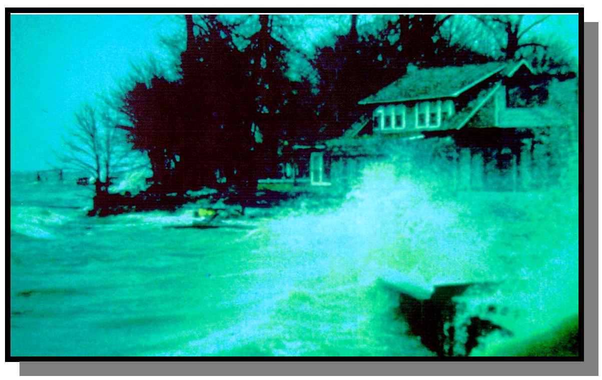
WAVE UPRUSH AND OVERTOPPING: METHODOLOGIES AND APPLICATIONS

GREAT LAKES - ST. LAWRENCE RIVER SYSTEM



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PREFACE

Wave uprush and wave overtopping are two important factors influencing the delineation of shoreline hazards and the design of coastal structures. Wave uprush (or runup) is the vertical height above the stillwater level to which water, from an incident wave, will rush up to on a shoreline or shoreline structure. Wave overtopping refers to the water which passes over the top of a shoreline bank or shoreline structure due to incident wave attack. Overtopping occurs when the limit of wave uprush exceeds the top elevation of the shoreline or structure.

This report is an updated and revised compilation of three earlier reports (Atria 1991a, 1991b and 1992) which presented methodologies for estimating shoreline wave uprush and overtopping. The first report (Atria 1991a) provided a literature review of various available methodologies for the prediction of wave uprush. The second report (Atria 1991b) summarized the wave uprush methodologies and provided typical example of applications. The third report (Atria 1992) presented a literature review of existing methodologies for the prediction of wave overtopping and included example applications. All three reports were based on the previous work of others.

At present, the understanding of uprush processes and wave overtopping is limited, and there seems to be no generic methodology for the prediction of wave uprush limits and overtopping rates. Existing guidance is mainly based on empirical research work, carried out in laboratory facilities, particularly for monochromatic waves. Review of the literature has shown that several uprush and overtopping methodologies may be considered "accepted" practice if they are used in the same context as which they are based. If a greater degree of certainty is required that the wave uprush predicted by the empirical methods will not be exceeded (e.g., for management of *flooding hazard* shorelines on the Great Lakes and the St. Lawrence River System), an upper-bound limit of the "accepted" methods can be used. Other factors such as local bathymetry (e.g. offshore bars and composite slopes), berms in front of structures, wind speed, oblique wave attack, etc., may also change the magnitude of the wave uprush and overtopping.

Coastal protection structures which are overtopped by waves are common and their proper use is considered acceptable practice. Design of structures that preclude overtopping may be cost prohibitive, so that structures may have to be lower and some risk accepted. Structures which are subject to overtopping must be carefully designed to withstand the forces and scouring effects of the overtopping water. Special attention must be given to the details of the crest and backside of the structure. Also, proper provisions for the drainage of the overtopping water must be specifically incorporated into the design of the shoreline structure to prevent upland flooding and ponding.

The procedures outlined for the various "accepted" methods must be followed closely and they should not be extrapolated much beyond the tested conditions. It may be appropriate when a methodology is used that the proponent provide a brief summary of how the methodology was derived and why it is applicable to the situation under study.

The guidance provided in this report should not be the only information used for final design. Each final design must be site specific and should be carried out by a qualified coastal engineer. Physical model testing should be used when justified by the scope of the project.

DEFINITIONS, UNITS AND SYMBOLS

Definitions

100 year flood level - the peak stillwater level due to the combined occurrences of mean monthly lake levels and wind setup having a total probability of 1% of being equalled or exceeded during any year. In connecting channels and the St. Lawrence River the 100 year flood level is the peak instantaneous stillwater level that is equalled or exceeded in 1% of all years.

Approach slope - the slope of the lakebed or nearshore lakeward of the shoreline or structure slope. The approach slope is measured perpendicular (i.e., at right angle) to the shoreline.

Armour stone - a relatively large quarried stone or fieldstone that is of nearly uniform size and usually large enough to require individual placement. It is commonly used as primary wave protection.

Bathymetry - the measurement of depths of water in oceans, seas, and lakes; also information derived from such measurements.

Breaking wave - a progressive wave in which the wave crest spills, curls over and plunges, surges, or collapses; syn. breaker. The waves breaking process is an important factor for uprush on structures. Wave uprush is different for non-breaking waves, breaking waves and broken waves.

Breakwater - a structure protecting a shore area, harbour, anchorage or basin from waves.

Bulkhead - a structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

Composite slope - when a structure is composed of two or more different slopes, it is called a composite slope structure.

Deep water - water so deep that surface waves are little affected by the lake bottom. Generally, water deeper than one-half the surface wavelength is considered deep water.

Deep-water waves - surface waves travelling in water with a depth greater than 1/2 of the wavelength.

Depth of breaking - the stillwater depth at the point where the wave breaks.

Embankment - an artificial bank such as a mound or dyke, generally built to hold back water or to carry a roadway.

Freeboard - the additional height of a structure above design high water level to prevent overflow. Also, at a given time, the vertical distance between the water level and the top of the structure.

Hindcast waves - the use of historic synoptic wind charts or wind speed and direction measurements to calculate the wave.

Irregular waves - waves as they naturally occur in the Great Lakes - comprised of a combination of various waves heights and periods.

Monochromatic waves - a wave train, generated in the laboratory basin, with the same wavelengths and periods; regular waves.

Other water related hazards - water associated phenomena acting on shoreline areas other than flooding and wave uprush. This includes, but is not limited to, wave spray, ponding due to wave overtopping, ice accumulation, and ice forces.

Pile, sheet - a pile with a generally slender flat cross section to be driven into the ground or lake bed and meshed or interlocked with like members to form a diaphragm, wall or bulkhead.

Plunging breaker - a wave breaking on a shore, over a reef, etc. Crest curls over air pocket; breaking is usually with a crash. Smooth splash-up usually follows.

Refraction (of water waves) - (1) the processes by which the direction of a wave moving in shallow water, at an angle to the contours, is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) The bending of wave crests by currents.

Relative freeboard - the ratio of the structure freeboard to the wave height.

Revetment - a facing stone, concrete, etc. built to protect a scarp, embankment, or shore structure against erosion by wave action or currents.

Rip-rap - a layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

Rubble - (1) loose angular waterworn stones along a beach. (2) Rough, irregular fragments of broken rock.

Rubble-mound - a mound of random-shaped and random-placed stones protected with a cover layer of selected stones or specially shaped concrete armour units. (Armour units in primary cover layer may be placed in orderly manner or dumped at random).

Static water level - elevation of surface of the water in the absence of wind, wave, atmospheric and/or tidal disturbances.

Storm surge - a rise above the normal static water level on the open coast due to the action of wind stress on the water surface.

Seawall - a structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. See also Bulkhead.

Shallow water - water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water or depths less than 1/25 the wavelength as very shallow water and between ½ and 1/25 as transitional water.

Stillwater level - the elevation that the surface of the water would assume if wind setup and other atmospheric and/or tidal displacements of the water body occurred, but wave action was absent.

Transmission coefficient - the ratio of the transmitted wave height (wave height on the lee side of a structure, such as a breakwater) to the incident wave height.

Wave height - the vertical distance between a crest and the proceeding trough.

Wavelength - the horizontal distance between similar points on two successive waves measured perpendicular to the crest.

Wave overtopping - passing of water over the top of a structure as a result of wave uprush or wave action. Generally overtopping does not mean some spray or splash due to a combination of splitting of water by impact or wind action but describes overrun by clear water ("green water").

Wave period - the time for a wave to crest to traverse a distance equal to one wavelength. The time for two successive wave crests to pass a fixed point.

Wave setup - super-elevation of the water surface, averaged over time shoreward of the breaking point, over normal surge elevation due to onshore mass transport of the water by wave action alone.

Wave uprush/runup - the vertical height above the still-water level to which water, from an incident wave, will rush up to on a shoreline or shoreline structure.

Wind waves - (1) waves being formed and built up by the wind. (2) Loosely, any wave generated by wind.

Units

The units of measurement comply with the SI-system throughout and all formulae postulate that values are inserted using base SI-units. In the SI-system, all parameters such as length, volume, mass, and force are to be used in a formula with the value given in its base unit.

Mass is the term used to specify the quantity of matter contained in material objects. The base unit is the kilogram (kg).

Density is the unit mass, that is, the mass per unit volume. The base unit is kg/m^3.

Force is given in the base unit newton (N).

Weight is a measure of the gravitational force acting on a material object at a specified location. The base unit is the newton (N). The standard gravity at sea level is 9.81 m/s^2.

Unit weight is the gravitational force per unit volume. The base units are N/m^3. The unit weight is determined as the product of density and gravity.

Stress and pressure are expressed as the force per unit area. The base units are N/m^2 or Pa (pascal).

Volume is expressed in m^3 (also, 1.0 m^3 = 1000 litres (l)).

Symbols

2D = two-dimensional

3D = three-dimensional

b_w = berm width (m)

D_{50} = median stone diameter (m)

d_b = water depth at breaker point (m)

d_B = depth of berm below SWL (m)

d_s = water depth at toe of structure (m)

F = freeboard of structure from stillwater level (m)

$F^{/}$ = dimensionless freeboard parameter
= $F/(H_{mo}^2 L_p)^{1/3}$ (after Ahrens and Heimbaugh 1988b)

$F^{//}$ = dimensionless freeboard parameter
= $F \cot \alpha / (H L_o)^{1/2}$ (after Pilarczyk 1990)

$F^{///}$ = dimensionless freeboard parameter
= $F/H_s * F^*$ (after Bradbury 1988)

F^* = dimensionless freeboard parameter
= $F/(T_z(gH_s)^{1/2})$ (after Owen 1982)

g = acceleration due to gravity (m/s^2)

H = wave height (m)

H_b = breaking wave height (m)

H_{ds}	=	wave height at depth d_s (m)
H_{max}	=	maximum wave height (m)
H_{mo}	=	four times standard deviation of sea surface elevations (m). In deep water, $H_{mo} \approx H_s$
H_0	=	deep-water significant wave height (m)
H_o'	=	deep-water wave height equivalent to the observed shallow-water wave height if unaffected by refraction and friction (m)
	=	H/K_s
	=	$H_0 K_f K_R$
H_s	=	significant wave height, or average height of the highest one-third individual waves in record (m)
H_{rms}	=	the equivalent root-mean-square wave height (m)
H_2	=	on average, 2% of all waves will exceed this wave height (m)
H_{10}	=	on average, 10% of all waves will exceed this wave height (m)
H_m	=	mean wave height (m) (also $\overline{H}$)
K_f	=	wave height reduction factor from friction (-)
K_R	=	refraction coefficient (-)
K_s	=	shoaling coefficient (-)
K_t	=	transmission coefficient (-)
k	=	wave number (-)
	=	$2\pi/L$
k_α	=	wave direction modification factor
L	=	wavelength (m)
L_m	=	Airy wavelength (m) using T_m
L_o	=	deep-water wavelength (m)
L_p	=	Airy wavelength (m) using T_p

m	=	approach or nearshore slope (rise:run = m:1)
N	=	proportion of overtopping waves to the number of incident waves
P	=	dimensionless measure of permeability (-)
Q	=	average overtopping rate ($m^3/s \bullet m$)
$Q^{/}$	=	dimensionless overtopping rate
	=	$Q/(gH_{mo}{}^3)^{1/2}$ (after Ahrens and Heimbaugh 1988b)
$Q^{//}$	=	dimensionless overtopping rate
	=	$QT(\cot\alpha)^{1/2}/0.1HL_o$ (after Pilarczyk 1990)
Q^*	=	dimensionless overtopping rate
	=	$Q/(T_z g H_s)$ (after Owen 1982)
q	=	overtopping rate per wave (m^3/wave$\bullet$m)
q_f	=	maximum permissable discharge (prior to failure) associated with a characteristic wave and not the time-averaged discharge ($m^3/s \bullet m$)
R	=	wave uprush or runup (m)
R_m	=	average uprush (also $\bar{R}$) (m)
R_2	=	on average, 2% of all uprush values will exceed this uprush level
R_h	=	wave uprush Hunt's method (1959) (m)
R_{max}	=	maximum uprush (m)
R_s	=	significant uprush, or the average height of the highest one-third of all individual uprush heights on record (m)
r	=	rough slope reduction factor (-)
SWL	=	stillwater level (m)
T	=	wave period (s)
T_p	=	wave period corresponding to frequency at highest peak of energy density spectrum (s)
T_m	=	average wave period (also $\bar{T}$) (s)
T_z	=	mean zero crossing wave period (s)

U_{10} = wind speed (m/s) at 10 m above surface

W_f = wind speed coefficient

α = angle of structure backslope (°)

β = angle of approaching waves to the shoreline (°)

Δ = relative density

Δ_r = roughness height (m)

δ_o = magnitude of superelevation of the wave midpoint above the mean water level (m)

θ = angle of the front slope of the structure or shoreline above the horizontal (°)

θ_r = angle of the front slope of the structure or shoreline above the horizontal (radians)

ρ = mass density of water (kg/m^3)

ξ = surf similarity parameter or Iribarren's Number (-)
= $\tan\theta / (H_s / L_o)^{1/2}$

Subscripts

$[\ ]_c$ = critical value

$[\ ]_{nb}$ = non-breaking wave value

$[\ ]_p$ = plunging wave value

$[\ ]_{ss}$ = smooth slope value

$[\ ]_r$ = rough slope value

$[\ ]_t$ = transition value

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1.0 INTRODUCTION

Delineation of the *flooding hazard* limit, along the shorelines of the Great Lakes - St. Lawrence River System (Provincial Policy Statement, Policy 3.1: Public Health and Safety) is based on the combined influence of the 100 year flood level plus an allowance for wave uprush and other water related hazards. This report provides direction on the determination of wave uprush and wave overtopping.

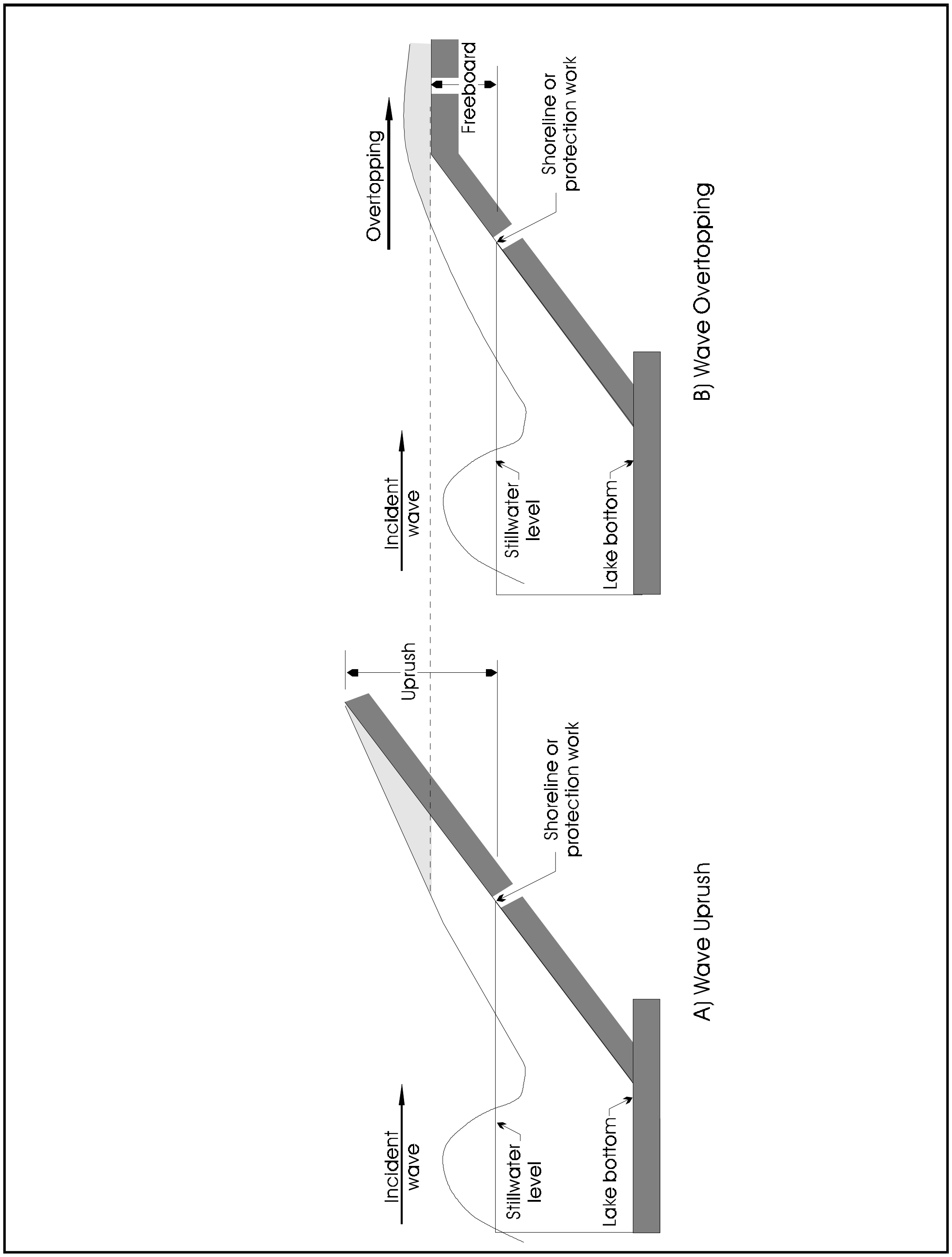
Wave uprush is typically defined as the vertical height above the stillwater level to which water, from an incident wave, will rush up to on a shoreline or shoreline structure (see Figure 1a). Because most uprush data are empirical, based on laboratory tests, the elevations measured (relative to the stillwater level) include both wave setup and wave uprush automatically. As such, the available engineering guidance generally includes wave setup in the uprush predictions in an inseparable way.

In the event that the top elevation of the existing natural shoreline or shoreline structure is lower than the limit of wave uprush, wave overtopping will occur (see Figure 1b). Wave overtopping is defined as the passing of water over the top of a shoreline or structure as a result of wave uprush or wave action. Generally, overtopping does not mean some spray or splash due to a combination of splitting of water by impact or wind action but describes overrun by clear water ("green water").

When wave overtopping occurs, water will pass over the top of the bank or shoreline structure to the backshore area. Depending on the backshore land use, the overtopping waves can be a risk to life and property and a hazard to shoreline users. If adequate drainage facilities are not provided, the backshore area will be subject to flooding and ponding. Flooding can threaten the emergency access to or egress from the area. Property can be damaged due to the flooding and/or the impact of the flowing water. The flow of the overtopping water can also cause scour at the top of the shoreline structure. Continued erosion behind the structure, due to the overtopping waves, can cause a structural failure of the protection work.
Shoreline protection structures that permit some wave overtopping are not uncommon and their proper use is considered acceptable practice (U.S. Army Corps of Engineers 1984; Goda 1985; Bruun 1985; Pilarczyk 1990). Initial costs of structures that preclude overtopping may be prohibitive and depending on the proposed land use, in the lee of the structure, a non-overtopping structure may not be necessary, nor required. Structures may have to be lower and some risk accepted. Bruun (1985) notes that "every 'practical' break or seawall is 'overtopped' by wave action once in a while mostly at low frequency of occurrence". If the shoreline or structure height is lowered, the amount of wave overtopping increases. As overtopping increases, the level of risk increases. The level of acceptable risk depends on the potential for loss of life, the proposed land use, the value of potential property damage, the environmental impacts, and the ability to undertake repairs or reconstruction if the structure is damaged.

Structures which are subject to overtopping must be carefully designed to withstand the forces and scouring effects of the overtopping water. Special attention must be given to the details of the crest and backside of the structure. Also, proper provisions for the drainage of the overtopping water must be specifically incorporated into the design of the shoreline structure to prevent upland flooding and ponding.

Figure 1: Definition Sketch for Wave Uprush and Wave Overtopping



1.1 Controlling Parameters

As waves move from the deep water offshore, into the shallower nearshore region, their direction changes so that the wave crests tend to align themselves more parallel to the shore. This is known as refraction. The degree of refraction, or change in angle, depends on the wavelength and water depth. Refraction may increase or decrease the wave height at shore locations (by focusing the waves together or by spreading them out) as well as change the wave direction. In addition to wave refraction effects, the shape of the wave changes significantly as the wave moves into shallow water. Generally, the length of the wave decreases and the height increases. This process is known as shoaling. Some reduction in the wave height may also result from energy loss caused by the roughness of the lake bottom in very shallow water.

Under certain conditions (such as for steep structure slope, long period and small amplitude waves) waves may reach the structure (or beach) without breaking. In this case, the waves can be assumed to be reflected totally like a standing wave, and the uprush is then directly related to the wave amplitude.

If waves break on the beach (or structure) due to instability caused by decreasing depths, the incident wave energy will be distributed in wave uprush, wave reflection, wave breaking, slope roughness losses, and losses due to permeability. Wave uprush is a function of the following:

- the incident wave climate (wave height, H, and wave period T (and hence wavelength, L));
- the beach (or structure) slope ($\tan\theta$);
- the approach or lake bottom slope (m);
- the water depth at toe of the structure slope or beach slope (d_s); and
- surface roughness and structure permeability (Δ_r and P).

Using these parameters, wave uprush may be expressed in dimensionless form as follows:

$$\frac{R}{H} = f\left(\frac{H}{L}, \tan\theta, m, \frac{d_s}{H}, \frac{\Delta_r}{H}, P\right)$$

where R is the wave uprush, H is the incident wave height, L is the wavelength, $\tan\theta$ is beach (or structure) slope, m is the approach or lake bottom slope, d_s is the water depth at toe of the structure slope or beach slope, Δ_r is a roughness height and P is a dimensionless measure of permeability.

Figure 2 outlines some of the conditions and variables in the wave uprush and wave overtopping process. Other factors such as the local bathymetry (e.g. offshore bars and composite slopes), berms in front of structures, oblique wave attack, etc., may also change the magnitude of the uprush.

Figure 3 shows the relationship between the vertical uprush value, R , and the horizontal offset for wave uprush. The same geometrical consideration could be used to obtain the horizontal component of wave uprush for other simple slope configurations.

Figure 2: Definition Sketch for Some Variables Applicable to Wave Uprush and Overtopping

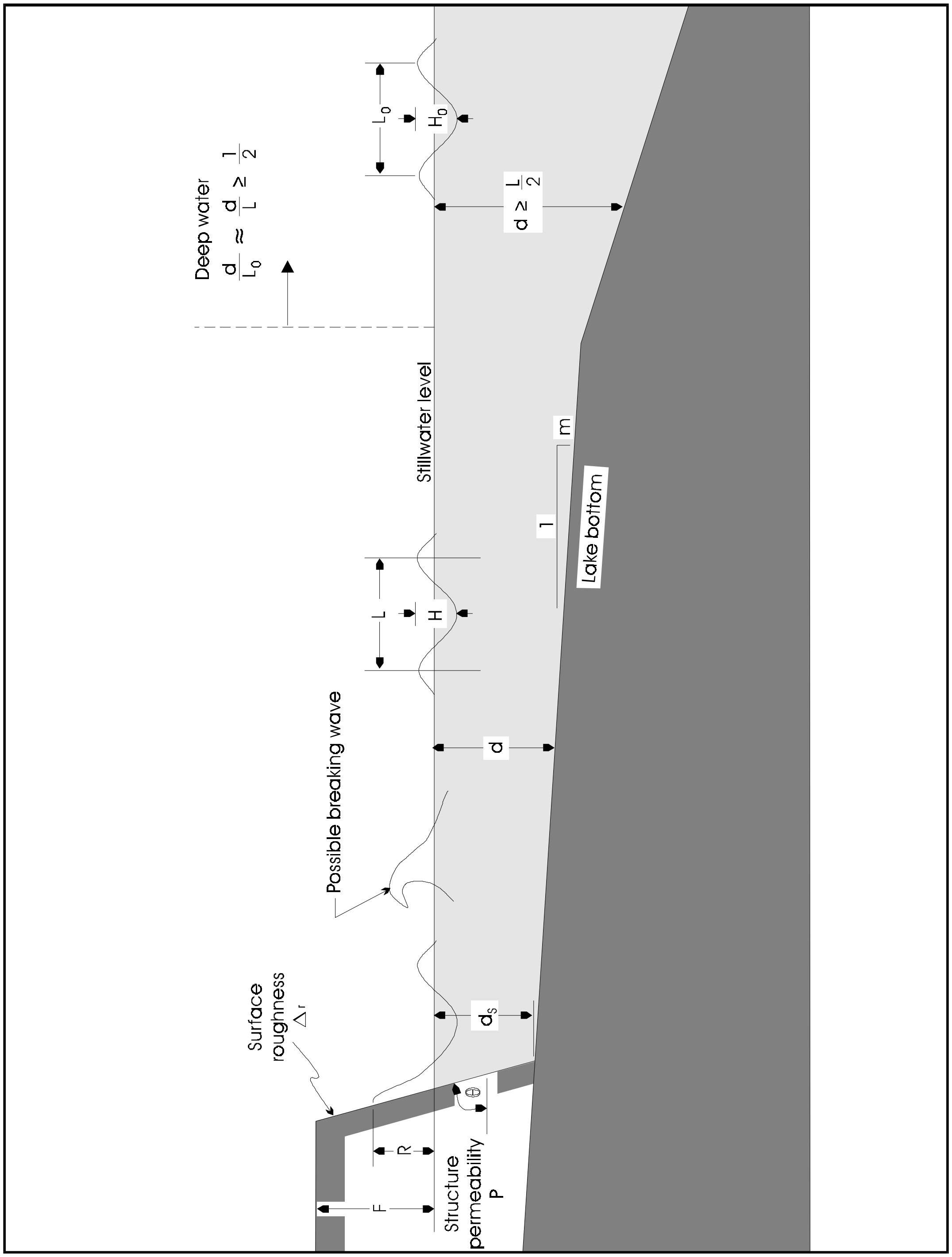
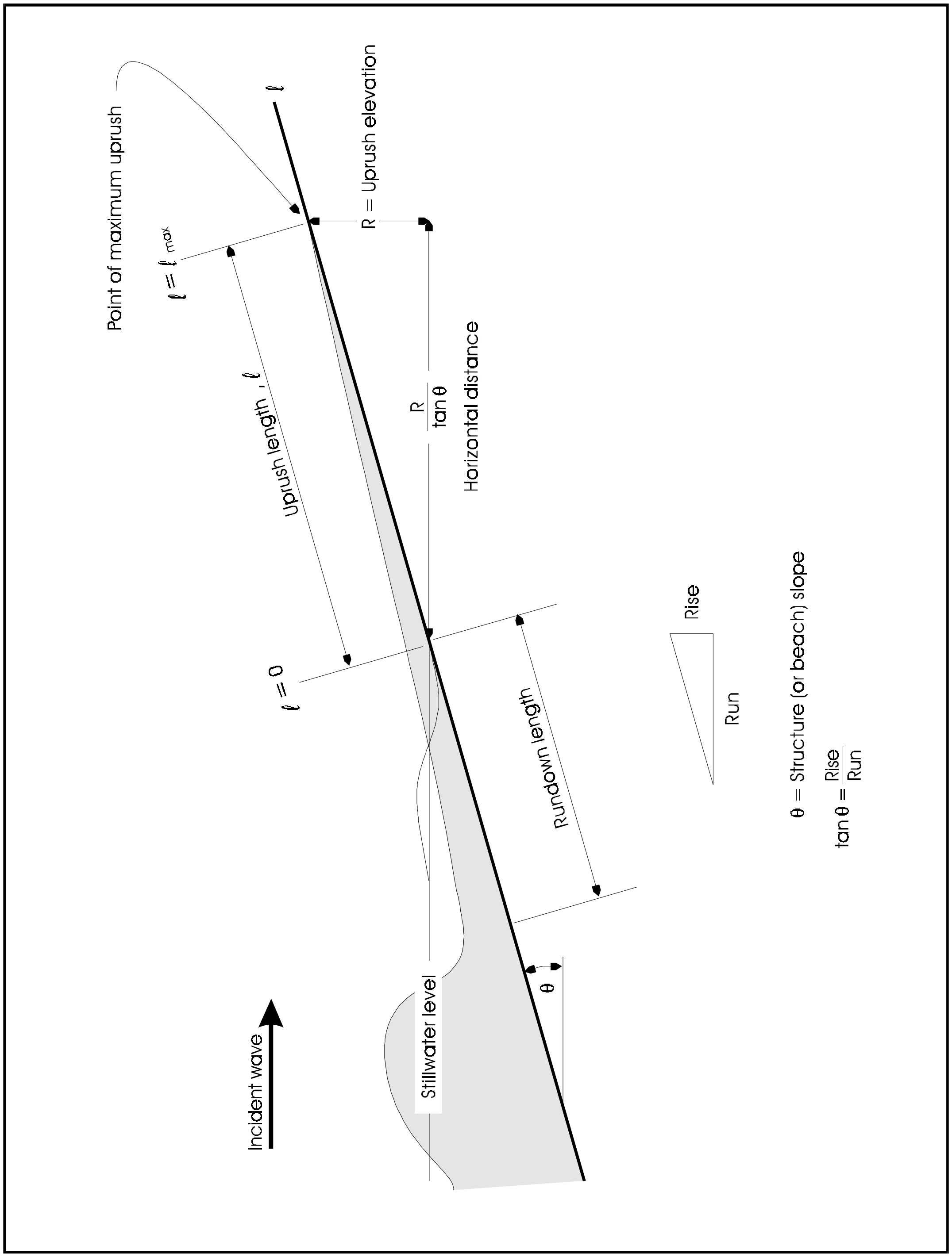


Figure 3: Uprush Characteristics for Wave Breaking on Slope



Relative wave uprush (*R/H*) was found to be a function of the surf similarity parameter (or Iribarren Number) ξ , a parameter representing the wave-structure interaction processes:

$$\xi = \frac{\tan\theta}{\sqrt{H/L_o}}$$

where H is the wave height, L_o is the deep-water wavelength and $\tan\theta$ represents the structure (or beach) slope. H/L_o is the wave steepness. Figure 4 (from Pilarczyk 1990) provides an example plot of relative wave uprush, *R/H* (and downrush) versus the surf similarity parameter ξ.

It may be seen from Figure 4, for example, that relative uprush increases, as ξ increases, for small values of ξ and becomes approximately constant or slightly decreases for $\xi > \approx 3.5$. These regions on the *R/H* vs ξ graph are called the 'breaking wave', the 'transition' and the 'non-breaking' wave regions.

Typically, in the breaking wave region

$$\frac{R}{H} \approx a\,\xi^{b} \qquad \xi < \approx 3$$

where a and b are empirical coefficients. In the non-breaking region, the relative uprush was found to be a function of the slope

$$\frac{R}{H} \approx c\left(\frac{\pi}{2\theta}\right)^{1/4} \qquad \xi > \approx 4$$

where c is a constant.

Also, it may be seen from Figure 4 that:

1. Roughness and permeability play a major role in the uprush processes.

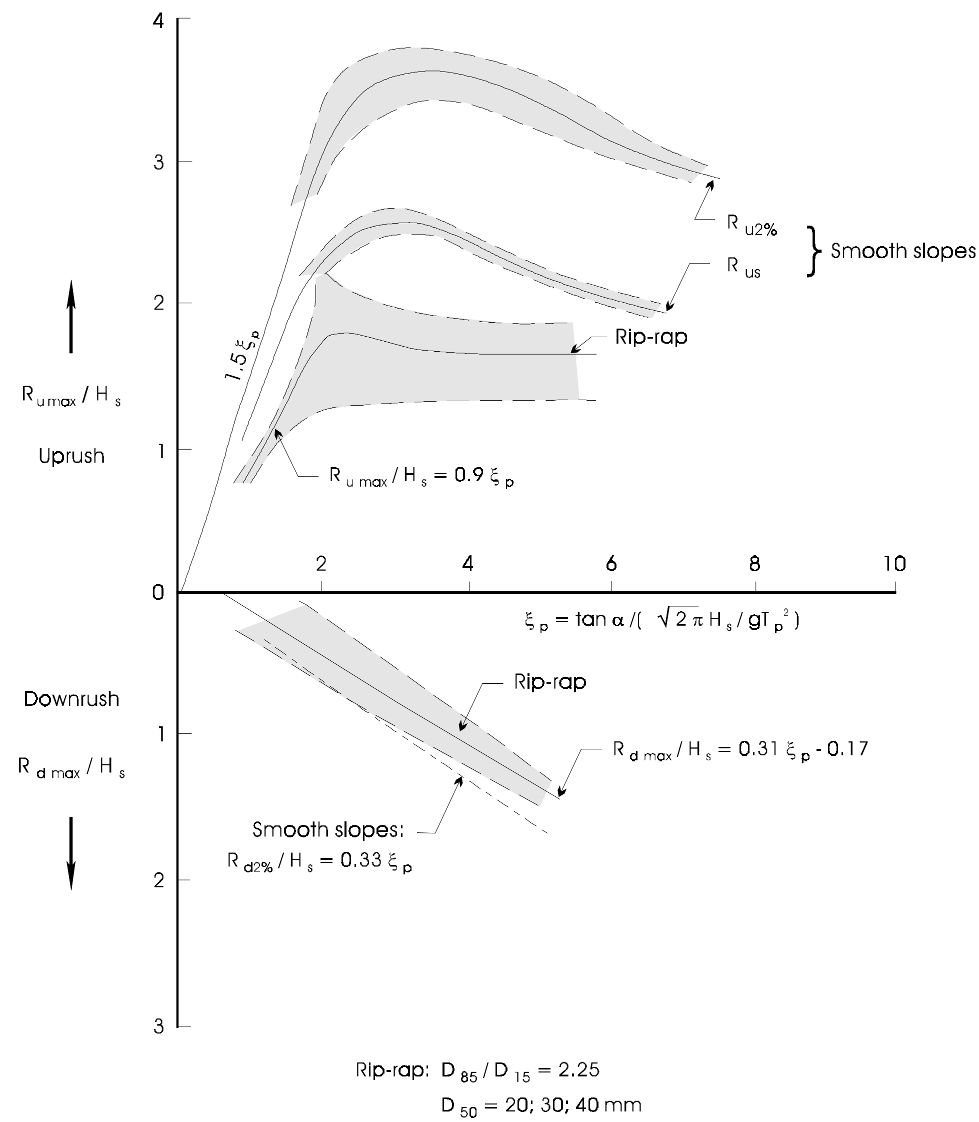
2. It is difficult to develop a generic predictor for uprush due to the different flow regimes and complex wave-structure interaction processes.

3. Irregular wave uprush data brings about the problem of the definition of characteristic wave climate parameters (to represent the incident wave train) in the calculation procedures.

Further complications in applying uprush predictors include the definition of the slope (e.g. some sites have composite slope situations) as well as the choice of the nearshore wave transformation procedure. All of the above show the need for site-specific evaluations of wave uprush and overtopping of shoreline protection schemes by a competent coastal engineer, using updated methodology(ies) as well as physical model testing, if warranted.

When the height of the natural shoreline, or of the shoreline structure, above the stillwater level is less than the limit of uprush, wave overtopping occurs. The distance between the stillwater level and the top of the structure is known as the freeboard, F (see Figure 1b). Thus the basic parameters controlling wave overtopping are essentially those affecting wave uprush and are shown in Figure 2.

Figure 4: Uprush and Downrush Functions for Irregular Waves (after Pilarczyk 1990)



The commonly accepted measures of overtopping are the mean overtopping discharge, *Q*, and the proportion of overtopping waves to the total number of incident waves, sometimes known as *N*. The proportion of overtopping waves, *N*, is not a particularly good measure of overtopping, and cannot be used directly in design. The mean overtopping discharge, *Q*, is of much greater use, and many researchers have attempted to produce prediction methods to calculate *Q* for a variety of design conditions. It should be noted that *Q* is usually given in terms of mean discharge per unit length of shoreline or structure, e.g. $m^3/s{\bullet}m$, or $l/s{\bullet}m$ ($1000\ l = 1\ m^3$). A volume of water per wave, *q*, may also be defined for a unit length of shoreline, e.g. $m^3/wave{\bullet}m$.

1.2 Shoreline Structures

There are innumerable types and configurations of shoreline structures along the Great Lakes - St. Lawrence River system. As noted earlier, there is no generic methodology for predicting wave uprush and overtopping. However, there are methods for a few types of structures with simple profiles. Methods for wave overtopping prediction are the most limited. For wave overtopping computation, only two basic shoreline structures will be considered:

- vertical seawalls; and
- sloping armour stone revetments.

Schematic profiles of a typical vertical seawall and a typical sloping armour stone revetment are shown in Figure 5. These structures are common along the Great Lakes - St. Lawrence River system.

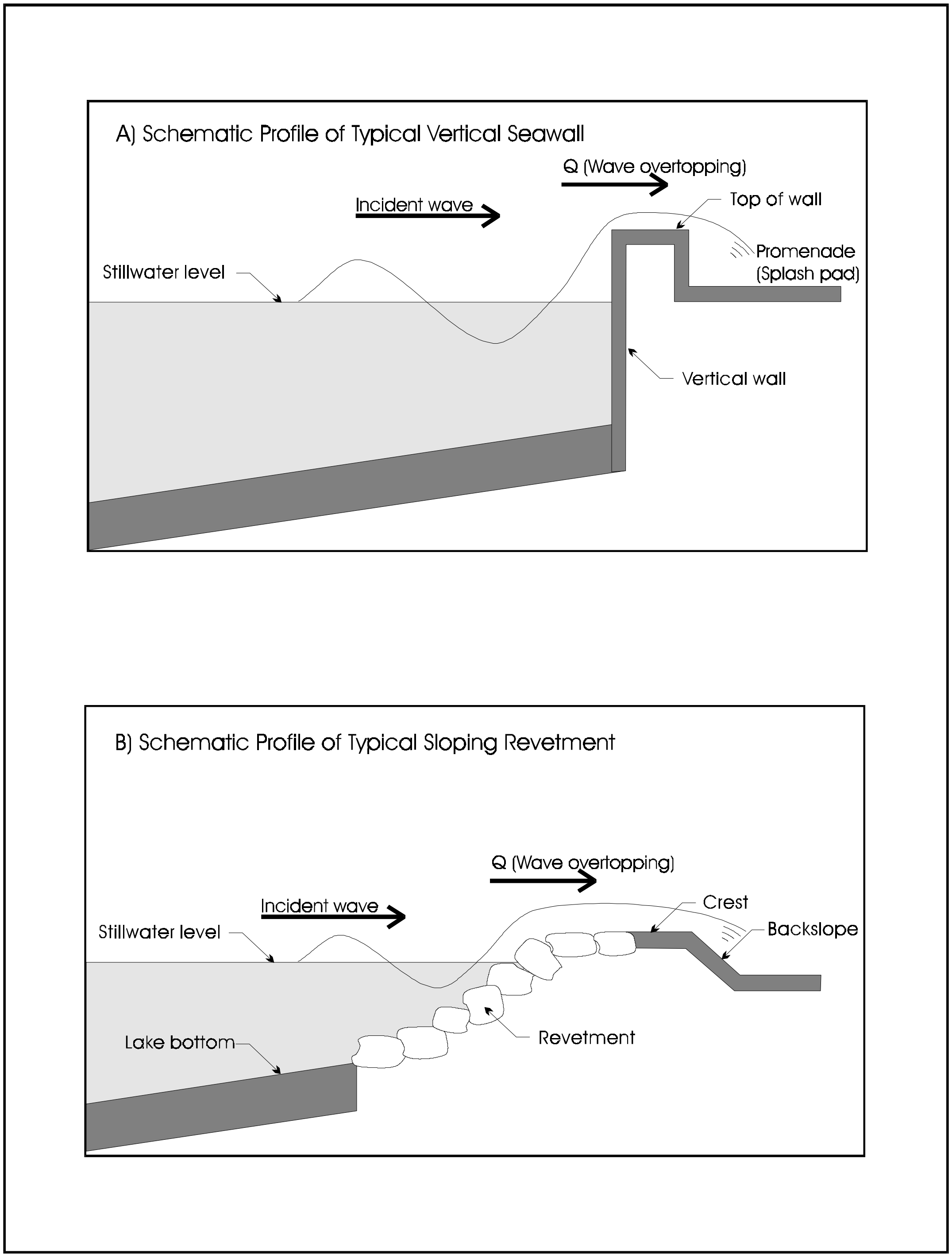
The vertical seawall is simply a vertical wall extending up from the lake bed (the bed being below water) to a distance above the stillwater level (see Figure 5a). It is often constructed of reinforced concrete, large concrete blocks or steel sheet pile. The sloping revetment structure can be either an embankment type of structure protected by rip-rap or armour stone (see Figure 5b), or it can be a composite structure where a vertical wall is fronted by a mound of armour stone (not submerged) and the elevation of the crest of the vertical wall is not much greater than the elevation of the crest of the revetment. These composite structures will considered as revetments.

1.3 Scope of Report

The factors which influence wave uprush are identified. Definitions are provided along with a summary of the units and the notations or symbols which are used. A literature review of wave uprush and overtopping methodologies available to 1992 and 1994 respectively is presented in Sections 2 and 3 respectively. Section 4 outlines design practice including specific problems that may be encountered as well as design guidance. Typical examples of wave uprush and overtopping calculations for commonly found shoreline structures and beaches in the Great Lakes - St. Lawrence River System are provided in Section 5.

This report is intended to provide technical guidance in the review of wave uprush and overtopping levels along natural shorelines and shoreline structures. This guidance alone should not be used for final design. Each final design must be site specific and should be carried out by a qualified coastal engineer.

Figure 5: Typical Vertical Seawall and Sloping Revetment



2.0 WAVE UPRUSH

At present, the understanding of uprush processes is limited, and there seems to be no generic methodology for the prediction of the limit of wave uprush. Extensive data are available for monochromatic (regular) waves and smooth impermeable slopes (i.e., smooth asphalt or concrete slopes; also, saturated sand beaches are often assumed to be smooth slopes). These smooth slope data, when coupled with coefficients representing further analysis of the influence of several roughness elements (i.e., rip-rap, armour stone) and permeabilities (i.e., stone core versus earth core with geotextile filter), are then used for practical applications. Figure 6 shows the influence of roughness and permeability on wave uprush calculated using several methodologies. Recently, more data became available on the uprush of irregular waves, in larger laboratory flumes (e.g., see Figures 4 (after Pilarczyk 1990) and 7 (after Mase 1989)).

2.1 Literature Review of Uprush Processes

The methodology for predicting wave uprush on structure slopes includes the following approaches: theoretical, empirical and hydraulic modelling. Hydraulic modelling is discussed in Section 4.0. Other reviews of uprush methods are available in Horikawa (1978), Allsop et al. (1985a) and Walton et al. (1989a). The format of this literature review section generally follows the layout presented in the report by Allsop et al. (1985a).

a) Theoretical Approach

It is not possible to treat wave uprush theoretically when waves break on a gentle slope (Horikawa 1988), since the flow processes involved are highly complex. As a result, many researchers in the past have approached the uprush question theoretically mainly for non-breaking waves and steep structure slopes.

Also, theoretical treatments of the turbulent breaking processes, the energy dissipation processes on roughened structure surfaces, and the mixing processes within structures due to permeability, are all very difficult, and the methods to describe these processes are not yet ready for practical engineering use. For these reasons, only a review of theoretical uprush on smooth steep slopes under non-breaking wave conditions is presented in this section. Other reviews of the wave uprush theory may be found in Horikawa (1978), LeMéhauté et al. (1968), Meyer and Taylor (1972), Allsop et al. (1985a) and Walton et al. (1989a).

