Wave Overtopping of Seawalls

Design and Assessment Manual

HR Wallingford Ltd

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Statement of use
This manual is the final output from Project W5/006. The manual will be of use to flood and coastal defence engineers responsible for the design of new seawalls or the assessment of existing structures. Recipients of this report are to pass on any comments to Keith Slaney, the Environment Agency's Project Manager for R&D Project W5/006.

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GLOSSARY

Armour	Protective layer of rock or concrete units
Composite sloped seawall	A sloped seawall whose gradient changes
Composite vertical wall	A structure made up of two component parts, usually a caisson type structure constructed on a rubble mound foundation
Crown wall	A concrete super-structure located at the crest of a sloping seawall
Deep water	Water so deep that that waves are little affected by the seabed. Generally, water deeper than one half the surface wavelength is considered to be deep
Depth limited waves	Breaking waves whose height is limited by the water depth
Crest Freeboard	The height of the crest above still water level
Impacting waves	Waves that tend to break onto the seawall
Mean overtopping discharge	The average flow rate passing over the seawall
Mean wave period	The average of the wave periods in a random sea state
Normal wave attack	Waves that strike the structure normally to its face
Oblique wave attack	Waves that strike the structure at an angle
Peak overtopping discharge	The largest volume of water passing over the structure in a single wave
Reflecting waves	Waves that hit the structure and are reflected seaward with little or no breaking
Return period	The average length of time between sea states of a given severity
Run-up	The rush of water up a structure or beach as a result of wave action.
Shallow Water	Water of such a depth that surface waves are noticeably affected by bottom topography. Customarily water of depth less than half the surface wavelength is considered to be shallow

GLOSSARY continued

Significant wave height	The average height of the highest of one third of the waves in a given sea state
Toe	The relatively small mound usually constructed of rock armour to support or key-in armour layer
Tolerable overtopping discharge	The amount of water passing over a structure that is considered safe.
Wave return wall	A wall located at the crest of a seawall, which is designed to throw back the waves
Wave steepness	The ratio of the height of the waves to the wave length

NOTATION

a	Weibull scale parameter
b	Weibull shape parameter
A,B	empirical coefficients (see Table 1)
A_{c*}	dimensionless crest freeboard = $A_c/T_m(gH_s)^{0.5}$
C	empirical coefficient (see Table 5)
A_c	freeboard of the top of the slope (or the base of the return wall)
A_f	crest freeboard adjustment factor
C_r	permeable crest berm reduction factor
C_w	crest berm width
d	water depth over mound
d_*	mound parameter (composite structures) = $(d/H_s)(2\pi h/(gT_m^2))$
D_f	return wall discharge factor
g	acceleration due to gravity
h	water depth at toe of structure
h_*	wave breaking parameter = $(h/H_s)(2\pi h/(gT_m^2))$
H_s	significant wave height at the toe of the seawall
H_{so}	significant offshore wave height
N_{ow}	number of waves overtopping
N_w	number of waves in the sequence
O_r	overtopping ratio (ratio of mean discharge at angled wave attack to that at normal wave attack)
Q	mean overtopping discharge rate per metre run of seawall
Q_*	dimensionless mean discharge (sloped seawalls) = $Q / (T_m g H_s)$
Q_b	mean discharge per metre run of seawall at crest of structure or reaching base of the return wall (impermeable revetment)
Q_{b*}	dimensionless base discharge at crest of structure
Q_c	mean discharge per metre run of seawall reaching base of the return wall (permeable revetment)
Q_{c*}	dimensionless discharge reaching base of wave return wall (permeable revetment).
Q#	dimensionless mean discharge (vertical walls, reflecting waves) = $Q/(gH_s^3)^{0.5}$
Q_h	dimensionless mean discharge (vertical walls, impacting waves) = $\{Q/(gh^3)^{0.5}\} / h_*^2$
Q_d	dimensionless mean discharge (composite structures) = $\{Q/(gd^3)^{0.5}\} / d_*^2$
r	roughness coefficient
R_d	dimensionless crest freeboard (composite structures) = $(R_c/H_s)d_*$
R_h	dimensionless crest freeboard (vertical walls, impacting waves) = $(R_c/H_s)h_*$
R_*	dimensionless crest freeboard = $R_c/(T_m(gH_s)^{0.5})$
R_c	freeboard of the seawall (the height of the crest of the wall above still water level)
S_{op}	offshore sea steepness
T_m	the mean wave period at the toe of the seawall
T_{po}	peak offshore wave period
V_{bar}	mean individual overtopping volume
V_{max}	maximum individual overtopping volume
W_*	dimensionless return wall height = W_h/A_c
W_h	return wall height
X_*	adjusted dimensionless crest freeboard
β	angle of wave attack
γ	reduction factor for angle of attack

EXECUTIVE SUMMARY

Man-made defences, predominantly in the form of seawalls protect approximately 860 kilometres (23%) of the English coast. These defences range from simple earth embankments through vertical concrete walls and onto more complex composite structures often involving wave return walls and/or rock armouring. Regardless of structural type, the purpose of a seawall is usually to prevent erosion of the coastline and to limit the risk of marine inundation of the hinterland.

Over the past twenty years much research has been undertaken in the UK and elsewhere into the overtopping performance of seawalls and related structures. In the UK the Ministry of Agriculture, Fisheries and Food under Commission FD02 has funded most of this research effort.

The research has concentrated on providing techniques for predicting the mean overtopping discharge, and hence consequent flood volumes and drainage requirements, for a range of commonly occurring seawall types. Little, or no attention, has been paid to either the peak individual (wave-by-wave) overtopping discharge or to the number of waves likely to overtop a particular defence, despite the importance of these two parameters in determining the overall standard of performance of a sea defence.

It is now recognised that :-

- peak overtopping discharges represent the most hazardous events for pedestrians and vehicles moving behind the wall, and that for certain seawall designs the peak individual discharge may also be the event initiating damage to, or failure of, the defence
- the number of waves overtopping a seawall is the most easily recorded indicator of overtopping performance, being amenable to both visual observation and capture on video cameras etc. As such, if suitable relationships can be established, the number of overtopping waves provides the most obvious means of calibrating prediction techniques defined in terms of either mean or peak individual discharge.

This manual draws together and summarises previous research into the overtopping performance of seawalls. Data is re-analysed to provide a set of consistent design techniques, representing the most reliable approach to the assessment of seawall overtopping by wave action. The manual consists of five principal sections. Following introductory sections 1 and 2, the third deals with the estimation of mean overtopping discharges. The fourth deals with the estimation of the number of overtopping waves and the peak individual overtopping discharges. The fifth section then examines the concept of a tolerable overtopping discharge and its potential application in design practice.

The manual is intended to be used by flood and coastal defence engineers responsible for the design of new seawalls or the assessment and possible remediation of existing structures.

KEY WORDS

Flood control works, coastal structures, breakwaters, vertical seawalls, sloping seawalls, mean and peak overtopping discharges, tolerable discharge, normal and oblique wave attack.

1. INTRODUCTION

1.1 Background

Over the last ten years the Ministry of Agriculture, Fisheries and Food (MAFF) under commission FD02 has funded a long-term research programme into methods to predict overtopping discharges over a range of different types of sea defences. The performance of many structures such as embankments and vertical walls have been analysed for a range of conditions, under both normal and oblique wave attack. In addition, complimentary work has been carried out in Holland and Italy, for dyke and composite caisson structures. This manual presents, in a concise form, the design methods derived from the wider programme of research. A full description of the work which went into deriving and assessing these methods can be found in the accompanying project record, Overtopping of Seawalls, Project Record W5/006/5.

This manual brings together the results of the work carried out to date and recommends approaches for calculating mean and peak overtopping discharges and the number of waves overtopping the seawall crest. The manual will help engineers to establish limiting tolerable discharges for design wave conditions, and then use the prediction methods to confirm these discharges are not exceeded.

1.2 Seawalls

Historically, seawalls have been the most widely used option for coastal defence. They have been built along the coastlines to protect the land from erosion and flooding and sometimes provide additional amenity value. Typically structures are either massive vertical retaining walls or sloping revetments.

Some coastal structures are relatively impermeable to wave action. These include seawalls and breakwaters formed from blockwork or mass concrete, they may have vertical, near vertical, or sloping faces. Such structures are liable to experience intense local wave impact pressures, may overtop severely, and will reflect much of the incident wave energy. Reflected waves cause additional wave disturbance and/or may initiate or accelerate local bed scour.

A second type of coastal structure consists of a mound of quarried rock fill, protected by layers of rock or concrete armour units. The outer armour layer is designed to resist wave action without significant displacement of armour units. Under-layers of quarry or crushed rock support the armour and separate it from the fine material in the embankment or mound. These porous and sloping layers dissipate a significant proportion of the incident wave energy in breaking and friction. Simplified forms of rubble mounds may be used for rubble seawalls or protection to vertical walls or revetments.

1.3 Overtopping discharge

Overtopping discharge occurs as a result of waves running up the face of the seawall. This manual does not directly examine wave run-up but concentrates on the resulting discharge rates. A full description of wave run-up can be found in Simm (1991).

If wave run-up levels are high enough water will reach and pass over the crest of the wall. This defines the 'green water' overtopping case where a continuous sheet of water passes over the crest.

A second form of overtopping occurs when waves break on the seaward face of the structure and produce significant volumes of splash. These droplets may then be carried over the wall either under their own momentum or as a consequence of an onshore wind.

Another less important method by which water may be carried over the crest is in the form of spray generated by the action of wind on the wave crests immediately offshore of the wall. Without the influence of a strong onshore wind however this spray will not contribute to any significant overtopping volume.

Overtopping rates predicted by the various empirical formulae described within this report will include green water discharges and splash, since both these parameters were recorded during the model tests on which the prediction methods are based. The effect of wind on this type of discharge will not have been modelled. Model tests suggest that onshore winds have little effect on large green water events, however they may increase discharges under 1 l/s/m. Under these conditions the water overtopping the structure is mainly spray and therefore the wind is strong enough to blow water droplets inshore.

1.4 Use of this manual

The design methods described in the latter sections of this manual are based upon a deterministic philosophy in which overtopping discharges are calculated for wave and water level conditions representing a given return period. All of the design equations require the wave conditions at the toe of the seawall. If appropriate, these input conditions should take account of wave breaking. Methods of calculating depth limited wave conditions are outlined by Owen (1980) and Simm (1991).

The input water level should include a tidal and, if appropriate, a surge component. Surges are usually comprised of a number of components, including contributions due to wind set-up and barometric pressure.

All of the prediction methods given in this report have intrinsic limitations to their accuracy. The physical model data from which these design equations have been derived generally exhibit significant scatter. A study by Douglass (1985) concluded that calculated overtopping rates, using empirically derived equations, should only be regarded as being within, at best, a factor of 3 of the actual overtopping rate. It is generally reasonable to assume that the overtopping rates calculated using the methods contained in this report are accurate only to within one order of magnitude. Overtopping rates are very sensitive to small variations in seawall geometry, local bathymetry and wave climate. The methods presented here are generally based upon the results of model tests conducted on models intended to represent generic structural types, such as vertical walls, armoured slopes etc. The inevitable differences between these structures and site-specific designs may lead to large differences in overtopping performance. The methods presented here will not predict overtopping performance with the same degree of accuracy as structure-specific model tests.

The manual has been kept deliberately concise in order to maintain clarity and brevity. For the interested reader a full set of references is given so that the reasoning behind the development of the recommended methods can be followed.

2. WATER LEVELS AND WAVE CONDITIONS

2.1 Water levels, tides and surges

2.1.1 General

Water levels consist of two components: an astronomical or predicted tide caused by planetary motions and a surge, which is related to local weather conditions. Surges may either be positive, causing an increase in water level, or negative where water levels are reduced.

Tidal elevations are well documented, as the motions of the planets are repeatable. However, surge levels are not repeatable and must therefore be treated on a probabilistic basis.

The easiest means of predicting extreme water levels is to analyse long term water level data from the site in question. However, where no such data exists, it may be necessary to predict surge levels using theoretical equations and combine these levels with tidal elevations in order to obtain an estimation of extreme water levels.

