

The Effect of Wave Grouping on Shoaling and Breaking Processes

by

Thomas D. Shand

A thesis submitted for the
Degree of Doctor of Philosophy



**School of Civil and Environmental Engineering
The University of New South Wales
Australia**

August, 2009

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Abstract

Determining the largest breaking wave height which can occur in water of finite depth is a fundamental reference quantity for the design of coastal structures. Current design guidelines are based on investigations which predominantly used monochromatic waves, thereby neglecting group effects which are inherent to the free propagation of waves in deep water. The Coastal Engineering Manual (CEM) states that wave grouping and its consequences is of significant concern, with breakwater armour damage being generally attributed to higher waves associated with wave groups. However, the CEM also acknowledges that there is little guidance and few formulae for use in practical engineering.

This thesis describes a laboratory-based investigation into the effect of wave groupiness on wave shoaling, breaking and surf zone processes. New optical-based techniques for data abstraction, developed within this study, have allowed examination of the interaction between deep water intra-wave group processes and shallow water shoaling processes. The applicability of existing methods for predicting breaking wave height and position is evaluated, along with the implications of groupiness on engineering design in the nearshore. The effect of wave groupiness on overtopping and hazard on emerged rock platforms is similarly assessed.

Wave group testing has revealed that the spatial phasing of intra-group processes during shoaling can result in considerably different shoaling and breaking regimes. Under certain regimes, wave breaking occurred further shoreward and in a more plunging manner than under other regimes. Within the mid to inner surf zone, waves were also observed to propagate into shallower water before breaking than is predicted by existing design guidelines. This could result in under-prediction of wave height by up to 100%. Expressions are developed for the prediction of maximum wave heights and surface elevation on plane slopes. These expressions implicitly include non-linear group effects and group-induced water-level variations within the surf zone, and are found to provide conservative upper envelopes for the range of data observed within the current testing regimes.

Predictive schemes are similarly developed for overtopping hazard on emerged rock platforms based on critical wave and water-level conditions. Variations in maximum overtopping flow values due to intra-wave group processes of up to $\pm 35\%$ were found. These group effects were found to reduce by up to 30% the threshold wave conditions before the initiation of hazard.

Acknowledgements

Firstly, I would like to express my sincere gratitude to Dr. Bill Peirson for his technical expertise and advice and for his continued support, encouragement and patience throughout this project. Thanks are also extended to Assoc Prof. Ron Cox for his practical and useful comments and to academic and technical members of the Water Research Laboratory for their continued advice and assistance, particularly Mr John Hart for his considerable workshop assistance and good humour.

I would like to thank Assoc. Prof. Donald Bailey from the Institute of Information Sciences and Technology, Massey University, New Zealand for his advice and assistance in developing numerical routines for data extraction. Also to Dr Mike Shepherd of Massey University for his grammatical and linguistic reviews and comment.

Comments and assistance over the duration of the project from Drs Karin Bryan and Willem de Lange of The University of Waikato, Dr Giovanni Coco of the National Institute of Water and Atmosphere, New Zealand and Dr Derek Goring of Mulgor Consulting Ltd are greatly appreciated. The NSW Department of Primary Industries and UNSW Water Research Laboratory, are both thanked for their financial support of portions of this study. I would like to thank Tonkin and Taylor Ltd for their support and flexibility in allowing me to peruse this project whilst in their employ and in particular to Mr Richard Reinen-Hamill for his continual encouragement, support and mentoring.

Finally, my sincere thanks to friends and family for their encouragement and patience. For the hospitality shown during my frequency trans-Tasman dashes, I am grateful. In particular, to my parents, Roger and Susan; I cannot thank you enough for instilling in me a desire to learn, explore and understand. And also for the support and encouragement you have unconditionally given me. I dedicate this thesis to you both.

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List of Symbols

Symbol	Parameter
a	wave amplitude
a_0	initial wave amplitude
BP	wave break point
c	wave celerity
c_g	wave group velocity
d	local water depth
d_b	depth at break point
d_{mwl}	depth with respect to mean water level
d_{swl}	depth with respect to still water level
E	energy (N-m)
E_f	energy flux
E_k	kinetic energy
E_p	potential energy
g	acceleration due to gravity (assumed to be 9.812 ms^{-2})
h	local water depth
H	wave height
H_0	energy-based significant wave height [$=4\sqrt{\eta^2}$]
H_b	breaking wave height
H_{br-s}	spatial measure (temporally discrete) of breaking wave height
H_{max}	maximum wave height
H_{rms}	root mean squared wave height
H_{sig}	significant wave height (statistical)
k	wave number [$= 2\pi/L$]
k_0	initial wave number [$= 2\pi/L_0$]
m	beach slope [$= \tan^{-1} \theta$]
L	wavelength
L_0	deep water wavelength
L_g	wave group length
mwl	mean water level

N	Number of waves generated at the paddle within an individual wave group (spatial measure)
R_c	relative freeboard of emerged model platform above swl
s	wave steepness [$= ak = H/L$]
S_0	initial wave steepness [$= (ak)_0 = (H/L)_0$]
S_{br-s}	spatial measure of breaking wave steepness
S_1	slope offshore of emerged model platform
S_2	slope of front face of emerged model platform
swl	still water level
swx	intersection of the sloping bed and still water level
t	time
T	wave period
T_g	wave group period
w	emerged model platform width
x	horizontal distance
x'	horizontal distance from wave paddle in the direction of wave propagation
x'_{swx}	horizontal distance from the paddle to the intersection of the sloping bed and still water level
x_{swx}	horizontal distance from the intersection of the sloping bed and still water level (swx) opposite the direction of wave propagation
X_p	horizontal distance traveled by the plunging lip between break point and impact
Y_s	vertical distance between plunging jet impact and maximum splashup elevation
z	local surface elevation in relation to the still water level
z_{max}	maximum local surface elevation in relation to the still water level
z_{min}	minimum local surface elevation in relation to the still water level
z_{mean}	local mean surface elevation in relation to the still water level
ρ_w	density of water
λ_{br}	visual breaker type (1-10)
ω	wave radian frequency [$= 2\pi/T$]

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*The outcome of any serious research can only be to
make two questions grow where only one grew before*

Thorstein Veblen (1857 – 1929)

Chapter 1 Introduction

1.1 Background and rationale

There are few natural processes as complex as the life cycle of a water wave; from its generation under an applied wind field, its propagation across a water body, interaction with other wave-forms and finally its breaking and dissipation.

Determination of the largest wave height, H , which can occur in water of depth, d , without breaking is a fundamental requirement for the design of coastal structures. For example, the unit mass of rock armour specified by design codes based on the Hudson formula (USACE, 2003) exhibits a wave height cubed dependency. The ratio of breaking height (H_b) to depth (H_b/d) is known as the breaker index and has been the subject of much research over the past 150 years. McCowan (1894) theoretically derived a maximum wave height on depth ratio prior to breaking of 0.78 above flat beds; however, this value was found to be inadequate on steeper slopes by Iversen (1952). Subsequent studies by Galvin (1969), Goda (1970), Weggel (1972) and others further refined breaker index values for waves and beds of varying steepness and slope, with Goda (1970) and Weggel (1972) producing design curves based on their own and earlier published investigations. While additional work by Smith and Kraus (1990), Battjes (1974), Nelson (1985) and others have been undertaken, these studies form the basis of modern design guidelines (Goda, 2000 and USACE, 2003) which define the breaker index according to wave steepness and bed slope.

Original raw measurement data, upon which design guidelines are based, exhibit significant scatter (i.e. Figure 1-1), with data points evident above the recommended design curves. This is of interest as, in the absence of turbulent perturbations, monochromatic waves should always break consistently in the same manner and in the same location. Goda (2007) identifies an inherent variability in wave breaking and in the derived breaker index of 6 to 14%, depending on bed slope, with steeper slopes exhibiting greater scatter.

Additionally, there are a number of intrinsic problems associated with the application of these guidelines. These include the definitions of both the wave break point and the depth at break point and, perhaps most crucially, that these investigations have tended to focus on

idealised wave-trains consisting of regular, monochromatic waves and therefore neglected possible group or irregular wave effects.

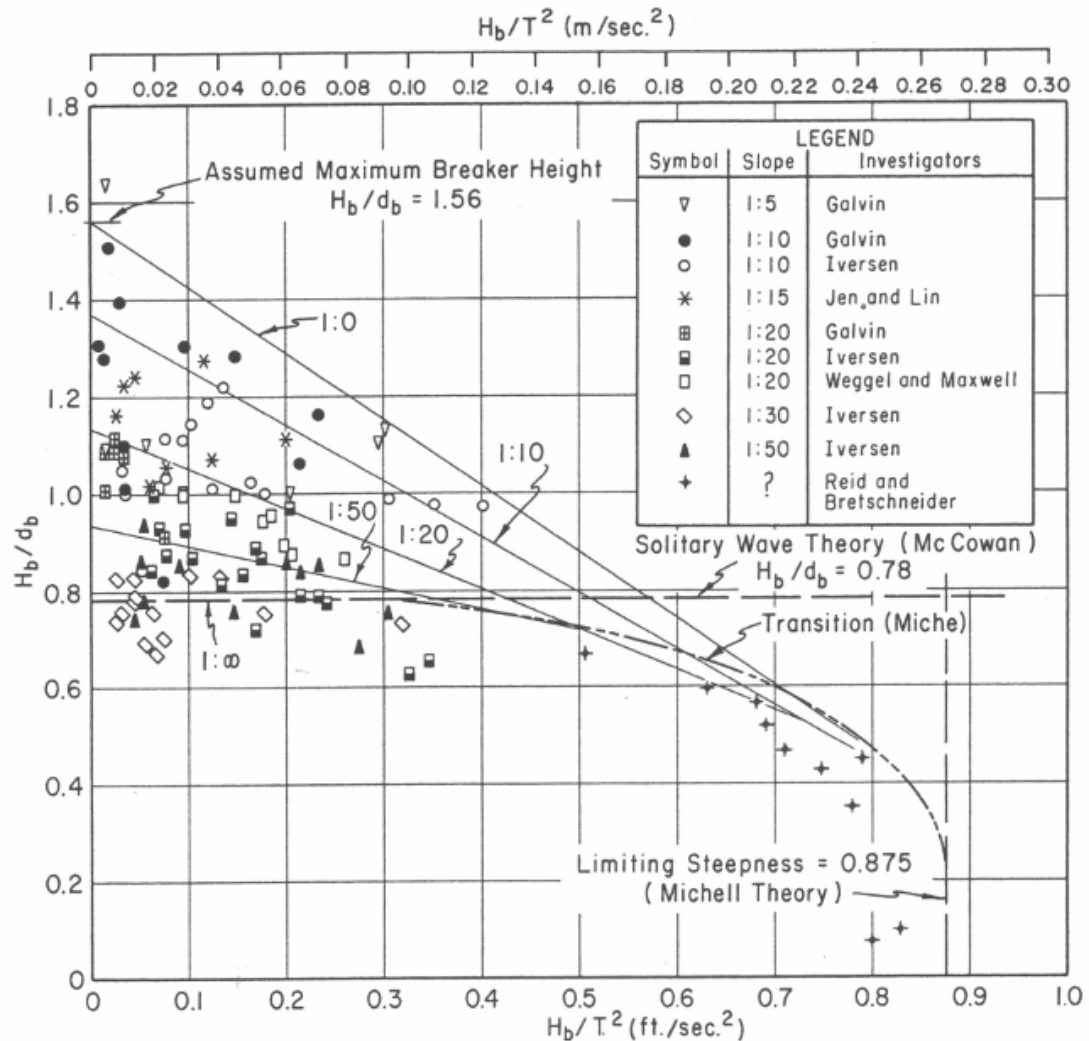


Figure 1-1 Measured breaker depth index according to wave steepness with curves of best-fit based on bed slope indicated (source: Weggel, 1972). These curves form the bases of modern design guidelines such as within the Coastal Engineering Manual (USACE, 2003).

Monochromatic water waves are inherently unstable due to sideband frequency instabilities and wave-trains must, over time and space, form groups (Benjamin and Feir, 1967; Hwung and Chiang, 2005). The Coastal Engineering Manual (USACE, 2003) suggests that higher waves associated with wave groups are generally thought responsible for armour layer damage (CEM, 2003; VI-5-66), and states that that wave grouping and its consequences is of significant concern. However, there is little guidance and few practical formulae for use in practical engineering (CEM, 2003; II-1-97). The manual recommends that practitioners be aware of the existence of groupiness and evaluate its

importance in situations known to be sensitive to group-related phenomena. However, procedures for this evaluation are stated to be beyond the scope of the Manual.

Recent studies have highlighted the role of wave groups in deep water wave breaking due to convergences in wave energy density (Rapp and Melville, 1990; Song and Banner, 2002; Banner and Peirson, 2007). These convergences are inherent to the groups themselves rather than due to the effects of depth or subsurface currents. Group-related fluctuations in energy density have also been found to influence breaking properties in shallow water (Dack et al., 2005; Peirson et al., 2007; Shand et al., 2007). Using idealised group spectra and judicious selection of the spatial phasing of group evolution relative to shoaling, breaker indices up to 10%, 27% and 35% higher than design guidelines were observed on horizontal, 10:1 and 5:1 sloped beds respectively.

These preliminary results indicated that current design guidelines based on monochromatic waves under-estimate the ‘worst case’ scenarios associated with wave groupiness and justified a further, more comprehensive, investigation. Additionally, limitations in the accuracy and extent of measurement possible using conventional instrumentation, particularly within the surf zone, became apparent during initial studies. This more comprehensive study has therefore included development of an optical-based laser-induced fluorescence (LIF) technique to better visualise and capture flow during all stages of wave evolution.