Figure 6: Influence of Roughness and Permeability on Wave Uprush

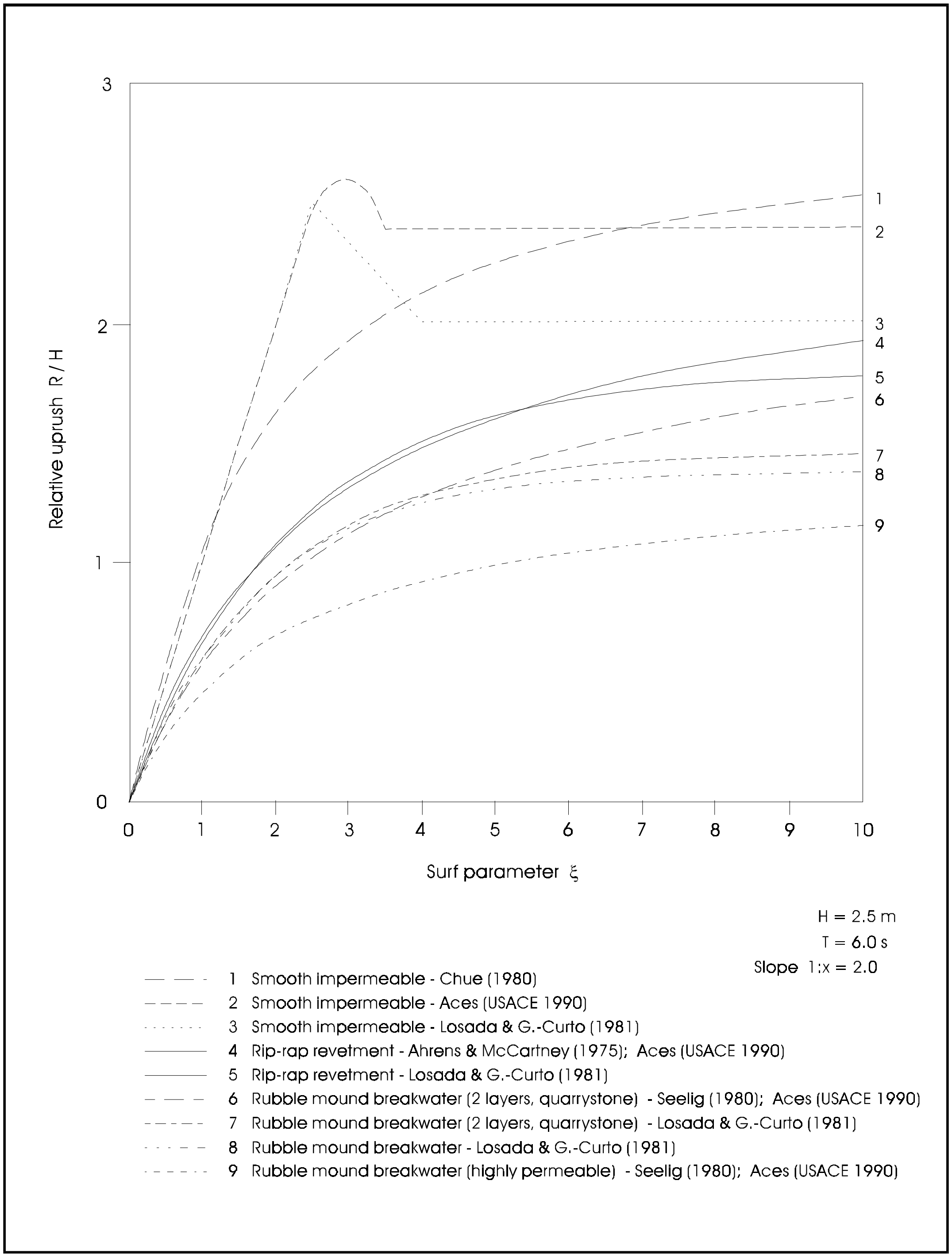
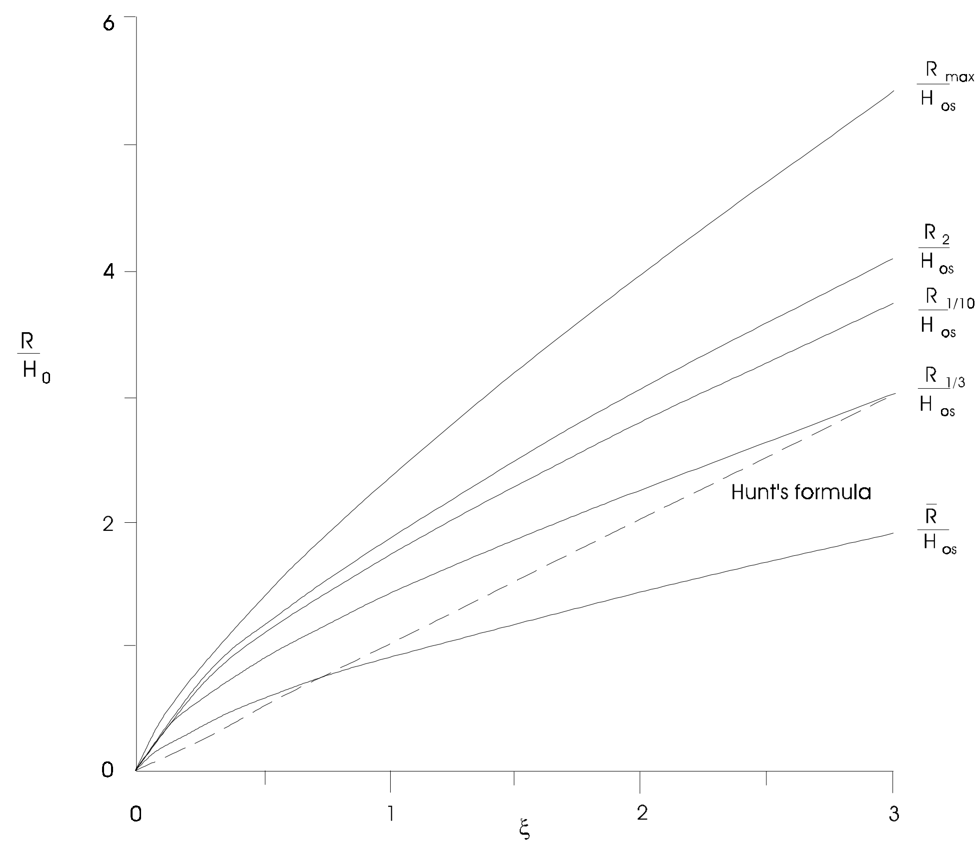


Figure 7: Influence of Irregular Waves on Wave Uprush (after Mase 1989)



Proposed uprush height formula for breaking random waves

. Miche (1951)

Miche (1951) proposed that for a structure with a steep slope, waves are completely reflected to form a standing wave pattern in front of the slope provided that the wave steepness is less than $(\sqrt{2\theta/\pi})(\sin^2\theta/\pi)$. Also, Miche (1951) developed a theoretical equation to predict wave uprush for non-breaking waves on a structure in deep water as

$$\frac{R}{H} = \sqrt{\frac{\pi}{2\theta}}$$

with θ in radians. This equation can be applied only to structures with steep slopes and with the wave height given at the toe of the structure (or in deep water). In shallower water, Walton et al. (1989a; 1989b) pointed out that the right-hand side of this equation should include the shoaling coefficient, K_s, as well as higher order terms (first proposed by LeMéhauté et al. 1968) as follows:

$$\frac{R}{H} = \left(\frac{1}{K_s}\right)\sqrt{\frac{\pi}{2\theta}} + O(H;L)$$

where $O\,(H;\,L)$ represents higher order nonlinear terms.

. LeMéhauté, Koh and Hwang (1968)

However, due to non-linear effects of standing waves, a super-elevation (i.e. the midpoint of the wave crest and wave trough does not lie at the mean water level) occurs in front of the structure. To take the superelevation into account, LeMéhauté et al., (1968) postulated that the relative uprush at a sloping structure (or steep beach) could be approximated by adding the super-elevation term δ_o to Miche's (1951) equation as:

$$\frac{R}{H} = \sqrt{\frac{\pi}{2\theta}} + \frac{\delta_o}{H}$$

The magnitude of the super-elevation was derived by Miche (1944) based on linear theory and can be expressed as:

$$\delta_o = \frac{kH^2}{2}\left(\frac{1}{\tanh(kd_s)}\right)\left(1 + \frac{3}{4\sinh^2(kd_s)} - \frac{1}{4\cosh^2(kd_s)}\right)$$

where k=2π/L. This approximation is valid provided that the slope is not very gentle.

. **Takada (1976)**

Takada (1976; see also Horikawa, 1978) proposed a similar expression as given by LeMéhauté et al. (1968) except that the superelevation term was derived by Sainflou (1928) based on the trochoidal wave theory. The magnitude of the superelevation was obtained by:

$$\delta_o = \frac{k H^2}{2} \coth(k d_s)$$

LeMéhauté et al. (1968) commented that the solutions for the superelevation by Sainflou (1928) and Miche (1944) do not satisfy the continuity equations exactly. They suggested that the solitary wave theory or conoidal wave theory should be used.

. **Nagai and Takada (1972)**

Nagai and Takada (1972) considered second, third and fourth order modifications and the incident wave steepness $H/L < \sqrt{2\theta/\pi}\,(\sin^2\theta/\pi)$ to predict the super-elevation, and derived another equation as:

$$\delta_o = \frac{kH^2}{8}\left[3\coth^3(k d_s) + \tanh(k d_s)\right]$$

. **Wallace (1963)**

Wallace (1963) investigated the reflection of a solitary wave from a vertical wall and obtained an approximate result as

$$\frac{R}{H} = 2.5$$

. **Keller and Keller (1965)**

Keller and Keller (1965) employed linear wave theory for waves on a plane slope with a horizontal sea bed and derived

$$\frac{R}{H} = \left[J_0^{\,2}\left(2\,\frac{\sqrt{\omega^2 d_s / g}}{\theta}\right) + J_1^{\,2}\left(2\,\frac{\sqrt{\omega^2 d_s / g}}{\theta}\right)\right]^{-1/2}$$

where J_0 and J_1 are Bessel functions of zero and first order and $\omega=2\pi/T$. According to Allsop et al.

(1985a), this equation can be asymptotically approximated by:

$$\frac{R}{H} = \pi \left[\frac{2}{\theta T} \sqrt{\frac{d_s}{g}} \right]^{1/2}$$

Van Dorn (1966) compared Keller and Keller (1965) equation with Savage's (1958) data and found good agreement for the smallest waves, but errors increased as the wave height became larger.

b) Empirical Approach

As noted previously, prediction of wave uprush was the subject of several laboratory studies. Physical modelling of uprush processes were carried out with both regular (monochromatic) and irregular waves, for both smooth and rough slopes. A review of these empirical attempts at describing wave uprush is presented, in chronological order for smooth slopes and regular waves, rough slopes and regular waves, and irregular waves.

i) Smooth Slopes - Regular Waves

The following discussion relates to the empirical analysis of wave uprush on smooth impermeable slopes due to regular (monochromatic) waves.

. Saville (1956, 1958)

Saville (1956) conducted a large number of two-dimensional (2D) wave flume tests investigating the effects of relative depth, relative wave steepness, structure slope, and beach slope. Tests of beach-slope effects were limited to structures sited on the horizontal wave tank bottom and on a 1:10 slope. Saville's uprush research with shore structures, including composite slopes (Saville, 1956; 1958) has been widely used, and was included since the 1960's in all versions of the U.S. Army manuals (U.S. Army Corps of Engineers 1966; 1973; 1977; 1984). A summary of his work was also given in Wiegel (1964).

. Savage (1958)

Savage (1958) performed a series of laboratory experiments to investigate the effect of roughness and permeability of the slope surface on uprush. The tested slopes ranged from 1:30 to vertical. The effect of roughness was tested by covering the smooth slopes with a single layer of material (e.g. sand, gravel, etc.). The effect of permeability was tested on slopes composed of the material to be tested.

. Hunt (1959)

Hunt (1959) conducted experiments to investigate uprush due to breaking and surging waves on uniform structure slopes and obtained the well-known formula for breaking waves (when tan θ ≤ √(H/T ^{2}) as:

$$\frac{R}{H} = \frac{2.3\tan\theta}{\sqrt{H/T^2}}$$

in English units. This equation is valid provided that the oncoming waves will not break before the structure, but will break on the structure slope. The wave height, *H*, should be measured at the toe of the structure. Since, the waves do not break before the structure, *H* can be assumed to be the deep-water wave height ($H \approx H_o$).

This equation has been widely used to predict the wave uprush in natural beaches. Allsop et al., (1985a) stated that Hunt's formula is "remarkably accurate for many natural beaches which are relatively smooth and for which $\xi < 2.5$". Also, Hunt (1959) suggested, based on the experiment data, that the uprush due to non-breaking surging waves could be approximated as:

$$\frac{R}{H} \approx 3$$

. U.S. Army Corps of Engineers (1973, 1984)

Based on the data collected by Saville (1956, 1958), Savage (1958) and others, the Shore Protection Manual (SPM; U.S. Army Corps of Engineers 1973, 1984) presented a set of uprush curves for smooth impermeable slopes and regular waves. Figure 8 shows a typical set of SPM uprush curves. The SPM notes that the uprush results predicted by the set of curves are likely less than actual uprush values due to scale roughness effect of the model experiments, and thus recommended using a scale effect correction factor.

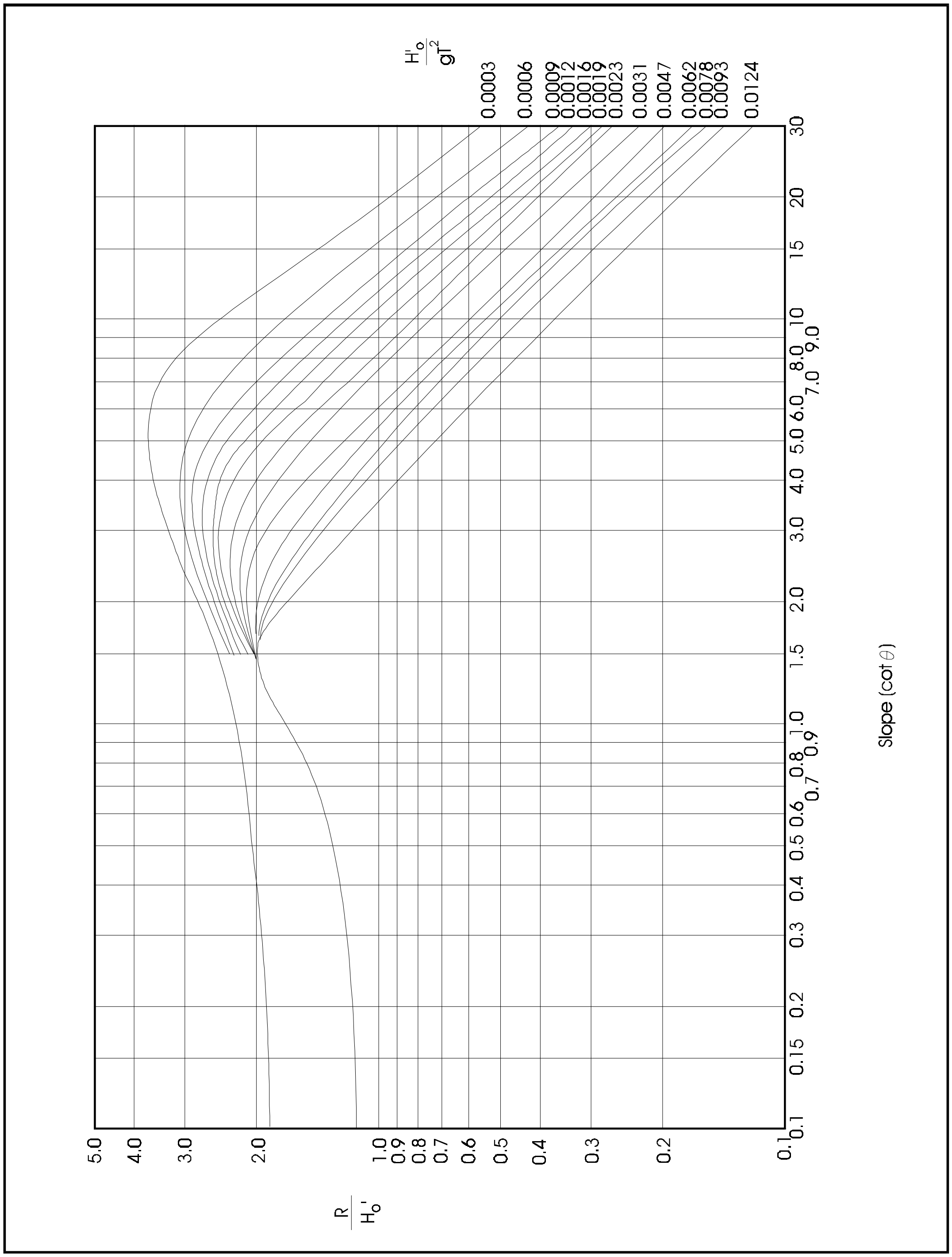
It is important to note here that the SPM uprush curves for slopes were reanalysed by Stoa (1978a, 1978b). However, his work, which supersedes the design curves of the SPM, was not included in the latest version of the manual (U.S. Army Corps of Engineers 1984).

. Battjes (1974a)

Battjes (1974a, 1974b) rewrote Hunt's (1959) equation in dimensionless form in terms of the surf similarity parameter (or Iribarren number), ξ, as:

$$\frac{R}{H} = \xi = \frac{\tan\theta}{\sqrt{\frac{H}{L_o}}}\ ; \qquad \xi < 2.3$$

Figure 8: Wave Uprush on Smooth, Impermeable Slopes when $d_s/H_o' \geqslant 3.0$ (after USACE 1984)



Singamsetti and Wind (1980) compared Hunt's equation with data of Saville (1956), Savage (1958) and Roos (1972) and found that for $0.2 < \xi < 2.0$, 84% of the measured data fell within ±10% confidence interval of the equation, for slopes from 1:3 to 1:40. Bruun (1985) found the results given by this equation to be accurate within 10% when compared with the experimental uprush data on slopes of 1:3 to 1:7.

Gunbak (1979), Losada and Gimenez-Curto (1981), and Sawaragi, Iwata and Kobayashi (1982) found that Hunt's formula (1959) is only valid when ξ is less than 2.5. Maximum uprush was found in the transition region of breaking to non-breaking conditions where $2.0 < \xi < 3$. They attributed this phenomenon to a resonance effect on the slope.

. Stoa (1978a, 1978b)

Stoa (1978a) reanalysed the laboratory test data from Saville (1956) and Savage (1958) and developed a wave uprush equation as:

$$\frac{R}{H_o'} = (\cot\theta)^{-1.04}\,(4.23)\,(10)^{2(q-1)}\left(\frac{H_o'}{gT^2}\right)^{q-1} ; \qquad \cot\theta \geq 2.0$$

where H_o' is the unrefracted wave height at deep water. The value of q can be estimated as a function of the structure slope using Figure 9 (Stoa 1978b), or by the following fitted equations:

$q = \exp\,[\,0.086 - 0.605\ \ln(\cot\theta)]$; for $2.0 \leq \cot\theta < 4.5$,
$q = \exp\,[-0.650 - 0.107\ \ln(\cot\theta)]$; for $4.5 \leq \cot\theta < 10$.

Stoa's (1978a) equation is valid for a structure on a flat bottom or structures on sloping bottoms provided that $d_s / H_o' > 3$, and assumed that the waves do not break before reaching the structures but completely break on the structure slope.

Stoa (1978b) presented a set of relative uprush curves for both breaking and non-breaking waves, as a revision to the SPM design uprush curves. The curves are valid for structure slopes fronted by horizontal and 1:10 bottom slopes for a range of d_s/H_o'. A set of curves from Stoa (1978b) along with the scale-effect correction factor figure and a flow chart are presented in Appendix A. Scale effect is discussed further in Section 4.0. Dewberry and Davis (1990) produced an uprush model for FEMA based on the method of Stoa (1974b). For additional details see Stoa (1979) in Section 2.1(b)(ii).

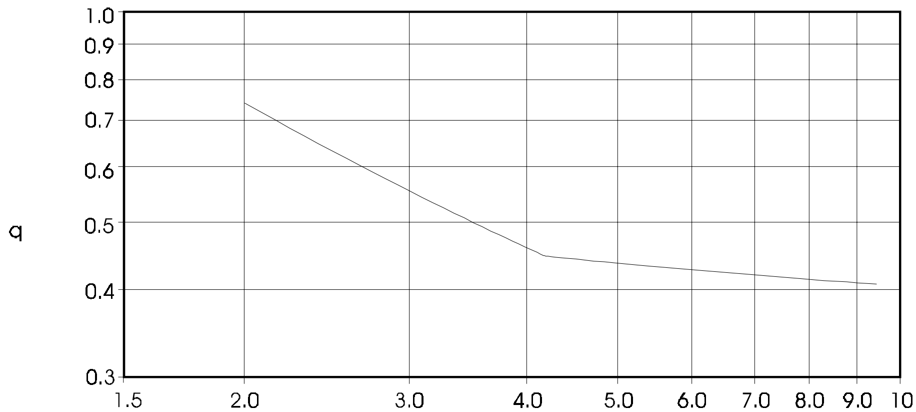
. Chue (1980)

Chue (1980) fitted an equation to the data from a wide range of wave and slope conditions (including breaking and non-breaking waves, steep and gentle slopes) and obtained:

$$\frac{R}{H} = 1.8\,(\,1 - 3.111\frac{H}{L_o})\,\xi_o\left(1 - \exp\left[-\sqrt{\frac{\pi}{2\theta}}\,(\,\frac{1}{\xi_o}\,)\right]\right)$$

where $\xi_o = \tan\theta\,/\,(\,H\,/\,L_o\,)^{0.4}$ (note the different steepness exponent used by Chue).

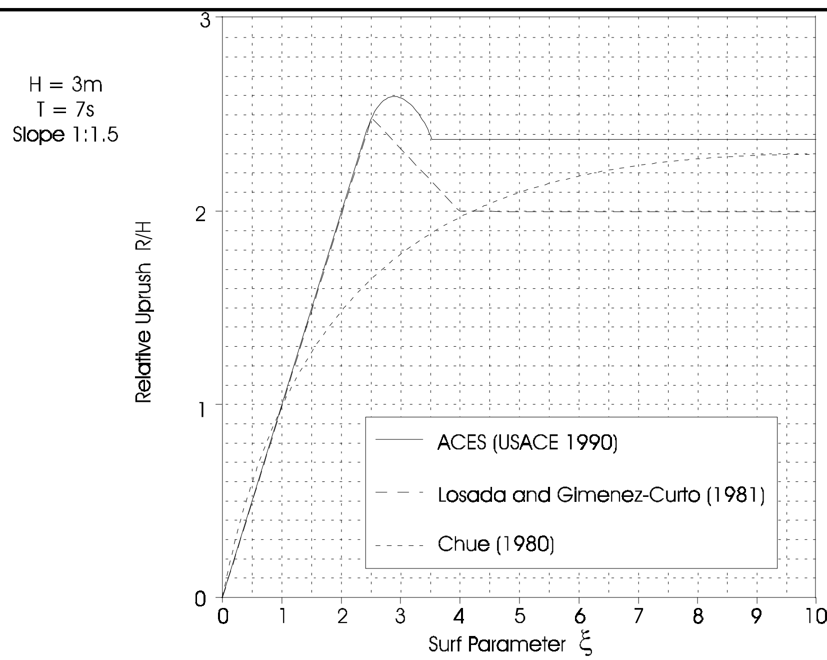
Figure 9: Values for q for Stoa (1978b)



Values of q for use in Stoa (1978b) equation

$$\frac{R}{H'_o} = (4.23)(\cot\theta)^{-1.04}[10^{2(q-1)}](\frac{H'_o}{gT^2})^{q-1}$$

Figure 10: Relative Uprush on Smooth Slopes



Chue's equation is also shown in Figure 10. Based on the data collected by Saville (1956) and Savage (1958), Chue observed that for non-breaking wave conditions, the relative uprush, *R/H*, decreases when wave steepness, H/L_o, increases; as indicated in his expression. Ahrens and Titus (1985) criticized that this trend is contrasted with Savage's (1958) data and the work of LeMéhauté et al. (1968).

. Losada and Gimenez-Curto (1981)

Working with Iribarren's number, Losada and Gimenez-Curto (1981) proposed three expressions to cover the entire range of breaking and non-breaking wave conditions for smooth slopes as (shown in Figure 10, H = 3 m and T = 7 s):

$R / H = \xi$; $\quad 0.0 < \xi < 2.5$ (breaking),
$R / H = 2.5 - (\xi-2.5)/3.0$; $\quad 2.5 < \xi < 4.0$ (transition),
$R / H = 2.0$; $\quad 4.0 < \xi$ (non-breaking).

. Ahrens and Titus (1985)

Based on Saville's and Savage's data, Ahrens and Titus (1985) also developed the following equations for smooth slopes and for the regions of breaking, non-breaking and transition:

$$\left[\frac{R}{H}\right]_p = 1.002\,\xi \quad or \quad \left[\frac{R}{H_o'}\right]_p = 0.967\,\xi\ ; \qquad \text{for } \xi \leq 2.0 \text{ (plunging)}$$

$$\left[\frac{R}{H}\right]_{nb} = 1.181\left(\frac{\pi}{2\theta}\right)^{0.375}\exp\left[3.187\left(\frac{\eta_c}{H}-0.5\right)^2\right]; \qquad \text{for } \xi \geq 3.5 \text{ (non-breaking)}$$

$$\left[\frac{R}{H}\right]_t = \left(\frac{\xi-2.0}{1.5}\right)\left[\frac{R}{H}\right]_{nb} + \left(\frac{3.5-\xi}{1.5}\right)\left[\frac{R}{H}\right]_p\ ; \qquad \text{for } 2.0 < \xi < 3.5 \text{ (transition).}$$

where η_c is the crest wave height above the stillwater level calculated from Dean's (1974) Stream Function Wave Theory. Walton et al., (1989a; 1989b) proposed a new empirical equation as a modification to the non-breaking uprush expression of Ahrens and Titus (1985) as follows:

$$\frac{R}{H} = 1.087\,\left(\sqrt{\frac{\pi}{2\theta}}\right) + 0.775G$$

for $\xi \geq 3.5$ (non-breaking), where G is Goda's nonlinearity parameter (Goda, 1985)

$$G = \frac{H/L}{\tanh^3(kd)}$$

. Horikawa (1988)

Horikawa (1988) summarised the wave uprush study results in Figure 11. It can be seen that Hunt's formula was used when $\xi < 2.3$, Miche's formula was used when $\xi > \sim 4.3$, and the equation proposed by LeMéhauté et al., (1968) was chosen in the region $2.3 < \xi < 4.3$.

. Walton and Ahrens (1989)

Walton and Ahrens (1989) indicated that Hunt's formula cannot be applied when the structure slope approaches vertical, i.e. $\xi \rightarrow \infty$. hence they proposed to modify Hunt's (1959) expression by substituting tan θ with sin θ.

Using linear wave theory, Miche (1951) developed a breaking criterion for the limiting non-breaking waves on smooth uniform slopes, extending to deep water as:

$$\left[\frac{\sin\theta}{\sqrt{H/L}}\right]_c = \sqrt{\frac{\pi}{K_s}}\left(\frac{\pi}{2\theta}\right)^{1/4}; \qquad \text{for} \quad \theta \le \frac{\pi}{4}$$

where $K_s = \sqrt{\dfrac{1}{2\, n \tanh(kd_s)}}$ and $n = \dfrac{1}{2} + \dfrac{kd_s}{\sinh(2kd_s)}$.

Based on a non-linear shallow water theory, Keller (1961) found a similar expression for the limiting non-breaking waves as:

$$\left[\frac{\theta}{\sqrt{H/L}}\right]_c = \sqrt{\frac{2\pi}{K_s}}\left(\frac{\pi}{2\theta}\right)^{1/4}$$

Using the breaking criteria of Miche (1951) and Keller (1961) to define the upper limit of wave steepness on a particular structure slope, they derived the upper limit of wave uprush under non-breaking wave conditions as:

$$\frac{R}{H} = \sqrt{2\pi}\left(\frac{\pi}{2\theta}\right)^{1/4}$$

Figure 12 shows the comparison of the laboratory data collected by Saville (1956) and Savage (1958) with the modified Hunt's formula and the derived upper limits for nine slopes ranging from vertical to 1:10. It should be noted that the equation above gives an upper limit of 2.5 for non-breaking waves on vertical wall.

Figure 11: Summary of Breaking and Swash of Regular Waves (after Horikawa 1988)

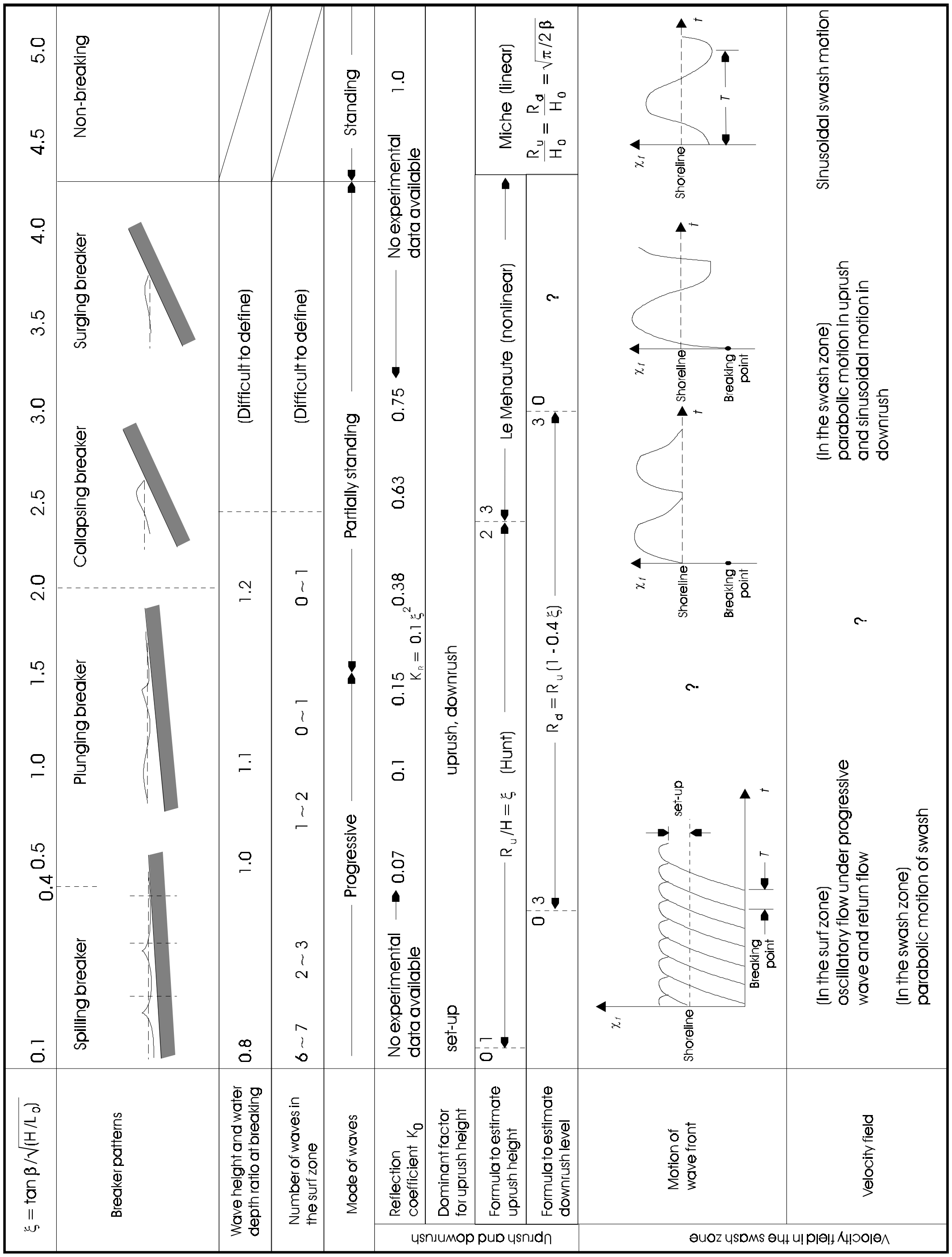
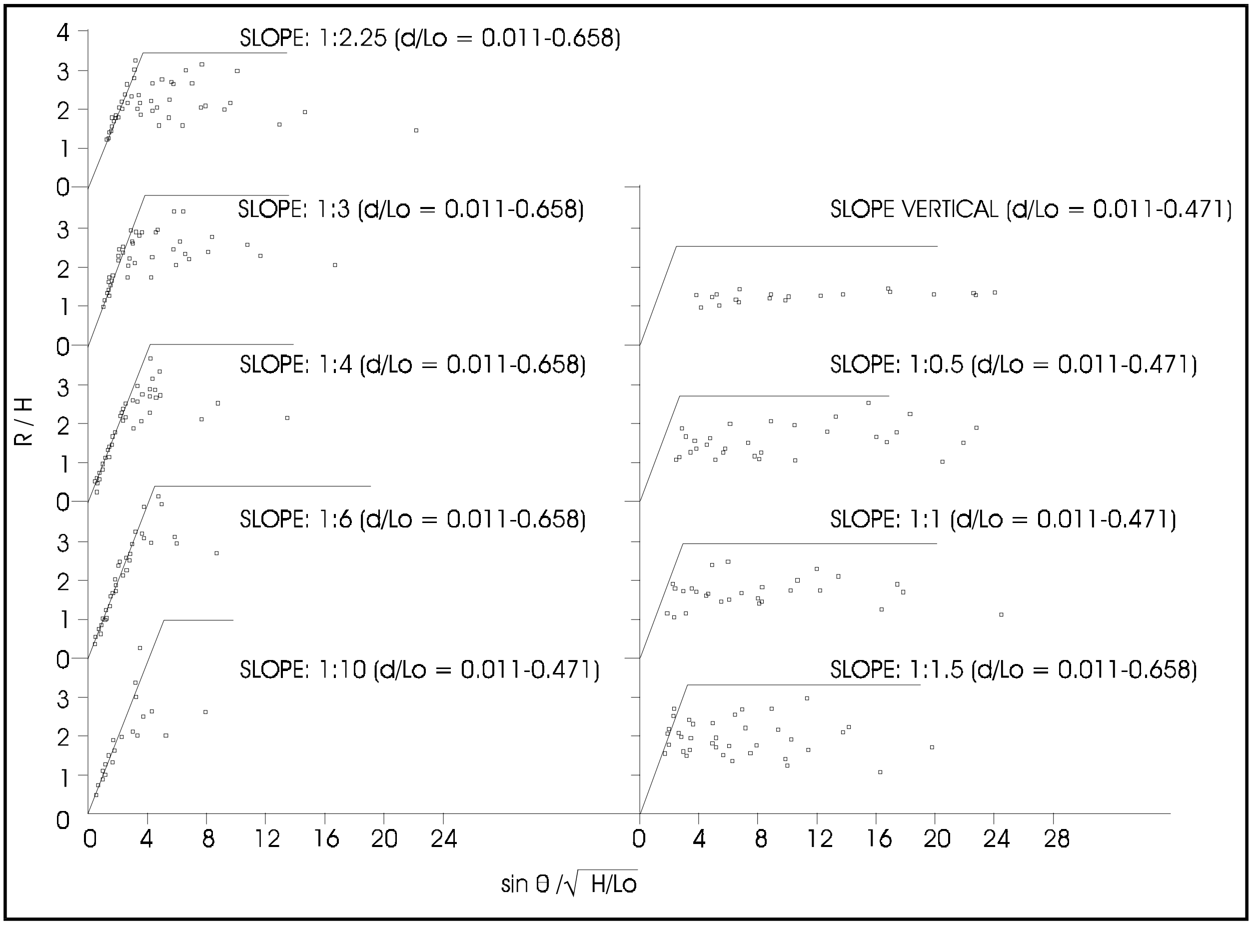


Figure 12: Relative Uprush versus Modified Surf Similarity Number (after Walton and Ahrens 1989)



. U.S. Army Corps of Engineers (1990)

In the software package Automated Coastal Engineering System (ACES), developed by the Coastal Engineering Research Center (U.S. Army Corps of Engineers 1990), the empirical equations used for predicting wave uprush on smooth, impermeable sloped structures are the same as those proposed by Ahrens and Titus (1985) except that the non-breaking wave uprush is estimated by the following equation (derived by Ahrens and Burke (1987), see also Walton et al. 1989a; 1989b):

$$\left[\frac{R}{H}\right]_{nb} = 1.087\sqrt{\frac{\pi}{2\theta}} + 0.775\frac{H/L}{\tanh^3(kd_s)}\ ; \qquad \text{for } \xi \geq 3.5 \text{ (non-breaking).}$$

The complete ACES curve is also shown in Figure 10 for H = 3 m, T = 7 s and cot θ = 1.5.

ii) Rough Slopes - Regular Waves

Wave uprush on rough slopes is less than uprush on smooth slopes. This concept lead to the development of a 'so called' reduction factor, *r*. This reduction factor was then applied to the uprush formulas developed for smooth slopes in order to obtain a uprush value for the comparable rough slope.

. Savage (1958)

Savage (1958) observed that *r* decreased with decreasing wave steepness and decreasing slopes. However, Savage's data was limited to sand covered slopes and could not be extended to other types of slope surfaces.

. U.S. Army (1973, 1984)

Based on many laboratory data by Saville (1956; 1958; 1959), Savage (1958) and others, the SPM has published design curves for uprush on rough slopes. Figure 13 (after USACE, 1984) shows that wave uprush is dependent on wave steepness, slope and slope material (roughness and permeability). Also, Figure 13 shows that uprush on rough slopes is quite a different process than on smooth slopes, so that a formula based on the concept of a reduction factor multiplier (to smooth slope expressions) would not predict well the uprush on rough slopes.

The reduction factors for various type of structures were estimated and summarized in the SPM (USACE 1984) based on the Corps' laboratory data and are reproduced in Table 1.

Figure 13: Relative Uprush on Smooth Slopes and Rough Permeable Slopes (after USACE 1984)

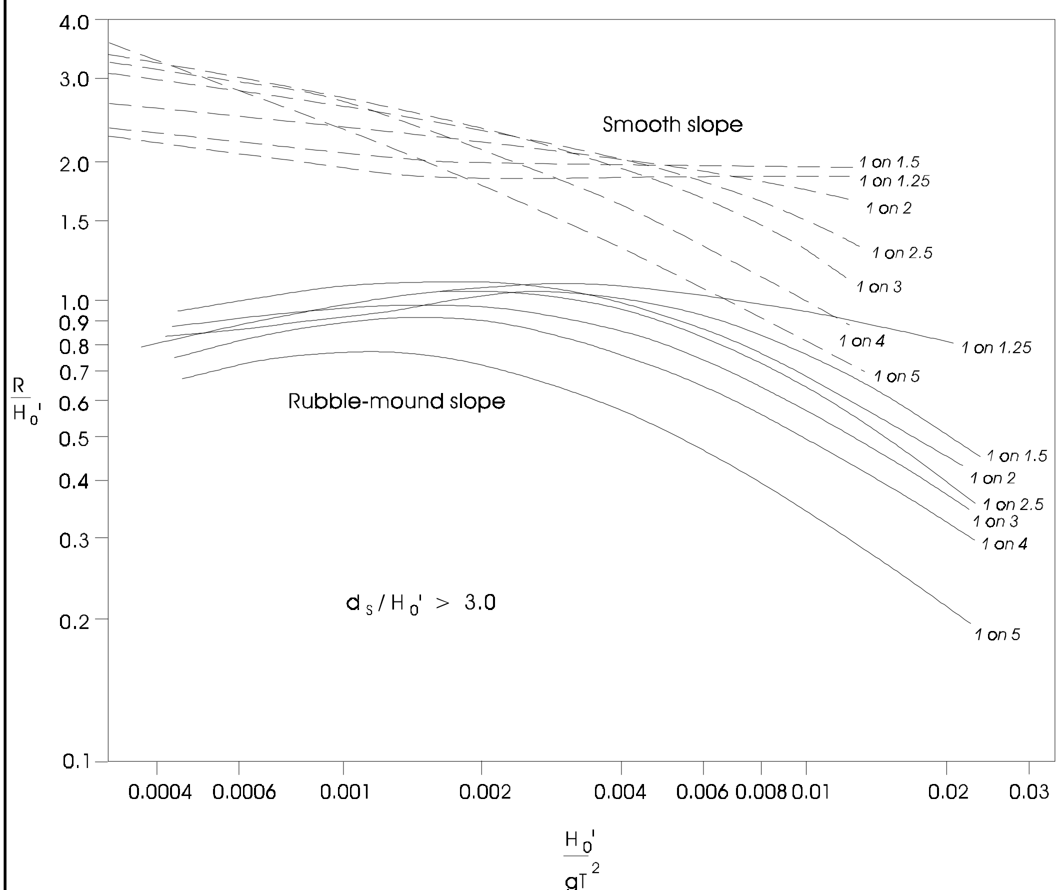


Table 1: Surface Reduction Factors - SPM (USACE 1984)

Slope Surface Characteristics	Placement	*r*
Smooth, impermeable	---	1.00
Concrete blocks	Fitted	0.90
Basalt or Gobi blocks	Fitted	0.85 - 0.90
Grass	---	0.85 - 0.90
Quarrystone	Fitted	0.75 - 0.80
Quarrystone (rounded)	Random	0.60 - 0.65
Quarrystone, 1 layer (impermeable base)	Random	0.80
Quarrystone, 3 layers (impermeable base)	Random	0.60 - 0.65
Quarrystone	Random	0.50 - 0.55
Concrete armour units (~50% void ratio)	Random	0.45 - 0.50

. Ahrens and McCartney (1975)

Ahrens and McCartney (1975) and Gunbak (1979) proposed to use the following expression for uprush for the entire range of breaking and non-breaking conditions:

$$\frac{R}{H} = \frac{a\xi}{1 + b\xi}$$

where *a* and *b* are empirical coefficients determined for a particular type of armour units in place. Ahrens (1981b) states that this equation gives "reliable estimates of monochromatic wave uprush for $d_s/H > 3$ and for slopes 1 on 2 to 1 on 10". For the breaking region (i.e. small ξ), the equation reduces to a form similar to Hunt's formula as $R/H = a\,\xi$. For non-breaking region (i.e. very large ξ), the equation reduces to a constant as in linear standing wave theory as $R/H = a/b$. Table 2 gives the values of *a* and *b* (Seelig, 1980). This method is used in FEMA (1991) and ACES (USACE 1990) to determine wave uprush on impermeable rip-rap revetments.

Table 2: Coefficients for Ahrens and McCartney (1975) Method (see also Seelig, 1980)

Armour Stones	*a*	*b*
Rip-rap revetment* (impermeable base)	0.956	0.398
Rubble-mound breakwater* (quarrystone, 2 layers)	0.775	0.361
Rubble-mound breakwater (highly permeable core)	0.692	0.504

* see Figures A.1 and D.1 in Appendix B for comparative illustrations of rip-rap revetments and rubble-mound breakwaters.

. Gunbak (1979)

Gunbak (1979) measured wave uprush on armour slopes of breakwaters and obtained:

$$\frac{R}{H} = \frac{0.8\,\xi}{1 + 0.5\,\xi}$$

When compared with the measured relative uprush data from a rock armoured slope, the estimates using the above equation were slightly over-predicted. Gunbak (1979) proposed the following equations for rock armoured slopes:

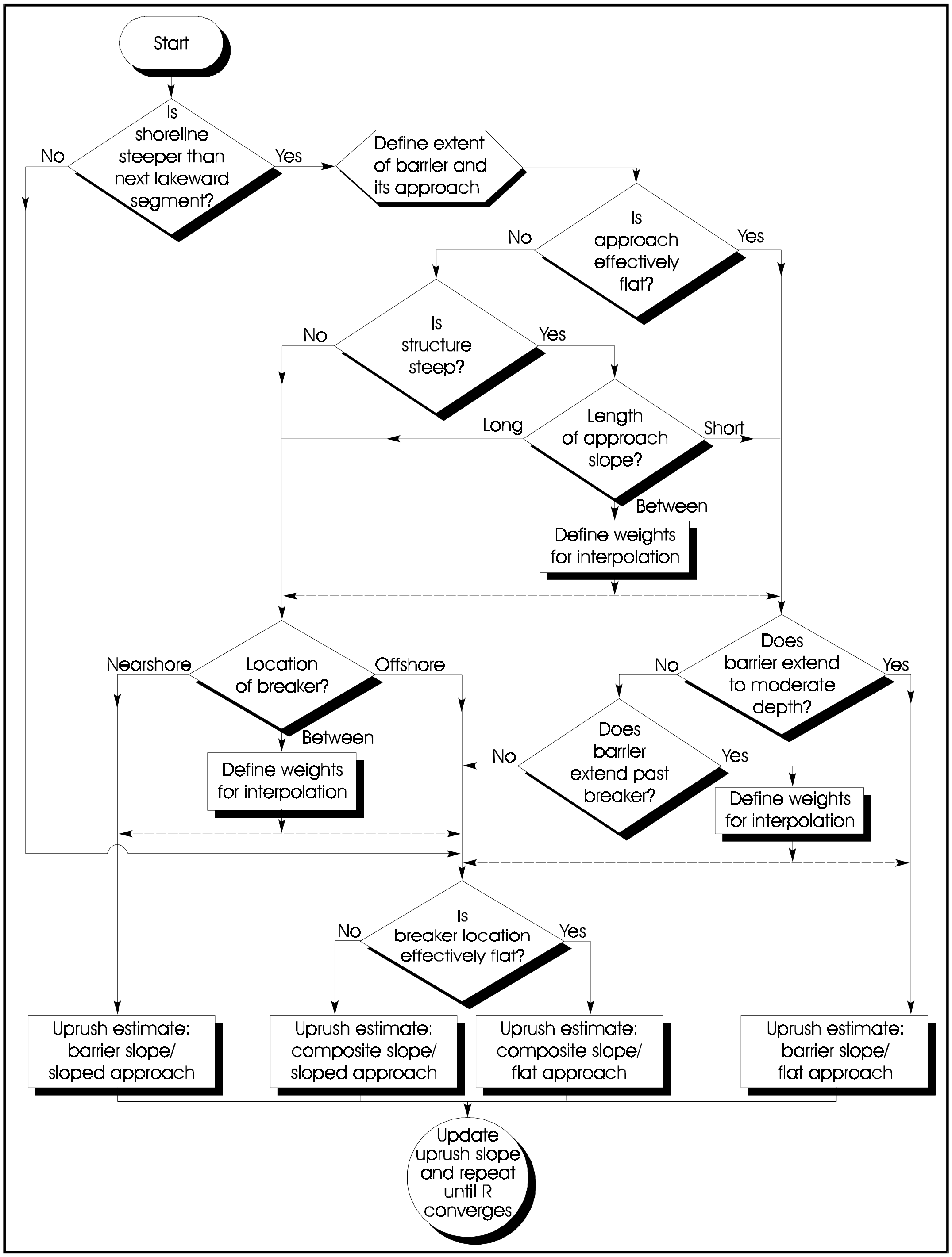
$R/H = 0.4\ \xi$; $\quad 0.0 < \xi < 3.0$,
$R/H = 1.2$; $\quad 3.0 < \xi$.

. Stoa (1979)

Stoa (1979) presented relative uprush curves for particular rough slope and wave conditions based on previous tests ($d_s / H_o' \geq 3$ and slopes from 1:1.5 to 1:5) and a procedure to estimate rough-slope uprush as a function of uprush on a comparable smooth slope for untested conditions. A set of curves, along with a table of slope surface reduction factors, *r*, for rip-rap size material, and flow charts (Stoa, 1979) are presented in Appendix B.

Stoa's method was codified by Stone & Webster (1981) and further modified and improved by Dewberry & Davis (1990) for use by the Federal Emergency Management Agency (FEMA) in the United States. The Dewberry and Davis model was developed for ocean shorelines but is applicable to the Great Lakes (B. Hallermeier, pers. comm). Figure 14 reproduced from Hallermeier et al., (1990) gives the flowchart of the FEMA wave uprush model. Also, an evaluation of the model predictions was presented in Hallermeier et al., (1990) for field and laboratory data for both smooth and rough structures.