2.1.2 Extreme water levels

Extreme high water levels are caused by a combination of high tidal elevations plus a positive surge. At the present time there are two publications that are used to estimate extreme water levels.

<u>Graff (1980)</u>
Graff (1980) analysed sea level maxima at 67 ports distributed around the UK. The data are analysed by a standard method of extreme analysis. The number of maxima in the data series ranged from a minimum of 10 to a maximum of 133, throughout the period 1813 – 1978. The data can be used to estimate extreme sea levels for return periods up to 1:250 years.

<u>P.O.L (1995)</u>
Recently P.O.L (1995) with funding from MAFF have published a report for extreme sea levels for the east coast. Data from 41 sites are considered in this stage of the report, and results for another 19 sites were determined. Furthermore, by use of a spatial model, estimates of extreme still water levels are given for sites at regular intervals along the UK east coast for which data are non existent.

This P.O.L publication only covers the eastern coast of Britain. However, at the time of printing another report by P.O.L. (1997) exists which covers the whole of the UK. MAFF will recommend the use of this report as it supersedes the two previous data sets.

2.1.3 Astronomical tides

Around the UK coast, and indeed around much of the world, the largest fluctuations in water level are caused by "astronomical" tides. These are caused by the relative rotation of both the sun and the moon around the earth each day. The differential gravitational effects over the surface of the oceans cause tides with well defined periods, principally semi-diurnal and diurnal. Around the British Isles the semi-diurnal tides are much larger than the diurnal components.

In addition to the tides that result from the earth's rotation, other periodicities are apparent in the fluctuation of tidal levels. The most obvious is the fortnightly spring-neap cycle, corresponding to the half period of the lunar cycle.

Further details on the generation of astronomic tides, and their dynamics, can be found in the Admiralty Manual of Tides. These give daily predictions of times of high and low waters at selected Standard Ports. Also listed are data for Secondary Ports and details of calculating the differences in level between them and the nearest Standard Port are provided. Unfortunately, in practice, the prediction of an extreme water level is made much more complicated by the effects of weather, as discussed below.

2.1.4 Surges

Generally speaking the difference between the level of highest astronomical tide and, say, the largest predicted tide in any year is rather small (i.e. a few centimetres). In practice, this difference is unimportant at least in the UK, when compared with the differences between predicted and observed tidal levels due to weather effects.

Extreme high water levels are caused by a combination of high tidal elevations plus a positive surge. Positive surges comprise three main components:
- a barometric effect caused by a variation in atmospheric pressure from its mean value.
- wind set-up. In shallow seas, such as a the English Channel or the North Sea, a strong wind can cause a noticeable rise in sea level within a few hours
- a dynamic effect due to the amplification of surge-induced motions caused by the shape of the land (e.g. seiching and funnelling).

A fourth component, wave set-up causes an increase in water levels within the surf zone at a particular site due to waves breaking as they travel shoreward. Unlike the other three positive surge components, wave set-up has only an extremely localised effect on water levels. Wave set-up is implicitly reproduced in the physical model tests on which the overtopping equations are based. There is therefore no requirement to add on an additional water level increase for wave set-up when calculating overtopping discharges using the methods reported in this document.

Negative surges are made up of two principal components: a barometric effect caused by high atmospheric pressures and wind set-down caused by winds blowing offshore. Large positive surges are more frequent than large negative ones. This is because a depression causing a positive surge will tend to be more intense and associated with a more severe wind condition than anticyclones.

The east and south coasts of the UK are most likely to be affected by surges. Although high water levels similar to those which occurred in January 1953 are rare, it is normal for surges of about a metre to occur several times a year along these stretches of coast. It is thus necessary to make considerable allowance for surges in the design of any structure intended to last for more than a few months.

Where long term water level data is not available, surge components can be estimated from information given in Simm (1991).

2.1.5 Future sea level rise

MAFF (1993) prescribe allowances to be made for relative sea level rise according to the Environment Agency Region in which the scheme is located. The allowances, tabulated below, should still be used for planning and design unless there is a scientifically strong and sound reason for them to be reviewed for a particular scheme. These allowances are embodied within the Project Appraisal Guidance Notes (PAGN) and are subject to periodic review.

Rates of relative sea level rise – MAFF (1993)

EA Region	Allowance
Anglian, Thames, Southern	6mm/year
North West, Northumbria	4mm/year
Remainder	5mm/year

2.2 Wave conditions

2.2.1 General

In defining the wave climate at the site, the ideal situation is to collect long term instrumentally measured data at the required location. There are very few instances in the UK in which this is even a remote possibility. It is however more likely that data in deep water, offshore of a site will be available either through the use of a computational wave prediction model based on wind data, or more recently from the UK Meteorological Office wave model. In both of these cases the offshore data can be used in conjunction with a wave transformation model to provide information on wave climate at a coastal site. If instrumentally measured data is also available, covering a short period of time, this can be used for the calibration or verification of the wave transformation model, thus giving greater confidence in its use.

2.2.2 Wind-sea

Wind generated waves offshore of most coasts have wave periods in the range 1s to 10s. The height, period and direction of the waves generated will depend on the wind speed duration, direction and the 'fetch', i.e. the unobstructed distance of sea surface over which the wind has acted. In most situations, one of either the duration or fetch become relatively unimportant. For example, in an inland reservoir or lake, even a short storm will produce large wave heights. However, any increase in the duration of the wind will then cause no extra growth because of the small fetch lengths. Thus such waves are described as 'fetch limited". In contrast, on an open coast where the fetch is very large but the wind blows for only a short period, the waves are limited by the duration of the storm. Beyond a certain limit, the exact fetch length becomes unimportant. These waves are described as ‘duration limited'.

2.2.3 Swell

On oceanic shorelines, including some of the UK coastline (especially the south west coast), the situation is usually more complicated. Both the fetch and duration may be extremely large, waves then become "fully developed" and their height depends solely on the wind speed. In such situations the wave period usually becomes quite large, and long period waves are able to travel great distances without suffering serious diminution. The arrival of ‘swell’, defined as waves not generated by local and/or recent wind conditions, presents a more challenging situation from the viewpoint of wave predictions.

2.2.4 Wave breaking

Wave breaking remains one phenomenon that is difficult to describe mathematically. One reason for this is that the physics of the process is not yet completely understood. However, as breaking has a significant effect on the behaviour of waves, the transport of sediments, the magnitude of forces on coastal structures and the overtopping response it is represented in computational models. The most frequent method for doing this is to define an energy dissipation term which is used in the model when waves reach a limiting depth compared to their height

There are also two relatively simple empirical methods of estimating the incident wave conditions in the surf zone. The methods by Goda (1980) and Owen (1980) are regularly used.

Goda (1980)
Inshore wave conditions are influenced by shoaling and wave breaking. These processes are influenced by a number of parameters such as the sea steepness and the slope of the bathymetry. To take all the important parameters into account Goda (1980) provided a series of graphs to determine the largest and the significant wave heights (H_{max} and H_s) for 1:10, 1:20, 1:30 and 1:100 sloping bathymetries.

Owen (1980)
In this method, the wave height is represented by an equivalent post-breaking wave height H_{sb}. It should be noted that H_{sb} is not necessarily the wave height which would be obtained by direct measurement. It is an equivalent wave height designed to give the correct overtopping discharge as confirmed from physical model tests where significant wave breaking took place. Again the wave height is dependent on the slope of the bathymetry, the depth of water and the wave period at the structure.

2.3 Currents

Where waves are propagating towards an oncoming current, for example at the mouth of a river, the current will tend to increase the steepness of the waves by increasing their height and decreasing their wavelength. Refraction of the waves by the current will tend to focus the energy of the waves towards the river mouth. In reality both current and depth refraction are likely to take place producing a complex wave current field. It is clearly more complicated to include current and depth refraction effects, but at sites where currents are large they will have a significant influence on wave propagation. Computational models are available to allow both these effects to be represented.

2.4 Joint probability analysis

Extreme coastal engineering events, such as wave overtopping of a seawall or severe erosion of a beach, are often brought about by a combination of severe wave conditions and extreme water levels. Therefore, when engineers are designing coastal defences, be they seawalls, groynes or breakwaters, they should look at the likelihood of both conditions occurring simultaneously. A joint probability study of wave heights and water levels will do this and can be a powerful engineering tool in reducing the design conditions at particular sites where there is not a strong correlation between high waves and high water levels.

In the case of waves and water levels, the assumption of complete dependence would lead to a very conservative design since the one in hundred year event would have to comprise a one hundred year water level and a one hundred year wave height. Conversely, the assumption of independence would lead to under-design in some cases, since any increase in the probability of high wave heights at times of very high water levels would have been ignored. The correlation between waves and water levels will usually lie between these two extremes of complete dependence and complete independence. The precise degree of dependence is best determined from the analysis of actual data. Any correlation between wave heights and water levels is likely to be clear in a scatter diagram of wave heights against surges, as both are related to meteorological conditions. The inclusion of predicted astronomical tidal level (which is not related to weather conditions) in the water level data would tend to mask this correction.

Methods for assessing joint probability
The list below outlines four commonly used methods for addressing joint probability, in order of increasing sophistication:

- Calculate design wave height and assume a water level.
- In situations where wave heights are limited by the depth of water, a design water level can be assumed and a depth limited wave height calculated.
- Calculate extreme wave heights and water levels and assess correlation intuitively.
- Derive time series waves and water levels and analyse correlation and joint probability extreme rigorously.
- Any of the above with wave period dependent upon wave height.
- Any of the above as a function of wave direction.

Hawkes and Hague (1994) and Simm (1996) discuss these approaches in more detail.

2.5 Application of design conditions

The selection of a given return period for a particular site will depend on several factors. These will include the expected lifetime of the structure, expected maximum wave / water level conditions and the intended use of the structure. If for instance the public are to have access to the site then a higher standard of defence will be required than that to protect farm land.

It should be remembered that there will not be exactly T_r years between events with a given return period of T_r years. If the events are statistically independent then the probability that a condition with a return period of T_r years will occur within a period of L years is given by $p = 1 - (1\text{-}1/nT_r)^{nL}$, where n is the number of events per year, e.g., 2920 storms of three hours duration. Thus for an event with a return period of 100 years there is a 1% chance of recurrence in any one year. For a time interval equal to the return period, p is given by $1 - (1\text{-}1/nTr)^{nTr}$ or $p \sim 1\text{-}1/e = 0.63$. Therefore there is a 63% chance of occurrence within the return period. Further information on design events and return periods can be found in the British Standard Code of practice for Maritime Structures (BS6349 Part 1 1974 and Part 7 1991).

3. PREDICTING MEAN OVERTOPPING DISCHARGES

3.1 Introduction

This section presents methods of predicting mean overtopping discharges over a variety of seawall types. A knowledge of mean discharge is required when designing features such as drainage capacity of areas protected by a seawall. It is also of relevance when attempting to assess risk of damage to the crest and rear slope of the seawall itself. A method is presented for each of the major types of seawall in common use around the UK coastline.

3.2 Smooth impermeable slopes

A considerable number of studies have been undertaken into the overtopping performance of seawalls. The most comprehensive was that completed by Owen (1980) who investigated the performance of simply sloping and bermed seawalls shown in Figure 3.1 and Figure 3.3 respectively. The bermed structures used in this study all had berms located at or below still water level and had the same slope angle above and below the berm. Owen proposed a design method, which is widely used in the civil engineering industry, to calculate the mean discharge overtopping a simply sloping seawall. In this method the discharge and freeboard are non-dimensionalised as follows:-

$$Q_* = Q / (T_m g H_s)$$

$$R_* = R_c / (T_m (gH_s)^{0.5})$$

where Q is the mean overtopping discharge rate per metre run of seawall
T_m is the wave period at the toe of the wall
g is acceleration due to gravity
H_s is the significant wave height at the toe of the wall
R_c is the freeboard of the seawall (the height of the crest of the wall above still water level)

The dimensionless discharge, Q_*, and freeboard, R_*, are related by the following equation:-

$$Q_* = A \exp(-BR_*)$$

Where A and B are empirically derived coefficients which depend on the profile of the seawall.