1.1.1 Rock fishing

Wave group behaviour has potentially significant implications for human safety at the coast. On average, eight people per year lose their lives during rock fishing incidents on the New South Wales (NSW) Coast. The NSW coroner has identified rock fishing as having one of the highest fatality rates of any sport in the state. There is a pressing need to understand the critical factors that cause wave groups to suddenly initiate significant overtopping of rock shelves and to characterise the risks to people exposed to such conditions. If such risks can be related to prevailing wave conditions offshore, appropriate warnings can be incorporated within daily weather forecasts.

A recent report (Jones, 2003), quantifying the risks associated with rock fishing, found that the dominant factor causing people to be swept into the water was unanticipated

wave impact (94% of fatalities). While a high proportion of fatalities occur during larger than average wave conditions, fatalities have also occurred during smaller wave conditions, indicating that even during relatively benign conditions, some hazard may exist. An example of significant overtopping occurring during relatively low energy conditions (significant wave height $\approx 1.5\text{m}$) is presented within Figure 1-2. Other studies have shown that proportionally higher fatalities also occur during long wave period conditions (Evans, 2007) and during incoming tides. A study of incidents of fishermen being washed off rocky shores and breakwaters in Taiwan (Tsai et al., 2004) identified wave groupiness as a significant factor contributing to dangerous wave overtopping.

A laboratory-based study investigating the effect of hazardous waves, particularly those associated with wave groups, on run-up and overtopping of rock platforms, and implications for the safety of rock fishermen, has been completed and forms part of this thesis.



Figure 1-2 A fisherman at Flat Rock, Harbord on the NSW coast braces as a wave overtops a rock platform.

1.2 Study objectives and approach

This present study aims to expand the work of Shand et al. (2007), Banner and Peirson (2007) and Dack and Peirson (2005) on the interaction between nonlinear, intra-wave group processes and shoaling transformations prior to break point. It is anticipated that the interaction of these respective horizontal and vertical energy convergences plays a key role in defining the breaking position and therefore the breaker height index. Additionally, the study aims to improve our technical understanding of the wave conditions that are likely to lead to hazardous conditions on rock shelves along the NSW coast and to quantify the effect of groupiness in a wave-train on this hazard.

The study question is therefore *to what extent do deep water intra-wave group processes influence shallow-water wave-breaking and overtopping processes*, with the hypothesis that *the spatial phasing of group evolution at some critical depth will influence future shoaling patterns and breaking position and type, with these interactions being most pronounced on steeper coastal bathymetries*. Two types of representative coastal bathymetries have been investigated:

1. plane slopes;
2. coastal rock platforms.

The following approach has been employed to test the study hypothesis and achieve the study aims:

1. review literature relevant to the study question and related subject area, including wave group processes, depth-induced wave breaking and implications for engineering design;
2. development of an improved laboratory method for observing and analysing water surface profiles, applicable across all water depths and wave states;
3. analysis of intra-wave group processes in deep water, with deep water results guiding the selection of subsequent shallow-water testing regimes;
4. testing of monochromatic, ‘control’ cases on plane-sloped bathymetry. Shoaling and breaking processes are observed and resulting breaker indices compared to existing guidelines;
5. testing of wave groups by adjusting the spatial phasing of group evolution with the sloping bed position;

6. quantifying the effect of a sloping bed on the intra-group processes by comparing shallow water results to the deep water control cases;
7. analysis of the shoaling and breaking regimes for wave groups as a function of group spatial phasing;
8. evaluation of breaker indices and related design parameters, comparison to existing design guidelines and suggestion of modifications to guideline techniques as required to fit data;
9. assessment of the threshold wave and water level conditions that are likely to lead to hazardous conditions on rock shelves along the NSW coast and quantification of the effect of groupiness in a wave-train on this hazard;
10. derive conclusions as to the effect of wave groupiness on shoaling, breaking and overtopping processes, implications for engineering design and the applicability of existing methods for determining wave height and design conditions in the presence of wave groups.

1.3 Thesis outline

This introductory chapter provides a brief background to the subject area, the rationale for the study, and the approach used to achieve the study aims. The remainder of the thesis is presented within seven chapters as follows:

Chapter two provides a review of literature detailing past investigations in the subject area and current understanding of the study problem. Experimental facilities and data collection techniques are detailed within Chapter three and a description of data processing techniques within Chapter four. Chapter four includes discussion of errors and limitations associated with the testing facilities and analysis techniques.

The results and discussion are divided into two parts. Part one presents the results and discussion of experiments undertaken on a plane sloped bed and Part two presents results and discussion on wave overtopping of emerged platforms. Within Part one, Chapter five presents results and discussion associated with monochromatic experiments, Chapter six presents the fluid dynamic aspects of the group influence on shoaling and breaking water waves and Chapter seven provides results and discussion on the implications of wave groupiness on structural design in the nearshore.

Part two comprises Chapter eight and includes a brief discussion of the background and fieldwork pursuant to the laboratory study which examines the influence of groupiness on overtopping hazard at emerged platforms. Using established hazardous flow guidelines, a method for predicting the onset of hazardous conditions as a function of wave and water level characteristics is presented, with the influence of nonlinear evolving wave-trains discussed.

Chapter nine presents a summary of conclusions for both the plane bed slope and emerged platform investigations and provides recommendations for future studies.

Chapter 2 Background and literature review

2.1 *Introduction*

The maximum wave height that can be achieved in water of finite depth is of key importance in the design of coastal structures and the assessment of nearshore currents and water levels. There have been many experimental, field and numerical studies investigating shoaling and breaking waves and the effect of wave and seabed properties on these processes. However, past investigations have often followed dissimilar experimental procedures, making different assumptions and evaluating a variety of characteristics (Black and Rosenberg, 1992). These differences have resulted in inconsistent and inconclusive results. To assess the maximum breaking wave height possible in water of limited depth, two key parameters are required: breaking wave height and water depth at break point. The exact definitions of break point, wave height and water depth remain ill-defined and have been subject to differing interpretation by past investigators (i.e. Goda, 1970; Seyama and Kimura, 1988; Black and Rosenberg, 1992).

Current design guidelines are based on investigations which predominantly used monochromatic waves, thereby neglecting group effects. Groupiness or height modulation in wave-trains is an inherent characteristic of freely propagating waves in deep water. Whilst our understanding of the formation and propagation of *wave groups* remains limited, significant progress has recently been made in the prediction of the onset and strength of breaking of wave groups in deep water (Banner and Peirson, 2007). Recent two-dimensional laboratory studies have also shown that wave group effects have a marked effect on wave shoaling and breaking, influencing both the position and form of the breaking waves. The phenomenon of the wave group influence on breaking waves in shallow water is the primary focus of the present investigation.

This chapter reviews the formation and propagation of wave groups, their associated long-waves and changes which occur as water depths shallow. Additionally, the wave-breaking process is reviewed with particular reference to the terms and definitions used to characterise breaking and to the established engineering guidelines used to identify the maximum wave height possible in depth-limited water.

2.2 Wave groups in deep water

2.2.1 Wave generation and development

Although ocean waves may originate from a variety of sources, by far the largest contributor of energy is surface wind. Descriptions of wind-wave growth remain far from complete and multiple explanations for the initial generation mechanism exist. Texts such as Kinsman (1965), Komar (1998), Janssen (2004) and Holthuijsen (2007) summarise and discuss these initial wave growth theories. However, once initial waves have formed, the theory of their growth becomes somewhat simpler. By offering an uneven surface, airflow and therefore surface pressure is modified leading to areas of higher and lower pressure. The pressure is found to be highest windward of a wave crest and lowest in the lee of a crest. As these pressures are in phase with the water motion (i.e. high pressure occurs where the water position is falling behind the crest and low pressure where the water is rising before the crest), energy is transferred from the wind to the waves (Miles, 1957). This differs from the supposition that surface friction is the major contributor to wave growth. Recent work by Peirson and Garcia (2008) shows that both processes play a significant role. The restoring force of waves generated in this manner is gravity; hence waves of this type are often described as *gravity waves*.

Wind generated waves form a directional wave spectrum whose peak frequency decreases with fetch. This leads to a *sea-state* comprised of instantaneous wave forms of varying amplitude, wavelength, phase and direction that exist simultaneously.

Higher frequency waves within deep water may transfer energy to lower frequency waves by nonlinear wave-wave interactions (quadruplets). This enables more rapid growth of the lower frequency domain of a wave field than would be possible with wind input alone (Holthuijsen, 2007).

While a wave-field may be regarded as a distribution of wave heights, period and directions, the wave-field may be quantified using discrete values to simplify description and analysis. To facilitate this, a wave-field is often characterised by the average of the largest one third of waves, also known as the *significant wave height* (H_{sig}) and the corresponding *significant wave period* (T_{sig}). Other statistical measures of

wave characteristics including the maximum wave height ($H_{\max}$), the average of the highest 1% or 10% of waves ($H_{1\%}$ or $H_{10\%}$) or the arithmetic mean or root mean squared wave height (H_{mean} or H_{rms}). Longuet-Higgins (1952) developed relationships between certain wave statistics based on theoretical distributions, with good agreement to field data (Komar, 1998).

Early developments in wave prediction (e.g. Sverdrup and Munk 1946, 1947 and Bretschneider, 1952; 1957) produced nomograms to determine the significant wave height and period resulting from wind of a certain velocity blowing for a certain duration or over a certain distance or fetch. These semi-empirical forecasting methods are known as S-M-B methods after the early developers. More recent methods employing spectral parameters to define the wave field have superseded these early methods and are described further in the following sections.

Once a wave-field decouples from its source of generation (i.e. moves away from the generating wind system) the sea-state becomes swell. At this point it becomes useful to describe the wave forms according to simplified theories which define relationships between wave velocity, height and length and changes in these relationships as currents and shallow water are encountered.

Wave theories have their origins in the late 18th and mid 19th centuries with Laplace (1776) and Airy (1845) developing linear wave theory, followed closely by Stokes' investigation of higher order theories in 1847 (Stokes, 1947). Further developments in wave analysis and theory continued through the latter part of the century with Boussinesq (1872) developing solitary wave theory and Korteweg and de Vries (1895), cnoidal theory. These classical models form the basis of most modern wave analysis and have changed little over the past century. Detailed discussions of the development and workings of these theories are given in Dean and Dalrymple (1984), Komar (1998) and Craik (2004).

2.2.1.1 Linear wave theory

Linear Airy wave theory (Laplace, 1776; Airy, 1845) is the simplest of these wave theories and is based on the Navier-Stokes equations of motion. These equations balance the change in momentum of flowing water against those forces which cause the

change. While more complete descriptions are presented within Kinsman (1965), Dean and Dalrymple (1991) and Mei et al. (2005a), the theory assumes a sinusoidal wave-form of small amplitude compared to depth (i.e. $H/d \ll 1$) where H is wave height and d is water depth. Deep water is generally defined as $d > L/2$ where L is wavelength, however linear wave theory may also be applied to shallower water than this and has been shown to retain reasonable accuracy right up to the break point (Kamphuis, 1994). Water between $L/2$ and $L/20$ is said to be of transitional or intermediate depth and water shallower than $L/20$, shallow. The theory assumes that wave energy is conserved (i.e. no frictional loss), that motion is irrotational and imparts no angular momentum to water particles, and that there is no net movement in particle position after a wave has passed. So whilst linear waves do not impart energy or momentum to the particles they propagate through, energy and momentum are transported along the surface by the waves.

The water surface relative to the still water level (η) may be defined in any depth by:

$$\eta(x, t) = a \cos(\omega t - kx) \quad (2-1)$$

where a is wave amplitude or $H/2$, k and ω are the wave number and radian frequency and x and t are distance and time. The relationship between the wave number and radian frequency are given by the dispersion relationship

$$\omega^2 = gk \tanh(kd) \quad (2-2a)$$

$$\text{or} \quad L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right) \quad (2-2b)$$

As wavelength occurs on both sides of the dispersion equation (Eqn. 2-2b), iterative solutions or approximations must be employed. A number of techniques are available including those by Eckart (1952), Hunt (1979), Neilson (1982) and Fenton (1990). These methods range in accuracy for differing water depths, with the method of Hunt (1979) providing small errors across a range of depths and the Neilson (1982) method better suited for shallow water application (de Lange, 1999). Wave period (T), length (L) and celerity (c) are related by:

$$c = \frac{L}{T} = \frac{\omega}{k} \quad (2-3)$$

Wave celerity and wavelength may be similarly described by Eqns. 2-4a and 2-4b.

$$c = \frac{gT}{2\pi} \tanh(kd) \quad (2-4a)$$

$$L = \frac{gT^2}{2\pi} \tanh(kd) \quad (2-4b)$$

In deep water the wave celerity simplifies to $c_0 = gT/2\pi \approx 1.56T$ and wavelength to $L_0 = gT^2/2\pi$ or $\approx 1.56T^2$. In shallow water wave celerity becomes $c_s = (gd)^{1/2}$ and wavelength, $L_s = T(gd)^{1/2}$.