Figure 14: Flow Chart for FEMA Uprush Model (after Hallermeier et al. 1990)



. Seelig (1980)

Based on the data from Ahrens and McCartney (1975), Gunbak (1976) and Hudson (1958), Seelig (1980) obtained the values of *a* and *b* shown in Table 2. The resultant equations are shown in Figure 15. It can be seen that the relative uprush is increasing monotonically with increasing ξ. The uprush on a conventional rubble-mound breakwater armoured with quarry stones is much less than that on a rip-rap revetment. Uprush is even less for a rubble-mound breakwater with a high degree of permeability.

Ahrens (1981b) compared the methods of Ahrens and McCartney (1975) and Stoa (1979) with the laboratory test results of Ahrens and Seelig (1980) which used a 1:2 rip-rap revetment slope and a submerged fronting slope of 1:15. Both methods were found to over-predict the observed maximum uprush by an average of 38 percent for Stoa (1979) and 29 percent for Ahrens and McCartney (1975). Ahrens (1981b) cautioned that the data were for only one slope and that it was not clear how general was the tendency for over-prediction.

. Losada and Gimenez-Curto (1981)

Losada and Gimenez-Curto (1981) re-analyzed the regular wave tests on rough, permeable slopes and proposed a generalised expression for relative uprush as:

$$\frac{R}{H} = A(1 - \exp[B\xi])$$

where *A* and *B* are empirical coefficients. Based on the data from Ahrens and McCartney (1975), Gunbak (1976), and Dai and Kamel (1969), the values of *A* and *B* were obtained and indicated in Table 3 and the resultant equations are plotted in Figure 16. Figures 17, 18 and 19 show actual data from the tests of Ahrens and McCartney (1975), Gunbak (1976) and Dai and Kamel (1969) respectively.

Table 3: Coefficients for Losada and Gimenez-Curto (1981) Method

Armour Layer	*A*	*B*
Rip-rap revetment (impermeable base)	1.789	-0.455
Rubble-mound breakwater (quarrystone, 2 layers)	1.451	-0.523
Rubble-mound breakwater	1.370	-0.596

Figure 15: Relative Uprush - Ahrens and McCartney Method

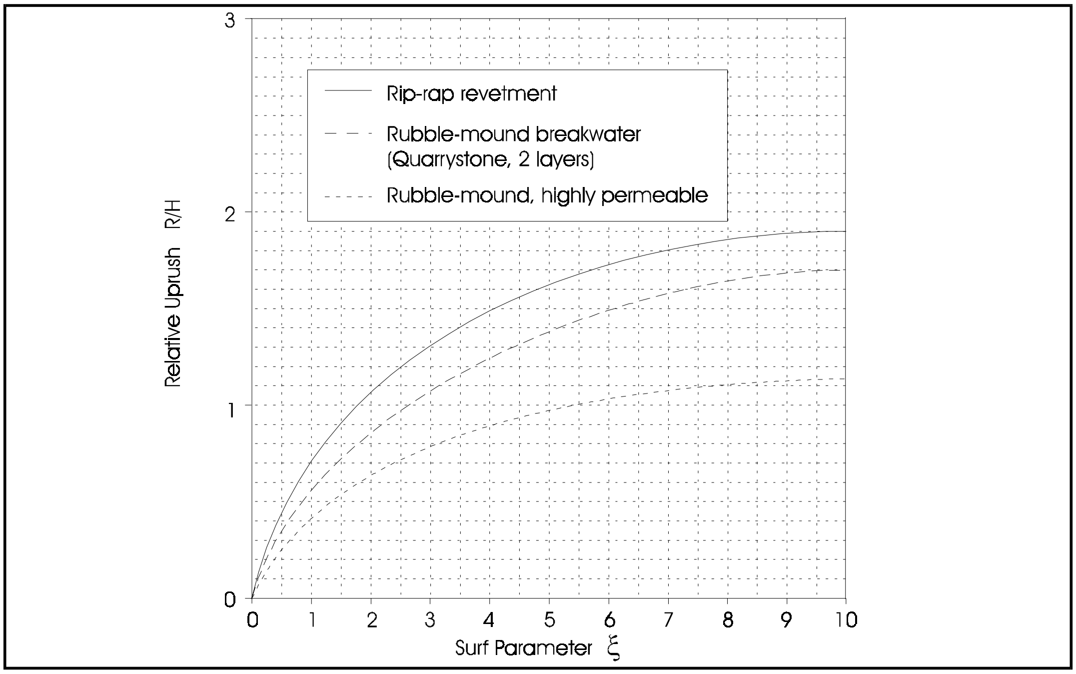


Figure 16: Relative Uprush - Losada and Gimenez-Curto Method

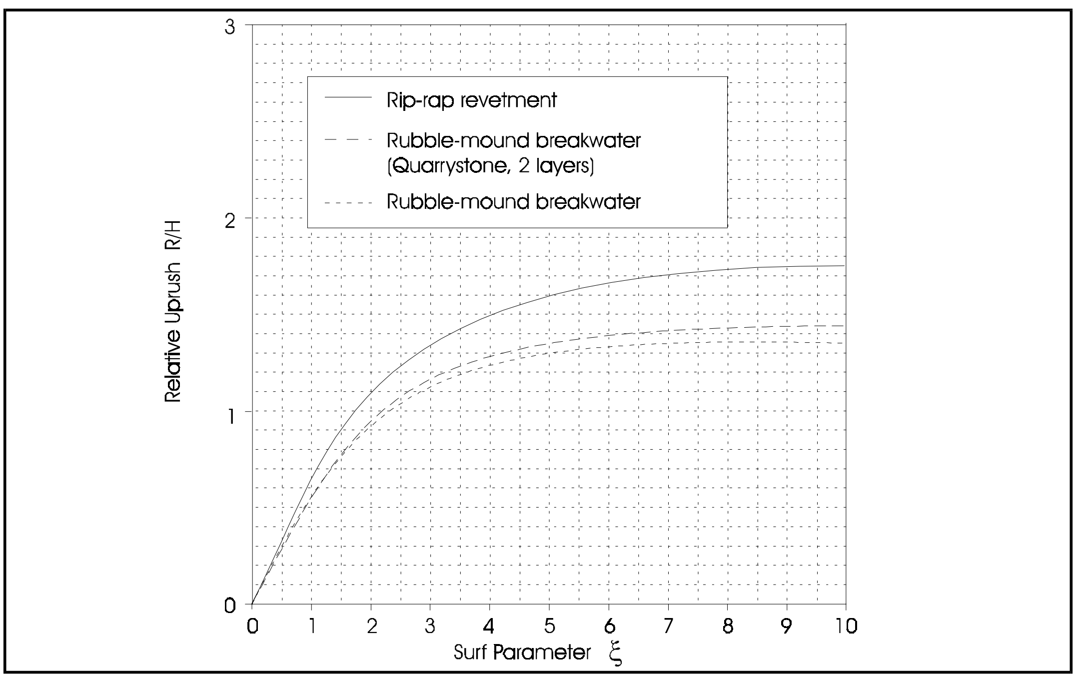
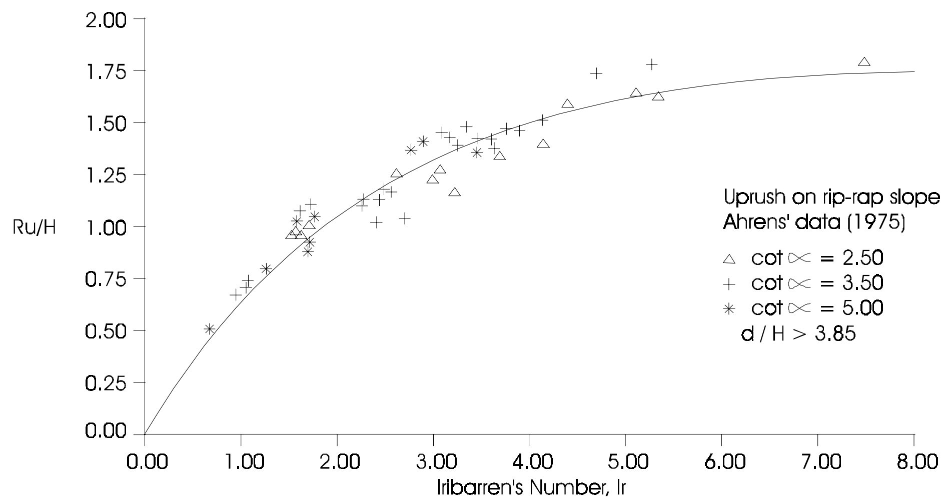
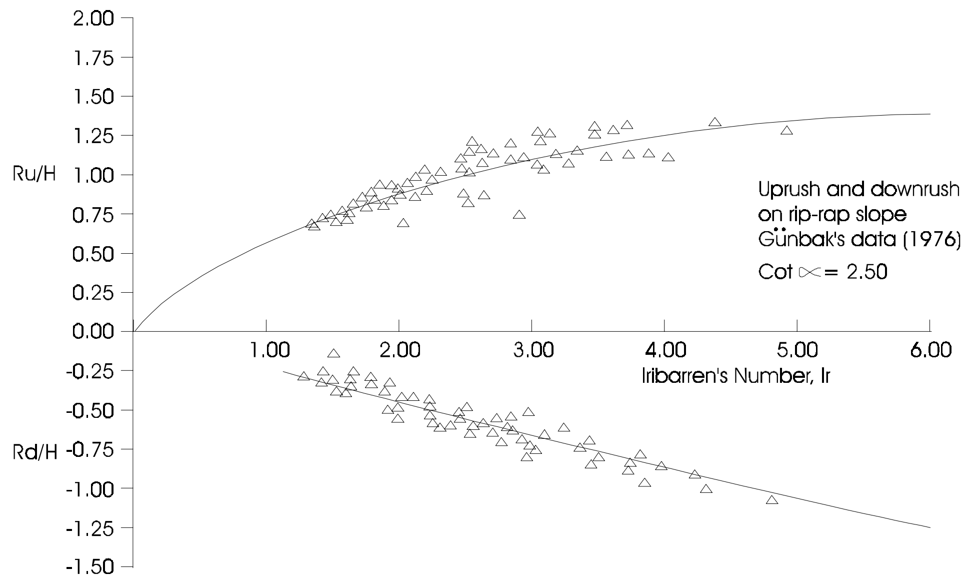


Figure 17: Relative Uprush on Rip-rap Slopes (after Bruun 1985)



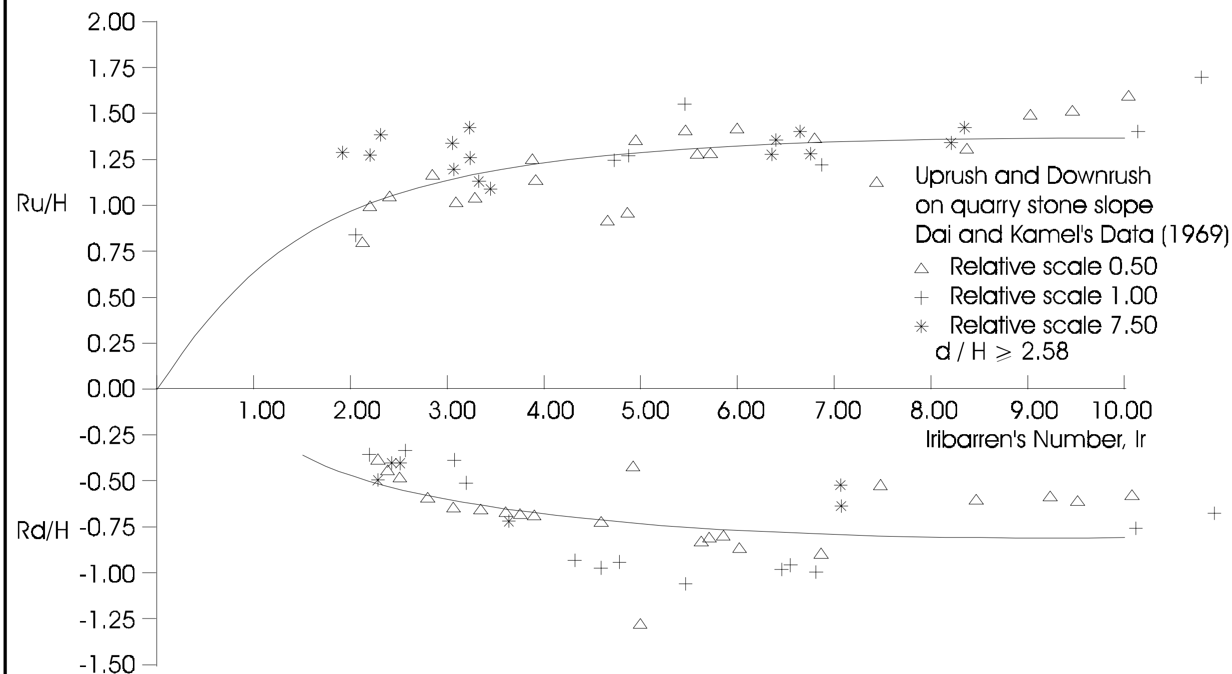
Note: The line represents the equation of Losada and Gimenez-Curto (1981).

Figure 18: Relative Uprush and Downrush on Rip-rap Slopes (after Bruun 1985)



Note: The lines represent the equations of Losada and Gimenez-Curto (1981).

Figure 19: Relative Uprush and Downrush on Quarry Stone Slope (after Gunbak 1979)



Note: The lines represent the equations of Losada and Gimenez-Curto (1981).

. **Stevenson (1983)**

Stevenson (1983) conducted laboratory tests to investigate regular wave uprush on shingle beaches of slopes of 1:6, 1:8 and 1:10.45. The uprush on the shingle beaches was found to be about 35% of that predicted by Hunt's equation thus giving:

$$\frac{R}{H} = 0.35\,\xi$$

. **Walton, Tsai, Dean and Richardson (1989)**

Walton et al. (1989a, 1989b) recognized that the information on uprush reduction due to roughened surfaces is limited and that the reduction factor, *r*, cited in all literature should be considered as a combination of the effects of turbulent energy dissipation on the slope face due to roughness and the effects of turbulent energy dissipation within the structure due to permeability. They posed a possible form of the reduction factor as:

r = (*roughness factor*) X (*permeability factor*).

They observed that the reduction factors appearing in all existing laboratory data range from 0.5 to 0.9. Owing to lack of confidence in extrapolating this information to field conditions, they suggested using the reduction factors obtained from the high end of the laboratory measurements (see Table 4).

Table 4: Surface Reduction Factors - Walton et al. (1989a, 1989b)

Slope Surface Characteristics	*r*
Smooth	1.0
Concrete or Gobi blocks	0.9
Grass	0.9
Quarrystone, rubble	0.8
Stepped surface	0.8

. **Pilarczyk (1990)**

Pilarczyk (1990) suggested the surface reduction factors for wave uprush calculations given in Table 5, for application in the design of seawalls, dykes and revetments in the Netherlands.

Table 5: Surface Reduction Factors - Pilarczyk (1990)

Slope Surface Characteristics	*r*
Asphalt, smooth concrete	1.0
Concrete blocks, geotextile-mats, open stone-asphalt, grass-mat	0.95
Pitch stone, basalton	0.9
Rough, permeable block mats	0.8
Gravel, gabions	0.7
Rip-rap (minimum thickness 2 D_{50})	0.6

. U.S. Army Corps of Engineers (1990)

In the software package, Automated Coastal Engineering System (ACES), developed by the Coastal Engineering Research Center (USACE 1990), the empirical equations used for prediction of uprush on rough impermeable sloped structures are the ones developed by Ahrens and McCartney (1975). The coefficients for this formula were given in Table 2.

iii) Irregular Waves

Wind waves in the sea are random in nature. It is difficult to define the absolute maximum wave height and period. Waves are generally described by a probability distribution function such that the wave with a specified level of exceedance probability can be used for design. For example, the significant wave height, H_s , is defined as the average of the highest one-third of the waves. In the most recent research with irregular waves, the wave uprush caused by a wave train is also defined in the same manner.

A probability distribution such as Rayleigh, Weibull, etc., is usually fitted to the observed or measured uprush data. For irregular waves, the wave parameters H_s and T_p will be used to compute the surf similarity parameter as:

$$\xi_p = \frac{\tan\theta}{\sqrt{\frac{H_s}{L_p}}} = \frac{\tan\theta}{\sqrt{\frac{2\pi H_s}{gT_p^2}}}$$

Saville (1962) linked regular wave uprush to irregular uprush using the "hypothesis of equivalency" which assumes that the relative uprush distribution, R_p / R_s, is distributed similar to the relative wave height distribution, H_p / H_s, which has a Rayleigh distribution (in deep water).

The parameter R_p is the wave uprush associated with a particular probability of exceedance, P, and R_s is the wave uprush of the significant wave height, given as:

$$\frac{H_p}{H_s} = \frac{R_p}{R_s} = \sqrt{-\frac{\ln P}{2}}$$

For the purpose of design, the uprush of 2% exceedance probability, R_2, is commonly used in the Netherlands (Pilarczyk 1990). If the uprush distribution is Rayleigh, then

$$\frac{R_2}{R_s} = \sqrt{-\frac{\ln 0.02}{2}}$$

giving $R_2 \approx 1.4\, R_s$.

However, there is no evidence available to prove that the uprush of a particular level of exceedance probability is caused by the waves with the same exceedance probability. Also, the probability distribution of the uprush may not be the same as that of the incoming waves creating the uprush.

The assumption of Rayleigh distribution for both incident waves and uprush is not proven. In shallow water, there will be a truncation in the wave height distribution due to depth limited and steepness induced breaking. Ahrens (1981b) suggested use of the Goda (1975) model for establishing the ratio of H_{max} / H_s. Further discussion of regular and irregular wave parameters is given in Section 4.3(a).

. van Oorschot and d'Angremond (1968)

Laboratory tests with irregular waves on smooth slopes of 1:4 and 1:6, performed by van Oorschot and d'Angremond (1968), indicated that the shape of the incident wave spectrum had an important influence on the shape of the resulting uprush distribution. They concluded that wider spectra gave rise to higher extreme uprush values and proposed a modified Hunt formula for the 2% uprush as:

$$\frac{R_2}{H_s} = \sqrt{2\pi}\, C_2\, \xi_p$$

where the coefficient C_2 is determined by the spectral width, ϵ, ($\epsilon^2 = (m_o m_4 - m_2^2) / (m_o m_4)$, where m_o, m_2 and m_4 are the zeroth, second and fourth moments of the spectrum). Gunbak (1979) suggested the values in Table 6 to be used.

Table 6: Coefficients for van Oorschot and d'Angremond (1968) Method (after Gunbak 1979)

ϵ	C_2
0.3	0.55
0.4	0.61
0.5	0.67
0.6	0.73

. Kamphuis and Mohamed (1978)

Kamphuis and Mohamed (1978) conducted experiments for irregular wave uprush on smooth slopes of 1:1, 1:1.5, 1:2 and 1:3 and concluded that both wave height and uprush distributions were approximately Rayleigh. For non-breaking irregular waves, they concluded that Miche's (1951) equation may be valid for irregular waves provided that the mean uprush and wave height were, i.e.,

$$\bar{R}/\bar{H} = \sqrt{\frac{\pi}{2\theta}}$$

but they found that $R_2/\bar{R} = 2.4$ instead of 2.23 for Rayleigh distribution.

. Ahrens (1981a)

Ahrens (1981a) employed another approach to predict irregular wave uprush for smooth slopes. Based on the data from Oorschot and d'Angremond (1968), Kamphuis and Mohamed (1978) and Ahrens (1979), Ahrens developed an equation to predict irregular wave uprush, R_2, R_s and $\bar{R}$ for structure slopes of 1:1, 1:1.5, 1:2, 1:2.5, 1:3 and 1:4, as:

$$\frac{R_i}{H_s} = c_1 + c_2 \frac{H_s}{g T_p^2} + c_3 \left(\frac{H_s}{g T_p^2} \right)^2$$

where R_i can be either R_2, R_s or $\bar{R}$, c_1, c_2 and c_3 are dimensionless regression coefficients found in Table 7.

For structures with slopes flatter than 1:4, he recommended the following equations similar to Hunt (1959):

$$\frac{R_2}{H_s} = 1.61\,\xi\,; \qquad \frac{R_s}{H_s} = 1.25\,\xi\,; \qquad \frac{\bar{R}}{H_s} = 0.84\,\xi$$

Table 7: Coefficients for Ahrens (1981a) Method

Regression Coefficients for R_2 / H_s			
cot θ	c_1	c_2	c_3
1.0	2.32	71.5	0
1.5	2.52	195	0
2.0	3.21	71.9	0
2.5	3.39	129	-16100
3.0	3.70	0	-17000
4.0	3.60	-222	0

Regression Coefficients for R_s / H_s			
cot θ	c_1	c_2	c_3
1.0	1.34	66.1	0
1.5	1.38	318	-19700
2.0	1.64	357	-30900
2.5	1.94	279	-32100
3.0	2.11	187	-26700
4.0	2.52	-79.4	0

Regression Coefficients for $\overline{R} / H_s$			
cot θ	c_1	c_2	c_3
1.0	0.71	110	-8070
1.5	0.75	197	-11400
2.0	0.93	242	-19300
2.5	1.00	278	-31300
3.0	1.19	209	-29600
4.0	1.47	72.5	-17000

. Ahrens (1983)

Ahrens (1983) compared uprush caused by regular and irregular waves on structure slopes ranging from 1:1 to 1:4 and found that irregular wave uprush can be described by a Weibull distribution and the relative uprush caused by non-breaking irregular waves had the same trend as Miche's (1944) equation. Also, he concluded that the effect of non-linearity on irregular wave uprush was not significant.

. Ahrens and Heimbaugh (1988a)

Based on model studies, including Great Lakes sites, and using a wide variety of water depths, Ahrens and Heimbaugh (1988a) developed a method to compute irregular wave uprush on rip-rap protected embankments for structure slopes of 1:2, 1:3 and 1:4. They proposed the following formula:

$$\frac{R_{\max}}{H_{mo}} = \frac{a\,S}{1 + b\,S}$$

where R_{max} is the elevation of maximum wave uprush during test observations (i.e., the "extreme excursion of green water" as measured by an "experienced observer"), H_{mo} is the energy-based zero-moment wave height, S is a surf parameter defined by either ξ_M or ξ_L and a and b are dimensionless coefficients given in Table 8.

Table 8: Coefficients for Ahrens and Heimbaugh (1988a) Method

Equation	Wave-length used	Surf parameter used	Uprush coefficients
Recommended	L_p	$S = \xi_L$	$a = 1.154$ $b = 0.202$
Alternative	L_o	$S = \xi_M$	$a = 1.022$ $b = 0.247$

L_p = Airy wavelength calculated using the peak wave period of the energy spectrum T_p and the water depth at the structure toe. L_p can be estimated from Appendix D.

$L_o = g\,T_p^2 / 2\pi$. For definitions of S, see Table 10.

. **Mase (1989)**

Mase (1989) conducted an extensive series of laboratory tests to develop a formula to predict irregular wave uprush on gentle, impermeable slopes ranging from 1:5 to 1:30. The deep-water significant wave height and period were used. The wave steepness values ranged from 0.004 to 0.07. From the experimental study results, Mase found that relative uprush becomes large as the wave steepness becomes small and as the slope becomes steep. This tendency is opposed to those measured for steep slopes by Ahrens (1983) and Kamphuis and Mohamed (1978). The following formulae were developed by Mase (1989) for wave uprush predictions (see Figure 7):

$$\frac{R_{\max}}{H_{os}} = 2.32\,\xi^{0.77}$$

$$\frac{R_2}{H_{os}} = 1.86\,\xi^{0.71}$$

$$\frac{R_s}{H_{os}} = 1.38\,\xi^{0.70}$$

$$\frac{\overline{R}}{H_{os}} = 0.88\,\xi^{0.69}$$

These equations give an envelope for the maximum of the scattered test observations. To describe an average trend, multiply the right side of the equations by 0.5. Mase noted that the differences between predicted values and observations measured on a natural beach seemed to depend on the differences in permeability and roughness.

. **Pilarczyk (1990)**

In the Netherlands, Pilarczyk (1990) recommended to use $C_2 = 0.70$ (wide spectrum coefficient of van Oorschot and d'Angremond, 1968, equation) and a wave steepness value of 0.05 (typical of storm waves for the North Sea coast). This was described by Pilarczyk (1990) as a "safe approach" for determining uprush due to wind waves for smooth slopes. Pilarczyk proposed:

$R_2 / H_s = 1.75\,\xi_p$ for $\xi_p < 2.5$,
$R_2 / H_s = 3.5$ for $\xi_p \geq 2.5$.

. Van der Meer and Stam (1992)

Van der Meer and Stam (1992) reported on model tests of irregular wave uprush on rock slopes, including revetment (impermeable core) and breakwater (permeable core) structures with slopes ranging from 1:1.5 to 1:4. They presented the following relationships for impermeable and permeable rock (rough) slopes:

$$\frac{R_n}{H_s} = a\,\xi_m \qquad \text{for } \xi_m \leq 1.5$$

$$\frac{R_n}{H_s} = b\,\xi_m^c \qquad \text{for } \xi_m \geq 1.5$$

For surging wave conditions (i.e. $\xi_m > 2$ or 3), the relative uprush for permeable rock slopes is limited to

$$\frac{R_n}{H_s} = d$$

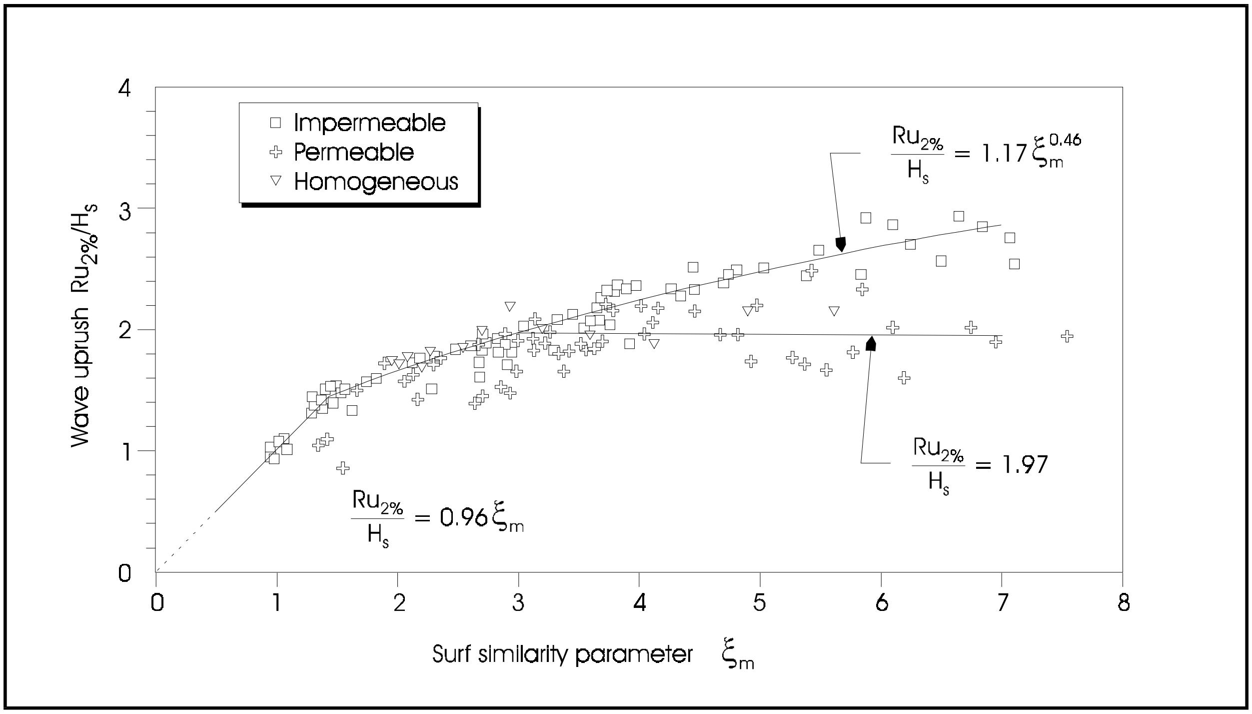
where $\xi_m = \dfrac{\tan\theta}{\sqrt{H_s/L_m}}$; $L_m = \dfrac{g\,T_m^2}{2\pi}$; and T_m is the mean wave period ($T_p = 1.1 \sim 1.2\ T_m$).

Values for the coefficients for the above equations are given in Table 9 and the relationships for R_2 are shown in Figure 20. This approach is outlined in CIRIA/CUR (1991). The equations are only valid for relatively deep water. Wave breaking in the nearshore results in a truncation of the uprush distribution. Van der Meer and Stam found that this results in lower maximum uprush heights but could sometimes produce higher mean uprush values. Therefore the relationships will yield conservative estimates of high (i.e., R_2) uprush values under depth-limited conditions. Van der Meer and Stam also presented a procedure describing wave uprush as a Weibull distribution and includes consideration of the slope angle, wave steepness and permeability.

Table 9: Coefficients for Van der Meer and Stam (1992) Method

Uprush R_n	*a*	*b*	*c*	*d*
$R_{0.13}$	1.12	1.34	0.55	2.58
R_1	1.01	1.24	0.48	2.15
R_2	0.96	1.17	0.46	1.97
R_5	0.86	1.05	0.44	1.68
R_{10}	0.77	0.94	0.42	1.45
R_s	0.72	0.88	0.41	1.35
R_{mean}	0.47	0.60	0.34	0.82

Figure 20: Influence of Permeability of Structure on Wave Uprush (after Van der Meer and Stam 1992)



2.2 Accepted Wave Uprush Methodologies

Several uprush methodologies may be considered "accepted" practice for determining *flooding hazards* along Great Lakes - St. Lawrence River System shorelines if the methodologies are used in the same context as which they are based. For the purpose of this Technical Report, these "accepted" methods include:

Smooth Slopes		Rough Slopes	
Regular Waves	Irregular Waves	Regular Waves	Irregular Waves
• Hunt (1959)	• Ahrens (1981a)	• Ahrens and McCartney (1975)	• Ahrens and Heimbaugh (1988a)
• Stoa (1978b)	• Mase (1989)	• Stoa (1979)	• Van der Meer and Stam (1992)
• Ahrens and Titus (1985)	• Pilarczyk (1990)	• Losada and Gimenez-Curto (1981)	
• Walton and Ahrens (1989)	• Van der Meer and Stam (1992)		

Table 10 summarizes the "accepted" methods for the prediction of wave uprush. Tables 1 to 5 and 7 to 9 give the various coefficients and reduction factors quoted in Table 10. Note that the work of Stoa (1978) has revised and superseded the guidance for uprush on slopes provided by the Shore Protection Manual (SPM) (U.S. Army Corps of Engineers 1973; 1984). The Automated Coastal Engineering System (ACES) (USACE 1990) includes methods listed above. In addition, physical model tests, when properly designed and executed, are considered accepted practice for both smooth and rough slopes.

The procedures outlined for the various "accepted" uprush methods must be followed closely and they should not be extrapolated much beyond the tested conditions. It may be appropriate, when any one of the methodologies are used to estimate wave uprush, for the proponent to provide a brief summary of how the selected methodology was derived and why it is applicable to the situation under study. This would help to demonstrate the proponents' understanding of the limits of the method used. The proponent should then evaluate whether or not any adjustments should be made to the predicted uprush. This should be valid for both small and large project, where the former is done with existing guidance and the later with the aid of site-specific model tests.

Table 10: Summary of Wave Uprush Methodologies

Table 10: Summary of Wave Uprush Methodologies

Method	References	Equations and Boundary Conditions	Remarks
Hunt	Hunt (1959) Battjes (1974)	$\frac{R}{H} = \xi = \frac{\tan\theta}{\sqrt{H/L_o}}$ $\quad \xi < 2.3$ **Smooth slopes; regular waves**.	Used for uprush on natural beaches. The definition of input parameters (wave height and bottom slope) is open for discussion due to the wave transformation processes including breaking (e.g. Silvester, 1974).
Stoa (smooth slopes)	Stoa (1978b) Hallermeier et al., (1990)	Graphs of method are given in Appendix A, as well as correction factors for rough slope applications in Tables 1 and 4. **Smooth slopes** fronted by flat bottom or by 1:10 slope. **Regular waves**.	For application see flow chart (Figure 12) of Appendix A. Stoa's method is used for uprush on beaches and with correction factors for rough sloping structures. This method was further developed by Dewberry and Davis (1990) for the Federal Emergency Management Agency (FEMA), in the U.S.A.
Ahrens and Titus (modified)	Ahrens and Titus (1985) Walton et al. (1989a; 1989b) USACE (1990)	$\left[\frac{R}{H}\right]_p = 1.002\,\xi$; or $\left[\frac{R}{H'_o}\right]_p = 0.967\,\xi$ $\quad \xi \leq 2.0$ (plunging) $\left[\frac{R}{H}\right]_t = \left(\frac{\xi - 2.0}{1.5}\right)\left[\frac{R}{H}\right]_{nb} + \left(\frac{3.5 - \xi}{1.5}\right)\left[\frac{R}{H}\right]_p$ $\quad 2.0 < \xi < 3.5$ (transitional) $\left[\frac{R}{H}\right]_{nb} = 1.087\left(\sqrt{\frac{\pi}{2\theta}}\right) + 0.775\,\frac{H/L}{\tanh^3(kd_s)}$ $\quad \xi \geq 3.5$ (nonbreaking) **Smooth slopes; regular waves**. Tables 1 and 4 show the surface reduction factors recommended for rough slopes.	The expression for the nonbreaking region ($\xi \geq 3.5$) is as modified by Walton et al. (1989a; 1989b). This method is included in the Automated Coastal Engineering System (ACES, USACE 1990). The first equation may be used for natural beaches. Also used for sloping and vertical structures, depending on the value of ξ. The equation using H'_o for plunging regions is used in FEMA (1991) for beaches.

Table 10: Summary of Wave Uprush Methodologies

Method	References	Equations and Boundary Conditions	Remarks
Walton and Ahrens	Walton and Ahrens (1989) Walton et al., (1989a; 1989b)	$\frac{R}{H} = \frac{sin\theta}{\sqrt{H/L_o}}$ Upper limit: $\frac{R}{H} = (2\pi)^{1/2} \, (\frac{\pi}{2\theta})^{1/4}$ **Smooth slopes; regular waves**. Table 4 shows the surface reduction factors recommended for rough slope applications.	Upper limit expression is an envelope for uprush data in the nonbreaking zone, on the conservative side. Note that for small values of θ, sinθ ≈ tanθ, so that for natural beach applications this method gives similar results as Hunt's. May be used for both beaches and protective structures. This work (Walton et al. 1989a; 1989b) was sponsored by FEMA.
Ahrens and McCartney	Ahrens and McCartney (1975) Seelig (1980) USACE (1990)	$\frac{R}{H} = \frac{a\xi}{1 + b\xi}$ Coefficients for the above equation are given in Table 2. **Rough slopes; regular waves**.	This method gives reliable estimates of regular wave uprush for $d_s / H > 3$ and for slopes 1:2 to 1:10. For small ξ (breaking region) this equation reduces to a form similar to Hunt's (1959). Used for sloping revetments and rubble-mound breakwaters. Method used by FEMA (1991) for revetments and is included in ACES (USACE 1990).
Stoa (rough slopes)	Stoa (1979)	Graphs of method are given in Appendix B. For particular rough slopes only (1.5 < cot θ < 5), and for $d_s / H_o' \geq 3$. **Rough slopes: regular waves**.	For application see flow chart (Figure 2) of Appendix B. This method has limited applications for uprush on sloping structures due to its limited boundary conditions. To be used in conjunction with Stoa (1978b).
Losada and Gimenez-Curto	Losada and Gimenez-Curto (1981)	$\frac{R}{H} = A(1 - \exp[B\xi])$ Coefficients for the above equation are given in Table 3. **Rough slopes; regular waves**.	This method gives similar results as Ahrens and McCartney (1975), and may be used for sloping revetments.

Table 10: Summary of Wave Uprush Methodologies

Method	References	Equations and Boundary Conditions	Remarks
Ahrens	Ahrens (1981a)	$\frac{R_i}{H_s} = c_1 + c_2 \frac{H_s}{gT_p^2} + c_3 \left(\frac{H_s}{gT_p^2} \right)^2$ where R_i can be either R_2, R_s or $\bar{R}$, and c_1, c_2 and c_3 are dimensionless regression coefficients found in Table 7. **Irregular waves, smooth slopes** of 1:1, 1:1.5, 1:2, 1:2.5, 1:3 and 1:4. For slopes flatter than 1:4; $\frac{R_2}{H_s} = 1.61\,\xi$ $\frac{R_s}{H_s} = 1.25\,\xi$ $\frac{\bar{R}}{H_s} = 0.84\,\xi$	This method provides a means for estimating the R_2, R_s or $\bar{R}$ values of wave uprush.
Mase	Mase (1989)	$\frac{R_{max}}{H_s} = 2.32\,\xi^{0.77}$ $\frac{R_2}{H_s} = 1.86\,\xi^{0.71}$ $\frac{R_s}{H_s} = 1.38\,\xi^{0.70}$ $\frac{\bar{R}}{H_s} = 0.88\,\xi^{0.69}$ **Irregular waves**; **smooth**, impermeable, gentle (1:5 to 1:30) **slopes**.	Wave steepness tested ranged from 0.004 to 0.07.

Table 10: Summary of Wave Uprush Methodologies

Method	References	Equations and Boundary Conditions	Remarks
Pilarczyk	Pilarczyk (1990)	$\frac{R_n}{H_s} = C_n \sqrt{2\pi}\,\xi_p$ *where* $\xi_p = \frac{\tan\theta}{\sqrt{H_s/L_p}}$ **Smooth slopes**; **irregular waves**. Surface reduction factors for the above equation are given in Table 5 for rough slope applications.	The values for $C_n = C_{2\%}$ (uprush exceeded by 2% of waves), were estimated from measurements, and are roughly equal to: $C_{2\%}$ = 0.5 to 0.6 - for a narrow spectrum and $C_{2\%}$ = 0.7 - for a wide spectrum. Values for $C_n = C_s$ (significant waves) were not reported.
Ahrens and Heimbaugh	Ahrens and Heimbaugh (1988a)	$\frac{R_{max}}{H_{mo}} = \frac{aS}{1 + bS}$ where $S = \xi_M = \tan\theta / (H_{mo} / L_o)^{1/2}$ [alternative] or $S = \xi_L = \tan\theta / (H_{mo} / L_p)^{1/2}$ [recommended by authors] Coefficients for the above equation are given in Table 8. **Rough slopes** only (1:2, 1:3 and 1:4); **irregular waves**.	This method allows for a choice of surf parameter to be used: the Irribaren number ξ_M or a local wavelength parameter ξ_L (see Table 8). Also, H_{mo} is recommended, rather than H_s. This method is limited to sloping rubble-mound revetment structures, with slopes of 1:2 to 1:4, when irregular wave data are available.
Van der Meer and Stam	Van der Meer and Stam (1992)	For **irregular waves**. For impermeable and permeable rock (**rough**) **slope**: $\frac{R_n}{H_s} = a\,\xi_m$ for $\xi_m \le 1.5$ $\frac{R_n}{H_s} = b\,\xi_m^c$ for $\xi_m \ge 1.5$ to a limit of $\frac{R_n}{H_s} = d$ for permeable slopes when $\xi_m > 2$ or 3; where $\xi_m = \frac{\tan\theta}{\sqrt{H_s/L_m}}$; $L_m = \frac{g T_m^2}{2\pi}$ and T_m is the mean wave period ($T_p = 1.1 \sim 1.2\ T_m$). Coefficients for the above equations are given in Table 9.	Model tests of uprush on rock slopes, including revetment (impermeable core) and breakwater (permeable core) structures with slopes ranging from 1:1.5 to 1:4. When rock slope is permeable, relative uprush will reach a limit for surging wave conditions (i.e. $\xi_m > 2$ or 3). Valid for relatively deep water. Wave breaking in nearshore results in a truncation in uprush distribution (results in lower maximum uprush heights but sometimes higher mean uprush values) giving conservative estimate of high uprush values under depth limited conditions.

2.3 Limitations of Wave Uprush Predictors

Problems with the present state-of-the-art of uprush predictions include:

- Roughness and permeability play a major role in the uprush processes (see, for instance, Figure 21 which shows that the relative difference between uprush on smooth and rough slopes, r, is not constant, but varies with surf similarity factor, ξ).
- It is difficult to develop a generic predictor for uprush due to the different flow regimes and complex wave-structure interaction processes.
- Irregular wave uprush data brings about the problem of the definition of characteristic wave climate parameters to represent the incident wave train in the calculation procedures.
- The choice of the nearshore wave transformation procedure requires experience.
- Considerable judgement is required for the definition of the structure or shore slope (e.g. some sites have composite slopes) and for the nearshore approach slope to be used in the equations.

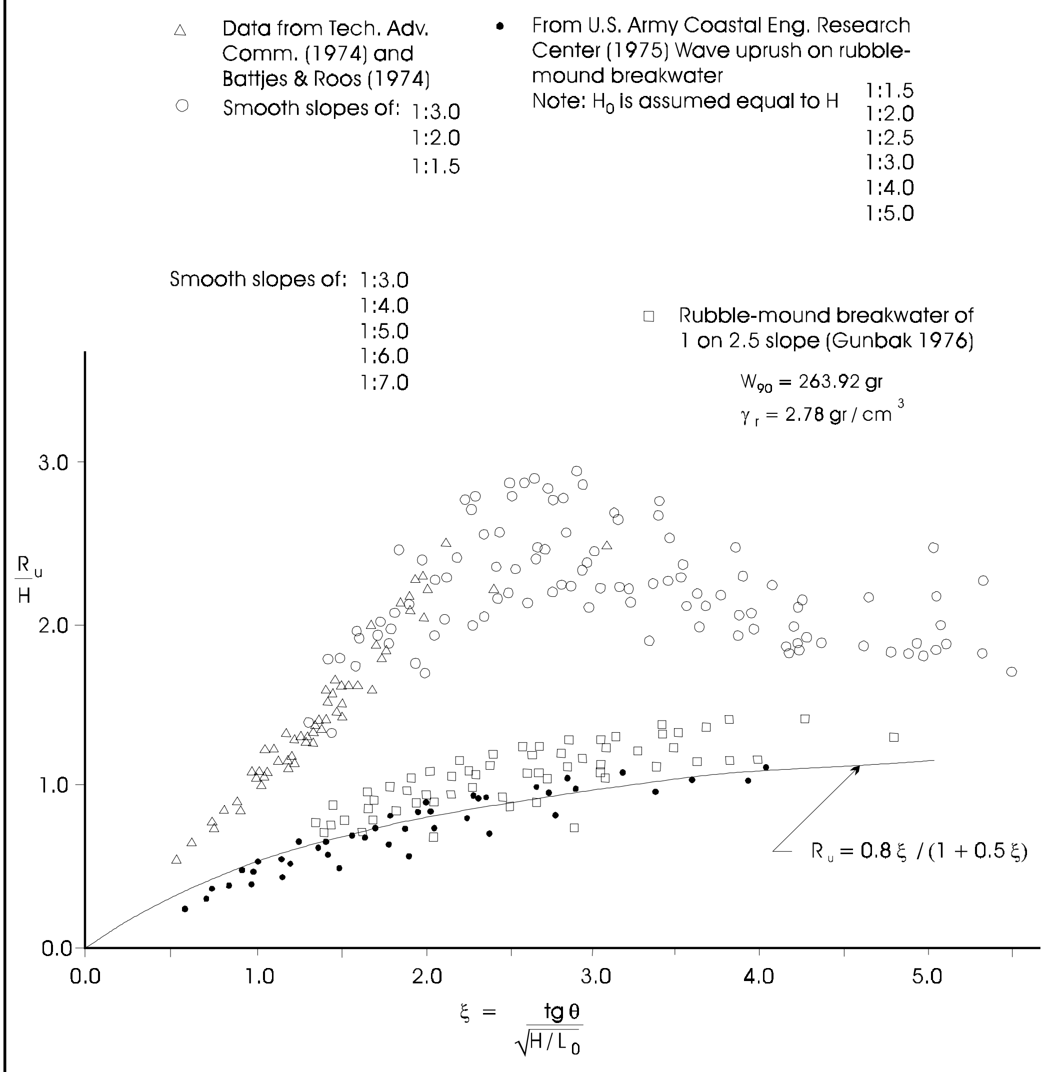
Due to a paucity of data on comparisons between uprush on smooth slopes and uprush on rough slopes under similar wave and bathymetric conditions, information on uprush reduction factors is inadequate to provide more than an approximation of the reduction offered by a rough (and possibly limited permeability in the case of stone on a hard surface) surface. This inadequacy may be compensated by the use of roughness uprush reduction factors "on the high end of the laboratory measurements because of the present lack of confidence in extrapolating such information to field conditions" (Walton et al. 1989a). However, Figure 13 shows that uprush on rough slopes is quite a different process than on smooth slopes, so that an expression based on the concept of a reduction factor multiplier (to smooth slope expressions) would not predict well uprush on rough slopes.

Many of the uprush predictive methods are based on regular waves, smooth impermeable slopes and relative depths of $d_s/H \geq 3$. However, most protective structures along the Great Lakes - St. Lawrence River System shorelines have rough, permeable slopes and are subjected to irregular waves that break at or near the toe of the slope (i.e., $d_s/H < 3$). Relatively few laboratory studies have been completed for these conditions.

To use many of the available wave uprush methodologies, for the design of shallow water flood protection structures, requires the local (transformed) wave height, H, at the toe of the structure. This would introduce the need to perform nearshore wave transformations for uprush predictions, which complicate matters due to the fact that several methods are in existence for the computation of wave climate in shallow waters.