Owen (1980) derived, or interpolated, values of A and B for simply sloped seawalls ranging in slope angle from 1:1 to 1:5, these are shown in Table 1. Owen (1980) also found that the equations used for simply sloping seawalls could be equally applied to bermed structures, albeit with modified empirical coefficients (Table 2). The original coefficients proposed by Owen (1980) have been revised as further data has become available.

Recently, van der Meer and de Waal (1992) proposed an alternative series of equations to estimate overtopping of simply sloping and bermed seawalls. The methods of both Owen and van der Meer and de Waal have their advantages and disadvantages. Recent work suggests that the van der Meer method generally under predicts overtopping discharges for wave periods greater than 10 seconds, while Owen errs on the conservative side. Owen has measured data for a number of different types of simply sloping structures. The data is therefore more structure-specific than the van der Meer method, which combines all the data together. The authors therefore recommend that the method proposed by Owen (1980) is used for estimating overtopping discharges at smooth, simply sloping and bermed seawalls around the UK coastline.

The methods discussed above were derived from tests conducted in 2-dimensional wave flumes and are thus applicable only to waves approaching normal to the structure. Several authors have investigated the effect of oblique angles of wave attack. Banyard and Herbert (1995) have reported on the behaviour of simply sloping and bermed seawalls. There was some indication that under a few conditions there was a slight increase in overtopping discharge at small angles of wave attack, however, overtopping was generally found to reduce with increased angle of attack. This reduction was smaller for short-crested seas than for long-crested seas. Equations developed by Banyard and Herbert (1995) enable an overtopping ratio, O_r, to be calculated. O_r is defined as the ratio of overtopping at a given angle of wave attack, β, to that predicted under normal wave attack. The following equations have been developed for short-crested seas, and can thus be applied conservatively to long-crested seas.

For simply sloping seawalls :-

$$O_r = 1 - 0.000152\beta^2$$

and for bermed seawalls :-

$$O_r = 1.99 - 1.93\ (1.0 - ((\beta - 60) / 69.8)^2)^{0.5}$$

The behaviour of the two types of seawall differed considerably, with the bermed structure exhibiting a greater reduction in overtopping, for a given wave angle, than the simply sloping seawalls. This difference in performance was particularly noticeable at small angles of wave attack. In both cases the predicted overtopping discharge is lower for all oblique angles of attack than for normal attack. The slope of the seawall was found to have little effect.

Box 3.1 outlines the design method for smooth, simply sloping, impermeable seawalls. The adaptation of the method for bermed seawalls is described in Box 3.2.

BOX 3.1
SMOOTH, IMPERMEABLE, SIMPLY SLOPED SEAWALLS
MEAN OVERTOPPING DISCHARGE

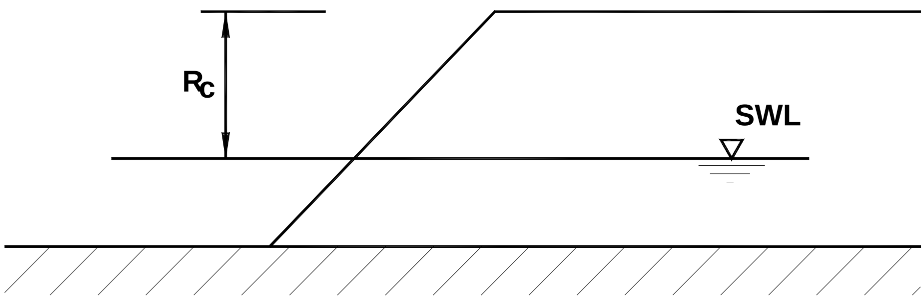


Figure 3.1 Smooth, impermeable, simply sloped seawall

BOX 3.1 (continued)

BOX 3.1 (continued)

NORMAL WAVE ATTACK

Use the following equations to calculate the mean discharge :-

$$R_* = R_c/(T_m(gH_s)^{0.5}) \qquad (1)$$

$$Q_* = A \exp(-BR_*) \qquad (2)$$

$$Q = Q_* T_m g H_s \qquad (3)$$

Where R_c is the freeboard (the height of the crest of the wall above still water level) (m)
H_s is the significant wave height at the toe of the seawall (m)
T_m is the mean wave period at the toe of the seawall (s)
g is acceleration due to gravity (m/s^2)
A,B are empirical coefficients dependent upon the cross-section of the seawall (see Table 1)
Q is the mean overtopping discharge rate per metre run of seawall ($m^3/s/m$)

Equation 1 is valid for $0.05 < R_* < 0.30$

The geometry of the structure and a knowledge of the wave and water level conditions enables the dimensionless freeboard, R_*, to be calculated using equation (1). The dimensionless mean discharge, Q_*, is then calculated using equation (2) along with the appropriate values of A and B from Table 1. Use of equation (3) then enables the mean overtopping discharge, Q, to be calculated.

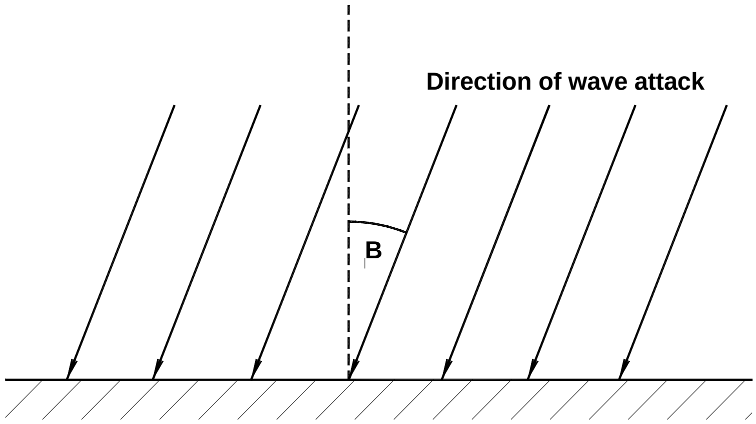


Figure 3.2 Angled wave attack

ANGLED WAVE ATTACK

The overtopping performance of a simply sloping seawall under angled wave attack from short-crested seas can be determined by firstly calculating the discharge assuming normal wave attack, i.e., using equations (1) - (3). For a given angle of wave attack the overtopping ratio, O_r, can then be calculated from the following equation:-

$$O_r = 1 - 0.000152\beta^2 \qquad (4)$$

where O_r is the ratio of discharge under angled wave attack to that under normal attack
β is the angle of wave attack to the normal, in degrees (see Figure 3.2).

Knowledge of the overtopping ratio and the overtopping under normal wave attack thus enables the discharge at a particular angle to be determined. Equation (4) may also be used to provide a conservative solution for long crested seas.

Equation (4) is valid for $0^o < \beta \leq 60^o$. For angles of approach greater than 60^o it is suggested that the result for $\beta = 60^o$ be applied.

BOX 3.2
SMOOTH, IMPERMEABLE, BERMED SEAWALLS
MEAN OVERTOPPING DISCHARGE

NORMAL WAVE ATTACK

For bermed slopes, where the berm is located at or below still water level (Figure 3.3), the method given in equations (1) to (3) should be employed, using the modified empirical coefficients given in Table 2. If the slope angles above and below the berm differ, then the empirical coefficients applicable to the upper slope angle should be used.

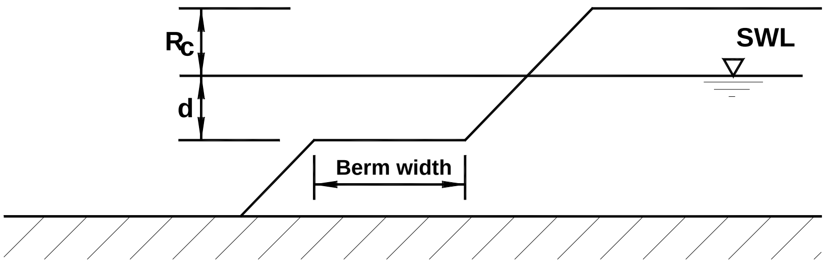


Figure 3.3 Bermed seawall, berm below SWL

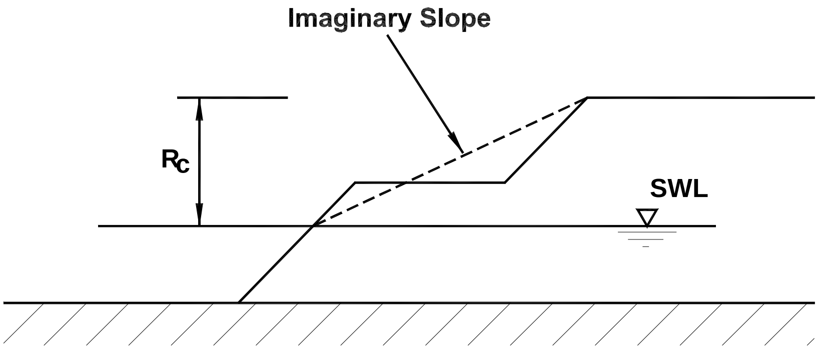


Figure 3.4 Bermed seawall, berm above SWL

For berms located above still water level (Figure 3.4) it is suggested that an imaginary simple slope be constructed between the still water level/seawall intersection point and the top of the seaward slope of the structure. Equations, (1) to (3), and coefficients (from Table 1) applicable to simple slopes may then be applied to this imaginary slope in order to estimate mean overtopping discharge rates.

ANGLED WAVE ATTACK

In order to determine overtopping due to angled wave attack in short-crested seas the discharge assuming normal wave attack should first be determined as described above. For a given wave angle, β, the overtopping ratio, O_r, can then be calculated from the following equation:-

$$O_r = 1.99 - 1.93\,(1.0 - (\,(\beta - 60) / 69.8\,)^2\,)^{0.5} \qquad (5)$$

where O_r is the ratio of discharge under angled wave attack to that under normal attack
β is the angle of wave attack to the normal, in degrees (see Figure 3.2)

Knowledge of the overtopping ratio and the overtopping under normal wave attack thus enables the discharge at a particular angle of wave attack to be determined. Equation (5) may also be used to provide a conservative solution for long-crested seas.

Equation (5) is valid for $0^o < \beta \leq 60^o$. For angles of attack greater than 60^o it is suggested that the result for $\beta = 60^o$ be applied.

There are many types of composite seawalls which are amalgams of slopes of different angles and which have two or more berms (see Figure 3.5). Data on these types of structures is rare, but limited information from site specific studies enables the advice contained in Box 3.3 to be given.

BOX 3.3
COMPOSITE SEAWALLS
MEAN OVERTOPPING DISCHARGE

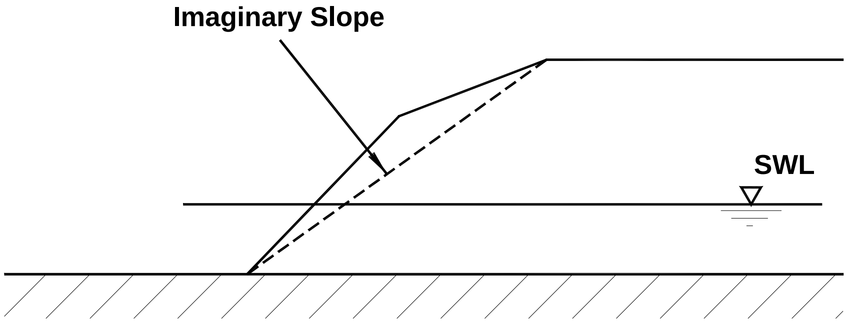


Figure 3.5 Composite sloped seawall

NORMAL AND ANGLED WAVE ATTACK
For composite slopes in relatively shallow water, $h < 0.55\ H_s$, it is suggested that an imaginary simple slope be constructed between the toe and crest of the structure. The methods described in Boxes 3.1 and 3.2 may then be applied to this imaginary slope.

3.3 Rough and armoured slopes

Owen (1980) extended his work on simply sloping and bermed seawalls to cover rough impermeable and rough permeable (i.e. armoured) structures, (Figure 3.6). Owen (1980) related the dimensionless parameters Q_* and R_*, as previously used to analyse smooth slope overtopping, by the following equation:-

$$Q_* = A \exp(-B\, R_*/r)$$

where A,B are the empirical coefficients applicable to a smooth slope
r is a roughness coefficient

Owen (1980) produced typical values of the roughness coefficient based upon the relative run-up performance of alternative types of construction (Table 3). These coefficients were originally derived for simple slopes but can also be conservatively applied to bermed slopes.

