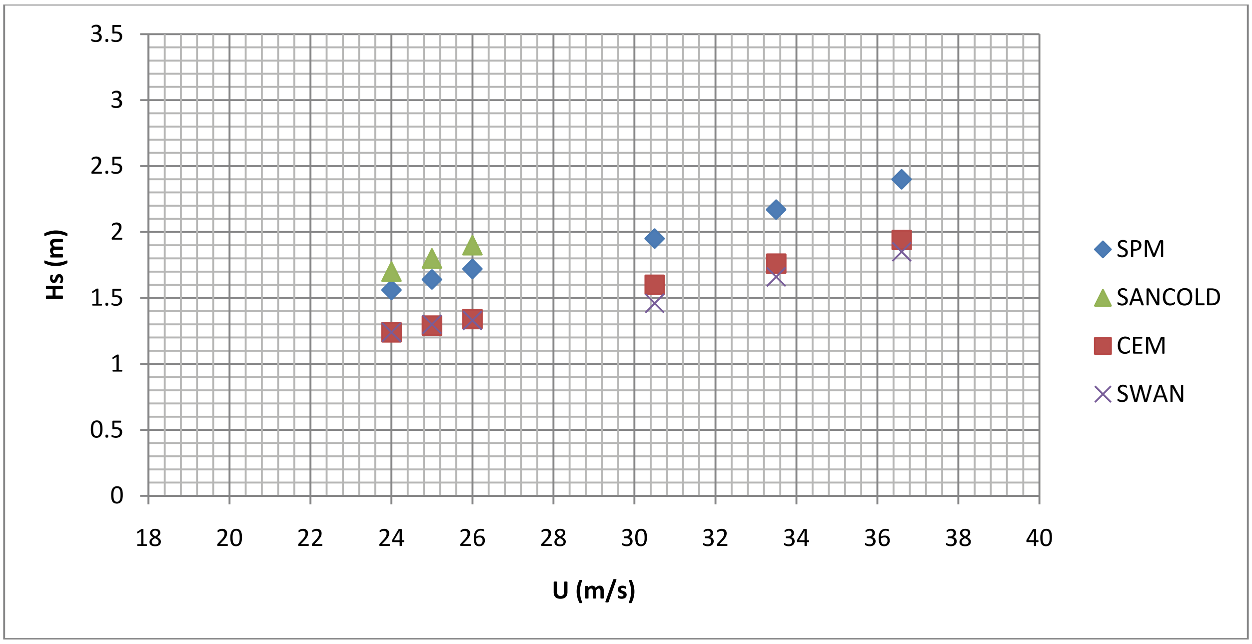


Figure 6.6-5 Significant wave height vs. wind speeds. Voëlvlei Dam

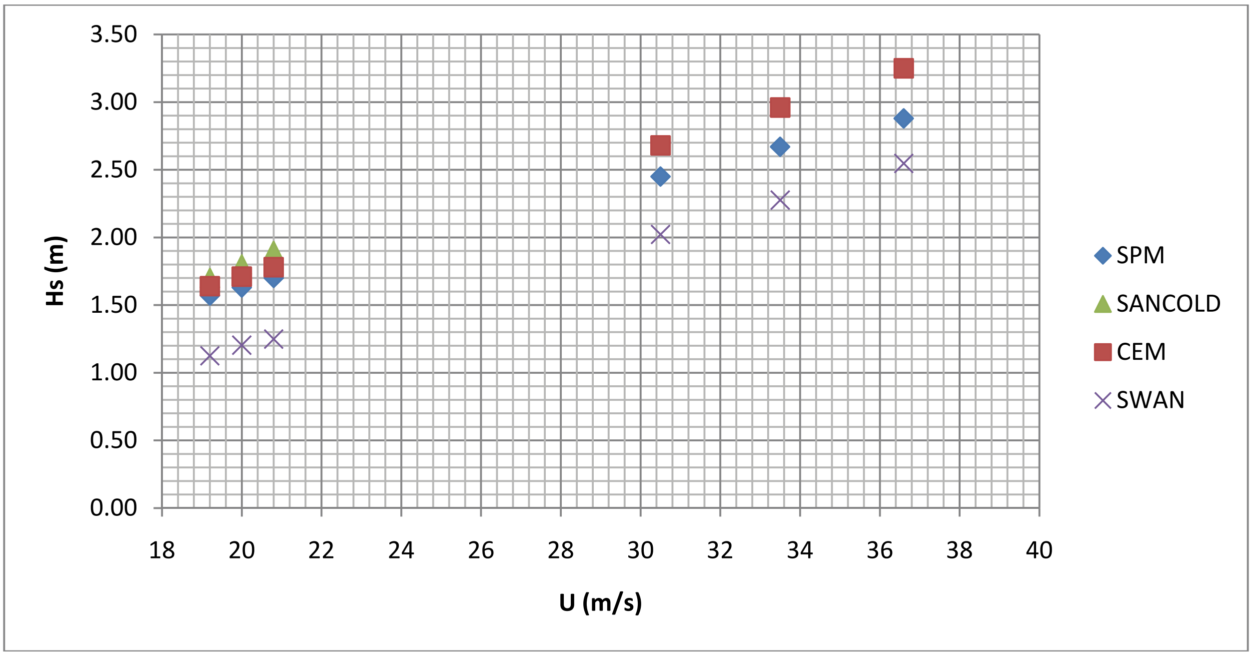


Figure 6.6-6 Significant wave height vs. wind speeds. Bloemhof Dam

7. UNSTEADY FREEBOARD COMPONENT PREDICTION FOR RESERVOIR SEICHES, SURGES, SET-UP USING HYDRODYNAMIC MODELLING

7.1 BACKGROUND

For category II and III dams it is proposed that hydrodynamic mathematical models (1D to 3D) are used to simulate the unsteady flow patterns in a reservoir.

The following were simulated in a one dimensional model Mike 11 and are evaluated in this section:

- River flood inflow surge wave or oscillating effects
- Low pressure oscillations
- Wind setup
- Dam outflow adjustment

7.2 SURGE AND OSCILATION EFFECTS DUE TO FLOOD INFLOWS

A simple reservoir model setup with about 2 km reservoir length and 100 m width was used as initial condition. The initial reservoir water level is shown in Figure 7.2-1 and a typical cross-section in Figure 7.2-2. An inflow hydrograph (Fig 7.2-3) was selected rising from 0 to 100 m^3/s in only 20 minutes. The simulated water level at the dam responds to the flood by rising rapidly as the reservoir fills and due to the flood surge and the water mass starts to oscillate but dampens out over time. Oscillation at the dam is shown in Figure 7.2-4. The water level drops slowly after the flood as the reservoir empties slowly through a bottom outlet.

In another scenario with a hydrograph falling stage duration of 80 minutes (Figure 7.2-5), the simulated oscillations at the dam is as indicated in Figure 7.2-6. The oscillations for the two scenarios are very similar.

INFLOW

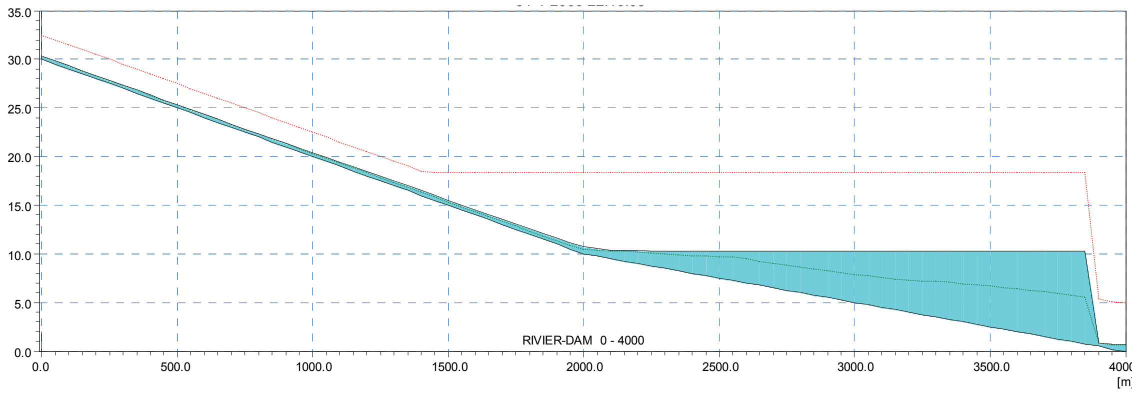


Figure 7.2-1 Longitudinal section of initial condition in reservoir

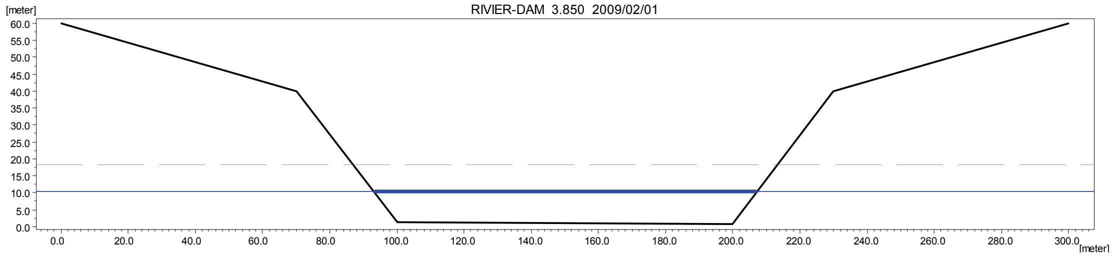


Figure 7.2-2 Typical cross section of reservoir

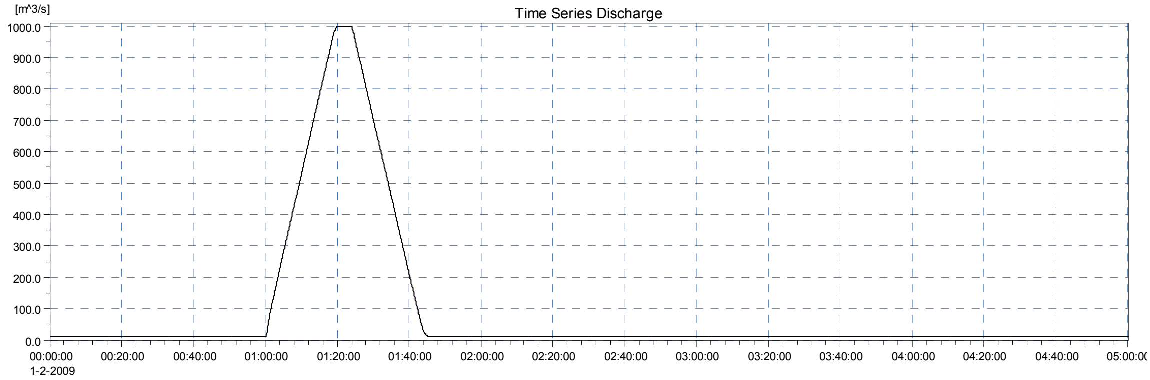


Figure 7.2-3 Inflow hydrograph 20 min rising and falling stages

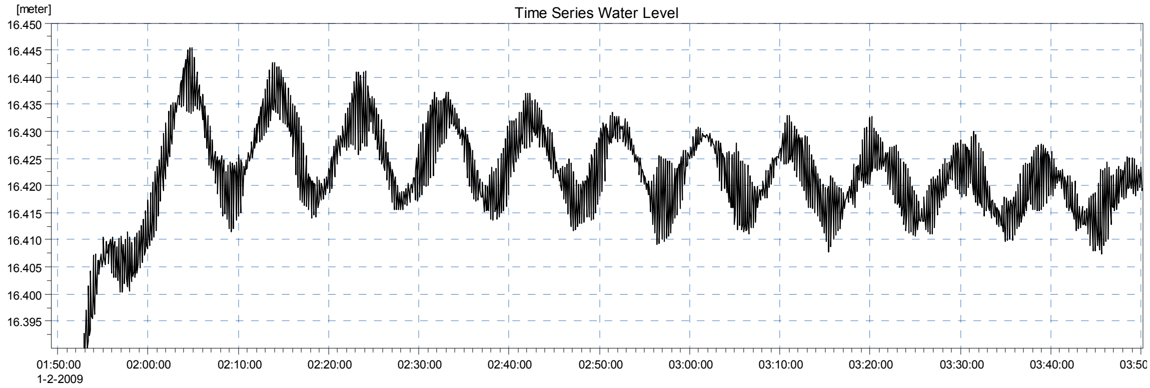


Figure 7.2-4 Oscillation water levels at dam with 20 min falling stage hydrograph

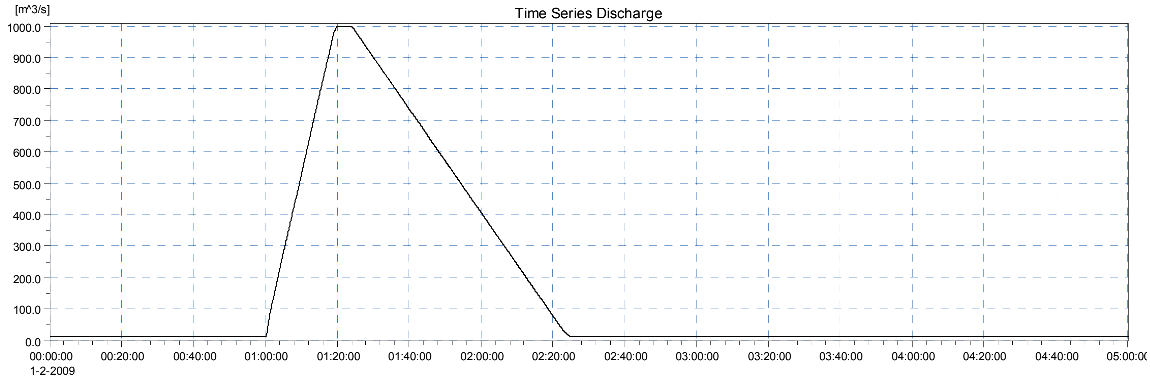


Figure 7.2-5 Inflow hydrograph 60 min falling stage

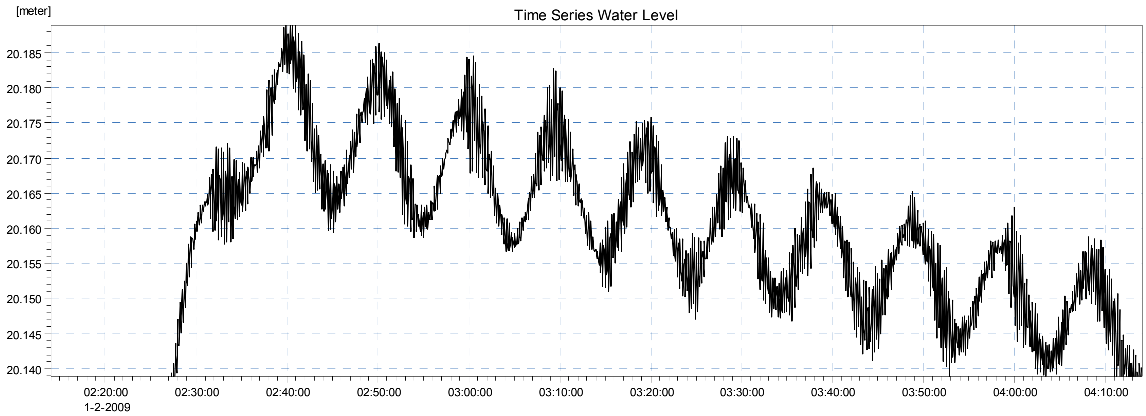


Figure 7.2-6 Simulated oscillating water level at dam with 60 min falling stage hydrograph

7.3 LOW ATMOSPHERIC PRESSURE OSCILLATIONS

On a large reservoir, low atmospheric pressure could raise the water level by as much as 0.3 m, which could lead to oscillations in the reservoir. In a scenario of a reservoir 50 km in length, the simulated oscillations that could be experienced at the dam is shown in Figures 7.3-1 to 7.3-5. The oscillations could last for several days.

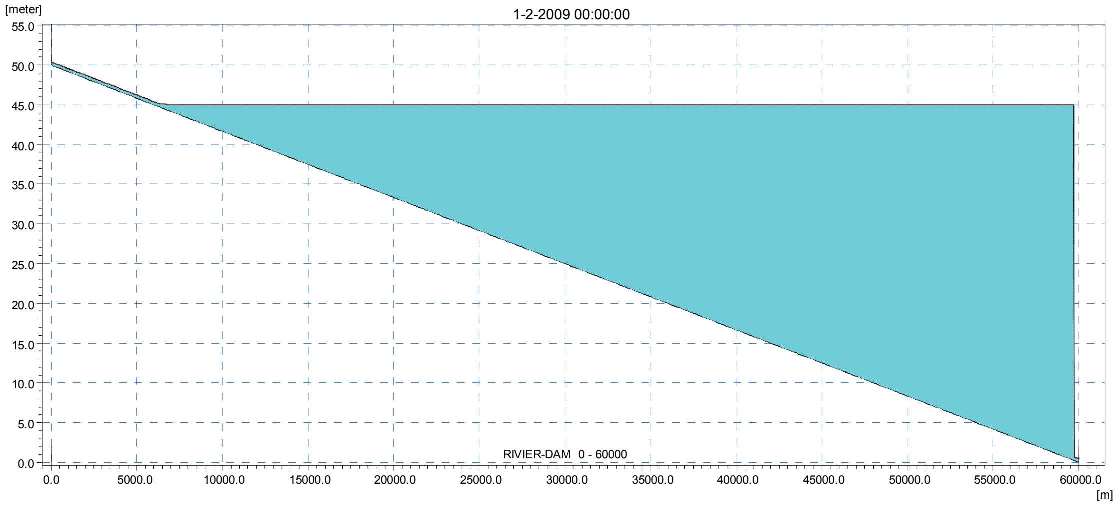


Figure 7.3-1 Longitudinal profile of 54 km dam + 5 km upstream river with base flow of 5 m^3/s (slope 1:1200)

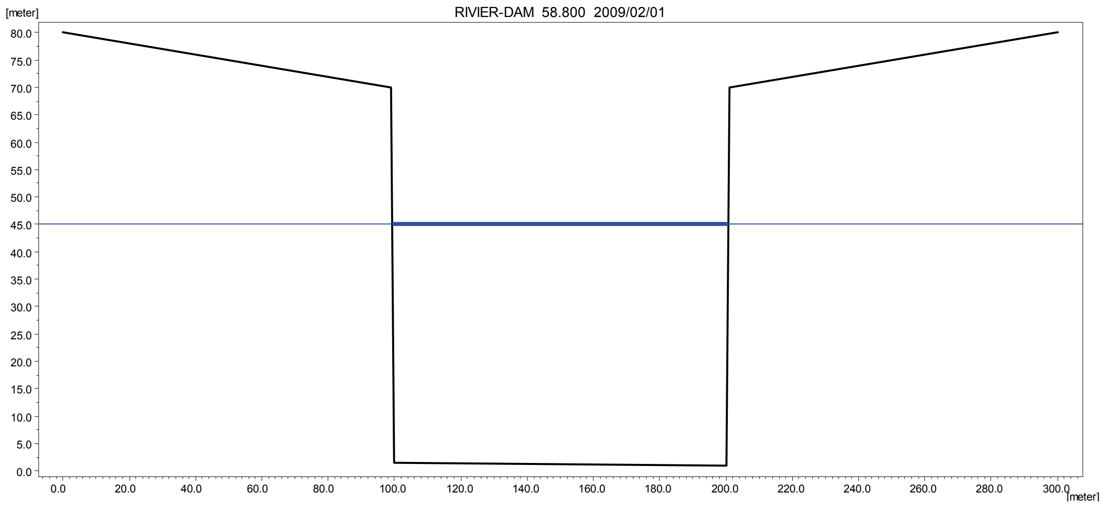


Figure 7.3-2 Cross section of 54 km length reservoir

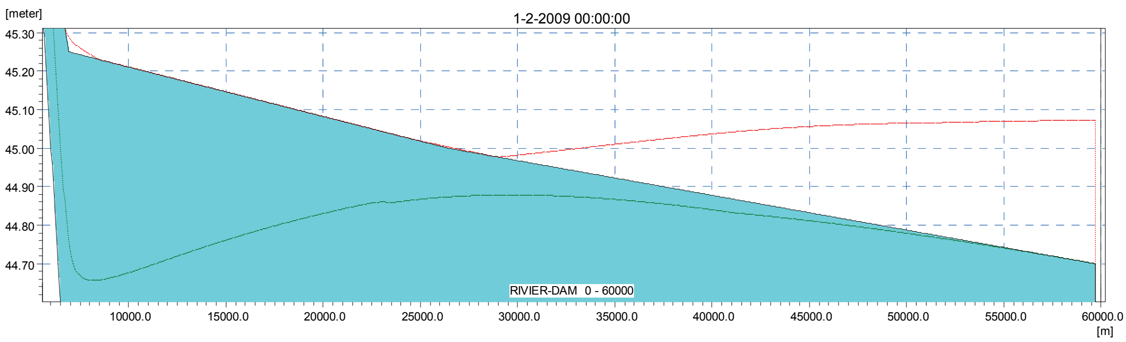


Figure 7.3-3 Profile view of 600 mm pressure difference on dam, starting position

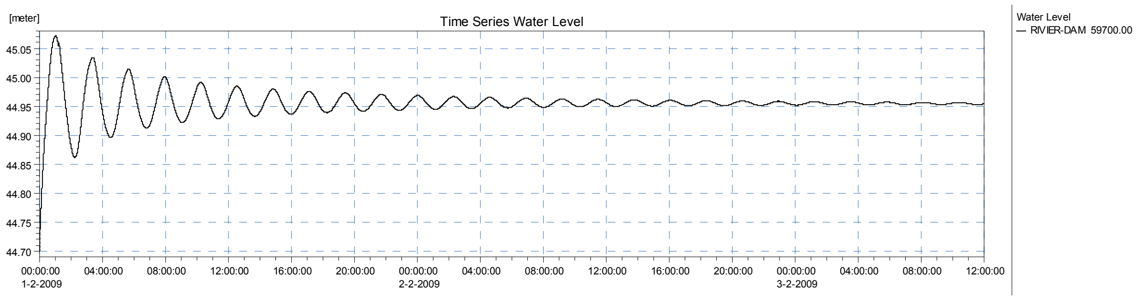


Figure 7.3-4 Oscillating at dam wall (Stable Level = 44.955 m)

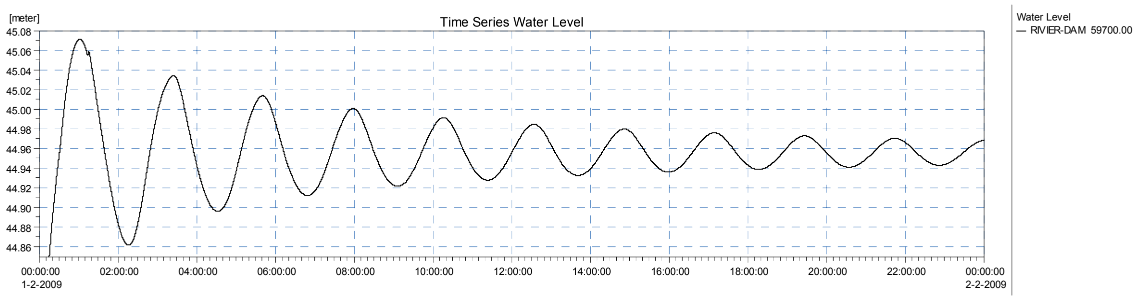


Figure 7.3-5 Oscillating at dam wall Zoomed (Stable Level = 44.955 m)

7.4 WIND SET-UP

Mike 11 simulates the wind shear stress by using the equation:

$$t_w = t_{fac}\, C_w\, \rho_a\, V^2_{10*}$$

With	t_{fac}	=	Wind friction coefficient
	C_w	=	Topography surrounding factor
	P_a	=	Density of air
	V^2_{10*}	=	Velocity 10 m above water surface

For a reservoir water level starting condition indicated in Figure 7.4-1, and wind boundary conditions as shown in Fig 7.4-2, the simulated water level at the dam is shown in Fig 7.4-3. Fig 7.4-4 shows a longitudinal profile of the reservoir. The rise in water level at the dam is 0.031 m for a wind speed of 25 m/s.

With a wind blowing down the length of the reservoir at 35 m/s, the setup at the dam is 0.061 m (Figures 7.4-5 to 7.4-7). The analytical setup calculated by using the SANCOLD (1990) guidelines is 0.101 m but the method increases the fetch length by a factor 2.

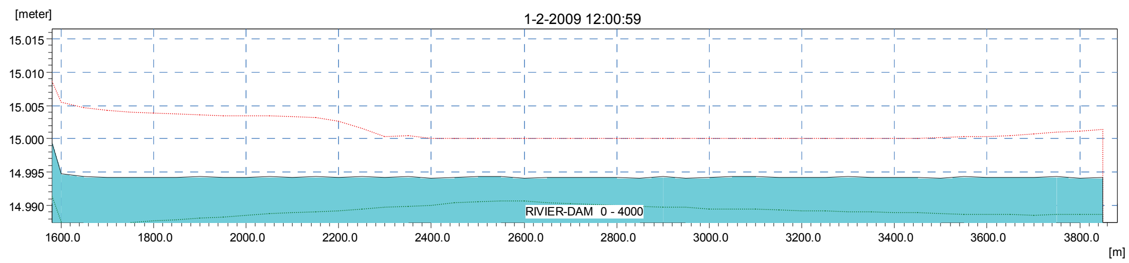


Figure 7.4-1 Water level without wind

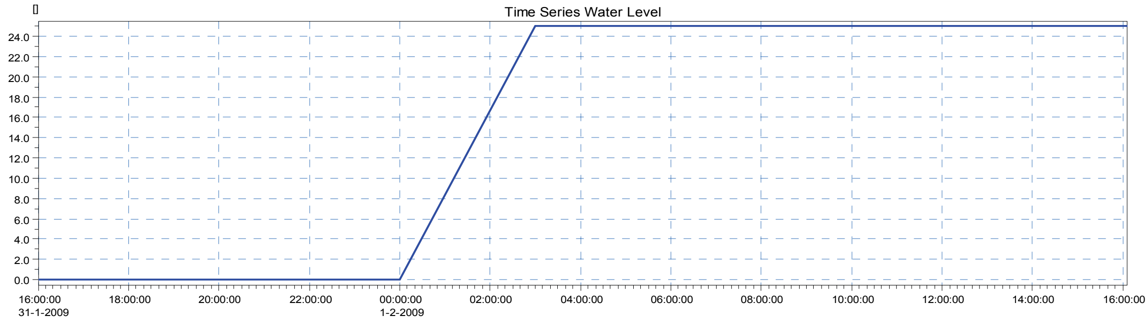


Figure 7.4-2 Wind boundary conditions (m/s)

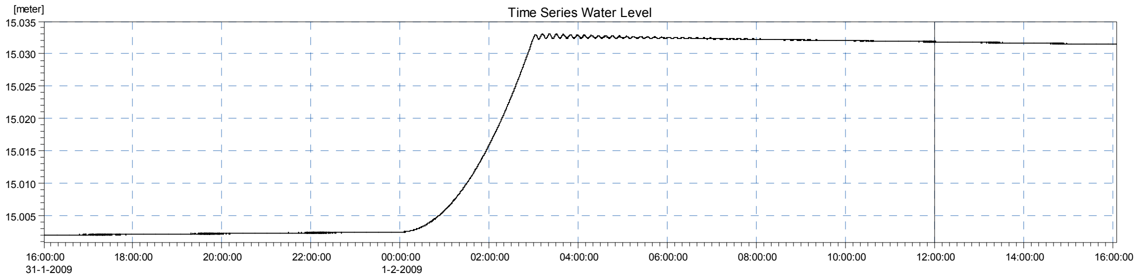


Figure 7.4-3 Simulated water level at dam at 25 m/s

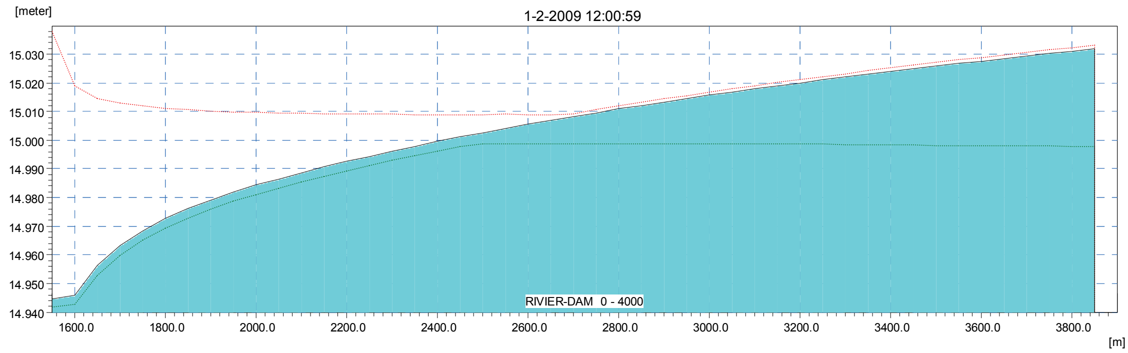


Figure 7.4-4 Reservoir water level with wind 25 m/s

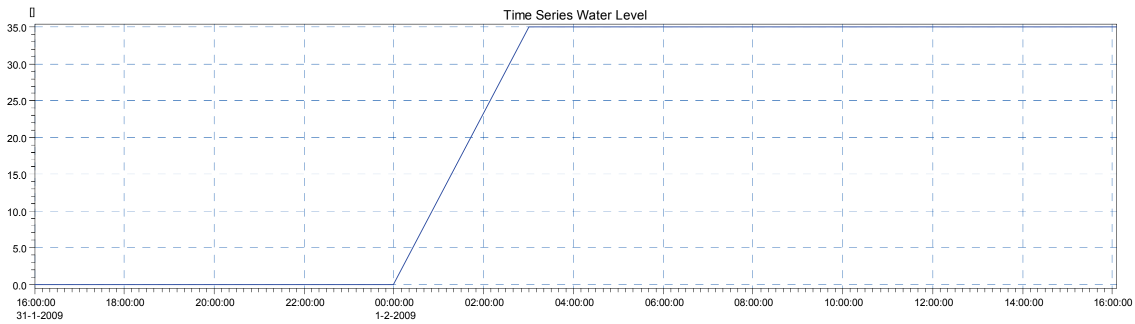


Figure 7.4-5 Wind boundary condition 35 m/s

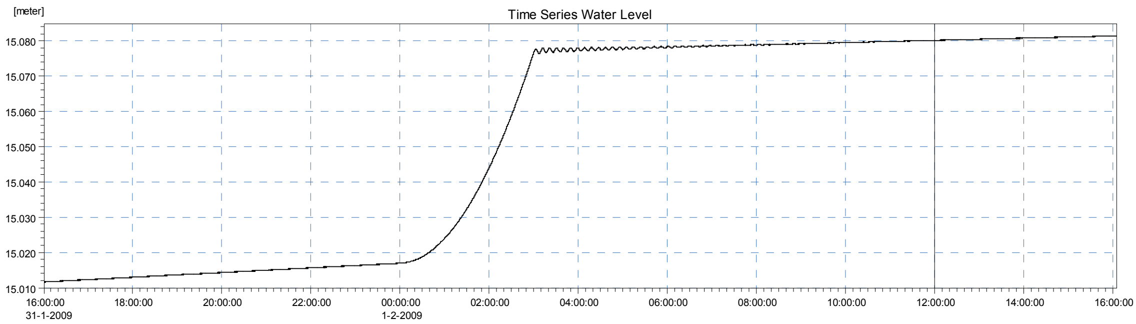


Figure 7.4-6 Water level set-up at dam at 35 m/s

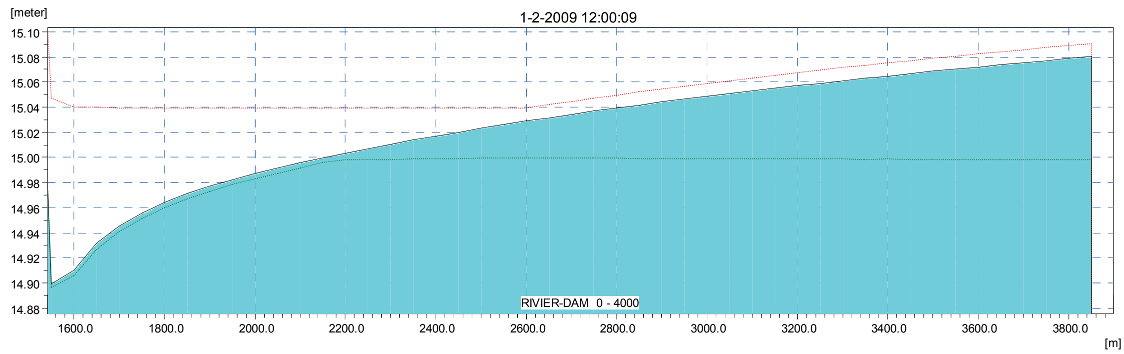


Figure 7.4-7 Reservoir water level at wind 35 m/s

7.5 DAM OUTFLOW ADJUSTMENT

Sudden gate adjustments at a dam to regulate the outflow could lead to unsteady flow patterns in the reservoir. For a case where the outflow is increased from 0 to 500 m^3/s in 5 minutes and closed again after about 30 minutes (Figure 7.5-1). The effect of the release is a general lowering of the water level (Figure 7.5-2), but if one looks closely oscillations are created in the reservoir (Fig 7.5-3) which originates during closure of the outflow.