At present, the computation of wave uprush on composite slope structures has not been supported by any analytical method, rather, it is based on approximations and experience for varying structure or beach characteristics, wave climate, and site specific bathymetry. This is more important in the case of dynamic or reshaping structures, where the width of the structure vary with wave action. In the cases of complex structure designs, a physical model testing may be the only way to assess wave uprush and overtopping for the flood protection structure. Physical model tests of wave uprush and overtopping on shoreline protection structures are generally expensive. The decision regarding the need for model tests should be made on a site by site basis, depending on the costs of the particular project, and on the potential risk of loss of life and damages.

Figure 21: Wave Uprush on Smooth and Permeable Slopes (after Bruun 1985)



3.0 WAVE OVERTOPPING METHODOLOGY

As it is with wave uprush, the present understanding of wave overtopping processes is very limited. It is not yet possible to predict wave overtopping from an entirely theoretical basis. The complex processes which govern wave interaction with structures can not yet be fully explained and put into the form of mathematical equations. The parameters that govern wave uprush (i.e., including the characteristics of the incoming waves and the shore protection structure, see Section 2.0) are also factors in wave uprush. Then, the movement of the overtopping water plume over the shoreline or structure, and onto the onshore area, also becomes a function of such factors as air entrainment, wind speed and direction, shore elevation and slope, ground friction and degree of saturation and permeability of the in-place soils and ground cover. A detailed literature review of the most common wave overtopping prediction methodologies is provided in this section.

3.1 Literature Review of Overtopping Processes

. Ahrens and Heimbaugh (1988b)

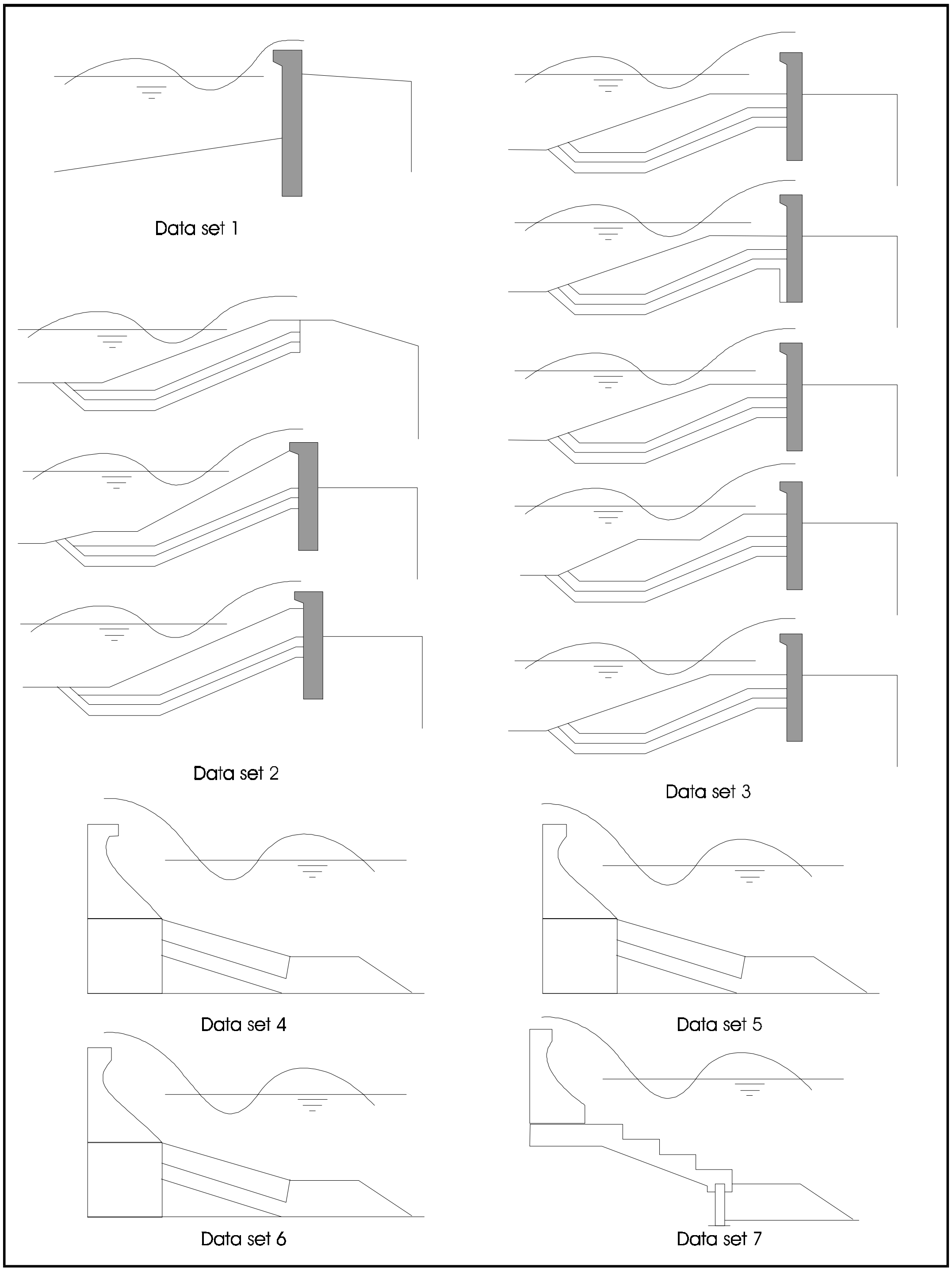
Ahrens and Heimbaugh (1988b) conducted an analysis of a series of laboratory flume tests of irregular wave overtopping for a number of seawall and seawall/revetment configurations. Data for 13 configurations was collected and grouped into 7 data sets representing relatively similar geometrical characteristics (i.e., 7 seawall/revetment configurations were classified, see Figure 22). Ahrens and Heimbaugh expressed the overtopping rate as an exponential function of a dimensionless freeboard parameter, F', regardless of whether the overtopping rate was expressed as a dimensional or dimensionless variable. F' is defined as:

$$F' = \frac{F}{(H_{mo}^{2} L_p)^{1/3}}$$

where F is the freeboard, the average vertical distance form the mean local water level to the crest of the seawall, H_{mo} is the energy based zero-moment wave height either measured near the structure or at the structure toe and L_p is the local Airy wavelength calculated at the or near the toe of the structure using the wave period of the energy spectrum T_p.

It is suggested that the use of F' is efficient because it contains, in one term, information about the water depth, structure height, and wave conditions. That is, F' can consolidate all of the overtopping data for similar structure configurations into a single, well defined trend. Ahrens and Heimbaugh claim that as wave conditions become relatively more severe, a point is reached where details of the structure's geometry seems to have little influence on the overtopping rate. This point occurs when a combination of a high water level and large waves causes the structure to be virtually swamped or inundated by wave action. Functionally, Ahrens and Heimbaugh claim that inundation occurs when $F' \leq 0.3$.

Figure 22: Seawall Structures - Datasets 1 to 7 (after Ahrens and Heimbaugh 1988)



Ahrens and Heimbaugh (1988b) proposed three overtopping models of increasing complexity:

$$Model\ 1 \quad Q = Q_o e^{(C_1 F')}$$

$$Model\ 2 \quad Q' = Q'_o e^{(C_1 F')}$$

$$Model\ 3 \quad Q' = Q'_o e^{(C_1 F' + C_2 X_2)}$$

where Q is the overtopping rate in $m^3/m \bullet s$ and the dimensionless overtopping rate Q' is expressed as a function of g, acceleration of gravity, and overtopping rate, Q, by:

$$Q' = \frac{Q}{\sqrt{gH_{mo}^3}}$$

Q_o is an overtopping coefficient having the same units as Q. Overtopping coefficients Q'_o, C_1 and C_2 are dimensionless coefficients determined by regression analysis. The term X_2 in Model 3 can be any one of several dimensionless variables which improves the predictive ability of Model 3 over Model 2. Table 11 presents all the above coefficients for the seven different types of structures that were initially grouped.

Each of the models has certain advantages and disadvantages. Model 1 is expressed in dimensional units which can be directly related to potential flooding, level of damage, or levels of danger. For a variety of data sets, model 3 provides the best predictions of the three models, however, the secondary variable X_2 is a variable parameter dependent on the type of structure and geometric conditions. One of the data sets provided the best regression coefficients when the relative freeboard F/d_s was used as the secondary independent variable. Other data sets (i.e., structure geometries) provide better correlation when the secondary variable $\sqrt{(H_{mo}/L_o)}$ is used where L_o is the deep-water wavelength using peak wave period T_p. Furthermore, Ahrens and Heimbaugh state that it is not always clear why one secondary variable provided better results in Model 3 than others, leaving the process rather arbitrary. Therefore, Model 3 was not regarded as producing an overly general formula. Of perhaps significant importance, Ahrens and Heimbaugh report that small changes in geometry of a seawall/revetment configuration can have an important influence on the overtopping rate, but this is difficult to properly account for in a simple overtopping model.

Table 11: Overtopping Coefficients Summary Table (after Ahrens & Heimbaugh 1988)

Data Set No.	Model	Regression Coefficient	Overtopping Variables	No. of Observations	Correlation Coefficient
1	1	6.684 -13.586	Q_o C_1	89	0.889
	2	0.212 -10.526	Q_o' C_1		0.90
	3	0.338 -7.385 -2.178	Q_o' C_1 C_2, $X_2 = F/d_s$		0.923
2	1	3.006 -13.091	Q_o C_1	118	0.777
	2	0.1472 -11.138	Q_o' C_1		0.789
	3	0.308 -10.732 -6.629	Q_o' C_1 C_2, $X_2 = \sqrt{(H_{mo}L_o)}$		0.794
3	1	5.454 -16.723	Q_o C_1	111	0.825
	2	0.279 -14.885	Q_o' C_1		0.811
	3	1 -14.371 -11.411	Q_o' C_1 C_2, $X_2 = \sqrt{(H_{mo}L_o)}$		0.841
4	1	36.66 -20.676	Q_o C_1	62	0.93
	2	1 -17.555	Q_o' C_1		0.915
	3	1 -12.69 -20.87	Q_o' C_1 C_2, $X_2 = W_b/L_p$		0.943
5	1	8.663 -14.747	Q_o C_1	57	0.953
	2	0.332 -12.414	Q_o' C_1		0.934
	3	0.541 -11.702 -5.771	Q_o' C_1 C_2, $X_2 = \sqrt{(H_{mo}L_o)}$		0.947
6	1	0.817 -6.334	Q_o C_1	37	0.771
	2	0.0232 -3.791	Q_o' C_1		0.615
	3	1 -7.558 -1.366	Q_o' C_1 C_2, $X_2 = H_{mo}/D_B$		0.918
7	1	23.59 -18.26	Q_o C_1	68	0.927
	2	0.348 -11.232	Q_o' C_1		0.923
	3	1 -11.174 -10.664	Q_o' C_1 C_2, $X_2 = \sqrt{(H_{mo}L_o)}$		0.948

Note Q_o in Model 1 is dimensional while Q_o' in Models 2 and 3 are dimensionless.

. Owen (1982)

Owen (1982) conducted a series of model tests to measure the overtopping discharge for a variety of sea defences and wave climates. The structures tested were of the earth embankment or dyke type, fronted in some cases with a submerged berm (see Figure 23). There were no splash or parapet walls incorporated into these structures. The model tests were carried out at a 1 to 25 geometric scale (with the exception of a few tests) and the following structural and hydrodynamic parameters were varied to assess their influence on wave overtopping:

seawall geometry	●	seaward slopes tested	1:1, 1:2, 1:4
	●	freeboard (m)	0.0, 1.0, 1.5, 2.0, 2.5, 3.0
	●	berm depth below SWL (m)	0.0, 1.0, 2.0, 4.0
	●	berm width (m)	0, 5, 10, 20, 40, 80
wave climate	●	sig. wave height, H_s (m)	0.75, 1.25, 1.75, 2.25, 4.0
	●	wave steepness, H_s/L_o	0.035, 0.045, 0.055
	●	angle of wave attack (degrees)	0, 15, 30, 45, 60

To obtain mean overtopping rates, the overtopped water was measured for 100 average wave periods, five separate times. The reported mean overtopping was simply the average of the five tests, expressed in units of $m^3/m{\bullet}s$.

Owen found (also reported by Ahrens and Heimbaugh (1988)) that a general exponential relationship in the form of:

$$Q^* = A \exp\left(-BF^* / r \right)$$

expressed the overtopping relationship for all test configurations (uniform and bermed smooth slopes) quite well, where the dimensionless freeboard, F^*, and overtopping, Q^*, are expressed as:

$$F^* = \frac{F}{T_z\sqrt{gH_s}}, \quad Q^* = \frac{Q}{T_z g H_s}$$

where H_s and T_z are significant wave height and zero crossing wave period respectively. A and B are dimensionless coefficients which are dependent on the test configuration (structure geometry and wave climate). Values for the coefficients A and B are given in Table 12 for both straight, smooth slopes and bermed, smooth slopes.

Owen found a close similarity in overtopping discharges for seawall slopes of 1:1 and 1:2 for almost all berm dimensions, and noted that this was contrary to published wave uprush data where one would expect lower discharges for slopes of 1:1. The 1:4 slope produced significantly less overtopping than the 1:1 and 1:2 slopes.

Figure 23: Smooth Bermed Revetment (Owen 1982)

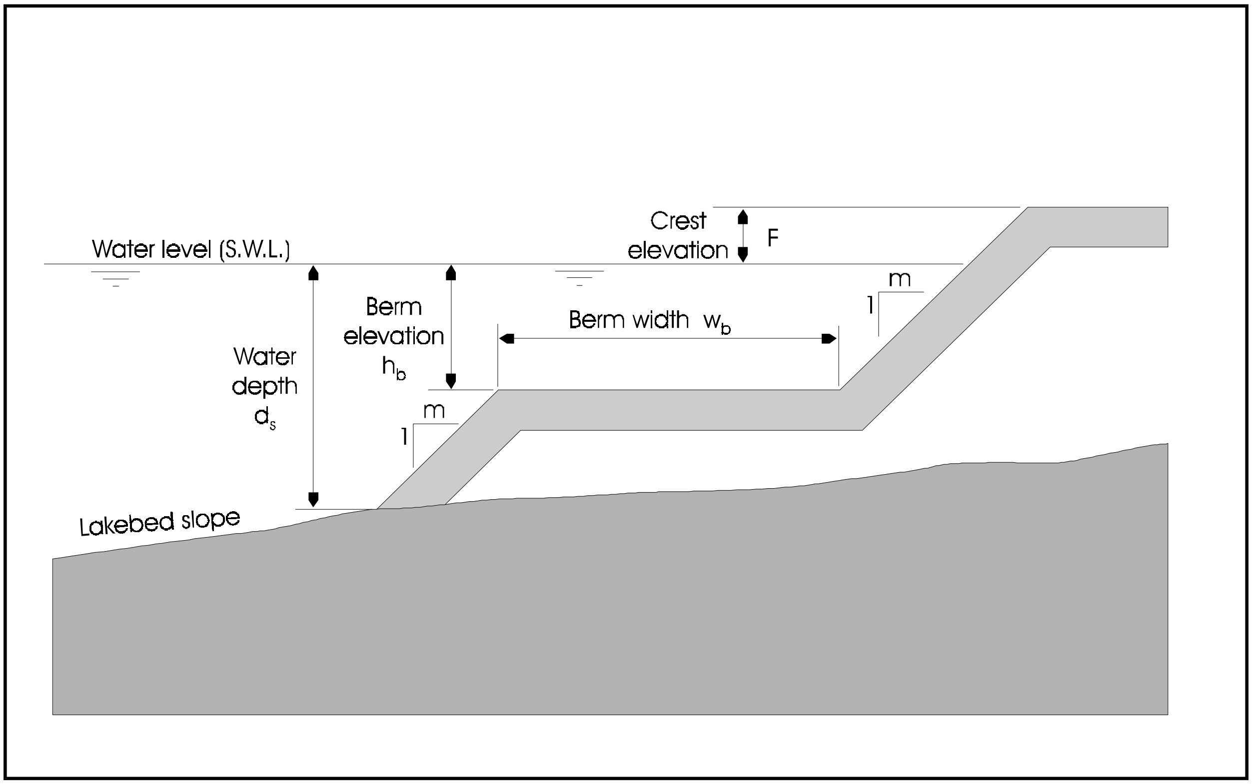


Table 12: Overtopping Coefficients for Owen (1982)

Straight, Smooth Slopes		
Slope	*A*	*B*
1:1	0.00794	20.12
1:1.5	0.0102	20.12
1:2	0.0125	22.06
1:3	0.0163	31.9
1:4	0.0192	46.96
1:5	0.025	65.2

Bermed, Smooth Slopes (see Figure 23)				
Slope	**Berm elevation** h_B	**Berm width** w_B	*A*	*B*
1:1	-4.0	10	6.40×10^{-3}	19.50
1:2			9.11×10^{-3}	21.50
1:4			1.45×10^{-2}	41.10
1:1	-2.0	5	3.40×10^{-3}	16.52
1:2			9.80×10^{-3}	23.98
1:4			1.59×10^{-2}	46.83
1:1	-2.0	10	4.79×10^{-3}	18.92
1:2			6.78×10^{-3}	24.20
1:4			8.57×10^{-3}	45.80
1:1	-2.0	20	8.80×10^{-4}	14.76
1:2			2.00×10^{-3}	24.81
1:4			8.50×10^{-3}	50.40
1:1	-2.0	40	3.80×10^{-4}	22.65
1:2			5.00×10^{-4}	25.93
1:4			4.70×10^{-3}	51.23
1:1	-2.0	80	2.40×10^{-4}	25.90
1:2			3.80×10^{-4}	25.76
1:4			8.80×10^{-4}	58.24
1:1	-1.0	5	1.55×10^{-2}	32.68
1:2			1.90×10^{-2}	37.27
1:4			5.00×10^{-2}	70.32
1:1	-1.0	10	9.25×10^{-3}	38.90
1:2			3.39×10^{-2}	53.30
1:4			3.03×10^{-2}	79.60
1:1	-1.0	20	7.50×10^{-3}	45.61
1:2			3.40×10^{-3}	49.97
1:4			3.90×10^{-3}	61.57
1:1	-1.0	40	1.20×10^{-3}	49.30
1:2			2.35×10^{-3}	56.18
1:4			1.45×10^{-4}	63.43
1:1	-1.0	80	4.10×10^{-5}	51.41
1:2			6.60×10^{-5}	66.54
1:4			5.40×10^{-5}	71.59
1:1	0.0	10	9.67×10^{-3}	41.90
1:2			2.90×10^{-2}	56.70
1:4			3.03×10^{-2}	79.60

The effect of berm elevation varied slightly depending on the exact seawall slope and berm width, but in all cases the major effect occurred as the berm was raised from 2.0 m to 1.0 m below SWL, with only minor further reductions in overtopping discharge when the berm was located exactly at SWL. Owen notes that the effect of berm depth, h_B, on overtopping discharge seemed to be independent of the incident wave height. This implies that h_B/H_s is not a dimensionless parameter governing wave overtopping.

The effect of berm width, w_b, was found to be significant in that the overtopping discharge reduced as the berm width increased. For berms at 2 or more metres below the SWL, the effect of berm width was less marked, but the reduction was still significant.

Perhaps the most significant finding from Owen's work was the effect of angle of wave attack. Contrary to expectations, the highest overtopping discharges were recorded not at normal incidence, but for those waves striking the seawall at about 15° off normal. Owen found that overtopping rate only began to decrease for incidence angles greater than 30°. Owen concluded that overtopping rates can be reduced by raising the crest elevation, flattening the seaward slope, by increasing the berm width, by reducing the depth of water over the berm or by increasing the angle of wave attack beyond 30° from the normal.

Owen's work was based on smooth slopes ($r = 1.0$). Owen suggests that the rough slope surface reduction factor, r, could be used in his equations to obtain an estimate of overtopping for simple armoured slopes. Recommended values of r are provided in Tables 1 and 4. CIRIA/CUR (1991) expects this approach for rough slopes would give conservative results.

Walton et al. (1989b) conducted a review of overtopping predictive techniques and assessed their applicability for the Federal Emergency Management Agency (FEMA). They found that in the United Kingdom, the present state of the art in overtopping (for design) is reflected in publications by Owen (1980; 1982). They note a weakness with the Owen approach is that the coefficients A and B are site and structure specific. They acknowledge that the work conducted by Ahrens and Heimbaugh (1986) used a very similar approach to predict overtopping rates to that of Owen with the primary exception being that the wave parameters used by Ahrens and Heimbaugh are those at the toe of the slope.

. Goda (1985)

Goda (1985) provides a review of the overtopping phenomenon with design information. He points out, as do others (e.g., Ahrens and Heimbaugh, 1988b), that the primary parameter influencing the degree of overtopping is the absolute height of individual waves relative to the crest elevation of the seawall. An important feature of wave overtopping of prototype seawalls is the random nature of the event. That is, overtopping is not a continuous process but an intermittent occurrence of individual high waves among a series of waves. Goda concludes that if regular wave data is used to compute overtopping of a seawall, by interpreting the regular wave height as equivalent to the significant height, the error introduced in the estimate of the overtopping rate may be large. Of specific importance, for a seawall with a relatively high crest elevation, the overtopping rate will be underestimated when using regular wave data, because the estimation ignores the existence of individual waves higher than the significant wave.

Goda compiled design charts to estimate wave overtopping rates of vertical and revetment type seawalls (see Figures 24 and 25 respectively) based on irregular wave data. Others, as discussed later, claim that Goda's design curves may be based on regular waves. Goda's data is based on typical shapes of seawalls (vertical walls jutting directly up from the seabed) and "revetment seawalls" (sloping mounds of rubble stones and concrete blocks fronting vertical walls) and two foreshore slope conditions; 1:10 and 1:30.

Goda's method is summarized by a series of dimensionless overtopping graphs with the following form:

$$\frac{Q}{\sqrt{2gH_o'^{3}}} = f\left(\frac{H_o'}{L_o}, \frac{F}{H_o'}, \frac{d_s}{H_o'}\right)$$

Three dimensionless parameters are required to determine overtopping rates. First, there is a family of curves for a given wave steepness H_o'/L_o, where H_o' is the unrefracted wave height and L_o is the deep-water wavelength. Each family of curves represents a range of relative crest heights, F/H_o', and the abscissa for each family is relative water depth d_s/H_o'.

If either the wave steepness, relative crest height or the bottom slope differs from the specifics listed in the design charts, interpolation or extrapolation becomes necessary. In general, if the bottom slope is gentler than 1 on 30, the wave overtopping rate in water shallower than $2H_o'$ becomes less than that given by Figure 24(b).

The wave overtopping rate for a revetment (block mound) seawall has additional design parameters based on the geometric details of the mound in front of the seawall, the main parameters being the width and height of the mound. However, the compilation of generalized design diagrams for the overtopping rate of block mound seawalls is more difficult than for the case of vertical revetments. Considering this, Goda (1985) proposed design diagrams for blocked seawalls with two block units (tetrapods) across the top and a steep front slope (1 on 1.5).

The design curves are for specific seawall geometries, and Goda emphasizes that even a small modification of the seawall geometry may change the amount of wave overtopping. That is, wave overtopping is very sensitive to structure shape. In fact, Goda claims that wave overtopping can be reduced to zero by redesigning the parapet into a curved shape, if the seabed conditions and the frontal shape of the seawall are suitable. In the case of the block mound seawall, overtopping can be reduced to some extent by replacing the entire volume of rubble stones in the underlayer and the core of the mound with concrete energy dissipating blocks. On the other hand, a sloped seawall with a smooth surface, which is typical of coastal dikes, usually has a greater rate of wave overtopping. Goda provides modification curves for block mound seawalls to account for variable front slopes and widths of the mound crest. It is important to recognize that the design curves provide overtopping rates that have been averaged over a long period of time in comparison to the wave period of individual waves. In a shorter time interval, a much larger amount of wave overtopping than estimated by the design curve might occur due to wave grouping.

Allsop (1986) and Douglass (1984) suggest that Goda's results appear to be based on regular wave testing. Walton et al. (1989b) note that Goda's statement that his charts "were prepared on the basis of irregular wave tests and calculations of wave deformation in the surf zone" does not appear to be consistent with his use of the monochromatic wave parameter H_o'.

Figure 24a: Design Diagrams of Wave Overtopping Rate of Vertical Seawalls on a Lake Bottom Slope of 1:10 (after Goda 1985)

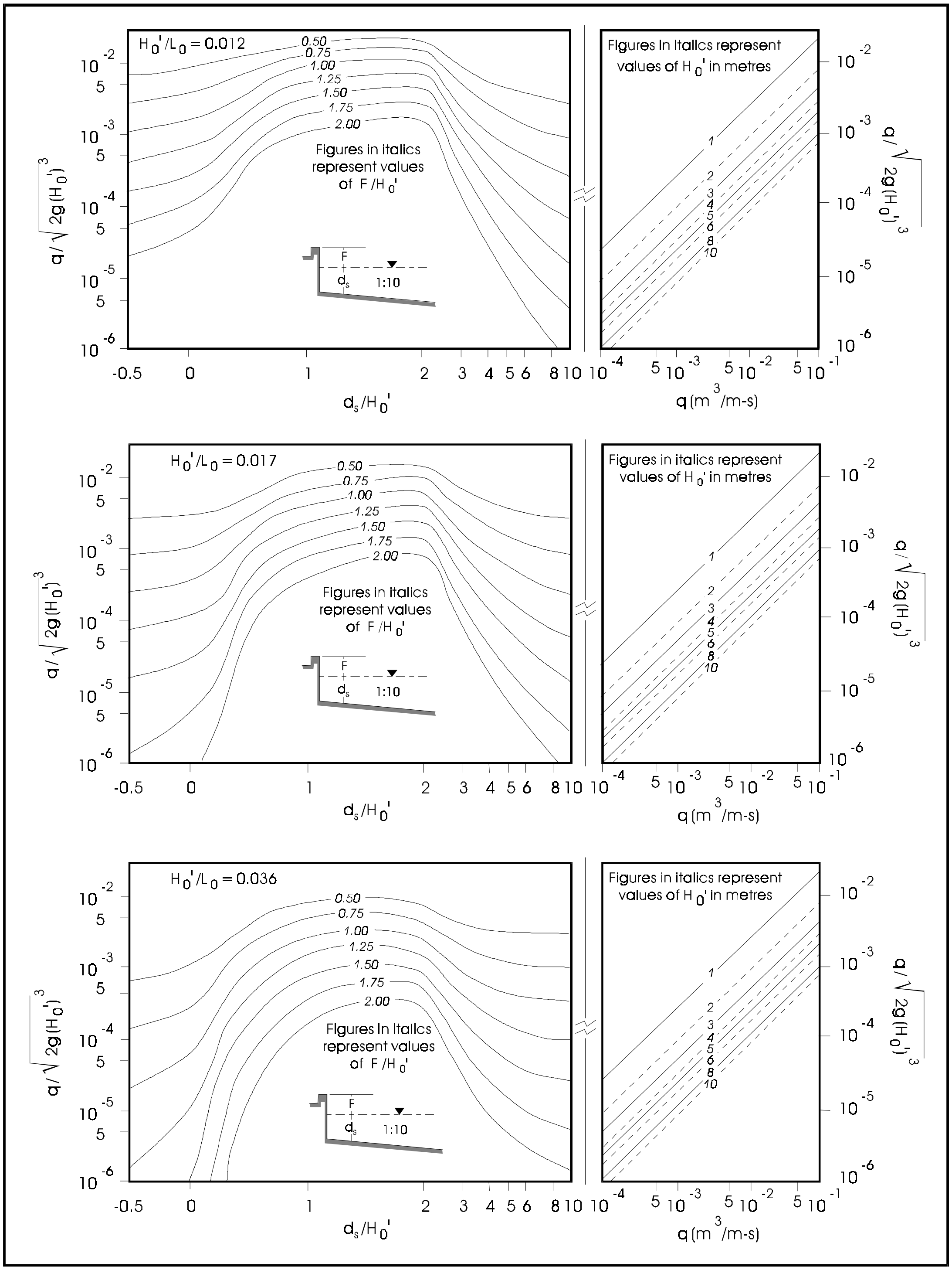


Figure 24b: Design Diagrams of Wave Overtopping Rate of Sloping Vertical Seawalls on a Lake Bottom Slope of 1:30 (after Goda 1985)

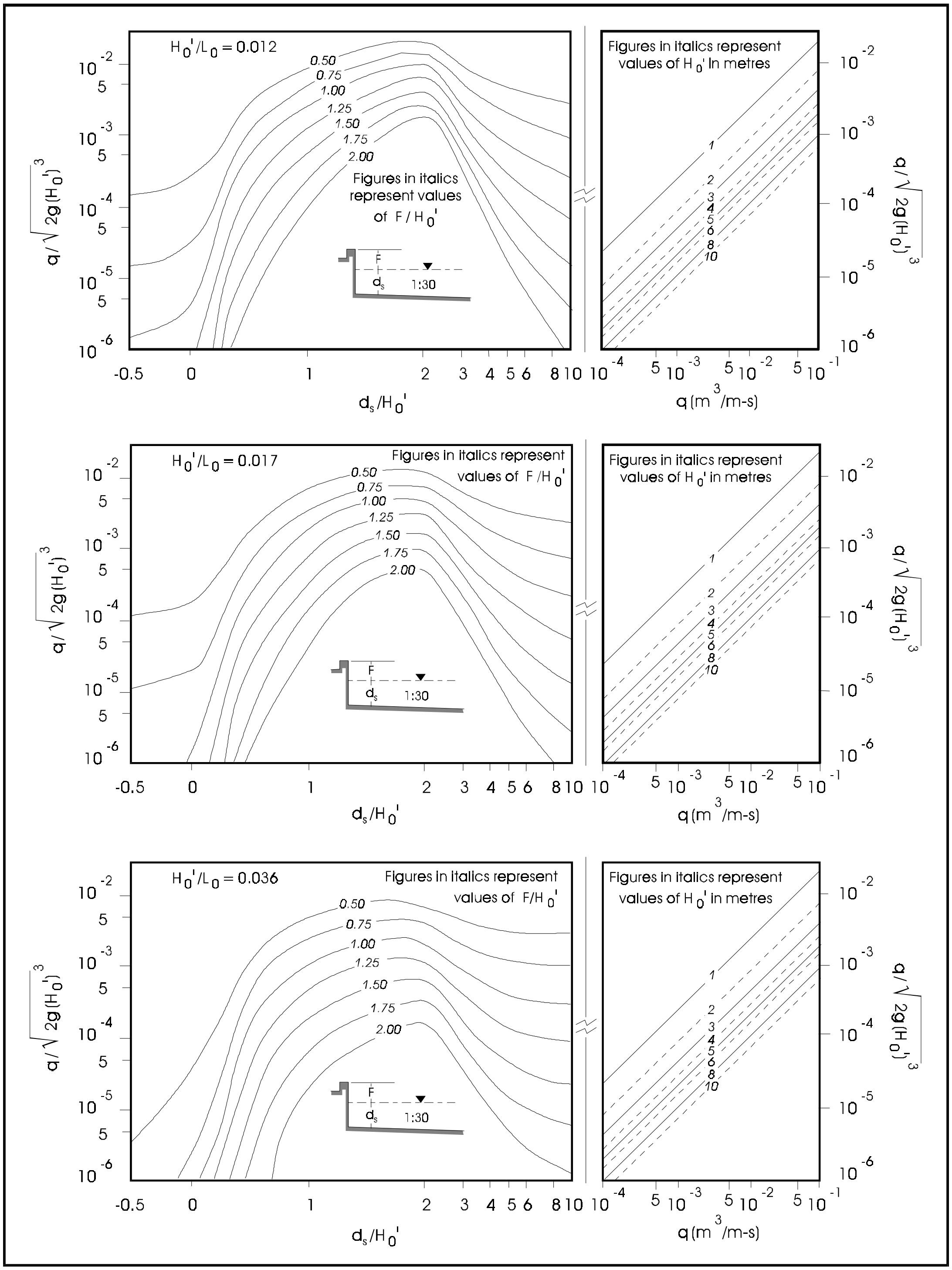


Figure 25a: Design Diagrams of Wave Overtopping Rate of Sloping Revetment Seawalls on a Lake Bottom Slope of 1:10 (after Goda 1985)

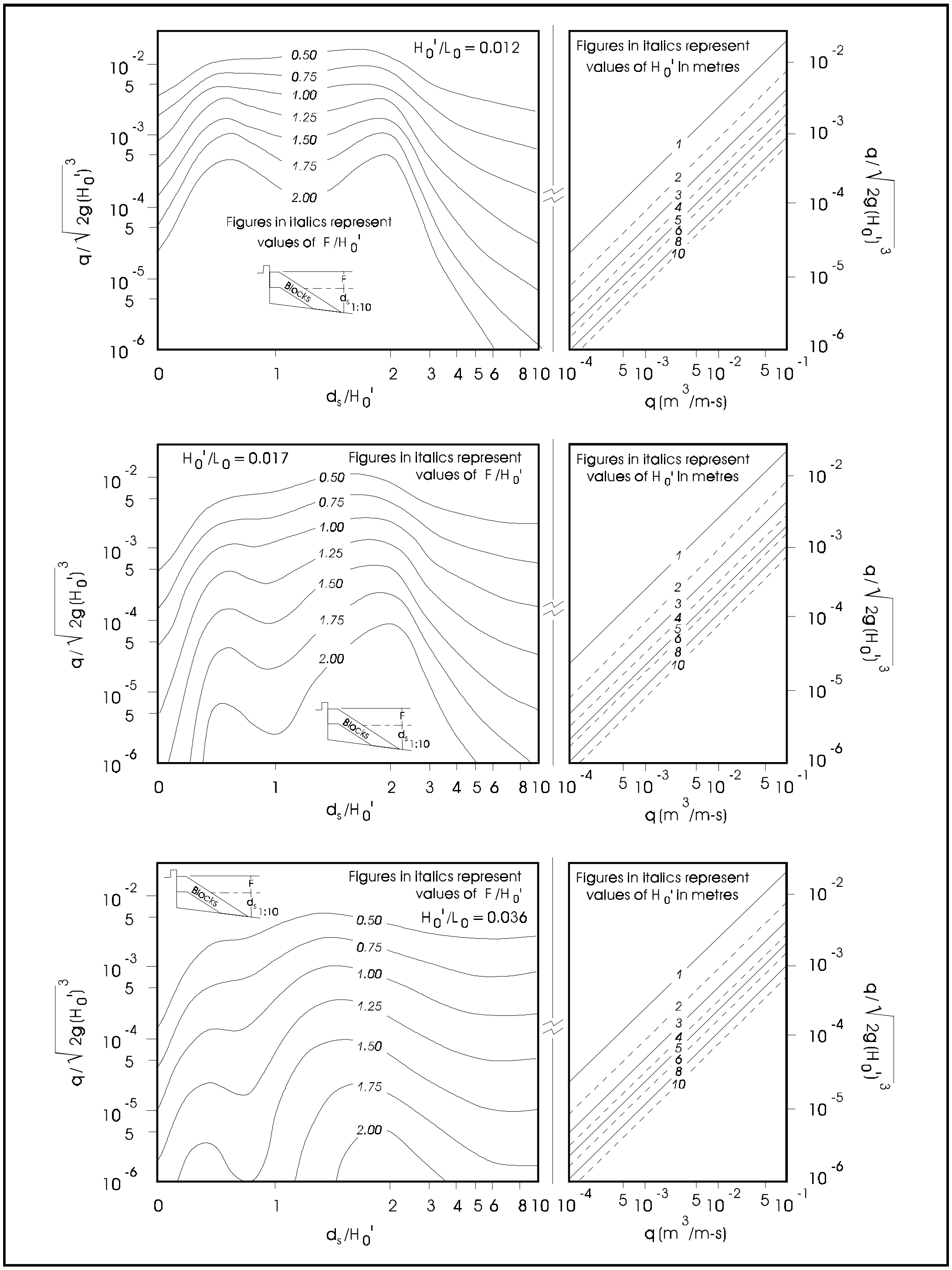
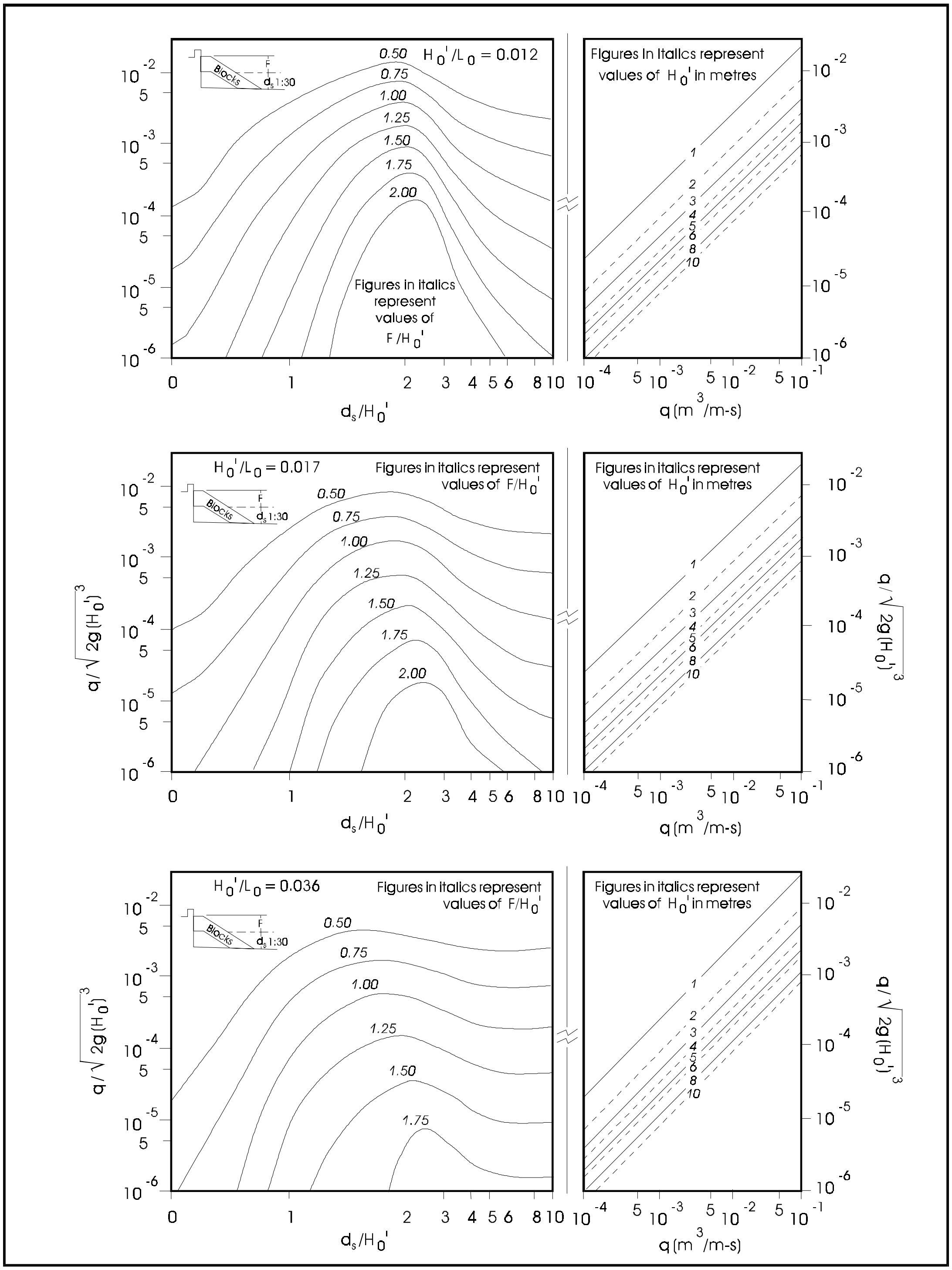


Figure 25b: Design Diagrams of Wave Overtopping Rate of Sloping Revetment Seawalls on a Lake Bottom Slope of 1:30 (after Goda 1985)



. Shore Protection Manual (U.S. Army Corps of Engineers, 1984)

The Shore Protection Manual (SPM) (U.S. Army Corps of Engineers 1984) provides a technique based on the work of Weggel (1976) and is presented in the form of the following equation:

$$Q=\sqrt{g\ Q_o^*\ H_o'^3}\ \exp\left[\ -\left(\frac{0.217}{\alpha_1}\right)\ \tanh^{-1}\left(\frac{F}{R}\right)\right]$$

where Q_o^* and α_1 are fitted empirical coefficients, F is the structure freeboard and R is the wave uprush as defined in the SPM (USACE 1984, see Section 2.1). It is a monochromatic wave approach to calculate wave overtopping and relies on a semi-empirical nonlinear equation in which two coefficients must be fitted. Additionally, the use of uprush data to fit empirical coefficients of the equation provides additional uncertainty in estimating the overtopping. The method also uses deep-water wave conditions as input to the model, thus ignoring the possible inaccuracies of the nearshore wave transformation. Kobayashi and Raichle (1994) found that the SPM method significantly under-predicted the average overtopping rate when compared to test results. Douglass (1984), as discussed later, found from limited data that the SPM method likely under-predicts overtopping.

Methods suggested by Ahrens (1983) are used in the SPM to extrapolate for random waves. It is argued that the overtopping discharge for a sequence of random waves may be given by summing the overtopping contribution of individual run-ups. It is assumed that wave uprush levels fit a Rayleigh probability distribution. This method embodies a number of fairly significant assumptions, and in some instances correction factors are proposed.

. Jensen and Juhl (1987)

Jensen and Juhl (1987) present the results of their experience with wave overtopping from model tests and comparisons with prototype data. Field data is scarce and Jensen and Juhl studied two field data sets: Fukuda (1974); and DHI measurements at the breakwater at the Port of Hundested (1977, no reference provided). Supplementary model tests, at scales of 1:8 and 1:10, were made on the same breakwater and the model measurements compared well.

Both in the model and in prototype, measurement of the overtopping rate, Q, was determined by collecting the amount of overtopping water in separate trays placed at different distances behind the breakwater. In this way not only the total overtopping quantities were determined, but also the intensity of water falling as a function of the distance from the breakwater. The results showed that the overtopping varied from structure to structure, but some general conclusions were derived:

1) The amount of overtopping increases rapidly with the parameter of H_s/F. The logarithm of Q is almost a linear function of H_s/F.

2) The influence of the wave period is very different from structure to structure.

3) No clear delineation exists between the wind carried spray and mass overtopping of green water.

Jensen and Juhl found that the intensity of overtopping behind a breakwater decreases very rapidly with the distance from the breakwater. They report that in all the tests performed, as well as in the available prototype measurements, that on average the intensity of overspill decreases exponentially with the distance *x* from the breakwater, expressed by:

$$q(x) = q_o 10^{-(x/\gamma)}$$

where q is the intensity at a distance x, and q_o is the intensity for x=0. The parameter γ is a constant and equal to the distance for which the overspill intensity decreases by a factor of 10. As such, the total amount of overtopping Q may be calculated by integrating the above equation which results in the following formula:

$$Q = q_o \gamma / ln10$$

The parameter γ is reported to be independent of both wave and wind condition. The only exception to this is the overspill behind breakwaters with a high parapet wall where the intensity close to the wall is more evenly distributed before the exponential decrease begins. Jensen and Juhl note that the ratio γ/B (B is the horizontal distance from the point where the armour layer intersects with the SWL to the limit of the reclamation or to the rear side of the crown wall) seems to be rather constant in the range of 0.40 to 0.70.

In addition to the horizontal distribution of overtopping behind a breakwater, Jensen and Juhl also investigated the overtopping discharge of individual waves and found this to be very important. It is interesting to note that although average intensities of wave overtopping are commonly used to set structure dimensions based on acceptable criteria, it is not the average intensity that determines the level of inconvenience or danger of overtopping waves. Further discussion is provided in Section 4.7.

. Pilarczyk (1990)

Pilarczyk (1990) provides a review of coastal protection encompassing many aspects of the subject including the computation of overtopping. For straight and relatively smooth slopes, the overtopping can be roughly approximated by the following equation:

$$Q'' = \exp^{-2.5F''}$$

where the dimensionless overtopping rate Q'' and freeboard parameter F'' are expressed as:

$$Q'' = Q T_m \frac{\sqrt{(\cot\theta)}}{0.1 H_m L_o}$$

$$F'' = \frac{F \cot\theta}{\sqrt{H_m L_o}}$$

where H_m is the mean wave height, value exceeded by 50% of the wave heights (approximately 0.625 H_s), and T_m is the mean wave period and θ is the structure front slope angle with respect to the horizontal.

. Bradbury et al. (1988)

Bradbury et al. (1988) conducted two tests of wave overtopping on rock armoured seawalls without crown walls and revised Owen's (1982) parameter of relative freeboard to give:

$$F^{///} = \frac{F}{H_s} \times F^{*} = \frac{F^2}{T_z\sqrt{gH_s^3}}$$

Overtopping predictions can then be made using:

$$Q^{*} = A\,(F^{///})^{-B}$$

Values of *A* and *B* have been calculated from the results of tests with a rock-armoured slope at 1:2 with crest details shown in Figure 26.

. Walton et al. (1989b)

Walton et al. (1989b) conducted a review of overtopping predictive techniques and assessed their applicability for the Federal Emergency Management Agency (FEMA). They concluded that none of the existing methods to calculate overtopping were sufficient to adequately address the desirable qualities (from a FEMA point of view) for a predictive overtopping method. This is because FEMA required the ideal overtopping prediction equation to be based on a depth limited wave height criteria.