Armoured seawalls often include a crest berm that will dissipate significant wave energy and thus reduce overtopping. Owen’s equation does not take into account crest berms and hence discharges are over-predicted. For the purposes of this report a series of model tests were conducted to investigate the effect of the crest berm on the mean discharge. An equation was derived for rock armoured slopes which can be applied conservatively to other permeable structures.

Studies of the overtopping discharge performance of rough and armoured seawalls under angled wave attack are mainly limited to site-specific studies. Juhl and Sloth (1994) used long-crested seas to investigate the effect of wave angle on the overtopping performance of breakwaters. They noted that for small angles of wave attack a few tests exhibited overtopping ratios, O_r, greater than unity, although on average a reduction in overtopping was found. This was similar to the behaviour of smooth slopes noted by Banyard and Herbert (1995). Juhl and Sloth (1994) concluded that the overtopping ratio was dependent upon the freeboard, R_c, but derived no empirical equations to describe the overtopping performance.

BOX 3.4
ROUGH AND ARMOURED SEAWALLS
MEAN OVERTOPPING DISCHARGE

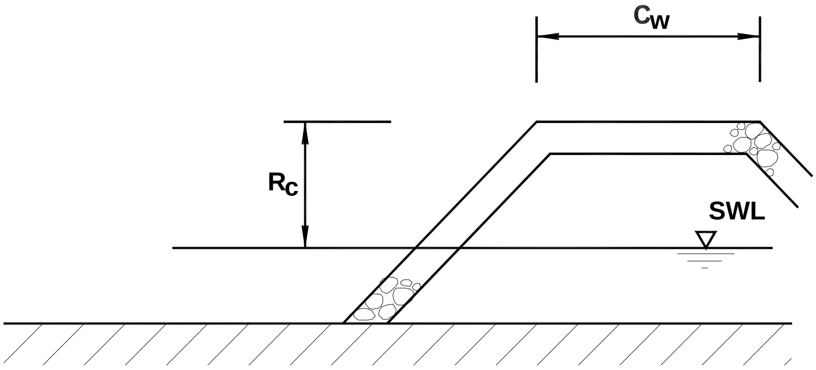


Figure 3.6 Armoured seawall

NORMAL WAVE ATTACK
Use the following equations to calculate the mean discharge :-

$$R_* = R_c/(T_m(gH_s)^{0.5}) \quad (6)$$
$$Q_* = A \exp(-BR_*/r) \quad (7)$$
$$Q = Q_* T_m g H_s \quad (8)$$

where R_c is the freeboard (the height of the crest of the wall above still water level) (m)
H_s is the significant wave height at the toe of the seawall (m)
T_m is the mean wave period at the toe of the seawall (s)
r is the roughness coefficient (see Table 3)
g is acceleration due to gravity (m/s^2)
A,B are empirical coefficients dependent upon the cross-section of the seawall (see Table 1)
Q is the mean overtopping discharge rate per metre run of seawall (m^3/s/m)

Equation 7 is valid for $0.05 < R_* < 0.30$

The geometry of the structure and a knowledge of the wave and water level conditions enables the dimensionless freeboard, R_*, to be calculated using equation (6). The dimensionless mean discharge, Q_*, is then calculated using equation (7) along with the appropriate values of A and B from Table 1 and the coefficient r from Table 3. If the structure includes berms then the procedure described in Box 3.2 for berms on smooth impermeable slopes should be followed, including the use of the modified coefficients (Table 2). Use of equation (8) then enables the mean overtopping discharge, Q, to be calculated. To take account of a permeable crest berm, a reduction factor, C_r, is determined as follows :-

$$C_r = 3.06 \exp(-1.5\, C_w/H_s) \quad (9)$$

where C_w is the crest berm width in metres. When $C_w/H_s < 0.75$ assume that $C_r = 1$.

The discharge should first be calculated using equations (6) to (8) assuming that there is no crest berm and then multiplied by the reduction factor, C_r.

ANGLED WAVE ATTACK
Due to the limited information on the performance of rough and armoured seawalls under angled wave attack it is suggested that the methods described in Boxes 3.1 (simple slopes) and 3.2 (bermed slopes) are applied.

3.4 Wave return walls

A limited amount of research work has been completed into the performance of wave return walls sited at the crest of seawalls. Two methods are presented here, one for permeable structures and one for impermeable structures. The permeability of the structure, in particular that of the crest, was found to be an important factor in the performance of the return wall. Owen and Steele (1991) undertook the most comprehensive study, investigating the performance of recurved wave return walls on top of 1:2 and 1:4 simply sloping seawalls (see Figure 3.7). Owen and Steele (1991) quantified the performance of return walls with a discharge factor, D_f, i.e., the ratio of the discharge overtopping the recurve wall to the discharge which would have occurred if the recurve wall had been absent. It was discovered that the performance of a recurve wall is primarily dependent on its height and on the discharge which is incident upon it.

Banyard and Herbert (1995) studied the performance of recurve walls under angled wave attack. The recurve walls exhibited similar behaviour in both short and long-crested seas. Under angled wave attack significant increases in overtopping discharges can occur. The largest measured increases in overtopping were over six times that predicted for normal wave attack. Banyard and Herbert (1995) found that the parameter which most effected the overtopping ratio, O_r, was the wall's discharge factor, D_f, rather than the angle of attack. The greatest increases in overtopping occurred when the discharge factor was low. For $D_f > 0.31$ however, discharge under angled wave attack decreased. The work of Owen and Steele (1991) and Banyard and Herbert (1995) is summarised in Box 3.5 below and advice is provided on its application to other types of seawall cross-section.

BOX 3.5
WAVE RETURN WALLS ON IMPERMEABLE SEAWALLS
MEAN OVERTOPPING DISCHARGE

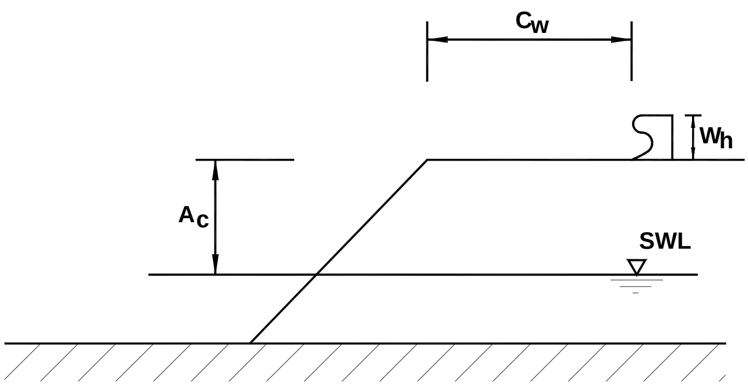
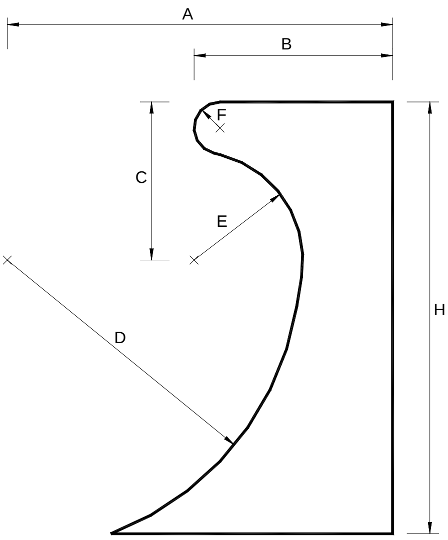


Figure 3.7 Wave return wall

The following design method is applicable to the recurved return wall profile proposed by Berkeley-Thorn and Roberts (1981) and illustrated in Figure 3.8. This is a very efficient type of return wall. Alternative profiles may be significantly less efficient.

BOX 3.5 (continued)



H	0.61	0.91	1.22	1.52	1.83	
A	0.53	0.79	1.05	1.33	1.63	
B	0.38	0.50	0.61	0.72	0.84	
C	0.33	0.42	0.52	0.60	0.67	(metres)
D	0.31	0.52	0.75	0.99	1.24	
E	0.15	0.23	0.30	0.38	0.46	
F	0.10	0.10	0.11	0.11	0.11	

Figure 3.8 Basic form of recurved wall profile

NORMAL WAVE ATTACK

First use the following equations to calculate the discharge which arrives at the base of the return wall :-

$$A_{c*} = A_c/(T_m(gH_s)^{0.5}) \quad (10)$$

$$Q_{b*} = A \exp(-BA_{c*}) \quad (11)$$

$$Q_b = Q_{b*}\, T_m\, g\, H_s \quad (12)$$

where A_c is the freeboard of the top of the slope (or the base of the return wall) (m)
H_s is the significant wave height at the toe of the seawall (m)
T_m is the mean wave period at the toe of the seawall (s)
g is acceleration due to gravity (m/s^2)
A,B are empirical coefficients dependent upon the cross-section of the seawall (see Table 1)
Q_b is the base discharge per metre run of seawall, i.e., that which arrives at the base of the return wall. For impermeable seawalls this is the same discharge that reaches the crest of the slope (m^3/s/m)

Equation 11 is valid for $0.02 < A_{c*} < 0.30$

The dimensionless wall height is defined as :-

$$W_* = W_h/A_c \quad (13)$$

Where W_h is the height of the wave return wall (m)

BOX 3.5 (continued)

BOX 3.5 (continued)

Knowledge of the dimensionless wall height, W_*, the seaward slope of the seawall and the distance of the return wall behind the top of the seaward slope, C_w, allows an adjustment factor, A_f, to be obtained from Table 4.

The adjusted slope freeboard, X_*, is then given by :-

$$X_* = A_f\, A_{c*} \qquad (14)$$

The values of X_* and W_* allow the use of Figure 3.9 in order to obtain a discharge factor, D_f. The mean discharge, Q, is then determined from:-

$$Q = Q_b\, D_f \qquad (15)$$

The adjustment factor, A_f, is presently only available for slopes of 1:2 and 1:4 and therefore some interpolation will be required for alternative slopes. It is recommended that for slopes between 1:1 and 1:2½ adjustment factors applicable to the 1:2 slope be employed in the analysis. For slope angles between 1:2½ and 1:4 it is suggested that linear interpolation be carried out between the available adjustment factors based upon the cotangent of the slope of the structure. For slopes shallower than 1:4 a conservative solution will be ensured if adjustment factors applicable to a 1:4 slope are employed. Although the adjustment factors may be obtained by interpolation, the value of Q_{b*} in this analysis should always be calculated using coefficients of A and B applicable to the precise seawall profile.

ANGLED WAVE ATTACK

Overtopping discharge can substantially increase under angled wave attack. In order to determine overtopping due to angled wave attack in short or long crested seas the discharge assuming normal attack should first be calculated. For angles of wave attack from 0^o - 45^o the overtopping ratio, O_r, may then be determined from the following equation:-

$$O_r = -1.18\,\mathrm{Ln}(D_f) - 0.40 \qquad (16)$$

where O_r is the ratio of the discharge under angled wave attack to that under normal attack and D_f is the discharge factor of the return wall (see Figure 3.9).

A minimum value of $O_r = 0.1$ should be assumed if a value of $O_r < 0.1$ is calculated from equation (16). For angles of attack greater than 45^o an overtopping ratio of 0.1 should be assumed.

IMPERMEABLE SEAWALLS WITH BERMS AND/OR ROUGHNESS

For smooth bermed and rough impermeable seawalls the relevant slope should be converted to an equivalent smooth simply sloping structure which, for the same wave conditions, water level and crest level, would give the same base overtopping discharge. The method of determining the overtopping performance of smooth bermed and rough impermeable seawalls is outlined in Boxes 3.1 to 3.4. Once an equivalent seawall slope has been obtained, the performance of the return wall can then be assessed using the method outlined above for simply sloping structures.