In another simulation where the outflow is adjusted, with an inflow hydrograph (Fig 7.5-4), the resulting oscillations at the dam are shown in Figure 7.5-5.

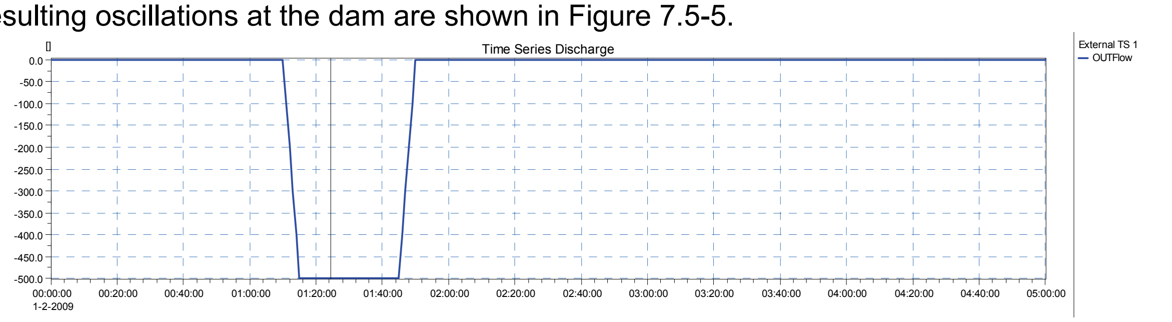


Figure 7.5-1 Outflow at dam

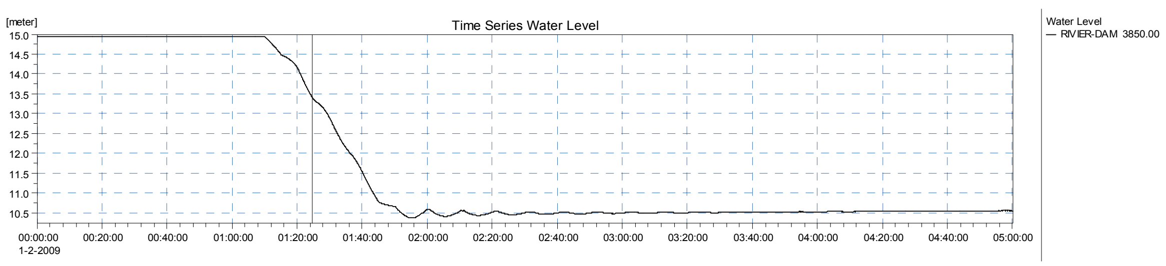


Figure 7.5-2 Simulated water level at dam

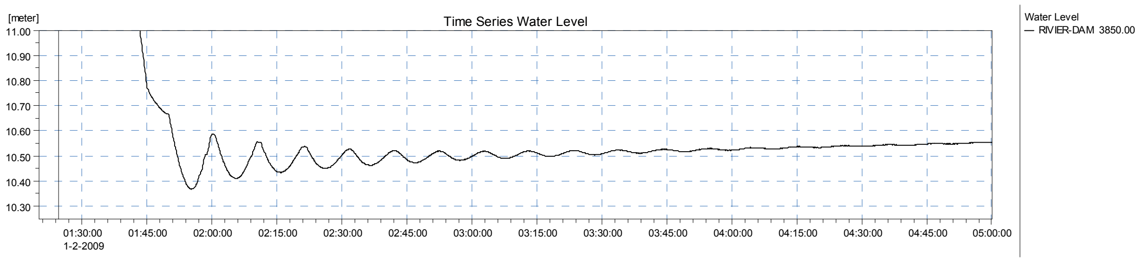


Figure 7.5-3 Simulated water level zoomed in at dam

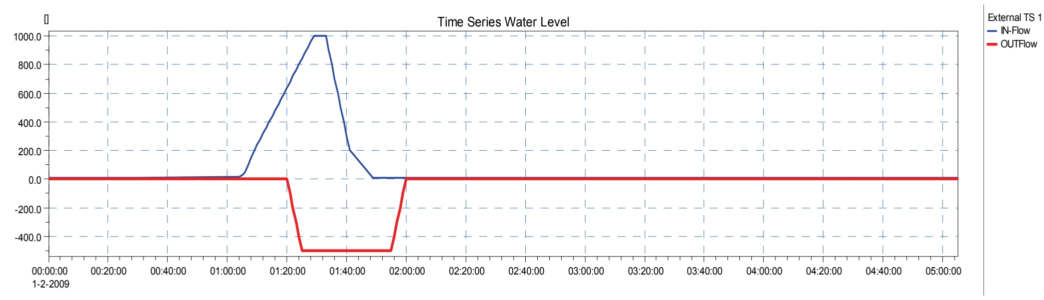


Figure 7.5-4 Boundary flow conditions (negative flow is outflow)

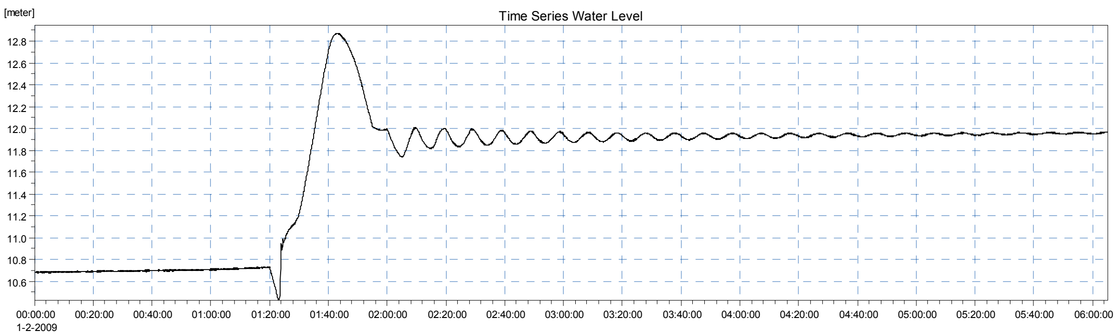


Figure 7.5-5 Simulated water level at dam

7.6 CONCLUSIONS AND RECOMMENDATIONS

The above simulations using a 1D mathematical model show that the unsteady flow patterns in a reservoir could be analyzed by mathematical model. 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
- Local low pressure effects

8. COMBINING FREEBOARD COMPONENTS

8.1 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, 1991) with flood surcharge freeboard only, is the flood which may cause substantial damage to structure and surroundings but must not be such as to cause the dam to fail catastrophically causing loss of human life and economic loss. It will be found that in many cases the determining condition for freeboard will be that for the SEF at non-overspill crest, i.e. where overtopping is not allowed. However, the other case must also be checked to ascertain the most critical condition.

In the Guidelines on Safety in relation to Floods" (SANCOLD,1991) spillage under any of generally accepted criteria must not endanger the safety of the structure. These criteria are not listed but are proposed here. Combination of freeboard components are grouped in combination numbers (Table 3.1 in Volume II) that should be tested as shown in Table 3.2 (refer to Volume II). All conditions in the combinations mentioned and indicated (for a specifically numbered combination of criteria) are to be met simultaneously.

Site specific conditions at the reservoir which say influence wave run-up and wind set-up should be duly taken into consideration.

Adjustments for direction of wind and possible wind tunnelling effects should be made when specific data are available to substantiate such adjustments.

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

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

Certain practical guidelines for the determination of freeboard have been developed by various organisations over the years and are given in Table 3.3 (Volume II). These practical rules of thumb are often of application to small dams and medium sized dams with a low hazard rating and provide also a check on freeboard calculations. These practical guidelines are also discussed in further paragraphs dealing with different types of dams (Volume II).

8.2 RISK ANALYSIS APPROACH

8.2.1 Objectives of assessment

The DEFRA *interim guide to Quantitative Risk Assessment for UK reservoirs* (DEFRA-QRA) (see Brown & Gosdin 2004[1]) has been identified as a possible basis for risk assessment of

[1] Brown AJ & Gosden JD (2004). Interim guide to quantitative risk assessment for UK reservoirs. Thomas Telford.

dams according to which the SANCOLD *Interim guidelines on freeboard for dams* could be converted into a quantitative risk assessment procedure for the determination or assessment of freeboard allowance.

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 is recognised as a suitable framework according to which the low probabilities of failure with large consequences of civil engineering works could be considered on a rational basis. The methodology makes best use of limited information together with transparent judgement, to estimate safety levels and how they will be influenced by design changes and modifications of existing facilities. The need for improved methods of assessment derives from general trend such as the increasing magnitude and number of facilities, aging of existing facilities, budgetary constraint whilst higher safety levels are demanded by society. 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 design procedures. The lack of appropriate probabilistic data is the most important basis for criticism of risk assessment. The counterargument is that no method should be acceptable under such conditions, but that risk assessment combines statistical and judgement based data in a transparent manner. The operational use of risk assessment then also facilitates the gathering of suitable data and experience for the systematic improvement of engineering practice.

The *DEFRA-QRA Guidelines* are selected for evaluation because it is available as an operational procedure for the risk assessment of dams, albeit constricted to a limited scope of application of operating embankment dams in the UK. The objective is to determine how it could be used to incorporate quantitative risk based procedures into the SANCOLD Guidelines on freeboard allowance for dams. For this purpose the essential features of DEFRA-QRA are presented here, with reference to Brown & Gosdin (2004) for a concise outline and software for application of the procedures.

8.2.2 Outline of DEFRA-QRA procedure

The DEFRA-QRA Guidelines were developed from the DEFRA (2002) investigation as discussed in §4.1 of the Review Report, which had the main aim to: *Propose and demonstrate an integrated system which provides a framework for decision making … on the annual probabilities of occurrence, consequences and tolerability of all the various threats to reservoir safety*. The intended use of the DEFRA-QRA Guide is to complement to existing dam safety standards for the purpose of screening the risk that a dam poses to the downstream public. It provides a procedure to quantify the risk of dam failure through:
the calculation of probabilities of certain external and internal threats, causing failure through possible event trains (*F*);
determining the consequences of failure through dam-break analysis and flood routing down the valley; the likely loss of life (*N*) and the expected third party monetary loss (£);
the tolerability of the resulting risk is then determined by classification of the dam in terms of the magnitude of {*N*; £} and comparison of its risk characteristics to set criteria on an F-N chart.

8.2.2.1 General Scheme

The process of application of the integrated system is presented in Table 4.1.1 of the Review Report. Supporting models are provided according to which the annual probabilities of the occurrence of certain threats and event trains can be determined from the characteristics of the dam under investigation. A number of core threats are identified, with the option of considering specific additional threats. The matrix showing the relationship between identified threats and subsequent failure modes on which the DEFRA-QRA Guidelines are based is shown in Table 4.1.2 of the Review Report (Table A.1 from Brown & Gosden 2002).

In addition to procedures for the calculation of the dam critical flood and the consequent probability of failure due to extreme rainfall, information is provided on historic failure probabilities for the core threats, and procedures for adjustment of the probabilities of the event trains as derived from characteristics and condition of the dam. The annual probability of failure of the subject dam due to the contribution of the dam break of a single upstream dam is included in the procedure.

8.2.2.2 Risk Analysis Software

The risk calculations and tolerability assessment is done in a pre-formatted and programmed manner as multiple Microsoft Excel spread-sheet calculations. The analyst is lead in gathering appropriate information and providing the quantitative input systematically, including the selection of appropriate options and procedures.

The arrangement of the calculation procedures makes it possible to extend the scope of the guide by adding threats and event trains with their associated probability models as additional spread-sheets.

The main elements of the DEFRA-QRA Guide consist of the following main steps:
Background information: gathering related information on the characteristics of the subject dam, including background data; field inspections; upstream dams; downstream installations and reservoirs;
Application: the scope of application for which probability models are provided;
Risk elements: the risk calculation process in terms of the two main risk elements of calculating annual probabilities and consequences respectively;
Calculation models: particulars of the respective generic models which are incorporated in the procedures;
Assessment: the risk-tolerability determination and assessment, including suitable criteria.

8.2.3 DEFRA-QRA SCOPE OF APPLICATION

The scope of application of the DEFRA-QRA Guideline is limited to risk assessment of existing operational embankment dams. The risk calculation as such is of a general nature and is therefore widely applicable and in principle it is extendible. However the limitations in dam types and situation derive from the calculation models which are included. The models in turn are limited by the availability of historic data as compiled by a number of supporting studies which are referenced in the Guidelines.

Although the scope of application is limited by the availability of risk models and data for the various threats and failure modes, this feature also represents the strength of the procedure, by making the procedures fully operational within this limited scope. The analyst if guided in gathering all the necessary information, apply judgement where needed and do the risk calculations to the final point, without having to develop the necessary models from first principles.

The DEFRA-QRA Guideline is set up to follow a specific procedure in the review of the results of the assessment, mainly to consider whether the results are reasonable and what the implications are. There are a number of well established ways in which the results of risk assessment can be applied, either by considering the specific facility in a more refined manner such as to improve weaknesses, or by regarding it in a wider context such as managing a portfolio of dams within budgetary constraints. Another classification considers whether the risk analysis is used for operations (design, maintenance, safety management, etc.) or development such as the formulation of effective generic guidelines and standards.

8.2.4 Risk calculation

The calculation of the risk *R* consist of the separate determination of the conventional two components of the annual probability of dam breach (p_B) and its consequences (*C*), consisting in turn of the likely loss of life (N_{Fat}) and direct third party damage in monetary terms ($C_£$). This could be expressed as

$$\begin{aligned} R &= (p_B) \quad \text{x} \quad (C) \\ &= (p_B) \quad \text{x} \quad \{N_{Fat}; C_£\} \end{aligned} \qquad (1)$$

8.2.4.1 Probability Calculation

The annual probability of failure is composed of multiple threats that could possibly develop through different event trains into identified modes of failure. The total annual probability of failure (*APF*) is calculated by the simple addition the probabilities of each failure mode as:

$$p_B = APF^{ALL} = \sum(APF^{i}) \qquad (2)$$

i Specific Threats & Failure modes causing uncontrolled sudden large release of water

See Table 8.2.1 for definition of failure modes.

Table 8.2.1 Matrix showing mechanisms of deterioration linking threats to failure modes

Threats				Modes of failure due to *T*						Annual *p* of individual *T* causing failure
Type		*T*	Annual *p* for *T* individual	External erosion		Internal erosion		Sliding	Appurtenant works	
				Over-topping	Other	Through fill	Along interface			
		Symbol for mode *M*		***OT***	***EE***	***IF***	***IS***	***SL***	***EM***	
External	Extreme rainfall/flood	FL	APT_{FL}	Inflow > spillway capacity	Local runoff down face	Increased head		High load / increased pwp	High load	$APT_{FL}^{ALL} = \sum APT_{FL}^{OT}$ $APT_{FL}^{EE} + +$
	Upstream reservoir failure	CA		As for extreme flood						APT_{CA}^{ALL}
	Wind	W	etc	Waves blown over crest	Wave attack of up-face	Unlikely		Waves saturate down-face	Un-likely	etc
	Snow/ice	SN		Blockage of spillway	Unlikely	Unlikely		Unlikely	Various	
	Earthquake	EQ		Seiches	Unlikely	Disrupt filters	Disrupt contact stresses	Liquefaction; horizontal load	Various	
	Sabotage	TR		Blockage of spillway	Unlikely	Explosion within appurtenant structure		Blow up dam	Various	
Internal	Human error	HE		Blockage of spillway	Unlikely	Unlikely		Unlikely	Various	
	Aircraft strike	AS		Physical disruption of crest	Unlikely	Disrupt filters		Impact load	Unlikely	
	Internal stab – embankment	IE		Settlement	Unlikely	Various		Various	Na	
	IS – app works	IS		Inability to open spillway gates	Unlikely		Broken pipe	Unlikely	Various	
Annual probability of failure				APT_{ALL}^{OT}	APT_{ALL}^{EE}	etc				$p_B = APT_{ALL}^{ALL}$

The probability calculation is therefore done as a Fault Tree analysis, where all the contributing events that lead and contribute to a single event are considered. The different failure events are treated as independent and the probabilities can therefore simply be added. Since the probability of occurrence of each threat & failure event is small such addition of probabilities for independent events is reasonable. In cases where these events share common modes and are therefore correlated, the result provides a conservative upper limit probability.

The use of point values for the annual probabilities is indicative of the relatively simple level at which risk analysis is applied. The use of more advanced convolution models, where the joint occurrence of various sources of uncertainty is taken into account, is warranted in such a scoping risk assessment. Generally the annual probability of failure is primarily determined by the extreme deviation of a single component, which could be used together with expected probabilities of associated events. . This approach which is effectively taken by DEFRA-QRA Guidelines is similar to that used by Eurocode for the accidental design situation (see EN 1990:2002). A systematic assessment of this simplified approach would improve confidence in the method.

An important feature of this process is that the *probability of occurrence* of specific chains of events that lead to dam failure is determined for a given set of conditions. This is in contrast to the design process, towards which the SANCOLD freeboard procedures are formulated, where the *required freeboard* is determined for given probabilities of occurrence as specified.

8.2.4.2 Consequence Calculation

In spite of all the failure modes *i* for which probabilities are determined, only a single (deterministic) calculation of the consequence of dam failure is done, although the outcome is expressed in terms of the dual parameters {likely loss of life (N); monetary loss ($C_{£}$)}. The consequence analysis consists of the following steps:

- Dam breach hydrograph established
- Setting up the flood routing – distance, slope of valley
- Routing of dam-break flood down valley
- Estimating population at risk
- Estimating likely loss of life (N)
- Estimating cost of direct third-party damage ($C_{£}$).

8.2.4.3 Risk Tolerability and Review

Risk tolerability of the dam is assessed by comparing the results of the estimated probability and consequences to criteria for acceptable and tolerable levels of risk. Firstly the consequence class of the dam is established by considering only the consequences on a consequence diagram {likely loss of life (*N*); third-party direct damage ($C_£$)} to assign a “Dam Category” according to which statutory requirements such as requiring a Flood Plan or the frequency of surveillance are set.

Risk criteria consist limits to the combination of {annual probability; likely loss of life} which are conveniently expressed on an F-N diagram, where the F-axis represents the annual probability of an event which leads to a likely loss of life larger than N as represented on the N-axis; the criteria is given in terms of a line defining the limit below which combinations of F-N values are *Broadly Acceptable*; a limit above which it is *Unacceptable*; with an ALARP region in between.

The ALARP region is generally used to indicate risk conditions which require some management and decision making in terms of whether the cost of risk improvement would be disproportional to the gain in risk performance. In the case of the DEFRA-QRA Guide the ALARP region represent conditions where improvements are required if its cost is below some criterion of the “value” of the reduction of the likely loss of life expressed at its net present value.

The review of the QRA results encompasses a systematic consideration of the input data, assumptions and results to ensure that they are reasonable. The final step is draw conclusions from the analysis and to come to a final decision on the resulting course of action.

8.2.5 EVALUATION OF DEFRA-QRA

The DEFRA-QRA Guidelines could be applied as the basis for the standardised assessment within a given field of application such as for the dams under the jurisdiction of a given authority. In such an environment it could also form the basis for the development of more refined assessment procedures, capturing of risk related data and experience with dam design, maintenance and safety management activities. In this investigation it is however considered in order to determine how risk analysis can be employed to improve the guidelines for a specific aspect of dam safety performance, namely the provision of freeboard against overtopping which could lead to failure.

The DEFRA-QRA Guidelines are assessed here firstly in terms of its merit in using it for general integral risk assessment applications. Its relevance to the specific issue of the provision of freeboard is then regarded.

8.2.5.1 DEFRA-QRA as General Risk Assessment Procedure

As indicated above, the main strength of the DEFRA-QRA Guidelines is that it is set up to provide a comprehensive risk assessment, which includes the essential steps, starting off from the gathering of all related information from records and an inspection, through all the elements of determining annual probabilities, consequences, risk tolerability and the final decisions on following up activities; including also generic data on historic failure rates and ways in which the specific characteristics of the subject dam could be taken into account.

The main elements of strength of the Guidelines are the following:

Main components of risk assessment: the main steps required to perform a comprehensive risk assessment of a dam are identified and set out, to be followed systematically by the analyst.
Risk models: supporting models to be used to calculate the relevant annual probabilities and consequences are provided, together with supporting guidance on how to use them, including references to the supporting investigations on which the risk models are based.
Pre-formatted calculation procedures: spread sheet software is set up to assist in logical data input, allowing the application of proper judgement, automatic calculation of subsequent steps and presentation of the results, all in a relatively user friendly manner.
Simplicity of models and procedures: such a standardised procedure can only be compiled at a relatively simplified level, using only point estimates of probability, without taking account of the propagation of uncertainties or the effects of common modes or correlations; nevertheless it serves as a useful integral basis for raising the level of sophistication if justified.
Flexibility of procedures: the relative importance of the various components of the process can easily be established by sensitivity analysis, the presentation of intermediate results, and even the addition of other relevant models.

8.2.5.2 Relevance of DEFRA-QRA to SANCOLD Freeboard Guidelines

A direct comparison between the DEFRA-QRA Guidelines and the SANCOLD Guidelines for dam freeboard is needed in order to establish the role that the DEFRA-QRA Guidelines could play in improving the SANCOLD Guidelines. This requires both the way in which the safety performance of dams is fundamentally approached in the respective procedures, and the practical application of the resulting methodologies.

There are two main issues causing some discrepancies between the DEFRA-QRA Guidelines and the SANCOLD Guidelines on freeboard for dams: there are firstly fundamental differences in the approaches taken to determine the required freeboard for a dam, or to determine the risk characteristics of a given dam;

The fundamental approaches taken by the Defra-QRA & SANCOLD and the modes of implementation are compared in Table 8.2.2.

Table 8.2.2 Comparison of Defra QRA and SANCOLD Freeboard Guidelines

#	Element	Defra-QRA	SANCOLD Freeboard
1	Fundamental approach	Integral determination of risk for given dam and its properties existing operational dam sufficient experience base to provide models for reference failure probabilities	Determination of total required freeboard height* h_f (design) as for design of dam geometry or sufficiency of existing dam models for the calculation of contributing heights
2	Threats	Identified threats included in procedure: flood surcharge as major element; differentiated into external & internal consequent failure modes	Disturbances of water level caused by processes primary: flood surcharge secondary: wind, wave & surcharge, etc.
3	Treatment of probabilities	Calculation of probability as one of the main components of risk: individual annual probability ($p_{B,i}$) of threat (i) at level causing dambreak	Deterministic procedure with: disturbance magnitude specified at given levels of probability differentiated probabilities in terms of {dam size; hazard rating}
4	Combination of threats	Total probability of dambreak (p_B) by addition of probabilities ($p_{B,i}$); implying the individual threats (i) are statistically independent.	Worst disturbances of water level (h_f) addition of individual heights ($h_{f,i}$) for reasonable combination of disturbances (i).
5	Treatment of consequences	Main component of risk (C) annual likely loss of life (N) third party monetary loss ($C_£$)	Qualitative classification apply three level hazard rating
6	Assessment	Principal objective of assessment point estimate of risk { p_B; C} measured against set risk criteria consider implications	Deterministic assessment typical of design process required freeboard height (h_f) sufficiency of given height {Yes; No}
7	Freeboard	Taken as given characteristic of dam when assessing of flood surcharge other combined processes not critical resulting from treating threats as independent.	Risk implications embedded qualitatively as basis for deterministic procedures Table II) matrix of dam size (risk potential) & hazard rating (consequences) used to specify disturbances probabilities & combinations
* height = vertical distance between FSL & nominal crest level			

From this comparison it is clear that the differences between Defra-QRA and the SANCOLD Guidelines are so fundamental, that the two approaches cannot merely be merged. Furthermore, the Defra-QRA Guidelines are at such a simplified level of treating the probabilities of the consequences of insufficient freeboard that it is not suitable to perform a calibration of the SANCOLD specifications for the combinations of the respective disturbances taken into account when determining the necessary freeboard.

8.2.6 CONCLUSIONS

Risk assessment provides an invaluable complementary tool to established dam safety design procedures. It is particularly useful to identify critical components of a portfolio of dams, or the relative importance of the safety components of individual dams.

Its rational and quantitative nature makes it a useful aid in risk management and decision making, allowing for optimised programs within the constraints of budgets and an ever increasing level of safety and accountability demanded by society.

In spite of the limitations of the lack of appropriate data, it provides a proper framework for integrating all sources of information for present risk management, and to gather the data for future development – there is no motivation to gather data properly, unless the need for such information is revealed by the limitations of present risk assessment exercises.

The Defra-QRA Guidelines provide an effective tool for assessing the risk characteristics of operational embankment dams. It's useful features include the systematic approach taken, providing useful guiding information, leading the analyst to gather the relevant data and formulating assumptions transparently, automating the risk calculations whilst allowing full control into the process, providing criteria on risk acceptability and assessment of the results.

8.2.7 RECOMMENDATIONS

It is recommended the present SANCOLD procedures are essentially maintained for establishing freeboard requirements on the basis of calculating freeboard requirements at specified reliability (probability) performance levels, including the improvements that derive from related investigations.

Since the intention of the Defra-QRA Guideline procedures is to complement established dam safety procedures, they can be implemented as an effective operational tool to consider the results from the SANCOLD freeboard assessment or design from an integral risk perspective. The SANCOLD Guidelines which are essentially reliability based (Table I)

deterministic process which is done against a qualitative risk framework (Table II) should be taken as the operational procedure within the present field of application of these Guidelines. The view obtained from the Defra-QRA Guidelines should be used in a complementary manner to obtain the dam safety and risk context of the specific application.

Useful experience would be gained through such supporting application of the Defra-QRA Guidelines, without compromising the effective use of the SANCOLD Guidelines. This process would enhance the further development of quantitative risk assessment procedures, as is indeed one of the objectives with the establishment of the Defra-QRA Guidelines. It would also contribute to the gathering of appropriate data and information, an break through the obstacles of the lack of application due to the lack of data for risk assessment and vice versa.

8.3 APPLICATION OF THE DEFRA QRA GUIDE FOR UK RESERVOIRS ON A TYPICAL PROTOTYPE

8.3.1 The DEFRA-QRA guide for UK reservoirs

The United Kingdom (UK) Department for Environment, Food and Rural Affairs (DEFRA), sponsored a research project in 2002 from which the Interim Guide to Quantitative Risk Assessment for UK Reservoirs (see Brown & Gosdin 2004[2]) was developed, hereinafter called the DEFRA-QRA. The DEFRA-QRA was compiled as a tool for the management of reservoir safety in the UK. It provides a screening level framework for decision-making by experienced dam professionals on the annual probabilities of occurrence, consequence and tolerability of the risk of reservoir failure.

The probability of failure element of the risk assessment was devised for embankment dams which have been in service for longer than five years; the wear-in period. Note that the DEFRA-QRA does not apply to concrete and masonry dams due to insufficient historical data on the performance in relation to internal threats such as concrete degradation, geotechnical and geological threats.

As indicated in Section 8.2 of this document, it is clear that the differences between DEFRA-QRA and the SANCOLD Guidelines are so fundamental, that the two approaches cannot merely be merged.

Following is a discussion on the application of the risk model presented in the DEFRA-QRA. The prototype on which the model was applied is the Hardap Dam in Namibia. This dam was chosen due to its high risk regarding likely loss of life (LLOL) should the dam break, the town of Mariental with several thousand inhabitants lies in the path of the expected flood wave.

8.3.2 Summary of the DEFRA-QRA model as presented in the Guide

DEFRA-QRA quantifies the risk of the **failure of a dam** which can be defined as an uncontrolled sudden large release of water from the reservoir it retains. DEFRA-QRA includes a Microsoft Excel Workbook which comprises 11 sections which are divided into the following three parts:

- Part 1 Estimating the overall annual probability of failure (Sections 1 to 7).
- Part 2 The consequences if the dam did fail (Sections 8 to 10).
- Part 3 The consequent risk (probability of failure × consequences) (Section 11).

The DEFRA-QRA Excel Workbook contains the entire quantitative risk assessment model and uses a UK case study as an example to illustrate the way it works. The workbook uses tools commonly applied in dam safety practice in the UK; such as broadcrest weir formulae for overtopping, Probable Maximum Flood (PMF) calculation for the extreme flood size and JONSWAP formulae for wind wave calculation.

The components making up the three parts of DEFRA-QRA are discussed below. Note that the components related to dam freeboard will be discussed in more detail.

---

[2] Brown AJ & Gosden JD (2004). *Interim guide to quantitative risk assessment for UK reservoirs.* Thomas Telford.

8.3.2.1 Part 1: Estimating overall annual probability of failure

Section 1 requires information on the characteristics of the subject reservoir; background data, site inspection information regarding upstream and downstream dams and conditions downstream related to flood routing and also the population at risk.

Section 2 deals with the annual probability of failure of the dam due to extreme floods and adverse weather. **Three freeboard components addressed in this section; these are the Dam Critical Flood, Wind Generated waves and wave run-up**. This section also requires the user to estimate the primary failure modes through an observational method (formulate an event train). This process is necessary to qualitatively identify possible mechanisms of deterioration to ensure the annual probability of failure is reasonable.

The **Dam Critical Flood (DCF)** used in the UK does not correspond at all to the Safety Evaluation Flood (SEF) proposed by SANCOLD. The DCF is the flood that would cause the dam to fail whereas the SEF will overtop the NOC causing some damage without causing the dam to fail.

The sum of the water spilling over the NOC, the spillway and any additional outlet/s provides the total outflow from the dam. Allowing for flood attenuation in the reservoir, the outflow figure can be adjusted to give the inflow flood which is the DCF. At this stage no recurrence interval is allocated to the DCF.

The annual probability of failure due to extreme floods is calculated by expressing the flood as a fraction of the PMF. The PMF has a probability of occurrence of 1.0E-6.
Wind Waves are calculated using the JONSWAP method (Joint North Sea Wave Project: 1970). After the significant wave height and run-up has been calculated, the discharge of spill water over the crest is calculated for cases of zero wave freeboard (water level is equal with NOC) and also for 20%,40% and up to 100% wave freeboard.

Previous research (Brown and Gosden; 2002) has indicated that the discharge over the NOC caused by wind waves is not commonly significant, hence they do not include it in the DCF calculations. However if the wave and wave run-up are found to be significant, one can reduce the flood level for the DCF by the height of the wind wave (e.g. 60% wave freeboard) at the point where it creates critical flow on the downstream embankment. In so doing the DCF is reduced.

Sections 3, 4, 5 and 6 allocate an annual probability of failure to the subject dam due to failure of the upstream dam, due to internal stability regarding the embankment and appurtenant works of the embankment respectively and due to other external threats which can be identified, such as seismic activity.

Section 7 produces an annual probability of failure by adding the separate probabilities: the annual probabilities estimated by the DEFRA-QRA, and the summing of rates is mathematically acceptable (Brown and Gosden; 2004).

8.3.2.2 Part 2: The consequences if the dam did fail

Section 8 deals with a dam break analysis by estimating the extent of inundation. A dam break hydrograph is generated using Froelich (1995). The flood is then routed along the downstream valley.

Section 9 makes an estimation of the population at risk and then calculates the LLOL taking into account the damage potential of the water expressed in m^2/s. Consideration is given to no warning as well as to some warning which could reduce fatalities.

Section 10 estimates the direct cost of third party flood damage for residential and non residential properties. Types of damage include inundation damage, partial and total destruction of buildings.

8.3.2.3 Part 3: The consequent risk

Section 11 The likely loss of life (LLOL) and annual probability (AP) are plotted onto an FN chart to compare the risk for the subject reservoir with the criteria proposed by the UK Hazardous Installations Criteria.

The risk can plotted in one of three zones: the broadly acceptable zone, the intermediate "as low as reasonably possible" zone (ALARP), and the unacceptable zone. In the ALARP zone one can perform a sensitivity analysis to determine best utilisation of funds to gain the most risk profit.

8.3.3 Hardap dam results for DEFRA-QRA

To perform a risk analyses on the Hardap Dam, Namibia, according to the DEFRA-QRA method, several manipulations of the formulae in the DEFRA model had to be made:

- In this example the Regional Maximum Flood method (RMF) was known and not the PMF and the RMF is equated to the 1:10000 year flood (based on DWAF methodology) and factored by 2 (according to the DEFRA-QRA model) to determine a PMF applicable to the Hardap Dam.
- The Full Supply Level (FSL), which is the top of the gated spillway, had to be reduced to the OGEE spillway level to allow the DEFRA model to calculate the spillway discharge capacity.
- Results for the Hardap Dam were obtained for four (4) freeboard combinations found in the DEFRA-QRA approach as well as in the SANCOLD Freeboard Guidelines. Note that in all four cases the flood surcharge does not form a part of the freeboard components because the DEFRA-QRA model calculates the flood which the dam can accommodate after subtracting the freeboard components from the total available freeboard.
- The four freeboard cases are discussed below:

1) DEFRA typical approach: The wind wave and surge freeboard has a negligible effect on the risk of the dam and it is ignored. Hence the still water level of the flood is equal to or slightly higher than the NOC for allowable spill.
2) SANCOLD Freeboard combination number 1: Allow for a 25 year wind wave and set-up.
3) SANCOLD Freeboard combination number 2: Allow for a 25 year wind wave, set-up and flood surge which is taken as 0.5 m in this case (a SANCOLD 1990 recommendation).
4) SANCOLD Freeboard combination number 3: Allow for a 100 year wind wave, set-up and flood surge which is taken as 0.5 m in this case (a SANCOLD 1990 recommendation).