Energy is transmitted in two ways: as potential energy (E_p) as particles are elevated and kinetic energy (E_k) as the particles are moved in an orbital motion within the wave-form. Energy density may be obtained along the full wavelength by incorporating both these forms of energy with the resultant energy density defined as:

$$E = E_p + E_k = \frac{1}{8} \rho g H^2 \quad (2-5)$$

The time-averaged energy flux (E_f), which describes the energy transmitted in the direction of wave propagation per metre of wave crest, may be written:

$$E_f = \frac{1}{16} \rho g H^2 c \left(1 + \frac{2kh}{\sinh 2kh} \right) \quad (2-6)$$

The rate at which a wave transfers momentum can be described by the momentum flux or radiation stress (Longuet-Higgins and Stewart, 1962, 1964). This is described in both the direction of wave advance (x direction - S_{xx}) and along the wave crest (y direction - S_{yy}) by:

$$S_{xx} = E \left[\frac{2kd}{\sinh(2kd)} + \frac{1}{2} \right] \quad (2-7a)$$

$$\text{and} \quad S_{yy} = E \left[\frac{kd}{\sinh(2kd)} \right] \quad (2-7b)$$

where d is the water depth and k is the wave number ($= 2\pi/L$).

2.2.1.2 Nonlinear theories

As wave height increases and the assumption of small amplitude is increasingly violated, the wave-form becomes increasingly nonlinear. Theories such as higher order Stokes wave theory (Stokes, 1847; Craik, 2005) become more appropriate in these cases. Stokes linear theory is developed to include higher order terms which describe waves as a function of a^n , where a is the wave amplitude and $n \geq 2$. Stokes waves travel faster than linear waves in all water depths and have a longer wavelength. The resultant

wave-forms are characterized by narrow, peaked crests and wide, flat troughs. The resulting orbital velocities below Stokes waves produce a flow of higher strength but shorter duration beneath the wave crest and of lower strength but longer duration beneath the trough. This results in a net drift of water particles in the direction of wave propagation known as Stokes drift. Solving the wave equation according to Stokes theory is substantially more complex than for linear theory with the second order solution for wave profile given by (Komar, 1998):

$$\eta = \frac{H}{2} \cos(kx - \omega t) + \frac{\pi}{2} \frac{H^2}{L} \frac{\cosh(kd)[2 + \cosh(2kd)]}{[\sinh(kd)]^3} \cos[2(kx - \omega t)] \quad (2-8)$$

Both linear and Stokes theory become increasingly inaccurate as water depth becomes very shallow. In cases of very shallow water, unbroken wave theories such as solitary theory (Boussinesq, 1872) and cnoidal theory (Korteweg and de Vries, 1895) provide a better approximation. Both these theories describe waves as they become steep prior to break point and as the ratio of deep water wave length to water depth becomes large.

Solitary waves are non-periodic, progressive waves which contain a single crest and have no intrinsic wavelength or wave period. As solitary waves do not have wavelength or period, wave celerity is a function of wave height and water depth and the resultant velocity is greater than that of linear waves.

Cnoidal waves are periodic, progressive waves based on an elliptic cosine function. The description of cnoidal waves changes with depth, reducing to linear waves in deep water, solitary waves in shallow and Stokes waves in water of intermediate depth. This ability to change form with depth provides the cnoidal wave with the greatest potential for use in all depth ranges prior to break point. A more precise summary of nonlinear aspects of wave theory is presented within Mei et al. (2005b).

2.2.1.3 Wave spectra

Real ocean waves are comprised of fields of superimposed waves, varying in direction, amplitude, phase and wavelength. Due to the transient nature of real wave fields, observations of the surface at differing times or positions yield different results. To

characterise all possible representations of the wave field, a linear wave spectrum is employed (Holthuijsen, 2007).

This spectrum assumes a wave field may be accurately represented by the superposition or sum of a large number of harmonic waves at varying discrete frequencies. The assumption for an ocean wave model is that the phase and amplitude of these harmonic waves is random and thus each may be defined according to a probability distribution. Combining these distributions gives amplitude and variance density spectra. These describe the energy present at various frequencies. While the shape of the surface elevation may change over time, the energy spectrum remains constant, and thus allows statistical characterisation of the surface elevation.

As waves of longer wavelength (lower frequency) will, according to linear theory, travel faster than shorter waves, a sorting process known as frequency dispersion occurs during propagation (Barber and Ursell, 1948). This dispersion causes spatial and temporal variance in the wave spectrum. The further from the source of generation a wave field travels, the more apparent this sorting process becomes. Therefore, spectra which may be broad banded and contain energy in a wide range of frequencies near generation, become narrow banded as they propagate, containing primarily waves of similar wavelength.

The use of spectra to characterise wave fields have significant limitations when nonlinearity and wave-wave interaction is strong, especially within shallow water.

2.2.2 Group formation and propagation

Groupings of waves of similar amplitude or modulation within ocean wave-trains are evident in both visual observation and wave-records (Masson and Chandler, 1993). Groupiness is important because it is anticipated as playing a significant role in the stability of coastal and marine structures (List, 1991; Medina et al., 1994), may influence the formation of extreme, ‘rogue’ waves in both deep water (Stansberg, 1997; Banner, 2005; Gramstad and Trulsen, 2007 and others) and nearer the coast (Tsai et al., 2003) and may generate long waves both in deep water (Longuet-Higgins and Stewart, 1962) and the nearshore (Symonds et al, 1982). Long waves within the nearshore, often termed *surf beat* (Munk, 1949) or *infragravity waves*, have furthermore been suggested

as drivers of nearshore circulation and harbour resonance (McComb et al., 2005) and the development of rhythmic morphological features (Holman and Bowen, 1982).

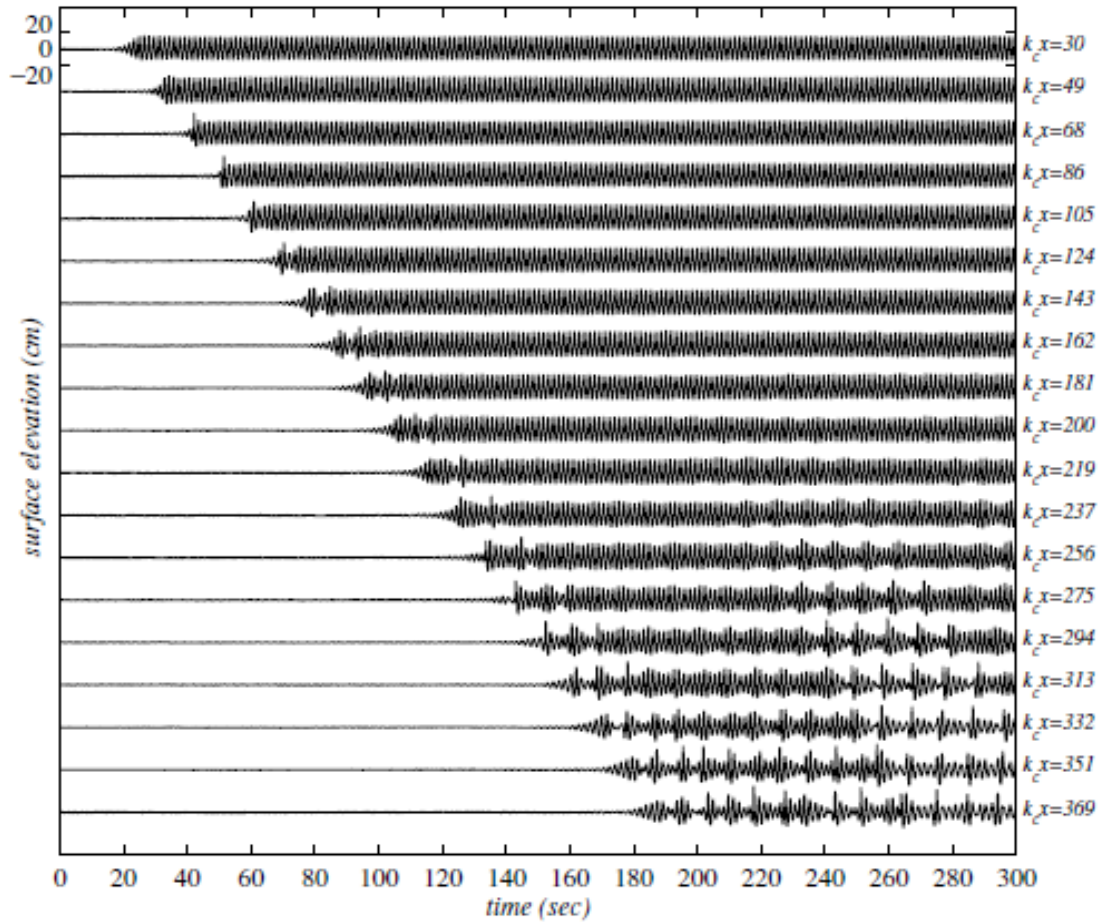


Figure 2-1 Evolution and modulation of an initially regular wave-train in a wave flume with time (x-axis) and space (y-axis). The top-most time series is nearest the wave generation source and the bottom-most series, furthest away (source: Hwung and Chiang, 2005, Figure 5).

The formation of wave groups has been attributed (Mase and Iwagaki, 1986) to either statistical causes, i.e. the superposition of a number of independent waves, or to modulational instabilities associated with group propagation. The difficulty of maintaining a regular wave-train in a flume was first explained by Benjamin and Feir (1967). They suggested that unstable side-band disturbances occur in Stokes waves with frequencies differing slightly from the fundamental carrier wave frequencies. These side-bands may induce nonlinear evolution of even initially regular wave-trains (Melville, 1982; Hwung and Chiang, 2005). Development of amplitude modulation for initially regular, monochromatic waves travelling within a controlled laboratory flume is shown in Figure 2-1 (Hwung and Chiang, 2005). Melville (1982) showed that along

with amplitude modulation, nonlinear instabilities may induce frequency modulation. Furthermore, both amplitude and frequency modulation develop asymmetry, with the modulation exhibiting steeper front faces and more gently sloping backs.

2.2.2.1 Group form and velocity

The surface position of a bichromatic wave group (Figure 2-2) formed by the superposition of two sinusoidal wave trains (Eqn 2-1) can be written as (Kinsman, 1965):

$$\eta(x, t) = a \cos(k_1 x - \omega_1 t) + a \cos(k_2 x - \omega_2 t) \quad (2-9)$$

Eqn 2-9 can be rewritten using trigonometric identities (Dingemans, 1997) as:

$$\eta(x, t) = \left[2a \cos\left(\frac{k_1 - k_2}{2} x - \frac{\omega_1 - \omega_2}{2} t\right) \right] \sin\left(\frac{k_1 + k_2}{2} x - \frac{\omega_1 + \omega_2}{2} t\right) \quad (2-10)$$

The motion of the group envelope is defined by left hand part of the equation and so the group wave number and group angular frequency are given by $k_g = (k_1 - k_2)/2$ and $\omega_g = (\omega_1 - \omega_2)/2$. The velocity of the group envelope as k_1 tends to k_2 thus becomes (Dingemans, 1997):

$$c_{envelope} = \lim_{k_1 \rightarrow k_2} \frac{\omega_1 - \omega_2}{k_1 - k_2} = \frac{d\omega}{dk} \quad (2-11)$$

Thus, the velocity at which a wave group travels differs from the celerity of individual waves within that group (Dean and Dalrymple, 1991; Craik, 2004). This so-called group velocity (c_g) is defined as:

$$c_g = cn \quad (2-12)$$

where c is the linear wave celerity and n is given by:

$$n = \frac{1}{2} \left[1 + \frac{2kd}{\sinh(2kd)} \right] \quad (2-13)$$

Due to this difference in velocity, individual waves do not remain ‘bound’ to a particular group within deep water. Rather, individual waves ‘grow’ from the back of the group, pass through the group centre and diminish as they move towards the front of the group. This dispersive process slows as the wave group enters shallow water and the

individual wave celerity approaches that of the group. For sinusoidal waves, this relationship is generally accepted to reduce to $c_g = c/2$ in deep water and to approach $c_g \approx c$ in shallow water (Phillips, 1977, Dean and Dalrymple, 1991).

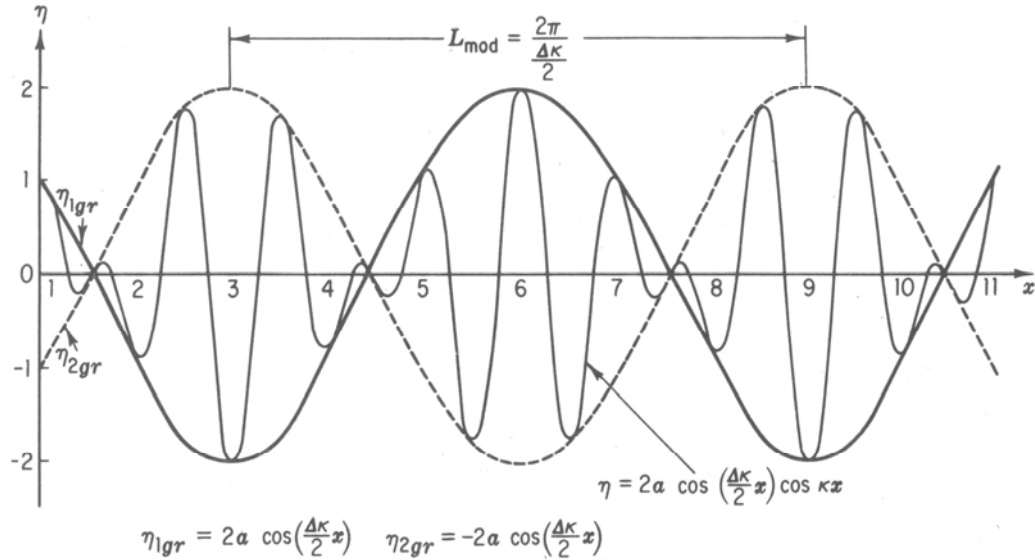


Figure 2-2 Group behaviour of an amplitude-modulated wave formed by the superposition of two sinusoidal wave trains of nearly the same frequency (source: Kinsman, 1965).