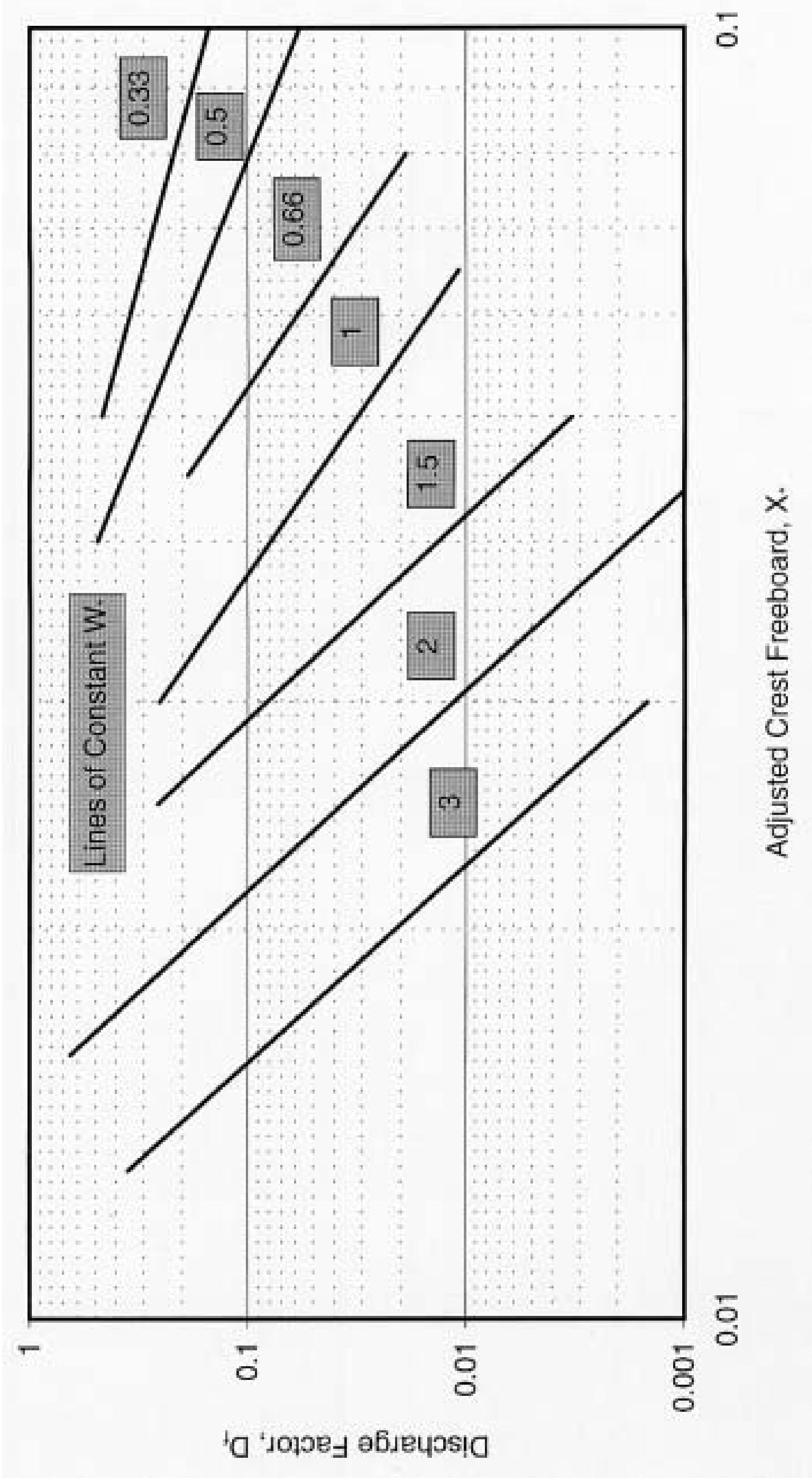


Figure 3.9 Discharge factors, walls on impermeable slopes

Bradbury and Allsop (1988) measured overtopping discharges for crown walls mounted on the top of rock breakwaters. It was discovered that crown walls were more effective at reducing overtopping when located on top of a permeable crest than on an impermeable one. The present work re-analysed Bradbury and Allsop's data and produced a design chart (Figure 3.10) which relates the discharge factor of the wall to the dimensionless discharge arriving at its base Q_{c*}. Details are given in Box 3.6.

BOX 3.6
WAVE RETURN WALLS ON PERMEABLE SLOPES

The following design method was developed from data based on rectangular section crown walls rather than more efficient recurved walls. The results can therefore be applied conservatively to recurved walls.

NORMAL WAVE ATTACK

First use the following equations to calculate the discharge which arrives at the crest of the armoured slope :-

$$A_{c*} = A_c/(T_m(gH_s)^{0.5}) \quad (17)$$

$$Q_{b*} = A \exp(-BA_{c*}/r) \quad (18)$$

$$Q_b = Q_{b*}\, T_m\, g\, H_s \quad (19)$$

where A_c is the freeboard of the top of the slope(or the base of the return wall) (m)
H_s is the significant wave height at the toe of the seawall (m)
T_m is the mean wave period at the toe of the seawall (s)
g is acceleration due to gravity (m/s^2)
A,B are empirical coefficients dependent upon the cross-section of the seawall (see Table 1)
Q_b is the mean discharge per metre run of seawall at the crest of the armoured slope ($m^3/s/m$)

Equation 18 is valid for $0.02 < A_{c*} < 0.30$

Then use equation (9) to reduce Q_b to take account of the permeable crest berm. i.e. $Q_c = C_r\, Q_b$ or in non-dimensional terms $Q_{c*} = C_r\, Q_{b*}$

where Q_c is the mean discharge per metre run of seawall, which arrives at the base of the return wall.

The dimensionless wall height, W_* is defined as :-

$$W_* = W_h/A_c \quad (20)$$

where W_h is the height of the wave return wall

Use Figure 3.10 to determine a wall discharge factor, D_f. The mean overtopping discharge, Q, is then given by :-

$$Q = Q_c D_f \quad (21)$$

ANGLED WAVE ATTACK

No data is available on angled wave attack. An estimate of the discharge may be obtained by using the method outlined in Box 3.5 for angled wave attack on crest walls on impermeable slopes.

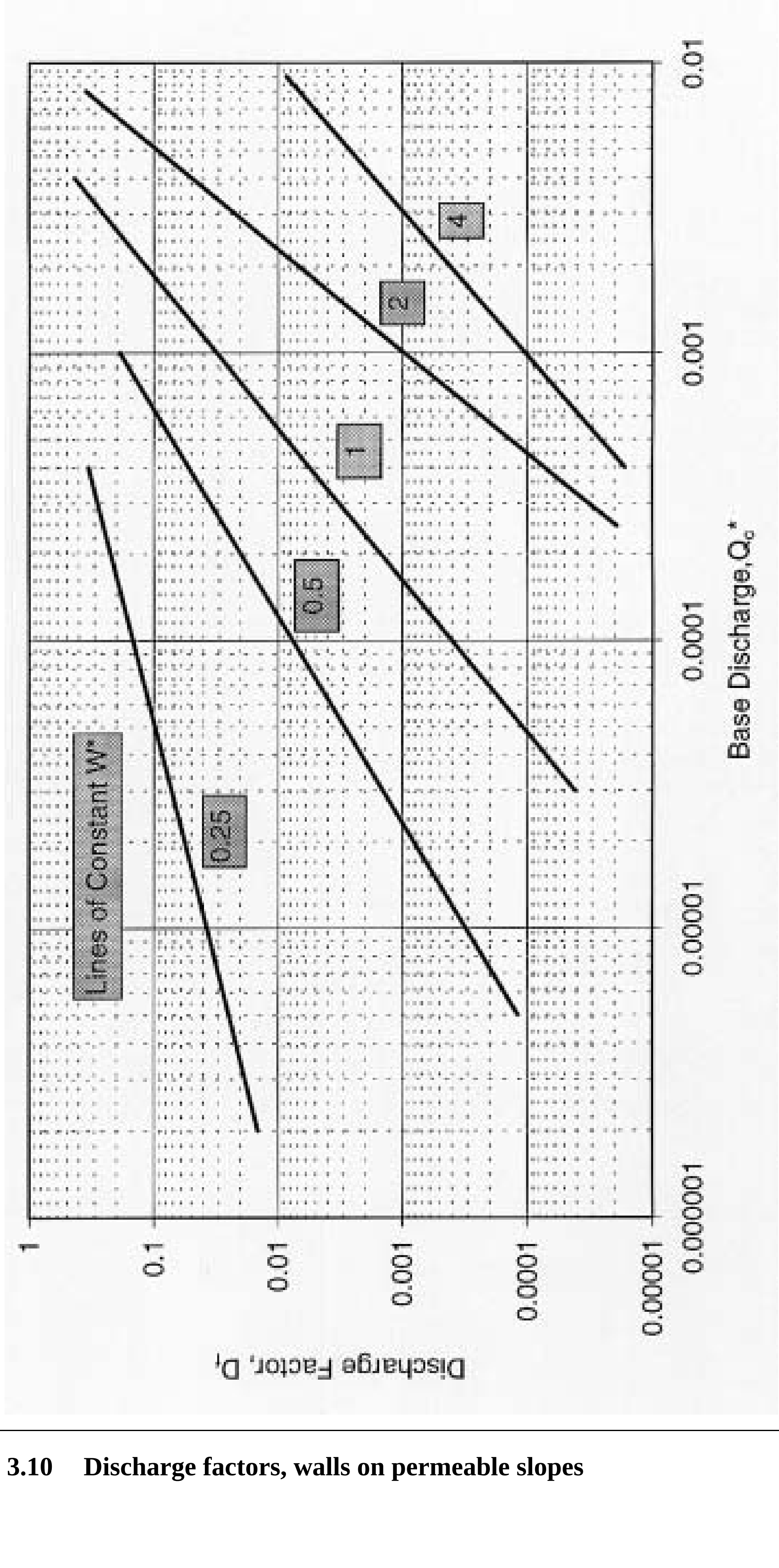


Figure 3.10 Discharge factors, walls on permeable slopes

3.5 Plain vertical walls

A variety of authors have examined the overtopping performance of vertical walls, including Goda (1985), whose method was confirmed and extended by Herbert (1993).

The method recommended however is that of Allsop et al (1995) who initially derived the following empirical equation:-

$$Q/(g\,H_s^3)^{0.5} = 0.03\exp(-2.05\,R_c/H_s)$$

This equation covers a range of relative freeboards of $0.03 < R_c/H_s < 3.2$, and is applicable to vertical walls in both deep and shallow water.

However, the results of Allsop et al (1995) indicated that the overtopping performance of vertical walls is dependent upon the predominant incident wave conditions. In deep water, waves hit the structure and are generally reflected back seawards (so-called reflecting waves). However, as the waves become limited by the water depth they are prone to break over the seawall (so-called impacting waves), causing a change in the overtopping performance. Allsop et al (1995) determined a parameter, h_*, which determined whether waves were in reflecting or impacting mode was defined as:-

$$h_* = (h/H_s)(2\pi h/(gT_m^2))$$

Reflecting waves predominate when $h_* > 0.3$; impacting waves when $h_* \leq 0.3$. New dimensionless parameters were developed for impacting waves and overtopping equations were derived for both types of wave action. The goodness of fit of the data was superior to all previously derived equations and, for this reason, this method is recommended for use with vertical walls. The method is summarised in Box 3.7.

Franco (1996) carried out a comprehensive series of experiments on the influence of 3-dimensional waves on vertical walls in deep water. It was discovered that the overtopping discharge generally reduced as the angle of attack diverged from the normal. A reduction parameter, γ, was derived as a function of angle of attack, β. The reduction parameter, γ, modifies the basic deep water overtopping equation thus:-

$$Q/(g\,H_s^3)^{0.5} = A\exp(\,(-B/\gamma)\,(R_c/H_{si}))$$

Where A and B are the coefficients for normal wave attack and γ is generally < 1 for angled wave attack. Discharge reduced for angles of attack from normal to 45^o. At angles of attack greater than 45^o discharge remained almost constant. The reduction coefficient, γ can be approximated by a linear equation as shown in Box 3.7.