The required freeboard increases from option 1 to option 4 (zero freeboard for option 1 to 2.45 m freeboard for option 4). Consequentially the flood size reduces from option 1 to 4 which causes the risk to increase from option 1 to 4.

The results of the freeboard case studies, 1 to 4, are presented in the same number format in Figures 8.3-1 to 8.3-4. The required freeboard height is indicated in each graph description. Note that SANCOLD (1990) Freeboard combination numbers 4, 5 and 6 are not included in the case studies. These cases represent earthquakes, landslides or blocked spillways which all can have a random number which may be site specific. For the Hardap Dam case the annual probability of an earthquake or landslide is very small due to its location in a low seismic active area and the solid rock hills surrounding the dam basin. Hence the worst options would be one of the first three in the SANCOLD list.

Special notes regarding the Hardap Dam model

- Regarding the allocation of annual probability of failure due to appurtenant works: Risk due to failure of appurtenant works is reasonably high due to poor maintenance on the gated spillways. If the gates do not open a large portion of the flood will spill over the embankment and failure of the dam is imminent. It may also be noted that due to poor maintenance, the bearings of one of only two radial gates on another dam in Namibia has seized, rendering the gate inoperable. This in general increases the risk of appurtenant works failing in Namibia.

Note that the tables for AP of appurtenant works failing, as proposed by DEFRA-QRA, was not available to the compiler of the workbook at the time of this report, hence an assumption was made in this regard. The outcome is a high risk due to low maintenance on appurtenant works as indicated in Figure 8.3-1 below.

On Figure 8.3-2 a line was drawn in representing the Boundary Line for Involuntary Risk which is used by the Department of Water Affairs and Forestry (DWAF) in South Africa to differentiate between acceptable and unacceptable risk. It can be noted that the DWAF line is less conservative regarding the LLOL when compared to the DEFRA model.

A more comprehensive description of the legend items is given in the paragraph below Figure 8.3-1.

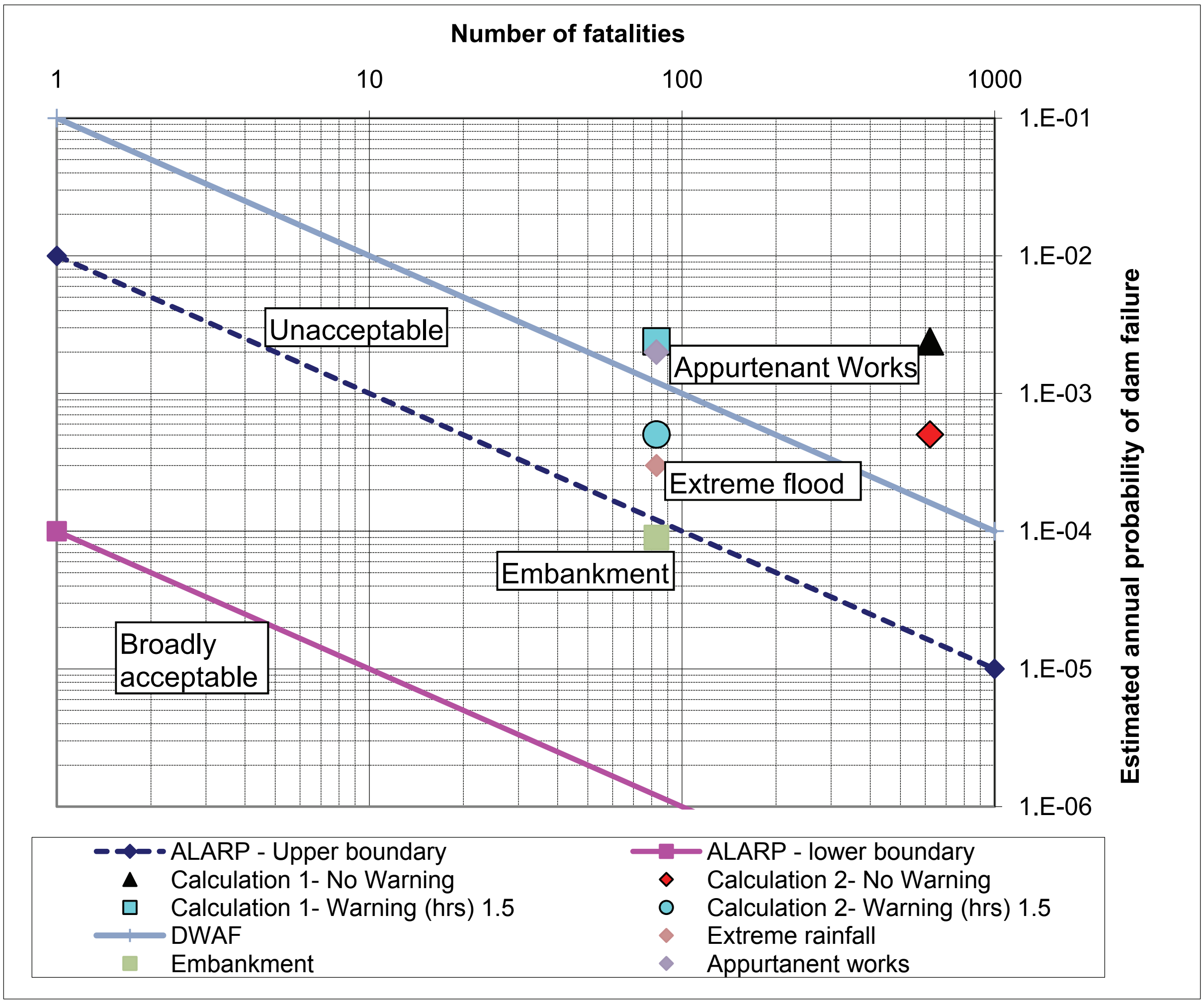


Figure 8.3-1 The FN table for Hardap Dam allowing zero freeboard for wind or other freeboard components (The entire freeboard height is utilized to pass the flood over the spillway and dam NOC where allowable)

Description of Legend items in the Graphs:

- The **ALARP upper and lower boundary** provides the evaluator with limits for unacceptable and broadly acceptable risk. The area in between the lines is the ALARP region which allows the dam owner to decide how to utilize available funds to gain the most risk profit from any betterments to the dam.
- The **DWAF** line represents the risk tolerance line used by DWAF Dam Safety Division; above the line the risk is considered too high and below the line the risk is acceptable. It is included for comparison purposes only.
- **Calculation 1** represents the condition of the dam at present, with and without warning time for people living downstream, hence the difference in fatalities between the two.
- **Calculation 2** represents the proposed future condition of the dam after betterment works have been performed to improve the risk of the dam. Warning time as previously indicated.
- **Extreme rainfall, Embankment and Appurtenant works** represent the three individual risk components which make up the total risk of "Calculation 1 – Warning 1.5 hours". These

components specifically apply to the present condition of the dam (Calculation 1) with a 1.5 hour warning to inhabitants downstream which will significantly reduce the number of fatalities.

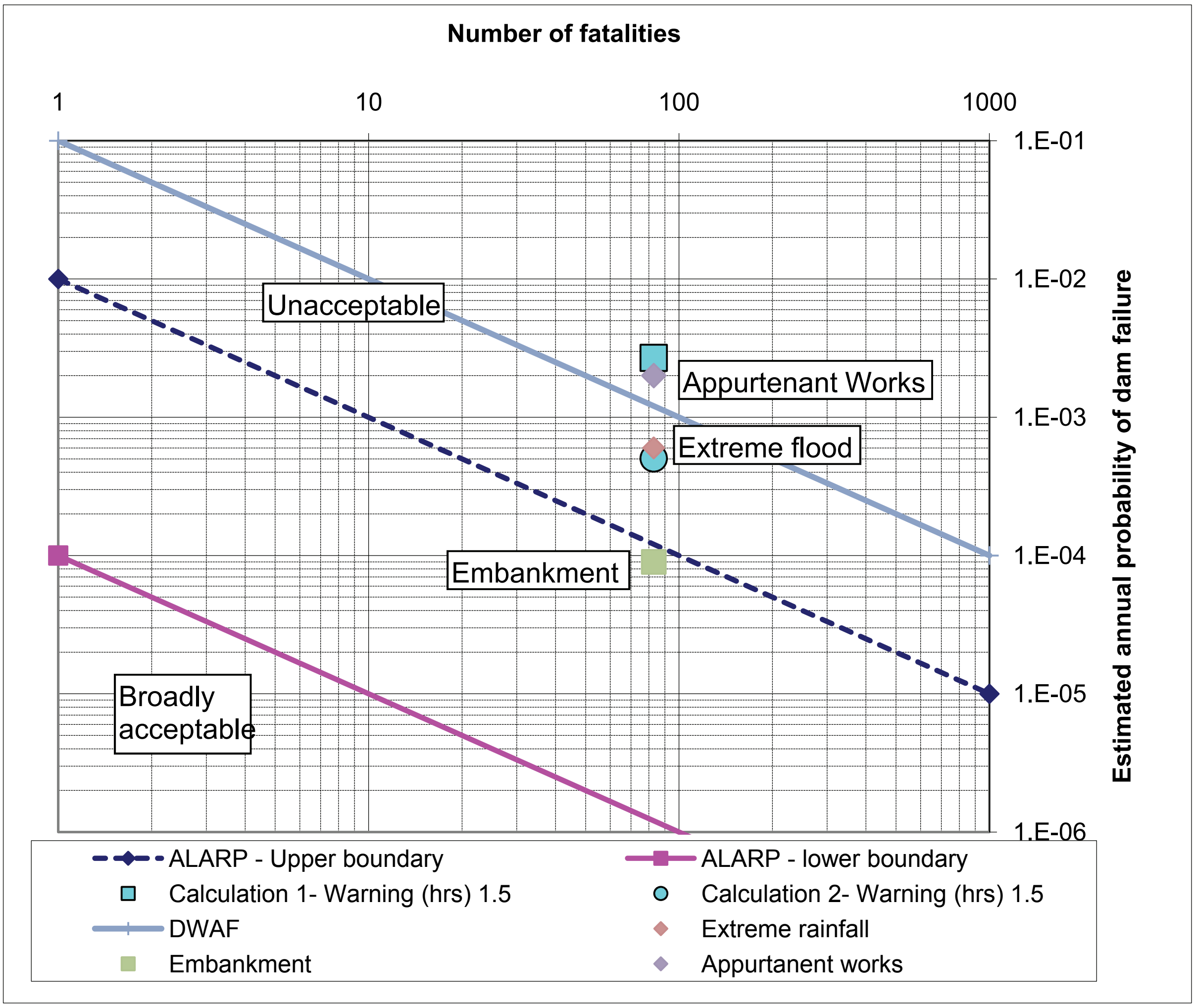


Figure 8.3-2 SANCOLD freeboard combination number 1 (25 year wind event and set-up = 1.72 m).

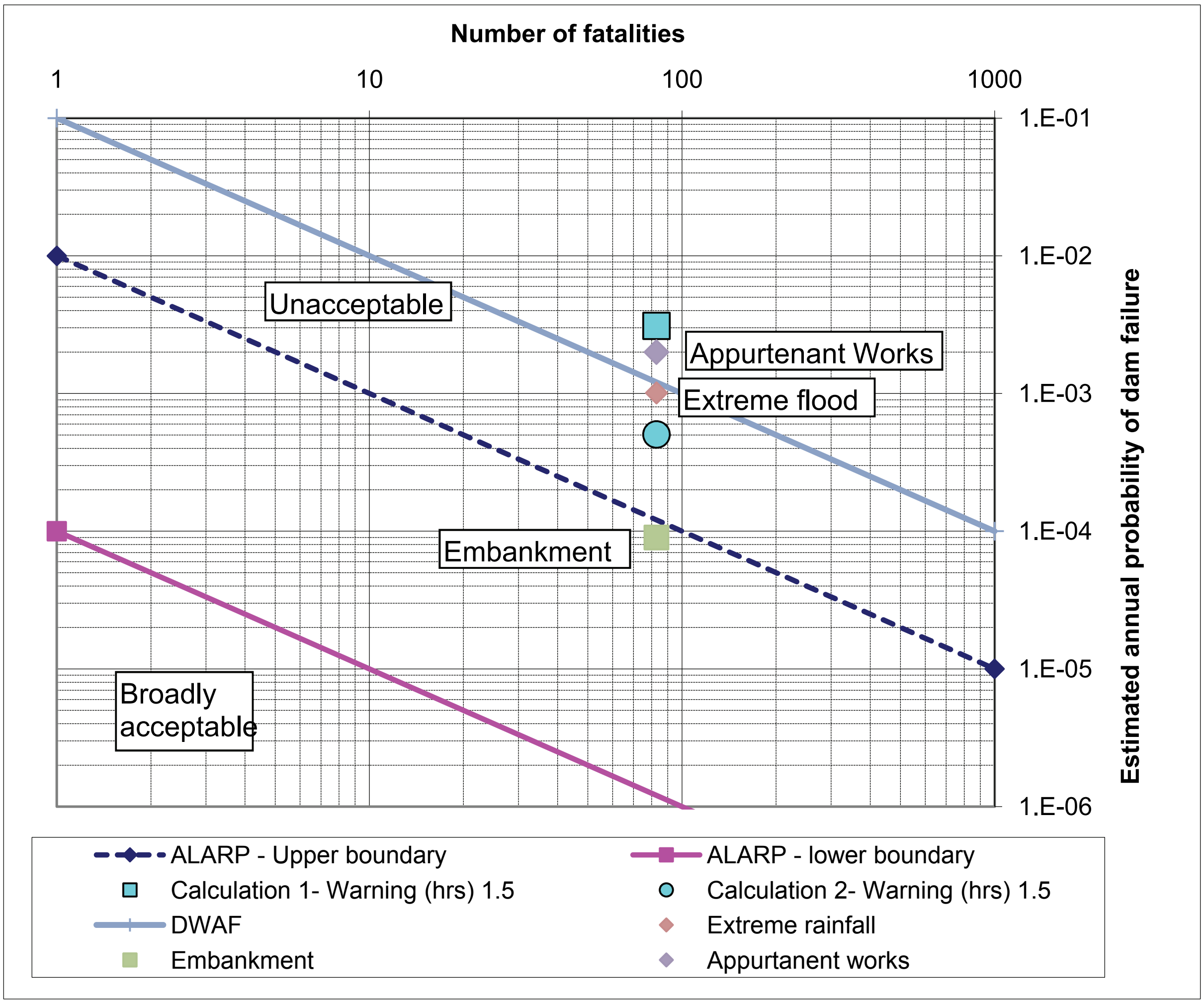


Figure 8.3-3 SANCOLD freeboard combination number 2 (25 year wind event, set-up and flood surge = 2.22 m).

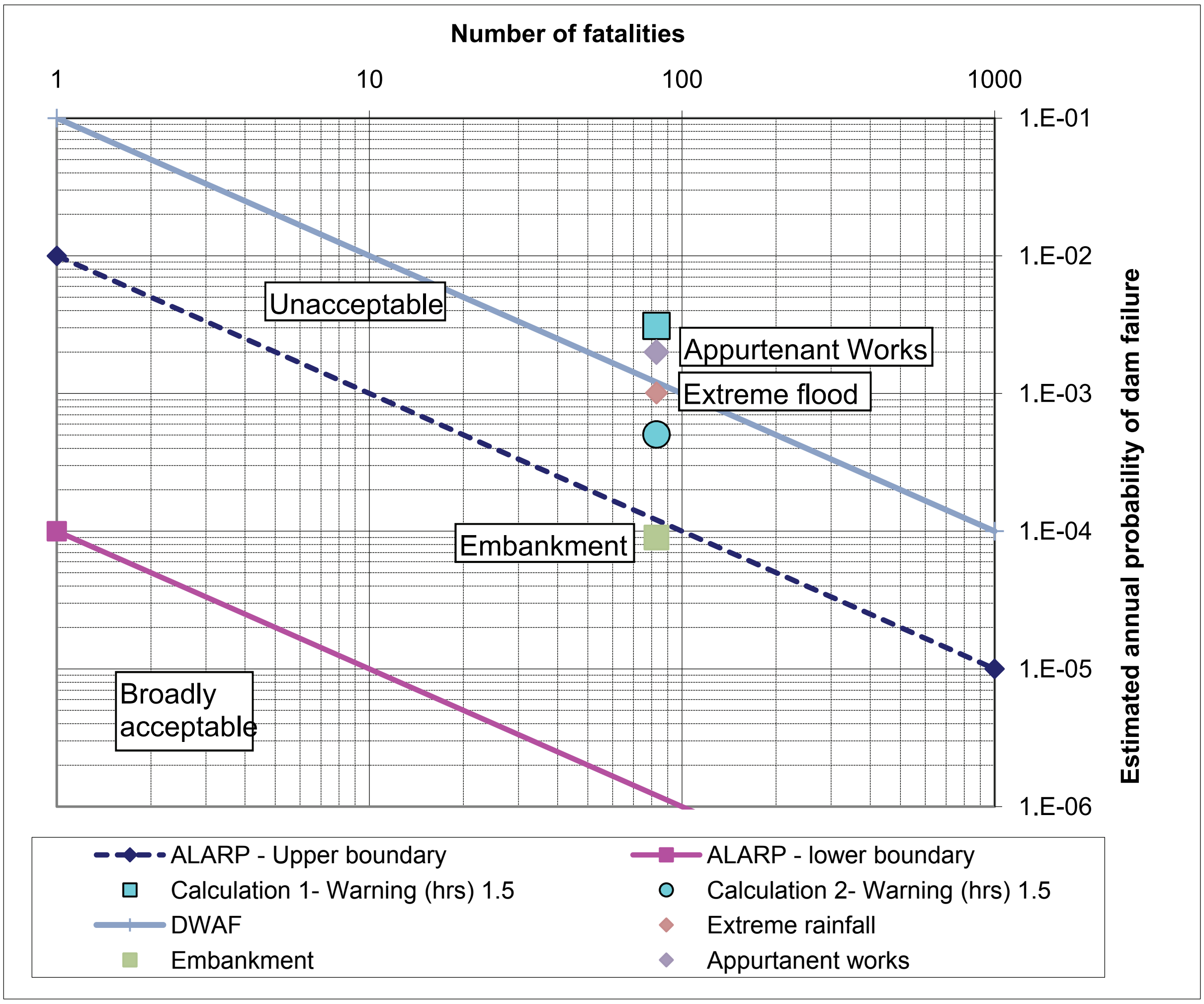


Figure 8.3-4 SANCOLD freeboard combination number 3 (100 year wind event, set-up and flood surge = 2.45 m).

8.3.4 SANCOLD vs. DEFRA-QRA

The DEFRA model focuses on the condition that the dam does break and it assigns an annual probability to the event. The SANCOLD model provides a conditional probability to determine what freeboard is required for the dam not to be exposed to the risk of breaking.
The result of the two approaches is that the DEFRA model allows flexibility regarding decision making in which of the various risk components need to be addressed to bring the dam within acceptable limits. The SANCOLD model allows no room for flexibility in the freeboard, either the dam complies with the freeboard requirements or it does not.

The advantage of the DEFRA model is that the dam owner can now decide how to spend his budget to get the most profit regarding risk reduction.

8.3.5 Conclusions

From the findings of this evaluation the following conclusions can be drawn:

- The DEFRA-QRA could be applied to Southern African dams with some modifications to the workbook as well as to the annual probability of failure due to appurtenant works.
- Freeboard components such as wind waves, set-up, seiches, etc. are not included directly in the extreme flood scenario, but could be incorporated.
- The DEFRA-QRA method could be used to complement established dam safety procedures such as the SANCOLD Freeboard guidelines.

8.3.6 Recommendations

From the conclusions of this evaluation the following recommendations are made:

- Adjust the DEFRA-QRA model to apply to South African dams to include annual probabilities of failure applicable to internal erosion of the embankments and also appurtenant works applicable to southern Africa.
- Include structural stability as a risk with an annual probability of occurrence.
- Use the revised SANCOLD Freeboard guidelines (which may include proposed modifications accepted by WRC) in the design of new dams. Include a row for the Safety Evaluation Flood in the Freeboard Component Table of DEFRA.
- Use the modified DEFRA-QRA in the safety evaluation of existing dams in Southern Africa.
- In the modified DEFRA-QRA, include the freeboard components as described in the SANCOLD Freeboard document and subtract this freeboard component from the NOC or parapet wall when calculating the dam critical flood.

9. DESIGN OF RIPRAP FOR WIND WAVE EROSION PROTECTION AT DAMS

9.1 INTRODUCTION

Riprap rock protection is one of the more frequently used methods to protect the upstream slope of earthfill (embankment) dam walls against wind waves action. This paper deals only with riprap protection against wind waves on the upstream slope of embankment dam walls (refer Figure 9.1-1 below for a typical cross section). Protection of the downstream slope of embankment dam walls against normal rain runoff and wind wave overtopping falls outside the scope of this document.

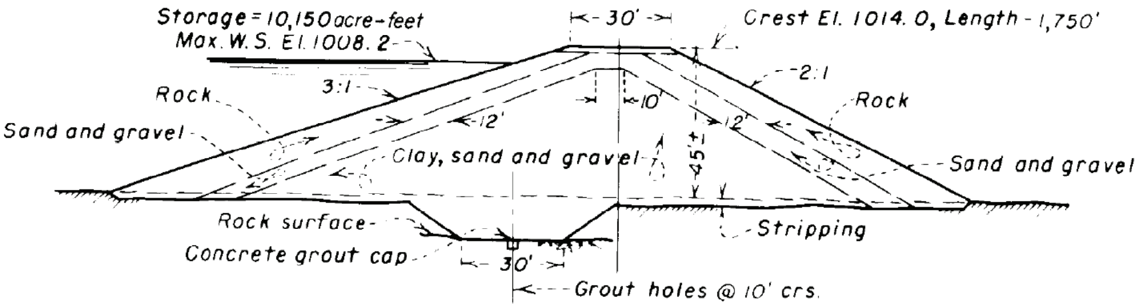


Figure 9.1-1 Typical illustrative example of a cross section of an earth fill (embankment) dam wall in imperial units with impermeable core and rock riprap protection against wind waves on its upstream slope (USBR, 1987)

The design aspects of the riprap armour, underlayer and filterlayer between the riprap layer and the impermeable core's upstream slope as well as the configurations of the riprap at both the bottom toe and crest are addressed.

An example of an embankment upstream riprap protection (comprising of a relative large rock grading) is presented in Figure 9.1-2 below.

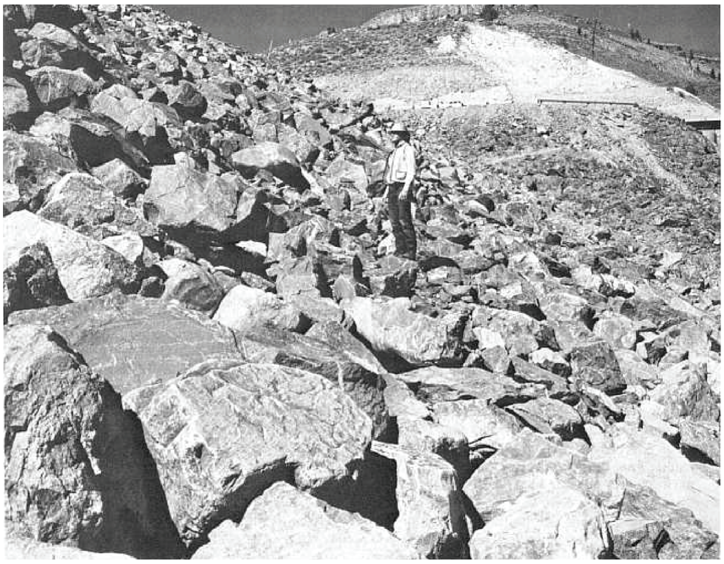


Figure 9.1-2 Example of relatively large rock grading as riprap protection on the upstream face of an embankment dam wall (USBR, 1987)

An example of a simple/generalised guideline to estimate riprap rock size and grading, which could be considered as a rule of thumb, is that given in USBR (1987), presented in Table 9.1.1 below.

Table 9.1.1 An Illustrative example in imperial units of generalised guidelines for thickness and rock gradation limits of a riprap protection layer on 1:3 slopes for angular quarried rock (USBR, 1987)

Reservoir fetch, mi	Nominal thickness inches	Weight of rock (in pounds) at various percentages (by weight)[1]			
		Maximum size	40 to 50%	50 to 60 %	[2]0 to 10%
≤2.5	30	2,500	>1,250	75 to 1,250	<75
>2.5	36	4,500	>2,250	100 to 2,250	<100

[1]Sand and rock dust shall be less than 5 percent, by weight, of the total riprap material.
[2]The percentage of this size material shall not exceed an amount that will fill the voids between the larger rocks.

This document presents both simplified and recently developed more refined formulae to estimate required rock riprap size. The document also attempts to integrate recent developments in coastal and river defence works in the design of riprap protection for dams.

Determination of site specific wind wave conditions falls outside the scope of this paper and will therefore not be addressed. Only relevant design wave parameters will be defined, more detailed information on wave parameters and rock characteristics can be found in The Rock Manual, (2007). The determination of dam wall freeboard (to prevent overtopping) will also not be treated in this document.

This chapter is structured as follows:

- Estimation of armour rock size, grading and layer thickness
- Definition of underlayer and filter layers
- Appropriate toe and crest configuration of riprap protection
- Survey methods for purposes of construction and long term monitoring for maintenance purposes

The main references which were consulted include:

- The Rock Manual (2007). Download website: www.ciria.org and www.cetmef.equipement.gouv.fr
- The Coastal Engineering Manual (CEM, 2006). Download website: http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=ARTICLES;104
- The Design of Small Dams (USBR, 1987). Download website: https://www.usbr.gov/

9.2 ESTIMATION OF ARMOUR ROCK SIZE, LAYER THICKNESS, UNDERLAYER AND FILTER LAYERS

9.2.1 Estimation of riprap rock size

To estimate riprap rock sizes under given wave conditions, formulae for preliminary estimate (first estimates in pre-feasibility stage) and formulae for more refined estimates (in the feasibility stage of a project) are treated below. For final design purposes the feasibility stage estimate of riprap size should be verified by model tests.

9.2.1.1 Estimate of riprap rock size for pre-feasibility purposes (modified Hudson formula):

The Hudson formula has been developed in the 1950's. The Hudson formula has however a number of limitations including: it has been derived from model tests using regular waves, it does not account for wave period and storm duration, it does not describe damage level, it is limited to non-overtopping and permeable structures only. However, because of the simplicity of the Hudson formula, it is suitable to determine a first estimate of riprap rock size in the pre-feasibility stage of a project.

Van der Meer and Van Gent (The Rock Manual, 2007; §5.2.2.2) modified the Hudson formula and recommended the following modified Hudson formula:

$$\frac{H_s}{\Delta D_{n50}} = 0.7\left(K_D \cot\alpha\right)^{1/3} S_d^{0.15}$$

Where:

H_s	=	significant wave height (average of largest third wave heights in a in a wave train, at the toe of the riprap slope)
K_D	=	stability coefficient [K_D =1 for structures with permeable core and K_D =4 for structures with impermeable core; such as in case of earthfill embankment dam walls]
S_d	=	damage level $A/(D_{n50})^2$, where *A* = erosion in cross-section (refer The Rock Manual, Eq 5.100 for detail definition; the value of S_d is normally taken as 2 for "start of damage")
D_{n50}	=	nominal median rock diameter, or equivalent cube size, $d_n = (M/\rho_S)^{1/3}$
Δ	=	relative mass density $(\rho_s - \rho_w)/\rho_w$ where ρ_s is mass density of stone and ρ_w is mass density of water

α = angle of the riprap slope

9.2.1.2 Estimate of riprap rock size for feasibility purposes

Two sets of formulae are recommended for estimation of riprap rock size, i.e. Van der Meer (1988) for deepwater conditions (where water depth at toe of riprap > $3H_{s,design}$, design) and the formulae derived by Verhagen & Mertens (2009) based on the re-analysis of the row data of Van der Meer (1988) and Van Gent (2003) and which are applicable to both shallow and deep water cases:

(a) Van der Meer (1988) – (The Rock Manual, 2007; §5.2.2.2):

The application of the Van der Meer (1988) formulae (for cases where the water depth at the toe of the rock protected slope is larger than 3 x Hs at the toe – termed deep water conditions) are recommended for the evaluation of rock size for riprap on embankment dams. These formulae include for the effect of storm duration, wave period, the structure's permeability and a clearly defined damage level. The formulae are presented in Boxes 9.2.1 below.

Box 9.2.1 Van der Meer formulae for deep water (non depth limited waves); (Excerpt from The Rock Manual,2007; Eqs 5.136 and 5.137)

For ***plunging waves*** ($\xi_m < \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_{pl}\, P^{0.18} \left(\frac{S_d}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5} \qquad (5.136)$$

and for ***surging waves*** ($\xi_m \geq \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_s\, P^{-0.13} \left(\frac{S_d}{\sqrt{N}} \right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^{P} \qquad (5.137)$$

where:

N	=	number of incident waves at the toe (-), which depends on the duration of the wave conditions *Refer The Rock Manual, 2007 for decision on value of N*
H_s	=	significant wave height, $H_{1/3}$ of the incident waves at the toe of the structure (m)
ξ_m	=	surf similarity parameter using the mean wave period, T_m (s), from time-domain analysis; $\xi_m = \tan\alpha/\sqrt{(2\pi/g \cdot H_s / T_m^2)}$ (-) *$Tm/Tp \approx 0.76$*
α	=	slope angle (°)
Δ	=	relative buoyant density, ρ_r/ρ_w - 1 (-)
P	=	notional permeability of the structure (-); the value of this parameter should be: $0.1 \leq P \leq 0.6$ (see Figure 5.39) **NOTE:** the use of a geotextile reduces the permeability, which may result in the need to apply larger material than without a geotextile.
c_{pl}	=	6.2 (with a standard deviation of $\sigma = 0.4$; see also Table 5.25) *Refer The Rock Manual, 2007 for 95% confidence deign.*
c_s	=	1.0 (with a standard deviation of $\sigma = 0.08$).

Box 9.2.1 (continued)

The transition from plunging to surging waves is **derived from the structure slope (not from the slope of the foreshore)**, and can be calculated with Equation 5.138, using a *critical* value of the surf similarity parameter, ξ_{cr} :

$$\xi_{cr} = \left[\frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan\alpha} \right]^{\frac{1}{P+0.5}} \qquad (5.138)$$

For $\xi_m < \xi_{cr}$ waves are **plunging** and Equation 5.136 applies.
For $\xi_m \geq \xi_{cr}$ waves are **surging** and Equation 5.137 applies.

NOTE: For slope angles more gentle than 1:4 ($\cot\alpha \geq 4$) only Equation 5.136 (for plunging waves) should be used, irrespective of whether the surf similarity parameter, ξ_m , is smaller or larger than the transition value, ξ_{cr} .

The definition and applicable range of the parameters in the Van der Meer formulae as well as calculation steps to follow and a worked example are presented in detail in Section 5.2.2.2 of The Rock Manual (2007).

(b) Verhagen & Mertens (2009)

Verhagen & Mertens (2009) re-analysed the raw data of Van der Meer (1988) and Van Gent (2003) and unified the deepwater and shallow water cases in one set of formulae applicable to both deep water and shallow water cases. These formulae (presented below) are therefore an update and improvement of the shallow water formulae of Van der Meer, i.e. Equations 5.139 and 5.140 in The Rock Manual (2007).

$$\frac{H_{2\%}}{\Delta D_{n50}} = (1+c_f \xi_\beta)\frac{c_{pl}}{\gamma_{Latham}}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(\xi_{H_{2\%},T_{m-1,0}}\right)^{-0.5} P^{0.18}$$

[for plunging waves ($\xi_{H_{2\%},T_{m-1,0}}$ < ξ_{cr}) as defined for Van der Meer (1988) formulae]

$$\frac{H_{2\%}}{\Delta D_{n50}} = (1+c_f \xi_\beta)\frac{c_s}{\gamma_{Latham}}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(s_{m-1,0}\right)^{-0.25} \left(\xi_{H_{2\%},T_{m-1,0}}\right)^{P-0.5} P^{-0.13}$$

[for surging waves ($\xi_{H_{2\%},T_{m-1,0}}$ ≥ ξ_{cr}) as defined for Van der Meer (1988) formulae]

Definition of parameters:

$H_{2\%}$	=	wave height exceeded by 2% of waves in a wave train (at the toe of the riprap slope). In the case of waves breaking upstream of the toe of the riprap slope, $H_{2\%}$ and $T_{m-1,0}$ can be determined by means of the freeware program SWAN1D at http://www.kennisbank-waterbouw.nl/Software/index.htm
$T_{m-1,0}$	=	period of waves, calculated from the first negative moment of the wave spectrum (at the toe of the riprap slope) – Refer The Rock Manual for detail definition
$s_{m-1,0}$	=	fictitious wave steepness: 2π $H_{2\%}$ /(g $T^2_{m-1,0}$)
$\xi_{H_{2\%},T_{m-1,0}}$	=	surf similarity parameter; tan α / $\sqrt{s_{m-1,0}}$
D_{n50}	=	nominal median block diameter, or equivalent cube size, $d_n = (M/\rho_S)^{1/3}$
Δ	=	relative mass density $(\rho_s - \rho_w)/\rho_w$ where ρ_s is mass density of stone and ρ_w is mass density of water
N	=	number of waves in a wave storm (Refer The Rock Manual for value of N)
S	=	damage level $A/(d_{n50})^2$, where *A* = erosion in cross-section
α	=	angle of the seaward slope of a structure
P	=	notional permeability coefficient
c_{pl}	=	8.4 with a standard deviation, σ = 0.7

c_s	=	1.3 with a standard deviation, σ = 0.15
c_f	=	0.035
ξ_β	=	surf similarity parameter calculated with the slope of the foreshore, β, the local $H_{2\%}$ and the local $T_{m-1,0}$
γ_{Latham}	=	1/(coefficient given for c_{pl} or c_s in Table 5.30 in The Rock Manual, 2007)

9.2.2 Rock grading and layer thickness of riprap layer

The Hudson, Van der Meer and Verhagen formulae which are presented in Section 9.2.2 give an estimate of the required riprap armour size, Dn50. The rock grading for the estimated Dn50 rock size of the riprap layer can be defined in accordance with European Norm, EN 13383 – see Box 9.2.2 below for guidelines.