The discrepancy between the group velocity and individual wave celerity and the dispersive nature of grouped waves also means that the number of waves observed within a group depends on whether the group is examined from a temporal (fixed spatial point over time) or spatial (fixed moment in time) viewpoint. The number of waves in space can be evaluated by the ratio of group wavelength to individual wavelength and the number of waves in time, by the ratio of group period to individual wave period. Thus, the ratio of waves in space to waves in time can be calculated by the ratio of group velocity to wave celerity. In deep water this is approximately two, and so twice as many waves will be observed at a single point in space through time, than when observed spatially at a single point in time.

Song and Banner (2001), investigating the numerical evolution of nonlinear, evolving deep-water wave groups (further described in Section 2.2.3), found however, that the velocity of the group envelope maximum (c_{se}) was not constant as indicated above, but rather exhibited a rapid fluctuation. This fluctuation was found due to the highly asymmetric carrier wave profiles associated with nonlinear wave groups. Maximum

values (up to $c_{se}/c = 0.8$) were found to occur when the surface envelope maximum corresponded with a wave crest and minimum values (as low as $c_{se}/c = -0.05$), occurred when the surface envelope maximum corresponded with a wave trough.

2.2.2.2 Describing wave groups

While certain spectral properties of a wave-train such as spectral width and shape parameters (Rice, 1945; Longuet-Higgins, 1952, both detailed in Goda and Kudaka, 2007) and Benjamin-Feir index (Gramstad and Trulsen, 2007; Nakamura, 1996) provide some insight into wave group properties from a spectral viewpoint, more comprehensive descriptions of the groupiness characteristics are obtained by direct analysis of time series.

The degree of modulation (groupiness of a wave-train) may be expressed by various parameters. The run length of a wave group (Goda, 1970) provides a measure of the number of consecutive waves exceeding a threshold height, typically the significant wave height. Correlation coefficients (Goda, 1970) may be similarly determined to express the probability of a large wave (which exceeds a certain threshold) following another large wave.

Funke and Mansard, 1979 proposed a groupiness factor (GF) based on a smoothed instantaneous wave-energy history (SIWEH). The SIWEH method uses a lowpass filter, related to the peak period, to smooth the squared surface elevation. The groupiness is calculated by the ratio of variance in the SIWEH spectra to the variance in short wave spectra. However as List (1991) highlights, this method is highly sensitive to the choice of lowpass cut-off frequency and may become contaminated by additional long-wave energy present.

An alternative envelope-based groupiness factor (GF) was proposed by List (1991). This method uses the ratio of the standard deviation of the group envelope to the mean envelope amplitude to define groupiness. Unlike the SIWEH method, this bounds the GF to a range of 0 to 1, with a GF of 1 indicating a fully-modulated group with wave heights of zero at the group nodes and a GF of 0 indicating a non-modulated, regular wave-train. A similar method is employed by Svendsen and Veeramony (2001) who use

the simpler ratio of the variation in wave height (ΔH) to mean wave height (H_{mean}) to define groupiness, G .

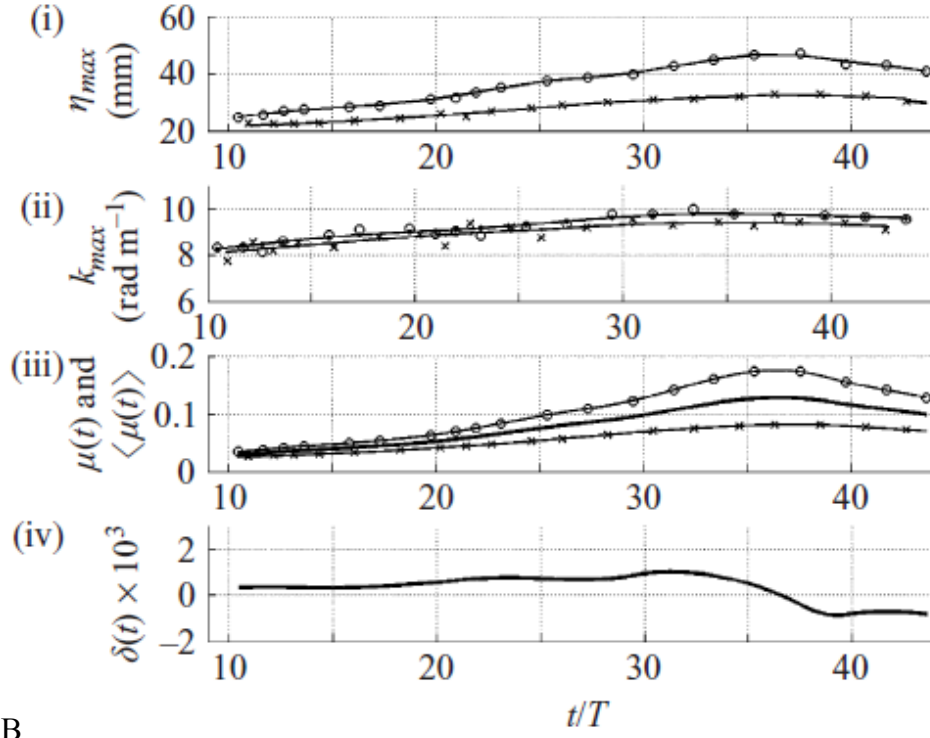
2.2.3 Deep water wave breaking

Whilst wave breaking is most typically associated with beaches and shallow water, waves can also be observed to break in deep water. Banner and Peregrine (1993) provide a review of wave breaking in deep water and discuss types of breaking, measurement techniques and limitations, theoretical descriptions of the occurrence of breaking and implications for wind-wave modelling.

While the use of kinematic or geometric wave properties such crest velocity compared to wave celerity, or wave steepness have traditionally been used to define criterion for wave breaking, as further discussed within Section 2.3.2.3, they have not been found to be universally applicable (Banner and Peirson, 2007). Studies of waves breaking in deep water (Melville, 1982; Rapp and Melville, 1990) have shown that breaking is initiated by nonlinear wave interaction. More recent studies by Stansberg (1997), Tulin and Wadesa (1999), Svendsen and Veeramony (2001), Song and Banner (2002) and Banner and Peirson (2007) have shown that wave group effects play a key role in the initiation of this breaking.

Convergence of wave energy density within an evolving nonlinear wave group leads to the formation of locally-steep waves. Figure 2-3 depicts the evolution of two nonlinear wave groups of differing initial steepness. The first (A) does not break, but reaches a point of recurrence, after which nonlinearity decreases and waves become locally less steep. Waves within the second group, however, break (B). Song and Banner (2002) suggested that the non-dimensional mean growth rate in local energy convergence $\delta(t)$ must exceed a threshold value of $0.14 (\pm 0.1) \times 10^{-3}$ before breaking must occur. This was supported within a more recent experimental investigation (Banner and Peirson, 2007) and appears to provide a robust predictor of the onset of breaking several wavelengths in advance of the occurrence of breaking.

A



B

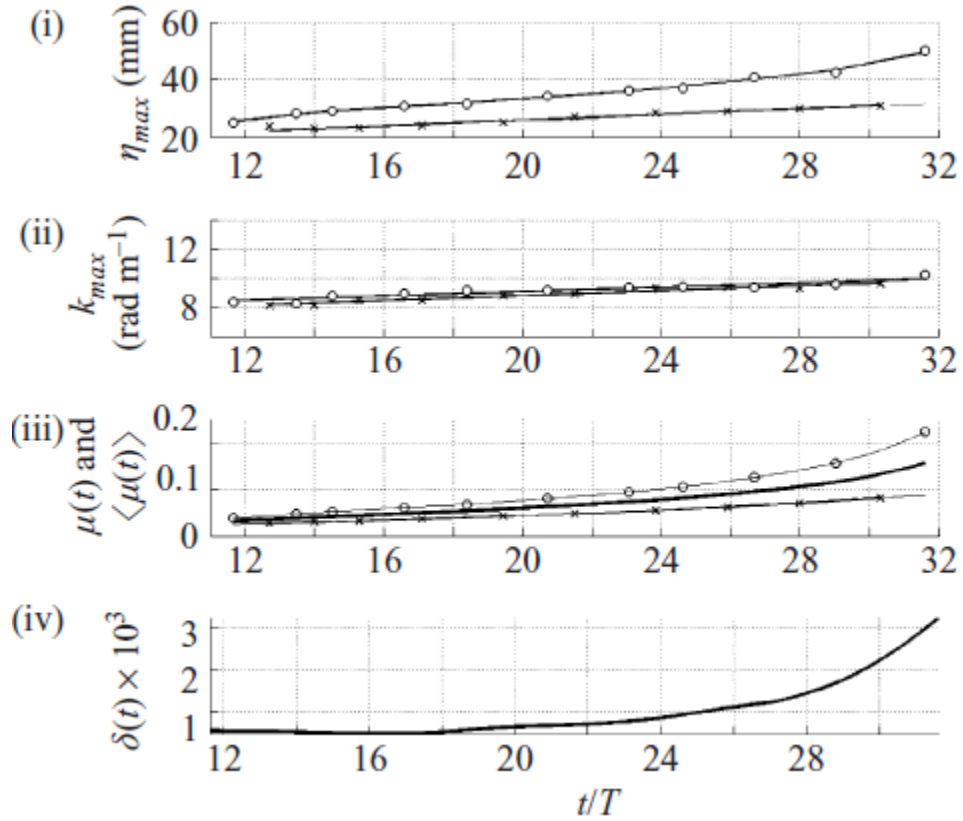


Figure 2-3 Evolution of nonlinear wave groups to recurrence (A) and breaking (B). (i) carrier wave at crest (upper curve) and trough (lower curve) maxima, (ii) corresponding local wave number, (iii) evolution of $\mu(t) = Ek^2$ and (iv) the non-dimensional mean growth rate $\delta(t)$, representing the convergence rate of wave energy to the envelope maximum (source: Banner and Peirson, 2007).

2.3 Wave groups in shallow water

2.3.1 Shoaling on a plane slope

As waves move into shallower water depth, systematic changes in the geometry and kinematics of the wave-forms begin to occur. Waves begin to (non-negligibly) interact with the bed, or ‘feel bottom’ (Komar, 1998), in water depths less than $L_0/2$. This implies that longer period waves begin to interact with the bottom earlier than shorter period waves. From this point until the wave-form becomes unstable and breaks, the waves undergo shoaling transformations.

The most observable transformation of waves during shoaling are changes in the wave shape (Komar, 1976). These are observed within the vertical asymmetry, with increased peakedness of the wave crests and flattening of the troughs (Hedges, 1980) and horizontal asymmetry, with steep front faces and more subdued backs. Changes to the vertical asymmetry may be observed by the growth of higher order harmonics within the wave spectrum which influence wave shape (Flick et al., 1981). The shape of orbital velocity paths beneath wave-forms also changes from near circular in deep water, to elliptical within intermediate water depths, becoming progressively flatter and near linear in shallower depths.

While linear theory is generally intended for describing waves of small amplitude in comparison to water depth, it may be used to provide a reasonable estimate of the general changes which wave-forms undergo during shoaling. As the dispersion relationship (Eqn. 2-2) remains valid in shallow water, Dean and Dalrymple (1991) give the following relationship:

$$\frac{L}{L_0} = \frac{c}{c_0} = \tanh(kd) \quad (2-14)$$

This implies a reduction in wavelength and celerity with decreasing water depth. Along with the reduction in wave celerity, c , the ratio n (Eqn. 2-13) begins to increase. This affects the group velocity, which is a function of both c and n (Eqn. 2-12). As n is inversely proportional to depth, whereas c is proportional to the squared root, n initially increases faster than c decreases (de Lange, 1999). This results in a temporary increase in group velocity before the rate of decrease in celerity exceeds the increase in n and group velocity decreases. Thus, as waves progress into shallower water, wave celerity

approaches the group velocity and wave groups become less dispersive, i.e. waves move relatively more slowly through the group.

As energy and mass must also be conserved throughout the shoaling process according to linear theory, the energy flux (E_f) must remain constant (Eqn. 2-15)

$$E_f = Ecn \quad (2-15)$$

Due to the variations in C and n as waves shoal, variation also occurs in E and consequently wave height, H , from Eqn. 2-5. Initially wave height decreases slightly as the group velocity increases, then increases as group velocity decreases. The ratio of shoaling to deep water wave height is given by Eqn. 2-16 and the relationships of deep to transitional and shallow water wave properties within Figure 2-4 (de Lange, 1999). This wave height increase has been referred to as ‘energy bunching’ (Holthuijsen, 2007) where energy is compacted in the horizontal axis.

$$\frac{H}{H_0} = \sqrt{\frac{c_{g0}}{c_g}} = \sqrt{\frac{c_{g0}}{2nc}} \quad (2-16)$$

While linear wave theory has been found to provide surprisingly good results for predicting shoaling changes in characteristic wave heights of random waves up to break point (Kamphuis, 1994), the theory tends to under predict changes in individual waves, particularly in shallower depths where nonlinearity dominates (Iversen, 1952). This is evidenced within Figure 2-5, where the initial decrease in wave height is greater than predicted by linear theory and the subsequent increase in wave height is substantially greater than predicted by theory. More complex solutions, as obtained by higher order Stokes theory and Solitary theory provide a better approximation of wave evolution in intermediate (Stokes) and shallow water (Solitary) where the growth of higher order harmonics in the spectra becomes important (Figure 2-6).