BOX 3.7
PLAIN VERTICAL WALLS
MEAN OVERTOPPING DISCHARGE

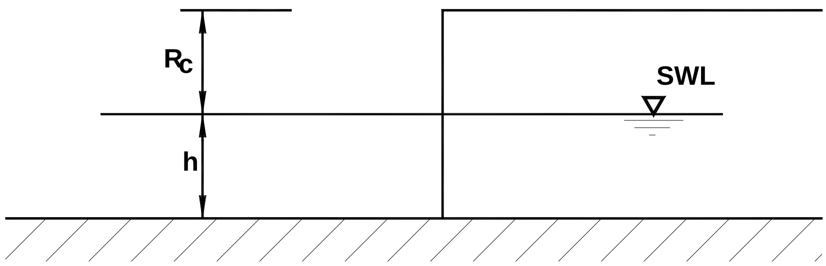


Figure 3.11 Plain vertical wall

NORMAL WAVE ATTACK

First calculate h_* to determine whether the waves are predominantly impacting or reflecting. The parameter h_* is given by :-

$$h_* = (h/H_s)(2\pi h/(gT_m^2)) \qquad (22)$$

where
- h is the water depth at the toe of the structure (m)
- H_s is the significant wave height at the toe of the structure (m)
- g is acceleration due to gravity (m/s^2)
- T_m is the mean wave period at the toe of the structure (s)

Reflecting waves predominate when $h_* > 0.3$, in which case the following equation applies:-

$$Q\# = 0.05 \exp(-2.78\, R_c/H_s) \qquad (23)$$

where
- $Q\#$ is the dimensionless discharge, given by $Q/(gH_s^3)^{0.5}$
- Q is the mean overtopping discharge rate per metre run of seawall (m^3/s/m)
- R_c is the freeboard (the height of the crest of the wall above still water level) (m)

Equation 23 is valid for $0.03 < R_c/H_s < 3.2$

Impact waves predominate when $h_* \le 0.3$, in which case the following equation applies:-

$$Q_h = 0.000137\, R_h^{-3.24} \qquad (24)$$

where Q_h is the dimensionless discharge, given by :-

$$Q_h = \{Q/(gh^3)^{0.5}\} / h_*^2 \qquad (25)$$

and R_h is the dimensionless crest freeboard, given by :-

$$R_h = (R_c/H_s)h_* \qquad (26)$$

Equation 26 is valid for $0.05 < R_h < 1.00$

ANGLED WAVE ATTACK

For reflecting waves modify equation (23) as follows :-

BOX 3.7 (continued)

BOX 3.7 (continued)

$$Q\# = 0.05 \exp\{(-2.78/\gamma)(R_c/H_s)\} \qquad (27)$$

γ is the reduction factor for angle of attack and is given by :-

$$\gamma = 1 - 0.0062\beta \qquad \text{for } 0^o < \beta \leq 45^o \qquad (28)$$

$$\gamma = 0.72 \qquad \text{for } \beta > 45^o \qquad (29)$$

where β is the angle of attack relative to the normal, in degrees.

For impacting waves
No data is available to describe the effect of angled wave attack on the mean discharge when waves are in impacting mode.

3.6 Composite vertical walls

Many vertical walls are fronted by a rock-armoured mound usually designed with the intention of limiting overtopping or protecting the toe of the structure from bed erosion. The size and geometry of the armour can vary considerably and thus the overtopping behaviour of the structure can differ quite markedly. Three basic types of mound can be identified :-

i) Small toe mounds which have an insignificant effect on the waves approaching the wall.

ii) Larger mounds, which significantly affect the incident wave conditions and have crests below still water level.

iii) Emergent mounds in which the crest of the armour protrudes above still water level.

These mound types, together with the relevant parameters, are illustrated in Figures 3.12 and 3.13.

Allsop et al (1995) completed a comprehensive analysis of composite vertical structures identifying empirical equations for all three mound types. A parameter, d_*, was identified which determined whether the mound could be classified as large or small. As defined, d_* plays a similar role to the h_* parameter for vertical walls, the difference being that the relative wave height is determined with respect to the water depth over the mound d, rather than the depth at the toe, h. The discharge is then dependent upon whether the mound causes the incident waves to impact on to the structure or to reflect. Overtopping due to impacting waves is significantly greater than that caused by reflecting waves but it is not yet possible to distinguish the parameters that identify the two wave types. In order to take a conservative approach it is therefore recommended that the equations for impacting waves be used. Details are given in Box 3.8.

Structures with a small freeboard ($R_c/H_{si} < 1.5$) were discovered to behave as plain vertical walls. No distinction was made between deep and shallow water.

BOX 3.8
COMPOSITE VERTICAL WALLS
MEAN OVERTOPPING DISCHARGE

Although only determined for rock armoured mounds, it is considered that the following equations may also be applied to mounds armoured with concrete units.

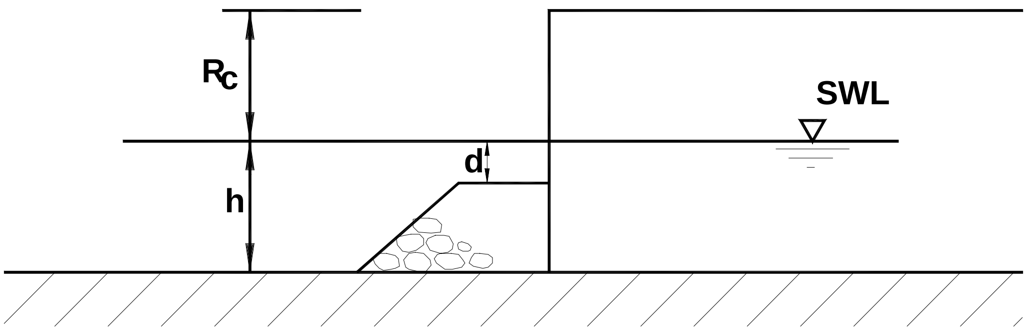


Figure 3.12 Composite vertical wall, submerged mound

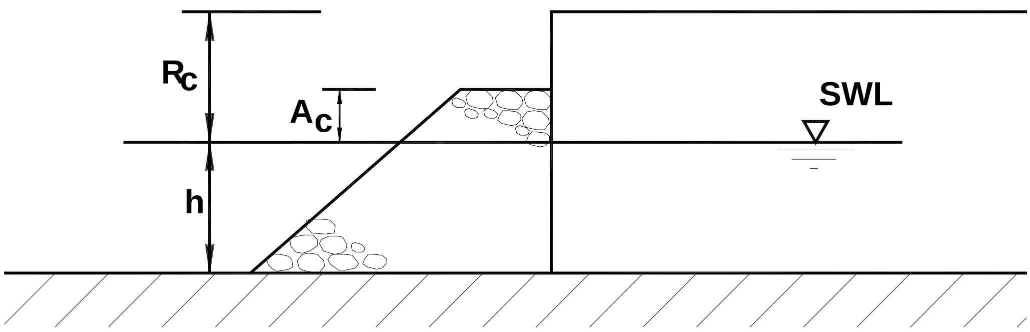


Figure 3.13 Composite vertical wall, emergent mound

NORMAL WAVE ATTACK
First determine whether or not the waves are affected by the presence of the mound. The parameter which governs this is d_*, given by the following equation :-

$$d_* = (d/H_s)(2\pi h/(gT_m^2)) \qquad (30)$$

where d is the water depth over the mound (m)
h is the water depth at the toe of the structure (m)
H_s is the significant wave height at the toe of the structure (m)
g is acceleration due to gravity (m/s^2)
T_m is the mean wave period at the toe of the structure (s)

Mounds are classified as small when d_* > 0.3, in which case the structure behaves in a similar manner to a plain vertical wall subject to reflecting waves. The procedure outlined in Box 3.7 should therefore be followed.

BOX 3.8 (continued)

BOX 3.8 (continued)

Mounds are classified as large when $d_* \leq 0.3$, in which case the mound begins to affect the overtopping performance of the seawall. It is recommended that the following equations, which are strictly applicable to impacting waves only, be employed in order to ensure a conservative design :-

$$Q_d = 4.63 \times 10^{-4} (R_d)^{-2.79} \qquad (31)$$

where Q_d is the dimensionless discharge given by :-

$$Q_d = \{Q/(gd^3)^{0.5}\} / d_*^2 \qquad (32)$$

and R_d is the dimensionless crest freeboard given by :-

$$R_d = (R_c/H_s)d_* \qquad (33)$$

Equation 31 is valid for $0.05 < R_d < 1.00$

Composite structures with emergent mounds behave as armoured slopes with rectangular crest walls. The procedures outlined in Box 3.6 are therefore recommended.

A further category of composite structures is those with a small relative freeboard, i.e. for which $R_c/H_s < 1.5$. These structures behave in a similar manner to plain vertical walls. The equation which applies to vertical walls in all water depths is recommended, i.e.,

$$Q\# = 0.03 \exp(-2.05 R_c/H_s) \qquad (34)$$

where $Q\#$ is the dimensionless discharge, given by $Q/(gH_s^3)^{0.5}$
Q is the mean overtopping discharge rate per metre run of seawall ($m^3/s/m$)
R_c is the freeboard (the height of the crest of the wall above still water level) (m)

Equation 34 is valid for $0.03 < R_c/H_s < 3.2$

ANGLED WAVE ATTACK
In the absence of data on the overtopping performance of composite vertical walls under angled wave attack it is suggested that the method outlined in Box 3.7 for plain vertical walls should be employed. In the case of composite structures with emergent mounds the methods for armoured slopes with rectangular crest walls, outlined in Box 3.6 should be followed.

4. PREDICTING THE MAXIMUM INDIVIDUAL OVERTOPPING EVENT

4.1 Number of waves overtopping

4.1.1 Introduction

The preceding section presented methods for predicting the mean overtopping discharge for a variety of structures. In terms of safety, however, the tolerable limits may be more usefully defined in terms of the maximum individual overtopping event. This section presents methods for estimating these peak events. The wave conditions and structural parameters, as well as the mean discharges, are required to assess individual overtopping volumes.

An intermediate step in the estimation of the maximum individual overtopping event is an estimate of the number of waves that overtop the structure. Data is available for a variety of structural types, although it is much more limited in extent than data concerning mean discharges.

The number of waves overtopping a structure can be equated to the number of waves with a calculated run-up greater than its crest elevation. This is an approximation in that it assumes there is no interaction between successive waves. In reality such interaction exists for some structures, as a wave overtopping a structure can allow greater run-up in the following wave.

4.1.2 Sloping Seawalls

Owen (1982) examines the number of waves overtopping a smooth, impermeable slope. If the incident waves, and hence the run-up levels, are Rayleigh distributed then the proportion of waves overtopping is described by an equation of the form :-

$$N_{ow}/N_w = \exp(-C\, R_*^2)$$

where N_{ow} is the number of waves overtopping
N_w is the number of waves in the sequence
C is a parameter which depends on the seawall slope
R_* is the dimensionless freeboard given by $R_c / (T_m\, (gH_s)^{0.5})$

Values of C were calculated for a variety of smooth, impermeable slopes (Table 5). There was generally good agreement between computed values of C and model test results at slopes of 1:2 and 1:4. At a slope of 1:1 the equation tended to under-predict the proportion of waves overtopping because of the wave interaction discussed above.

The run-up level of an individual wave on an armoured slope will be reduced compared to that on a smooth, impermeable slope in proportion to the roughness factor, r. The proportion of waves overtopping a rough, permeable slope will therefore be :-

$$N_{ow}/N_w = \exp(-C\, (R_*/r)^2\,)$$

This equation was validated, with good agreement, by test results on 1:1.5 slopes with a variety of armour types (Besley et al (1998)). Results of tests on other slopes are unavailable. Box 4.1outlines the method.

Banyard and Herbert (1995) calculated overtopping reduction factors for angled wave attack on sloping seawalls, although no data was recorded on the number of waves overtopping. Franco (1996) concluded that as a result of an increase in the angle of wave attack the mean discharge reduced. Similarly the number of waves, overtopping were reduced but to a lesser extent. It should be noted that a particular level of discharge distributed over a smaller number of waves will give larger values of individual volume, therefore a conservative approach is to under-predict the number of waves overtopping. In the absence of definitive data it is therefore recommended that the same reduction factor calculated for the mean discharge be applied to the number of waves overtopping.