Box 9.2.2 EN 13383 system for standardisation of rock gradings (Excerpt from Rock Manual, 2007 – §3.4.3.2)

EN 13383 divides armourstone products into:

- **Heavy gradings ("HM")** for larger sizes appropriate for armour layers – normally handled individually
- **Light gradings ("LM")** appropriate for armour layers, underlayers and filter layers – produced in bulk, usually by crusher opening and grid bar separation
- **Coarse gradings ("CP")** often used for filter layers – of such a size that all pieces can be processed by production screens with square openings (ie typically less than 200 mm).

The system for defining heavy gradings requirements is based on setting limit values with an associated percentage passing by mass (see Figure 3.19). A set of nominal limits corresponds to the target size of the armourstone. A set of extreme limits corresponds to tolerances. The standard grading requirements and associated passing values are summarised in Table 3.5.

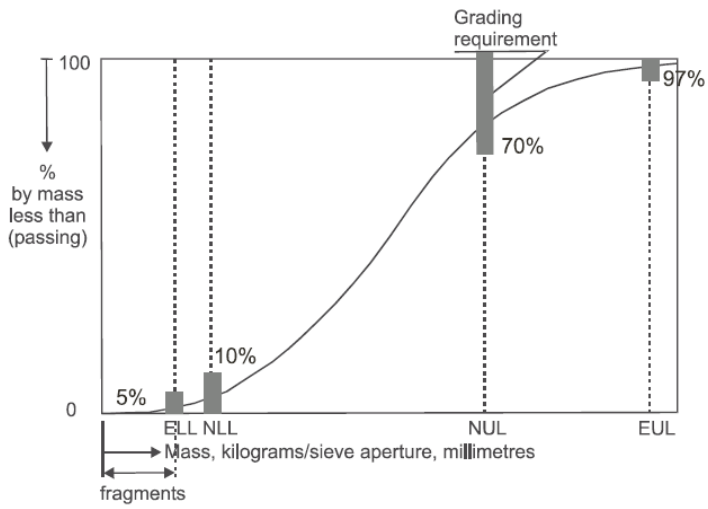


Figure 3.19 *System for limits of EU standard gradings – percentages of passing as given are for heavy grading*

For **heavy gradings**, the associated limits are:

- ELL (Extreme Lower Limit) – the mass below which no more than 5 per cent passing by mass is permitted
- NLL (Nominal Lower Limit) – the mass below which no more than 10 per cent passing by mass is permitted
- NUL (Nominal Upper Limit) – the mass below which no less than 70 per cent passing by mass is permitted
- EUL (Extreme Upper Limit) – the mass below which no less than 97 per cent passing by mass is permitted.

In Table 3.5 limits for M_{em} are also given, defined as *effective mean mass*, ie the average mass of a sample of stones without *fragments* (those below the ELL-value of the grading, see Section 3.4.3.5).

The layer thickness, t_d, for the riprap layer can be derived according to equation below (refer Equation 3.25 in §3.5.1 in The Rock Manual (2007)):

$$t_d = n.k_t.\ D_{n50}$$

where:
n = number of layers (normally a minimum of two layers)
k_t = porosity coefficient (refer §3.5.1 in The Rock Manual for definition and values as a function of blockiness and length to thickness ratio)

9.2.3 Underlayer and filter layer

Underlayer:

In the case of heavy graded riprap one or more underlayers will be necessary to enable the specification of a practical grading for filter(s) between the smallest underlayer and the clay core. The minimum underlayer thicknesses should be based on a two layer system, i.e. n =2.

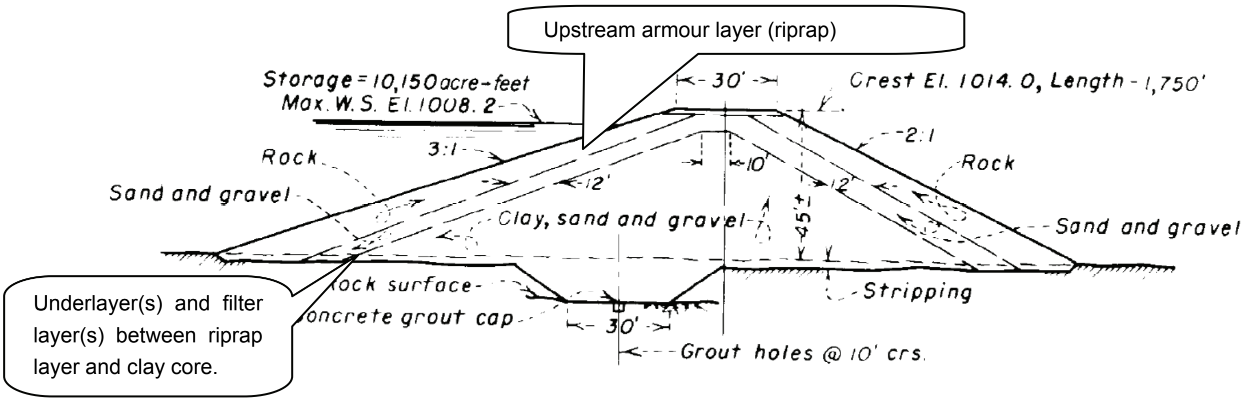


Figure 9.2-1 Typical dam cross-section indicating riprap layer (armour) relating to the underlayer

Underlayers (and filter layers) should be designed to prevent the transport of finer material below it, but allow for movement of water through it without loss of material from it.

The size of the rock in the underlayer can be estimated, using the criterion in Box 9.2.3 below.

The thickness of the underlayer can be calculated similar to that given under Section 9.2.2 for riprap thickness, with at least a double layer, i.e. “n” ≥ 2.

Box 9.2.3 Estimation of underlayer rock size (excerpt from The Rock Manual, 2007).

Rock structures are normally constructed with an armour layer (often a double layer, $2k_t D_{n50}$ thick, where k_t is the layer coefficient (-), see Section 3.5.1), one or more thin granular underlayers or filters, and a core of rather finer material. The core may consist of quarried rock (quarry run) or clay or sand. A geotextile filter may be placed between core and granular layers.

The *Shore protection manual* (SPM) (CERC, 1984) recommends for the ratio of the stone mass of the underlayer M_{50u} (t), and that of the armour, M_{50a} (t), a value as indicated in Equation 5.192:

$$\frac{M_{50u}}{M_{50a}} = \frac{1}{15} \text{to} \frac{1}{10} \qquad (5.192)$$

This criterion (Equation 5.192) could also be expressed in terms of D_{n50} :

$$\frac{D_{n50a}}{D_{n50u}} = 2.2 \text{ to } 2.5$$

The grading specification of the underlayer should be based on the EN 13383 standard rock gradings in Section 9.2.2 (Box 9.2.2).

Filter layers:

Requirements for filter layers are described in several guidelines including that of ICOLD (1994) and HEC 11 (1988). The criteria for filter layers according to HEC 11 (1988) are presented here as a guideline:

"For rock riprap, a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15 percent particle size (D15) of the coarser layer to the 85 percent particle size (D85) of the finer layer. An additional requirement for stability is that the ratio of the 15 percent particle size of the coarser material to the 15 percent particle size of the finer material should exceed 5 but be less than 40 (32). These requirements can be stated as:

$$\frac{D_{15}(Courserlayer)}{D_{85}(Finerlayer)} < 5 < \frac{D_{15}(Courserlayer)}{D_{15}(Finerlayer)} < 40 \qquad \textit{Equation A}$$

The left side of the inequality in Equation A is intended to prevent piping through the filter, the center portion provides for adequate permeability for structural bedding layers, and the right portion provides a uniformity criterion. If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter material if more than one layer is used, and between the filter blanket and the riprap cover or underlayer.

In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of fine material from the finer layer to the coarser layer. Not more than 5 percent of the filter material should pass the No. 200 sieve. The thickness of the filter blanket should range from 150 mm to 380 mm for a single layer, or *from 100 mm to 200 mm for individual layers of a multiple layer blanket. Where the gradation curves of adjacent layers are approximately parallel, the thickness of the blanket layers should approach the minimum. The thickness of individual layers should be increased above the minimum proportionately as the gradation curve of the material comprising the layer departs from a parallel pattern".*

9.3 CREST AND TOE CONFIGURATION OF RIPRAP PROTECTION

9.3.1 Crest configuration

The termination of riprap protection at the crest of a dam wall is important for the extreme case where some wave overtopping might occur. A formal termination of the riprap layer in the form of a horizontal shoulder of width ≈ $3D_{n50}$ is advised because of the overtopping absorption capability of such a configuration. This could be terminated with a concrete wall (e.g. an L-shape wall) as shown schematically in Figure 9.3.1 (left). The concrete wall in configuration C in Figure 9.3.1 (left) could either have a simple L-shape configuration or a more sophisticated form as shown in Figure 9.3.1 (right). The latter type of wall will deflect the wave run-up, that could overtop the dam wall, towards the upstream direction. The configuration of this type of wave wall (normally mass concrete) should be optimised by means of physical model tests to ensure effective functioning.

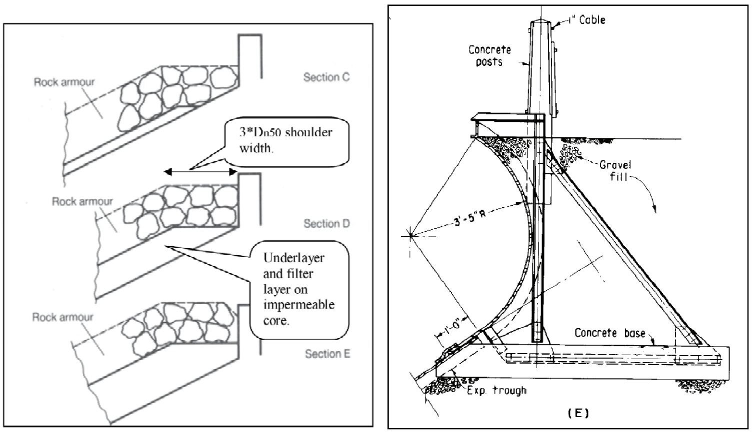


Figure 9.3-1 Termination of riprap at dam crest (Rock Manual, 2007 – on left) and (USBR, Design of small dams, 1987 – on right)

9.3.2 Toe configuration

Since the toe of an earth fill dam is vulnerable when wave attack on it is in relative shallow water, its deterioration could undermine the riprap protection which could lead to failure of the riprap layer above that undermined portion. For this reason, it is recommended that special attention be given to the termination of the toe part of the riprap layer.

Examples of appropriate termination of toe armour (riprap) is presented in Box 9.3.1 below.

Box 9.3.1 Examples of appropriate termination of toe armour (riprap) on a rock and a gravel foreshore; (The Rock Manual, 2007)

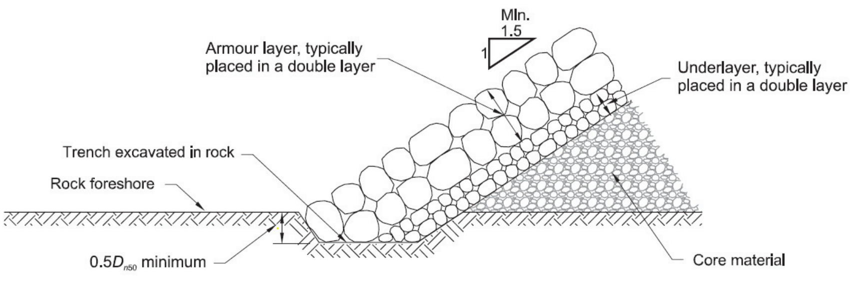


Figure 6.58 *Toe detail 1b: rock foreshore – excavated trench*

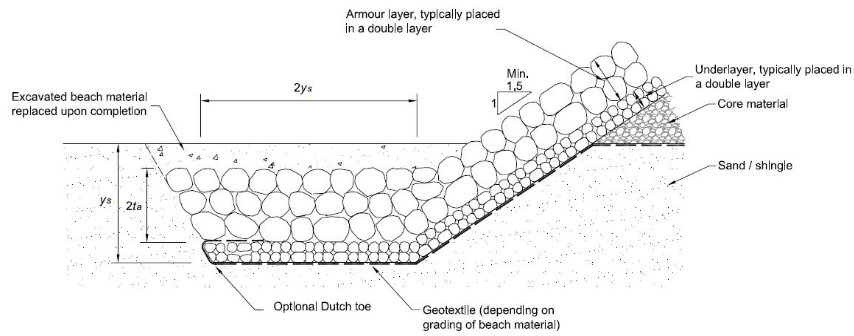


Figure 6.63 *Toe detail 3c: severe scour potential – excavated trench*

9.4 SURVEY METHODS TO CHECK CONSTRUCTION ACCURACY AND MONITORING OF RIPRAP LAYER FOR MAINTENANCE PURPOSES

9.4.1 Survey method to check construction accuracy:

The control of riprap and under layers of rock with relative large sizes could use the ball-and-crane method as a control to ensure that the design profile and thickness is accomplished by the contractor. This method is used in construction of a rubble mound breakwater in the marine environment.

The ball and crane survey method uses a spherical bottom probe with the diameter of the sphere = $0.5D_{n50}$. The top of the underlayer would be surveyed first and after placement of the rip rap armour layer the top of the rip rap will be surveyed (both using the ball and crane method). Spacing of survey spots is normally taken as D_{n50} in both x and y directions of a rectangular survey grid. The profiles of underlayer and rip rap layer would then define the layer thickness and could be compared with the design layer thickness before acceptance of the constructed rip rap layer.

Figure 9.4.1 illustrates the crane and ball survey method.

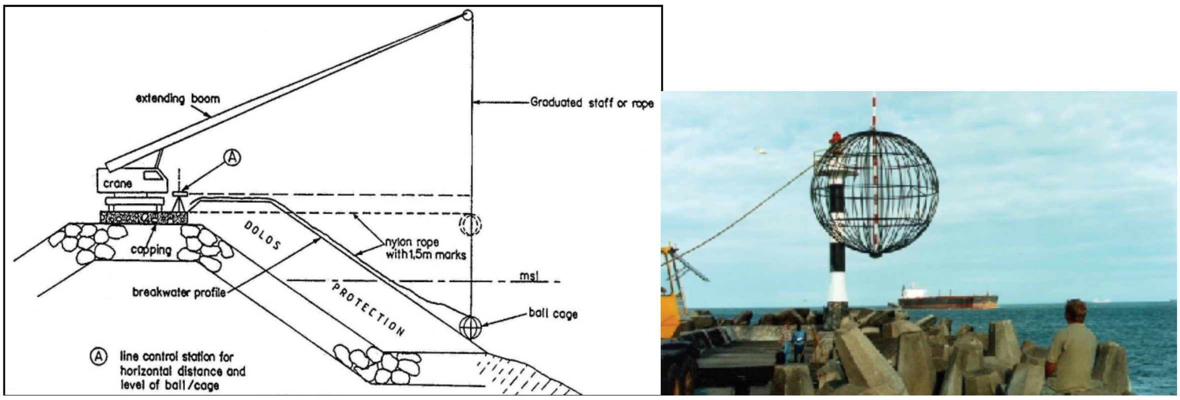


Figure 9.4-1 Illustration of ball and crane survey of armour profile (horizontal square survey grid, with spacing = D_{n50} and ball diameter = $0.5D_{n50}$); (CSIR 2008)

9.4.2 Survey method to monitor of riprap layer for maintenance purposes:

The monitoring of the state of a riprap rock protection layer on the upstream slope of a dam wall is useful to decide if maintenance to the riprap layer is required. One of the methods used in the monitoring of rubble mound breakwaters in the marine environment is a photographic method utilising a helicopter. The Helicopter takes up a predetermined position (x, y, and z) with the aid of an on board GPS system. A photo is then taken of the surface of the slope protection. In this way a panoramic coverage (with overlap) of the entire riprap surface is obtained. For comparison purposes photographs are taken at subsequent surveys from the same locations. The method is illustrated in Figure 9.4.2 below.

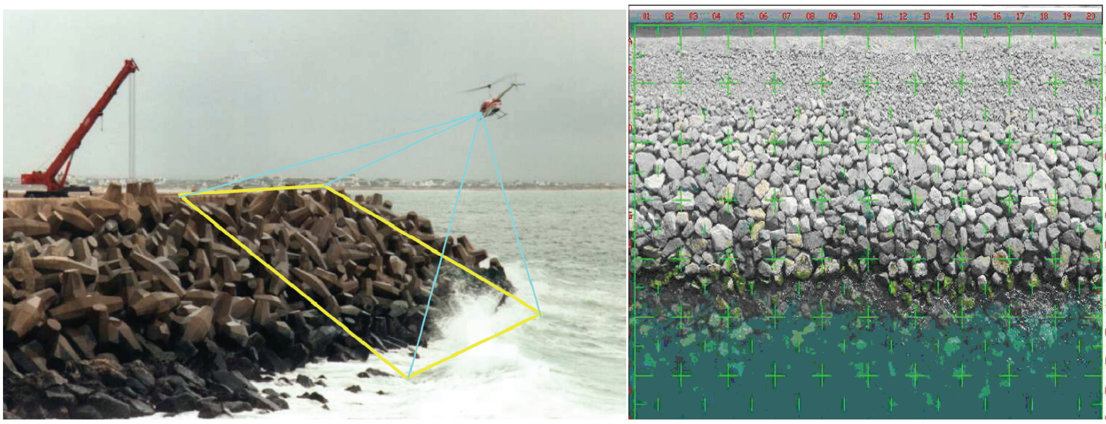


Figure 9.4-2 Illustration of an aerial photographic survey method by helicopter (left) and an example of one of a series of overlapping photographs that is obtained from the helicopter survey method for analysis; (CSIR 2008)

A laser scanner method to cover the entire rip rap surface is currently in an advance stage of development by the CSIR. The laser scanner method obtains a relatively dense cloud of coordinates of the exposed rock slope. From these coordinates a detailed contour map could be generated, which enables comparison of subsequent surveys to quantify differences between subsequent surveys.

9.5 SUMMARY AND CONCLUSION

In conclusion the following remarks, regarding the design of rock riprap for erosion protection against wind wave action on the upstream face of embankment dam walls, could be made:

a) Formulae to estimate rock size for riprap protection on upstream slopes of embankment dam walls to withstand a selected wave condition, are the formula of Hudson as modified by Van der Meer (1988) for pre-feasibility estimates, the deepwater formulae of Van der Meer (1988) and the recently developed formulae by Verhagen & Mertens (2009) applicable to both deep water and shallow water cases. The Van der Meer and Verhagen & Mertens formulae are considered more accurate than the Hudson formula and could be used in feasibility studies. Final designs should however be verified by means of physical model studies.
b) From the rock size (Dn50), derived for riprap protection on a dam wall's upstream slope, rock grading (according to the European Norm – EN 13383) and corresponding riprap layer thickness can be derived from information provided in The Rock Manual (2007).
c) The underlayer rock size and grading can also be derived with guidance information in The Rock Manual (2007).
d) Information is also provided to design a granular filter layer between the underlayer and the impermeable core of an embankment dam wall.
e) Detail is given on configurations to effectively terminate the riprap upstream wall face protection at the toe and crest of the embankment dam wall.
f) A method to survey riprap and underlayer thickness during construction is presented.
g) A method to monitor the stability of a rock riprap layer, for maintenance purposes, is also described.

10. CONCLUSIONS AND RECOMMENDATIONS

The key findings of this study are:

a) The 1990 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 with the addition of cyclone data along the East Coast. Such regionalized maps could be used for planning purposes of category I and II dams, but for any detail design studies local wind data should be analysed.

c) For planning purposes and for the detail design of category I dams wind wave height, run-up and set-up calculation should be based on the Rock Manual. In all other cases the SWAN model should be used to determine wind wave height.

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 $H_{2\%.}$

d) 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 (1D) model could be used, but for category II and III dams 2D or 3D models are recommended.

e) The design of riprap at dams for protection against wind wave erosion should be based on the Rock Manual.

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 using a revised DEFRA methodology to incorporate component scenarios could be used complementary to the deterministic approach and is recommended for detail design studies of category II and III dams.

11. REFERENCES

CEM (2006). Coastal Engineering Manual. U.S. Army Corps of Engineers (USACE). Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

DELFT 3D [Web address: http://www.delftcluster.nl/website/nl/page107.asp]

DCM (2006). Wind and precipitation design criteria for freeboard. South Florida Water Management District (SFWMD).

DEFRA (2002). R*esearch Contract* “Reservoir Safety – Floods and Reservoir Safety Integration”. Ref. XU0168 Rev A05. August 2002.

ESDU (1990). Mechanisms leading to high wind speeds.

ICOLD, 1994, Embankment dams granular filters and drains; Review and recommendations (Bulletin No 95). HEC 11, 1988, Design of Riprap Revetment, USA Federal Highway Administration.

ICOLD (2003). Dams and Floods Guidelines and Case Studies. International Commission on Large Dams. http://www.icold-cigb.org/

Kamphuis. (2002). Introduction to Coastal Engineering and Management. Advanced Series on Ocean Engineering (Volume 16) ISBN 981-02-3830-4. Website:
http://www.worldscibooks.com/engineering/4064.html

Meteosat (2003). Website: http://www.sierrapapa.it/e_meteos.htm

MIKE11, MIKE 21 and MIKE 3.
[Website: http://www.dhigroup.com/Software/Marine/MIKE21.aspx]

Milford (1987). Maximum wind speeds for South Africa. The Civil Engineer in South Africa.

Pullen et al. (2007). Wave Overtopping of Sea Defences and Related Structures. EurOtop, Assessment Manual, August 2007. Website: http://www.overtopping-manual.com/index.html

Rossouw. C (1999). The probability of occurrence and the intensity of tropical cyclones along the Southern African East Coast. Master’s Thesis. University of Stellenbosch.
SANCOLD (1990). Interim Guideline on Freeboard for Dams. Safety Evaluation Report No 3. The South African National Committee on Large Dams. Pretoria. September 1990.

SANCOLD (1990). Interim Guidelines on Freeboard for Dams. Safety Evaluation Report No.3 The South African National Committee on Large Dams. Pretoria. September. 1990.

Shore Protection Manual (1984). Department of the Army. Waterways Experiment Station. U.S. Army Corps of Engineers (USACE). Coastal Engineering Research Centre.

Stormsurf (2007). Website: http://www.stormsurf.com/mdls/menu_wx.html

STWAVE. Coastal and Hydraulics Laboratory – Engineer Research and Development Canter. Waterways Experiment Station – Vicksburg, Mississippi – US Army Corps of Engineers. Website: http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Software;9

SWAN (Booij et al., 2006). Website: http://vlm089.citg.tudelft.nl/swan/index.htm

The Rock Manual (2007). The use of rock in hydraulic engineering (2nd edition). CIRIA; CUR; CETMEF. London, 2007. ISBN 978-0-86017-683-1.
(<www.ciria.org><www.cetmef.equipement.gouv.fr>).

Tyson et al. (2000). The Weather and Climate of Southern Africa. A book published by Oxford University Press.

USACE, 2006, The Coastal Engineering Manual (2006). Website:
http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=ARTICLES;104

USBR, 1987, The Design of Small Dams. Website: https://www.usbr.gov/

Verhagen & Mertens, 2009, Riprap Stability for deep water, shallow water and steep foreshores. ICE conference, September 2009, Edinburgh. CIRIA et al., 2007, The Rock Manual. **Website:** www.ciria.org **and** www.cetmef.equipement.gouv.fr

Wium (2007). Personal communication (In preparation for publication).

WRC Report No 1420 (2007). Review of the Selection of Acceptable Flood Capacity for Dams in South Africa in the context of Dam Safety. ISBN 978-1-77005-571-1.

WRC Report No K5/1759/1 (2008). Review of the SANCOLD Guidelines on Freeboard for Dams; Part 1 – Literature Review

Appendix A: South Florida Water Management District (SFWMD) – Design Criteria Memorandum: DCM-2 (2006)

Design Criteria Memorandum: DCM-2

Effective Date: February 6, 2006 **Acceler8 Ref #: P 599**

Subject: Wind and Precipitation Design Criteria for Freeboard

1. Introduction

DCM – 2 presents the SFWMD design criteria for the determination of the wind speed coincident with precipitation and/or normal full storage level. Determination of the controlling conditions for establishing freeboard on storage impoundments will be defined and detailed. This memorandum is an overview and is not intended to be a comprehensive presentation of hydrologic design criteria.

Freeboard allowance is determined from wave characteristics, wind setup and wave run-up, as well as a several other factors. This memorandum addresses the combined wind and precipitation design criteria for freeboard determination of high, significant and low hazard dams. Dams assigned the high hazard potential classification are those where failure or mis-operation will probably cause loss of human life (FEMA 333). Significant hazard dams will have similar freeboard design criteria because of potential for high economic losses or loss of highly valued environmental ecosystems. Low hazard dams may have different freeboard design criteria based on socio-economic analysis performed on a case-by-case scenario. Hazard potential classification is covered in **DCM – 1**.

This memorandum presents the joint criteria and recommendations developed by the South Florida Water Management District (SFWMD), the U.S. Army Corps of Engineers (USACE) and the Florida Department of Environmental Protection (FDEP) for developing wind speed and precipitation events for freeboard determination of CERP impoundments.

2. Freeboard Determination

Freeboard is defined as the vertical distance between the crest of a dam and some specified pool level, usually the normal full storage level or the maximum flood level (USACE, 1997). Freeboard and flood control storage are required to provide the capacity to store and/or route the design storm through the reservoir considering inflows, precipitation on the reservoir basin and wind generated waves without overtopping of the embankment. The objective in selection of design freeboard is to assure that failure of the dam will not result from wind setup, wave action, uncertainties in analytical procedures, and uncertainties in project function in combination with the most critical pool elevation (USACE, 1991). Freeboard determination should be based on site-specific conditions that can be reasonably expected to occur simultaneously. Settlement, waves induced by potential landslides, or malfunction of the spillway and/or outlet works are also important considerations for freeboard determination. This memorandum identifies and develops design criteria for logical combinations of reservoir levels/precipitation, and wind conditions for freeboard determination. Freeboard is shown graphically on **Figure DCM 2-1**. The reference elevation for freeboard determination will be the “normal full storage level” (NFSL) elevation which is defined as the maximum elevation of storage at an un-gated, uncontrolled level where outflow would begin with continued storage. For a reservoir with outflow

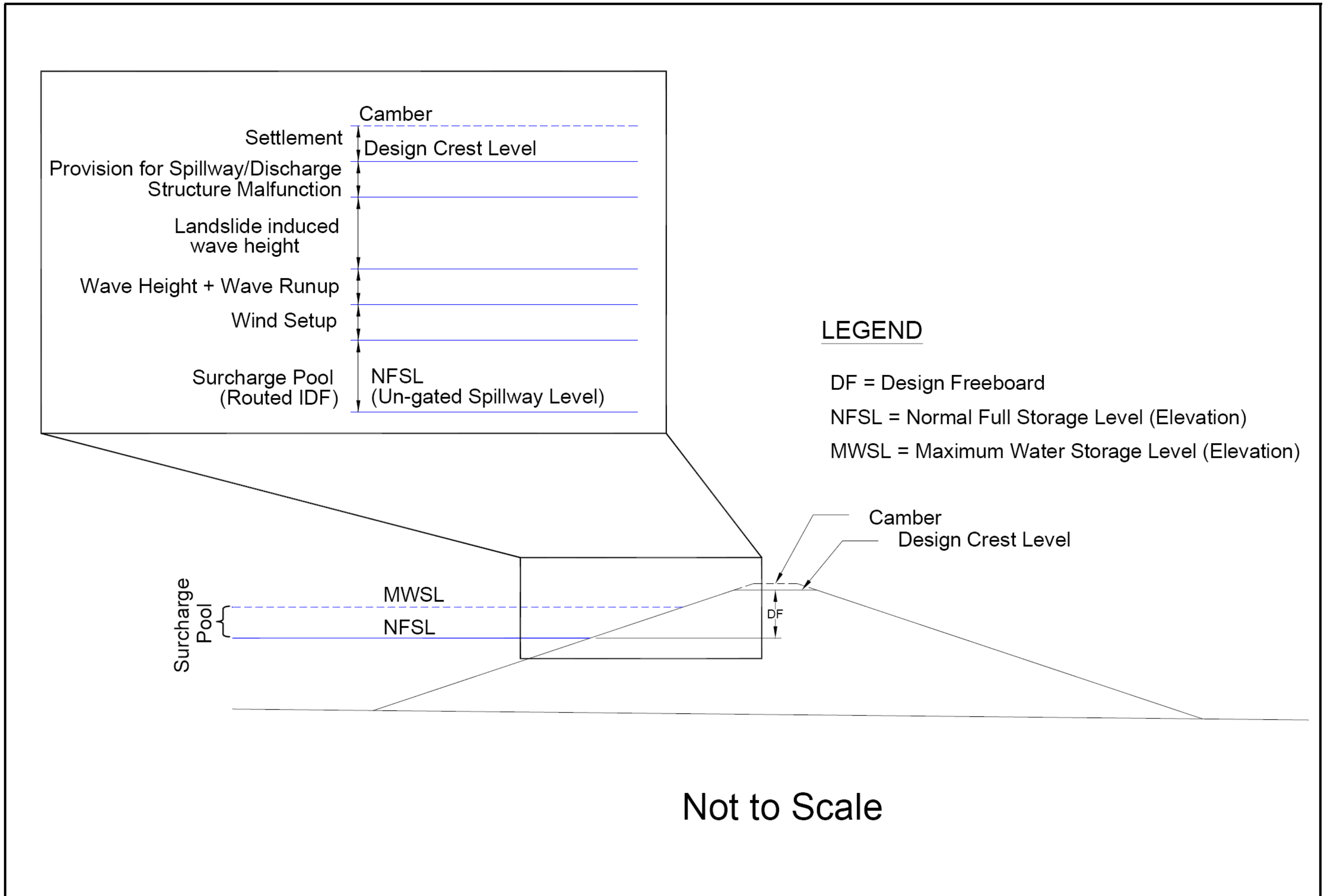


FIGURE DCM 2-1

DESIGN FREEBOARD

controlled in part by moveable gates or other means, the NFSL is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge. The maximum water storage level (MWSL) is defined as the maximum (still-water) level resulting from routing of the inflow design flood (IDF). Surcharge pool is defined as the storage between the normal full storage elevation and the maximum water storage level. Spillway design and routing of the IDF is an integral part of freeboard determination and is to be included in design of the impoundments. The IDF, wind and wave design requirements for CERP impoundments are included in **DCM-2**. Spillway requirements for CERP impoundments are included in **DCM-3**.

Embankment freeboard will be determined as the minimum required embankment height above the reference pool level (normal full storage level) to prevent overtopping from the defined condition. As defined by the USACE (1997):

Freeboard is generally based on probable maximum wind conditions when the reference elevation is the normal operating level. When estimating the reservoir level to be used with the probable maximum reservoir level, a lesser wind condition is used because it is improbable that maximum wind conditions will occur simultaneously with the maximum flood level.

Neither the probable maximum wind (PMW) condition nor the lesser wind condition associated with the maximum flood level is numerically defined in any standard reference document. In addition, the duration of the flood level near the maximum pool level needs to be included in the freeboard determination. As described in ER 1110-2-8 (FR) paragraph 9c., *"when the IDF pool hydrograph is within three feet of the maximum pool for 36 hours or longer or where the project has been designed with little surcharge for the maximum pool above full pool elevation, the minimum freeboard will be five feet for embankment dams and three feet for concrete dams or greater …."*

Therefore, design criteria to be used for high and significant hazard water storage impoundments and dams for the CERP program were developed as detailed herein. Design criteria for impoundments classified as low hazard potential are also included.

2.1 HIGH AND SIGNIFICANT HAZARD POTENTIAL

Several impoundments to be constructed for CERP will be high or significant hazard structures with the IDF defined as the flood resulting from the Probable Maximum Precipitation (PMP) event. Determination of design criteria will include combinations of reservoir level, precipitation and wind conditions that are reasonably expected to occur simultaneously or within 36 hours of the maximum surcharge pool level. A list of four combinations of precipitation and wind conditions that could be reasonably expected to occur are presented in **Table DCM 2-1.** Each combination of precipitation and wind will be hereafter referred to as **Case *x***. These four cases are also described in the following paragraphs.