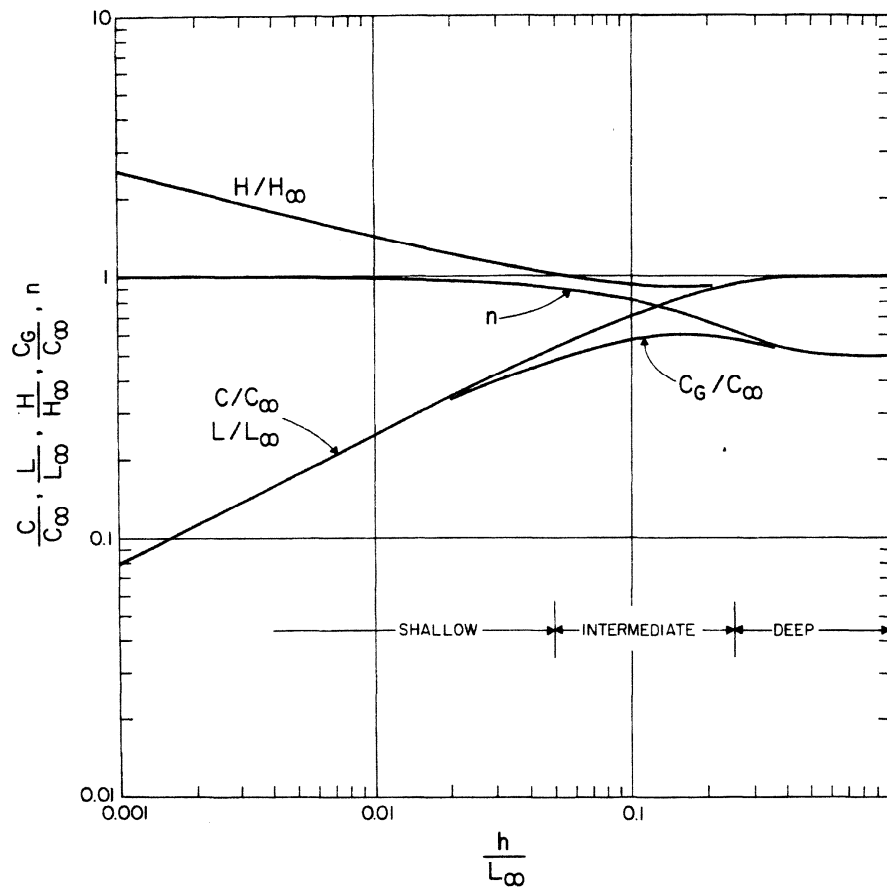


Figure 2-4 Shoaling transformations of linear waves as a function of the ratio of depth (h) to deep water wavelength (L_0) (source: Komar, 1998).

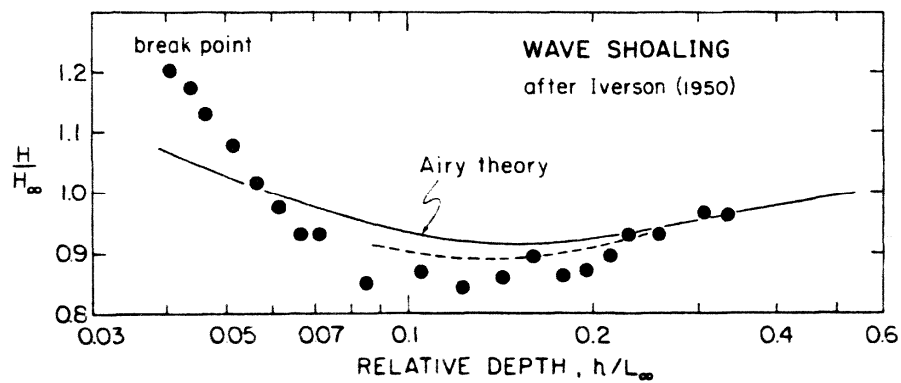


Figure 2-5 Comparison of linear theory (—) to measured changes in wave height (•) during laboratory shoaling (source: adapted by Komar, 1998 from Iversen, 1952).

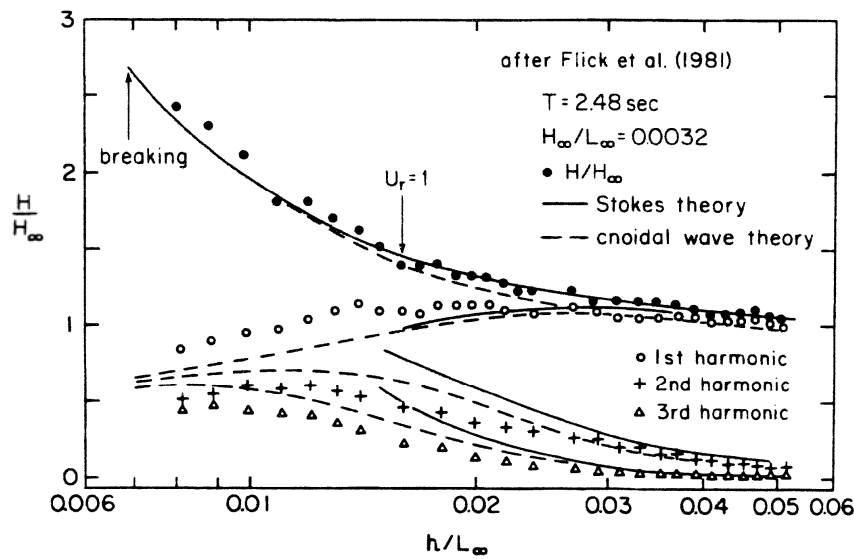


Figure 2-6 Variation in wave height and higher order harmonics during shoaling on a 1:35 plane slope (source: adapted by Komar, 1998 from Flick et al., 1981).

More recently, with the advent of more powerful computational tools, numerical-based depth-integrated and potential flow/Navier-Stokes models have become practical. Examples of depth-integrated models include those based on Boussinesq theory (Peregrine, 1966) and the nonlinear shallow water (NSW) equations. These models incorporate the nonlinear behaviour of shallow water waves to a higher degree (Battjes, 2006), including the generation of bound sub- and super-harmonic waves during shoaling and near-resonant triad interactions (Janssen et al., 2007).

Weakness in the Boussinesq-type models include the non-exponential type velocity profile which give poor representation of intermediate to deep water velocity profiles, slight over-representations of wave heights immediately before breaking and their inability to independently define the breaking position or post-breaking behaviour. While the velocity profile problem can be somewhat overcome by incorporating multi-layer models (Lynett & Liu, 2004), artificial mechanisms such as a limiting wave face steepness or empirical breaker index must be used to define break point (Svendsen, 2006).

Three dimensional, potential flow modelling solves the irrotational equations of motion for a single-phase fluid. This fluid may be discretized by a boundary element (i.e. Jansen, 1986; Drimer and Agnon, 1998) or boundary integral (Dold and Peregrine, 1986; Yasuda, 1997) type scheme. These models are capable of simulating fully

nonlinear and dispersive waves up to the break point without the requirement of an external mechanism to trigger breaking. The models have limitations in describing fluid motion following impact of the overturning jet (Battjes, 2006), after which volume of fluid (VOF) models (i.e. Lin and Liu, 1998) or Lagrangian-based smoothed particle hydrodynamics (SPH) models (i.e. Dalrymple and Rogers, 2006) may be better suited.

The boundary element models (BEM) of Drimer and Agnon (1998) and Dold and Peregrine (1986) were used by Song and Banner (2002, 2004) to model complex intra-wave group processes in deep and intermediate water depths. Local energy density convergences leading to recurrence or breaking (Section 2.2.3) were simulated well into the overturning regime without the requirement of numerical smoothing (Song and Banner, 2004). Simulations incorporating group-induced breaking within shallow water were found to be less reliable.

2.3.2 Wave breaking

While the wave breaking process is the most catastrophic phase of wave development, its description is the least complete. Peregrine (1983) provides a definitive discussion, referencing significant contributions by Longuet-Higgins (1976, 1981), Galvin (1968, 1969), Cokelet (1977), Peregrine (1981) and many others, on the causes and mechanics wave breaking as identified at that time. While a significant number of field (Thornton and Guza, 1986; Massel and Brinkman, 1998; Mead et al., 2001), laboratory (Smith and Kraus, 1990; Chanson, 1997; Duncan, 2001) and, particularly, numerical (Grilli et al., 1997; Lin and Lui, 1998; Helluy, 2005; Dalrymple and Rogers, 2006) studies have since been undertaken, with significant progress made, complete descriptions and solutions to the problem, either theoretical or numerical, have yet to be achieved.

The break point of a wave may be defined in a number of ways. Classical analytical theories (Stokes, solitary and cnoidal) presume the break point is reached when the velocity of particles within the wave crest equals the celerity of the wave-form (*Kinematic criterion*). Alternatively, the *dynamic criterion* (Longuet-Higgins, 1982) assumes that breaking will occur when the horizontal acceleration of the water particles at the surface is equal to or exceeds the gravitational acceleration.

Singamsetti and Wind (1980) summarized more recent definitions of break point as:

- the point at which the wave cannot further adapt to the changing bottom configuration and starts to disintegrate;
- the point where the horizontal component of the water particle velocity at the crest becomes greater than the wave itself (Kinematic);
- the point where the wave height reaches a maximum;
- the point where part of the wave front becomes vertical;
- the point at which the radiation stresses start to decrease;
- the point where the water particle acceleration at the crest tends to separate the particles from the water surface;
- the point where the pressure at the free surface given by the Bernoulli equations is incompatible with the atmospheric pressure.

While a universal definition of break point has not been agreed, Smith and Kraus (1990) considered the point at which the wave front becomes vertical to be a practical definition. However, it is worth noting that from a purely fluid-mechanic viewpoint, *nothing special is noted at the point of verticality but rather, at the point of overturning crest impact* (Peregrine, 1983), where the free surface is strongly deformed and vorticity and turbulence generation is triggered.

Furthermore, from an irregular wave point of view, the point at which the crest first becomes vertical is of little use (Goda, 2007), as this point is unavailable in laboratory studies (due to the variable breaking position). Rather, the concept of characteristic waves and incipient breaking (Kamphuis, 1991b) is employed. To determine incipient breaking, the significant wave height is defined from a distance offshore and through the breaking and surf zone. Curves are fitted to the shoaling portion of the measurements and to the wave height decay portion. The intersection of these curves defines the incipient break point (Goda, 2007). Kamphuis defined the significant wave height at incipient break point, H_{sb} by Eqn. 2-17 where L_{pb} is the wavelength at break point corresponding to the peak frequency, m is beach slope and d_b is the breaking depth.

$$H_{sb} = [0.095 \exp(4.0m)] L_{pb} \tanh(2\pi d_b / L_{pb}) \quad (2-17)$$

2.3.2.1 *Breaking dynamics*

As a wave shoals, its height increases and celerity decreases. However, with the increase in wave height and therefore displacement, the individual water particles within the wave begin to attain higher velocities. The wave becomes increasingly asymmetric (Hedges et al., 1980) and water particles accelerate towards the top and front of the wave crest (Yasuda et al., 1997) until the particles can no longer be constrained and are ejected as a free jet of liquid. Remaining water particles within the wave crest with the greatest accelerations and velocities converge towards this jet adding to its volume and size. The jet moves in a forward and downward trajectory controlled by the velocities of the free water particles and gravity. At present, explanations for the size, shape and trajectory of plunging jets are limited, although numerical findings by Yasuda et al. (1997) indicate that the jet shape is independent of breaking wave height and that the jet size increases as the breaking type progress from spilling to plunging.

If a wave breaks by spilling (Section 2.3.2.2), the jet will impact the wave face in front almost instantly and the resulting turbulence propagates down the face as the wave moves into the surf zone (Ting and Kirby, 1996). If the wave breaks by plunging, the jet projects over the steep wave face in front and impacts the surface at a *plunge point* (Peregrine, 1983). A void forms between the free plunging jet and the wave face. Again, properties of this void in relation to incident wave or seabed properties have yet to be adequately quantified. Longuet-Higgins (1982) showed that a cubic curve gave a good description of the forward face of a plunging wave and Mead et al. (2001) assigned vortex length and vortex width to the void, which combine to give a dimensionless vortex ratio. Attempts were made to link this vortex ratio with the orthogonal bed slope prior to breaking with limited success.

Once the plunging jet impinges the water-free surface in front of the wave a number of phenomena occur. The jet moves into the surface and a body of water is ejecting upward and shoreward; this body is referred to as splash up (Peregrine, 1983). Peregrine (1981) suggested several possible mechanisms for the splash up including a ‘rebound’ of the plunging jet off the free surface or ‘pushing up’ of the initially still surface by the jet.

Experimental investigations into the entrainment of air following the impingement of the plunging jet (Chanson and Lee, 1997) suggest that the dominant jet characteristics influencing entrainment are the impact velocity and the angle between the plunging jet and receiving fluid. Video analysis within a laboratory wave flume (Chanson and Lee, 1997) shows that air bubbles are entrained 1.2 – 2 times the breaking wave height below the surface unless deflected earlier by the bed. The bubbles then become trapped in the surrounding vortical motion before becoming driven by buoyancy and rising to the surface. Experimental studies by Lamarre and Melville (1994) found that up to 50% of wave energy may be spent entraining air and resisting buoyancy forces in deep water.