BOX 4.1
NUMBER OF WAVES OVERTOPPING SLOPED STRUCTURES
(SIMPLE SLOPES ONLY)

NORMAL WAVE ATTACK
The proportion of waves overtopping a sloped seawall is given by :-

$$N_{ow}/N_w = \exp(-C\,(R_*/r)^2\,) \qquad (35)$$

where N_{ow} is the number of waves overtopping
N_w is the number of waves in the sequence
C is a parameter, which depends on the slope (see Table 5)
R_* is the dimensionless freeboard given by $R_c / (T_m\,(gH_s)^{0.5})$
r is the roughness coefficient (see Table 3)

Equation 35 is valid for $0.05 < R_* < 0.3$

This method applies to simply sloping structures only. On smooth, impermeable slopes it has been validated for slopes of between 1:1 and 1:4. For armoured slopes it is recommended for slopes of between 1:1 and 1:2 only.

ANGLED WAVE ATTACK
Apply the reduction factor calculated using equation 4 for the mean discharge directly to the number of waves overtopping. This will underestimate the number of waves overtopping and will therefore be conservative in its estimate of individual volumes.

For other sloped structures, including those with crest walls and berms, an empirical relationship between the proportion of waves overtopping and the dimensionless discharge has been determined during the preparation of this manual (Box 4.2). This method was based on the results of tests conducted on a variety of sloped structures. There was a considerable degree of averaging involved in compiling the equations and the results may be less accurate than those given in Box 4.1, especially at low values of Q_*.

BOX 4.2
NUMBER OF WAVES OVERTOPPING SLOPED STRUCTURES
(GENERAL)

NORMAL WAVE ATTACK
First calculate the dimensionless discharge, Q_*, using the appropriate method. The proportion of waves overtopping a sloped structure is then given by :-

$$N_{ow}/N_w = 55.41 Q_*^{\ 0.634} \qquad \text{for } 0 < Q_* < 0.0008 \qquad (36)$$

$$N_{ow}/N_w = 2.502 Q_*^{\ 0.199} \qquad \text{for } 0.0008 \le Q_* < 0.01 \qquad (37)$$

$$N_{ow} = N_w \qquad \text{for } Q_* \ge 0.01 \qquad (38)$$

where N_{ow} is the number of waves overtopping
N_w is the number of waves in the sequence
Q_* is the dimensionless overtopping discharge, given by $Q_* = Q / (T_m g H_s)$

This method was compiled from the results of tests conducted on a variety of structural types. It is therefore generally less accurate than that given in Box 4.1, especially at low values of Q_*.

ANGLED WAVE ATTACK
Apply the reduction factor calculated using equation 4 for the mean discharge directly to the number of waves overtopping. This will underestimate the number of waves overtopping and will therefore be conservative in its estimate of individual volumes.

4.1.3 Vertical Walls

Franco et al (1994) developed the following equation, based on the results of tests conducted in relatively deep water :-

$$N_{ow}/N_w = \exp \{-(1/0.91)^2 (R_c/H_s)^2\}$$

where N_{ow} = number of waves overtopping
N_w = number of waves in sequence

As for sloped structures, the expression for the number of waves overtopping the structure in deep water is described by Rayleigh distribution.

Franco measured the number of waves overtopping a vertical wall under angled wave attack. The number of waves overtopping reduces with increasing angle of attack, although not to the same extent as the mean discharge. The above equation was modified to take account of angled attack thus:-

$$N_{ow}/N_w = \exp \{-(1/C)^2 (R_c/H_s)^2\}$$

Where C = 0.91 for normal wave attack and is generally < 0.91 for angled wave attack. The coefficient C can be approximated as a linear function of the angle of wave attack, β. Details are given in Box 4.3.

Whilst Franco et al's equation provides a good description of the number of waves overtopping in deep and intermediate water, it under-predicts the number of waves overtopping in shallow water. Where breaking waves exist in significant numbers the distribution of individual wave heights diverges from the Rayleigh form. In addition the mechanism by which individual waves overtop the structure is altered, being dominated by impacting waves rather than run-up.

As described in Chapter 3, Allsop et al (1995) found that, when predicting overtopping discharge, reflecting and impacting waves (as identified by h_* from equation 22) require separate treatment. When $h_* > 0.3$, the overtopping discharge is accurately described by an equation relating it to R_c/H_s. When $h_* \leq 0.3$, however, it was found that the overtopping discharge was better described by relating it to a parameter, R_h, as defined by equations 24 and 26.

Similarly it has been found that the number of waves overtopping a vertical wall in shallow water is described by relating it to R_h, thus :-

$$N_{ow}/N_w = 0.031\, R_h^{-0.99}$$

Recommendations for plain vertical walls are given in Box 4.3.

No data is available describing the number of waves which overtop vertically composite structures (see Figures 3.12 and 3.13). However, based on the mean discharge behaviour of such structures the recommendations in Box 4.4 can be made.

BOX 4.3

NUMBER OF WAVES OVERTOPPING A VERTICAL STRUCTURE

NORMAL WAVE ATTACK

First calculate h_* to determine whether the waves are predominantly impacting or reflecting using the following equation :-

$$h_* = (h/H_s)(2\pi h/(gT_m^2)) \qquad (39)$$

where h is the water depth at the toe of the structure (m)
H_s is the significant wave height at the toe of the structure (m)
g is acceleration due to gravity (m/s^2)
T_m is the mean wave period at the toe of the structure (s)

Reflected waves predominate when $h_* > 0.3$, in which case the following equation applies:-

$$N_{ow}/N_w = \exp\{-(1/0.91)^2 (R_c/H_s)^2\} \qquad (40)$$

where N_{ow} is the number of waves overtopping
N_w is the number of waves in the sequence
R_c is the freeboard (the height of the crest of the wall above still water level) (m)
H_s is the significant wave height at the toe of the wall (m)

Equation 40 is valid for $0.03 < Rc/Hs < 3.2$

Impact waves predominate when $h_* \leq 0.3$, in which case the following equation applies:-

$$N_{ow}/N_w = 0.031\, R_h^{-0.99} \qquad (41)$$

Equation 41 is valid for $0.05 < R_h < 1.0$

where R_h is the dimensionless crest freeboard given by :-

$$R_h = (R_c/H_s)\,(h/H_s)(2\pi h/(gT_m^2)) \qquad (42)$$

ANGLED WAVE ATTACK

For reflecting waves use equation (40) modified thus :-

$$N_{ow}/N_w = \exp\{-(1/C)^2 (R_c/H_s)^2\} \qquad (43)$$

C is a correction factor for angled wave attack and is given by :-

$$C = 0.91 - 0.00425\beta \qquad \text{for } 0^o < \beta \leq 40^o \qquad (44)$$

$$C = 0.74 \qquad \text{for } \beta > 40^o \qquad (45)$$

The convention for the angle of wave attack is shown in Figure 3.2.

For impacting waves

No data is available to describe the effect of angled wave attack on the proportion of overtopping waves when the waves are in impacting mode.

BOX 4.4
NUMBER OF WAVES OVERTOPPING A VERTICALLY COMPOSITE STRUCTURE
NORMAL AND ANGLED WAVE ATTACK

Limited information is available on the number of waves overtopping composite structures. Based on the observations of mean discharges, however, the following recommendations can be made.

First, determine whether or not the waves are affected by the presence of the mound, by calculating d_* using equation (30).

Structures with a small mound ($d_* > 0.3$) should be treated as plain vertical walls subject to reflecting waves, as in Box 4.3.

Structures with a large mound ($d_* \leq 0.3$) should be treated as plain vertical walls subject to impacting waves, as in Box4.3.

Structures with an emergent mound should be treated as sloped structures with crest walls, as in Box 4.2.

4.2 Maximum individual overtopping event

Given that the number of overtopping events and the mean discharge can be predicted using the methods described above, it is then possible to estimate the magnitude of the largest individual overtopping event. The method used has been developed from data acquired from model tests in which the volume associated with each overtopping wave was measured individually.

It was found that the distribution of the volumes, V, of individual overtopping events can be described by the two parameter Weibull probability distribution :-

$P(V) = 1 - \exp(- (V/a)^b)$

where $P(V)$ is the probability of non-exceedance of a given volume, V
b is the shape parameter
a is the scale parameter and can be calculated from V_{bar} and b

The Weibull distribution corresponds closely to the distribution of individual volumes at higher values of V and can therefore be used to accurately represent extreme values of V. The maximum expected individual overtopping volume, V_{max}, in a sequence of N overtopping waves is given by :-

$V_{max} = a (\ln(N_{ow}))^{1/b}$

Analysis of model test results produced values of a and b for a variety of structural types. The quantity of available data on individual overtopping volumes is quite limited, but the range of structures for which data could be obtained showed fairly consistent behaviour.

Franco et al (1994) showed that the shape parameter, b, was 0.75 for a vertical wall subject to reflecting waves. This was confirmed by further analysis undertaken during the preparation of this manual. Franco (1996) found that b was dependent on wave steepness and short-crestedness, being lower for long-crested waves. Since a lower value of b dictates a higher V_{max} the result for long-crested waves will be used as a conservative approach.

The analysis undertaken for this study showed that b = 0.85 for impacting waves on a vertical wall. No dependency on wave steepness was apparent.

Franco (1996) found that for sloping structures b was generally 0.1 higher than the equivalent result for a vertical wall in deep water. Further analysis for this manual determined a similar result, for a variety of sloped structures. There was more variation in the parameters a and b determined for sloped structures than for vertical walls. This is to be expected, as the structures included in the study incorporated a variety of structural features, such as berms and crest walls etc. However, no distinct pattern emerged to enable particular values of a and b to be associated with specific structural types.

For all structural types and wave heights no dependency on angle of attack was perceived.

Recommendations are given in Box 4.5 below.

BOX 4.5
MAXIMUM INDIVIDUAL OVERTOPPING VOLUME

NORMAL AND ANGLED WAVE ATTACK

In a sequence of overtopping waves the maximum individual overtopping volume per m run of the seawall, V_{max}, is given by :-

$$V_{max} = a\,(\ln(N_{ow}))^{1/b} \qquad (46)$$

where N_{ow} is the number of overtopping waves
a, b are empirical coefficients

N_{ow} can be estimated using the techniques outlined in Boxes 4.1, 4.2, 4.3 or 4.4, depending on the seawall type. Equation 46 should only be used for $N_{ow} \geq 5$. The coefficients a and b depend on the seawall type and the wave conditions, as follows :-

For vertical walls first determine whether the waves are in reflecting or impacting mode, as described in Box 3.7.

If the waves are in reflecting mode then :-

$a = 0.74\ V_{bar}$, $b = 0.66$ for $s_{op} = 0.02$

and $a = 0.90\ V_{bar}$, $b = 0.82$ for $s_{op} = 0.04$

where s_{op} is the offshore wave steepness, given by $2\pi H_{so}/(gT_{po}^2)$

For values of s_{op} between 0.02 and 0.04 interpolate between these values.

If the waves are in impacting mode then :-

$a = 0.92\ V_{bar}$ $b = 0.85$

with no dependency on wave steepness.

For sloped seawalls :-

$a = 0.85\ V_{bar}$ $b = 0.76$ for $s_{op} = 0.02$

BOX 4.5 (continued)

BOX 4.5 (continued)

and $a = 0.96\ V_{bar}$ $b = 0.92$ for $s_{op} = 0.04$

where s_{op} is the offshore wave steepness given by $2\pi H_{so}/(gT_{po}^2)$

For values of s_{op} between 0.02 and 0.04 interpolate between these values.

In each case V_{bar} is the average overtopping volume per overtopping wave, given by :-

$$V_{bar} = QT_mN_w / N_{ow} \qquad (47)$$

where Q is the mean discharge ($m^3/s/m$)
N_{ow} is the number of overtopping waves
N_w is the number of waves in the sequence
T_m is the mean period of the waves at the toe of the structure (s)

Q and N_{ow} are calculated using the method appropriate to the structure (e.g. vertical walls Boxes 3.7 and 4.3).