- **Case 1** consists of a 100-year wind combined with the PMP.
- **Case 2** consists of a 100-year precipitation event combined with a Category 5 hurricane, wind condition.
- **Case 3** consists of a PMW combined with the normal full storage level, and
- **Case 4** consists of a storm specific (historical) event.

Case 1 (the 100-year wind combined with the PMP) and **Case 3** (a PMW assumed to be 200 mph wind combined with the normal full storage level) are consistent with the USACE criteria (USACE, 1997). The remaining two cases (**Case 2** and **Case 4**) were developed to address specific combinations of wind and precipitation that have occurred or could reasonably be expected to occur in southern Florida.

Table DCM 2-1: Reservoir Level/Precipitation and Wind Conditions Reasonably Expected to Occur Simultaneously for Freeboard Determination

		Precipitation or Reservoir Level		
		Normal Full Storage	100-Year Storm	PMP
Wind Condition	100-Year Wind			▤
	156 mph Category 5		▤	
	200 mph (PMW)	▤		
	Hurricane Easy (1950)	Storm Specific Wind and Precipitation		

The values of wind speed and definition of the type of wind (gust, one-minute, fastest-mile) required further analysis to identify the method to be used in analyzing conditions for freeboard determination. In particular, since there is no consensus definition of a "lesser wind condition." Comparison of various wind speeds that could be used for design criteria were evaluated for each of the reservoir level/precipitation events described in **Table DCM 2-1**, and each case is presented below.

2.1.1 CASE 1: PMP COMBINED WITH 100-YEAR WIND

The Probable Maximum Flood (PMF) results from routing the Probable Maximum Precipitation (PMP) event. The PMP is determined using the National Oceanic and Atmospheric Administration (NOAA) published data in Hydrometeorological Report (HMR) Numbers 51, 52, and 53. As shown in HMR 51, the PMP depth for southern Florida ranges from about 32 to 60 inches for 6 hour and 72 hour storm duration's, respectively. For CERP reservoirs, inflow is derived from a combination of pumping into the impoundment and direct rainfall rather than gravity inflow from a contributory drainage area. Therefore, the PMF will primarily result from the PMP falling directly on the reservoir surface, plus an insignificant contribution of runoff from the surrounding perimeter crest road and interior slope. The PMF for design will depend on the PMF hydrograph (duration) that produces the critical condition within the impoundment and at the dam. The critical condition could result from the inflow rate, timing or total volume of storm event, dependent on the spillway design and reservoir storage available at the beginning of the storm event. The PMF hydrograph for design could result from a high-intensity local storm or a general storm with a long duration. Therefore, the PMP event that develops the most critical condition, determined from the various storms identified in HMR 51, will be the design PMF for the impoundment and associated structures.

Identification of an appropriate design wind to be used in conjunction with the unlikely PMF does not have a documented standard practice or requirement in Florida. Therefore, different sources of wind conditions that are typically used in facility design were evaluated. Sources included:

- USACE regulations,
- US Bureau of Reclamation Design Guidelines for Dams,
- Federal Energy Regulatory Commission,
- Florida Building Code,
- The American Society of Civil Engineer's Manual on Wind Load Conditions ASCE 07-02 (ASCE, 2003), and
- Comparable projects.

The basic wind speeds for building design presented by the ASCE for hurricane prone areas and in the Florida Building Code are based on 3-second gusts for 50-year to 100-year return periods (ASCE, 2003, Fig. 6-1b and Table C 6-3). ASCE Figure 6-1b (ASCE, 2003) corresponds to Figure 1606 in the Florida Building Code.

Various wind conditions were considered in developing combinations of precipitation/reservoir level and wind events that could reasonably be expected to occur simultaneously. Since there is little documentation in this area, other areas of standard practice that could be used as a potential guideline were reviewed. One analogous condition that is used in standard practice is the method for determining the PMF based freeboard for impoundments in the northern States. The PMF for cool season months in the northern States is determined as the PMP in combination with snowmelt from the 100-year snow-pack (FERC, 2001, pg. 66). In many places in northern States, particularly in mountainous areas, the contribution of snowmelt of the 100-year snow-pack with the PMF runoff is very significant. This can be summarized as using an extreme precipitation event, in combination with a "lesser" snowmelt condition. In identifying a combination of events for freeboard determination, a level of risk or probability of occurrence similar to the PMP plus 100-year snow-melt would be an analogous selection. While a PMF in Florida obviously would not have a snowmelt contribution, a similar critical condition (i.e. level of risk or probability of occurrence) that is characteristic of Florida would be the combination of the PMP with an extreme wind speed of some lesser condition. An example of a lesser wind condition would be a 100-year recurrence interval wind speed.

The 100-year wind condition (has a 1 in 100 probability of occurrence each year) represents a severe criterion when placed in combination with the PMP event. The 100-year wind speed was identified as a "lesser wind condition" that could be reasonably expected to occur simultaneously with a PMP event, and will be used for defining **Case 1** in the freeboard determination. In addition, a recurrence interval of 100 years will be used for the frequency of occurrence for the analysis since this will allow the magnitude to vary with the site location in Florida, such as distance inland. Wind speeds for any given frequency are logically greater near the coast than they would be near the center of Florida as hurricanes generally decrease in strength/intensity once the eye of the hurricane makes land fall. Faster moving hurricanes tend to only weaken slightly as they move across the peninsula of Florida, but generally have less rainfall due to this fast progression. Significant surcharge pool levels may not result from an individual storm. However, sequential precipitation events frequently occur with storms that provide extreme precipitation events. Therefore, a minimum design wind speed will be used for all of the impoundments. Slower moving hurricanes generally have greater rainfall but diminish more in strength as they move across the peninsula of Florida.

An alternative wind speed for freeboard determination has been considered in a working draft document by the USACE (Jacksonville District, 2004). The USACE selected a 125 mph wind speed as the value to be used in combination with the PMP. The 125 mph wind speed would be applied to all reservoirs to be designed in the CERP study area. Based on our understanding, the 125 mph wind speed represents a wind at the reference level of 33 feet above ground surface, and is interpreted to represent a wind of sufficient magnitude and duration to develop significant reservoir waves. In summary, the December 2004 Working Draft by the USACE (Jacksonville District) states that the maximum design wind speed recommended for CERP impoundments is 125 mph based on the following:

1. Acknowledging South Florida's Building Code requirement for 120 mph.

2. The theorized maximum sustained wind velocities of 130 mph for Water Conservation Area (WCA) 1 and 2 which is further referenced to C&SF GDM summarized data as *"It is estimated that the maximum sustained velocity on Lake Okeechobee would be about 115 mph, with a lake wide average of 85 mph. WCA-1 and –2 maximum sustained velocity would be 130 mph (area average of 100 mph) with WCA-3 having a 120 mph (area average of 85 mph)."*

3. GDM specifies for provision of protection against wave action for 3 or 4 hours in the case of a stagnated hurricane.

4. Category 3 Hurricane on Saffir-Simpson Scale.

The basis for selecting the 100-year wind speed from the Florida Building Code and ASCE (2003) for CERP projects, and the method to be used to calculate wind speeds for design for **Case 1** are described in the following section.

2.1.1.1 100-Year Wind Speed.

Determination of the 100-year wind speed at a site using the Florida Building Code (www.sbcci.org/floridacodes.htm, Section 1606 Wind Loads) begins with Figure 1606, which is reproduced herein as **Figure DCM 2-2**. The wind speeds on **Figure DCM 2-2** are 3-second gusts in miles per hour that would be expected to occur 33 feet (10 meters) above the ground surface. The 33-ft height for wind speeds is a standard reference height. The wind speeds are further identified as representing ***50-year to 100- year*** peak gusts.

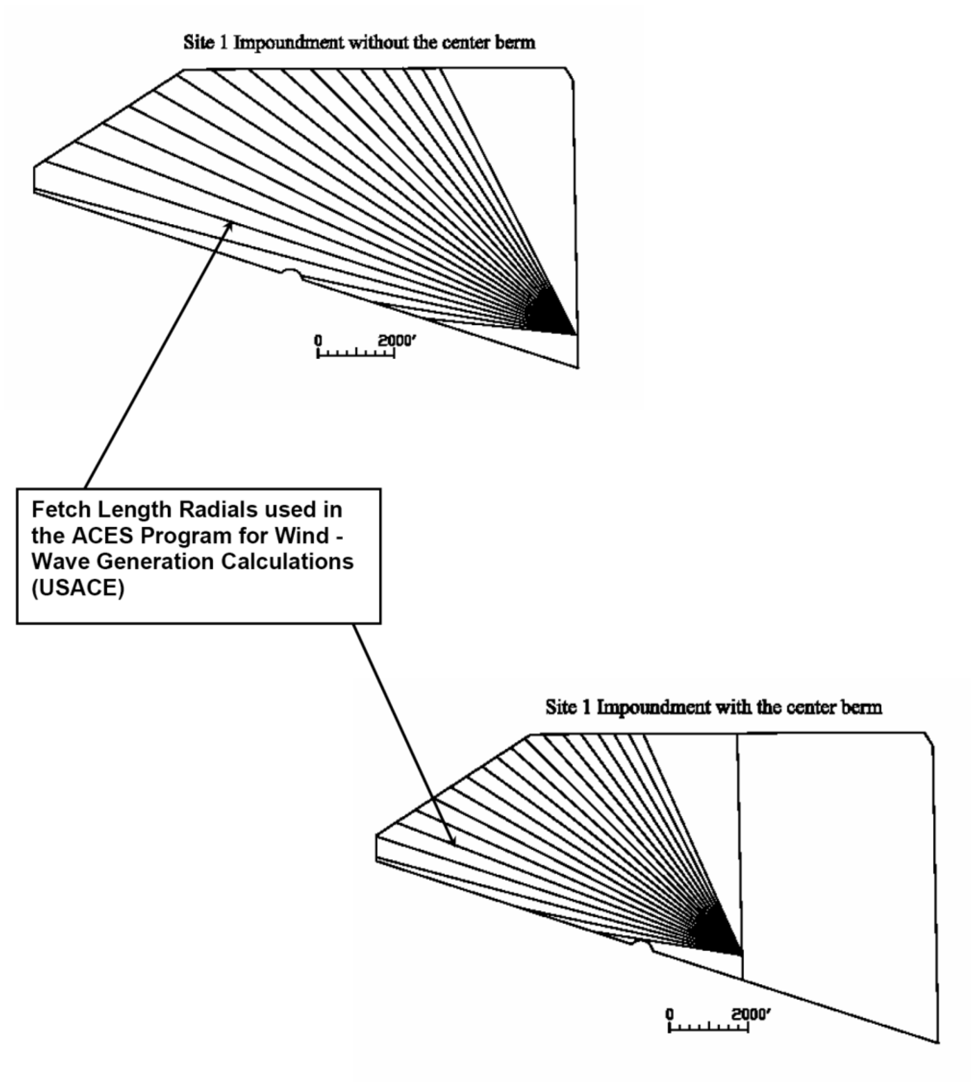


FIGURE DCM 2-2

EXAMPLE ACES Fetch Length Radials
(Site 1)

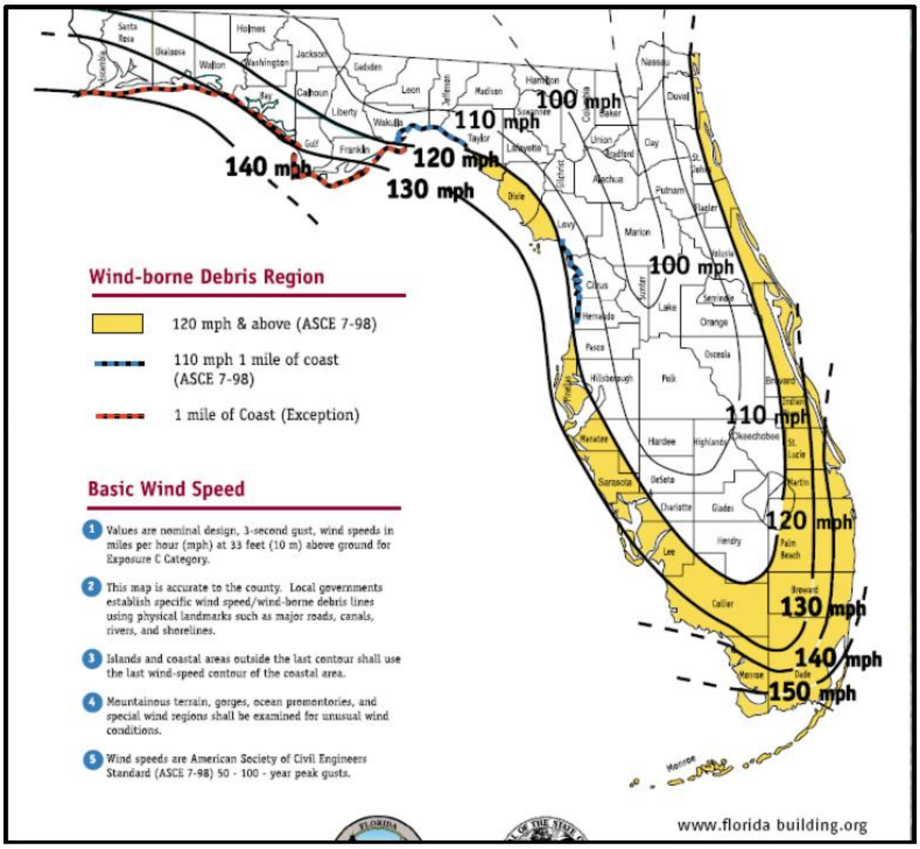


Figure DCM 2-2: Florida 50-Year to 100-Year Gust Wind Speeds (mph) (ref. ASCE 7-02, Figure 6-1b; Florida Building Code, Figure 1606)

The conservative assumption is to assume that the wind speeds on **Figure DCM 2-2** represent 50-year wind speeds and convert the values to 100-year wind speeds, as shown below. The equivalent 100-year wind speeds listed in **Table DCM 2-2**, use a factor of 1.07 to convert hurricane wind speeds from a 50-year to a 100-year return (from the standard structural design reference of the American Society of Civil Engineers (ASCE) Table C 6-3, ASCE 2003). For example, if an area with a 125-mph wind speed is selected from **Figure DCM 2-2**, the corresponding 3-second gust 100-year wind speed would be 134 mph.

Table DCM 2-2: Wind Speed Conversion Factors by Return Period

Return Period (years)	Conversion Factor
500	1.23
200	1.14
100	1.07
50	1.00
25	0.88

The 3-second gust wind speeds shown on **Figure DCM 2-2** are of an insufficient duration to fully develop significant waves. Specifically, the wind speed duration of 3 seconds is insufficient to develop the wave height, which would occur due to the traction of a constant wind at this speed and propagate the wave across the reservoir. Therefore, wind speed adjustments appropriate for developing significant reservoir waves are to be developed as shown below.

ASCE Figure 6-1b (ASCE, 2003) shows the same wind speeds as Figure 1606 in the Florida Building Code. Figures 6-1a, 6-1b, and 6-1c (ASCE, 2003) cover the Gulf and Atlantic coasts from southern Texas to Florida to North Carolina and to Maine. The figures provide "smoothed" contours of wind speed and are based on a substantial amount of data and analysis. It would be impractical for CERP dam design consultants to duplicate the scope of data analysis used in developing the ASCE figures and is generally not necessary. Use of these figures provides reliable and consistent data from location to location. The analysis of local wind speed gage data, even from first-order stations, would provide unreliable and inconsistent results from location to location because of routine gage malfunctions at extreme wind speeds and the "hit and miss" nature of hurricanes relative to data recording stations. Freeboard criteria for CERP reservoirs, therefore, will be based on Figure 1606 of the Florida Building Code for wind speed data, rather than developing wind speeds based on local gage data, with the appropriate adjustments described below. Figure 1606, which is identified as being based on 50-year to 100-year wind speeds, will be conservatively interpreted as representing 50-year wind speeds for application to freeboard determination.

2.1.1.2 Duration Adjustment of Wind Speeds

When evaluating wave generation in water bodies of different sizes, different wind speed averaging intervals are appropriate. The current Coastal Engineering Manual notes that if extreme wind speeds are being considered, wind speed should be adjusted from the averaging time of the observation (3-second on **Figure DCM 2-2**) to an averaging time appropriate for wave prediction (USACE, 2003, pg. II-2-34). To adjust the wind speeds for duration, the Coastal Engineering Manual references a figure on page II-2-4, which also includes the defining equations? The following equations (USACE, 2003, pg. II-2-4) show the calculation adjustments for the ratio of wind speeds by various durations, to 1-hour average wind speeds.

For $t \le 3{,}600$: $$\frac{U_t}{U_{3,600}} = 1.277 + 0.296 \tanh\left(0.9 \log_{10}\left(\frac{45}{t}\right)\right) \qquad \text{Eqn. 1}$$

For $3{,}600 < t < 36{,}000$: $$\frac{U_t}{U_{3,600}} = -0.15 \log_{10} t + 1.5334 \qquad \text{Eqn. 2}$$

where:

t = time (seconds)

U_t = wind speed (mph) at time t (seconds)

$U_{3,600}$ = 1-hour average wind speed (mph)

For a given wind speed and fetch, there is an associated wind duration that, if exceeded, cannot increase the wave size because of the limiting effects of the fetch and depth. The following equation (USACE, 2003, Eqn. II-2-35) can be used to determine whether or not waves in a particular situation can be categorized as fetch limited.

$$t_{x,u} = 77.23 \frac{X^{0.67}}{u^{0.34} g^{0.33}} \qquad \text{Eqn. 3}$$

where:

t = wind duration time (seconds)

X = fetch length (meters)

u = wind velocity (meters/sec)

g = gravitational constant (meters/sec^2)

In English units, the above equation would be:

$$t = 112 \frac{X^{0.67}}{u^{0.34}} \qquad \text{Eqn. 4}$$

where:

t = wind duration time (minutes)

X = fetch length (miles)

u = wind velocity (mph)

The above Eqn. 4 is also presented in a current publication on freeboard analysis (Holler, 2004). An example can clarify the procedure for determining the 100-year wind speed that would be coincident with the PMP.

2.1.1.3 Example Wind Speed Adjustments

The following wind speed determination generally follows the example procedure for estimating winds for wave prediction provided in the Coastal Engineering Manual beginning on page II-2-34 (USACE, 2003).

1. Assume the reservoir is in an area where the gust-wind is 125 mph based on the Florida Building Code (**Figure DCM 2-2)**.

2. Adjust the 50-year, 3-second wind speed to an equivalent 100-year wind speed:
(125 mph) * 1.07 = 134 mph

3. Adjust the134 mph, 3-second wind speed to a 1-hour wind speed duration with Eqn. 1

$$\frac{U_3}{U_{3,600}} = 1.51$$

The resulting 1-hour average wind speed would be 89 mph (134 mph/1.51).

4. Adjust the over land wind speed to an over water wind speed (USACE, 2003, pg. II-2-36). Considering that various sources provide some conflicting information regarding overland versus over water wind speed adjustments, the conservative approach has been taken that adjusts over land wind speeds to an over water wind speed using a factor of 1.2. The resulting over water wind speed is (89 mph)*1.2 = 107 mph.

5. Assume the reservoir fetch is 5 miles. For this 107 mph wind speed, the minimum duration from Eqn. 4 is 67 minutes (4,020 seconds). Eqn. 4 is used to determine whether or not waves in a particular situation should be categorized as fetch limited.

6. Adjust the 1-hour 107 mph wind speed to 67 minutes duration using Eqn. 2.

$$\frac{U_{4,020}}{U_{3,600}} = 0.993$$

The resulting 67-minute average wind speed to be used for the freeboard determination would be 106 mph. Note, that if the fetch distance was longer, the adjusted wind speed could be less than 100 mph. The minimum wind speed for impoundment design will be a 100 mph over-water, one hour wind speed.

In summary, the application of a 100-year frequency wind speed with the PMF is judged to be of a similar level of risk or probability for freeboard determination, as applied in engineering practice elsewhere for reservoir design. Based on reviews and discussions it was agreed that the 100-year wind speed will be used to represent an extreme, but reasonably possible wind speed in combination with the PMF, for **Case 1** as described above. The 100-year wind speed will be determined from the Florida Building Code or ASCE 07-02 map (and assuming the values have a recurrence interval of 50 years) for the specific project site, and adjusted to the 100-year recurrence interval at the applicable over water wind height, and duration required for wave setup and wind run-up analysis.

2.1.2 CASE 2: 100-YEAR PRECIPITATION COMBINED WITH CATEGORY 5 HURRICANE WINDS

Case 2 was identified as one combination of precipitation and wind conditions that could be reasonably expected to occur simultaneously. Freeboard **Case 2** defined in **Table DCM 2-1** includes a Category 5 wind speed as defined by the Saffir-Simpson Hurricane Scale with a 100-year precipitation event. The Saffir-Simpson Hurricane Scale classifies hurricanes based on intensity and damage potential using five categories from 1 through 5, with 5 being the most intense. The Saffir-Simpson Hurricane Scale categorizes hurricanes based on 1-minute sustained wind speeds (ASCE, 2004, pg. 116). The Saffir- Simpson hurricane wind speeds are presented in **Table DCM 2-3**, including adjustments to comparable gust (3-second) and 1-hour average wind speeds based on Eqn. 1.

Table DCM 2-3: Saffir-Simpson Hurricane Categories with Duration Adjusted Winds

Hurricane Category	Reference Wind Speed (1-min. avg.)	Duration Adjusted Wind Speed (3-sec. Gust)	Duration Adjusted Wind Speed (1-hour avg.)
1	74 - 95 mph	90 - 115 mph	59 - 76 mph
2	96 - 110 mph	116 - 133 mph	77 - 88 mph
3	111 - 130 mph	134 - 158 mph	89 - 104 mph
4	131 - 155 mph	159 - 188 mph	105 - 124 mph
5	> 155 mph	> 188 mph	> 124 mph

From **Table DCM 2-3**, it is evident that the 3-second gust wind speeds for southern Florida shown in the Florida Building Code (see **Figure DCM 2-2)** mostly fall into the Category 2 equivalent wind speed classification when compared to the duration adjusted values. **Case 2** consists of a reference design wind speed selected for combination with the 100-year precipitation event. The wind speed for **Case 2**, the minimum wind speed that would classify a hurricane as Category 5 will be used, and interpreted as an over water wind speed (i.e. the factor to convert an over land wind speed to an over water wind speed is not applied). The Category 5, 156 mph wind speed for a 1-minute average converts to 125 mph for a 1-hour average wind speed that would have a sufficient duration to generate significant winds on the water surface.

The 100-year precipitation depths coincident with the Category 5 wind speeds can be determined from Technical Publication EMA # 390 – *Frequency Analysis of Daily Rainfall Maxima for Central and Southern Florida* (Pathak 2001). In the freeboard determination, the 3-day 100-year precipitation, as shown on **Figure DCM 2-3**, will be combined with the Category 5 wind speeds for **Case 2**.

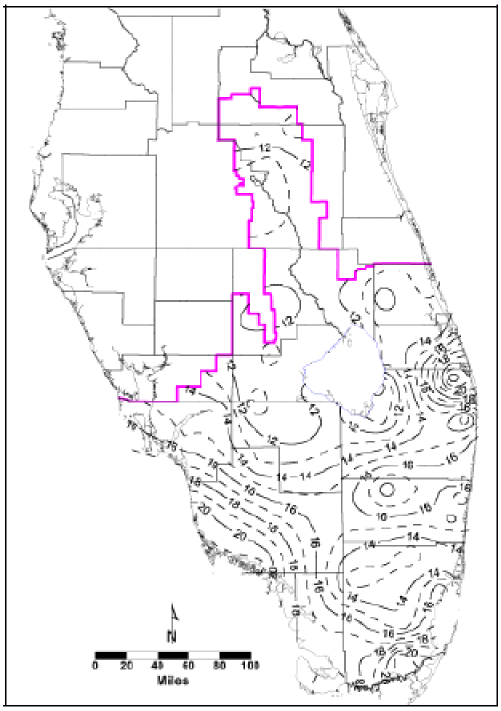


Figure DCM 2-3: 100-Year, 3-Day Precipitation (inches)
(ref. Pathak, 2001)

2.1.3 CASE 3: PROBABLE MAXIMUM WIND SPEED

A third combination of precipitation and wind conditions that could be reasonably expected to occur simultaneously would include a PMW. It is reasonable to assume that a PMW could occur, and reasonable combinations of extreme precipitation events and extreme (but lesser than maximum) wind conditions are considered in **Case 1** and **Case 2**. Therefore, another logical occurrence of events to be considered would be a PMW combined with the reservoir level at the normal full storage level, which is typically considered as the uncontrolled spillway level. There are no known standard references that define the PMW, applicable to southern Florida. Identifying a PMW in a hurricane area poses substantial difficulty, ranging from recorded readings limited by the maximum speed capability of the instrument or damage from flying debris, to fixed point land based instrument locations that most likely have not experienced/recorded the maximum wind conditions (which vary substantially in various sectors of a cyclonic event) generated by the storm. Therefore, selection and application of a PMW for freeboard determination will be used for evaluation of the sensitivity of an extreme wind speed in combination with a normal full storage level. Two hundred miles per hour was initially selected at the workshop as the preliminary wind speed for **Case 3**. A review of historic hurricane wind speeds was performed to evaluate wind speed records and develop an appropriate value to represent a PMW.

NOAA has listed the highest recorded wind speeds on its Atlantic Oceanographic and Meteorological Laboratory (AORL) Internet site (http://www.aoml.noaa.gov/hrd/tcfaq/E1.html). For the Atlantic region, Hurricane Camille (1969) and Hurricane Allen (1980) are listed as having maximum winds estimated at 190 mph. The AORL Internet site (http://www.aoml.noaa.gov/hrd/tcfaq/D4.html) also notes that the National Hurricane Center uses the 1-minute averaging time as standard for reporting sustained winds. Although the exact measurement location for these winds was not specified, they were assumed to be over water winds on the Saffir-Simpson Hurricane Scale (1-minute avg. duration). Hurricane Camille is listed as an example of a Category 5 hurricane.

The Coastal Engineering Manual notes that the central pressure of Hurricane Camille as it moved onshore at a speed of about 6 m/sec in 1969 was about 912 mb, which corresponds to a 1-minute wind speed of approximately 150 mph (USACE, 1993, pg. II-2-31). Hurricane Gilbert (1988) is listed on the AORL Internet site as having the lowest recorded sea level pressure of 888 mb, which corresponds to roughly a 170 mph 1-minute wind speed. A new low for a recorded sea level pressure for an Atlantic storm was apparently set by Hurricane Wilma in 2005, at 882 mb, with a corresponding 1-minute wind speed of approximately 181 mph.
An overview of reports on historical storms (between 1900 and 1995) with sustained wind speeds of at least 125 mph shows a significant number of recorded and estimated wind events greater than 175 mph (Hurricane Camille) for sustained winds (1 minute or longer in duration sustained wind) potentially up to 250 mph for a peak wind gust (1935). Wind readings reported include measured and estimated (including engineering and scientific estimates based on analysis of damage and/or barometric pressure gradients). More than 25 events of sustained winds greater than 125 mph have been reported (Barnes 1998). Some of the more significant events include:

a. September 1935 Labor Day Hurricane: An estimated sustained wind speed of 200 mph was developed based on damage and the estimate was supported by a scientific analysis of barometric pressure gradients recorded during the hurricane,

b. August 1969 Hurricane Camille: An estimated sustained wind of >175 mph,

c. September 1928 Lake Okeechobee Hurricane: Recorded and estimated sustained wind of >150 mph,

d. September 1926 Miami Hurricane: Recorded and estimated sustained wind >150 mph; sustained wind of 128 mph for 5 consecutive minutes; sustained wind of 138 mph for 2 minutes; and 42 minutes of sustained winds >120 mph,

e. September 1945 at Carysfort Reef Lighthouse: a 2 minute maximum recorded average wind speed of 170 mph,

f. September 1947 at Hillsboro Lighthouse: a sustained wind of 155 mph was recorded,

g. September 1949 Delray Beach: a 1 minute maximum wind speed of 153 mph and a 5 minute maximum wind speed of 132 mph were measured.

h. September 1960 Hurricane Donna: sustained wind speed of 140 to 150 mph were reported over a wide geographic area ranging from Fort Myers on the west coast to Ft. Meade in Polk County in Central Florida.

In review of historical data, design of facilities must consider that maximum sustained winds speeds (interpreted as 1 minute duration, minimum) would certainly exceed more than 150 mph (essentially a Category 5 hurricane) and would be expected to be in excess of 175 mph. A PMW does not have an established definition. However, in the context of determining criteria for reservoir design and freeboard determination, a correlation with the definition of a PMP would be appropriate. As such, the PMW would be defined as theoretically the highest wind speed for a given duration (1 minute sustained wind) that is physically possible from a cyclonic storm over land and at the reference elevation (33 feet, 10 meter). Additional considerations in determining a PMW would include the limited database of measurement of cyclonic winds based on fixed, land-based weather stations, equipment limitations, damage to instruments from debris, and the probability that extreme hurricane winds that vary around the hurricane vortex are not fully measured in any storm event.

At the December 2004 Design Criteria Workshop, a 200 mph value was identified as an appropriate value for the PMW considering the difficulty of measuring accurate hurricane wind speed, and the high incidence of failure of anemometers in high winds. The data presented in the previous paragraphs indicate the 200 mph wind speed represents a reasonable but conservative upper value for the PMW, if used in combination with a highly probable reservoir level. Therefore a 200 mph wind speed, which is well above the Category 5 minimum wind speed of 156 mph, will be considered the PMW for design of impoundments on CERP projects with a high or significant hazard potential classification (in accordance with **DCM-1**), in combination with the reservoir level at the normal full storage level (i.e. uncontrolled spillway crest level). The 200 mph assumed PMW will be interpreted as corresponding to an over-water, 1-minute average wind (i.e. the factor to convert an over-land wind speed to an over-water wind speed is not applied). Using a procedure similar to that provided for the 100-year wind speed, an example PMW wind speed for reservoir freeboard determination is shown as follows:

1. Adjust the 200 mph, 60-second wind speed to a 1-hour wind speed duration with Eqn. 1

$$\frac{U_{60}}{U_{3,600}} = 1.24$$

The resulting 1-hour average wind speed is 161 mph.

2. Assume the reservoir fetch is 5 miles. For this 161 mph wind speed, the minimum duration from Eqn. 4 is 59 minutes. Therefore, the 161-mph 1-hour wind speed would be the applicable wind coincident with the normal full storage level.

It was also considered that the PMW could vary by location in a manner similar to the 100-year wind speed. However, considering the lack of any data records or standard references providing a basis for variation of PMW conditions with location, the 200 mph, 1-minute over-water PMW was identified as the wind speed for **Case 3**. Due to uncertainties inherent in determining extreme winds and the application to wave generation (e.g. it has been reported that waves break up or "blow apart" under high wind conditions), the **Case 3** freeboard determination will be calculated based on the wave height with a 2 percent frequency, for manual computations using the SPM/CEM. **Case 3** is to be used only for "sensitivity identification" and not as a selected design condition. Wave models are unlikely capable of yielding results within a degree of confidence for design for these extreme wind speeds, especially over relatively shallow water bodies. Even for 125-mph wind, model capabilities are most likely being "stretched" for project conditions.

2.1.4 CASE 4: STORM SPECIFIC WIND AND PRECIPITATION

In addition to precipitation and wind conditions based on frequency analysis of the region, severe hurricane events have occurred in the State of Florida historically. Combinations of extreme precipitation and wind (Storm Specific) were also evaluated. A storm specific case of precipitation and wind conditions could be reasonably expected to occur and is included as **Case 4**. Storm records show that a 24-hour rainfall record occurred from Hurricane Easy in 1950. Hurricane Easy occurred in the upper Florida west coast over a three-day period, about 90 miles north of Tampa, Florida. Yankeetown was part of the area where the circling hurricane hit twice and a peak 24-hour rainfall of 38.7 inches was recorded, with a 72-hour rainfall depth of 45.2 inches. The precipitation depths from Hurricane Easy at Yankeetown, FL should be routed with the water surface elevation starting at the normal full storage level along with the assumed wind speed for the **Case 4** condition.

A maximum 3-second gust wind speed of 125 mph was recorded during Hurricane Easy, before the anemometer was knocked down by a falling tree (Barnes, 1998). It is unknown whether 125 mph is representative of the maximum wind speed generated by the storm specific hurricane event, or what wind condition occurred in the latter portion of the storm. However, this represents the best available information for use in analysis of wind and precipitation from a recorded storm event that has occurred simultaneously, and is too be used as the assumed wind speed.

A wind speed of 125 mph (3-second wind gust) is similar to the 100-year wind, location specific wind speed determined in the Example comparison, earlier. **Case 4** analysis will consist of the precipitation depths from Hurricane Easy at Yankeetown, FL (both the 24-hour and 72 hour rainfall durations) in combination with a 125 mph (3-second gust) wind speed. Appropriate adjustments to meet the required over-water conditions, wind duration for wave development, etc. are to be applied to the 125-mph, 3-second wind gust. Similarly, storm specific conditions from historical events near the site should also be considered.