A number of coherent turbulent flow structures (CTS's) are evident during and immediately following breaking (Dalrymple and Rogers, 2006). These include obliquely descending eddies (ODEs), 'fingers of typist' features on the back of overturning wave fronts and down-bursting. Obliquely descending eddies are formed after the passage of the wave and extend from the free surface to the bed with a near vertical axis. The 'finger of typist' features have been proposed as counter-rotating vortices occurring within the breaking wave. Down-bursting is the vertical rush of water extending from the free surface to the bed during the breaking process. These features have been observed within the field and laboratory (i.e. Nadaoka, 1989) and are beginning to be simulated using Lagrangian-based smoothed particle hydrodynamic models (i.e. Dalrymple and Rogers, 2006).

Between the location of jet impingement and the formation of a quasi-steady zone of wave propagation within the surf zone, a zone of transition occurs (Abadie, 1998). This region has also been described as the 'outer' surf zone (Svendsen et al., 1978). Description and quantification of the processes occurring within this transition zone have been problematic. Ting et al. (1995 and 1996) investigated the characteristics of turbulence induced by plunging and spilling breakers within a flume. Results of the study indicated that the main difference between surf zone turbulence induced by plunging and spilling waves was the method of energy transfer from periodic wave motion to turbulent motion. Under a plunging breaker, the turbulence was found to be dominated by large-scale motions associated with specific plunging conditions. These correspond to the cyclic, although decreasing, plunge and splash up motions noted by Peregrine (1983). Turbulent kinetic energy (TKE) was transported landward under the

plunging breaker and dissipated within one wave cycle. This is in contrast to the spilling breaker, where TKE dissipation occurs much more slowly.

2.3.2.2 Breaker type

The manner in which waves break varies dramatically and have significant implications for the interaction and stability of nearshore structures, sediment transport and nearshore hydrodynamics. Weggel (1964) found breaker type to be function of wave height, period and beach slope.

Wave breaking was categorized by Galvin (1968; 1972) into spilling, plunging, collapsing and surging types based on monochromatic laboratory data. Definitions of the characteristics of the above breaker types are defined within Table 2-1. Galvin notes that breaker type is a continuous variable and as such, the above definitions describe only 4 points on a *breaker type line*. This smooth graduation of breaking types is also noted by Peregrine (1983), with a suggestion that a single parameter family could possibly be used describe all breaking events. Field studies by Galvin (1972) were found in qualitative agreement with earlier laboratory studies (Galvin, 1968).

While early studies employed offshore or break point parameters, B_0 and B_b respectively to define breaking types based on the relation:

$$B_0 = H_0 / (L_0 m^2) \quad (2-18a)$$

$$\text{or} \quad B_b = H_b / (gmT^2) \quad (2-18b)$$

where m is the bed slope, a more commonly used parameter is the Irribarren number or surf similarity parameter of Battjes (1974). The deep and shallow water forms of this are defined by:

$$\xi_0 = \frac{\tan \beta}{\sqrt{H_0 / L_0}} \quad (2-19a)$$

$$\text{or} \quad \xi_b = \frac{\tan \beta}{\sqrt{H_b / L_0}} \quad (2-19b)$$

Transition values which define the breaker type have been derived empirically for the various parameters:

	Surging/Collapsing	Plunging	Spilling
Galvin (1972)	$B_0 < 0.09$	$0.09 < B_0 < 4.8$	$B_0 > 4.8$
Galvin (1972)	$B_b < 0.003$	$0.003 < B_b < 0.068$	$B_b > 0.068$
Battjes (1974)	$\xi_b > 2.0$	$0.4 > \xi_b > 2.0$	$\xi_b < 0.4$
Smith and Kraus (1991)	$\xi_b > 1.2$	$0.4 > \xi_b > 1.2$	$\xi_b < 0.4$

Table 2-1 Breaker type definitions (source: adapted from Galvin, 1972)

Breaker type	Description
Spilling	Foam, bubbles, and turbulent water appear at the wave crest and eventually cover the front face of the wave. Spilling starts at the crest when a small tongue of water moves forward faster than the wave form as a whole. In its final stages, the spilling wave evolves into a bore or an undulatory bore.
Plunging	The whole front face of the wave steepens until vertical; the crest curls over the front face and falls into the base of the wave; and a large sheet-like splash arises from the point where the crest touches down.
Collapsing	The lower part of the front face of the wave steepens until vertical, and this front face curls over as an abbreviated plunging wave. The point where the front face begins to curl over is landward of, and lower than, the point of maximum elevation on the wave.
Surging	The front face and crest of the wave remains relatively smooth and the wave slides up the beach with only minor production of foam and bubbles. Resembles a standing wave.

In general, and as evident by Eqns. 2-18 and 2-19, steeper waves on flatter slopes break by spilling. As the wave becomes lower or the profile steeper, waves progressively change from spilling to breaking in a plunging, collapsing or surging manner. This measure is a crude and often inaccurate approximation of the actual breaking process and has been shown to be inadequate on a number of occasions in the field (Weishar and Byrne, 1978) and laboratory (Sayce, 1997). This is primarily due to the inability of the index to accurately differentiate between intermediate breaker types. Variability in nearshore bathymetry also compromises the parameter's ability to predict breaker type in situations such as barred beaches or rock/reef platforms. For example, rapid changes in seabed slope may induce breaking of a different kind than would be predicted for the average cross-shore slope.

Breaker plunge distances are another example of an empirically determined wave descriptor. The wave plunge distance (X_p) is a function of the forward momentum of the plunging crest of the wave and the downward acceleration of gravity. A plunge distance expression was derived by Galvin (1969) from monochromatic experimental data based on the incident wave height and beach slope.

$$\frac{X_p}{H_b} = 4.0 - 9.25m \quad (2-20)$$

This expression implies that a wave crest will impact a surface sooner if the bed slopes more steeply. However, the breaker type indices (B_b and ξ_b) suggest that plunging is likely to be weaker on a flatter slope. Plunge distance in a weaker breaking wave would be expected to become reduced as the wave tends from plunging to spilling. These two empirical expressions (2-18, 2-20) therefore appear in contradiction. Singamsetti and Wind (1980), Smith and Kraus (1991) and others found the expression of Galvin (1969) to underestimate plunge distance. Smith and Kraus (1991) recommended:

$$\frac{X_p}{H_b} = 3.95\xi_0^{-0.25} \quad (2-21)$$

where ξ_0 is the deep water Irribarren number (Battjes, 1974) as given by Eqn. (2-19). While this expression includes the Irribarren number, which provides an indication of breaking type (Battjes 1974), the formula still predicts greater plunge distances for low Irribarren number spilling waves than for higher Irribarren number plunging waves. Both expressions are unsatisfactory for flatter slopes

A splash distance (X_s) corresponding to distance between plunge point and the impact of the resultant splash-up was originally proposed by Galvin (1969) as approximately equal to the plunge distance. Smith and Kraus (1991) re-evaluated this distance to:

$$\frac{X_p}{X_s} = 0.14\xi_0^{-1.50} + 0.70 \quad (2-22)$$

This expression again indicates a greater splash distance for spilling or gently plunging waves than for more extreme plunging waves.

2.3.2.3 Breaker criteria and predictors

Establishing the point at which wave breaking occurs has been the subject of extensive research since wave theory was in the early stages of development (Stokes, 1880) with a

plethora of indices currently available. These are summarised in table form within Appendix A.

These approaches can be broadly categorised into theoretical or empirical based approaches, and into limiting wave steepness or limiting wave height based approaches. The difficulty in establishing robust and reliable breaking predictor is evidenced by the sheer number of criteria developed, and by the new criteria currently being proposed (Le Roux, 2007; Camenen and Larson, 2007).

The most common approaches have tended towards defining a limiting steepness (H_b/L_b), after which breaking must occur (Stokes, 1880; Michell, 1893; Miche, 1944 and others), defining a breaking wave height with respect to deep water wave height (*breaker height index* - H_b/H_0) (Munk, 1949; Komar and Gaughan, 1972 and others) and defining the maximum wave height which may occur in water of finite depth before the wave becomes unstable and breaks (*breaker depth index* - H_b/d_b) (McCowan, 1891; Iversen, 1952; Galvin, 1969; Goda, 1970; Weggel, 1972 and others). In general the latter, depth-based criterion is most often employed in shallow water or nearshore situations (i.e. USACE, 2003; Goda, 2000).

The kinematic criterion for wave breaking states that when the water particle velocity within the wave crest is equal to or exceeds the wave celerity, the wave-form must become unstable and break (Stokes, 1880). This breaking condition was determined to occur for Stokes waves when the wave crest angle reached 120° . For an oscillatory wave in deep water, this corresponds to a limiting steepness of $(H_0/L_0)_{\max} = 0.142$ (Michell, 1893 in Galvin, 1972) although this value was revised by Yamada and Shiotani, 1968 (in Goda, 2007) for small amplitude waves to $(H_0/L_0)_b = 0.1684$. Miche (1944) gave a theoretical limiting steepness for waves propagating over a horizontal bottom in finite water depth as:

$$\frac{H_b}{L_b} = 0.142 \tanh(kh) = 0.142 \tanh\left(\frac{2\pi d_b}{L_b}\right) \quad (2-23)$$

Although in reasonable agreement with the envelope curve for laboratory data (Dean and Dalrymple, 1991), Madsen (1971) showed errors in its derivation and the horizontal bottom requirement severely limits its use.

Defining the breaking wave height based on deep water wave characteristics (*breaker height index*) provides a simple method for establishing the maximum expected wave height, provided offshore conditions are known. While early theoretical approaches by Munk (1949) neglected the effect of bed slope, subsequent empirical-based studies by Le Mehaute and Koh (1967), Sunamura and Horikawa (1974), Singamsetti and Wind (1980), Smith and Kraus (1991) and others included this slope effect. While this approach provides information on the breaking wave characteristic, the spatial position of wave breaking remains unknown. This is addressed by defining breaking wave height as a function of water depth (*breaker depth index*). This method has the advantage that the maximum expected wave height, provided this is represented by the breaking wave height, may be established for a given depth with little knowledge of the offshore wave conditions. This has benefits for engineering design and similar, where maximum wave height at a specific location or depth is required, rather than the maximum possible height based on offshore conditions.

The *breaker depth index* is generally defined by the ratio of breaker height (H_b) to break point water depth (d_b). The point of breaking according to solitary wave theory was found to depend on initial wave steepness and beach slope. For very flat slopes, a breaking ratio of 0.78 was derived by McCowan (1894). More recent studies by Nelson (1985), using field and laboratory data found that waves propagating over a horizontal slope could not attain this height and recommended a criterion of $H_b/d_b = 0.55$. This lower index was supported by further laboratory studies of regular and irregular waves by Riedel and Byrne (1986) and Gourlay (1994) and by theoretical work of Massel (1996) who proposed $H_b/d_b = 0.6$. Dack and Peirson (2005) achieved slightly larger laboratory values than Nelson and others at $(H/d)_{\max} = 0.59$ by using an idealised wave group spectrum and judicious selection of group phasing relative to a simple bathymetry.

The theoretic breaker index proposed by McCowan (1894) was also found to be inaccurate for waves propagating onto steeper slopes (Iversen, 1952). Based on Iversen's experimental findings, Weggel (1972) derived a series of critical conditions of wave breaking based on monochromatic wave steepness and beach slope. For beaches of negligible slope, this reduces to 0.78. Goda (1970) and Weggel (1972) produced

design curves based on their own and earlier published investigations. These studies form the basis of modern design guides by Goda (2000) and USACE (2003) (Figure 2-7). Equations defining their curves are given by Eqns. 2-24 and 2-25 respectively.

$$\frac{H_b}{d_b} = \frac{A}{d_b/L_0} \left\{ 1 - \exp \left[-1.5\pi \frac{d_b}{L_0} \left(1 + 11s^{4/3} \right) \right] \right\} \quad (2-24)$$

$$\frac{H_b}{d_b} = b - a \frac{H_b}{gT^2} \quad (2-25)$$

where A for Goda's Eqn. 2-24 is given as 0.17 for regular waves and a and b for Weggel's Eqn. 2-25 by $a = 43.8(1 - e^{-19\tan\beta})$ and $b = 1.56/(1 + e^{-19.5\tan\beta})$.