Limited information is available on individual wave overtopping volumes for composite structures. Based on the observations of mean discharges, however, the following recommendations can be made.

First determine whether or not the waves are affected by the presence of the mound, by calculating d_* from equation 30.

Structures with a small mound ($d_* > 0.3$) should be treated as plain vertical walls subject to reflecting waves.

Structures with a large mound ($d_* \leq 0.3$) should be treated as plain vertical walls subject to impacting waves.

Structures with an emergent mound should be treated as sloped structures.

5. TOLERABLE DISCHARGES

5.1 Tolerable mean discharges

Damage to seawalls, buildings or infrastructure has previously been defined as a function of the mean discharge. The guidelines given by Simm (1991) are reproduced in Box 5.1 below. Different limits are given for embankments (with back slopes) and revetments (without back slopes).

BOX 5.1
TOLERABLE MEAN DISCHARGES ($m^3/s/m$)

Buildings :-					
		Q	<	$1x10^{-6}$	No damage
$1x10^{-6}$	<	Q	<	$3x10^{-5}$	Minor damage to fittings etc
		Q	>	$3x10^{-5}$	Structural damage
Embankment Seawalls :-					
		Q	<	0.002	No damage
0.002	<	Q	<	0.02	Damage if crest not protected
0.02	<	Q	<	0.05	Damage if back slope not protected
		Q	>	0.05	Damage even if fully protected
Revetment Seawalls :-					
		Q	<	0.05	No damage
0.05	<	Q	<	0.2	Damage if promenade not paved
		Q	>	0.2	Damage even if promenade paved

5.2 Tolerable peak event

Overtopping limits have traditionally been specified in terms of mean discharge rates. When attempting to assess safety levels however, this approach is questionable, as the maximum individual event is expected to be of greater significance. The analysis presented in Chapter 4, demonstrates that, for a given level of mean discharge, the volume of the largest overtopping event will vary with wave conditions and structural type. It is thus inconsistent to specify safety levels with sole reference to mean discharge levels.

Data correlating individual overtopping events with hazard levels are rare. Franco et al (1994), investigated safe overtopping limits for pedestrians and vehicles. It was demonstrated, by means of model tests and experiments on volunteers, that the danger level which an individual overtopping event represents could be directly related to its volume. A volume was defined as “safe” if it created a less than 10% chance of a person falling over. An event was defined as “very dangerous” if it created a greater than 90% chance of a person falling over. It is felt that this higher limit represents an unacceptable risk to pedestrians and that the tolerable discharge should be closer to the lower 10% limit.

Franco et al (1994) discovered that the “safe” limit varied with structural type. A given volume overtopping a vertical structure was found to be more dangerous than the same volume overtopping a horizontally composite structure, with an emergent mound. The “safe” limit for a vertical wall was found to be $0.1m^3/m$, whilst for a horizontally composite structure it was $0.75m^3/m$. However, Franco et al (1994) also noted that a volume as low as $0.05m^3/m$ could unbalance an individual when striking their upper body without warning.

The latter figure was determined from experiments on volunteers rather than from model tests and can thus be considered more realistic.

Franco et al (1994) determined very different limits for different types of structure. The authors consider that such differences may be due to the structure's crest detail, in particular the height of any parapet wall, if one is present. This determines how the overtopping water jet impacts upon the individual.

Smith et al (1994) reported on full scale tests conducted on grass dykes. An observer stood on the crest of the dyke as it was being tested. The experiment was intended to determine safe overtopping limits for personnel carrying out inspection and repair work. Smith et al (1994) concluded that work was unsafe when the mean discharge exceeded 10 l/s/m. From examination of Smith et al's data this corresponded to a V_{max} of approximately $1.6m^3/m$. This is considerably higher than the limits determined by Franco et al (1994), and accords with Franco et al's observation that safe limit of V_{max} varies with structural type. One reason for this variation may be, as suggested above, the different way in which the water strikes the individual. Smith et al (1994) reported that the vast majority of the overtopping discharge acted on the observer's legs only. It must also be borne in mind that the safety limits for trained personnel working on a structure and anticipating overtopping are higher than those for other users.

Information on prototype safety is available from Herbert (1996) who monitored overtopping at a vertical seawall. During the installation and operation of the apparatus it was noted that personnel could work safely on the crest of the wall during mean discharges of up to 0.1 l/s/m. Individual overtopping volumes were not measured. However, the methods described in the preceding sections of this report can be used to provide an estimate of V_{max}, given that the mean discharge and the incident wave conditions were measured. This results in an estimated V_{max} of approximately $0.04m^3/m$ for the sea state which caused the 0.1l/s/m discharge. This is in close agreement with Franco et al's estimate of the volume ($0.05m^3/m$) which could cause someone to lose their balance.

Herbert (1996) also noted that overtopping became a danger to vehicles when the mean discharge exceeded 0.2 l/s/m. Using the process described above it was determined that this corresponds to a V_{max} of approximately $0.06m^3/m$. It is thus recommended that this be adopted as the upper safe limit for vehicles driven at any speed.

It was decided that a limiting individual volume of $0.04m^3/m$ should be applied to all structures for pedestrians, despite the fact that tests have suggested that the limit varies with structural type. The authors consider that it is possible for the most dangerous mode of impact to occur on all types of structure. Whenever green water overtopping occurs then it should not exceed these limits.

Analysis of the performance of a variety of structures showed that there are many potentially hazardous situations in which only a small number of green water overtopping events occur. These were generally situations in which structures with high crest freeboards are attacked by large, unbroken waves. Overtopping discharge was in the form of a small number of large events. Even if the incidence of overtopping is reduced, say by increasing the crest level, the events which do occur will still be dangerous. In these cases safety can be assured only when no overtopping events take place. However, the random nature of real seas makes it difficult to specify a situation in which overtopping events are completely eliminated. A probabilistic approach is therefore required. The risk that there will be at least one overtopping event during a sequence of N_w waves is given by :-

$$P(\text{overtopping}) = 1 - (1 - N_{ow}/N_w)^{Nw}$$

where N_{ow}/N_w is the proportion of waves overtopping. The acceptable risk of an overtopping event occurring may depend on the use of the structure in question. It is therefore recommended that when analysis of individual overtopping volumes indicates that very small numbers of overtopping events create unsafe conditions, the structure should be optimised by limiting the probability of an overtopping event taking place to an acceptable level.

A contrasting situation occurs whenever overtopping is in the form of a larger number of small events. In this case large numbers of overtopping events can be tolerated, provided that the predicted value of V_{max} is below the limits discussed above. Recommendations are summarised in Box 5.2.

BOX 5.2
TOLERABLE INDIVIDUAL OVERTOPPING EVENTS

The risk of at least one overtopping event occurring, P(overtopping) during the course of a storm is given by :-

$$P(\text{overtopping}) = 1 - (1 - N_{ow}/N_w)^{Nw} \qquad (48)$$

where N_{ow} / N_w is the proportion of overtopping waves

If the risk of an overtopping event occurring is unacceptably high then the maximum volume likely to overtop, V_{max}, must be estimated. It is suggested that for pedestrian and vehicle safety on structures to which the public have access, the risk of an overtopping event occurring during a sequence of 1000 waves should be less than 1%. Less stringent criteria may apply where access to structures is restricted.

V_{max} can be estimated using the method presented in Box 4.5

The following limits then apply:-

For pedestrians:-
All structures become dangerous for pedestrians when the largest overtopping event exceeds 0.04 m^3/m.

For vehicles:-
All structures become dangerous for vehicles driven at any speed when the largest overtopping event exceeds $0.06m^3/m$.

If the analysis suggests that even a small number of overtopping events will create conditions which are unsafe then the structure should be optimized by limiting the risk of overtopping to an acceptable level. When the analysis suggests that individual overtopping events can be tolerated then V_{max} should be limited to the values given above.

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Tables

Table 1 Empirical Coefficients – Simply Sloping Seawalls

Seawall Slope	A	B
1:1	7.94E-3	20.1
1:1.5	8.84E-3	19.9
1:2	9.39E-3	21.6
1:2.5	1.03E-2	24.5
1:3	1.09E-2	28.7
1:3.5	1.12E-2	34.1
1:4	1.16E-2	41.0
1:4.5	1.20E-2	47.7
1:5	1.31E-2	55.6

Table 2 Empirical Coefficients - Bermed Seawalls – Berm at or Below SWL

Seawall Slope	Berm Elevation	Berm Width	A	B
1:1 1:2 1:4	-4.0	10	6.40E-3 9.11E-3 1.45E-2	19.50 21.50 41.10
1:1 1:2 1:4	-2.0	5	3.40E-3 9.80E-3 1.59E-2	16.52 23.98 46.63
1:1 1:2 1:4	-2.0	10	1.63E-3 2.14E-3 3.93E-3	14.85 18.03 41.92
1:1 1:2 1:4	-2.0	20	8.80E-4 2.00E-3 8.50E-3	14.76 24.81 50.40
1:1 1:2 1:4	-2.0	40	3.80E-4 5.00E-4 4.70E-3	22.65 25.93 51.23
1:1 1:2 1:4	-2.0	80	2.40E-4 3.80E-4 8.80E-4	25.90 25.76 58.24
1:1 1:2 1:4	-1.0	5	1.55E-2 1.90E-2 5.00E-2	32.68 37.27 70.32
1:1 1:2 1:4	-1.0	10	9.25E-3 3.39E-2 3.03E-2	38.90 53.30 79.60
1:1 1:2 1:4	-1.0	20	7.50E-3 3.40E-3 3.90E-3	45.61 49.97 61.57
1:1 1:2 1:4	-1.0	40	1.20E-3 2.35E-3 1.45E-4	49.30 56.18 63.43
1:1 1:2 1:4	-1.0	80	4.10E-5 6.60E-5 5.40E-5	51.41 66.54 71.59
1:1 1:2 1:4	0.0	10	8.25E-3 1.78E-2 1.13E-2	40.94 52.80 68.66

Table 3 Typical roughness coefficients

Type of Seawall	Roughness Coefficient, r
Smooth concrete or asphalt	1.0
Smooth concrete blocks with Little or no drainage	1.0
Stone blocks, pitched or mortared	0.95
Stepped	0.95
Turf	0.9-1.0
One layer of rock armour on impermeable base	0.80
One layer of rock armour On permeable base	0.55 - 0.60
Two layers of rock armour	0.50 - 0.55

Table 4 Adjustment factors - wave return walls on impermeable seawalls

$W_h/A_c \geq 0.6$

Seawall Slope	Crest berm width (C_w)	A_f
1:2	0	1.00
1:2	4	1.07
1:2	8	1.10
1:4	0	1.27
1:4	4	1.22
1:4	8	1.33

$W_h/A_c < 0.6$

Seawall Slope	Crest berm width (C_w)	A_f
1:2	0	1.00
1:2	4	1.34
1:2	8	1.38
1:4	0	1.27
1:4	4	1.53
1:4	8	1.67

Table 5 Empirical coefficients - number of waves overtopping simply sloping seawall

Seawall Slope	C
1:1	63.8
1:2	37.8
1:4	110.5

SURVEY OF USERS

The objective of this survey is to assess the practical value of the overtopping manual to engineers. We would therefore be grateful if users of the manual could complete the following questionnaire. Responses can be faxed, if required, using a copy of the Faxback form overleaf.

Please consider the following points concerning the content and format of the manual.

A Content

Is the content of the manual sufficiently comprehensive?

Are any important aspects of overtopping design insufficiently covered or omitted?

Does the manual provide practical design and assessment methods?

B Format

Is the format of the manual convenient to use?

Are the design methods explained in sufficient detail?

Are the design methods presented in a clear and unambiguous manner?

Tolerable Discharge Data

We are interested in continually updating our data on all aspects of overtopping, but particularly in the field of tolerable overtopping discharges. If the users of this manual have any original data regarding this subject, we would be extremely interested. This could take the form of, for example, experiences of personnel working on structures subject to wave attack or the experiences of those responsible for public safety on seawalls and breakwaters. Please contact the address below if you are in possession of such information.

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A Content of Manual

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