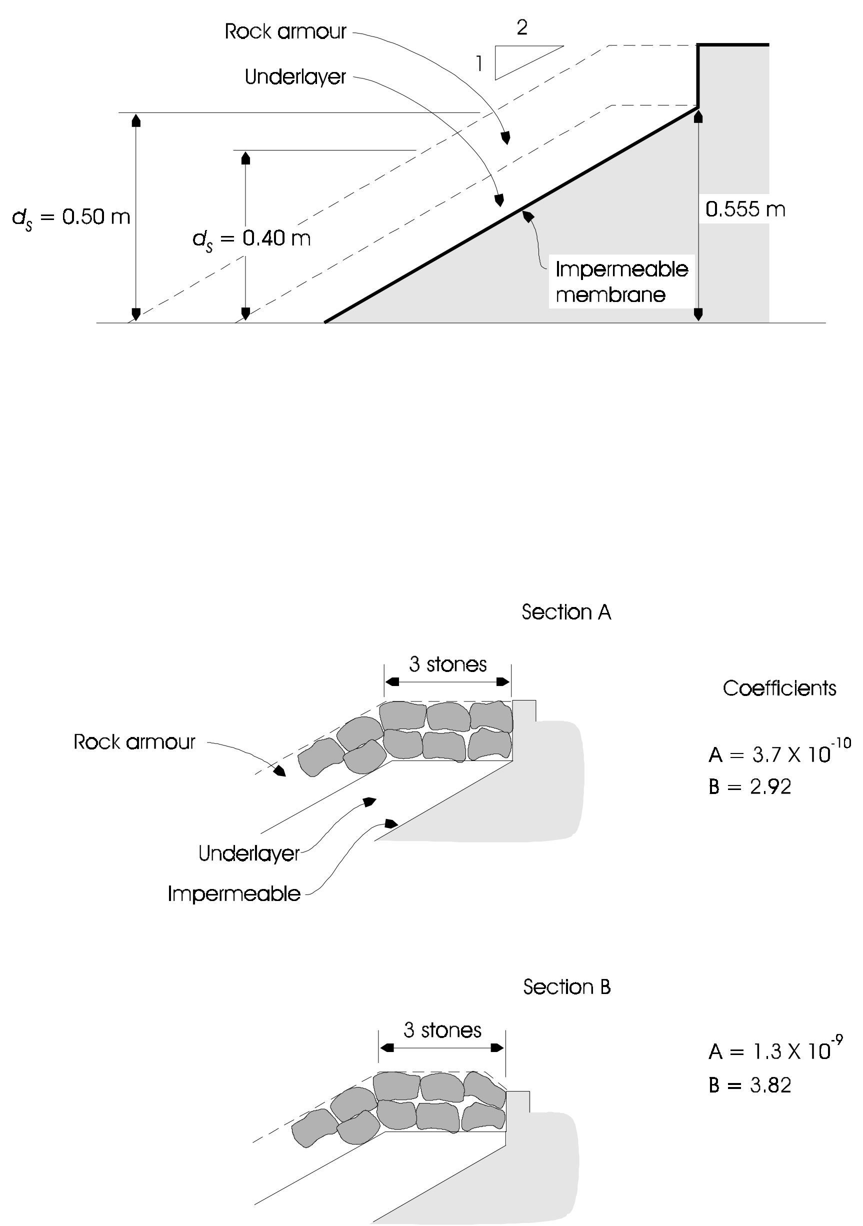
Comments of interest applicable to other cited references are duly noted. Here, comments by Walton et al. (1989b) are presented for those methodologies which were not reviewed in detail for this report.

<u>Kikkwawa et.al. (1968)</u> proposes another Japanese method in addition to Goda's which is based on an extension of a steady state weir flow equation. By extending the method to the dynamic (unsteady) case and assuming a triangular wave form, a solution was proposed of the form:

$$Q = H_o\sqrt{2gH_o}\left(\frac{2}{15}\right)\, mk^{3/2}\left(1 - \frac{F}{kH_o}\right)^{5/2}$$

where *k* is a dimensionless coefficient fit to data and *m* is a discharge coefficient (assumed = 0.5), H_o is the deep-water wave height, and *F* is the freeboard.

Figure 26: Overtopped Rock Structures with Low Crown Walls (after Bradbury et al. 1988)



Pappe (1960) presents some Dutch methods for estimating overtopping rates. These methods are limited to a narrow range of tests with slopes typical of dikes built in The Netherlands. They are comprised of plane smooth slopes (from 1 on 2 to 1 on 8) with horizontal foreshore which use irregular waves as input parameters. In dimensionless form, irregular wave overtopping is expressed as:

$$\frac{2\pi Q T}{H_m L} = f\left(\frac{\cot^{3/2}\theta}{H_m}\right)$$

where T is the average wave height, H_{50} is the median wave height, L is the first order wavelength and θ is the slope of the structure. The study was conducted in a wind-wave flume where there was no control of the generated spectra and as such may not resemble true wave spectra.

Battjes (1974b) proposed a semi-empirical equation for calculation of overtopping based on a limited set of monochromatic wave, smooth linear slope data taken at Delft. The equation is in the form:

$$Q\,T = H\,L_o\,(\tan\theta)\,A\left(1 - \frac{F}{R_h}\right)^2$$

where A is an empirically fitted coefficient ($\approx$ 0.1) and R_h is Hunt's (1959) uprush expression for breaking waves. This equation applies to smooth linear slopes (1:3 to 1:7) and monochromatic breaking waves.

. Bishop et al. (1985)

Bishop et al. (1985) measured wave overtopping of a 1:50 scale concrete caisson retained artificial island and compared the results to data collected at a prototype island. Model to prototype comparison ratios produced large discrepancies ranging from 2.0 to 6.4 depending on the storm stage. By varying parameters such as freeboard, incident wave height, incident wave period, wind vs. no wind and mean wave direction, overtopping parameter influences were studied. Bishop et al. attribute the discrepancies between model and prototype results to be primarily the result of modelling a three dimensional sea state (short crested waves) with two dimensional (long crested) waves in the laboratory. They showed that for incident wave trains of the same energy, unidirectional seas will result in more overtopping than multidirectional seas.

Bishop et al. concluded that the wind speed scaling is also important to the modelling process, but not a major reason to account for discrepancies between model and prototype data. Further conclusions are that overtopping rates are sensitive to freeboard and to wave period and/or groupiness factor and that errors in modelling wave direction do not account for the large discrepancies between model and prototype data.

Finally, Bishop et al. performed a brief analysis on the spacial distribution of spray as waves overtop the structure. They found that the model experienced a more rapid decrease in overtopping volumes with increased distance from the caisson wall than did the prototype. This was attributed to surface tension scale effects in that the terminal water drop size in model and prototype were approximately the same, thus the vertical component of velocity of spray was also the same in model and prototype resulting in a more rapid decease in spray trajectory in the model.

. Douglass (1984)

Douglass (1984) estimated wave overtopping using a variety of predictive techniques and made comparisons between results for similar structural situations. For vertical walls, Douglass compared SPM and Goda and found that the results were dependent on the relative depth d/H_s For relative depths of 3 and 1.5, the SPM method estimated more overtopping than Goda (Figure 27), however for a relative depth of 0.75, both methods yielded similar results. For very shallow water (i.e., relative depth 0.4), the SPM method estimated less overtopping that Goda's method. Douglass notes that the dependence on relative depth implies a dependence on wave breaking and appears to be a result of different approaches used to extrapolate monochromatic wave overtopping results to irregular waves. Goda (1985) however claims that his method is based on irregular wave tests. It is also important to note the influence of relative freeboard (F/H_s) on overtopping, in particular how either a slight decrease in freeboard or a slight increase in wave height causes a significant increase in overtopping.

For mildly sloping and smooth structures, Douglass found that Battjes predicts more overtopping than SPM (Figure 28). For dyke type seawalls, Douglass reports that Owen predicts larger values than SPM for 1:3 smooth slopes (see Figure 29) and for 1:1.5 smooth and rough slopes. When compared to a limited amount of wave overtopping field data collected by Aaen (1977) from a breakwater in Denmark (Figure 30), Douglass found that SPM under-predicts and Owen over-predicts Aaen's field data. Douglass explains the discrepancies as being the result of the unverified wind correction factor, and the ignored differences in slope and stone.

Fukuda et al. (1974) measured actual overtopping rates at a seawall fronted by artificial concrete blocks. Douglass reports that a comparison between this data set and Goda's method shows a significant over-prediction by Goda's method of between one and two orders of magnitude. Fukuda attributes this to the mild (and energy dissipative) offshore slope of 1:80 in prototype (Goda's method being applicable for a 1:30 offshore slope).

. Kobayashi and Raichle (1994)

Kobayashi and Raichle (1994) conducted overtopping tests of irregular waves over a revetment, located in the surf zone (i.e., $1.29H_{mo} \leq d_s \leq 1.66H_{mo}$), with a rough, impermeable 1:2 slope. Measured overtopping probability and average overtopping rate (see Figure 31) were found to be influenced by the spectral shape and wave grouping of the incident waves measured immediately outside the surf zone. The SPM procedure was found to under-predict the average overtopping rate yielding only order of magnitude estimates (see Figure 32). The measurements were also used to calibrate and evaluate a numerical model.

Figure 27: Comparison of SPM vs. Goda - Overtopping of a Vertical Wall (after Douglass 1984)

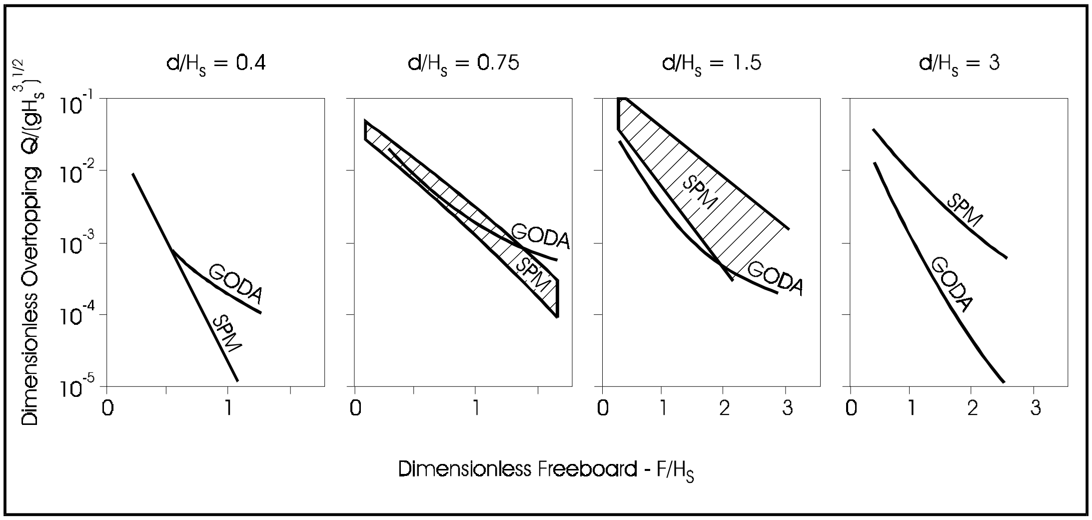


Figure 28: Comparison of Battjes vs. SPM - Overtopping of a 1:6 Smooth-slope Structure (after Douglass 1984)

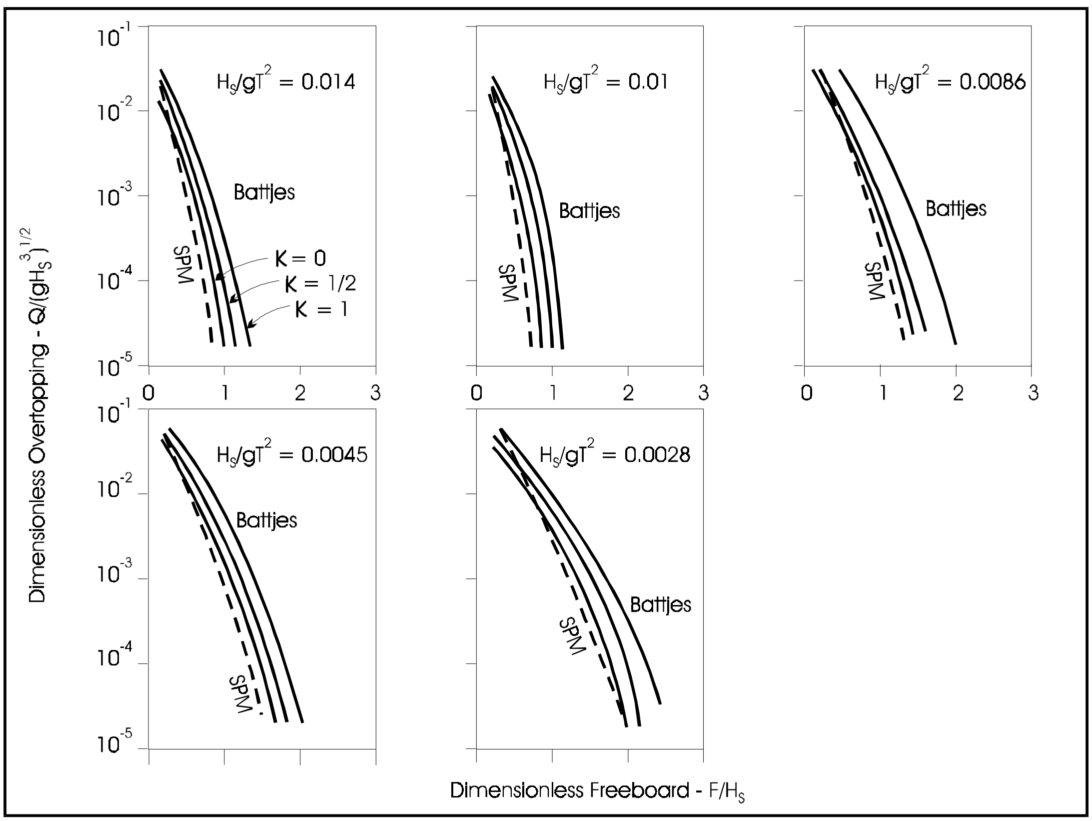


Figure 29: Comparison of Owen vs. SPM - Overtopping of a 1:3 Smooth-slope Structure (after Douglass 1984)

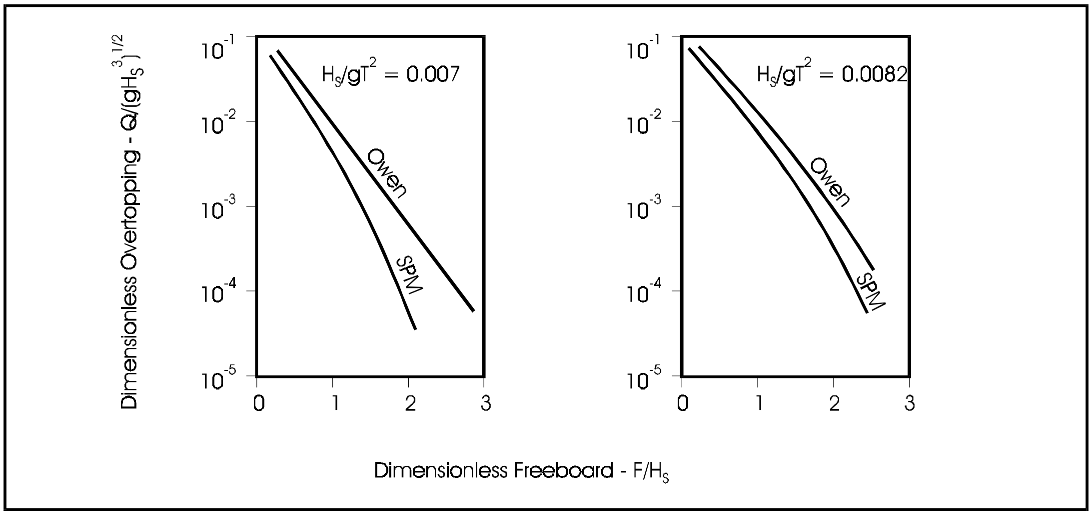


Figure 30: Comparison of Owen vs. SPM - with Field Data (after Douglass 1984)

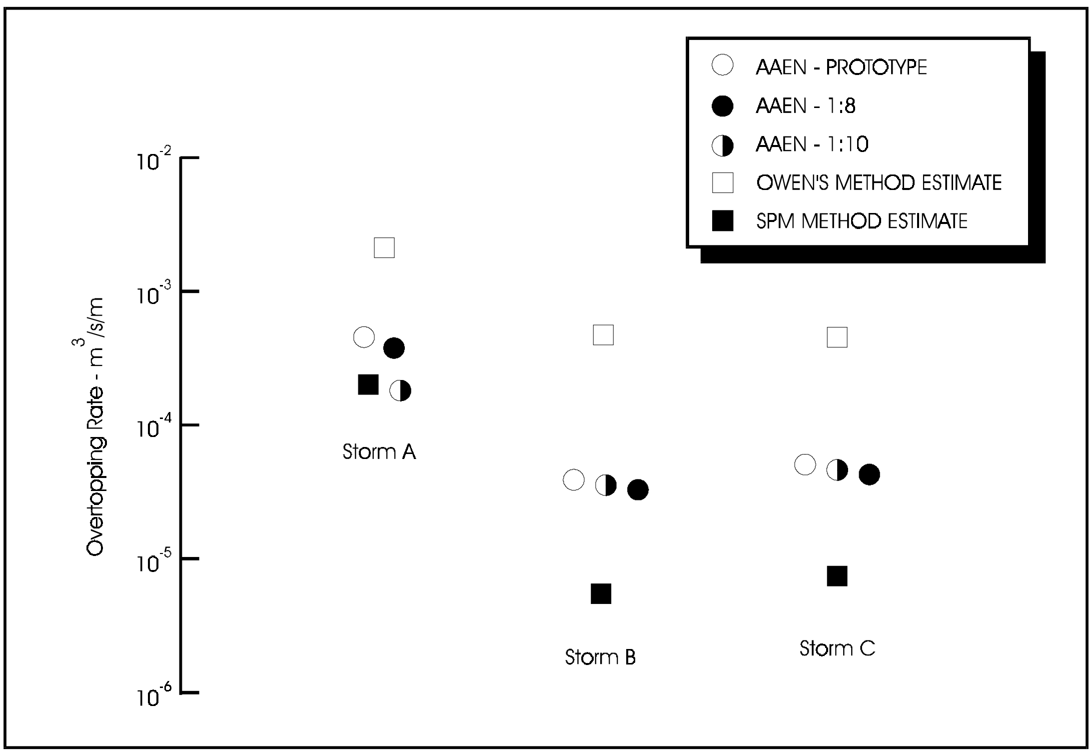


Figure 31: Measured Average Dimensionless Overtopping Rate Q'_m versus Relative Crest Height (after Kobayashi and Raichle 1994)

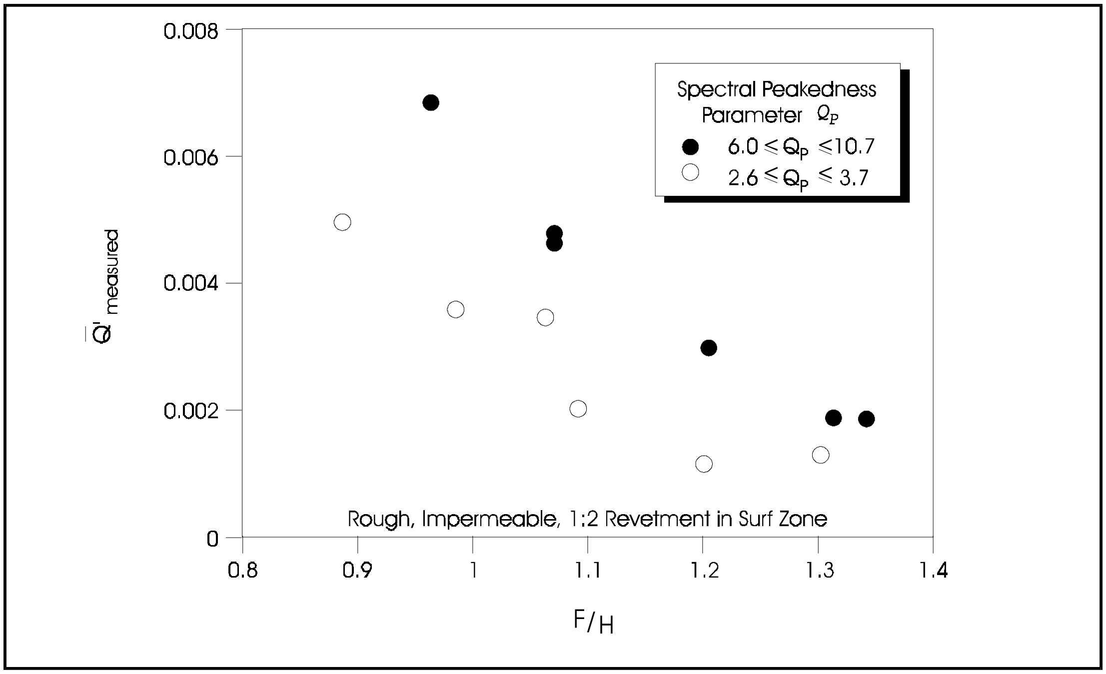
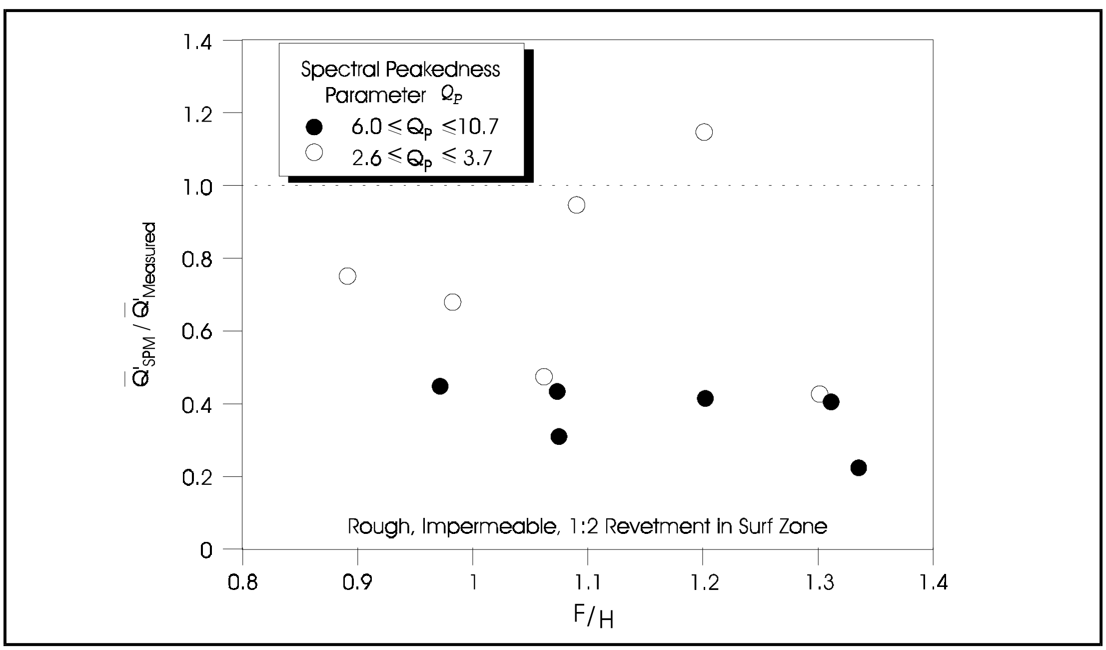


Figure 32: Ratio of SPM Average Dimensionless Overtopping Rate to Measured Dimensionless Rate in the Surf Zone (after Kobayashi and Raichle 1994)



3.2 Accepted Wave Overtopping Methodologies

There is no generic, empirical method to predict wave overtopping due to the limited number of laboratory studies, the variety of structure geometries and materials, and the range of test conditions. However, as discussed in Section 3.1, there is some limited guidance for predicting overtopping of some shoreline structures with simple profiles (i.e., uniform vertical seawalls and sloping armour stone revetments). For these simple structures, the results of a sufficient number of model tests have been analyzed to allow some predictions to be made based on empirical equations.

The following wave overtopping prediction methodologies are presented for vertical seawalls and sloping armour stone revetments:

Vertical Seawalls	Sloping Revetments
Ahrens and Heimbaugh (1988b)	Ahrens and Heimbaugh (1988b)
Goda (1985)	Goda (1985)
	Owen (1982)

These methods are summarized in Table 13. They have been commonly accepted within the coastal engineering industry as being the best estimates available and the most applicable to the types of shoreline structures and environmental conditions of the Great Lakes - St. Lawrence River System.

For the time being, these methods may be considered as accepted practice for estimating wave overtopping for small scale projects and where there is no risk of loss of life or significant property damage. The wave overtopping methodologies are applicable where a strong similarity exists between the geometry of the project structure and the geometry of the structure for which the overtopping predictor is based. These methods must be used in the same context on which they are based. The procedures outlined for the various "common" overtopping estimation methods must be followed closely and they should not be extrapolated much beyond the tested conditions.

It would be appropriate, when using one of the methodologies to estimate wave overtopping, for the proponent to provide a brief summary of how the methodology was derived and why it is applicable to the situation under study. This would help to demonstrate the proponents' understanding of the limits of the method used. Also, since most of the methods are based upon physical model tests, the proponent should discuss the differences between the model layout and the site situation and evaluate whether or not any adjustments should be made to the predicted overtopping to account for the differences. This should be valid for both small and large project evaluations, where the former is done with existing guidance and the later with the aid of site-specific model tests.

Other factors such as the local bathymetry (e.g. offshore bars and composite slopes), berms in front of structures, wind and oblique wave attack may change the magnitude of the wave overtopping and must be considered. For larger scale projects, where either loss of life or significant property damage may result from wave overtopping, a detailed physical model study of the site specific conditions should be required. The study should be undertaken by a qualified coastal engineer.

Table 13 Summary of Wave Overtopping Methods

Method	References	Equations and Boundary Conditions	Remarks
Ahrens and Heimbaugh	Ahrens and Heimbaugh (1988b)	For Vertical Seawalls: $Q = 6.684 \exp(-13.586\, F')$ where $F' = \frac{F}{(H_{mo}^2 L_p)^{1/3}}$ For Sloping Revetments: $Q = 3.006 \exp(-13.091\, F')$ where Q is the overtopping rate in $m^3/s \bullet m$, F is the freeboard (m), i.e. the average vertical distance from the stillwater level (including storm surge) to the crest of the seawall, H_{mo} is the energy based zero-moment wave height either measured near the structure or at the toe and L_p is the local Airy wavelength calculated using T_p. L_p can be estimated from Appendix D.	It should be noted that H_{mo} is different than H_s in shallow water. Relationship between H_{mo} and H_s is discussed in Section 4.3(a).
Goda	Goda (1985)	The method is summarized in a series of dimensionless overtopping graphs with the following functional form: $\frac{Q}{\sqrt{2gH_o'^3}} = f\left(\frac{H_o'}{L_o}, \frac{F}{H_o'}, \frac{d_s}{H_o'}\right)$ The design curves for vertical seawalls and revetment seawalls, with nearshore slopes of 1:10 and 1:30 are given in Figures 24 and 25. If either the wave steepness or the approach slope differs from the specifics listed in the design charts, interpolation is necessary. If the approach slope is gentler than 1:30, use 1:30. If the slope is steeper than 1:10, use 1:10.	The design curves are for very specific seawall geometries and wave overtopping is very sensitive to structure shape (Goda 1985). Goda stresses that use of the design curves should be restricted to preliminary engineering purposes or for the planning of physical hydraulic model studies for final design.
Owen	Owen (1982)	$Q = A\, T_z\, g\, H_s \exp\left(-B \frac{F}{r\, T_z \sqrt{g H_s}}\right)$ where Q is the overtopping rate in $m^3/s \bullet m$, F is the freeboard, H_s is the significant wave height at the toe of the structure, T_z is the zero crossing wave period ($T_z \approx 0.9\ T_p$) and r is rough slope surface reduction factor. Values for coefficients, A and B, are given in Table 12. Values for r can be found in Tables 1 and 4.	This method was put forth in CIRIA/CUR (1991) and described by Walton et al. (1989b) as the state-of-the-art in the United Kingdom. Using r for rough slopes is expected to yield conservative results. This model was intended for sloped revetment type, fronted, in some cases, with a submerged berm. Coefficients A and B are site and structure specific.

3.3 Limitations of Wave Overtopping Predictors

The existing wave overtopping predictors are rather crude and are applicable to only a few shoreline structure types with simple profiles. Overall, the predictors must be considered as providing order of magnitude estimates only. De Waal and van der Meer (1992) note that for $F/H < 1$ and also for cases when $Q > 10$ to 50 l/s/m, the reliability of overtopping predictions is small. Kobayashi and Raichle (1994) suggest that it is difficult to develop "an accurate and robust empirical method for predicting irregular wave overtopping over coastal structures situated inside the surf zone because the hydrodynamic processes are very complex and the number of parameters involved is large".

In the cases of complex structure designs, a physical model testing may be the only way to assess wave overtopping with any degree of certainty. Physical model tests of wave overtopping are generally expensive. The decision regarding the need for model tests should be made on a site-by-site basis, depending on the costs of the particular project, and on potential risk for loss of life and damages.

4.0 DESIGN PRACTICE

4.1 Critique of Existing Methodologies

In general, wave uprush and overtopping methodologies are mostly empirical in nature. For the most part, they are based on limited two-dimensional laboratory model studies with little prototype verification. Many of the tests were done using smooth slopes and regular waves. Differences in the definition of uprush, measurements techniques (e.g., the data measured in some laboratory investigations were obtained by visual observations) and laboratory setups can account for some of the range in the various uprush test results. It is known that due to scale effect of surface roughness, the uprush in small scale physical model testing is generally not conservative, i.e. it tends to be underestimated. On the whole, the data normally shows scattering around the fitted equation (e.g., see Figure 21). Even for specific, controlled, laboratory tests, some of the uprush results were above the fitted line and some of the results were below. To date, measurements have been shown to be extremely variable in field situations (Dewberry and Davis 1990). As such, it should be realized that there is a certain degree of uncertainty on the predicted uprush levels and overtopping rates using these empirical equations especially when they are extended to prototype conditions and conditions which differ from the test conditions. As well, with irregular waves, computed mean and significant uprush values are not the maximum expected values. In other words, the fitted equation should not be treated as the upper bound unless it is specifically identified as such. Shoreline managers should know that wave uprush levels and overtopping rates given by the present "state-of-the-art" methodologies are approximations only and in many instances do not represent the maximum values.

Walton et al. (1989a; 1989b) recently conducted a study to recommend a maximum wave uprush criterion for seawall design for <u>no overtopping and flooding</u>, and commented that:

> "An ideal [uprush] methodology would consist of the following points: (1) The methodology should be sufficiently robust to work on all structure slopes, roughnesses, and types; (2) The methodology should be independent of existing bathymetry leading to the structure, i.e. decoupled from wave transformation effects prior to encountering the structure; (3) The methodology should be verified by physical model testing at a scale sufficient to ensure that scale effects are minimized in the data or should provide a rationale to correct for such scale effects; (4) The methodology should be consistent with existing FEMA [Federal Emergency Management Agency, in the U.S.A] criteria for monochromatic depth limited breaking waves; (5) The methodology should provide answers consistent with existing knowledge of coastal flooding events at seawall sites.
>
> Present state of the art in [uprush] prediction is not sufficient to adequately address all of the above points. The primary reason for this inadequate state of knowledge is that generic research data sets of [uprush] for various structure types, locations, slopes, bathymetry, roughnesses, scales, etc. do not exist. The majority of [uprush] studies were made with the limited objective of designing a structure of a given type, slope, roughness, and offshore bathymetry, using prevalent wave conditions at that site. Existing studies which have addressed the physics of [uprush] are either verified (or more likely calibrated) on very limited data sets or not verified at all. A majority of the more recent research on wave [uprush] on structures consists of irregular wave input with corresponding irregular wave

> [uprush] measurements, again for a very limited number of site specific studies.
>
> In view of the above comments, it must be realized that any present approach to the problem of [uprush] prediction will be a pragmatic cost justifiable approach to a complex problem. For an improved answer to the [uprush] problem it will still be necessary, as in present coastal design, to do laboratory testing. Applicants requesting flood protection credit for their seawalls should be allowed the option of providing independent physical testing on their structure in lieu of any proposed 'cookbook' approach."

Allsop et al. (1985a) conducted a literature review on uprush equations on steep slopes and performed a comparison study. They concluded that (1) the method given by the SPM (USACE 1984) and by Losada and Gimenez-Curto (1981) appear to give reliable results; (2) wave uprush on rough slopes exhibits a different form of response to wave characteristics as compared with uprush on smooth slopes; and (3) no single general probability distribution fits all the measured uprush results well, the Rayleigh distribution may underestimate the extreme wave uprush.

Allsop et al. (1985a) further recommended that since wave uprush is estimated based on methods with relatively little validation, hydraulic physical modelling for site specific projects may be quicker, more certain and more economic than a complicated analyses of wave behaviour and structure characteristics. It is interesting to note that both studies emphasized that physical model testing should be treated as a viable option for uprush determination.

. Scale effect

Wave uprush in a physical hydraulic model is less than in prototype due to the viscous effects. This is a model scale effect. Most of the empirical uprush methodologies are based upon small-scale model tests and the use of these methods for prediction of uprush would yield underestimated values compared to prototype values. A limited number of large-scale uprush tests indicated the presence of scale effect. However, the literature on scale effect due to wave uprush (Saville 1987; Stoa 1978; Fuhrboter 1986; and Broderick and Ahrens 1982) is minimal and contradictory (Walton et al. 1989a; 1989b).

In their review of the literature, Walton et al. (1989a; 1989b) noted that there is some ambiguity as to whether the 'scale effects' are "true scale effects (i.e., due to modelling at different flow Reynolds numbers) or due (at least in part) to different relative roughnesses between the large scale tests and the small scale tests". The correction factor in the SPM (USACE 1984) is based upon Saville's work which included only a very limited amount of data. Also, the SPM correction factor increases sharply for steep sloped structures whereas Stoa's (1978b) curve for scale effects correction factor decreases to zero for steep sloped structures (vertical walls). Walton et al. (1989a; 1989b) concluded that due to the extreme lack of supporting theoretical and experimental work, no scale effects correction factor should be used even though this may lead to underestimated values of wave uprush. Dewberry and Davis (1990) found "no evidence of serious weakness in the scale-effect correction (Stoa 1978)".

. Physical Model Testing

In order to achieve dynamic similarity of wave forces on armour stones in the physical modelling of shoreline protection structures, the Froude, Reynolds and Weber numbers must be the same in model and prototype. The Froude number is the ratio of inertia and gravity forces; the Reynolds number is the ratio of inertia and viscous forces and the Weber number is the ratio of inertia and surface tension forces. In a Froudian model (gravity waves) the ratio of inertia and gravity forces are maintained. However, it is sometimes impossible to maintain similarity for all numbers at once. For instance, because both model and prototype use water as the fluid, the viscosity and density are the same and the Reynolds number (Re) criterion cannot be satisfied.

To properly design physical modelling experiments, it is necessary to ensure minimum departure from the similarity criteria listed above (i.e. to ensure minimum scale effect). Several experimental investigations have been carried-out to determine the magnitude of scale effect for hydraulic modelling of coastal structures. Hudson (1975) recommended, based on previous experimental results of Dai and Kamel (1969), that scale effect due to viscous flow through the model structure is minimized as long as the model Reynolds number is larger than a critical value given by $Re > 3x10^{4}$.

Scale effect may also result from differences in layer permeability, air entrainment and two-phase flow. Different permeabilities of the various layers of the model and prototype structures would effect uprush and downrush elevations as well as internal flow characteristics. Further information on physical modelling of coastal structures may be obtained in Hudson et al. (1979). Model effects such as two-dimensional (2D) versus three-dimensional (3D) testing also play a significant role in physical modelling of shoreline protection structures. Irregular wave testing (with the proper spectrum shape) is also preferable. Physical models should only be carried out by qualified, experienced coastal engineers.

4.2 General Procedure

a) Input Conditions

For wave uprush and overtopping computation, the required data include detailed data on the nearshore bathymetry (e.g. site specific sounding surveys and/or field sheets from the Canadian Hydrographic Service), lake bottom profiles at the study site, water level variations and design water levels, deep-water and/or local wave conditions, and configuration of the structure under investigation. With only limited knowledge of specific site conditions, a very detailed treatment of wave uprush and overtopping may not be warranted.

The general procedure is as follows:

- To establish the nearshore and shoreline profiles and elevations, topographic plans of the study site (e.g., site specific plans, 1:2000 Flood Damage Reduction Programme (FDRP) mapping) and local nearshore bathymetric surveys should be obtained.

- Define the stillwater level. Elevations, related to IGLD (International Great Lakes Datum), water level variations and design water levels must be defined. The 100 year flood levels are available from the *Technical Guide for Great Lakes - St. Lawrence River Shorelines* (MNR 1996). The 100 year flood level storm surge and can be considered as the stillwater level (SWL).

- Define the structure (or beach) slope, the approach slope and the depth of water at the toe of the structure, d_s, using the design information, shoreline elevations and bathymetry data. Additional explanation regarding the slopes and structure types can be found in Sections 4.4 and 4.5.

- Define the design wave conditions. Data on the deep-water wave climate may be required depending on the approach chosen. Most protective structures along the Great Lakes - St. Lawrence River System shoreline are located in shallow water, where $d_s / H < 3$. If this is the case, local wave condition at the toe of the structure may be required for the wave uprush and overtopping computations. If local wave conditions are required, wave refraction models, forward or backward tracking, or other wave transformation techniques, such as Goda (1985), can be used to transform waves from deep water (i.e., H_o and T_p) to the toe of the structure or slope (i.e., H_{mo} or H_s at d_s and T_p) as if no structure was present. This will provide the local wave conditions at the structure site assuming that the conditions are the same with and without the structure.

  Alternatively the local wave height may be estimated using depth-limited techniques. For methods which require the unrefracted wave height, the refraction coefficient should be computed. Further explanation on the input wave conditions is provided in Section 4.3.

b) Wave Uprush Computation

For wave uprush computation, the accepted methodology should be selected and its relevant input parameters must be determined. For methods which require the local wave condition, the design wave at the toe of the structure should be determined. For methods which require the unrefracted wave height, H'_o, the deep-water wave conditions should be obtained. If the depth-limited wave is used to estimate the unrefracted wave condition, the refraction coefficient K_r for the study site is required. However, the unrefracted wave height should not be greater than the deep-water wave height. It should be noted that the greatest value of R is not necessarily associated with the greatest value of relative uprush, R/H, or the greatest value of H. Therefore, the wave uprush computation must be repeated for a range of wave conditions expected at the study site in order to determine the maximum R. As discussed later in Section 4.6(b), the maximum calculated value of R_m or R_s is not the upper limit or maximum uprush value. An estimate of an upper limit of wave uprush, say $R_{2\%}$ (de Waal and van der Meer 1992), can be determined by multiplying R_m by a factor of 2.23 or R_s by 1.4.

Finally, it must be noted that the limits of wave uprush or overtopping as determined by the methodologies provided in this report do not delineate the floodproofing elevation as required by provincial policy for development within the *flooding hazard* component of *hazardous lands* along Great Lakes - St. Lawrence River System (see Part 7: Addressing the Hazards, Technical Guide for Great Lakes - St. Lawrence River Shorelines, MNR 1996). The limit of wave uprush or overtopping as may be calculated from this report only delineates the *flooding hazard* limit. The floodproofing standard elevation is determined separately.

i) Natural Beaches

In the case of natural beaches, Hunt's (1959) formula and its variations (Battjes 1974a; Ahrens and Titus 1985; USACE 1990) have been widely used for uprush prediction and may be considered accepted practice. FEMA (1991) uses the equation suggested by Ahrens and Titus (1985) for smooth slopes. Instead of using the local wave height as the input parameter (see Table 10), FEMA suggests using the unrefracted wave height. The equation becomes $\frac{R}{H_o^{\prime}} = 0.967 \frac{\tan\theta}{\sqrt{H_o^{\prime}/L_o}}$.

It is important to note that recent work on irregular wave uprush on gentle slopes by Mase (1989) yielded different uprush predictions than the ones using the Hunt-type formulation. As shown in Figure 7, predictions with the Hunt formula are shown to be equivalent to either the mean uprush or the significant uprush depending on the value of ξ. This difference remains to be further investigated when more research data with irregular waves become available. Ahrens (1981a) also provided a methodology for predicting the mean, significant and 2 percent uprush values.

ii) Sloped Shoreline Structures

For shoreline structures such as rip-rap and armour stone revetments, both surface roughness and structure permeability play a role in the uprush processes. Methods which are based on rough slope model tests include Ahrens and McCartney (1975), Stoa (1979), Losada and Gimenez-Curto (1981), Ahrens and Heimbaugh (1988a) and van der Meer and Stam (1992). The U.S. Army Corps of Engineers (1990) and FEMA (1991) use the method of Ahrens and McCartney (1975) for rough slopes. An alternative method is to use smooth slope uprush methods (i.e., Stoa 1978b; Walton and Ahrens 1989; Ahrens 1981a; Mase 1989; Pilarczyk 1990; and USACE 1990) with roughness correction factors from Stoa (1979; see Appendix B this report), USACE (1984; see Table 1), Walton et al. (1989a, 1989b; see Table 4) and Pilarczyk (1990; see Table 5).

For armour stone revetments, especially single layer armour structures using geotextile filters, surface roughness may play a greater role in the uprush processes than the permeability. For the purpose of wave uprush, the geotextile can be considered as impermeable (i.e., the velocity of the uprushing water is much greater than the ability of the geotextile filter to pass the water through). Therefore, the methods used for impermeable rip-rap revetments may apply here, with different values for the roughness/permeability coefficient. In the case of individually placed, tightly fitted armour stone (with a relatively uniform surface), the roughness correction factor should be increased accordingly. If the structure is highly permeable, van der Meer and Stam (1992) suggested that the relative uprush may reach a limit. If structures with vertical walls are present, the uprush methodology for non-breaking waves apply (e.g. Walton and Ahrens 1989).

Once the wave uprush elevations are calculated, the horizontal distances covered on land by the wave uprush process may be estimated based on Figure 3, for constant slopes. The horizontal distance will depend on the land characteristics (i.e., different values would be obtained for natural beaches, concave shore profiles, bluff faces, and vertical walls). For low bluffs, wave uprush may travel over the bluff crest similar to a wave bore. Procedures to estimate the extent of the wave travel are given in Section 4.5(e).

b) Wave Overtopping Computation

When the uprush level exceeds the crest elevation of the design structures, wave overtopping will take place. The next step is then to calculate the wave overtopping rate using the appropriate wave conditions and the selected accepted methodology. For revetment type seawalls (see Figure 5(b)), the methods of Ahrens and Heimbaugh (1988b), Owen (1982) and Goda (1985) may be used. For vertical type seawalls (see Figure 5(a)), the methods of Ahrens and Heimbaugh (1988b) and Goda (1985) apply.

As with the wave uprush computation, the wave overtopping computation should be repeated for the range of wave conditions that may be expected in order to determine the maximum Q. The overtopping rate should be adjusted for oblique wave attack (see discussions Sections 4.3(f)). It is recommended to apply a factor of 1.15 to all calculated overtopping rates to account for the direction of wave attack not being directly normal to the structure. If a detailed wave direction analysis is undertaken which justifies the appropriate correction factor, then the above factor of 1.15 is superseded.

For structural stability and drainage considerations, the 100 year flood level and accompanying wave conditions should be used as the design conditions. For usage considerations (i.e., pedestrians), the water level and wave conditions should be based on conditions likely encountered during the period of reasonably anticipated use.

Storm wave activity on the Great Lakes is the result of winds blowing across the water. At the height of a storm, when waves are at their maximum, onshore winds can be very strong and can enhance the wave overtopping. Goda (1985) notes that wind is an important factor in wave overtopping, but no reliable information exists on the wind effect because of the lack of a reliable modelling law. However, it is believed that the increase in wave overtopping by an onshore wind is large when the quantity of overtopping is small and that the wind effect decreases gradually as the overtopping rate increases. Contrary to Goda's beliefs, Takada (1976) claims that the wind effect is not negligible even when wave overtopping is intensive.

There is considerable difficulty in estimating the amount of wind induced or assisted overtopping and spray for any given seawall. This is compounded by the inability of small scale hydraulic models to correctly reproduce spray generation, due principally to surface tension scale effects which control droplet size (e.g., Bishop et. at. 1985). Few model studies have successfully used scale wind velocities to assist overtopping. Finally there has been virtually no reliable information reported on the measurement of such overtopping in the field (Allsop 1986). Gadd et. al. (1984) discuss some qualitative trends in the wind effect and conclude that more data is needed to improve upon the SPM (USACE 1984) wind correction method for overtopping.

The SPM (USACE 1984) provides a simple procedure for estimating a wave overtopping wind enhancement factor:

$$k' = 1 + W_f \left(\frac{F}{R} + 0.1 \right) \sin\theta$$

where W_f is a coefficient depending on wind speed, U_{10}, as follows:

for	$U_{10} > 25$ m/s	$W_f = 2.0$
	$U_{10} = 13$ m/s	$W_f = 0.5$
	$U_{10} = 0$ m/s	$W_f = 0.0$

and θ is the structure slope. However, it must be realized that the SPM correction is merely an engineering judgement approximation of a very complex phenomenon. Sustained wind speeds (i.e., not gusts) that could be expected during a severe storm can be obtained from local weather offices or meteorological records.

Once the range of wave overtopping rates has been calculated, the structure performance may be assessed based on a comparison between the calculated rates and the acceptable criteria as detailed in Section 4.7. Finally, overtopping protection (refer to Section 4.8) and drainage considerations (refer to Section 4.9) must be addressed.

c) Sensitivity Analysis

A range of wave conditions (height and period) should be used to determine sensitivity of the wave uprush and overtopping methodologies. Depending on the site conditions (i.e., slope, approach slope and water depth), the maximum wave height may not necessarily produce the maximum value of wave uprush.

4.3 Input Wave Conditions

a) Wave parameters

The various methodologies for computing wave uprush and overtopping are complicated by the input wave parameters which were used in the original model tests. These wave parameters include H, H_o', H_{mo}, H_s, and H_m for wave height and T, T_z, T_p, T_s, and T_m for wave period. As discussed later in this Section, these wave parameters are different in shallow water. Therefore, it is important to understand which parameter is used for engineering design and planning.