2.2 MINIMUM FREEBOARD

Combined wind and precipitation design criteria for freeboard determination at high and significant hazard potential impoundments are described in the previous section. Minimum freeboard as described in this section, applies to high, significant and low hazard potential impoundments. The minimum will be the greater of the freeboard determined from the combined wind and precipitation criteria, or the minimum defined in this section.

CERP impoundments will generally be constructed with sandy soil as embankment material. At some sites rockfill or roller-compacted concrete may also be viable alternatives for dam construction. Slope protection consisting of rock riprap, soil-cement or other erosion-resistant layer will be provided on the reservoir (interior) side of the dam to provide wave protection. The exterior slope of the embankment will typically consist of sandy soil with vegetative cover. “Hardening” of the downstream slope to allow for over-topping is not planned.

Freeboard is an integral part of all dams. The objective in selection of the design freeboard is to assure that failure of the dam will not result from wind setup, wave action, uncertainties in analytical procedures, and uncertainties in project function in combination with the most critical pool elevation (USACE, ER 1110-8-2 (FR)). Because the impoundments will generally be constructed with highly erodible embankment shell material, freeboard design criteria of zero over-wash resulting from wind setup and wave action is desireable. Zero over-wash is not always required under infrequent high pool conditions. However, it is required that the over-wash not be of such magnitude and duration as to threaten the safety of the dam (USACE, ER 1110-8-2 (FR)).

Under some circumstances, freeboard may not be the same along the entire embankment length. Based on the geometry of many of the CERP impoundments, the fetch length and bathymetry of the storage area could vary significantly for different segments of the embankment. Where impoundments have significant embankment lengths, the risk conditions could vary significantly along the perimeter. Consequently, calculation of freeboard heights could consider the different wind setup/wave heights that could occur and the risk condition associated with embankment segments, in establishing the minimum crest height for a given precipitation and wind combination along various embankment segments.

In addition, a risk based assessment for final determination of the IDF, spillway sizing and freeboard height is required for all hazard classifications. Guidelines for risk based assessment have been prepared by various Federal agencies. The recommended guideline for Incremental Damage Assessment (IDA) for CERP impoundments is *Chapter 2, Selecting and Accommodating Inflow Design Floods for Dams*, Federal Energy Regulatory Commission, Engineering Guidelines for the Evaluation of Hydropower Projects (October 1993). Alternatively, other established methods of conducting and IDA would also be considered.

2.2.1 HIGH AND SIGNIFICANT HAZARD POTENTIAL

High and significant hazard potential reservoirs/impoundments with surcharge pool elevations within three feet of the maximum surcharge pool level (maximum water storage level-MWSL) for a period of 36 hours or longer, have increased probability that high winds in the critical fetch may coincide with this level. Therefore, when the inflow design flood hydrograph is within three feet of the maximum surcharge pool for 36 hours or longer or where the project has been designed with little surcharge for the maximum pool above the normal full storage elevation (NFSL), the minimum freeboard will be five feet (USACE, 1991) for embankment dams (i.e. five feet above maximum water storage level (MWSL)).

Freeboard height, over-wash rate, magnitude and duration are to be evaluated to determine the required freeboard height to safely pass the design event(s). Supporting data for freeboard determination is to include:

1. Freeboard height versus over-wash rate,

2. Maximum stage versus frequency of exceedance,

3. Narrative description of basis for selection of the freeboard height,

4. Include an analysis of each **Case** described in **DCM – 2**, as well as any other conditions as determined by the designer.

An example of the freeboard height versus over-wash rate and maximum reservoir stage versus frequency of exceedance are shown in **Attachment A**. The supporting data is to be provided for each cell or segment of the impoundment if different freeboard heights are recommended.

2.2.2 LOW HAZARD POTENTIAL

Low hazard potential (HPC) impoundments are facilities that are operated for storage (including **some** Stormwater Treatment Area's – STAs) that do not pose a significant risk, therefore less stringent criteria will be satisfactory. Low hazard potential impoundments are to be determined in accordance with the procedures described in **DCM –1**. The IDF produced MWSL will be determined by routing the 100-year, 24-hour recurrence interval precipitation event (versus the PMP that is required for impoundments with High and Significant HPC). The minimum freeboard for design of Low HPC impoundments is 3 feet on top of the MWSL (unless 36-hour exemption allowed under ER 1111-8-2(FR) is applicable). However, the design freeboard must take the following functional and operational conditions into consideration.

Freeboard requirements for low HPC storage impoundments will be the greater between the described minimum 3 feet or the combination impact of wind setup plus wave run-up due to a 60 mph 1-hour wind condition with

the MWSL. Low HPC storage impoundments are those impoundments that will most likely not sustain emergent vegetation that dampen waves significantly, due to dry season and operational "dry-outs."
Low HPC storage impoundments with shallow water depths and sustained emergent vegetation (sufficiently developed to impede the build up of significant wave trains), will have the minimum described 3 feet of freeboard. Impoundments meeting this requirement are expected to primarily be low HPC STAs that will be operated as flow through impoundments for the treatment of water to an improved water quality (i.e. treatment process attained in shallow – less than 4 feet of water depth – with sustained vegetation and dry out is not expected except under extreme drought conditions), for the express purpose of water treatment.

2.3 WIND DIRECTION

The height of the wave run-up on a slope depends in part on the incident wind direction and the prevailing wind direction. The prevailing wind direction is not readily identifiable in the Florida peninsula due to the variety of directions by which a hurricane can travel, in addition to the varying wind directions within the hurricane storm itself. Therefore, an assessment of probability of wind by sectors was evaluated from several weather stations. As described in the Conceptual High Levee Optimization Report (JM/JV dated May 21, 2004), the wind directions were grouped into 16 sectors, 22.5o for each sector. For example, the North (N) sector is 11.25o from each side of the true north, East (E) sector is 11.25o from each side of east direction, South (S) sector is 11.25o from each side of south direction, and West (W) sector is 11.25o from each side of west direction. **Figure DCM 2-4** shows the percentages of wind occurrences in 16 sectors at the West Palm Beach International Airport.

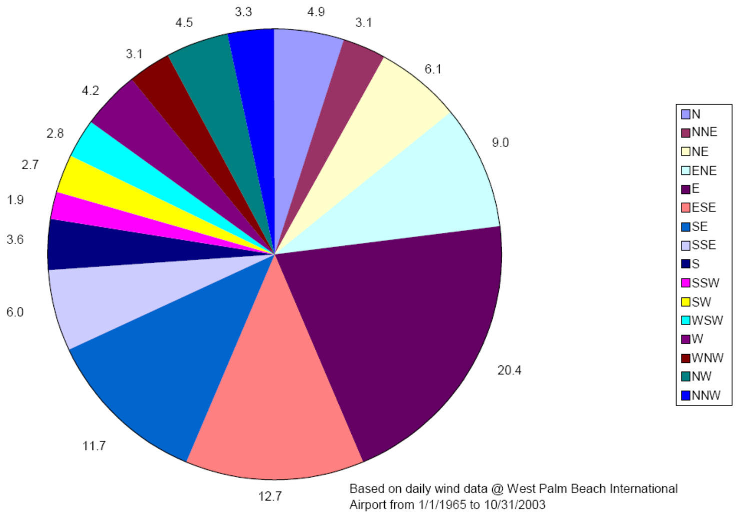


Figure DCM 2-4. Wind Speed Probability by Sector (22.5° interval)

Based on the probability of wind direction by sector at the Palm Beach International weather station, which yielded similar results to other weather stations evaluated in southern Florida, a prevailing wind direction will not be assumed. Winds occurring from any direction are to be considered in analyzing maximum fetch distances, wind setup and wave height plus run-up. Analysis of local wind records should be considered in selecting spillway locations
.

2.4 WAVE HEIGHT AND RUNUP

Analysis of conditions to develop freeboard is a complex process. Freeboard height calculations can be performed using spreadsheet and hand calculations using the procedures outlined in the Shore Protection Manual (USACE 1984) and other guidance documents. In recent years computer programs (numerical models) have been developed that allow for faster computations. Numerical models are required by the USACE for the **selected design alternative**. The computer program Steady-State Spectral Wave Model (STWAVE), which can be used for the development of wind-generated wave fields, and the computer program Automated Coastal Engineering System (ACES) which contains models for the computation of wave run-up and overtopping are the recommended numerical models. ACES (WES, 1992) was developed by the Waterways Experiment Station (WES) of the USACE. STWAVE (CHL, 2001) was developed by the Coastal and Hydraulics Laboratory (CHL) of the U.S. Army Engineer Research and Development Center. An example of the fetch length radials used for wind-wave generation calculations in the ACES program is shown on **Figure DCM 2-2**.

Wave fields include monochromatic (regular) waves and irregular waves. The irregular waves are to be used to evaluate wave fields to determine wave height for freeboard determination. Thorough understanding of the freeboard design process and the basis of the various factors to be applied is critical to effective use of these computer programs. Adequate training and access to technical support is recommended. The programs should not be used for specific project circumstances that are beyond the limitations of the programs.

The traditional spreadsheet/hand calculations using primarily the Coastal Engineering Manual – CEM (USACE, 2003) and the Shore Protection Manual – SPM (USACE, 1984) can be used for analysis of various alternatives. Although the Coastal Engineering Manual is the most current document, the Shore Protection Manual is referenced for wave run-up calculations since that portion of the Coastal Engineering Manual is in Draft form at this time. Wave heights from the CEM/SPM utilize Rayleigh Distribution to evaluate wave fields for structures (e.g. significant wave, 2 percent frequency wave, 5 percent frequency wave etc.). Irregular waves in STWAVE and ACES are similarly used to evaluate wave fields for structures.

The results of final computer-generated parameters are also to be compared with the traditional hand calculation methods for the final selected configuration. A direct one-to-one comparison may not result in an exact match. However, order of magnitude agreement should result. An example of the hand calculation method for developing wind speed, wave height and run-up for freeboard determination is presented in **Attachment A**. The example computation is presented for **Case 1** condition, and would be repeated in a similar manner for **Cases 2, 3 and 4** in the determination of the controlling freeboard condition and the required dam crest level.

2.5 WIND SETUP

Wind setup is one of the required components in the determination of freeboard. On the shallowest CERP impoundments, wind setup is a particularly important component of freeboard, and may even exceed the wave run-up component in magnitude. Three methods are available for determining wind setup. The most widely used wind setup equation as applied to reservoirs is probably the Zeider Zee equation, which normally results in the highest wind setup determination of the three methods. The Zeider Zee equation (USACE, 1997, pg. 15-3) is expressed as:

$$S = \frac{U^2 F}{1400d} \qquad \text{Eqn. 5}$$

Where:

S = wind setup (feet) above the stillwater level

U = average wind velocity (miles per hour) over the fetch

F = fetch length (miles)

d = average depth (feet) of water generally along the fetch line

Although the Zeider Zee equation is commonly suggested for determining wind setup in reservoirs, typical reservoirs in the U.S. are substantially deeper than the typical CERP impoundments. Because the Zeider Zee equation may yield results that excessively over-estimate wind setup at average reservoir water depths that are more typical of CERP impoundments, guidance is provided in this section to recommended alternative wind setup methods.

The recommended alternative to the Zeider Zee equation at lower impoundment depths (average stillwater depth less than 16 feet including surcharge depth), is the Bretschneider method (Bretschneider, 1966). The Bretschneider method normally results in wind setup heights that approach the mean value of results for the Zeider Zee and Sibul equations. For an enclosed reservoir of regular or somewhat irregular shape, an effective stress parameter in the Bretschneider method is presented in an integrated form as follows:

$$\frac{\kappa U^2 F}{g d^2} = \sum_{i=1}^{i=M} \left(\frac{\kappa U^2 \Delta x}{g (d_i)^2} \right) \qquad \text{Eqn. 6}$$

where:

κ = a constant equal to 3.3 x 10^{-6} (dimensionless)

U = average wind velocity (feet per second) over the fetch

F = fetch length (feet)

g = the gravitational constant (32.17 ft/sec^2)

d = average depth (feet) of water generally along the fetch line

i = section number

x = horizontal distance (feet)

The effective stress parameter is used for determining the wind setup (S in feet) as a function of S/d with Tables 5.2 and 5.3 as provided by Bretschneider (1966). The method accounts for wind conditions that may create a partially exposed reservoir bottom along the fetch line.

The Sibul equation (Brater and King, 1976) generally yields the lowest wind setup values of the three wind setup equations. The Sibul equation is expressed as:

$$\frac{S}{d} = 2.44 \times 10^{-5} \left(\frac{F}{d} \right)^{1.66} \left(\frac{U^2}{Fg} \right)^{2.02\left(\frac{F}{d} \right)^{-0.0768}} \qquad \text{Eqn. 7}$$

where:

S = wind setup (feet) above the stillwater level

d = average depth (feet) of water generally along the fetch line

F = fetch length (feet)

U = average wind velocity (feet per second) over the fetch

g = the gravitational constant (32.17 ft/sec^2)

The Bretschneider method is recommended for shallow impoundments with average depths less than 16 feet. Under conditions where the Bretschneider method does not have a numerical solution, the wind setup for the more shallow reservoirs can be computed as the average of the wind setup results for the Zeider Zee and the Sibul equations. For the deeper impoundments, with average depths greater or equal to 16 feet, the Zeider Zee equation is recommended because the equation provides a continuous solution without excessive over-estimation of wind setup.

3. SUMMARY

The state-of-the-practice for freeboard determination includes consideration of combinations of reservoir level/precipitation and wind conditions that are reasonably expected to occur simultaneously. Additional considerations include: settlement, waves induced by landslides or malfunction of the spillway and/or outlet works. At a minimum, freeboard determination for design of high and significant hazard potential impoundments will include the precipitation and wind combinations for **Cases 1** through **4** described in this memorandum, as well as settlement, and malfunction of the spillway and/or outlet works for freeboard determination. Freeboard determination will be based on zero over-wash or over-wash that is not of such magnitude and duration as to threaten the safety of the dam. The Rayleigh Distribution (irregular wave field) will be used for the wave analysis. The recommended method for assessing the over-wash potential is an evaluation of the over-wash rate versus embankment height, in conjunction with an analysis of the probability of exceedance of the precipitation plus wind condition (residual risk based on hydrologic conditions) versus a selected embankment height. A methodology for analysis of mixed populations (e.g. precipitation and wind conditions) is included in USACE EM 1110-2-1415.

Considering information in standard references for freeboard and discussions at the Acceler8 Design Criteria Workshop (Dec. 2004), it is most appropriate to determine freeboard for each impoundment for the four cases of combined precipitation and wind events, at a minimum. The most likely controlling case for freeboard would be for a reservoir level resulting from the PMP for the critical condition, combined with a 100-year wind speed – **Case 1**. The 100-year wind speed will be developed for the specific site location using the Florida Building Code and **Table DCM 2-2** (ASCE, 2003), and with the fetch length of the reservoir. The figures, which are identified as being based on 50-year to 100-year wind speeds, will be conservatively interpreted as representing

50-year wind speeds. Calculation of wind set-up and wave run-up will be in accordance with the applicable portions of the USACE Coastal Engineering Manual, with the adjustments shown above. Bretschneider's and Zeider-Zee equations will be used for wind setup analysis. **Cases 2, 3** and **4** will also be evaluated for each impoundment facility, to consider the different sizes, configurations, orientations and types of impoundments, to determine what conditions will be the controlling factor.

The minimum freeboard will be determined based on **Cases 1**, **2** and **4**. It is recognized that development of a PMW is difficult considering the available information. In addition, knowledge of wind and wave conditions on water surfaces at extreme velocities, such as 200 mph 60-second (or 161 mph 1-hour duration) wind speed for **Case 3**, are not well understood. Some reports note that waves may break up or "blow apart" at high wind conditions. Equations and models for determining wind-wave generation and wind setup therefore may be extremely conservative. Setup reduction has been observed over salt water only, and has been interpreted to be a result of the thick layer of foam developed due to salinity. Consequently, **Case 3** is to be evaluated and used as part of a sensitivity/parametric analysis of setup and wave height analysis, but will not be required as the final basis for determining the minimum freeboard height. Wave generation for **Case 3** will be based on the wave height with a 2 percent frequency. Wave models are unlikely capable of yielding results within a degree of confidence for design for these extreme wind speeds, especially over relatively shallow water bodies. Engineering judgment is to be used in interpreting and applying the results of the **Case 3** analysis. **Case 3** is to be used only for "sensitivity identification" and not as a selected design condition.

Routing of the precipitation event and spillway design will also affect the freeboard determination. For high and significant hazard structures, the design storm is the PMP event unless an IDA demonstrates that a flood less than the PMF results in consequences of dam failure at PMF flood flows that are acceptable. Development and routing of the probable maximum flood (PMF) is to include antecedent storm conditions and un-gated and gated spillway/outlet releases. Spillway design guidelines are covered in Design Criteria Memorandum **DCM-3**. However, antecedent storm conditions for flood routing of the 72-hour PMP will be as follows:

Flood routing for **Case 1** (100 year wind event in combination with the 72-hour PMP) to determine the governing conditions will include the following two scenarios:

1. Routing starts at normal full storage level (uncontrolled spillway crest level),

    a. Route 30 percent of the 72-hour PMF (Time scale 0-hr to 72-hr): un-gated spillway flows, gated structures are in-operable.

    b. A 3-day dry interval with un-gated spillway flow, gated structures are operable (Time scale 72-hr to 144-hr).

    c. Route 100 percent of the 72-hour PMF (Time scale 144-hr to 216-hr): un-gated spillway flows, gated structures are in-operable.

    d. A 10-day dry interval with un-gated spillway flow, gated structures are operable (Time scale 216-hr to 456-hr).

    e. Route 30 percent of the 72-hour PMF (Time scale 456-hr to 528-hr): un-gated spillway flows, gated structures are in-operable.

2. Routing starts at normal full storage level (uncontrolled spillway crest level),

    a. Route 40 percent of the 72-hour PMF (Time scale 0-hr to 72-hr): un-gated spillway flows, gated structures are in-operable.

    b. A 5-day dry interval with un-gated spillway flow, gated structures are operable (Time scale 72-hr to 192-hr).

    c. Route 100 percent of the 72-hour PMF (Time scale 192-hr to 264-hr): un-gated spillway flows, gated structures are in-operable.

    d. A 10-day dry interval with un-gated spillway flow, gated structures are operable (Time scale 264-hr to 504-hr).

    e. Route 40 percent of the 72-hour PMF (Time scale 504-hr to 576-hr): un-gated spillway flows, gated structures are in-operable.

Design of impoundments is a complex process of evaluation of the inflow design flood based on the HPC, spillway, outlet structure sizing and operation, and the potential consequences in the unlikely event of dam failure. A risk based assessment for final determination of the IDF, spillway sizing and freeboard height is required for all hazard classifications. Guidelines for risk based assessment have been prepared by various Federal agencies. The recommended guideline for IDA for CERP impoundments is *Chapter 2, Selecting and Accommodating Inflow Design Floods for Dams*, Federal Energy Regulatory Commission, Engineering Guidelines for the Evaluation of Hydropower Projects (October 1993).

This memorandum provides an overall assessment of referenced regulations/engineering manuals for CERP projects and engineering practice, and the interpretation of reasonable combinations of events that are to be evaluated in the design of water storage impoundments (as defined by the National Dam Safety Act) as a minimum. The project designer will have the most complete and thorough understanding of all of the conditions of the facility being designed, and therefore will be the most knowledgeable of conditions that could reasonably be expected to occur. Therefore, the designer for each facility is responsible for determining if other reasonable combinations of events are likely to occur that could be a controlling factor for the determination of freeboard. The designer is to perform further analysis of other reasonable combinations, if deemed necessary, for determination of the final freeboard height to be used for design.

4. REFERENCES

1. American Society of Civil Engineers, 2003. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-02, Second Edition.

2. American Society of Civil Engineers, 2004. *Guide to the Use of the Wind Load Provisions of ASCE 7-02*.

3. Barnes, Jay, 1998. *Florida's Hurricane History*, University of North Carolina Press.

4. Brater, Ernest F., and Horace W. King, 1976. *Handbook of Hydraulics*, Sixth Edition, McGraw-Hill Book Company.

5. Bretschneider, C.L., 1966. "Engineering Aspects of Hurricane Surge", Chapter 5 in *Estuary and Coastline Hydrodynamics*, Arthur T. Ippen, Ph.D., McGraw-Hill Book Company.

6. Coastal and Hydraulics Laboratory (CHL), U.S. Army Research and Development Center, February 2001. *STWAVE: Steady-State Spectral Wave Model User's Manual for STWAVE, Version 3.0*.

7. Federal Emergency Management Agency, October 1998 (reprinted 2004). *Federal Guidelines for Dam Safety: Hazard Potential Classification Systems for Dams*, FEMA 333.

8. Federal Energy Regulatory Commission (FERC), September 2001. *Engineering Guidelines*, Chapter VIII, Determination of the Probable Maximum Flood.

9. Holler, Albert G., PhD, P.E., 2004. "New Information for Designing Dam Freeboard", Association of State Dam Safety Officials (ASDSO), Annual Conference Proceedings.

10. Jacksonville District, Corps of Engineers, December 4, 2004. *Analysis and Hydraulic Design of Embankment Heights for Aboveground Impoundments and Reservoirs for Projects of the Comprehensive Everglades Restoration Plan*, Working Draft.
11. National Weather Service Hydrometeorological Report HMR 51, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", NOAA, August 1980.
12. National Weather Service Hydrometeorological Report HMR 52, "Application of Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", NOAA, August 1982.
13. National Weather Service Hydrometeorological Report HMR 53, "Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian", NOAA, April 1980.

14. Pathak, Chandra S. (January) 2001. Technical Publication EMA # 390 – *Frequency Analysis of Daily Rainfall Maxima for Central and Southern Florida*, Hydro Information Systems & Assessment Department, Environmental Monitoring & Assessment Division, South Florida Water Management District.

15. U.S. Army Corps of Engineers (USACE, 1984. *Shore Protection Manual*, Coastal Engineering Research Center, Waterways Experiment Station, Second Printing.

16. U.S. Army Corps of Engineers (USACE), June 30, 1995, Design *of Coastal Revetments, Seawalls and Bulkheads*, EM 1110-2-1614.

17. U.S. Army Corps of Engineers (USACE), March 1, 1991. *Inflow Design Floods for Dams and Reservoirs*, ER 1110-8-2(FR).

18. U.S. Army Corps of Engineers (USACE), March 5, 1993. *Hydrologic Frequency Analysis*, EM 1110-2-1415.

19. U.S. Army Corps of Engineers (USACE), March 1, 1996. *Risk-Based Assessment for Evaluation of Hydrology/Hydraulics, Geotechnical Stability, and Economics in Flood Damage Reduction Studies*, ER 1110-2-101.

20. U.S. Army Corps of Engineers (USACE), August 1, 1996. *Engineering and Design*, *Risk-Based Analysis for Flood Damage Reduction Studies*, EM 1110-2-1619.

21. U.S. Army Corps of Engineers (USACE), October 31, 1997. *Hydrologic Engineering Requirements for Reservoirs*, EM-1110-2-1420.

22. U.S. Army Corps of Engineers (USACE), July 31, 2003. *Coastal Engineering Manual*, EM-1110-2-1100, Part II Coastal Hydrodynamics.

23. Waterways Experiment Station (WES), U.S. Army Corps of Engineers, September 1992. *Automated Coastal Engineering System*, *Technical Reference*, Version 1.07.

5. ATTACHMENTS

Attachment DCM 2A Example Wind Speed and Wave Heights Calculation for Freeboard Determination

6. PREPARED BY:

John Haapala, P.E.	Robert C. Tucker, P.E.	Sean Smith, P.E.
Terry Arnold, P.E.	Lori Hadley, P.E.	
Yung Shen, Ph.D., P.E.	Stephen Partney, P.E.	

APPROVALS:

Tommy B. Strowd, P.E. **Date**
Assistant, Deputy Executive Director,
South Florida Water Management District

Stephen C. Duba, P.E. **Date**
Chief, Engineering Division - SAJ
U.S. Army Corps of Engineers

Jerry Brooks **Date**
Deputy Director, Division of Water Resource Management
Florida Department of Environmental Protection

DCM 2 Attachment A

Example of Determining Wind Speed and Wave Height for Freeboard Requirement Based on DCM-2, Case 1

Introduction

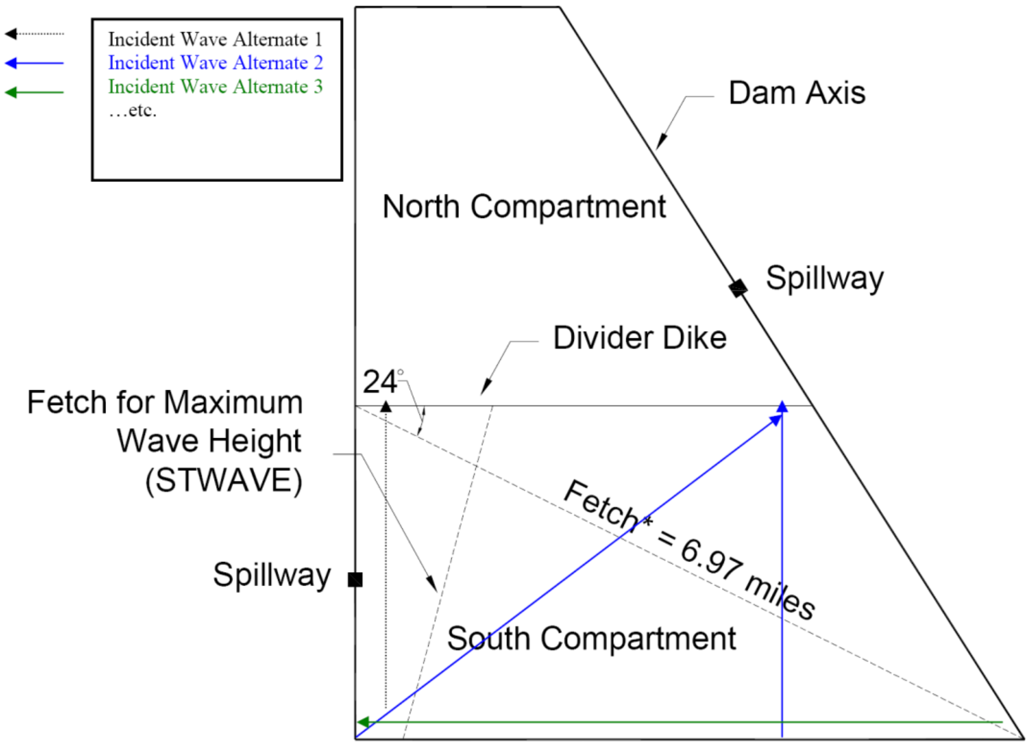
An example calculation of wind speed, wind setup and wave run-up for freeboard determination was prepared using Reservoir A-1, located about 17 miles south of Lake Okeechobee. In general the reservoir will be trapezoidal in shape, with a surface area of about 17,000 acres and a perimeter embankment length of about 120,000 lineal feet. Interior dikes are planned to reduce the maximum over-water fetch distance that would generate waves. Two spillways were assumed for Reservoir A-1, each with a capacity of 3,000 cfs. Each spillway consisted of an uncontrolled spillway overflow section and a gated spillway section. The uncontrolled spillway section consisted of a 100-foot long, concrete ogee crest with a capacity of approximately 3,000 cfs at 4 feet of head. The inflow design flood (IDF) of 51.9 inches (4.3 ft) was assumed for this analysis. The IDF was routed for the antecedent flood conditions shown in Section 3 (**DCM –2**) and the resulting routed flood peak surcharge depth was 4.5 ft. The gated section of the spillway was designed for additional discharge capacity and was sized for emergency evacuation of the reservoir in accordance with USACE ER 1110-2-50, and the gated section was assumed to be closed/inoperative for purposes of routing the IDF. The preliminary embankment layout and spillway locations are shown on **Figure DCM 2A-1**.

Reservoir freeboard related to wind conditions has traditionally been calculated following procedures in the Shore Protection Manual (SPM) and the Coastal Engineering Manual (CEM). In recent years, computer based numerical codes have been developed (e.g. STWAVE and ACES). The manual/spreadsheet approach based on the SPM/CEM (see discussion in Section 2.4, **DCM – 2**) is satisfactory for analysis of alternatives. However, the numerical programs STWAVE and ACES (or equivalent) are required by the USACE for the final selected alternative for CERP impoundments. In addition, the residual risk associated with over-wash rates versus freeboard height and the freeboard height versus frequency of exceedance are required for analysis and selection of the design freeboard.

The following example includes: 1) manual/spreadsheet following the procedures in the SPM/CEM, 2) computer programs STWAVE and ACES results, and 3) residual risk assessment. A comparison of the results by the manual/spreadsheet and computer program is also provided for information.

1. Manual/Spreadsheet (SPM/CEM Procedure)

Calculation of wind and wave related freeboard requirements involve numerous data sources and technical references, in addition to project specific conditions and engineering judgment. The steps below provide an example of the process of "weaving" through the various references required for freeboard determination. The example demonstrates determination of the required freeboard at



Not to Scale

(*) Note: The procedure for estimating the fetch length is to construct nine radials from the point of interest at 3 degree intervals. The average length of radials is used to represent the fetch length at the point of interest. In order to simplify the computations, only the length of the central radial is used for the computations of wind set-up and wave run-up in this example. Since the length of the central radial is the longest among nine radials, this approach is slightly conservative. The procedure of constructing nine radials to compute the representative fetch length should be followed for design phase.
When estimating fetch length, some of the directions along the central radials are not normal to the axis of the embankment. Corrections for central radials not normal to the axis of embankments should be made following the method described in Bureau of Reclamation's report "ACER Technical Memorandum No. 2, Freeboard Criteria and Guideline for computing Freeboard Allowances for Storage Dams, Revised 1992".

ACCELER8
evergladesnow.org

FIGURE DCM 2A-1

EXAMPLE RESERVOIR A-1
EMBANKMENT PLAN

Reservoir A-1 South Compartment, for one alternative and one of the **Cases** described in **DCM – 2**. This example is based on **DCM-2**, **Case 1**, with an inflow design flood based on the PMP event and a 100-yr annual recurrence interval (ARI) wind speed. The example does not include internal dikes in the South Compartment. The fetch length is 6.97 miles and the angle between the central radial and the axis normal to the embankment is 24 degrees, as shown on **Figure DCM 2A-1**. The maximum water storage depth is 16.5 ft (normal full storage level of 12 ft plus routed peak surcharge depth of 4.5 ft) resulting from the design PMP (**Case 1** specified in **DCM-2**) antecedent storm sequence. A complete determination of freeboard would also include analysis of the other **Cases** described in **DCM-2**.
Six primary references are used in this example calculation, which are:

- U.S. Army Corps of Engineers (USACE), 1984. *Shore Protection Manual*, Coastal Engineering Research Center, Waterways Experiment Station, Second Printing. Referred to as SPM in this document

- U.S. Army Corps of Engineers (USACE), July 31, 2003. *Coastal Engineering Manual*, EM-1110-2-1100, Part II Coastal Hydrodynamics. Referred to as CEM in this document.

- U.S. Army Corps of Engineers (USACE), March 5, 1993. *Hydrologic Frequency Analysis*, EM-1110-2-1415.

- U.S. Army Corps of Engineers (USACE), October 31, 1997. *Hydrologic Engineering Requirements for Reservoirs*, EM-1110-2-1420.

- U.S. Army Corps of Engineers (USACE), March 1, 1991. *Inflow Design Floods for Dams and Reservoirs*, ER 1110-8-2 (FR).

- U.S. Bureau of Reclamation (USBR), 1988. *Downstream Hazard Classification Guidelines*, ACER Technical Memorandum No. 11, U.S. Department of the Interior.

In general, freeboard determination for wind and wave conditions consists of three areas. They are:

a. Determine Design Wind Condition,

b. Wind Setup, and

c. Wave Height and Wave Run-up.

Steps 1 through 33 summarize the methodology described in **DCM-2**, and the technical references used in the calculations. The calculation steps consist of:

- Determine the appropriate wind speed – Steps 1 through 8.

- Determine the wind stress factor for use in equations that determine wave parameters – Steps 9 and 10.

- Determine deep-water wave parameters – Steps 11 through 13.

- Determine whether use of deep water or shallow-water wave parameters are indicated – Step 14.

- Because shallow water wave conditions were indicated in Step 14, determine shallow water wave parameters including the maximum wave height – Steps 15 through 17.

- Determine wind set-up and the resulting depth at the toe of the embankment – Steps 18 and 19.
- Determine whether depth limited wave breaking occurs – Step 20.
- Because run-up curves are based on deep-water wave heights, determine an equivalent deep-water wave height for the shallow-water wave height – Steps 21 and 22.
- Determine run-up parameters – Steps 23 through 27.
- Determine the smooth surface run-up – Step 28.
- Correct run-up for scale effects – Step 29.
- Reduce run-up for dam surface roughness – Step 30.
- Adjust run-up for fetch axis angle with dam – Steps 31 and 32.
- Determine IDF and wind related freeboard height for **Case 1** – Step 33.

A. Determine Design Wind Speed

The design wind speed is used to develop setup in the reservoir induced by wind acting on the surface of the reservoir, as well as wave height and run-up on the embankment slope. The wind speed used to calculate reservoir setup and wave height is defined as the maximum sustained over-water wind speed measured at height of 33 feet (10 meters). Wind speed records are typically reported as 3-second gusts or one-minute duration over land speeds measured at a height of 33 feet. The design wind speed in steps 1 through 10.