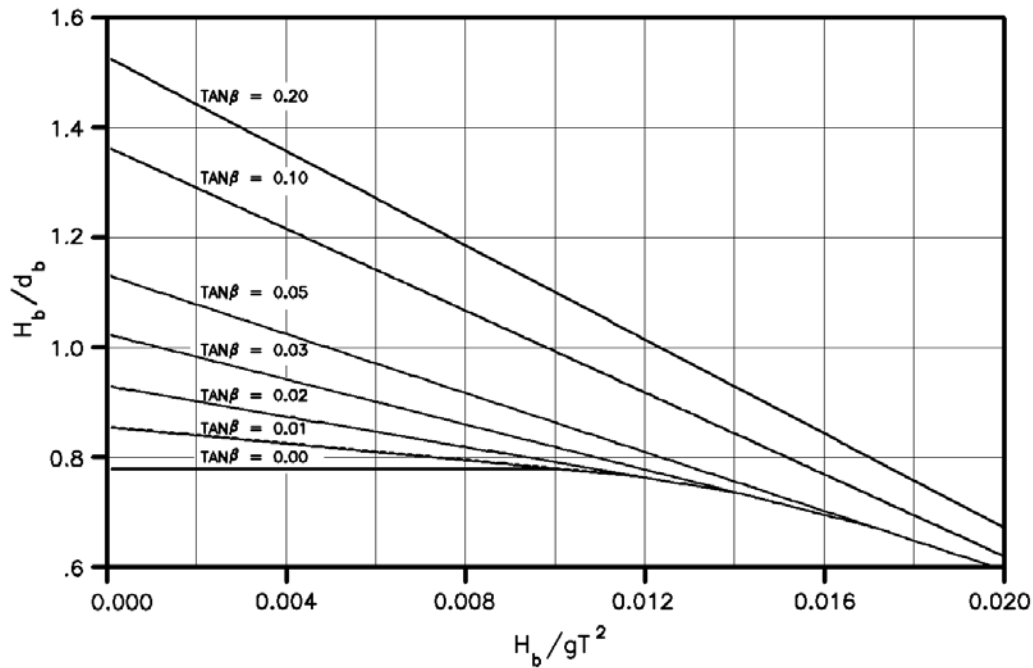


Figure 2-7 Breaker depth index guidelines for regular waves on a plane slope (source: USACE, 2003).

While these design curves and formula provide a simple and apparently robust method for assessment, the raw data, upon which the existing guides are based (Figure 1-1) show significant scatter, with data points evident above the recommended design curves. This scatter is of interest as regular monochromatic waves should, theoretically, always break consistently in the same manner and in the same location. Investigation of the validity of these curves for group-dominated wave-trains is a key aim of this thesis.

Motivated by this apparent scatter and deficiencies in existing criteria, experiments investigating the depth-limited breaking problem continued, with new criteria derived

almost yearly. These were typically based on the existing data, along with newly collected data. Rattanapitikon and Vivattanasirisak (2003) used 26 sources of existing data, comprising 695 combinations of bed slope and offshore wave conditions to derive a criterion. Smith and Kraus (1990) suggest that the majority of breaker depth indices use variables for the bed slope and deep water wave steepness and can thus be represented by:

$$\frac{H_b}{d_b} = C_1 m^{n_1} \left(\frac{H_0}{L_0} \right)^{n_2} + C_2 \quad (2-26)$$

where C_1 and C_2 are variable coefficients and n_1 and n_2 , variable exponents. Goda (2007) categorizes breaker indices into the following types:

$$H_b/d_b = f_1(0) = \text{constant}$$

$$H_b/d_b = f_2(d_b/L_0 \text{ or } d_b/L_b)$$

$$H_b/d_b = f_3(m)$$

$$H_b/d_b = f_4(m, d_b/L_0 \text{ or } d_b/L_b)$$

More recent studies have focussed on evaluating numerous existing criteria using large volumes of existing data to either appraise their performance and provide recommendations, or to slightly modify existing criteria (i.e. Rattanapitikon and Shibayama, 2000; Le Roux, 2007). While new criteria may result in lower errors across varying bed slopes and offshore wave conditions (Rattanapitikon and Vivattanasirisak, 2003), they are complex and difficult to use in practice (i.e. Camenen and Larson, 2007), represent average rather than design conditions and fail to identify fundamental causes of scatter.

Goda (2007) attributes this observed scatter in breaker index to an inherent variability in wave breaking (also noted by Smith and Kraus, 1990). Goda (2007) quantified this scatter at 6 to 14% depending on bed slope, with steeper slopes exhibiting greater fluctuation. He concludes that the breaking process is *stochastic*, and that a certain range of natural fluctuation should be accepted. This raises the question of whether a best-fit or an upper envelope type curve around measured data is more appropriate. Smith and Kraus (1990) suggest that *the average index from scattered data is more useful for most coastal-process studies while the maximum of the envelope is more appropriate for wave-force calculations*. Possible explanations for these discrepancies,

including disparity in the definitions of characteristics employed (i.e. Black and Rosenberg, 1992), and the effect of these on breaker criterion, are further discussed within Section 2.5.

Defining a breaking criterion for irregular waves becomes significantly more complicated than for regular waves (Smith and Kraus, 1990). This is due to spatial and temporal variation in breaking positions, interactions of breaking wave-forms with currents and turbulence induced by earlier breaking waves, and the influence of long waves and associated features characteristic to irregular wave fields. Firstly, actual definition of the breaking position and the associated wave characteristics at that point is very difficult for irregular waves, although this has been undertaken by the likes of Seyama and Kimura (1988) who analysed individual breaking waves using wave gauges and a video camera. To overcome this difficulty, a characteristic measure of wave height such as $H_{\max}$, H_{sig} or H_{rms} is generally employed, with the concept of incipient breaking, as defined earlier within this section, used to define the initial break point. For example, Thornton and Guza (1983) propose that for irregular waves, the ratios $H_{\text{rms},b}/d_b = 0.43$ and $H_{\text{mo},b}/d_b = 0.60$ may be assumed within the saturated surf zone. Durand (1997) notes however, the that use of $H_{\max}$ as a characteristic measure is highly dependent on sample size and thus inherently unstable, with H_{rms} or similar providing much more robust statistical measures.

Goda (2007) suggests that indices for regular waves may be assumed as an upper limit for irregular waves, i.e. $H_{\max(\text{irreg})}/d \approx H_{b(\text{reg})}/d$. This is supported by experimental findings of Durand (1997) where maximum wave heights within an irregular wave field are found to be well predicted by the regular methods of Weggel (1972) and Goda (1970). Goda found ratios based on significant wave height (H_{sig}/d) were approximately 30% lower than for regular waves at the outer boundary of the surf zone. This was supported by Seyama and Kimura (1988) and Kamphuis (1991) who found lower indices for irregular waves compared to regular. Goda (1975) proposed that his earlier criterion for regular wave breaking (Eqn. 2-24) could be employed for determining significant wave height in irregular wave breaking with the coefficient A taken at 0.12 (c.f. 0.17 for regular waves) at incipient breaking. This coefficient was found by several investigators to increase across the surf zone as the relative water depth decreased (Goda, 2007). The effect of this increase on the assumption that regular wave indices

provide an upper envelope for irregular wave breaking is further discussed within Section 2.5.

Svendsen (2001) investigated the effect of varying the groupiness of a wave-train (as defined by $G = \Delta H/H_m$, where $G = 0$ corresponds to a regular wave-train, and $G = 1$ to a fully modulated wave-train) on wave breaking on a plane slope of 1:35. Svendsen found that the horizontal distance over which waves broke increased with increased groupiness. Additionally, the breaker depth indices of individual waves within the group varied. Increased groupiness in a wave-train resulted in larger variations in breaker index. This is inconsistent with the assumption that waves near break point may be described according to solitary wave theory, for which the breaker index should be constant. In all cases, apart from the near monochromatic case, the larger waves within the group, particularly those toward the front of the wave group (waves 2 and 3 of 5), broke with higher breaker depth index values, similar to values expected for regular wave breaking (based on Hansen, 1990). Other, lower waves at either end of the group broke with smaller breaker depth indices. Suggested explanations for these differences in breaker indices include the smaller index waves having shorter wavelengths and therefore being steeper at break point, and the time-varying current associated with group-induced long waves inducing steeper waves. However, reasons for the differences in wavelength are not presented.

As discussed within Section 2.2.3, Song and Banner (2002) proposed the use of local energy convergence rates to predict the onset of breaking within deep water. Song and Banner (2004) used fully nonlinear models to investigate whether the deep water breaking criterion for two-dimensional wave groups, established by Song and Banner (2002), and experimentally verified by Banner and Peirson (2007), remained applicable in finite water depth. Although testing in shallow water was restricted due to limitations in the numerical code employed, they found the threshold growth parameter of $\delta(t) = 1.40 \pm 0.1 \times 10^{-3}$ to remain applicable in intermediate water depths ($d \geq L_o/3$). They found, however, that the presence of a sandbar, of length greater than $1/4$ the water depth, increased this threshold. This was attributed to energy convergences associated with the sandbar ‘competing’ with the group-induced convergences and inducing further nonlinearities into the wave-forms. These appeared to stabilise the wave-forms and inhibit breaking. This was supported by the findings of Dack and Peirson (2005) where

judicious phasing of an evolving wave group propagating onto a horizontal bed resulted in breaker indices in excess of those found by comparative studies using both regular and random waves. The studies showed that while intra-wave group interactions and local energy convergence remains a key factor in initiating wave breaking, bed effects also start to have influence. These bed effects distort the critical rate of energy convergence which dictates when breaking is initiated and appear to enable larger waves to occur prior to breaking.

2.3.3 Surf zone processes

After wave breaking, the oscillatory wave form is transformed to a translational form. This is generally evidenced within the field or laboratory by a bore of turbulent flow, or white water propagating at the front of the wave form. This turbulent bore is able to transport water forward within its crest, therefore causing onshore mass flow. The wave may continue to propagate through the surf zone as a turbulent bore, or may stabilize and reform into an oscillatory wave form (Smith and Kraus, 1990). Having accurate descriptors of wave decay and water level within the surf zone is important as it is these characteristics which drive a number of fundamental surf zone processes such as nearshore currents and sediment transportation.

As opposed to wave propagation prior to breaking where energy and momentum is, for the most part, conserved, translational waves within the surf zone lose energy as they progress. This dissipation of energy may occur through turbulence and entrainment of air (Lamarre and Melville, 1994) and bed interaction. As classical wave theories are incapable of describing waves past the break point, alternative methods such as the similarity method (Longuet-Higgins and Stewart, 1963) or an energy flux method (LeMehaute, 1962; Battjes and Janssen, 1978) are used to calculate change in wave height within the surf zone. The similarity method assumes a constant wave height to depth ratio between break point and the shore. As depth at the intersection of the bed and still water is assumed to be zero, wave height must have correspondingly reached zero at this point. The method has been found to overestimate wave height on slopes greater than around 1:30 and underestimate height on flatter slopes (USACE, 2003). Smith and Kraus (1988) found that a power law relationship between wave height and depth (Eqn. 2-27) gave a better approximation of wave height decay on lower beaches.

$$H = \gamma_b h_b \left(\frac{h}{h_b} \right)^n \quad (2-26)$$

where:

$$n = 0.657\gamma_b + \frac{0.043\gamma_b}{\tan \beta} - \frac{0.0096}{\tan \beta} + 0.032$$

Energy flux methods calculate the energy balance, given by Eqn. 2-28 (Battjes and Janssen, 1978), to predict change within the surf zone.

$$\frac{dEc_g}{dx} = \langle \varepsilon \rangle \quad (2-28)$$

Where x is the cross-shore distance and $\langle \varepsilon \rangle$ represents the average dissipation per unit area. The amount of energy dissipated during breaking and by the subsequent turbulent propagation of the wave-form is approximated using various methods. LeMehaute (1962) used an analogy with hydraulic jump, Battjes and Janssen (1978) used a periodic bore assumption (Figure 2-8) and Lippman (1996), a surface roller model. The energy dissipated per unit area using the periodic bore assumption of Battjes and Janssen (1978) is given by Eqn. 2-29 (Stoker, 1957).

$$\varepsilon_b = \frac{1}{4} \rho g \frac{(d_2 - d_1)^3}{d_1 d_2} Q \cong \frac{1}{4} \rho g \frac{(BH)^3}{d^2} Q \quad (2-29)$$

Where the depths d_1 and d_2 define depths at the trough and crest respectively, Q is the volume discharge per unit area across the bore and B is a breaker coefficient which depends on the breaker type. This coefficient is often defined as a function of the foam region on the wave face, but in practice is typically determined empirically. The dissipation due to bed friction (ε_f) may also be included but is generally found to be a minor contributor to dissipation compared to breaking and turbulence (Thornton and Guza, 1983). This conclusion is in contrast to the findings of Lowe et al. (2005), who tested the surface dissipation rates of waves propagating over a rough coral reef by measurement of water levels and velocities. Results of this study indicated that while frictional dissipation during and immediately following breaking was approximately equal to turbulent dissipation, dissipation on the reef flat was dominated by bed friction. Disparities were attributed to the extreme roughness of the coral bed with physical length scales in the order of 0.3 m. However, the study indicated that the influence of bed roughness on wave breaking should be taken into account in wave breaking and propagation models. Stive (1984) developed this technique further by adding correction

coefficients to the formula to take into account the effects of turbulent flow and explain observed discrepancies between classical theory and experimental results.

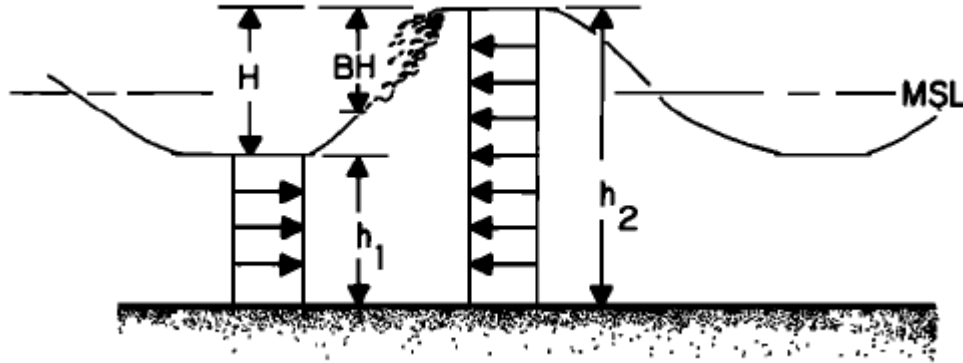


Figure 2-8 Periodic bore model used to describe waves within the surf zone (source: Thornton and Guza, 1983).