Since most of the uprush equations and relative uprush curves in the SPM (USACE 1984) were developed using regular waves (i.e., H and T), Ahrens (1981b) addressed the inherent difficulty in estimating extreme values and the specific difficulty of adapting results of monochromatic wave tests to irregular wave conditions in relatively shallow water. However, for irregular waves, statistical and spectral methods have been used to determine the representative wave height. The common ones are H_s (defined as the average of the highest one-third individual waves in a record) and H_{mo} (defined as four times the square root of the area under the energy spectrum). H_s is a statistical-based parameter, while H_{mo} is an energy-based parameter.

In deep water, the distinction between these two parameters is of little engineering importance and H_{mo} is approximately equal to H_s. The distinction is important where the water depth is shallow or the waves are very steep (Thompson and Vincent 1985). As such, it is important to know which parameter is used for design in shallow water.

There is no intrinsic relationship between regular wave parameters, H and T, and irregular wave parameters H_{mo}, H_m, H_s and T_p or T_m. Reference to regular wave tests for situations where irregular waves exist (i.e., nature) may not be representative. Thompson and Vincent (1985) suggested that if H_s is the primary concern, H could be specified as $H = H_s$ but it implies that the monochromatic wave train will contain twice as much energy as the irregular wave train. If energy is most important, H could be chosen as $H = 0.71\ H_{mo}$ so that the energy contained in the regular wave train is the same as that in the irregular wave train but many of the waves in the irregular wave train would be higher than the monochromatic wave height.

It should be noted that, based on the work of Kamphuis and Mohamed (1978) and Mase (1989), Dewbwerry and Davis (1990) point out that using H_s in the method of Stoa (1978b) provides an underestimate of measured uprush elevations. Dewberry and Davis (1990) recommend that the mean wave conditions (wave height, H_m, and period, T_m) are more appropriate wave parameters to determine the mean uprush value, R_m. A reasonable estimate of the significant wave uprush level, R_s, can be made multiplying R_m by 1.6 and the upper limit of wave uprush, $R_{2\%}$ can then be determined by multiplying R_m by a factor of 2.23 (assumes Rayleigh distribution). The mean wave conditions can be estimated (again assuming Rayleigh distribution) from: $H_m = 0.626\ H_s$; $H_2 = 2.23\ H_m$; and $T_m \approx 0.8$ to $0.9\ T_p$.

In cases where H_{mo} rather than H_s is used in the uprush and overtopping methodologies, this energy based parameter H_{mo} should be used throughout the computation. For deep water, H_{mo} can be set equal to H_s. However, recently it was found that the H_{mo} of broad banded, locally generated wind seas behaves contrary to linear wave theory (i.e., H_{mo} becomes smaller as it approaches from deeper water to shallower water). Thus using linear wave theory to determine the wave height at shallow water will give a value substantially larger than should be expected. Use of these wave heights in design formulae can lead to uneconomical over-design (Hughes and Miller 1987).

Since the empirical uprush equations were obtained in the laboratory under a controlled environment, the input waves were generally the waves in front of the structure or the waves generated by the paddle. Typically, the model layout was such that the structures were on a flat bottom or on sloping bottoms provided that $d_s / H'_o > 3$, and the waves did not break before reaching the structures. If the offshore slope fronting to the structure has great influence on the incident wave trains, it is necessary to know which wave conditions should be used as input to the uprush equations; the waves at the toe of the structure, or at deep water, or the broken waves. Resio (1987) studied extreme uprush on natural beaches and found that different uprush levels were obtained when the wave conditions at different locations were used.

Ahrens (1981b) noted that to use the equations of Ahrens and McCartney (1975) it is necessary to have the local significant wave height at the toe of the structure. He recommends using Goda's (1975) model for irregular waves to determine the local wave conditions. Diagrams for the estimation of the significant wave height in the surf zone can be found in Appendix C (from Goda (1985)).

Walton and Ahrens (1989) suggested to use transformed wave heights rather than deep-water wave heights. In this manner, the uncertainty of wave height transformation from deep water to the structure site would be handled as a separate problem.

b) Local Wave Conditions

Two procedures that can be used to estimate the local wave conditions (i.e., nearshore waves at the toe of the structure slope) are: 1) transformation of the deep-water waves to the nearshore using refraction, diffraction and shoaling procedures; and 2) depth-limited breaking wave criteria.

. Wave Transformation

Wave transformation to the nearshore can be crudely estimated using simple graphical techniques presented in the Shore Protection Manual (USACE 1984) or more accurately predicted using numerical models. Both approaches require some coastal engineering expertise to understand the data requirements and output limitations.

. Depth-Limited Breaking Waves

An alternative approach to determining the deep-water wave conditions and then transforming the waves to the nearshore is to assume depth-limited wave conditions in shallow nearshore waters. Depth-limited simply means that the wave height is physically limited by the depth of the water. That is to say, a given depth of water can only support a certain maximum wave height. There are a number of depth-limited wave height or breaking wave criteria as follows:

Linear wave theory (approach slope flatter than 1:100)

$$H_s = 0.78\, d_s$$

Hughes (1984) (engineering approximation for typical beach slopes)

$$H_{mo} = 0.6\, d_s$$

Kamphuis (1991)

$$H_{sb} = 0.095\, e^{4m} L_{pb} \tanh k_{pb} d_b$$

Two procedures which incorporate the depth-limited approach for estimating local wave conditions are Goda (1985) and FEMA (1991) and are outlined herein.

Goda (1985) Procedure to Estimate Local Wave Conditions

Diagrams for the estimation of the significant wave height in the surf zone, from Goda (1985), can be found in Appendix C.

FEMA (1991) Procedure to Estimate Local Wave Conditions

Assuming that deep-water wave conditions (H_s or H_{mo} and T) are known, the following procedures are recommended by FEMA (1991) to compute the H_{mo} at shallow water (but $d_s \geq 0$):

- *To define the cutoff wave period* $T^{/}$.

  Given the depth at the structure toe d_s; compute $T^{/} = 2\pi \sqrt{\frac{d_s}{g}} = 6.283 \sqrt{\frac{d_s}{g}}$.

- *To define wavelength L at* d_s.

  If $T \geq 1.5 \; T^{/}$, then compute $L = T\sqrt{g\, d_s}$. Otherwise, L can be estimated using linear theory (see Appendix D).

- *To compute wave height* H_{mo} *at* d_s.

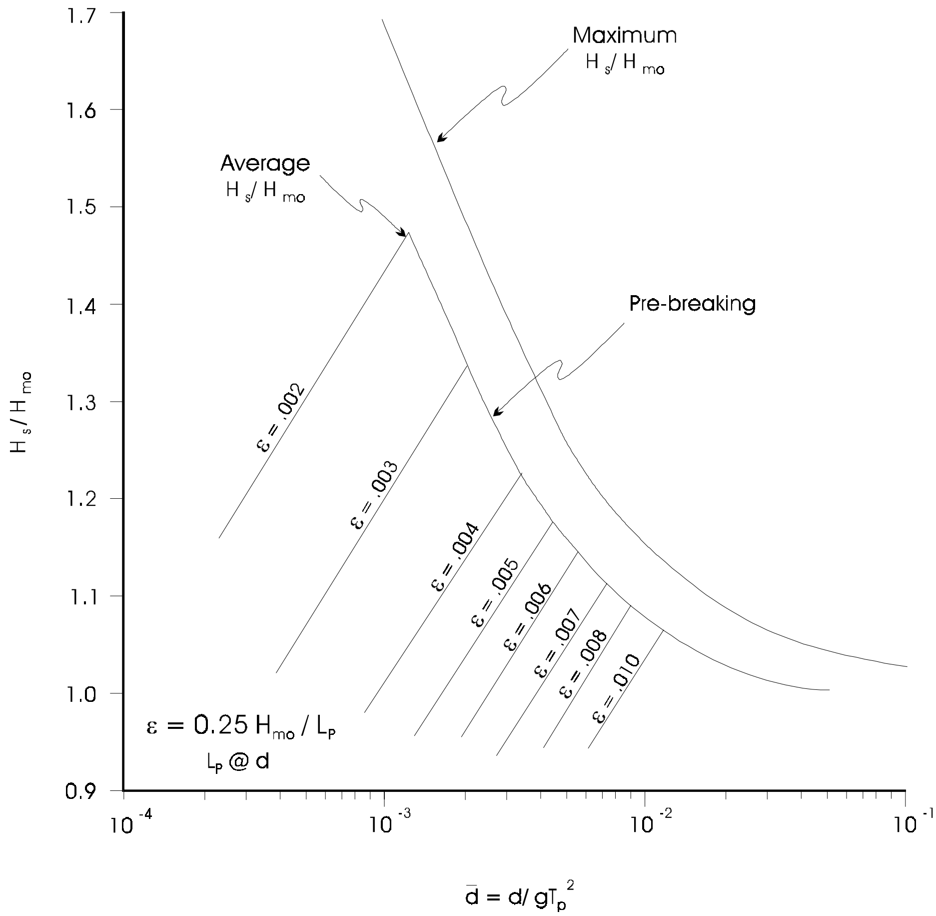
  $H_{mo} = \frac{1}{\pi}\sqrt{\alpha}\; L$ where $\alpha = 0.0078\, \kappa^{0.49}$ and $\kappa = 2\pi \frac{U^2}{g\,L}$. U is the wind speed at 10 m elevation and FEMA (1991) assumes a value of 40 mph (or 65 km/h).

- *To compute breaking wave height* $H_{mo,b}$ *at* d_s.

  At breaking, H_{mo}/d_s varies between 0.55 and 0.65 for most typical beach slopes. A value of 0.6 can be used for most engineering purposes (Hughes, 1984). Hence $H_{mo,b} = 0.6\, d_s$. If H_{mo} is greater than $H_{mo,b}$, then set $H_{mo} = H_{mo,b}$.

As discussed previously, linear wave theory may give a wave height value larger than expected. The depth limited wave height H_s (= 0.78 d_s) may be over-estimated. FEMA (1991) suggested to compute H_s by converting the computed wave height H_{mo} at d_s using the graph prepared by Thompson and Vincent (1985) as shown in Figure 33.

Figure 33: Relationship Between H_s/H_{mo} and d/gT_P^2 (after Thompson and Vincent 1985)



The following procedure is used:

- *To compute the dimensionless depth* $\bar{d}$ *and wave steepness* ϵ.

  Compute the values of $\bar{d} = \dfrac{d_s}{g\,T^2}$ and $\epsilon = 0.25\, H_{mo} / L_p$.

- *To compute wave height* H_s *using* H_{mo} *at* d_s.

  Using Figure 33, estimate the ratio H_s / H_{mo}. The upper curve in Figure 33, labelled "Maximum", represents an upper limit on H_s / H_{mo} for a given $\bar{d}$. This upper curve is appropriate for structure design, where some conservatism is desirable. The lower curve, labelled "prebreaking" represents a rough average from depths greater than the breaking depth (pre-breaking conditions). The lines labelled with the values of wave steepness, ϵ, permit estimates of H_s in the surf zone (post-breaking). FEMA (1991) recommends if the ratio H_s / H_{mo} is less than 1, set the ratio equal to 1 and then compute H_s.

In a recent presentation on "Wave Runup Guidance - Sensitivity" (author and date unknown) estimates of nearshore wave conditions based on the methods of FEMA (1991) and Goda (1985) were shown to be different. It was argued that in FEMA (1991), the treatment of depth-limited wave heights presumes that the breaker-zone bottom is nearly horizontal. Goda (1985) provides four approach slopes (1:10, 1:20, 1:30 and 1:100). The presentation concluded that wave height in the nearshore could be "appreciably underestimated" by FEMA (1991) wherever the slope of the approach slope was steeper than 1:100.

It is useful to know the deep-water wave height associated with the depth-limited wave height (i.e., what deep-water wave height is necessary to produce the depth-limited wave, including effects of wave refraction, shoaling, etc.). Comparison of the deep-water wave height necessary to produce the depth-limited wave with the actual deep-water wave statistics, characteristic of the site, will give some indication of how often the structure could be subjected to waves as high as the calculated depth-limited wave. It is likely for many shallow nearshore sites, depth-limited conditions will prove to be very frequent events (possibly annually or even more frequently).

c) Incident Versus Transmitted Waves

In instances where a structure, such as a detached breakwater, may act to reduce the incoming, or incident, wave action at a site, it may be necessary to estimate the transmitted wave height (i.e., the wave height on the leeside or shoreward side of the structure) for the purpose of determining the *floodproofing standard*. The transmission is dependent on the structure geometry (crest freeboard and width), permeability, water depth, and wave conditions, especially wave period. Estimates of transmitted wave height can be made by qualified coastal engineers using guidance from the literature (such as CIRIA/CUR 1991; Allsop 1983; Bremner et al. 1980; Seelig 1979; van der Meer 1990).

Figure 34 provides a simplistic relationship between the transmission coefficient, K_t and the relative freeboard, F/H_s, that may be sufficient for preliminary design. The upper and lower bounds represent the 90% confidence bands.

d) Oblique Wave Attack

It is often assumed that waves attacking a wall, with the wave crests parallel to the wall, will give rise to more severe effects than would oblique wave attack. The Shore Protection Manual does not consider the effect of wave attack at any angle of incidence, β, other than 0° (i.e., shore normal), that is with the wave crests parallel to the structure. The implicit assumption is that normal wave attack represents the most serious case.

Losada and Gimenez-Curto (1982) found that when waves approach the shore or structure at an angle, the uprush was found to be lower than that under perpendicular wave attack. Hosoi and Shuto (1964) proposed a reduction factor of $(1+\cos\beta)/2$ to reduce the uprush values for oblique wave incidence, where β is the wave approach angle.

Pilarcyzk (1990) suggested that oblique wave attack can be "roughly" taken into account by a reduction factor of $\cos(\beta\text{-}10°)$ when $\beta \leq 65°$. β is reduced by 10° to account for uncertainty in the value of β. When $\beta > 65°$, the uprush must be greater or equal to the incident wave H_s.

Allsop (1986) notes that normal wave attack may not give the greatest uprush (or overtopping). Test results, apparently originating from CSIR tests on dolos, and quoted by Gunbak (1979, but not referenced), illustrate that for waves of steepness H/L_o of 0.03 to 0.04, wave uprush (overtopping) is greater for $\beta \sim 30°$ than for $\beta = 0°$ or 45°. This is not commented upon, in fact Gunbak (1979) concludes that uprush (overtopping) may be reduced by the cos β factor.

Tautenhaim et al., (1982) conducted laboratory tests of oblique wave attack on uprush on a 1:6 smooth slope with regular waves and found that in contrast to previous investigations, an increase of uprush compared to normal wave approach would occur for wave directions in the range from 0° to 35°. Tautenhaim et al. (1982) propose that the effect of oblique wave attack is simply to modify the uprush (or overtopping) at normal incidence by a factor $k_\beta = \cos\beta\,(2 - \cos^3 2\beta)^{1/3}$. A graph of this function is shown in Figure 35.

A similar effect was noticed by Owen in tests measuring the overtopping of revetment seawalls. It was shown that generally the mean overtopping discharge at 15°, and sometimes 30°, exceeded that at $\beta = 0°$. It seems likely that, while the expression given by Tautenhaim et al. was only derived for smooth 1:6 slopes, a similar effect may be seen for seawalls or steeper slopes. It would appear from these two studies that the angle giving greater uprush (or overtopping) could be around $\beta = 15$ to 20°.

Figure 34: Wave Transmission Over and Through Low-Crested Structures (after CIRIA/CUR 1991)

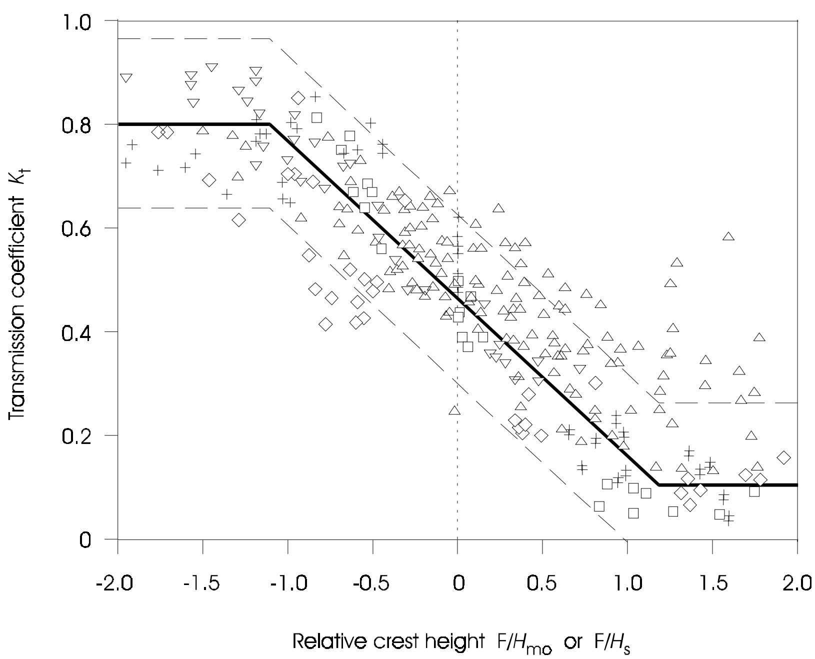
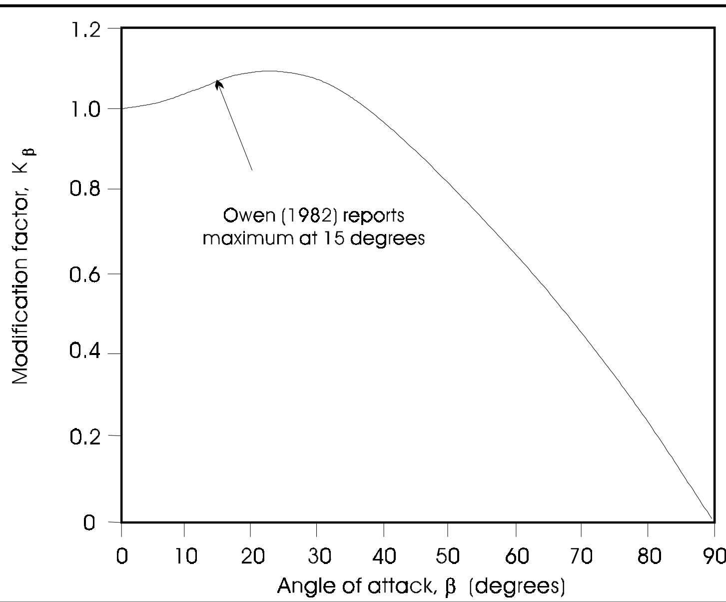


Figure 35: Wave Direction Modification Factor



Oblique wave attack was investigated by de Waal and van der Meer (1992) on smooth plane 1:2.5 and 1:4 slopes. For short-crested oblique waves, for β increasing from 0° to 90°, the uprush influence factor decreases linearly from 1.0 to 0.8 and the wave overtopping influence factor decreases linearly from 1.0 to 0.7.

Without a detailed analysis of the nearshore wave climate, it is recommended to increase wave overtopping rates by a factor of 1.15 to account for the influence of wave direction.

e) Waves and the Flooding Hazard

Storm surge and wave action are not independent events as both are wind driven phenomena; when you have a storm surge on the Great Lakes, you are likely to have wave action as well. Further, the 100 year flood level is a combination of mean lake level and storm surge. It follows that the 100 year flood level is not independent of wave action. In order to determine the limit of wave uprush for the purpose of determining the limit of the *flooding hazard*, it is necessary to know the wave conditions which are reasonably likely to accompany the 100 year flood level. There are three approaches that can be used: the first, is based on the appropriate return period of the deep-water waves; the second, is derived from the wave conditions resulting from a maximum sustained wind speed; and the third, is the based on the depth limited wave condition. Both the first and second approaches require transformation of the deep-water waves to the site in the shallow nearshore (i.e. local wave conditions). Wave transformation is described in Section 4.3(b).

. Return Period of Deep-Water Waves

Very little research has been carried out on deep-water wave conditions which are likely to accompany the 100 year flood level. In *Guidelines for Great Lakes Wave Runup Computation and Mapping*, the U.S. Federal Emergency Management Agency (FEMA 1991) recommends using the 100 year deep-water wave condition. To determine the 100 year deep-water wave, it is necessary to complete a return period analysis of the deep-water wave hindcast data (e.g., hourly wave heights over a period of 10 to 20 years). Two deep-water wave databases have been completed for the Great Lakes: Ministry of Natural Resources (MNR 1988a, 1988b and 1988c); and U.S. Army Corps of Engineers, Waterways Experiment Station, Wave Information Study (WIS) (Hubertz et al. 1991).

Preliminary work by Dewberry and Davis (1994), on wave action with extreme floods on the U.S. side of the Great Lakes, suggests that expected waves coincident with the 100 year flood on the four upper lakes (Superior, Huron, Michigan and Erie) may be described by the wave height with a recurrence interval of three years. On Lake Ontario, their preliminary work indicates that the expected wave height is summarized as having a recurrence interval of one-half year. Dewberry and Davis (1994) also reported that their guidance appears to be consistent with the approach of using wave heights derived from a maximum sustained wind speed of 65 km/h (40 mph).

It should be noted that the preceding findings of Dewberry and Davis are preliminary and were developed for the U.S. shores of the Great Lakes. The U.S. shores are exposed to prevailing and storm wind directions which are different from those at the Ontario shores. Hence, the Dewberry and Davis results may not be directly applicable to the Ontario shoreline. In developing the Great Lakes - St. Lawrence River System

Technical Guide (MNR 1996), wave heights were estimated for 13 of the MNR Great Lakes deep-water wave database sites: 2 on Lake Superior, 3 on Lake Huron, 2 on Lake St. Clair, 3 on Lake Erie and 3 on Lake Ontario. The wave heights were estimated by using a maximum sustained wind speed of 65 km/h (90 km/h for Lake St. Clair), as outlined in the following subsection. The results for the 13 sites were then compared to the return periods of the corresponding wave heights in the MNR wave database. It was estimated that wave heights calculated using the 65 km/h winds were generally less than the 10-year to 20-year wave heights from the MNR wave database for Lakes Superior, Huron, St. Clair (using the 90 km/h wind), and Ontario. For Lake Erie, the 5-year MNR database wave heights were greater than the waves generated by the 65 km/h wind.

It is recommended that until further study is carried out for the Canadian shore, that 10-year to 20-year return period wave heights, calculated from the MNR deep-water wave database, should be reasonably safe estimates of wave heights to be used in conjunction with the 100 year flood level to determine the limit of uprush for the purpose of delineating the *flooding hazard limit*.

. Maximum Sustained Wind Speed

If a deep-water wave database is not available, the deep-water wave conditions (H_s and T_p), to be used to determine the *flooding hazard limit*, can be estimated using wave prediction techniques presented in the 1977 edition of the Shore Protection Manual (USACE 1977) along with the maximum sustained wind speed for the Great Lakes and the appropriate straight fetch length. The deep-water wave is then transformed to the location at the structure toe. The 1977 edition of the Shore Protection Manual wave prediction method is recommended by Bishop et al. (1989).

A wind speed of 65 km/h is considered to be a reasonable estimate of the maximum sustained wind speed on the Great Lakes (FEMA 1991). Wind measurements at various lake-centre sites demonstrate that winds of 65 km/h constitute a "moderately extreme condition, occurring about 5 to 20 hours per year" (Dewberry and Davis 1994). For Lake St. Clair, because it is relatively shallow and has shorter fetch distances, the appropriate sustained wind speed used should be greater than 65 km/h. A preliminary recommendation is to use a wind speed of 80 to 100 km/h for Lake St. Clair.

. Depth-Limited Waves

The depth-limited wave approach is described in Section 4.3(b).

f) Waves and the Floodproofing Standard

The wave condition to be used for defining the *floodproofing standard* should be more extreme than the wave condition for the *flooding hazard*. The *floodproofing standard* for the Great Lakes is the sum of the 100 year monthly mean lake level <u>plus</u> the 100 year storm surge <u>plus</u> an allowance for wave action. Wave action that is likely to accompany the 100 year storm surge has not been defined by any studies but can be expected to be more extreme than wave action which accompanies the 100 year flood level.

It is recommended that until further study is carried out, a 50-year to 100-year return period wave height, determined from an appropriate deep-water wave database, be used in conjunction with the 100 year monthly mean lake level and the 100 year storm surge to determine the *floodproofing standard*.

4.4 Structure and Approach Slopes

It is necessary to define the structure slope or beach slope for to calculate wave uprush and overtopping. For a sloping structure, it may be easy to define the structure slope and the point of intersection with the approach slope. However, for beaches, the beach slope can be more difficult to define. Very often, the beach slope is confused with the approach slope which influences the waves propagating toward the shoreline. In order to standardize the approach to define the structure and approach slopes, the method given by Dewberry and Davis (1990) is suggested in this report.

The Dewberry and Davis (1990) method is summarized in Figure 36. A profile is defined by a set of co-ordinates which could be obtained from field sheets or sounding data. The profile is separated into two segments: structure and approach (see Figure 37). For the purpose of natural beaches, the structure slope M_s can be referred to as the beach slope; while the approach slope M_a can be described as the nearshore slope. The method starts by defining the structure slope using the stillwater level (SWL) and offshore wave height, H'_o. Subsequently, the approach slope is defined.

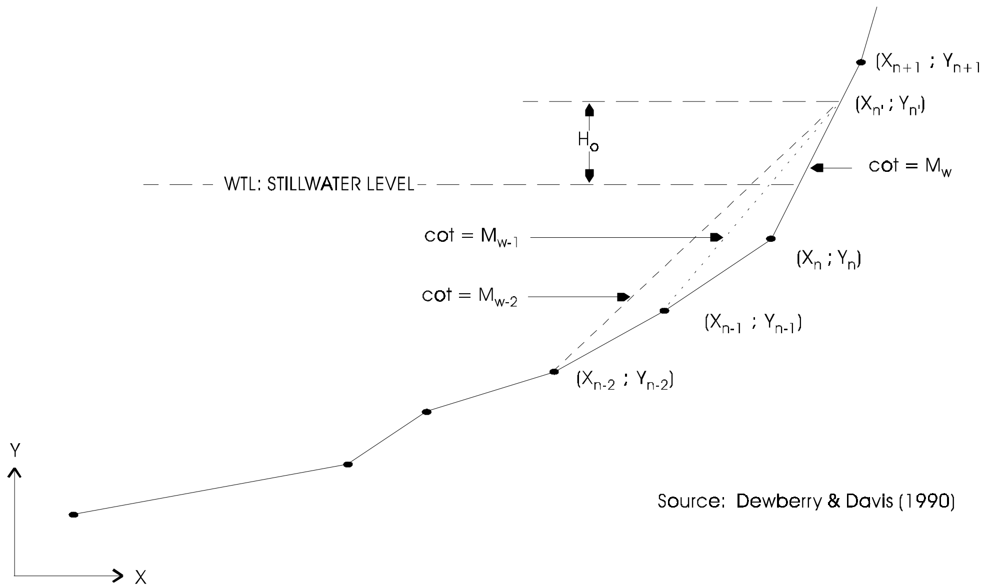
Detailed data on the nearshore bathymetry and onshore topography at the study site are needed to define the transition from the approach slope (lake bottom) to the shore or protection works slope. Bathymetry data can be obtained from site specific sounding surveys and/or field sheets from the Canadian Hydrographic Service. FDRP mapping or a site survey can provide suitable information regarding the onshore profile.

For graphical methods of determining wave uprush, the number of approach slopes available is limited. For example, Stoa (1978b, 1979) only provides graphs for a horizontal approach or for approach slopes of 1:10. Dewbwerry and Davis (1990) recommend that an approach be considered horizontal unless its overall slope is 1:15 or steeper and provided that the structure toe is not in "extremely shallow water".

Goda's (1985) method of determining wave overtopping provides for two approach slopes, 1:10 and 1:30. If the approach slope is 1:10 or steeper, Goda's graph for 1:10 slopes is recommended. If the approach slope is 1:30 or flatter use the graph for 1:30. If the approach slope is between 1:10 and 1:30, interpolate between the two graphs.

The point at which the structure slope and the approach slope intersect is normally at the toe of the structure. For natural beaches, this point may be referred as the 'toe' of the beach. Hence, for beach uprush calculations, the input wave parameters for the uprush formula should be defined at the depth of water at this point. In other words, deep-water waves should be refracted up to this depth and the refraction coefficient should be obtained at this location. FEMA (1991) suggested a value of approximately 8 m (26 feet) for d_s for beach wave uprush calculations.

Figure 36: Outline for New Geometrical Analysis of Basic Shore Situation



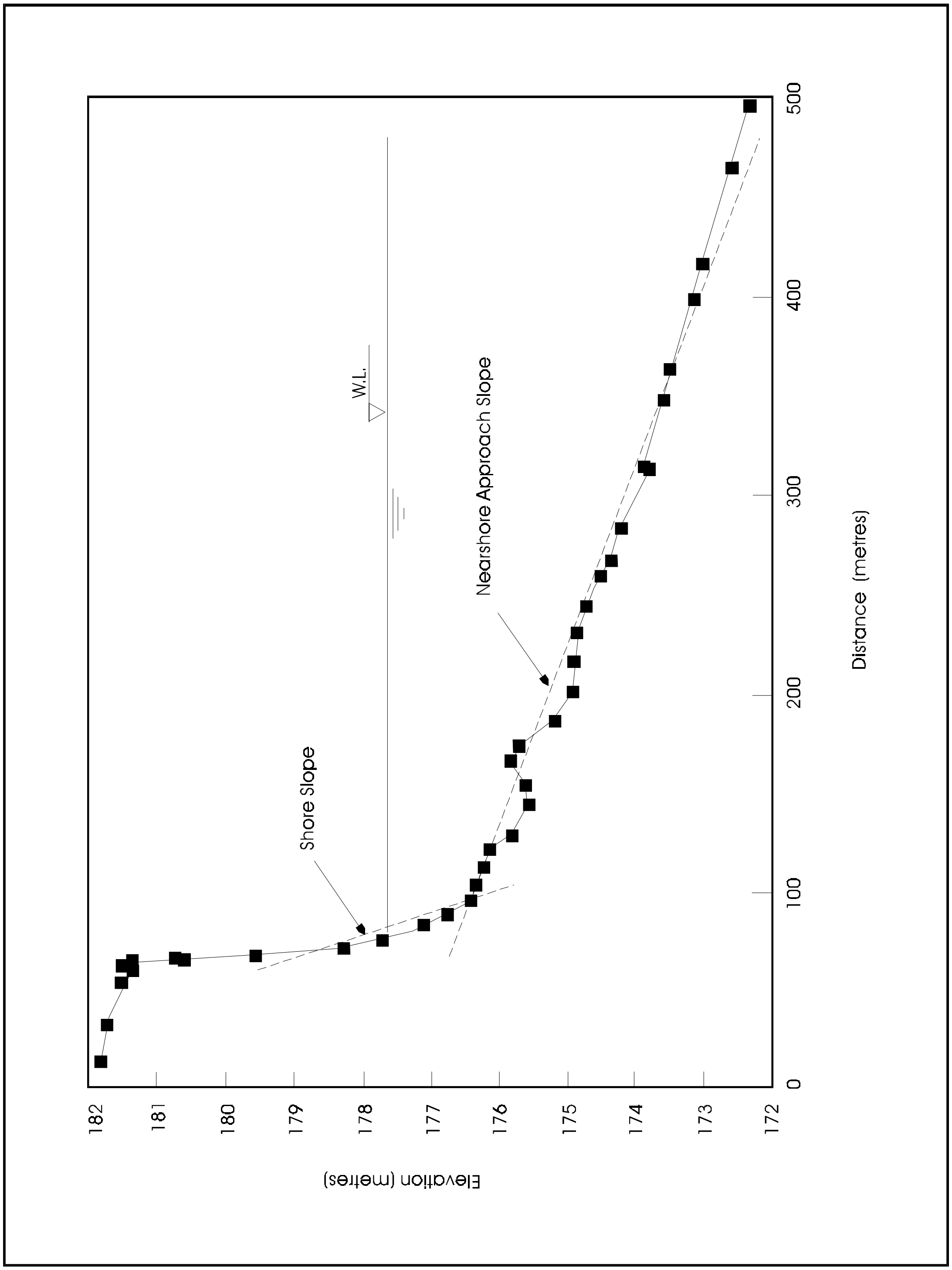
Analysis for Lakeward Extent of Shore Slope

1. M_w = Cotangent of emergent profile segment (i.e., including stillwater intercept). If segment extends to elevation exceeding ($WTL+H_o$), determine coordinates at that elevation, $X_{n'}$; $Y_{n'}$, and use in place of X_{n+1} ; Y_{n+1} in the following.
2. Add first fully submerged profile segment to emergent one, and determine overall slope of combination, namely $M_{w-1} = (X_{n+1} - X_{n-1})/(Y_{n+1} - Y_{n-1})$.
   If $M_{w-1} \leqslant 1.2\ M_w$, consider Shore Slope to include present segment, and proceed to next step; otherwise, Shore Slope extends lakeward only to X_n ; Y_n.
3. Add next lakeward profile segment and determine new overall slope M_{w-2}.
   If $M_{w-2} \leqslant 1.2\ M_w$, admit this segment to Shore Slope and repeat tentative extension; otherwise, do not.

Analysis for Lakeward Extent of Nearshore Approach Slope

1. M_{s-1} = cotangent of profile segment immediately lakeward of Shore Slope limit. Add next lakeward profile segment and determine overall slope M_{s-2}.
   If $M_{s-2} \leqslant 1.2\ M_{s-1}$ and $M_{s-2} \leqslant 15$, admit second segment to Nearshore Approach Slope, and proceed to next step; otherwise, Nearshore Approach Slope is limited to single segment.
2. Add next lakeward profile segment and determine overall slope M_{s-3}.
   If $M_{s-3} \leqslant 1.2\ M_{s-1}$ and $M_{s-3} \leqslant 15$, admit segment to Nearshore Approach Slope and repeat tentative extension; otherwise, do not.

Figure 37: Typical Profile for Wave Uprush Calculation



4.5 Shoreline Structure Types

a) Surface Roughness and Structure Permeability

The surface roughness and structure permeability is dependent on the structure type (see Tables 1 and 4, and Appendix C). Saturated sandy beaches are typically assumed to be smooth and impermeable. Permeability of a structure depends on the thickness and number of armour layers, underlayers and filters as well as the core. The results of the various investigations must be considered with respect to the differences in structure crest layout, crest height and permeability. To cite Bruun (1985):

> "[Uprush] data for rubble mound slopes have been considered separately for rubble mound structures and [rip-rap] revetments. There is no essential difference between the two types of structures. '[Rip-rap]' is commonly used for protection of an embankment (revetment) that is relatively high compared to the expected waves. 'Rubble mound' is usually applied to structures like breakwaters and jetties in which the top of a relatively impermeable core is near the SWL [stillwater level], and the part of the structure above the core is relatively permeable. Such rubble mound structure may absorb and transmit some energy in the upper, permeable part of the structure. '[Rip-rap]' is usually smaller size than 'rubble mound' due to lower exposures or shallow water depths."

Figures A.1 and D.1 in Appendix B show the relative differences between a revetment structure and a rubble mound breakwater. A designer should not use an empirical method for a certain structure when that method was developed for a different type of structure. For example, the method of Ahrens and McCartney (1975) with the coefficients for "quarry stone, 2 layers" applies to a rubble mound breakwater and should not be used for rip-rap revetments (see Table 2).

Single layer armour stone revetments with an underlayer of rip rap and a geotextile filter may be considered as a rough, impermeable structure for the purpose of calculating wave uprush. This is due to the fact that the velocity of the uprushing and downrushing water greatly exceeds the ability of the geotextile filter to pass water.

Van der Meer and Stam (1992) examined the influence of structure permeability. Tests conducted with an impermeable core, a permeable core and a homogeneous core revealed that up to $\xi_m = 3$, the uprush values were more or less the same for all three permeabilities (see Figure 20). For larger values of ξ_m, the permeable and homogeneous structures reach approximately a same constant level of $R_{2\%}/H_s$ around 2, while the impermeable core data still show increasing uprush with increasing ξ_m, up to $\xi_m = 7$. Ahrens and Heimbaugh (1988a) indicate that maximum uprush may not be too sensitive to the armour layer thickness.

. Gravel/Shingle Beaches

Stevenson (1983) suggested wave uprush on shingle beaches could be estimated by $R/H = 0.35\xi$. However, during a storm, the profile of a gravel/shingle beach will adjust to the wave conditions. Therefore uprush levels on a gravel/shingle beach are determined without reference to the initial slope. Typically if enough material is available, a beach crest will be formed and only the highest waves (i.e. approximately R_2) will overtop the beach crest. For small diameters (D_{50} < 0.10 m) the crest height above the stillwater level, h_c, can be determined as follows (CIRIA/CUR 1991):

$$h_c = 0.3 \frac{H_s}{\sqrt{s_m}}$$

. Reduction Factors

It has been mentioned that the characteristics of uprush on a smooth impermeable slope are different from those on a rough slope. The relative uprush on a smooth slope, as seen in Figures 6 and 21, increases linearly with the surf similarity parameter for $\xi < 2$. In the transition region, $2 < \xi < 3$, the relative uprush increases to its maximum value and then decreases to a constant value for the region of non-breaking wave conditions ($\xi > 3$). The relative uprush on a rough slope is monotonically increasing with ξ for the whole range of wave conditions. It is not possible to define a constant r value through the entire range of conditions. For rubble-mound breakwaters, r values outlined in the SPM (U.S. Army Corps of Engineers 1984) may be conservative for $\xi < 3.0$ but may exceed those values for $\xi \geq 3.0$ (see Figure 21). Van der Meer and Stam (1992) report that r values given by Battjes (1974b) and SPM (USACE 1984) are only valid for relatively gentle slopes with $\xi_p < 2$.

Regarding the structure characteristics, it may be difficult to determine the reduction factor due to uncertainties in the armour unit parameters, such as, porosity and roughness. The structure being examined may fall into one of the published categories (i.e., Tables 1, 4 and 5) and then the suggested reduction factor could be used. But the structure under consideration may have more porosity, such as the berm type revetment design, and therefore the reduction factor could be less than the value listed for the same category of structure (i.e., rip-rap revetment). In addition, the r reduction values presented are for the specific sizes tested in the laboratory investigations. Bruun (1985) notes that "almost all the coefficients with uprush on rubble mound slopes were obtained in the range of $D/H > 0.2$ where D stands for the average armour diameter" and H is the wave height. The rip-rap reduction factors of Stoa (1979), as presented in Appendix B, are for material with $0.2 \leq D/H \leq 0.33$. Stoa's r values increase as D/H decreases.

b) Composite Slopes and Berms

In many circumstances, the special bathymetry in front of a structure, such as offshore bars, composite slopes, berm in front of structure, etc. complicates the processes of wave uprush. When a structure is composed of two or more slopes, it is called a composite slope structure (see Figure 38).

Saville (1958) and Hunt (1959) proposed some methodologies to compute wave uprush for composite slopes. The method of Saville is the method presented in the SPM (USACE 1984). But these methods were only verified for a limited amount of case studies. The Technical Advisory Committee on Protection Against Inundation (1974) showed that Saville's method underestimates uprush for concave slopes. It is doubtful that the methods of Saville and Hunt can be applied to all situations. In fact, Bruun (1985) notes that Saville's method "should only be used in the range of geometric conditions in which experimental modification is available. Otherwise uprush may be underestimated."

Uprush is reduced in cases of slopes with a horizontal "berm" of width, b_w, (see definition sketch, Figure 39). Bruun (1985) summarized that a berm of a certain width is most effective when located at the average water level. Uprush decreases with increasing berm width, although past a certain value any further increase in width has little effect. The overtopping procedure of Owen (1982) includes provisions for a bermed slope. Pilarczyk (1990) and de Waal and van der Meer (1992) outline procedures for dealing with berms and composite slopes.

Computation of wave uprush for composite slopes and berm type structures is complicated and requires special attention. The procedures outlined here may be used for these types of structures, but it should be realized that a great amount of uncertainty is involved in the computed results.

c) Uprush Over Low Shores

A further complicating factor arises when the extent of the calculated uprush extends past the top of the shore or structure slope such as might occur along a low bluff or bank shoreline. For low bluffs, wave uprush may travel over the bluff crest similar to a wave bore (see Figure 40). Cox and Machemehl (1986) presented an extremely simplified approach to analyzing the overland propagation of the bore. The propagating bore is assumed to be a wave with an initial crest height equal to the uprush height if the bluff slope was extended. If the standing water depth, D (see Figure 40), is assumed to be 10% of H, then the distance of travel, or inland extent, x, of the wave bore is as follows:

$$x = \frac{T\sqrt{g}}{5}[(R-F)^{1/2} - H^{1/2}]$$

At the maximum inland extent ($x = L_s$), where H = 0, and

$$L_s = \frac{T\sqrt{g}}{5}(R-F)^{1/2}$$

Figure 38: Composite Slope Structure

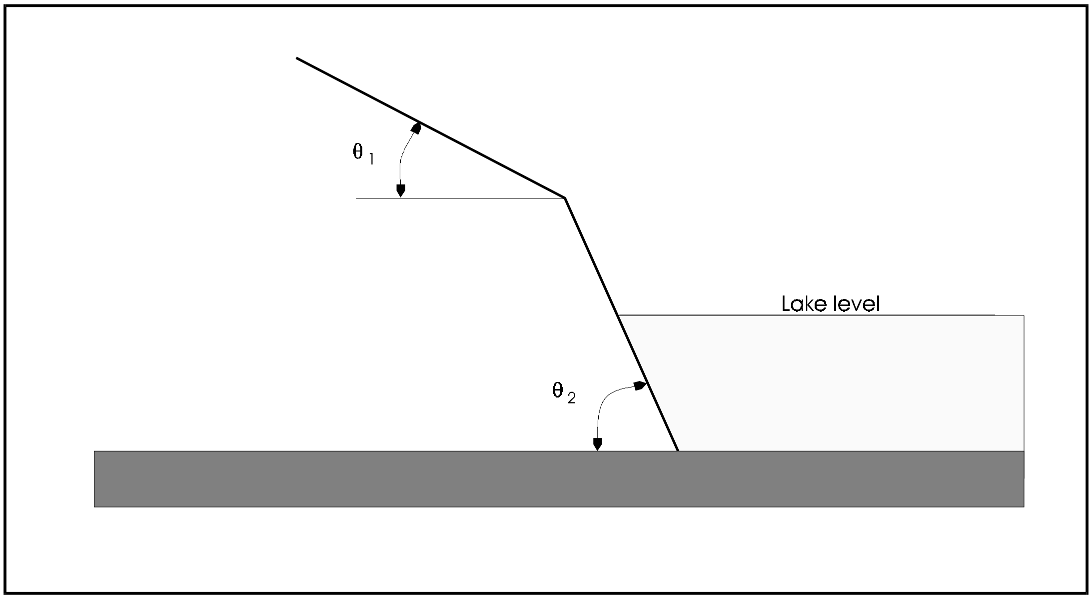


Figure 39: Definitions for Berm Slope (after Pilarczyk 1990)

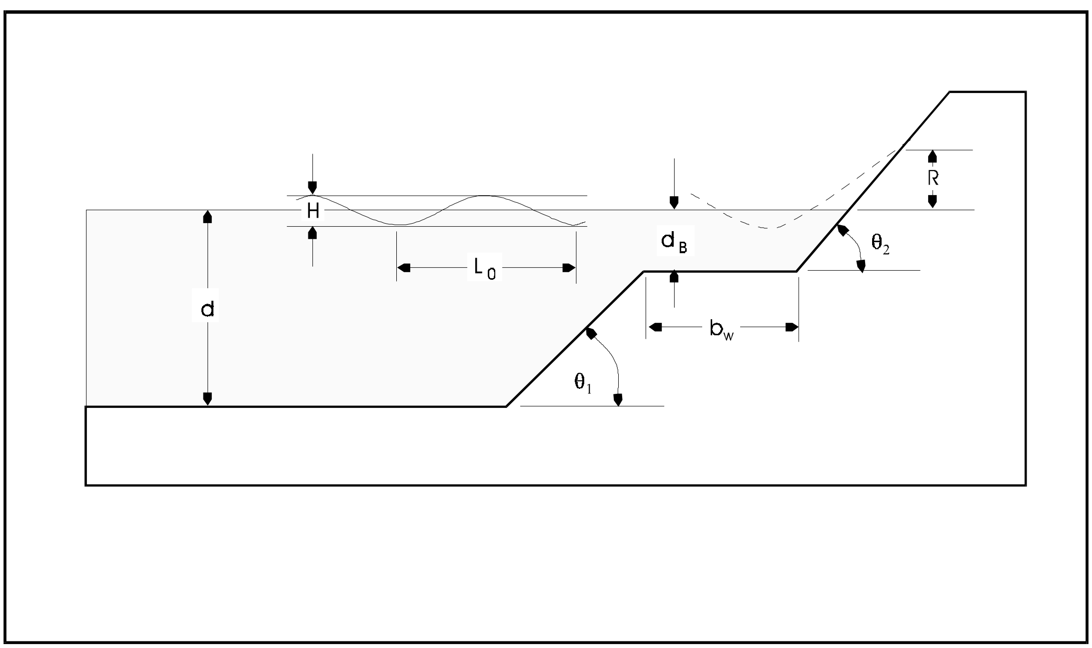
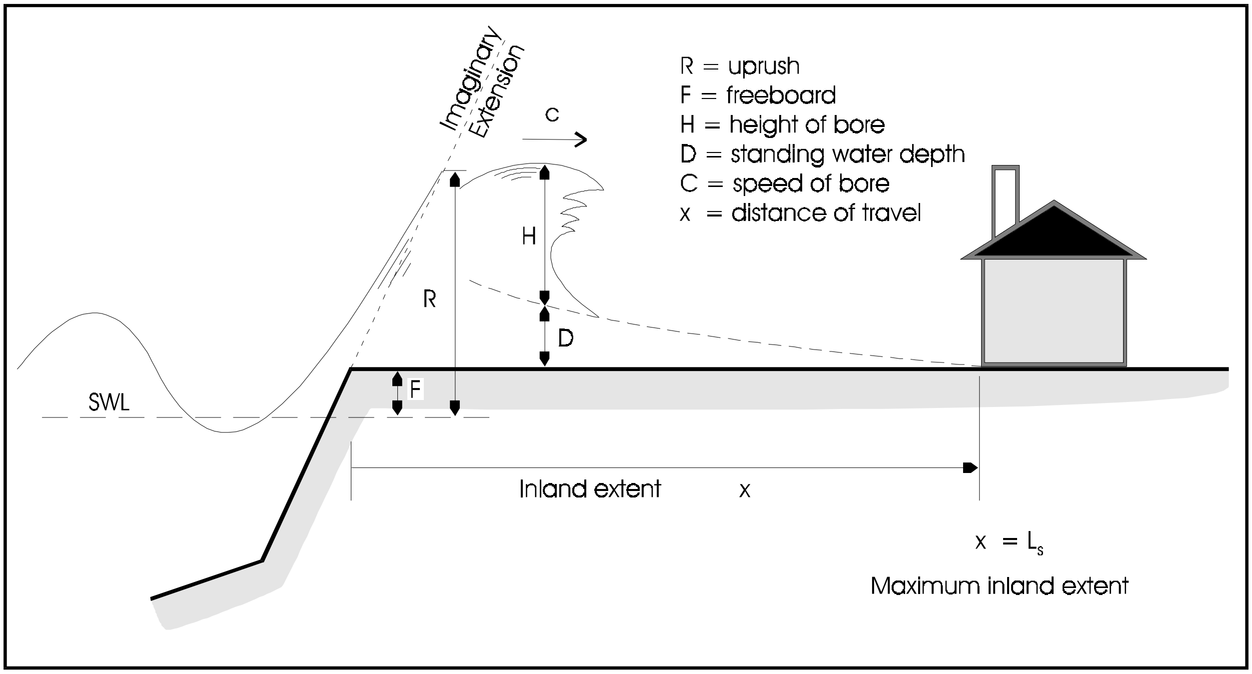


Figure 40: Definition Sketch of Wave Uprush Over Low Bluff (after Cox and Machemehl 1986)



FEMA (1991) recommends that where a bluff is overtopped by wave uprush, but not "flooded" (i.e., above flood level), and if there is a positive and mild slope on the landward side of the bluff crest, the inland extent of the wave uprush can be estimated using a methodology outlined in FEMA (1989). The FEMA (1989) procedure is based on some simplifications of the energy grade line, a Manning's "n" of 0.04 and is presented in Figures 41 and 42.