Step 1: Select 50-yr 3-sec gust wind speed

The wind speed map shown in ASCE 7-02 and the Florida Building Code (duplicated as **Figure DCM 2-2**) will be used to determine the site specific wind speed. The wind speed shown on the map is the 3-second, overland wind gust in miles per hour (mph), occurring 33 feet (10 meter) above the ground surface. The wind speed is interpreted as the 50-year annual recurrence interval (ARI). From **Figure DCM 2-2**, the 50-yr ARI, 3-sec gust wind speed at Reservoir A1 site would be 125 mph.

Step 2: Determine 100-yr ARI, 3-sec gust wind speed

Based on **Table DCM 2-2**, the wind speed conversion factor between 100-yr and 50-yr ARI wind speed is 1.07. Therefore, the 100-yr ARI, 3-sec gust wind speed would be 133.8 mph (125 mph in Step 1 multiply by 1.07).

Step 3: Determine 100-yr 1-hr wind speed

The 3-sec gust wind speed is then converted to a 1-hr wind speed. The 1-hr wind speed approximates the appropriate wind duration for the fetch distance being analyzed. From Eqn. 1 (**DCM-2)**, the ratio between 3-sec gust wind speed and 1-hr wind speed is 1.51. Therefore, the 1-hr wind speed would be 88.6 mph (133.8 mph in Step 2 divided by 1.51).

Step 4: Determine 1-hr wind speed over water

The wind speed computed in Step 3 is an over-land wind speed and will be converted into wind speed over-water in order to compute the wave height. Figure II-2-20 of the CEM is used for conversion of over-land to over-water winds speeds with fetch lengths less than 10 miles. The ratio of over-water to over-land wind speed (Figure II-2-20) is 1.2. Therefore, the 100-yr ARI, 1-hr over-water wind speed would be 106.3 mph (88.6 mph in Step 3 multiplied by 1.2).

Step 5: Estimate fetch-limited duration

Based on the length of travel over-water (fetch), the wind speed used to compute the wave-length, period and height is based on the fetch length and the wind duration required to develop the full wave height (i.e. fetch-limited duration). From Eqn. 4 (**DCM-2**), the fetch-limited duration for a fetch distance of 6.97 miles would be 84 minutes for the 106.3 mph wind speed calculated in Step 4.

Step 6: Estimate ratio between 84-minute wind speed and 1-hr wind speed
From Eqn. 2 (**DCM-2**), the ratio between 84-minute wind speed and 1-hr wind speed would be 0.978.
Step 7: Determine 100-yr 84-minute wind speed
This step adjusts the 1-hr wind speed to an equivalent 84-minute wind speed. From Step 6, the conversion ratio is 0.978. Therefore, the fetch-limited wind speed would be 104 mph (106.3 mph in Step 4 multiplied by 0.978).
Step 8: Stability correction
For some conditions an adjustment for stability of the boundary layer may be needed due to air-sea temperature differences. The adjustment (using a correction factor, R_T) may be needed for fetches longer than 16 km (10 miles) as described in the CEM (pg. II-2-36). Since the fetch length of this example the fetch length is 6.97 miles, an adjustment due to air-sea temperature difference is not needed. The wind speed of 104 mph is the effective wind speed (U) defined in Eqn. 3-27 of the SPM.
Step 9: Compute wind stress factor
The wind stress factor (U_A) is defined in Eqn. 3-28b of SPM Vol. I. This correction is needed to reflect the surface roughness due to wave over the water surface. From Eqn. 3-28b, the wind stress factor (U_A) was calculated to be 180 mph.
Step 10: Change unit of U_A
The wind speed computed in Step 9 is then converted to feet per second (ft/sec) to compute the shallow-water wave height and deep-water wave height. The wind stress factor (U_A) would be 264 ft/sec (180 mph in Step 9 multiplied by 1.467).

B. Wind Setup and Wave Properties

The design wind speed determined in A., above, is used to calculate wave properties for the reservoir. Wave development varies based on the geometry of the reservoir (depth of water, fetch length, etc.). The equations differ depending on whether deep-water, shallow water or transitional conditions occur. Steps 11 through 23 demonstrate wind setup calculations and development of wave properties for one condition.
Step 11: Compute deep-water significant wave height
Based on the fetch distance of 6.97 miles and a U_A of 264 ft/sec, the deep-water wave height is 14.25 ft. (Eqn. 3-33c of SPM Vol. I).
Step 12: Compute deep-water wave period
The deep-water wave period of 6.02 second, is calculated based on the fetch distance of 6.97 miles and U_A of 264 ft/sec (Eqn. 3-34c of the SPM).
Step 13: Compute deep-water wave length
The maximum wave height of 186 ft is then computed from Eqn. 2-8a (SPM Vol. I), using the deep-water wave period for the maximum wave height and the deep-water wave-length
Step 14: Compute relative depth
The relative depth is used to determine whether the computations of wave height and period should be based on the shallow-water equations or deep-water equations. The SPM (pg. 2-9) states that the shallow-water equations should be used to compute the wave period and height when the relative depth is less than 0.5. The relative depth (d/L_o) is the ratio between effective depth and deep-water wave-length. The calculated relative depth is 0.089 (Effective depth of 16.47 ft. divided by the deep-water wave-length (186 ft) in Step 13. Since the ratio is less than 0.5, transitional/shallow-water conditions are indicated.
Step 15: Compute shallow-water wave period
Based on the effective water depth of 16.47 ft, fetch distance of 6.97 miles and U_A of 264 ft/sec, the shallow-water wave period, computed from Eqn. 3-40 in the SPM, is 5.13 seconds.
Step 16: Compute shallow-water significant wave height
Based on the effective water depth of 16.47 ft, fetch distance of 6.97 miles and U_A of 264 ft/sec, the shallow-water wave height, computed from Eqn. 3-39 in the SPM, is 7.82 ft.

Step 17: Compute shallow-water wave height for zero over-wash

The freeboard height that results in zero over-wash is the design criteria for high and significant hazard dams, for wave run-up conditions and freeboard determination due to the erosive nature of embankment material (see **DCM-2**, Section 2 discussion). A wave height with a 2% frequency will be used to analyze a "low probability of over-wash" condition, to evaluate alternative embankment sections. In addition, the 2% frequency wave height will be used for a manual check of the STWAVE generated value.

The shallow-water wave height in Step 16 is the significant wave height. Using Equations 3-7 and 3-10 in the SPM, the 2% wave height ($H_{2\%}$) is calculated as 1.40 times the significant wave height. Therefore, the maximum wave height would be 10.95 ft (7.82 ft. in Step 16 multiplied by 1.40). Note: If deep-water wave length conditions are determined, Steps 15 through 17 would use deep water equations.

Step 18: Compute wind set-up

Based on the effective depth of 16.47 ft, fetch distance of 6.97 miles, and effective wind speed of 104 mph, the wind set-up is then computed using Eqn. 15-1 (USACE EM 1110-2-1420), as 3.31 ft. It should be noted that the effective wind speed (U) in Step 7 is used to compute the wind set-up not the wind stress factor (U_A) in Step 9 (Note, for average stillwater depth less than 16 feet, the Bretschneider method would be used. See Section 2.5, **DCM – 2**).

Step 19: Compute water depth at toe of the structure (dam)

The water depth at the toe of the structure (d_s) would be 19.78 ft, which is the sum of the effective depth of 16.47 ft plus wind set-up of 3.31 ft in Step 18.

Step 20: Check for depth limited wave breaking

Based on SPM Eqn. 2-91 and 3-50, the depth limited wave breaking height is 0.78 times depth. The depth limited wave height is 15.43 feet (0.78 multiplied by 19.78 ft from Step 19). Therefore, the maximum wave height of 10.95 feet from Step 17 is not limited by wave breaking.

Step 21: Estimate ratio between deep-water wave height and equivalent deep water wave height

Run-up curves were developed in reference to equivalent deepwater wave heights. The relative depth calculated in Step 14 is less than 0.5. Therefore, an equivalent deep water wave height (H'_o) needs to be computed. The value of H/H'_o (shallow-water wave height / equivalent deep water wave height) is estimated based on the value of d/L_o (effective depth / deep water wave length). The relationship between d/L_o and H/H'_o is listed in Table C-1 of SPM Vol. II, Appendix C. Based on d/L_o of 0.089 in Step 14, the H/H'_o would be 0.9433.

Step 22: Compute equivalent deep-water wave height

The equivalent deep-wave wave height (H'_o) would be 11.61 ft (10.95 ft in Step 17 divided by 0.9433 in Step 18).

Step 23: Compute ratio of d_s/H'_o

The ratio between the water depth at toe of dike and the equivalent deep-water wave height is then calculated. The ratio would be 1.70 (19.78 ft in Step 19 divided by 11.61 ft in Step 22).

B. Wave Run-up

Wave run-up is based on the wave height, slope and roughness of the run-up surface, and the incident angle of the wind to the slope surface. Steps 24 through 32 demonstrate the calculation of wave run-up for one condition.

Step 24: Compute ratio of $H'_o / (g * T_{o2})$

To determine the wave run-up, calculate $H'_o / (g * T_{o2})$. H'_o is the equivalent deep-water wave height in Step 22 and T_o is the deep-water wave period in Step 12. The computed value of $H'_o / (g * T_{o2})$ would be 0.0100. This value is used to estimate the wave run-up, R, at the dam. Figures 7-8 through 7-12 in SPM Vol. II show the relationships between R/H'_o and slope of upstream face of the dam (reservoir side) for a series of d_s/H'_o values. These relationships are also expressed as a function of a series of $H'_o / (g * T_{o2})$ curves. Figure 7-10 in the SPM shows the relationship between R/H'_o and slope of upstream face of the dike for the value of d_s/H'_o at 0.8. Figure 7-11 in the SPM shows the relationship between R/H'_o and slope

of upstream face of the dike for the value of d_s/H'_o at 2.0. Since the value of d_s/H'_o in Step 23 is 1.70, the value of R/H'_o needs to be interpolated between Figure 7-10 (d_s/H'_o at 0.8) and Figure 7-11 (d_s/H'_o at 2.0).
Step 25: Estimate R/H'_o for d_s/H'_o at 0.8
For slope of upstream face of dike at 3H to 1V, Curve 1 in the attached **Figure DCM 2-A2** was developed based on the values of R/H'_o and a series of H'_o / (g * T_{o2}) values for d_s/H'_o at 0.8. For value of H'_o / (g * T_{o2}) at 0.0100, the value of R/H'_o was estimated to be 1.20 from Curve 1 in **Figure DCM 2-A2**.
Step 26: Estimate R/H'_o for d_s/H'_o at 2.0
For slope of upstream face of dike at 3H to 1V, Curve 2 in the attached **Figure DCM 2-A2** was developed based on the values of R/H'_o and a series of H'_o / (g * T_{o2}) values for d_s/H'_o at 2.0. For value of H'_o / (g * T_{o2}) at 0.0132, the value of R/H'_o was estimated to be 1.25 from Curve 2 in **Figure DCM 2-A2**.
Step 27: Estimate R/H'_o for d_s/H'_o at 1.70
The value of R/H'_o was interpolated between Steps 24 and 25/26 for the value of d_s/H'_o at 1.70. The ratio of R/H'_o would be 1.24.
Step 28: Compute smooth surface wave run-up
Wave run-up for the design wind speed of 104 mph (from Step 8) and fetch length of 6.97 miles would be 14.40 ft (1.24 in Step 27 multiplied the equivalent deep-water wave height at 11.61 ft in Step 22).
Step 23: Compute ratio of d_s/H'_o
The ratio between the water depth at toe of dike and the equivalent deep-water wave height is then calculated. The ratio would be 1.70 (19.78 ft in Step 19 divided by 11.61 ft in Step 22).

C. Wave Run-up

Wave run-up is based on the wave height, slope and roughness of the run-up surface, and the incident angle of the wind to the slope surface. Steps 24 through 32 demonstrate the calculation of wave run-up for one condition.
Step 24: Compute ratio of H'_o / (g * T_{o2})
To determine the wave run-up, calculate H'_o / (g * T_{o2}). H'_o is the equivalent deep-water wave height in Step 22 and T_o is the deep-water wave period in Step 12. The computed value of H'_o / (g * T_{o2}) would be 0.0100. This value is used to estimate the wave run-up, R, at the dam. Figures 7-8 through 7-12 in SPM Vol. II show the relationships between R/H'_o and slope of upstream face of the dam (reservoir side) for a series of d_s/H'_o values. These relationships are also expressed as a function of a series of H'_o / (g * T_{o2}) curves. Figure 7-10 in the SPM shows the relationship between R/H'_o and slope of upstream face of the dike for the value of d_s/H'_o at 0.8. Figure 7-11 in the SPM shows the relationship between R/H'_o and slope of upstream face of the dike for the value of d_s/H'_o at 2.0. Since the value of d_s/H'_o in Step 23 is 1.70, the value of R/H'_o needs to be interpolated between Figure 7-10 (d_s/H'_o at 0.8) and Figure 7-11 (d_s/H'_o at 2.0).
Step 25: Estimate R/H'_o for d_s/H'_o at 0.8
For slope of upstream face of dike at 3H to 1V, Curve 1 in the attached **Figure DCM 2-A2** was developed based on the values of R/H'_o and a series of H'_o / (g * T_{o2}) values for d_s/H'_o at 0.8. For value of H'_o / (g * T_{o2}) at 0.0100, the value of R/H'_o was estimated to be 1.20 from Curve 1 in **Figure DCM 2-A2**.
Step 26: Estimate R/H'_o for d_s/H'_o at 2.0
For slope of upstream face of dike at 3H to 1V, Curve 2 in the attached **Figure DCM 2-A2** was developed based on the values of R/H'_o and a series of H'_o / (g * T_{o2}) values for d_s/H'_o at 2.0. For value of H'_o / (g * T_{o2}) at 0.0132, the value of R/H'_o was estimated to be 1.25 from Curve 2 in **Figure DCM 2-A2**.
Step 27: Estimate R/H'_o for d_s/H'_o at 1.70
The value of R/H'_o was interpolated between Steps 24 and 25/26 for the value of d_s/H'_o at 1.70. The ratio of R/H'_o would be 1.24.
Step 28: Compute smooth surface wave run-up
Wave run-up for the design wind speed of 104 mph (from Step 8) and fetch length of 6.97 miles would be 14.40 ft (1.24 in Step 27 multiplied the equivalent deep-water wave height at 11.61 ft in Step 22).
Step 29: Wave run-up correction due to scale effects
Figures 7-8 through 7-12 of the SPM were developed based on small-scale laboratory tests. The wave run-up values predicted by Figures 7-8 through 7-12 are expected to be smaller than the wave run-up on prototype structure because of the inability to scale roughness effects in small-scale laboratory tests

(reference SPM Vol. II). The wave run-up values from Figure 7-8 through 7-12 are therefore adjusted for scale effects by using Figure 7-13 in the SPM. The correction factor would be 1.12 for slope of upstream face of dam at 3H to 1V. The corrected wave run-up value of 16.13 ft is then calculated (14.40 ft in Step 28 multiplied by 1.12).

Step 30: Correction due to dam surface roughness

As described in the example description, the upstream slope is assumed to be constructed using random placed quarry stone. The surface roughness factor for random placed quarry stone is estimated to be 0.55, from Table 7-2 of the SPM. The corrected wave run-up due to dam surface roughness would be 8.87 ft (16.13 ft in Step 29 in multiplied by 0.55).

Step 31: Correction factor due to fetch axis not normal to the dam axis

The wave run-up computed in Step 28 is based on the assumption that the fetch axis is normal to the dam axis. If the fetch axis is not normal to the dam axis, the wave run-up in Step 28 needs to be corrected. A correction curve for incident angles greater or less than 90 degrees has been developed by the U.S. Bureau of Reclamation (Figure 8 of ACER TM No. 2). The angle between the axis of fetch length at 6.97 miles and the embankment axis was calculated to be 24 degrees. From Figure 8 in ACER TM No. 2, the correction factor would be 0.957.

Step 32: Wave run-up

The adjusted wave run-up would be 8.5 feet (0.957 in Step 31 multiplied by 8.87 ft in Step 30).

D. Freeboard Calculation

The required design freeboard (minimum height above the normal full storage level to the dam crest) is calculated as the sum of:

Flood Surcharge (routed flood) depth
+ Wind Setup
+ Wave height + run-up
+ Embankment settlement
+ Waves induced by potential landslides
+ Provision for malfunction of the spillway or discharge structure
= Minimum required design freeboard

Freeboard for this example is calculated in Step 33, below.

Step 33: Minimum required freeboard for **Case 1**

The IDF is 4.3 ft and the routed flood depth is 4.5 ft, resulting from the antecedent flood sequence described in Section 3, **DCM-2**. The spillway consists of an un-gated overflow structure and no provision for malfunction of the spillway or discharge structure is included. In addition there is no landslide potential. The calculated freeboard for this condition would be 16.3 ft (4.5 ft of surcharge plus 3.31 ft of wind setup in Step 18 plus 8.5 ft of wave height and run-up in Step 32), not including embankment settlement.

2. STWAVE Model Wave Heights/ACES Run-up and Overtopping.

A. STWAVE/ACES Analysis

STWAVE was used to develop the critical wave height, and wave run-up and overtopping were calculated using the formulations available in the USACE ACES model. STWAVE was used to determine the maximum wave height from the spectrum analysis (of the irregular wave field) for the southern compartment shown on **Figure DCM 2A-1.** The run-up and overtopping analysis resulted in a height of 8.37 ft, corresponding to a wind approaching from the southwest (195 degree measured CW from 0 degree north). The calculated wave height (Hs), minimum embankment height (determined by ACES) and the calculated freeboard are shown below.

STWAVE: Hs = 8.37 ft => ACES => Height = 29 ft,

Freeboard (not incl. flood surcharge) = 12.53 ft
A comparison of values from both manual computations and STWAVE/ACES for the various computation steps are summarized in **Table DCM 2-A2.**

B. STWAVE/ACES Comparison with Manual Calculations

Run-up and overtopping were also calculated using ACES for two additional wave heights (7.28 ft and 7.82 ft) to compare with the results of the manual computations. The wave height of 7.28 ft corresponds to the STWAVE wave height for a wind along the fetch axis described in the example problem (1., above). The significant wave height from the STWAVE model was different than the location of the longest fetch in the South Compartment (which was used for the example problem manual computations). The second wave height analyzed (7.82 ft from Step 16) corresponds to the significant wave height calculated during the manual computation of the example problem. For comparison, a minimum embankment height (normal full storage level (still water) depth + freeboard) was selected for the embankment height corresponding to an overtopping rate (using the irregular wave field) that rounds to 0.1 cfs/ft. The results of the analysis are summarized in **Table DCM 2-A2**. Embankment height and the associated freeboard calculated by the numerical models are shown below.

Longest fetch Axis. A comparative analysis using both the numerical modelling (STWAVE and ACES) and the manual computations was performed using the longest fetch distance for the southern compartment (shown on **Figure DCM 2A-1)**. The results of the analysis were:

STWAVE: Hs = 7.28 ft => ACES => Height = 28 ft,
Freeboard (not incl. flood surcharge) = 11.53 ft
Manual Comp: Hs = 7.82 ft => ACES => Height = 28 ft,
Freeboard (not incl. flood surcharge) = 11.53 ft

SPM/CEM Significant Wave Height. One additional comparison calculation was also performed. The calculated wave height (Hs) for the longest fetch from the manual calculation was input to the ACES model and used to calculate the freeboard. The results are shown below.
Hs = 7.82 ft from Manual Comp. => Procedure => Height = 28.14 ft,
Freeboard (not incl. flood surcharge) = 11.67 ft
Note that based on the manual calculations in **Table DCM2-A1**, the comparable value of freeboard is 11.80 feet (3.31 ft in Step 18 plus 8.49 ft in Step 32). The comparable height value from the manual calculation would be 28.3 feet (12.0 ft normal full storage level, plus 4.5 ft routed surcharge depth, plus 3.3 ft wind setup, plus 8.5 ft wave run-up).
From these results, it can be concluded that, for this particular case, the minimum freeboard determined using the hand computation method (with a wave height of 2 percent frequency using the Rayleigh Distribution) is in good agreement with the minimum freeboard determined using the combined STWAVE/ACES method (using wind conditions along the fetch axis) based on an overtopping rate of 0.1cfs/ft. Analysis for freeboard determination for design, must also include variable wind directions and the bathymetric conditions. As the incident wind direction changes, the wave heights at the down-wind embankment are affected not only by the fetch distance, but the water depth and geometry of the impoundment as well. The fact that the manual computation procedure does not necessarily calculate the maximum possible wave height at the embankment may be a concern. In order to analyze the maximum possible wave height at the embankment, extensive manual computations would be required. Numerical models capable of determining wave transformation over the entire impoundment allowing for variable wind directions and bathymetric conditions (such as STWAVE and ACES) provide an effective capability to analyze the conditions at the CERP impoundments.
Numerical modelling and manual computations can be used for determination of the wind-related portion of freeboard determination. The use of the numerical models requires significant setup and run-time, is not always possible or desire-able for the design of alternatives. Manual computations with sufficient analysis and sensitivity analysis related to wave transformation and bathymetric conditions would be suitable for

analysis of alternative embankment sections, but can require extensive analysis of complex conditions. Numerical models are required by the USACE for the selected design alternative. It is also recommended that the results of final computer- generated parameters for the selected design alternative, be compared with the traditional hand calculation methods. A direct one-to-one comparison may not result in an exact match. However, order of magnitude agreement should result.

Summary

The examples above provide an estimate of freeboard for only one of the Cases defined in **DCM-2**. In order to complete the determination of the design freeboard, analysis of **Cases 2, 3,** and **4** (described in **DCM-2 Section 3),** other combinations of conditions that could be likely to occur, and a more detailed analysis of the spillway capacity and routing of the IDF (as described in Section 3, **DCM-2** and **DCM-3, Spillway Capacity and Reservoir Drawdown Criteria)**, is required. Alternative spillway configurations, such as a wider spillway (gated or un-gated) that would reduce the peak routed flood depth should be analyzed. In addition, alternatives to reduce the fetch length and/or wave height/run-up should also be analyzed to reduce the wind-related portion of the freeboard height.

Due to the diverse nature of the data that is used, and the various reference documents containing all of the necessary data, it is important that designers fully understand and utilize the referenced documents and the background. The example “weave” through the referenced regulations/engineering manuals, and the **Case 1** condition is just one of the reasonable combinations of events described in **DCM-2** that is to be evaluated for the design of CERP impoundments classified as a dam. The project designer will have the most complete and thorough understanding of all of the conditions of the facility being designed, and therefore will be the most knowledgeable of conditions that could reasonably be expected to occur. Therefore, the designer for each facility is responsible for analyzing the four Cases presented in **DCM-2** and determining if other reasonable combinations of events are likely to occur. In addition, the designer must apply engineering judgement in analyzing the risk and acceptable over-wash rate that is selected for each embankment.

The example is based on the calculations in the South Compartment without internal dikes. The fetch length (F) is 6.97 miles and the angle between the central radial and the axis normal to the embankment is 24 degrees. The effective (routing of the IDF and the antecedent flood sequence) reservoir water depth is 16.47 ft (d), which is based on reservoir bottom at El. 12.0 ft plus the routed peak surcharge depth of 4.47 feet during the PMP.

Table DCM 2-A1 – Example Computation and Numerical Model Results: Wind Setup – Wave Runup - Freeboard

Description		Manual Calculations			
Step	**Computed Item**	**Value**	**Equation**	**Reference**	**Remark**
A.	**Determine design Wind Speed**				
1	50-yr 3-sec wind	125 mph		Figure DCM 2-2	
2	100-yr 3-sec wind	133.8 mph	= 125 * 1.07	Table DCM 2-2	
3	100-yr 1-hr wind	88.6 mph	= 133.8 / 1.51	Eq. 1, DCM-2	wind speed over land
4	wind speed over water(1-hr)	106.3 mph	= 88.6 * 1.2	Fig II-2-20, CEM	
5	fetch-limited duration	84 min.		DCM-2, Eqn. 4	
6	ratio of wind speed (84 min) to wind speed (1-hr)	0.978		Eq. 2, DCM-2	
7	wind speed over water(84-min)	104 mph	= 106.3 * 0.978		
8	stability correction	none		pp. II-2-36, CEM	
9	wind stress factor, U_A	180 mph	Eq. 3-28b	pp. 3-30, SPM Vol. I	
10	wind stress factor, U_A	264 ft/sec	= 180 *1.467		change mph to ft/sec
B.	**Wind Setup and Wave Properties**				
11	deep water significant wave	14.25 ft	Eq. 3-33c	pp. 3-44, SPM Vol. I	

	height				
12	deep water wave period, T_0	6.02 sec	Eq. 3-34c	pp. 3-44, SPM Vol. I	
13	deep water wave length, L_o	186 ft	Eq. 2-8a	pp. 2-9, SPM Vol. I	
14	relative depth, d/ L_o	0.089	=16.47 / 186	pp2-9, SPM	
15	shallow-water wave period	5.13 sec	Eq. 3-40	pp. 3-55, SPM Vol. I	
16	shallow-water significant wave height	7.82 ft	Eq. 3-39	pp. 3-55, SPM Vol. I	
17	shallow water 2% wave	10.95 ft	=1.40 * 7.82	Eq. 3-7 and 3-10 of SPM Vol. I	
18	wind set-up	3.31 ft	Eqn 15-1	EM 1110-2-1420	
19	water depth at toe of dam, d_s	19.78 ft	= 3.31 + 16.47		
20	depth limited wave breaking	15.43 ft	0.78*19.78	Eqn. 2-91 and 3-50 of SPM	
21	H / H'_o	0.9433		Table C-1, SPM Vol. II Appendix C	based on d/L_o @ 0.089
22	H'_o, equivalent deep-water wave height	11.61 ft	= 10.95 / 0.9433		
23	d_s / H'_o	1.70	= 19.78 / 11.61		
C.	**Wave Height and Runup**				
24	$H'_o / (g * T_0^2)$	0.0100	= 11.61 / (32.17 * 6.02^2)	Fig. 7-8 through 7-12, SPM Vol. II	
25	R / H'_o for d_s / H'_o @ 0.8	1.20		Fig 7-10, pp. 7-21, SPM Vol. I & Fig A2	based on $H'_o / (g * T_0^2)$ @ 0.0100
26	R / H'_o for d_s / H'_o @ 2.0	1.25		Fig 7-11, pp. 7-22, SPM Vol. I & Fig A2	based on $H'_o / (g * T_0^2)$ @ 0.0100
27	R / H'_o for d_s / H'_o @ 1.70	1.24			interpolate R / H'_o between steps 25 & 26

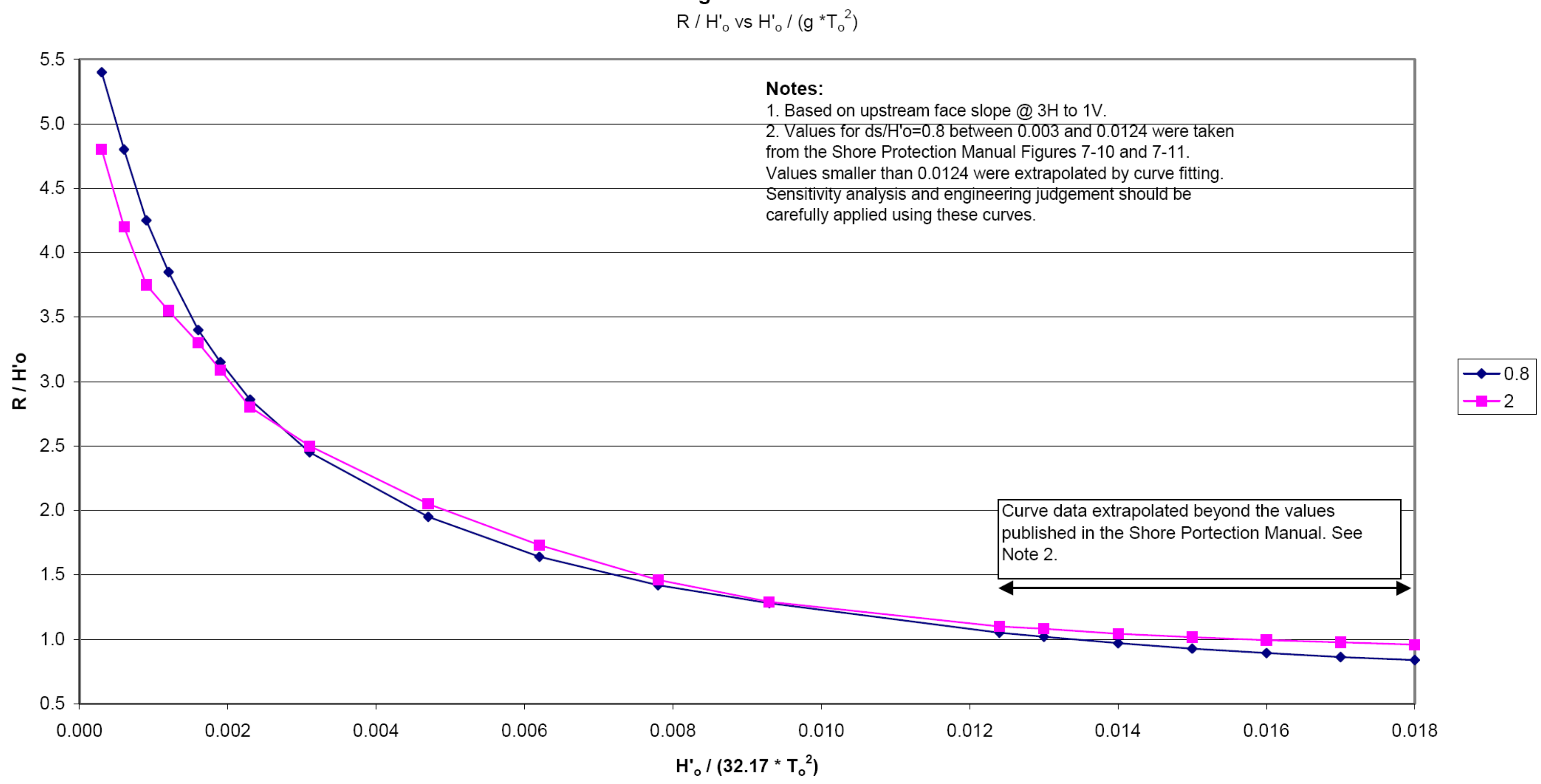
28	wave run-up, R	14.40 ft	= 1.24 * 11.61		
29	wave runup correction due to scale effects	16.13 ft	=1.12 * 14.40	Figure 7-13, SPM Vol. II	
30	wave runup corrected due to surface roughness	8.87 ft	= 16.13 * 0.55	Table 7-2, SPM Vol. II	Based on surface roughness factor of 0.55
31	correction factor due to angle	0.957		Figure 8	ACER TM No. 2
32	wave runup at embankment	8.5 ft	= 0.957 * 8.87		
D.	**Freeboard Calculation**				
33	Required Freeboard – **Case 1** (not including settlement, landslides induce waves or spillway/discharge structure malfunction)	~16.30 ft	= 4.5+3.31 + 8.5		wave runup + wind setup

Table DCM 2-A2: Example Problem - Reservoir A-1 Southern Compartment

	Still Water Depth (NFSL)	Wind Speed	Wind Setup	Effective Depth	Significant Wave Height	Wave Period	Slope Type	Runup	Qo*	Overtopping (cfs/ft) by embankment height										
	(ft)	(mph)	(ft)	(ft)	(ft)	(sec)		(ft)		20ft	25ft	26ft	27ft	28ft	29ft	30ft	31ft	32ft	33ft	34ft
STWAVE Maximum	16.47	106.33	3.27	19.74	8.37	6.7	Rough 1V:3H	8.25	0.020	22.208	1.895	1.026	0.528	0.272	0.129	0.057	0.024	0.009	0.003	0.001
STWAVE Fetch Axis	16.47	106.33	3.27	19.74	7.28	5.6	Rough 1V:3H	6.71	0.020	19.021	0.790	0.342	0.147	0.055	0.019	0.005	0.001			
Manual Comp Fetch Axis	16.47	106.33	3.27	19.74	7.82	5.1	Rough 1V:3H	6.63	0.020	20.895	0.824	0.351	0.148	0.055	0.018	0.005	0.001			

FINAL DRAFT

Figure DCM 2- A2



Appendix B: Example of wind generated wave software (cgWindWave)

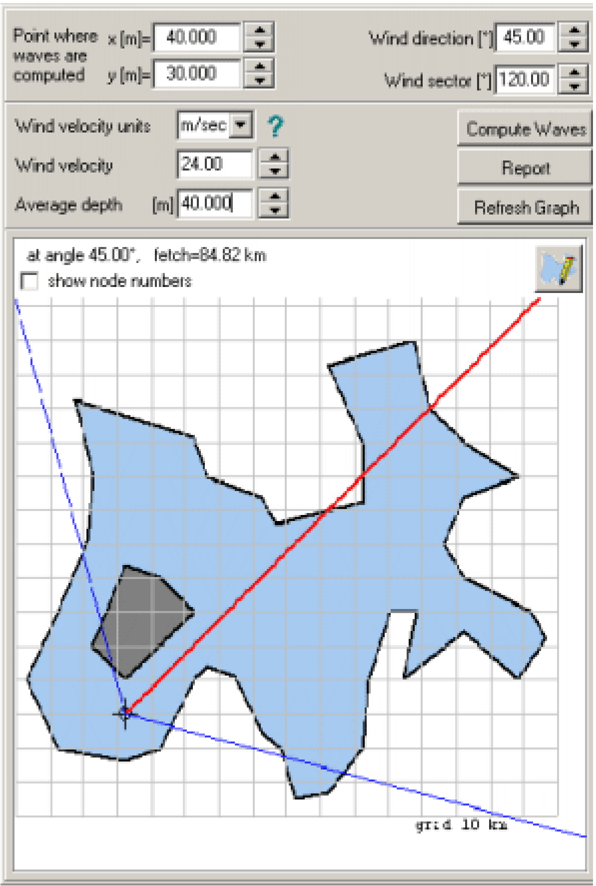
Background information on an Example of wind generated wave software (cgWindWave; website http://www.runet-software.com/cgWindWaves.htm)

cgWindWaves Forecasting of wind generated waves

The wave forecasting methods are both empirical and theoretical. Many factors are involved in wave forecasting, especially in restricted fetch areas. The wave forecasting methods are based on semi-empirical relations (SMB methods, Sverdrup, Munk, and Bretschneider), which link the significant wave height Hs and significant wave period Ts to wind speed U, fetch length F, and water depth. The wave forecasting procedures is largely graphical, and laborious.

The program **cgWindWaves** gives an estimate of the waves in restricted fetch water regions. From the water region defined by its map, the mean wind direction and wind velocity, you obtain the significant wave height Hs, and wave period Ts, and a wave spectrum S(f). The wave prediction is based on the combination of various theories for wave forecasting, for directional effects, and wave spectra, which are implemented in the program. Application of the program is for regions where refraction is negligible.

The program implements the directional wave effects using either Seymour's, Savil's or effective fetch method. The significant wave and period forecasting from the wind velocity and fetch is based on Bretschneider's or Wilson's method. For the wind energy Pierson-Moskowitz and JONSWAP spectra are used. All the data are in one main screen (water region, wind direction and velocity, selection of theories), and the forecasted wave characteristics, spectrum and time series sample, are shown immediately.



After specifying the water region, which is defined by its map, the mean wind direction and wind velocity, you can obtain the significant wave height, wave period and a wave spectrum.

The water region is defined by its outline specifying the coordinates of its nodes. More than one outline may be defined in order to describe regions with islands. The coordinates of the region nodes may be also read from a text file.

You can define the water region in three ways:

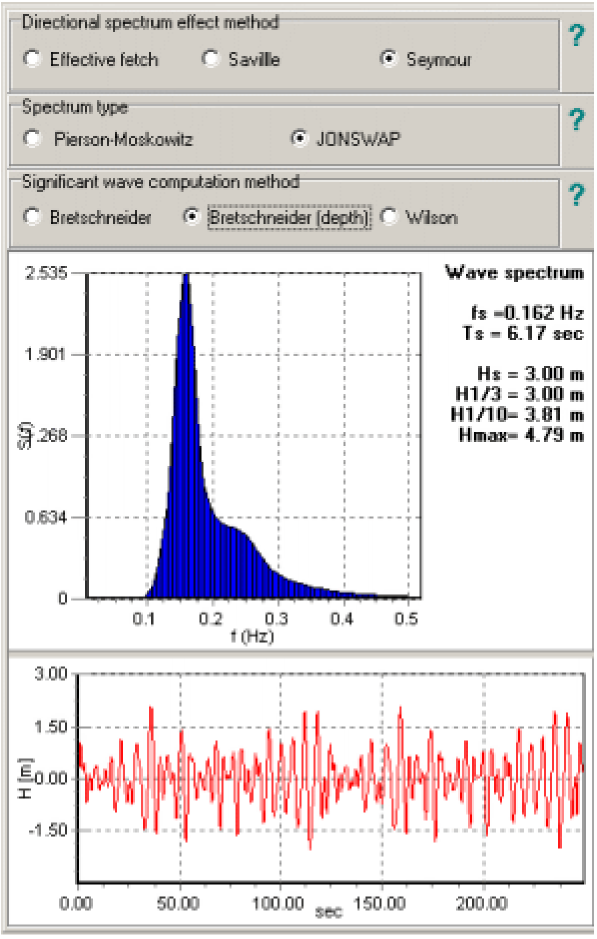
1) Graphically by clicking at the nodes, in the graphical package which is included in the program. The graphical pachage is simple and customized for the particular program.

2) By entering the X and Y nodal coordinates of the outline in a table.

3) By reading the nodal coordinates of the outlines from a text file.

Program units:

Metric (m, km, m/sec, km/h) or US Units (ft, miles, ft/sec, m.p.h)



User options:

- The directional spectrum effects

Effective fetch

Savil's method

Seymour's method

- The method of significant wave computation

Bretschneider method

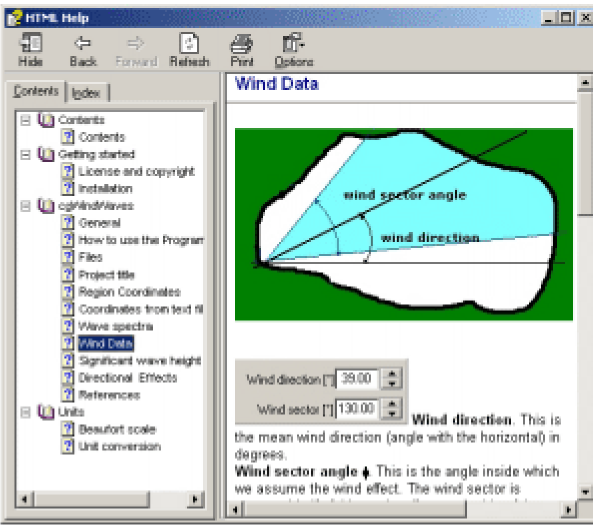
Wilson method

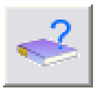
- The type of wave spectrum

Pierson-Moskowitz

JONSWAP

With a click of a mouse, you can change the wind direction or velocity. The resulting wave characteristics, wave spectrum and a sample wave record are shown simultaneously.

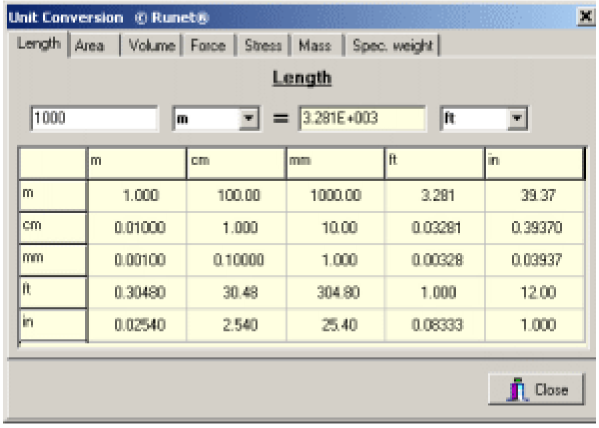




Complete Help

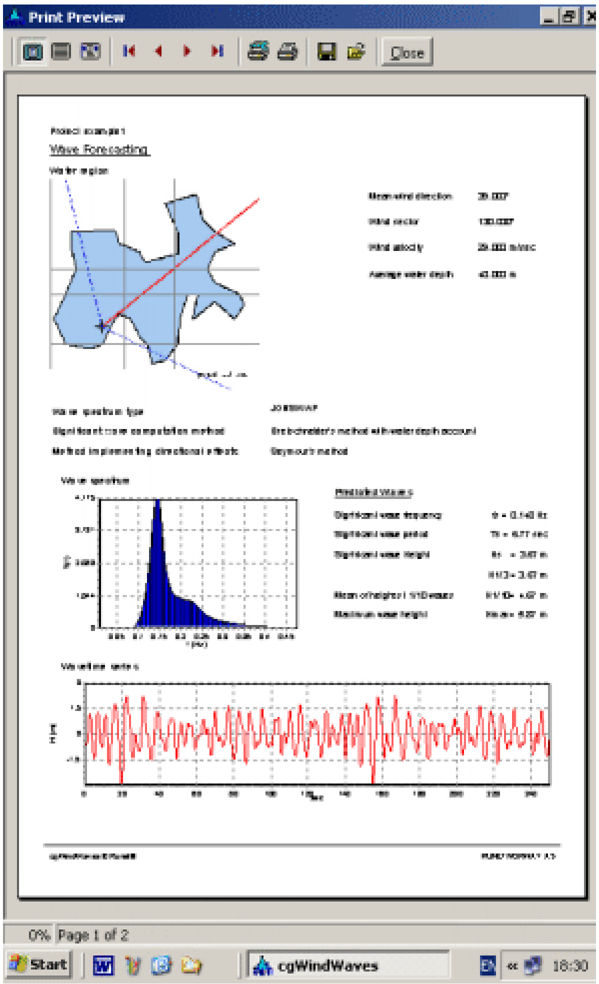
A context-sensitive Help system, guides you through the use of the program and the theoretical background.

On-line user's manual and frequently asked questions (F.A.Q.) are included in the program.



Engineering tools

Unit conversion



Reports

- Preview Reports
- Print reports
- Theoretical summary preview
- Theoretical summary print

Samples of report in pdf format

- Example 01
- Example 02
- Example 03

Short theoretical background

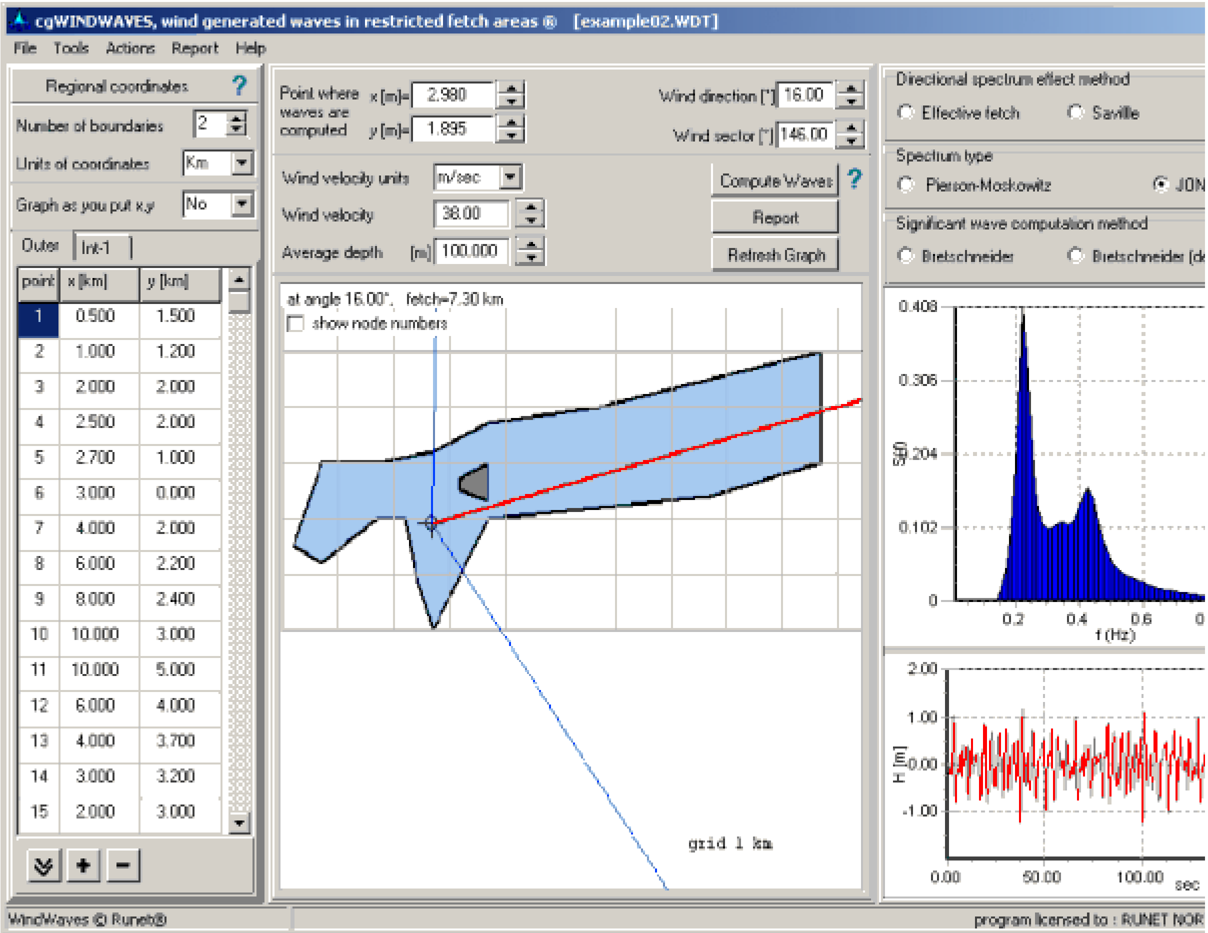
The wave forecasting methods are both empirical and theoretical. Many factors are involved in wave forecasting especially in restricted fetch areas where refraction is involved. The wave forecasting methods are based on semi-empirical relations (SMB methods, Sverdrup, Munk, and Bretschneider), which link the significant wave height and significant wave period to wind speed, fetch length and water depth.

In a water region defined by its map, the region of the water is divided in sectors radiating from the point where the wave characteristics are to be determined. These sectors are symmetrically disposed about a center line which is directed up-wind along the main wind direction. For each of these sectors the significant wave height (Hs) and wave period (Ts) are predicted, using one of three known methods: two methods by **Bretschneider**, one without taking into account the water depth and one taking into account the water depth, and one method by **Wilson**.

The wave forecasting of the water region is based on a wave spectrum which is obtained by taking into account the contribution of each of these sectors the water region is divided. Three methods are applied in the program for this contribution,**The effective fetch method, Saville's method and Seymour's method.**

For the wave spectra the two known spectra by **Pierson-Moskowitz** and **JONSWAP** are used.

Look at [Documentation] for the program for more detailed methodology.



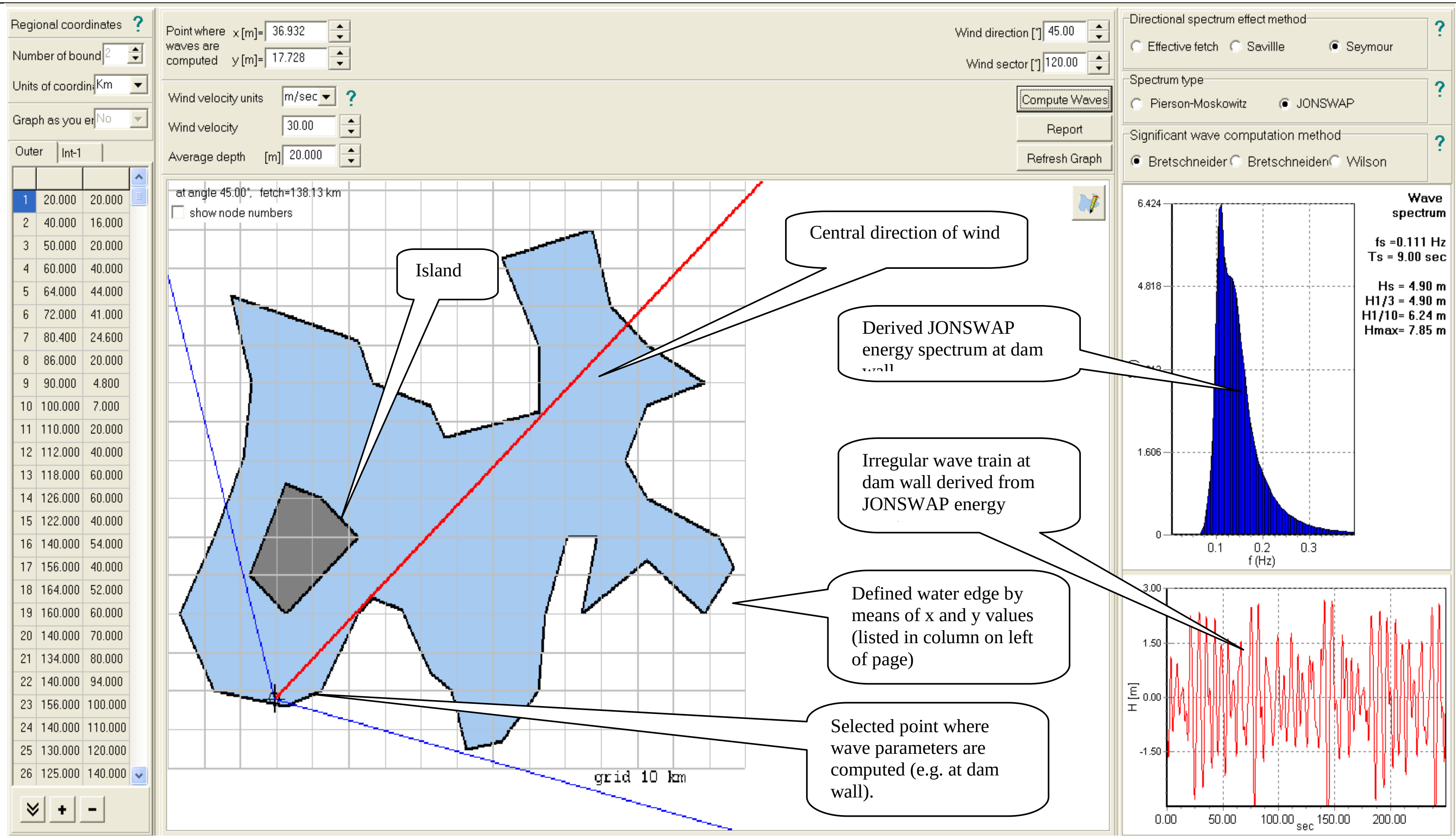


Figure B1: Example of input and output page of cgWindWaves software (water area of dam and wave energy spectrum (JONSWAP) with concurrent irregular wave train)

cgWindWaves, Methodology and references

Directional spectrum effects

In narrow fetch areas there is a dominant fetch for the wave prediction. In restricted fetch areas, there is not one dominant fetch to predict the waves. In this case three methods have been implemented in the program.

Effective fetch method

$$F_{eff} = \frac{\int_{-\alpha}^{+\alpha} F \cos(\phi)\, d\phi}{\int_{-\alpha}^{+\alpha} \cos(\phi)\, d\phi}$$

The effective fetch method is obtaining an equivalent fetch of the region by an weighted average process of the fetch of each sector, from this fetch the significant wave height and period are predicted, and from them the wave spectrum.

An effective fetch (Feff) obtained as a fetch weighted average, is used for the prediction of significant wave height and significant wave period. F is the fetch along a ray radiating from the point at which the waves is to be determined, and making an angle a with the main wind direction. The integral is over a wind sector area (usually from -p/2 to p/2)

Saville's Modified method

$$H_s = \frac{1}{2\alpha}\sqrt{\int_{-\alpha}^{+\alpha} [H(\phi)\cos(\phi)]^2\, d\phi}$$

$$T_s = \frac{\int_{-\alpha}^{+\alpha} T(\phi)\,[H(\phi)\cos(\phi)]^2\, d\phi}{\int_{-\alpha}^{+\alpha} [H(\phi)\cos(\phi)]^2\, d\phi}$$

Saville's method obtains an equivalent significant wave height and period, by a weighted average process of the predicted significant wave height and period for each sector. From the significant wave height and period the wave spectrum is predicted

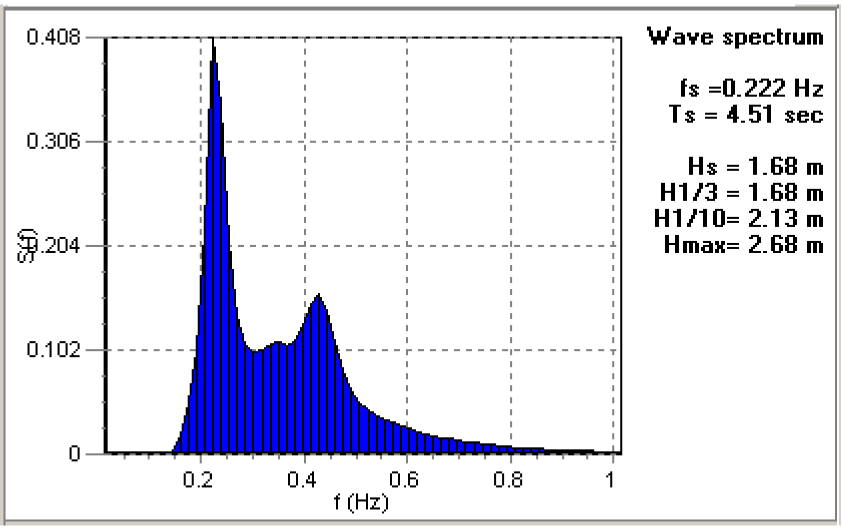
The significant wave height Hs obtained as weighted average, and the significant wave period as the energy weighted average. H(a) and T(a) are the wave height and wave period of waves generated along a ray radiating from the point at which the waves are determined, and making an angle a with the main wind direction. The integral is over a wind sector area (usually from -p/2 to p/2).

Seymour's method

According to this the wave spectrum is computed for each sector and the final predicted wave spectrum is obtained by adding up the energy of each sector's spectrum.

The water region is separated in small sectors at angles a around the main wind direction up to a boundary angle (usually from -p/2 to p/2). For each sector the significant wave height and significant wave period are computed, and from them a wave spectrum. The final wave spectrum is obtained from energy-average of all these spectrums, and from this spectrum the significant wave height and significant wave period are computed.

Significant wave height and period



The mechanics wave generation by winds acting over water surface is so complex factor that various semi-empirical methods have been developed. The wave forecasting methods are based on semi-empirical relations (SMB methods Sverdrup, Munk, and Bretschneider), which link the significant wave height Hs and significant wave period to wind speed, fetch, and water depth.

In the program are implemented three basic and commonly used methods.

In this methods F is the fetch, U the wind velocity, and g the acceleration of gravity, and D is the average depth f the region. the nondimensional terms for significant wave height H and significant wave period T are given as

Bretschneider's method

$$\frac{gH}{U^2} = 0.283 \tanh\left\{0.0125\left[\frac{gF}{U^2}\right]^{0.42}\right\} \qquad \frac{gT}{U} = 7.540 \tanh\left\{0.0770\left[\frac{gF}{U^2}\right]^{0.25}\right\}$$

[Seymour, R.J. "Estimating Wave Generation in Restricted Fetches", J. ASME WW2, May 1977, pp251-263.]

Bretschneider's method with depth effect

$$\frac{gH}{U^2} = 0.283 \tanh\left\{0.530\left[\frac{gD}{U^2}\right]^{0.75}\right\} \tanh\left\{\frac{0.0125\left[\frac{gF}{U^2}\right]^{0.42}}{\tanh\left\{0.530\left[\frac{gD}{U^2}\right]^{0.75}\right\}}\right\}$$

$$\frac{gT}{U} = 7.540 \tanh\left\{0.833\left[\frac{gD}{U^2}\right]^{0.375}\right\} \tanh\left\{\frac{0.0770\left[\frac{gF}{U^2}\right]^{0.25}}{\tanh\left\{0.833\left[\frac{gD}{U^2}\right]^{0.375}\right\}}\right\}$$

[Seymour, R.J. "Estimating Wave Generation in Restricted Fetches", J. ASME WW2, May 1977, pp251-263.]

Wilson's method

$$\frac{gH}{U^2} = 0.30\left[1 - \left\{1 + 0.004\left[\frac{gF}{U^2}\right]^{1/2}\right\}^{-2}\right] \qquad \frac{gT}{U} = 8.60\left[1 - \left\{1 + 0.008\left[\frac{gF}{U^2}\right]^{1/3}\right\}^{-5}\right]$$

[Bretschneider, Ch.I.."Topics in Ocean Engineering, Volume 1", p31-32, Gulf Publishing Company, Houston Texas, 1969.]

Wave spectra

Two commonly used wave spectral formulas are implemented in the program

Pierson-Moskowitz

$$S(f) = \alpha g^2 (2\pi)^{-4} f^{-5} \exp\left\{-\frac{5}{4}\left(\frac{f}{f_p}\right)^{-4}\right\}$$

JONSWAP

$$S(f) = \alpha g^2 (2\pi)^{-4} f^{-5} \exp\left\{-\frac{5}{4}\left(\frac{f}{f_p}\right)^{-4}\right\} \gamma^{\exp\left\{\frac{-(f-f_p)^2}{2\sigma^2 f_m^2}\right\}}$$

$\sigma 1=0.07$ for $f<=fp$, $\sigma 2=0.09$ for $f>fp$

References

1. Bretschneider, C. I.."Topics in Ocean Engineering, Volume 1", Gulf Publishing Company, Houston Texas, 1969.
2. Hallam M.G., Heaf N.J, Wootton, L.R., Dynamics of Marine Structures, Ciria Underwater Engineering Group, London 1977.
3. Kinsman B., " Wind Waves." Prentice Hall, Inc., Englewood Cliffs, New Jersey 1965.
4. Muga B., J., and Wilson J. F. ," Dynamic Analysis of Ocean Structures. " Plenum Press, New York, 1970
5. Newman, J., N. " Marine Hydrodynamics", MIT Press, Cambridge, Massachusetts, 1977.
6. Seymour, R.J. "Estimating Wave Generation in Restricted fetches", J. ASME WW2, May 1977, pp251-263.
7. World Meteorological Organization, "Handbook on Wave Analysis and Forecasting", WMO No 446, Geneva Switzerland 1976

APPENDIX C
Basic SWAN programming explained

C.1 GENERAL

Note that the programming used and explained in this appendix is only sufficient if you plan on using it to estimate wave heights in dams. It is however a good introduction to SWAN programming and should give you a basic feel for the programming. SWAN programming used to estimate wave heights on the Voëlvlei dam is used as an example in this appendix. See SWANProgramming.xls in the SWAN folder on the CD supplied with this written report.

C.2 PROGRAMMING

```
$************HEADING*****************************************
PROJ 'Voëlvlei' '1'


COORD CART

$**********MODEL INPUT*************************************
$
CGRID REG 2200.0 -3697040.0 0.0 3840.0 7720.0 96 193 CIR 36 0.033 1.0 40

$
INPGRID BOTTOM REG 2200.0 -3696920.0 0.0 96 193 40 40
READINP BOTTOM 1. 'Grid1.dat'  4 0 FREE
WIND 30.5 122

NUM ACCUR 0.02 0.02 0.0  90. 20
$

$
$*********** OUTPUT REQUESTS *************************

GROUP 'Table1' 0 96 0 193
$OUT
TABLE 'Table1' HEAD 'Output.dat' XP YP DEP HS PDIR


COMPUTE
STOP
```

C.3 EXPLAINED

Heading

`$`: Comment, SWAN does not use these lines

`PROJ 'Voëlvlei' '1'`: Project name and no.

`COORD CART:` Sets the coordinate system used (SWAN input and output) to Cartesian. The direction is the angle between the vector and the positive x-axis, measured counter-clockwise (the direction where the waves are going to or where the wind is blowing to).

Model input

```
CGRID REG 2200.0 -3697000.0 0.0 3800.0 7900.0 50 50 CIR 36 0.033 1.0 40
CGRID REG [xpc] [ypc] [alpc] [xlenc] [ylenc] [mxc] [myc] CIR [mdc] [flow]
[fhigh] [msc]
```

`CGRID`: Computational grid. This is the grid that SWAN will use for computations. See Figure D.1.

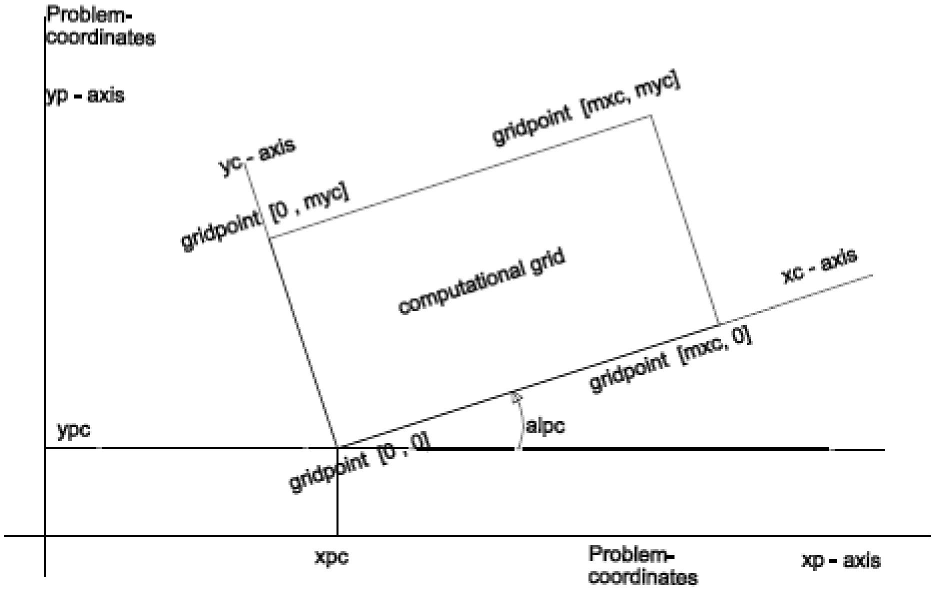


Figure C.1: Regular computational grid (SWAN User manual, 2004, p. 50)

`REG:` Regular grid: square or rectangular grid

`[xpc] [ypc]:` x and y origins

`[alpc]:` Angle of the x-axis of the computational grid. See Figure D.1.

`[xlenc] [ylenc]:` length of the grid in the x and y direction.

`[mxc] [myc]:` Number of meshes in the x and y directions. Maximum recommended meshes = 200.
(Number of Meshes = number of data points – 1)

`CIR:` indicates that the spectral directions cover the full circle.

`[mdc]:` this is the number of subdivisions of the 360 degrees of a circle, so $\Delta\theta$ = [360˚]/[mdc] is the spectral directional resolution.

`[flow]:` lowest discrete frequency that is used in the calculation (in Hz).
Typical field conditions: 0.04 Hz

`[fhigh]:` highest discrete frequency that is used in the calculation (in Hz).
Typical field conditions: 1 Hz

`[msc]:` one less than the number of frequencies. This defines the grid resolution in frequency-space between the lowest discrete frequency [flow] and the highest discrete frequency [fhigh]. The minimum number of frequencies is 4.

```
INPGRID BOTTOM REG 2200.0 -3697000.0 0.0 96 193 40 40
```

`INPGRID BOTTOM REG [xlenc] [ylenc] [alpc] [dxinp] [dyinp] [mxc] [myc]`

`INPGRID`: Input grid.

`BOTTOM`: defines the input grid of the bottom level

`[dxinp] [dyinp]`: Size of a mesh in the x and y direction.

`READINP BOTTOM 1. 'Grid1.dat'  4 0 FREE`

`READINP: reads input`

`1.`: indicates how bottom levels are to be read from file(bottom level positive downward relative to horizontal datum level)

`'Grid1.dat'`: The name of the grid file you created with Surfer (Appendix B: B.2) containing only the Z values.

`4`: prescribes the order in which the values of bottom levels should be given in the file. SWAN starts reading in the lower-left-hand corner.

`0`: number of header lines in your grid file SWAN should not read

`FREE`: Indicates that the values are to be read with free format.

`WIND 36.6 112`

`WIND [vel] [dir]`

`WIND`: Inputs wind

`[vel]`: Wind velocity in m/s

`[dir]`: Direction where wind is blowing to in degrees

`NUM ACCUR 0.02 0.02 0.0 90. 20`

`NUM ACCUR [drel] [dhova] [dtoval] [npnts] [limiter]`

`NUM ACCUR`: Number accuracy

`[drel]`: The change in the local significant wave height from one iteration to the next is less than fraction [drel] of that height

`[dhova]`: The change in the local significant wave height (Hs) from one iteration to the next is less than fraction [dhoval] of the average significant wave height (average over all wet grid points).

`[dtoval]`: The change in the local mean wave period from one iteration to the next is less than fraction [dtoval] of the average mean wave period (average over all wet grid points).

`[npnts]`: Above conditions are fulfilled in more than fraction [npnts]% of all wet grid points.

`[limiter]`: Determines the maximum change per iteration of the energy density per spectral.

Output requests

`GROUP 'Table1' 0 38 0 79`

`GROUP 'name' [x1] [x2] [y1] [y2]`

`GROUP`: define a set of output locations on a regular grid

`‘Table1’`: name

`[x1] [x2] [y1] [y2]: x and y start and end locations in table`

`TABLE 'Table1' HEAD 'Output.dat' XP YP DEP HS PDIR`

`TABLE`: Indicates that for each location of the output location set 'sname' one or more variables should be written to a file.

`HEAD`: Gives headers with column description to columns.

`'Output.dat'`: Determines the destination of the data.

`XP YP DEP HS PDIR`: Requested output.

`XP: x-coordinate`

`YP: y-coordinate`

`DEP: depth`

`HS: significant wave height`

`PDIR: peak direction`

`COMPUTE`: orders SWAN to start the computation(s).

`STOP`: Last command of input file. All information after this command will be ignored