The method of Cox and Machemehl (1986) typically results in lower values of inland extent of uprush than the FEMA (1989) method. The results should be interpreted with care and allowances made for the uncertainty of the procedures.

4.6 Upper Limit of Wave Uprush

a) Upper-Bound Limit of Wave Uprush Procedures

All model data to which empirical uprush equations and curves are fitted, exhibit, for the most part, a significant degree of scatter (e.g., Figures 20 and 21). Therefore, it is possible that uprush values, in specific instances, are greater than the values predicted by the fitted equation or curve. In addition, the tested conditions were limited in scope. If a greater degree of certainty is required that the wave uprush predicted by the empirical methods will not be exceeded (e.g., for management of *flooding hazards* on the Great Lakes - St. Lawrence River System shorelines), an upper-bound limit of the "accepted" methods can be used.

The approximate upper-bound limit of the "accepted" uprush procedures for smooth slopes, is as follows:

$$\frac{R_s}{H_s} = 1.25\ \xi$$

together with and limited by the maximum value as a function of the structure (or beach) slope as given by Walton and Ahrens (1989)

$$\frac{R_s}{H_s} = \sqrt{2\pi}\left(\frac{\pi}{2\theta_r}\right)^{1/4}$$

where θ_r is in radians.

Figure 43 shows this upper-bound uprush method for smooth slopes. The smooth slope uprush values can then be modified by the slope surface reduction factors *r*, as required, for rough slopes. For upper-bound estimates, it is recommended that the values suggested by Walton et al. (1989a; 1989b) be used, as given in Table 4.

Figure 41: Uprush on Low Bluffs (New England Methodology) (after FEMA 1989)

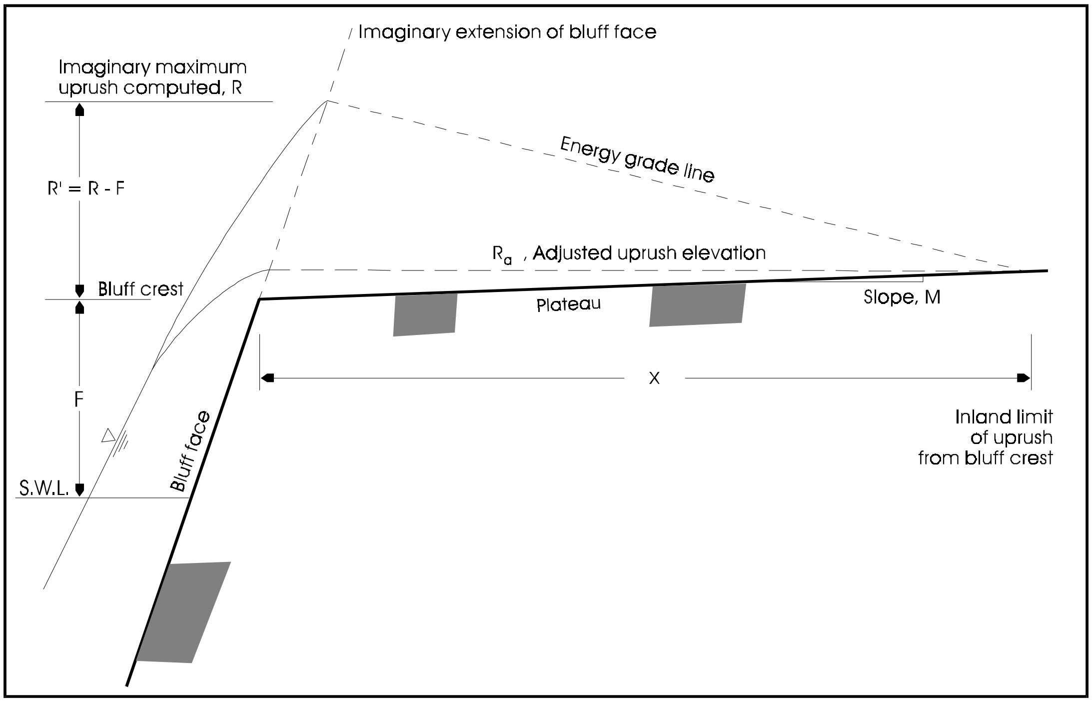


Figure 42: Computation of Uprush over Low Bluffs (New England Methodology) (after FEMA 1989)

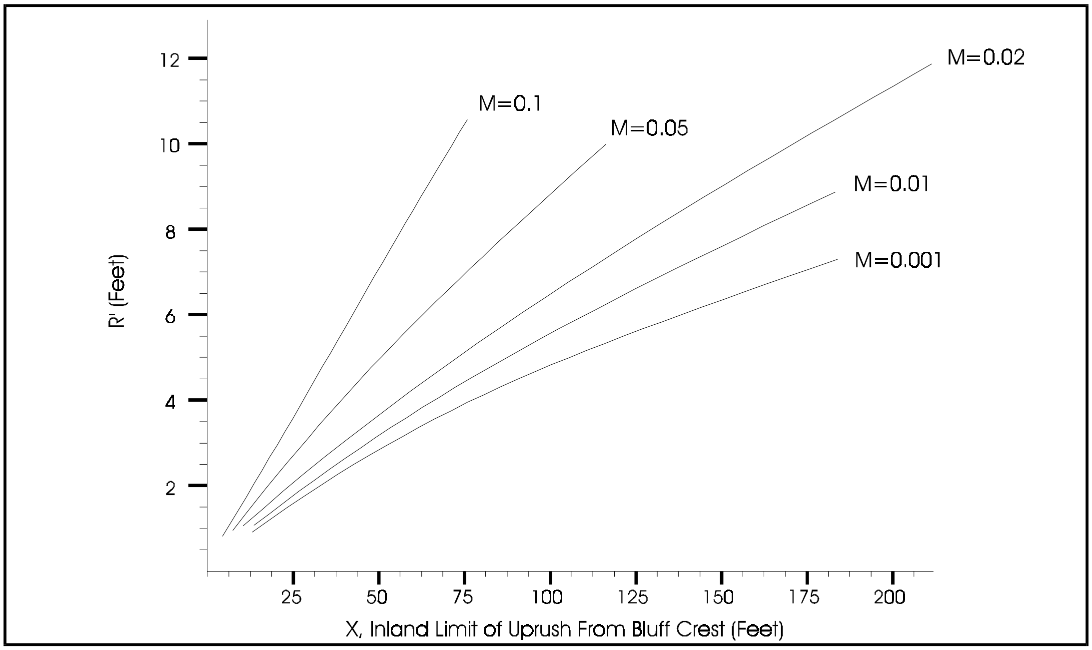
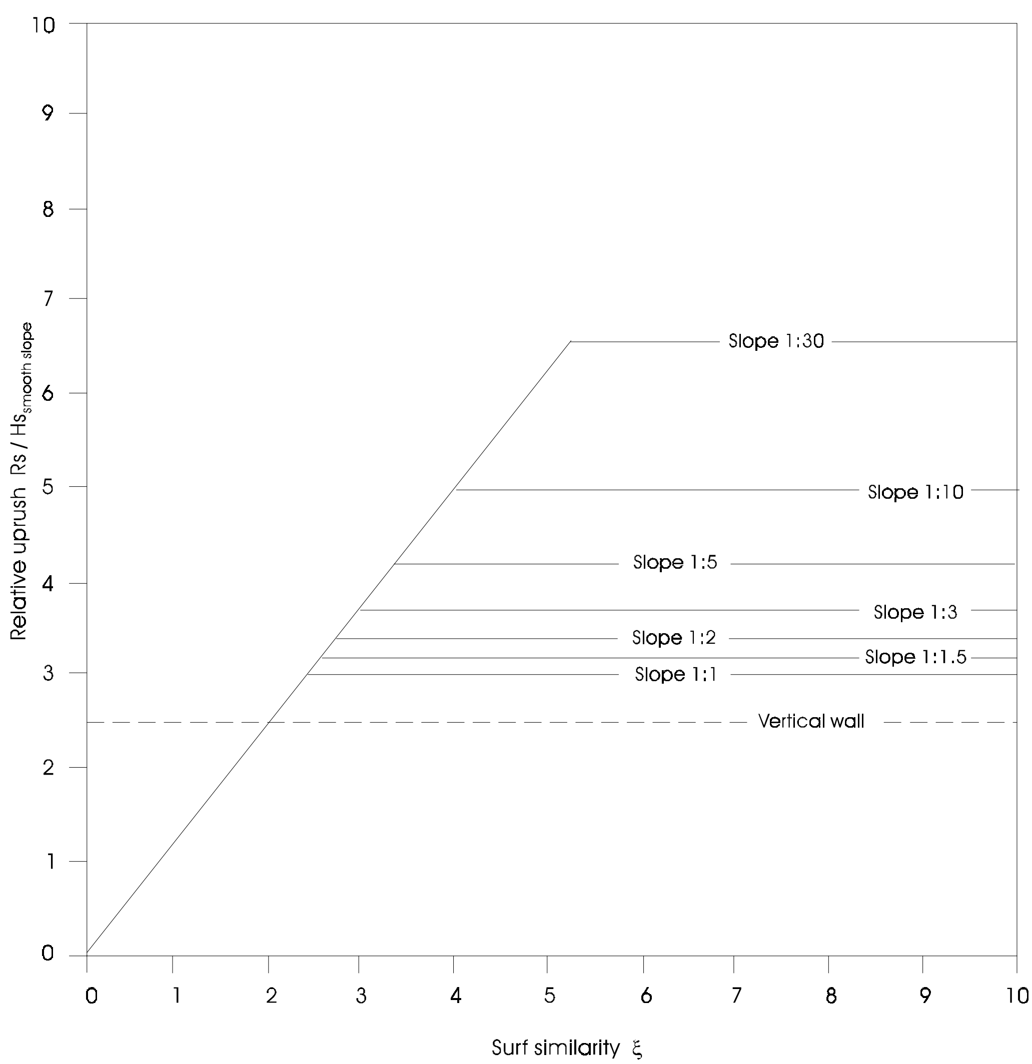


Figure 43: Relative Uprush: Upper-bound Curves for Smooth Slopes



For rough slopes apply the slope surface reduction factor, r.

$R/H_{rough\ slope} \approx R/H_{smooth\ slope} * r$

b) Distribution of Irregular Wave Uprush

During a storm, the maximum limit of wave uprush will exceed the value determined by R_s/H_s. In deep water, approximately 13% of the waves will be larger than H_s if Rayleigh distributed. This is because even though the wave conditions during a storm are characterized by the single value, H_s the waves actually vary in height (i.e., they are "irregular"). H_s represents the "significant wave height" and is described as the average of the highest one-third of all the wave heights. Therefore, there are waves that are higher than the H_s value. Additional discussion of wave parameters was presented earlier in Section 4.3(b).

It is also often assumed that wave uprush is Rayleigh distributed, resulting in:

$$R_2 = 1.4\ R_s \ ; \qquad R_2 = 2.23\ R_m$$

where R_2 is the uprush level exceeded by only 2% of all the uprush levels. As a result, overtopping will likely occur if the top height of the structure above the stillwater level, (i.e., freeboard, F) is less than the R_s value.

In shallow water, the assumption of Rayleigh distribution is not strictly correct. Due to the limiting effect of depth, the higher deep-water waves break before they reach the shore and there will be a truncation in the wave height distribution. Some research (Ahrens 1981a; Mase 1989) indicates that R_2/H_s is approximately equal to 1.3 to 1.4 times R_s/H_s. De Waal and van der Meer (1992) suggest for a gentle 1:100 approach slope and depth of water, d, the following relationships:

$$\gamma_d = \frac{H_2}{1.4H_s}$$

where

$$\gamma_d = 1 - 0.03\left(4 - \frac{d}{H_s}\right)^2 \qquad for \ 1 \le \frac{d}{H_s} \le 4$$

and

$$\gamma_d = 1 \qquad for \ \frac{d}{H_s} \ge 4$$

To obtain an upper-bound estimate of the R_2/H_s level, the upper-bound curves presented in Figure 41 could be increased by a factor of 1.4. Alternatively, the curves presented could be used with the local H_2 or H_{max} value at the toe of the protection work. H_2 is the wave height exceeded by only 2% of all the waves. H_{max} is the maximum wave height. H_2 or H_{max} could be estimated using an appropriate methodology such as Goda (1985) which is presented in Appendix C. However, as noted previously, the maximum wave does not necessarily produce the greatest uprush.

It may be appropriate to use different wave uprush values, R_m, R_s or R_2, for different design situations (e.g., non-habitable property, single dwellings or high density developments). R_m is the average uprush value. The R_2 uprush value could be considered as a reasonable limit for minimal overtopping for Great Lakes' shoreline protection works. Making allowances for safe overtopping is considered to be acceptable engineering design practice (see Section 3.0). As a general guide, if the freeboard is less than 2 times R_m, there may be considerable or excessive overtopping.

4.7 Guidelines for Acceptable Wave Overtopping Rates

The magnitude, duration and landward extent of overtopping water and wave spray are of direct concern in assessing shoreline flood susceptibility and degree of risk. The degree of risk is measured in terms of:

- the safety of persons and property behind the shoreline protection (i.e., usage considerations);
- the stability of the shoreline protection work itself; and
- the magnitude and impact of flooding/ponding landward of the protection work (i.e., drainage).

Protection works that permit a safe amount of wave overtopping are not uncommon and their proper design, installation and use is considered to be acceptable practice. Initial costs of protection works that preclude overtopping may be prohibitive and, depending on the proposed land use to the lee of the protection work, a non-overtopping work may not be necessary. The controlling criteria in any decision-making process is whether the intensity and/or the amount of wave overtopping endangers people or property, threatens the structural stability of the protection works and/or permits excessive water to pond in the onshore area.

When subject to overtopping, protection works must be carefully designed to withstand the forces of the overtopping water. Special attention must be given to the details of the crest and backside of the protection work to ensure that its stability is not jeopardized. Proper provisions for the drainage of the overtopping water must be specifically incorporated into the design of the shoreline protection work to prevent upland flooding and ponding. Design considerations for protection and drainage are provided in Sections 4.8 and 4.9 respectively. The following subsection provides a summary of acceptable mean overtopping rates based on a literature review as outlined in a later subsection.

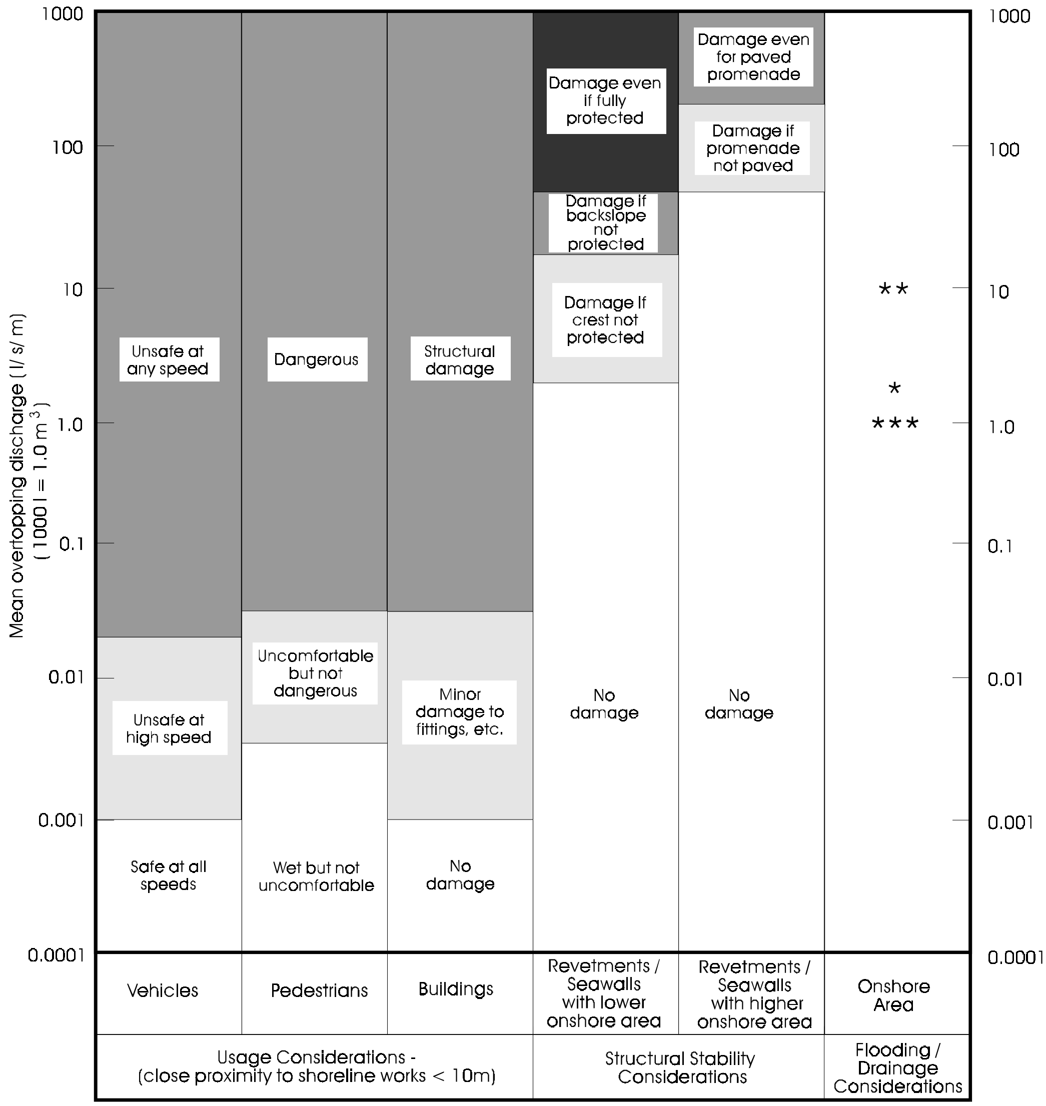
a) Summary of Acceptable Overtopping Rates

A summary of acceptable overtopping rates is presented in Figure 44. Figure 44 is adapted from CIRIA/CUR (1991) and is primarily based on the guidelines of Goda (1985) and Fukada et al. (1974) with additional information from Pilarczyk (1990) and Keillor and Miller (1987). The overtopping rate guidance provided by Figure 44 is grouped according to three factors:

- usage considerations;
- structural stability considerations of revetments/seawalls (with lower or higher onshore areas); and
- flooding/drainage considerations.

All three factors must be addressed when assessing the allowable rate of overtopping.

Figure 44: Summary of Acceptable Overtopping Rates (adapted from CIRIA / CUR 1991; GODA 1985; and Fukada et al. 1974)



* 2 l / s / m considered acceptable for dykes in Holland (Pilarczyk 1990).

** Suggested limit by GODA (1985) for utilization considerations and requires drainage channel of sufficient width (few tens of metres wide) and strong paving to withstand impact of falling water.

*** Suggested limit by GODA (1985) if ground elevation behind structure is lower than flood level or if insufficient facilities for proper drainage. Keillor and Miller (1987) suggest this limit for residential properties.

Usage is generally evaluated at water levels and wave conditions at normal operating times. For example, pedestrian walkways would be evaluated using typical storm conditions (waves and water levels) during the season(s) of primary use. It is not necessary to design a walkway to be fully protected from overtoppping at times of severe storms unless the walkway is a critical ingress/egress route or it has limited means of escape (i.e., long, isolated walkways with no intermediate points of egress). Structural stability and flooding considerations should be evaluated at the full flood level and associated storm conditions at any time of year.

. Usage

The usage guideline is subdivided into "vehicles", "pedestrians" and "buildings". These guidelines are generally applicable for situations in close proximity (less than 10 m) to the shoreline or protection works. A more quantitative assessment of the terms describing the degree of safety for pedestrians and vehicles is provided later in the literature review (e.g., see Fukada et al. (1974)). The study of Fukada et al. (1974) estimated the effects of overtopping rates on vehicles and pedestrians located 3 m behind the shoreline. The intensity of overtopping behind a structure decreases rapidly with the distance away from the shoreline. Jenson and Juhl (1987) and Muzik and Kirby (1992) found the decrease to be exponential.

It is important to note that during a storm, overtopping is characterized by sporadic, intense events when the larger waves overtop the shore. As noted, wind can also increase the intensity. It is during these brief, intense moments when most of the overtopping volume takes place. For example, over a period of one hour, approximately one-half of the total overtopping volume may be the result of the single largest overtopping wave (Jenson and Juhl 1987). The volume of water in this single overtopping wave, and the rapidness with which it occurs (of the order of a couple of seconds), will determine the hazard level. As such, it is not the average rate that determines the actual level of inconvenience or danger, although average rates can be used as criteria for acceptable overtopping. De Gerloni et al. (1991; see Section 4.7(b), *Literature Review of Acceptable Overtopping Rates*) found that an overtopping volume of approximately 0.1 m^3/m in one wave had about a 10% probability of knocking a person down. If that one wave represents approximately one-half of the total overtopping volume for 500 waves, and the average wave period is 6.5 s (i.e, slightly less than 1 hour duration), the average overtopping rate would be 0.06 l/s/m. This is in reasonable agreement with the guideline presented in Figure 44 for the lower limit of "dangerous" conditions for pedestrians.

Wind can also have an important influence on the quantity and extent of wave overtopping. Wave spray can be carried much further inland. Building and structure designers should be made aware of potential for significant icing during freezing weather.

. Structural Stability

"Revetments/seawalls with lower onshore area" are structures that are significantly higher than the backshore areas (see Figures 5a and 5b) and serve to protect the shoreline from flooding and erosion. These structures have also been described as "coastal dykes" (Goda 1985) or "embankment seawalls" (CIRIA/CUR 1991). The coastal dykes will have a crest and a backslope (see Figure 5b). "Revetment/seawalls with higher onshore area" are vertical seawalls or sloping revetments with top elevations that are not significantly higher than the backshore elevation and they primarily serve as erosion protection. They have also been termed "revetments" (Goda 1985) and "revetment seawalls" (CIRIA/CUR

1991). The term "promenade" refers to the area immediately behind the top of the structure and is also known as the "splash pad" or "splash area".

. Flooding/Ponding

Although it was noted that the overtopping intensity may decrease as you move away from the shoreline, the decrease in intensity does not necessarily diminish the flooding risk away from the shoreline. This is a particularly important consideration for low backshore areas which could be subject to flooding due to the total volume of water which comes over the top of the structure. Goda suggests that for the protection of a relatively densely populated coastal area, the overtopping rate of 0.01 m^3/s•m (10 l/s•m) is appropriate. Goda notes that the best way to handle this large volume of water (i.e., 3,600 m^3/hour for every 100 m of seawall) is "to provide a good channel of sufficient width and strong paving to withstand the impact of the falling water mass". The overtopped water should then be allowed to flow down the channel which should be "a few tens of metres wide if the overtopping rate is on the order of 0.01 m^3/s•m". Goda further states that if a wide channel can not be used, or if the ground elevation behind the seawall is lower than the flood level, the average overtopping rate, Q, should be approximately 0.001 m^3/s•m (1 l/s•m) or less. Keillor and Miller (1987) suggest that the latter (smaller) rate is more suitable for residential properties. In Holland, a value of 2 l/s•m is accepted for dykes (Pilarczyk 1990). De Vroeg et al. (1992) report that the design criteria for acceptable wave overtopping of sea defence works of a low lying area was 1.0 l/s•m at the 100 year design storm.

Design criteria for the seawall at the new airport at Chek Lap Kok in Hong Kong include: a) during any event with a 20 year return period, overtopping discharges shall not prevent the passage of vehicles adjacent to the seawall. For this purpose the average discharge volume behind the crest should be limited to 0.02 l/s•m; b) during any event with a 500 year return period, overtopping discharges shall not cause damage to unpaved ground behind the crest. For this purpose the average discharge volume behind the crest should be limited to 50 l/s•m.

Drainage considerations for overtopping water are discussed in Section 4.9.

b) Literature Review of Acceptable Overtopping Rates

. Goda (1985)

Goda (1985) outlined two seawall design philosophies. The first is to take the wave uprush height as the reference and to set the crest of the seawall higher than the uprush height so that no wave overtopping will occur. The inherent drawback with this approach is that extreme events can exceed the design event. The second approach is to take the wave overtopping amount as the reference and to set the crest elevation of the seawall at such a height as to keep the overtopping below some maximum tolerable quantity. With this second approach, the designer is aware of the existence of an overtopped water mass behind the seawall and is prepared to deal with it. The extent of damage in the event of an extraordinary storm will be much less than in the case of the uprush based seawall design approach.

The problem with applying the principle of overtopping based seawall design is how to determine the tolerance limit of wave overtopping. Goda analyzed about thirty cases of coastal dykes and revetments damaged in the aftermath of typhoons. This analysis yielded an estimate of the maximum tolerable rate of wave overtopping with respect to structural safety. Goda's rates are listed in Table 14.

Table 14 Allowable Overtopping for Structural Safety (after Goda 1985)

Coastal Structure Type	Surface Armouring	Overtopping Rate ($m^3/s \bullet m$)
Coastal Dyke (lower onshore area)	Concrete on front slope, with soil on crest and back slope.	less than 0.005
Coastal Dyke (lower onshore area)	Concrete on front slope and crest with soil back slope.	0.02
Coastal Dyke (lower onshore area)	Concrete on front slope, crest and back slope.	0.05
Revetment (higher onshore area)	No pavement of ground.	0.05
Revetment (higher onshore area)	Pavement of ground.	0.2

Goda also suggests that for the protection of a "relatively densely populated coastal area", the overtopping rate of 0.01 $m^3/s \bullet m$ is appropriate due to the utilization of this value by port areas in Japan. If the safe passage of vehicles is to be provided at all times along a coastal highway protected by a continuous seawall, the tolerable limit seems to be on the order of 10^{-4} $m^3/m \bullet s$.

. Fukuda et al. (1974)

Fukuda et al. (1974) present an evaluation of the effect of different wave overtopping rates based on prototype measurements and observations. They distinguished between the effect of the overtopping on different objects such as 1) a walking person; 2) a car; and 3) a building.

To evaluate the effect of overtopping on people and cars, a film was made simultaneously with measurements of the average rate. The film was later shown to eight experienced harbour engineers. The engineers were asked to evaluate the effect of the overtopping on cars and people placed 3 m behind the breakwater. The effect of the overtopping was divided into three classes "awful, a little", "awful", and "dangerous". Table 15 summarizes the results. The rates shown in Table 15 were by 10% of the observing engineers classified as one degree worse than indicated (safety standard 90%).

Table 15 Effect of Wave Overtopping (Fukada et al. 1974)

Degree of Inconvenience	Overtopping Rate, Q ($m^3/m\bullet s$)
Effect of wave overtopping on walking person 3 m behind the breakwater. Safety standard 90%.	
1. "awful, a little"	$< 4x10^{-6}$
2. "awful"	$4x10^{-6}$ - $3x10^{-5}$
3. "dangerous"	$> 3x10^{-5}$
Effect of wave overtopping on a car 3 m behind the breakwater. Safety standard 90%.	
1. "passable in high speed"	$< 10^{-6}$
2. "passable in slowing down"	10^{-6} - $2x10^{-5}$
3. "impassable in normal order"	$> 2x10^{-5}$

Irie (pers. comm.) provided the following "quantitative" interpretation of the terms used in Table 15:

> "The researchers showed a film of wave overtopping to eight engineers and requested them to evaluate on the following premises;
>
> 1) 'Awful a little' - one can walk without changing his posture even though being wet by a small spray due to overtopping.
> 2) 'Dangerous' - walking person can be thrust away or pushed down by a big spray due to overtopping.
> 3) 'Awful' - effects of overtopping on the walking person is in between 1) and 2).
> 4) 'Passable in high speed' - a vehicle can pass in high speed (i.e., roughly 60-80 km/h) just like it is running in a heavy rain.
> 5) 'Passable in slowing down' - a vehicle cannot pass without slowing down (this slow-downed speed can be taken roughly 10-30 km/h).
> 6) 'Impassable in normal order' - a vehicle is very hard to run because of wave overtopping (the vehicle might be thrust or damaged)."

. Jensen and Juhl (1987)

Jensen and Juhl (1987) present the available information on acceptable overtopping discharges and relate overtopping intensities to rainfall intensities. "The intensity of an extreme rainfall differs considerable from place to place with the largest intensities occurring in the tropics. In Denmark for example extreme rainfalls have an intensity in the order of $2x10^{-5}$ $m^3/m^2{\bullet}s$ while the world's largest reported intensity for a tropical rainfall and measurements in one minute is $5x10^{-4}$ $m^3/m^2{\bullet}s$. In order to compare such intensities with the intensity of wave overtoppings, it should be recognized...that the intensity of water falling behind a breakwater is highly irregular over time and that the maximum peak intensity is perhaps hundreds of times the average intensity."

"From model tests with a specific breakwater and based on visual observations and simultaneous wave measurements and measurements in the hydraulic model, it has been possible to estimate the intensities related to different degrees of wave overtopping. Notable overtopping corresponds to an average intensity of approximately $5x10^{-7}$ $m^3/m^2/s$. Inconveniences for the traffic on the roadway 3 m from the breakwater is experienced for an average intensity of approximately $1.5x10^{-6}$ $m^3/m^2/s$ [over] a distance of 1 to 15 m from the parapet wall. This means a discharge of approximately $2x10^{-5}$ $m^3/m{\bullet}s$. These figures seem to be in general agreement with the results of Fukuda [et al. (1974)]".

The following are "rough, preliminary and conservative guidelines for acceptable overtopping quantities" based on Fukuda et al. (1974) and related to rainfall quantities proposed by Jensen and Juhl.

> "Inconvenience for Persons - Inconvenience for persons behind a breakwater seems to occur for overtopping discharges of approximately $4x10^{-6}$ $m^3/m{\bullet}s$, corresponding to intensities of approximately 10^{-6} $m^3/m^2/s$ a few metres behind the breakwater. This intensity is only a fraction of the intensity of a heavy rainfall. The reason for such low intensities being of inconvenience is the highly irregular intensity of "spray-carry-over".
>
> Inconvenience for Vehicles - Inconvenience for vehicle traffic on a roadway just behind a breakwater occurs for an overtopping discharge of approximately 10^{-6} $m^3/m{\bullet}s$, corresponding to an intensity of approximately $3x10^{-7}$ $m^3/m^2/s$ a few metres behind the breakwater.
>
> Danger for Persons - An overtopping discharge of approximately $3x10^{-5}$ $m^3/m{\bullet}s$, corresponding to an intensity of approximately 10^{-5} $m^3/m^2/s$ a few metres behind the breakwater, is estimated as dangerous for persons. This is approximately 10 times the intensity which causes inconvenience.
>
> Impassable for Vehicles - From [Fukada et al. (1974)] it is estimated to be regarded as impossible to pass with a vehicle at a distance of 3 m from a breakwater when the average discharge of overtopping exceeds approximately $2x10^{-5}$ $m^3/m{\bullet}s$. Of course, this figure depends upon the type of vehicle."

. De Gerloni et al. (1991)

De Gerloni et al. (1991) performed a series of two dimensional model tests (1 to 20 scale) using irregular waves to assess the safety of overtopping rates on various rubble mound and vertical wall breakwaters. Mean overtopping rates and individual wave overtopping volumes were measured and their effect on people and vehicles was evaluated.

De Gerloni et al. reported that the crest elevation and wave period both have significant impacts on the mean overtopping rate. They measured overtopping volumes from individual waves and compared the maximum recorded individual volumes with mean overtopping rates. Although the two parameters appeared well correlated for different breakwater crest configurations, no unique relationship between the two parameters was found for all structure geometries. Figure 45 shows the correlation between maximum recorded volume per wave and mean overtopping discharge for the vertical seawall configuration they tested. De Gerloni et al. concluded that in addition to evaluating mean overtopping rates, the measurement of individual wave overtopping volumes is necessary when the safety of users is a concern.

Of particular interest was the use of model scaled vehicles and model scaled pedestrians to correlate individual overtopping volumes with safety. Vehicle modelling was accomplished with normal scaling practice, however the modelling of human behaviour presented a rather unique challenge. To accomplish this, a full size model human, or dummy, was developed whose weight was increased until the action of the model human (i.e., being knocked down) and a real human were similar under a surge of water of known volume. From this the equivalent dummy weight in prototype was determined which allowed for scaling. It was determined that a volume of 4 m^3/m/wave of overtopping water had a 90% probability of moving a car (see Figure 46) while a volume of 1.4 m^3/m/wave of overtopping water had a 90% probability of causing a person to fall (see Figure 47). A volume of about 0.1 m^3/m/wave of overtopping water had about a 10% probability of causing a person to fall.

4.8 Uprush and Overtopping Protection

The stability of a shoreline structure can be threatened when it is overtopped by waves. Overtopping can erode the area behind or above the structure thereby removing the material which supports it and failure of the structure can occur. A possible mode of failure is shown in Figure 48. The effects of overtopping can be reduced by increasing the crest elevation (i.e., increase the freeboard, *F*) or by protecting the area behind the structure (i.e. sufficient protection and drainage). The protection behind the crest, or the "splash pad", must have sufficient mass or strength to resist the overtopping wave action. Depending on the severity of the overtopping, the splash pad can consist of armour stone, rip-rap, concrete or asphalt pavement, proprietary reinforced grass or soil products or other suitable materials. A proper bedding and filter must be provided. Improper detailing of the connection between the splash pad and the primary protection works can lead to or contribute to the failure of a structure (see Figure 49). A literature review of wave overtopping protection methods, including Abt and Johnson (1991) and Pilarczyk (1990), is provided in the following subsections.

. Abt and Johnson (1991)

Abt and Johnson (1991) conducted near prototype flume studies in which rip-rap protected embankments were subjected to overtopping flows. Undersizing of the rip-rap or layer thickness may result in a fluidization of the protective layer, subjecting the embankment to severe erosive processes.

Figure 45: Correlation Between Mean Overtopping Discharge and Maximum Recorded Volume (after De Gerloni et al. 1991)

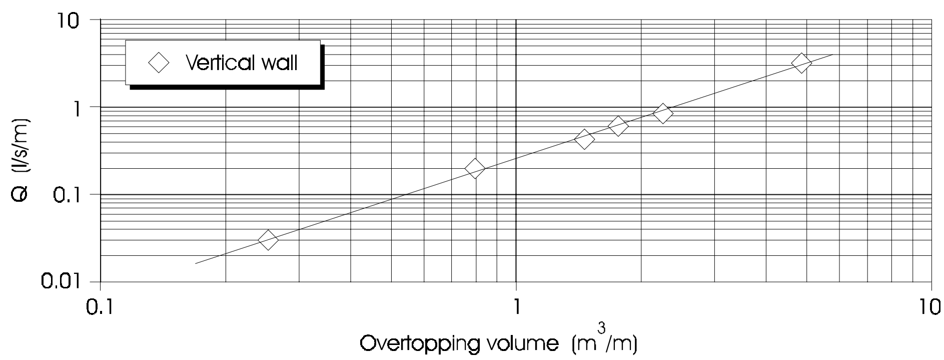


Figure 46: Overtopping Effect on Model Cars (after De Gerloni et al. 1991)

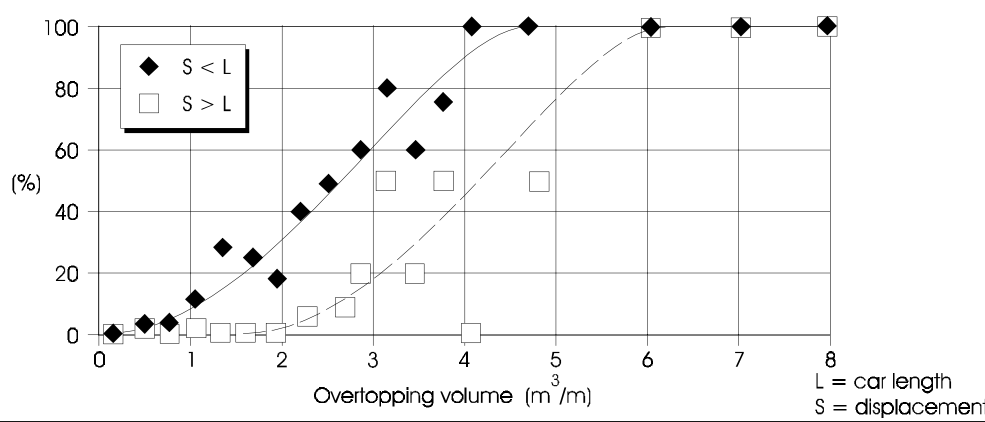


Figure 47: Overtopping Effect on Model Humans (after De Gerloni et al. 1991)

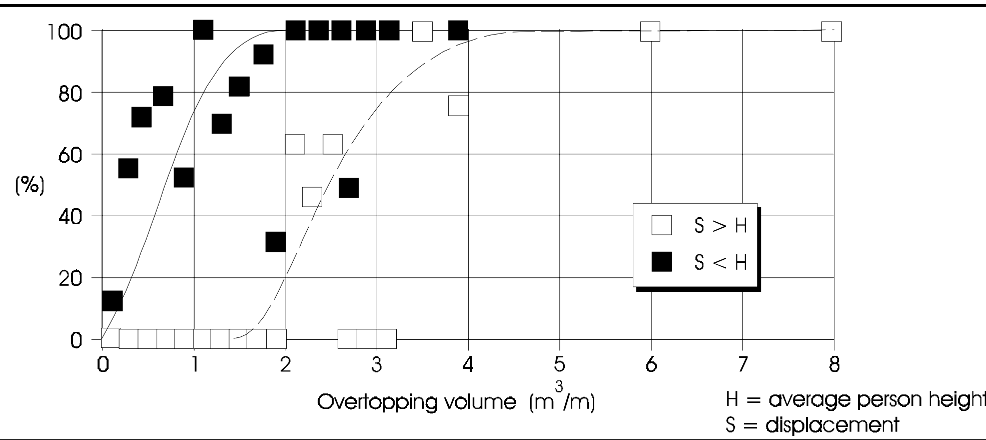


Figure 48: Collapse Mode of Revetment (after Smith and Chapman 1982)

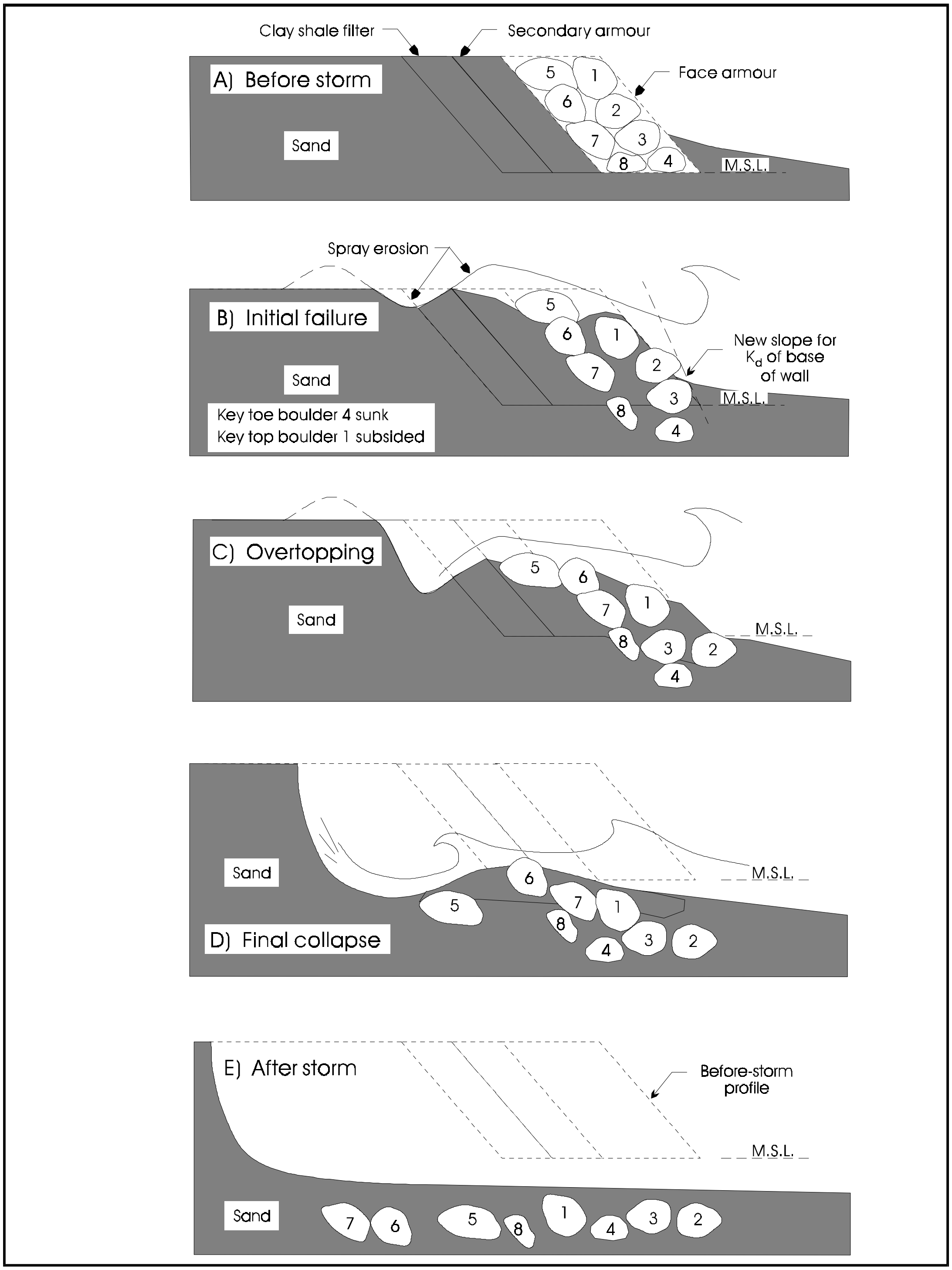


Figure 49: Wave Overtopping Damage

In an attempt to determine the rip-rap layer stability for angular shaped stones when subjected to overtopping flow, the rip-rap layer median stone size D_{50} was correlated to the overtopping unit discharge at failure, q_f, (q_f should be a momentary discharge per characteristic wave and not the time-averaged discharge, Q) and found to be:

$$D_{50} = 5.23\, S^{0.43}\, q_f^{0.56}$$

where S is the embankment slope (this function assumes a rip-rap specific gravity of 2.65).

Incipient stone movement occurred at approximately 74% of the rip-rap layer failure unit discharge. It is imperative that the rip-rap layer be designed to prevent failure therefore the median stone size should be sized to resist stone movement. To account for this Abt and Johnson recommend sizing the stone based on a design flow rate which is 1.35 times that of the overtopping flow rate:

$$q_{design} = 1.35\, q_f$$

In this way, the median stone size is designed to resist stone movement using the design unit discharge as follows:

$$D_{50} = 5.23\, S^{0.43}\, (q_{design})^{0.56}$$

where: D_{n50} in inches; and q in cubic feet per second.

It was determined that rounded stones should be oversized approximately 40% to provide comparable protection of angular stone. Also, flows can concentrate and form subchannels in the riprap layer. A flow concentration factor may be incorporated into the stone size analysis by multiplying q_f by a factor of approximately 1.0 to 3.0. The factor selected will depend upon the hazard level of the protected area.

. Pilarczyk (1990)

Pilarczyk (1990) says that no "definite method for designing against overtopping is known" because there is no proper way, as of yet, to estimate the hydraulic loading. He notes that in the standard Dutch practice a safe value of 0.002 $m^3/m{\bullet}s$ for grassed crest and rear slope is recommended. Recent experience suggests that this can be increased to 0.005 $m^3/m{\bullet}s$ or even to 0.01 $m^3/m{\bullet}s$ for "good" quality grass mat on clay sub-layer.

The following formula is proposed by Pilarczyk (1990) to aid in the design of crest armour stone protecting from overtopping flows. For rock, the equivalent stone diameter, D_{50} of the protective rock unit can be computed using the following:

$$\frac{H_s}{\Delta D_{50}} = \frac{2.25\ \cos\alpha}{\xi^{0.5}\left(1-\frac{F}{R}\right)^2}$$

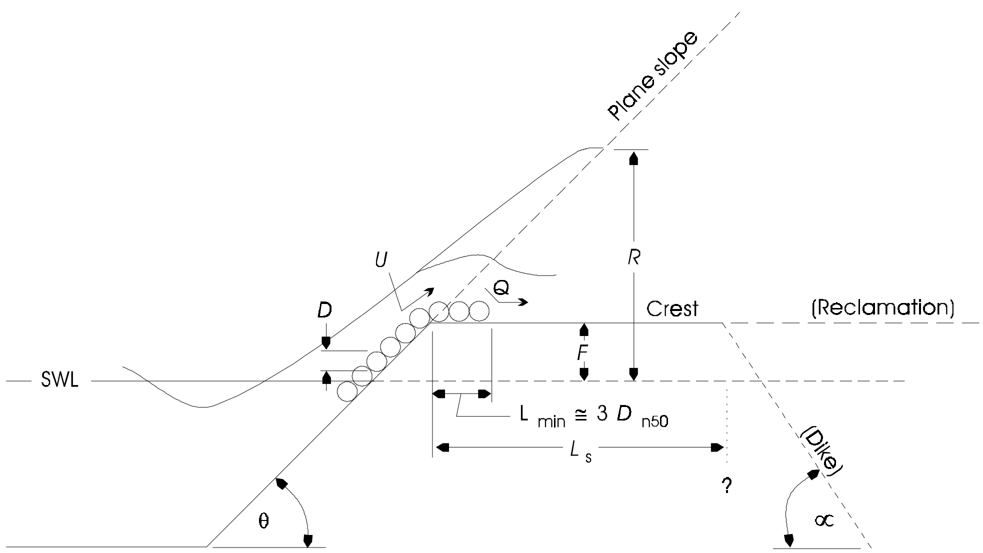
where α is the inner slope angle (see Figure 50), ξ is the surf similarity parameter, and R is the wave uprush on a plane slope.

The width of protection in the splash area L_s may be determined using the following equation (based on Cox and Machemehl 1986, see Section 4.5(c)):

$$L_s = \frac{\Psi}{5} T \sqrt{g(R - F)} \geq L_{\min}$$

where Ψ is an engineering judgement factor, more or less equal to 1, that is related to the local conditions and the importance of the structure. Figure 50 provides a definition sketch. L_{min} should be regarded to be about 3 times a rock unit dimension (i.e., $3D_{50}$). Pilarczyk notes that outside the area of L_s, the protection can be eventually extended with much finer units and a small sill at the end of the protected area can also be useful for erosion control.

Figure 50: Definition Sketch for Overtopping Protection (after Pilarczyk 1990)



Pilarczyk presents the method of Knauss (1979) for estimating the stable rock size dimension for overtopping breakwater to be used on the crest and backslope. The advantage of this method is that the maximum overtopping rate, q_f, can be used directly in the computation. The simplified version of the stability formula is:

$$q_f = 0.625\sqrt{g}\,(\Delta D_{50})^{1.5}\,(1.9 + 0.8\phi_p - 3\sin\alpha)$$

where ϕ_p is the stone arrangement packing factor ranging from 0.6 for natural dumped rockfill to 1.1 for optimal manually placed rock, and 1.25 for placed blocks.

4.9 Drainage Provisions

In the design of seawalls, the drainage system for the overtopped water should be well planned, to ensure that the volume of overtopping water due to storm waves, which can be considerable, is properly calculated and accommodated. The drainage system must ensure rapid drainage otherwise flood recovery operations may be hampered.

Factors to take into consideration, when designing a proper shoreline drainage system for any development, include the following:

- the temporal and spatial distribution of the overtopping water (i.e., most of the water volume is the result of a relatively small number of the larger waves during the course of a storm resulting in short periods of high flow with the greatest intensity closest to the protection works);

- while overtopping intensity may decrease as you move away from the shoreline, the increased distance does not diminish the flooding risk due to the total volume of water which comes over the structure, especially for low backshore areas;

- the potential for blockage of drains by wave carried sediments and debris; and

- the potential freezing or icing of the drains by wind-driven spray.

The predictors of wave overtopping can provide estimates of the average overtopping rate. Using established, standard civil engineering drainage design methods as a guide, a designer should then ensure that these factors are incorporated into a drainage system for the selected protection works. The resulting shoreline drainage system should be much larger than typical inland systems.

Additional guidance with respect to drainage considerations are outlined in Section 4.7(a).

Where drainage provisions landward of a protection work are uncertain or are not sufficient, the elevation of the development should be greater than the level of the limit of wave action (i.e., the development must meet the conditions of the provincial *floodproofing standard*).

5.0 EXAMPLE WAVE UPRUSH AND OVERTOPPING COMPUTATIONS

5.1 Wave Uprush

a) Example 1 - Sandy Beach

i) Site Conditions

The example site is a sandy beach situated on Lake Ontario. Figure 51(a) shows the beach profile extending to the offshore. Figure 51(b) shows the nearshore profile. The data points for the nearshore profile are given in Table 16. For the purpose of wave uprush calculations, the beach was considered to be a smooth impermeable surface (i.e., $r = 1.0$).

The deep water significant wave height, H_{os}, and the wave period, T_p, were provided in MNR (1988a) from the deep-water wave hindcast station nearest to the site (see Figures 52 and 53). Typical severe storm values are: $H_{os} = 4.20$ m; $T_p = 8.0$ s. The deep-water wavelength, L_{op}, is computed to be 99.9 m. Other values of H_{os} and T_p should be examined as well, to determine the maximum uprush for the site. If the refraction coefficient, K_r, is assumed to be 0.6 for the purpose of this example, the unrefracted wave height H'_{os} = 2.52 m. Assuming $T_m \approx 0.9T_p$, and a Rayleigh distribution, the mean wave conditions are: $H'_{om} = 0.626\,H'_{os} = 1.58$ m; $T_m = 7.2$ s; and $L_{om} = 80.9$ m.

The 100 year flood level was found to be 75.90 m. This elevation will be considered as the stillwater level (SWL) for uprush calculations.

Using the Dewberry & Davis (1990) profile segmentation method for the sandy beach site with SWL = 75.90 m and H'_o = 2.52 m, then (X'_n, Y'_n) is found to be (65.6, 78.40) and (X_n, Y_n) is (149.0, 75.83). The beach slope M_s and the nearshore slope M_a are found to be 1:34 and 1:91 respectively.

The toe of the structure for a sandy beach is the depth at the point where the beach slope and nearshore slope meet. This point was obtained from the profile data as (244.0 m, 73.23 m). Thus d_s = SWL - 73.23 = 2.67 m.

The significant wave height at the toe of the structure, H_{sds}, can be estimated using Goda's (1985) method (see Appendix C). With $\frac{H'_{os}}{L_{op}} = 0.025$ and $\frac{d_s}{H'_{os}} = 1.06$, from Appendix C one gets $\frac{H_{sds}}{H'_{os}} = 0.70$, with nearshore slope of 1:100. Therefore $H_{sds} = 1.76$ m.

Table 16: Profile Data for Sandy Beach Example

X_n, Distance (m)	Y_n, Elevation (m)
43	79.10
149	75.83
244	73.23
445	71.02
649	69.95
844	68.96
1045	67.67
1250	66.14
1445	65.76
1649	65.91
1945	64.62

ii) Uprush Calculations

Uprush was calculated using the methods of Stoa (1978b), Hunt (1959) and Mase (1989).

. Stoa (1978b)

Mean wave conditions were used, as prescribed by Dewberry and Davis (1990), to estimate the mean uprush, R_m. With $\frac{d_s}{H'_{om}}$ = 1.69 and $\frac{H'_{om}}{(gT_m^2)}$ = 0.0031, using Appendix A (Figure 2) for $\frac{d_s}{H'_o} \leq 3.0$ and extrapolating cot θ = 34, results in $\frac{R_m}{H'_{om}}$ = 0.20. Thus the uprush level using Stoa (1978b) is R_m = 0.32 m. From R_m, $R_s = 1.6R_m$ = 0.51 m and $R_2 = 2.23R_m$ = 0.71 m.

Figure 51: Example Sandy Beach

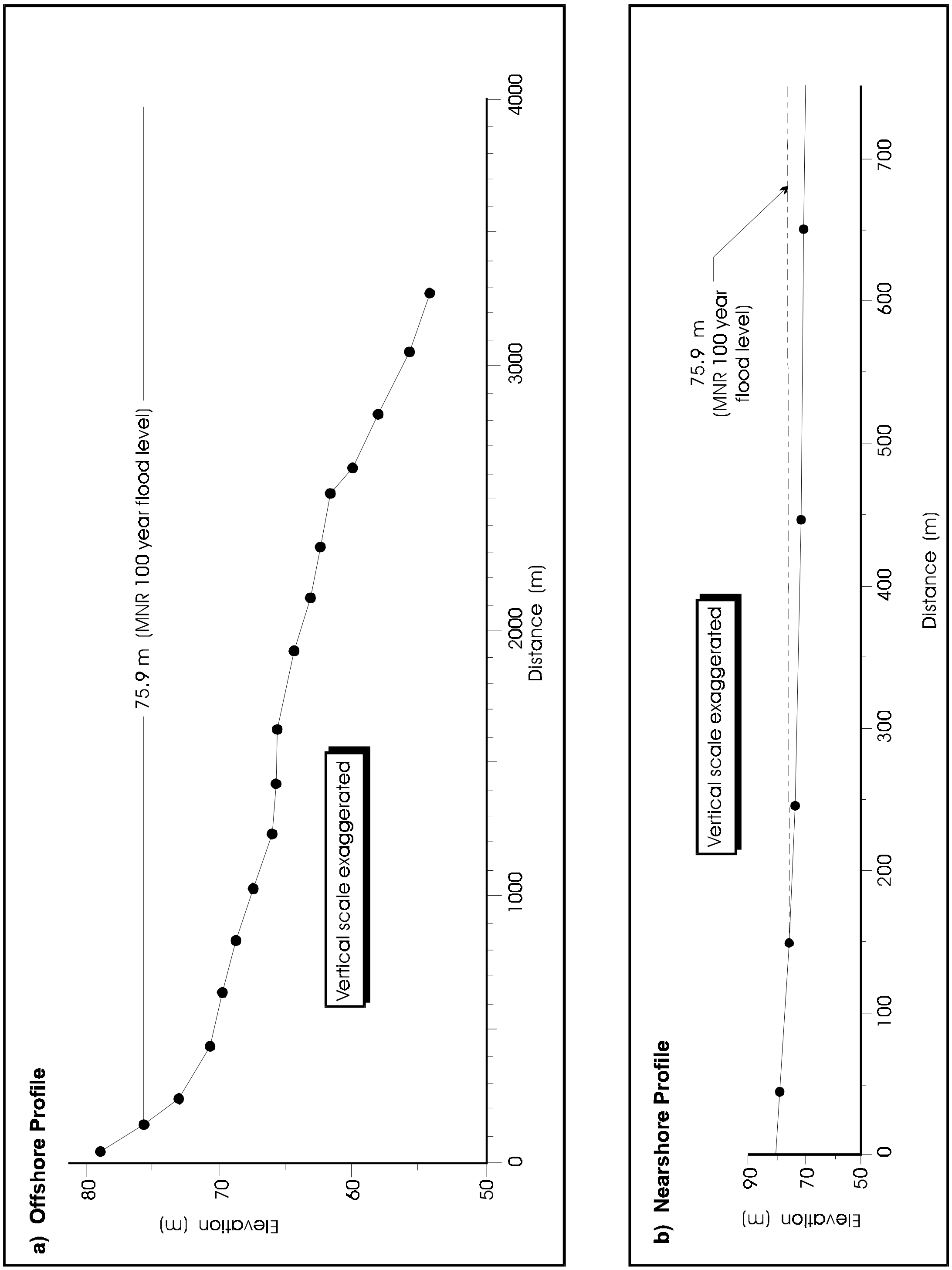


Figure 52: Example Summary of MNR Deepwater Wave Data

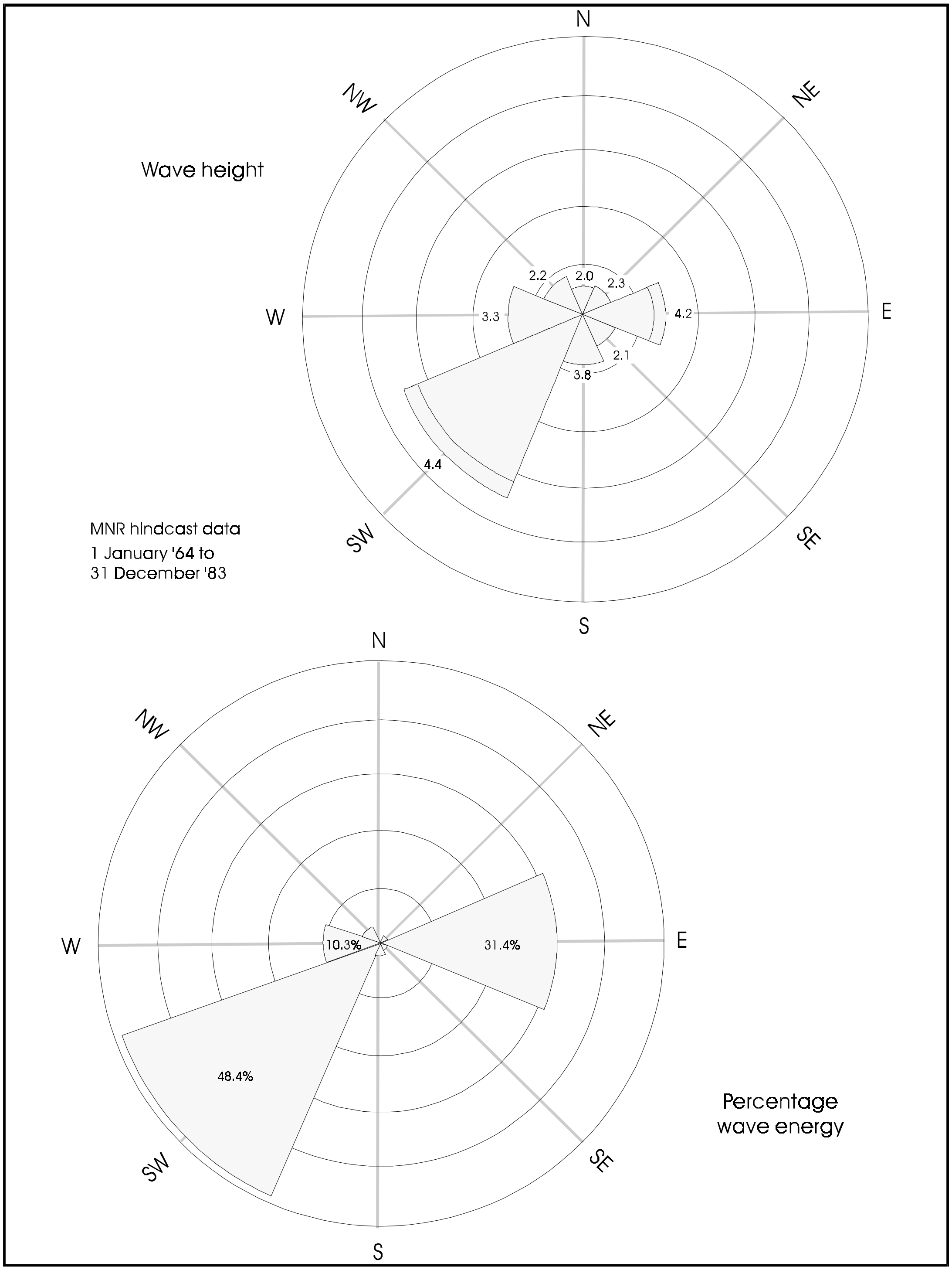


Figure 53: Typical Frequency Table of Wave Climate

FREQUENCY TABLE OF SIGNIFICANT WAVE HEIGHTS AND PEAK PERIODS

Selected from 1 JAN 1964 to 31 DEC 1983 | **Wave Direction: ALL**

WAVE HEIGHT (METRES)	NUMBER OF OCCURRENCES WAVE PERIOD (SECONDS)										
	0.0 TO 1.0	1.0 TO 2.0	2.0 TO 3.0	3.0 TO 4.0	4.0 TO 5.0	5.0 TO 6.0	6.0 TO 7.0	7.0 TO 8.0	8.0 TO 9.0	9.0 TO 10.0	ROW TOTAL
0.00 - 0.25	953	14784	6403	0	0	0	0	0	0	0	22140
0.25 - 0.50	0	9	54657	5106	0	0	0	0	0	0	59772
0.50 - 0.75	0	0	2688	37029	482	0	0	0	0	0	40199
0.75 - 1.00	0	0	0	13982	7101	0	0	0	0	0	21083
1.00 - 1.25	0	0	0	68	12339	543	0	0	0	0	12950
1.25 - 1.50	0	0	0	0	5693	2284	0	0	0	0	7977
1.50 - 1.75	0	0	0	0	194	4155	144	0	0	0	4493
1.75 - 2.00	0	0	0	0	0	2487	333	0	0	0	2820
2.00 - 2.25	0	0	0	0	0	708	991	13	0	0	1712
2.25 - 2.50	0	0	0	0	0	6	909	38	0	0	953
2.50 - 2.75	0	0	0	0	0	0	484	54	0	0	538
2.75 - 3.00	0	0	0	0	0	0	172	130	1	0	303
3.00 - 3.25	0	0	0	0	0	0	14	151	0	0	165
3.25 - 3.50	0	0	0	0	0	0	0	73	2	0	75
3.50 - 3.75	0	0	0	0	0	0	0	32	6	0	38
3.75 - 4.00	0	0	0	0	0	0	0	9	10	0	19
4.00 - 4.25	0	0	0	0	0	0	0	0	21	0	21
4.25 - 4.50	0	0	0	0	0	0	0	0	10	0	10
4.50 - 4.75	0	0	0	0	0	0	0	0	0	0	0
COL TOTAL	953	14793	63748	56185	25809	10183	3047	500	50	0	175268

Number of Hourly Records This Direction	175268
Total Records Used in Selection Interval	175272
Per Cent in this Direction	100.00

. Hunt (1959)

The equation for uprush is given as $\frac{R}{H} = \frac{\tan\theta}{\sqrt{H/L_o}} = \xi$. Again using the mean wave conditions outlined earlier, $\frac{R_m}{H'_{om}}$ = 0.21 and R_m = 0.33 m.

If the significant wave height at the toe of the structure H_{sds} = 1.76 m is used along with T_p, $\frac{R_s}{H_{sds}}$ is computed to be equal to 0.22 and R_s = 0.39 m.

. Mase (1980)

With the local significant wave height at the toe of the slope, H_{sds} and L_{op}, the equations of Mase (1989) result in values for R_m, R_s and R_2 of 0.55 m, 0.85 m and 1.12 m respectively.

Table 17 summarizes the uprush results along with various other input wave parameters For this sandy beach example, the Stoa and Hunt yield similar results. Mase predicts higher values as may be expected from Figure 7. Also, as discussed in Section 2.1, the Mase equations provide an upper envelop of scattered test observations. The horizontal allowances for wave uprush, based on the R_2 values for the severe storm, are 24 m for Stoa and 38 m for Mase.

b) Example 2 - Sloped Armour Stone Revetment

i) Site Conditions

The site selected is a typical fine-grained cohesive profile situated in Lake Ontario (profile type 3 according to Boyd, 1981). An armour stone revetment was proposed for the site. The toe of the structure will be placed at an elevation of 74.00 m and it was designed with a 1:2 slope (see Figure 54(a)). The nearshore survey is plotted in Figure 54(b) and the profile data are given in Table 18.

The deep-water wave conditions are the same as were used in Example 1 (i.e., H_{os}= 4.2 m; T_p=8.0 s; H'_{om}=1.58 m; T_m=7.2 s H'_{os} = 2.52 m). The 100 year flood level was found to be 75.90 m. This elevation was considered to be the stillwater level (SWL). The method described in Dewberry & Davis (1990) was used to separate the profile into structure and approach segments. In this case, the structure slope was defined by the armour stone revetment. Therefore M_s is 1:2. The toe of the structure is located at (70.7 m, 74.00 m), the next seaward point is (85 m, 73.75 m). Hence, the approach slope M_a is found to be 1:57.

Table 17: Uprush Results for Sandy Beach Example

Case	Uprush, R (m)		
Beach Slope = 1:34	**Stoa (1978b)**	**Hunt (1959)**	**Mase (1989)**
	Using H'_{om} and T_m	**Using H'_{om} and T_m**	**Using H_{sds} and T_p**
H'_{os}=2.52 m; T_p=8.0 s H'_{om}=1.58 m; T_m=7.2 s H_{sds} = 1.76 m	R_m = 0.32 R_s = 0.51 R_2 = 0.71	R_m = 0.33 R_s = 0.53 R_2 = 0.74	R_m = 0.55 R_s = 0.85 R_2 = 1.12
H'_{os}=1.75 m; T_p=6.0 s H'_{om}=1.10 m; T_m=5.4 s H_{sds} = 1.58 m	R_m = 0.21 R_s = 0.34 R_2 = 0.47	R_m = 0.21 R_s = 0.34 R_2 = 0.47	R_m = 0.42 R_s = 0.65 R_2 = 0.85
H'_{os}=1.00 m; T_p=7.0 s H'_{om}=0.63 m; T_m=6.3 s H_{sds} = 1.20 m	R_m = 0.15 R_s = 0.24 R_2 = 0.33	R_m = 0.18 R_s = 0.29 R_2 = 0.40	R_m = 0.36 R_s = 0.60 R_2 = 0.74

The depth at the toe of the structure is d_s = SWL - 74.0 = 1.90 m.

The significant wave height, H_{sds}, at the toe of the structure can be estimated using Goda (1985).

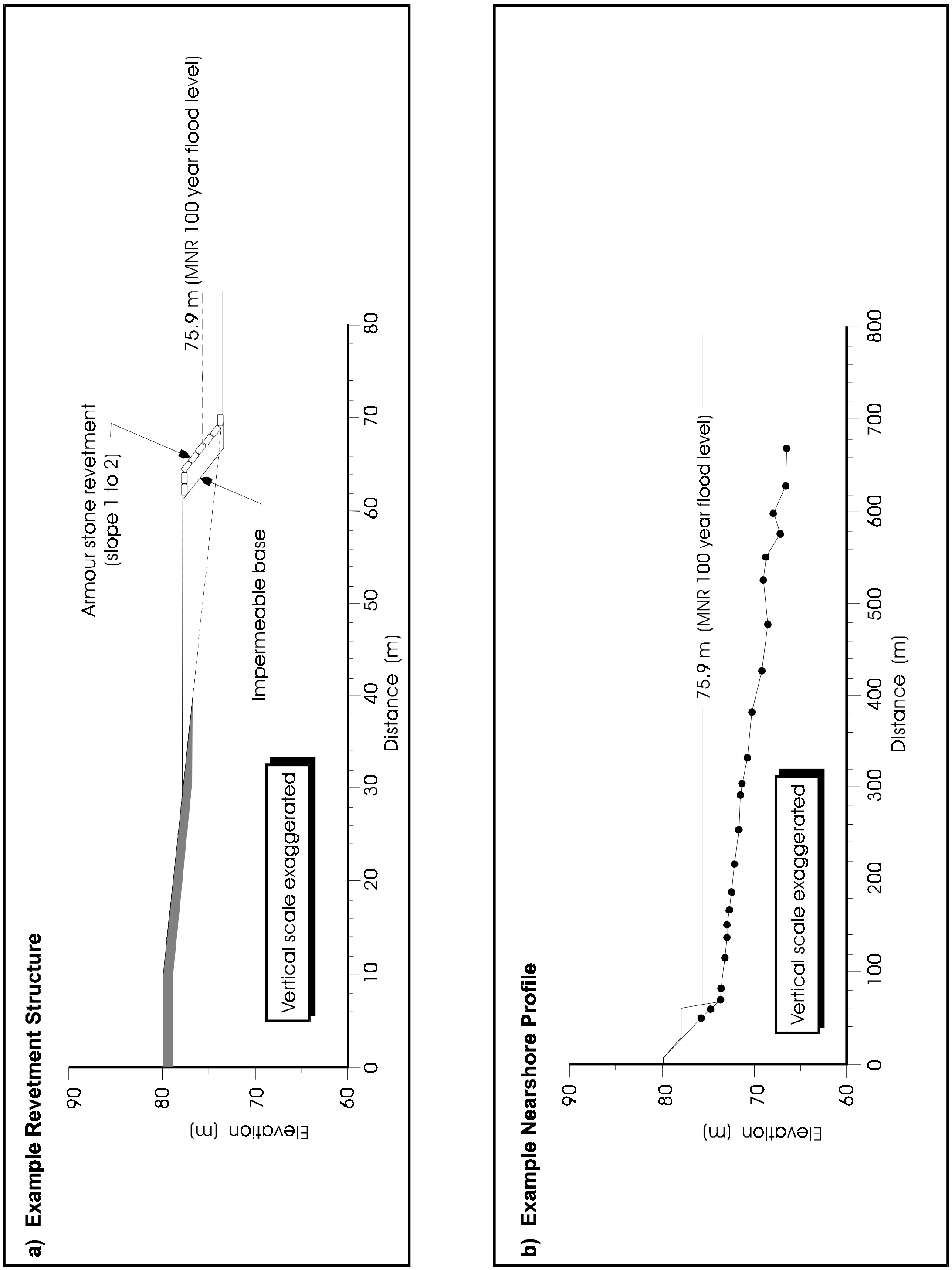
With $\dfrac{H'_{os}}{L_{op}}$ = 0.025 and $\dfrac{d_s}{H'_{os}}$ = 0.75, using Appendix C gives $\dfrac{H_{sds}}{H'_{os}}$ = 0.56. Hence, H_{sds} = 1.41 m.

Since H_{mo} is required for wave overtopping computation, the procedure described in Section 4.3 is employed here to estimate the wave height H_{mo} and wavelength L_p at the depth d_s. For T_p = 8 s, the cutoff wave period at d_s is computed to be $T' = 2\pi\sqrt{\dfrac{d_s}{g}}$ = 2.77 s. Since $T \geq 1.5\ T'$, the wavelength at d_s is $L = T\sqrt{g\,d_s}$ = 34.54 m. With U = 65 km/h (or 17.88 m/s), the wave height H_{mo} at d_s is estimated to be 1.51 m, which is greater than the breaking wave height ($H_{mo,b}$ = 0.6 d_s = 1.14 m). Therefore, H_{mo} is set equal to 1.14 m.

Table 18: Profile Data for Armour Stone Revetment Example

X_n, Distance (m)	Y_n, Elevation (m)
53	75.85
63	74.85
72	73.85
85	73.75
118	73.35
140	73.15
154	73.15
170	72.75
190	72.55
220	72.25
256	71.75
294	71.15
306	71.35
334	70.85
384	70.35
430	69.25
480	68.55
530	69.15
554	68.85
580	67.25
602	68.05
632	66.85
672	66.65

Figure 54: Example Revetment Structure and Nearshore Profile



Using the calculated information, the values of dimensionless depth parameter

$$\bar{d} = \frac{d_s}{g\,T_p^2}$$

and wave steepness

$$\epsilon = 0.25\, H_{mo} / L_{pds}$$

are computed to be 0.003 and 0.008 respectively. Using Figure 33, H_s / H_{mo} at d_d is found to be < 1.0 and is thus set equal to 1 following the recommended procedure of FEMA (1991). The wave height H_s at d_s is thus equal to 1.14 m, which is less than the value predicted by the Goda (1985) method. As a conservative approach and to be consistent with uprush calculations which use H_{sds} = 1.41 m, H_{mods} is set equal to 1.41 m.

ii) Uprush Calculations

Wave uprush was calculated using Stoa (1978b, 1979), ACES (USACE, 1990), Ahrens and Heimbaugh (1988a), and van der Meer and Stam (1992).

. Stoa (1978b, 1979)

Based on Stoa (1979) (see Appendix B, Figure 2), Stoa (1978b) with mean wave conditions and *r* was used.

Using $\dfrac{d_s}{H'_{om}}$ = 1.2; $\dfrac{H'_{om}}{(gT_m^2)}$ = 0.0031 and Appendix A, (Figure 2, flat approach slope), for $\dfrac{d_s}{H'_o} \leq 3.0$ with cot θ = 2, $\left(\dfrac{R_m}{H'_{om}}\right)_{ss}$ = 2.3. For rough slopes, $R_m = k\,r\,H'_{om}\,(R_m / H'_{om})_{ss}$, where the scale correction factor and surface reduction factor (see Appendix A and Table 1) are k = 1.00 and r = 0.80 respectively. Therefore, the uprush is R_m = 2.9 m. From R_m, $R_s = 1.6R_m$ = 4.6 m and $R_2 = 2.23R_m$ = 6.5 m.

. **ACES (USACE 1990) for smooth slopes with rough slope correction factor**

For $\xi \geq 3.5$; $\left(\dfrac{R_s}{H_s}\right)_{ss} = 1.087\sqrt{\dfrac{\pi}{2\theta}} + 0.775\,\dfrac{H_s / L_p}{\tanh^3\left(2\pi\, d_s / L_p\right)}$.

In this case $\dfrac{d_s}{L_{op}}$ = 0.0019, from Shore Protection Manual (Table C-1, see USACE, 1984), $\dfrac{d_s}{L_p}$ = 0.0561.

Thus, with H_{sds} = 1.41 m and θ=26.56°=0.4636 rad, $\left(\dfrac{R_s}{H_s}\right)_{ss}$ = 2.83. For rough slopes, $R_s = r\,H_s\left(\dfrac{R_s}{H_s}\right)_{ss}$

where the surface reduction factor is r = 0.80. Therefore, the uprush level using ACES for smooth slopes with a rough slope correction factor is R_s = 3.19 m.

. **Ahrens and Heimbaugh (1988a)**

The equation is given by $\dfrac{R_{\max}}{H_{mo}} = \dfrac{a\,S}{1 + b\,S}$ where $S = \xi_l = \dfrac{\tan\theta}{\sqrt{H_{mo} / L_p}}$ and a = 1.154 and b = 0.202 (see Table 8). With H_{mo} = 1.41 m and L_p = 33.8 m at d_s = 1.9 m (from Appendix D), $\dfrac{R_{\max}}{H_{mo}}$ = 1.89 and the uprush level is R_{max} = 2.7 m.

. **Van der Meer and Stam (1992)**

The equation is given by $\dfrac{R_s}{H_s} = 0.88\,\xi_m^{0.41}$ for $\xi_m = \dfrac{\tan\theta}{\sqrt{2\pi H_s / g T_m^2}} \geq 1.5$ for impermeable rock revetment.

Using H'_{os} = 2.52 m and T_m = 7.2 s,
ξ_m = 2.83 and R_s = 3.4 m. From the other equations, R_m = 2.2 m and R_2 = 4.8 m. However, as noted in Section 2.1, this method is valid for relatively deep water at the toe of the structure and will over estimate wave uprush in shallow water.

An alternative approach may be to use H_{sds} = 1.41 m and T_m = 7.2 s,
ξ_m = 3.79 and R_s = 2.1 m. From the other equations, R_m = 1.3 m and R_2 = 3.0 m.

Table 19 summarizes the uprush results for the armour stone revetment structure. It is evident that there is a variation in the uprush results particularly for the case of small relative depth. In shallow water (i.e., $d_s/H'_{os} < 3$), methods based on deep water conditions (Stoa 1978; van der Meer and Stam 1992) may tend to produce conservative results.

Table 19: Uprush Results for Armour Stone Revetment Example

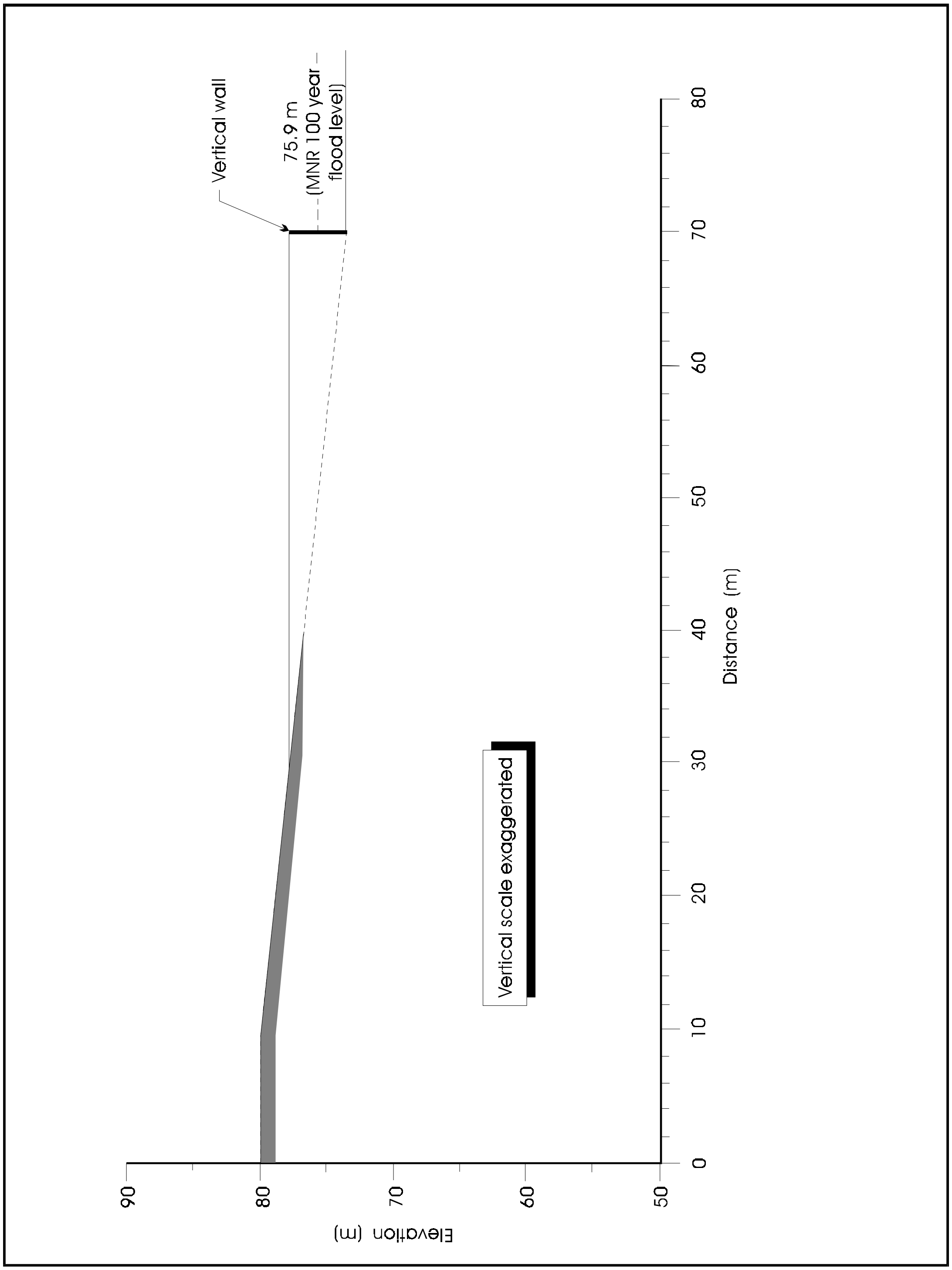
Case	Uprush, R (m)			
Impermeable 1:2 rock slope	**Stoa (1978b, 1979)**	**ACES (USACE 1990) Smooth Slopes**	**Ahrens and Heimbaugh (1988a)**	**Van der Meer and Stam (1992)**
	Using H'_{om}, T_m, and r = 0.8	**Using *Hsds*, T_p and r = 0.8**	**Using *Hmods* and T_p**	**Using *Hsds* and T_m**
H'_{os} = 2.52 m T_p = 8.0 s H'_{om} = 1.58 m T_m = 7.2 s H_{sds} = 1.41 m H_{mods} = 1.41 m	R_m = 2.9 R_s = 4.6 R_2 = 6.5	R_s = 3.3	R_{max} = 2.7	R_m = 1.3 R_s = 2.1 R_2 = 3.0
H'_{os} = 0.90 m T_p = 5.0 s H'_{om} = 0.56 m T_m = 4.5 s H_{sds} = 0.97 m H_{mods} = 0.97 m	R_m = 1.1 R_s = 1.7 R_2 = 2.4	R_s = 1.9	R_{max} = 1.8	R_m = 0.83 R_s = 1.3 R_2 = 1.8

c) Example 3 - Vertical Wall

i) Site Conditions

The site is the same as in Example 2. The nearshore survey is provided in Figure 54(b) and Table 18. A sheet pile wall has been proposed for this site. A vertical seawall will be located at 70.70 m from the baseline as shown in Figure 55, where the bottom elevation is 74.00 m.

Figure 55: Example Vertical Wall Profile



The same wave conditions as in Example 1 were used, i.e. H_{os} = 4.2 m; and T_p = 8.0 s; $H^{/}_{os}$ = 2.52 m. The 100 year flood level was found to be 75.90 m. This elevation was considered to be the stillwater level (SWL). The structure slope M_s is undefined (vertical wall). The approach slope, as defined in Example 2, is M_a = 57. The depth at the toe of the structure is d_s = SWL - 74.0 = 1.90 m. As computed in Example 2, the significant wave height at the toe of the structure, H_{sds} was estimated to be 1.41 m.

ii) Uprush Calculations

Wave uprush was calculated using ACES (USACE 1990), and the upper-bound method (Section 4.7).

. ACES (USACE 1990) for smooth slopes

For $\xi \geq 3.5$; $\left(\dfrac{R}{H_s}\right)_{ss} = 1.087\sqrt{\dfrac{\pi}{2\theta}} + 0.775\,\dfrac{H_s / L}{\tanh^3\left(2\pi\, d_s / L\right)}$.

In this case $\dfrac{d_s}{L_o}$ = 0.0019, from Shore Protection Manual (Table C-1, see USACE, 1984), $\dfrac{d_s}{L}$ = 0.0561.

Thus, with H_{sds} = 1.41 m and $\theta = 90^\circ = 1.571$, $\left(\dfrac{R_s}{H_s}\right)_{ss}$ = 1.92. Therefore, the uprush level using ACES is R_s = 2.71 m.

. Upper-bound method (Section 4.7)

Since ξ is undefined, $\left(\dfrac{R}{H_s}\right)_{ss} = \sqrt{2\pi}\left(\dfrac{\pi}{2\,\theta}\right)^{\frac{1}{4}}$ or $\left(\dfrac{R}{H_s}\right)_{ss}$ = 2.51. Therefore, the uprush level using the upper-bound method is R_s = 3.52 m.

Table 20 summarizes the uprush results for the vertical wall protection. Note that the upper limit of wave uprush could then be estimated by multiplying the calculated R_s by a factor of 1.4.

Table 20: Uprush Results for Vertical Wall Example

Case	Uprush, R_s (m)	
	ACES (USACE 1990)	Upper-bound Method (this report)
H_{sds} = 1.41 m T_p = 8.0 s	2.71	3.52
H_{sds} = 1.28 m T_p = 6.0 s	1.97	3.20
H_{sds} = 1.22 m T_p = 7.0 s	2.02	3.05
H_{sds} = 0.97 m T_p = 5.0 s	1.30	2.43

Note: The uprush values shown have not been increased by a factor of 1.4, and thus are not the upper limits of wave uprush.

5.2 Wave Overtopping

a) Example 5 - Sloped Armour Stone Revetment

i) Site Conditions

The example site and conditions are the same as Example 2 with the exception of the crest elevation. For the purpose of this example, the freeboard of the revetment crest will be varied according to the values of R_m, R_s and R_2 calculated in Example 2 (i.e., 1.3 m , 2.1 m and 3.0 m respectively) using the procedure of van der Meer and Stam (1992) with H_{sds} and T_m.

ii) Overtopping Calculations

Wave overtopping is calculated using the methods of Ahrens and Heimbaugh (1988b), Goda (1985), and Owen (1982).

. Ahrens and Heimbaugh (1988b)

The equation for revetment is Q = 3.006 exp(-13.091 F') where $F' = F/(H^2_{mods}L_p)^{1/3}$. With H_{mods} = 1.41 m; L_p = 33.8 m and F = 2.1 m, the dimensionless freeboard F' = 0.517. The overtopping rate, using the Ahrens and Heimbaugh (1988b) method for revetments and applying an oblique wave factor of 1.15 and a wind enhancement factor of 1.0, is Q = 0.0040 m^3/s•m (or 4 l/s•m).

. Goda (1985)

Goda's design diagrams for the estimation of wave overtopping rates of vertical seawalls and composite revetment/ seawall are shown in Figures 24 and 25. The three values of the dimensionless parameters required to determine overtopping rates are $\frac{H'_{os}}{L_{op}}$ = 0.025, $\frac{F}{H'_{os}}$ = 0.833 ($F = h_c$ in Goda's charts), and $\frac{d_s}{H'_{os}}$ = 0.754 ($d_s = h$ in Goda's charts). Entering Goda's overtopping chart for a 1:30 foreshore slope (Figure 25b) and interpolating between $\frac{H'_{os}}{L_{op}}$ = 0.017 and $\frac{H'_{os}}{L_{op}}$ = 0.036 and applying an oblique wave factor of 1.15 and a wind enhancement factor of 1.0, yields a Q of approximately 0.0024 m^3/s•m.

. Owen (1982)

Owen method has a general exponential relationship in the form of $Q^* = A\,exp(-B\frac{F^*}{r})$ where the dimensionless freeboard F^* and overtopping Q^* are expressed as $F^* = \frac{F}{T_z\sqrt{gH_s}}$ and $Q^* = \frac{Q}{T_z g H_s}$. Coefficients A and B are dimensionless coefficients which are dependent on the test configuration (structure geometry and wave climate) and r is the roughness coefficient.

For the present site condition, the coefficients A and B were determined to be 0.0125 and 22 respectively (see Table 12). The value of r was assumed to be 0.8 (see Table 4). Owen's dimensionless freeboard parameter F^* = 0.0784. Therefore, the dimensionless overtopping rate is Q^* = 0.00145. Solving for the dimensional overtopping and applying an oblique wave factor of 1.15 and a wind enhancement factor of 1.0, yields Q = 0.17 m^3/s•m.

Table 21 summarizes the various overtopping results for the sloping revetment. For the example presented, the methods of Ahrens and Heimbaugh (1988b) and Goda (1985) yield the same order of magnitude results. The rates estimated by Owen (1982) are 1 to 2 orders of magnitude greater. CIRIA/CUR (1991) expects Owen to produce conservative results.

Table 21: Overtopping Results for Armour Stone Revetment Example

Case	**Overtopping, Q (m^3/s•m)**		
Impermeable 1:2 rock slope	**Ahrens and Heimbaugh (1988b)** H_{mods} **= 1.41 m** T_p **= 8.0 s**	**Goda (1985)** H_o' **= 2.52 m** T_p **= 8.0 s**	**Owen (1982)** H_{sds} **= 1.41 m** T_z **= 7.2 s** r **= 0.8**
F = 1.3 m	0.053	0.018	0.38
F = 2.1 m	0.0040	0.0024	0.17
F = 3.0 m	0.00022	0.00012	0.066

Note: The overtopping values shown above include an oblique wave factor of 1.15 and a wind enhancement factor of 1.0.

6.0 CONCLUSIONS

It is evident that the state-of-the-art with respect to the prediction of wave uprush and overtopping of shoreline structures is insufficient to provide a single, accurate and robust empirical method. This is especially true for irregular waves in the surf zone due to the complicated hydrodynamic processes and the wide variability of structure types. There are, however, a number of methodologies which can be considered as "accepted practice" for many Great Lakes - St. Lawrence River System shoreline situations. The procedures outlined for the various "accepted" methods must be followed closely and they should not be extrapolated much beyond the tested conditions. The example calculations of wave uprush and overtopping have shown that there can be a marked variability in predictions depending on the methodologies used. Much work remains to be done to adequately address the uncertainty that arises with the use of the various methodologies. Even under controlled laboratory experiments, there is a considerable degree of scatter in the results of measured wave uprush and overtopping. In addition, measurements to date have been shown to be extremely variable in field situations.

To use some of the available wave uprush and overtopping methodologies requires the local (i.e., transformed) wave height, H, at the toe of the structure or slope. This introduces the need to perform nearshore wave transformations or to use a depth-limited wave approach.

As reported by Walton et al. (1989a), there is limited data on comparisons between uprush on smooth slopes and uprush on rough slopes, both permeable and impermeable, under similar wave and bathymetric conditions. Hence, information on uprush reduction factors is inadequate to provide more than just an approximation of the reduction offered by a rough surface. This inadequacy may be compensated by the use of roughness uprush reduction factors "on the high end of the laboratory measurements because of the present lack of confidence in extrapolating such information to field conditions".

Future work should focus on completing more investigations of wave uprush and overtopping on rough slopes with shoaling (i.e., sloping nearshore) irregular waves in limited depths of water as well as wave overtopping under wind assisted conditions (see, for example, van der Meer and Stam 1992; Kobayashi and Raichle 1994). These conditions are of the most interest along the shorelines of the Great Lakes - St. Lawrence River System. Extra efforts should be made to verify hydraulic model work with prototype investigations of wave uprush and overtopping. In addition, more research is needed in the subject of uprush on gravel/shingle beaches and very permeable structures such as berm (graded stone) revetments.

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