Studies into monochromatic wave decay within the surf zone by Masselink and Black (1995) found that waves do not remain at the critical water depth ratio at which they broke, but rather decay to a stable height. This stable ratio has been defined by several researchers at between 0.35 and 0.5 (i.e. Horikawa and Kuo, 1966; Thornton and Guza, 1983; Dally et al., 1985), significantly below the breaker depth indices which define initial wave breaking. Waves reaching ratios below this stable ratio are assumed to reform into a translational state. Using this stable wave height (κ), Dally et al. (1985) derived the loss in wave energy (Eqn. 2-30) from the difference between the local energy flux and the *stable* energy flux:

$$\frac{\partial E_f}{\partial x} = -\frac{\kappa}{h}(E_f - E_{f-stable}) \quad (2-30)$$

The transformation and dissipation of irregular waves is more difficult to determine as at any one spatial location there are likely to be broken, breaking and unbroken waves present. Irregular dissipation models, such as developed by Goda (1975), Battjes and Janssen (1978), Thornton and Guza (1983), Dally et al. (1985) and others, take this into account, with allowances made for the variable position of wave breaking. Two common approaches are wave by wave modelling, where the breaking and transformation of individual waves corresponding to an assumed spectrum are

calculated and a probability density function (pdf) approach where changes to a wave height distribution during breaking and through the surf zone are modelled.

The wave by wave approach using, for example, the decay model of Dally et al. (1985) allows for the consideration of uneven bed topographies, with the wave height distribution at any point within the surf zone determined by combining individual wave heights at that point. Characteristic measures of wave height such as $H_{\max}$, H_{sig} , H_{rms} may then be obtained. This method has been verified (USACE, 2003) with field studies by Dally (1990, 1992) and Larson and Kraus (1991).

The pdf transformation approach models the changes in the wave height distribution during propagation into and through the surf zone. An assumption is generally made that larger waves break at an earlier stage. Thus, larger waves within the distribution are truncated and thereafter treated as broken, with dissipation calculated according to one of the aforementioned decay models. An initial Rayleigh distribution is typically assumed (Collins, 1970; Battjes, 1972; Kuo and Kuo, 1974; Goda, 1975), using either a straight truncation at a predefined breaker index, or a variable index to account for observed variation in the breaker index of irregular waves (Goda, 1975). Goda (2002) found that wave distribution, originally Rayleigh distributed outside the surf zone, narrows within the surf zone due to truncation of larger waves, but return to Rayleigh nearer the shore due to regeneration post-breaking. This is also observed with a reduction of the ratio of $H_{2\%}/H_{\text{sig}}$ following breaking, followed by an increase in the ratio nearer the shore.

These differing approaches to truncation are illustrated within Figure 2-9 (Thornton and Guza, 1983). Battjes and Janssen (1978) base their truncation criterion on a limiting wave steepness, calculated according to the Miche (1944) criterion. This limiting steepness is advantageous in allowing for wave steepening within a current, together with depth induced steepness. The standard error of H_{rms} or H_{sig} compared to laboratory data was found to be around 10% (Battjes and Janssen, 2008).

Thornton and Guza (1983) assume a Rayleigh distribution throughout the surf zone, with waves exceeding a specified breaker index treated using a periodic bore dissipation model. The coefficient B , which describes breaking type (Eqn. 2-29) is calibrated

empirically to give best-fit results. Although this approach tends to overestimate waves within the tail of the spectrum and thus extreme characteristics such as $H_{\max}$, good agreement was found for central statistics such as H_{sig} and H_{rms} when compared to field data, with the H_{rms} standard error equal to 8.5% throughout the surf zone. More recent refinements of these older models by Baldock et al. (1998), Janssen (2006), Janssen et al. (2007) and Alsina and Baldock (2007) have resulted in small improvements in relative errors; however, the continued improvement appears restricted by fundamental limitations in robustly defining wave breaking thresholds and dissipation rates for a stochastic and inherently variable process (Goda, 2007).

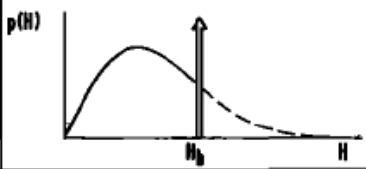
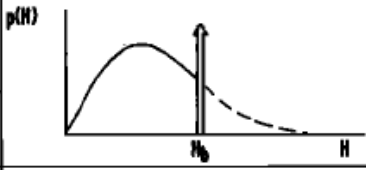
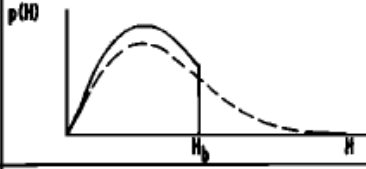
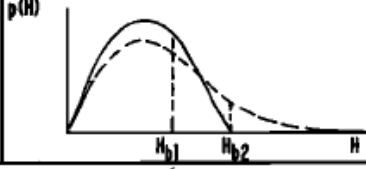
AUTHOR	DISTRIBUTION	SHOALING	BREAKER CRITERIA
COLLINS (1970)		LINEAR	$\frac{H_b}{H_d} = 0.76 \beta^{1/7} (H_d/L_d)^{-1/4}$ (AFTER LAMEHAUTE AND ROH, 1967)
BATTJES (1972)		LINEAR	$H_b = \frac{0.68}{k} \tanh\left(\frac{\gamma}{0.88} kh\right)$
KUO and KUO (1974)		LINEAR	$H_b = 0.63h$
GODA (1975)		NONLINEAR (SHUTO, 1974)	$\frac{H_b}{H_d} = A \frac{L_d}{H_d} \left\{ 1 - \exp \left\{ -1.5 \frac{h}{H_d} \frac{H_d}{L_d} (1 + K \tan^P \beta) \right\} \right\}$ (AFTER GODA, 1975)

Figure 2-9 Modified probability functions due to wave breaking. The dotted lines represent the original Rayleigh distributions and the heavy lines represent the modified distributions following breaking. H_b describes the breaking wave height of H_{sig} and arrows indicate all waves greater than H_b have broken (source: Thornton and Guza, 1983).

Studies into the spectral changes observed within the surf zone show that energy is transferred to higher and lower frequencies during wave breaking and propagation (Guza and Thornton, 1980; Flick et al., 1981). Dissipation has been shown to occur primarily from the high frequency tail of the spectrum (Herbers and Russnogle, 2000).

Similarly, Goda (2002) found nonlinearity, strongest at the outer edge of the surf zone, is destroyed by wave breaking within the surf zone.

The preservation of wave groupiness into the surf zone has also been noted by several investigators both within the field (List, 1991) and in laboratory studies (Svendsen et al., 2001) and has been noted as a possible mechanism for the generation of infragravity waves (Watson and Peregrine, 1992; Baldock et al., 1997), as discussed within the following section. Baldock et al. (1997) attributes the persistence of groupiness within the surf zone to non-saturation of incident waves, i.e. waves of varying height may occur at any defined depth and thus groupiness is able to remain. This is in disagreement with earlier findings of complete saturation at and above short wave frequencies within the inner surf zone, thus inferring groupiness is destroyed (i.e. Guza and Thornton, 1985). These studies were, however, undertaken on flat, dissipative beaches whereas the studies of List (1991) and Baldock et al. (1997) were carried out on steep slopes. This infers that the persistence of groupiness into the surf zone is a function of beach slope and saturation, with higher groupiness persisting in steeper environments (Baldock et al., 1997).

2.3.3.1 Mean water levels

In the nearshore zone, the mean water level is generally displaced from the still water level due to the presence of waves. The theoretical explanation for this was proposed by Longuet-Higgins and Stewart (1964), referred to hereafter as L-H&S64. For steady breaking, the average momentum flux is directed in an onshore direction. To balance this momentum flux, a pressure gradient is created by elevation of the water level. Water level is highest at the beach face, and drops towards the break point, creating an offshore gradient. However, this process is complicated by differences in the cross-shore seabed gradient and longshore wave height which may cause gradients in the alongshore direction. Maximum set down theoretically occurs at the break point before a rapid change in the direction of momentum flux occurs.

As waves propagate, they transmit energy and momentum. The rate at which a wave transmits momentum is the momentum flux or radiation stress. This flux occurs in both the direction of wave propagation (S_{xx}) and perpendicular to the direction of

propagation (S_{yy}). These fluxes per unit wave crest length according to linear wave theory are given in Eqns. 2-31a and 2-31b.

$$S_{xx} = E \left[\frac{2kd}{\sinh(2kd)} + \frac{1}{2} \right] \quad (2-31a)$$

and

$$S_{yy} = E \left[\frac{kd}{\sinh(2kd)} \right] \quad (2-31b)$$

Where E is total wave energy density. It is evident from these equations that the radiation stresses are affected by wave energy density, the wave number and local water depth.

The variant water level may be determined theoretically for monochromatic, linear waves based on expressions derived by L-H&S64. The momentum balance for steady breaking is given by Eqn. 2-32 and resultant set-up (η) by 2-33. Subsequent laboratory studies (Bowen et al., 1968; Van Dorn, 1976; both in Komar, 1998) found good agreement of set-up and set down with that predicted by L-H&S64.

$$\frac{dS_{xx}}{dx} + \rho g(\eta + d) \frac{d\bar{\eta}}{dx} = 0 \quad (2-32)$$

and

$$\eta = -\frac{1}{2} \frac{a^2 k}{\sinh(2kd)} \quad (2-33)$$

Subsequent studies (Stive and Wind, 1982) found that the linear model used by L-H&S64 tended to over predict the maximum set-up of steep waves. Nonlinear theories (i.e. Cokelet, 1977) were found to better predict the maximum set-up elevation; however, disparity is noted around the position of maximum set down. This is attributed to uncertainty of where momentum flux transitions from the offshore to onshore direction and is related to the manner and position of wave breaking. Studies of irregular waves breaking in the field (Guza and Thornton, 1981) and Holman and Salenger (1985) found that maximum set-up could generally be related to offshore wave height. Thornton and Guza, undertaking studies on a relatively flat beach, found a relationship of $\eta_{\max} = 0.17H_0$. However, Holman and Salenger (1985), studying set-up on a steeper beach of around 1:10, found the relationship of maximum set-up to offshore wave height to be considerably scattered. Improvement was obtained by incorporation of the deep water Irribarren number, with a best-fit line given by $\eta_{\max}/H_0$

$= 0.43\xi_0$. Considerably more scatter was observed during lower tidal levels, when offshore topography became more important to wave transformation. This finding was supported by Hanslow and Nielsen (1993), who found a weak relationship between average beach slope and maximum set-up on low dissipative beaches. This indicates that under these circumstances, offshore topography is of greater importance than average slope.

The relationship between water level and wave height within the surf zone is interrelated, with the rate of dissipation or change in wave height affecting momentum flux or radiation stress and therefore water level, and changes in water level affecting depth and therefore achievable wave height and rates of decay. Variation in mean water level and in the wave height distribution through the surf zone must therefore be solved simultaneously (Goda, 2000). This concept is incorporated into irregular wave models such as that of Goda (1975).

2.4 Long waves associated with wave groups

Waves at frequencies below that of incident swell were initially noted offshore of the surf zone by Munk (1949) during nuclear test monitoring and subsequently observed by Tucker (1950). These waves, with periods of 60 – 600 s (Tucker, 1950), were originally termed *surf beat*, although more recently have been referred to as *infragravity waves*. These waves were found to be highly correlated with the wave group period (T_g) and similarly high correlation was noted between the long wave trough and the incident wave maxima.

Long waves arise from several generation mechanisms. Ruessink (1998) describes three such mechanisms:

1. long waves forced by, and bound to, incident wave groups (Longuet-Higgins and Stewart, 1962). These bound waves may subsequently be released following breaking;
2. the time-varying break point position associated with wave height modulation within a group (Symonds et al., 1982; Battjes et al., 2004));
3. the persistence of wave groupiness within the surf zone, thus inducing changes in the mean water level at the shoreline (Watson and Peregrine, 1992; Baldock et al., 1997).

The propagation of wave groups has been shown to influence water levels through variations in radiation stress (Longuet-Higgins and Stewart, 1962, 1964). During the passage of larger waves, radiation stresses increase and water level beneath the waves is locally depressed, while during the passage of smaller waves, water level is locally elevated. Thus the group-bound long wave has a 180° phase difference with the incident wave envelope (Ruessink, 1998). This long-wave, bound to the wave group, is depicted in Figure 2-10 and may be described by Eqn 2-34 (Longuet-Higgins and Stewart, 1964).

$$\bar{\eta} = -\frac{g|a|^2}{2(gd - c_g^2)} \left(\frac{2c_g}{c} - \frac{1}{2} \right) \quad (2-34)$$

Where

$\bar{\eta}$ = the elevation of water surface induced by the wave group

a = wave-train amplitude

c_g = wave group velocity

c = wave velocity

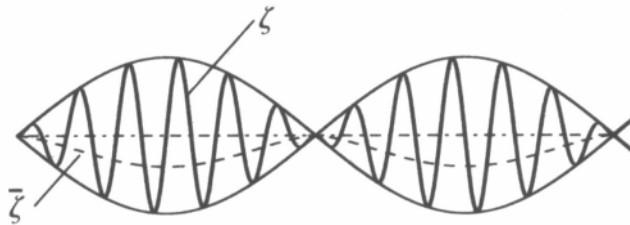


Figure 2-10 Long waves induced by radiation stresses associated with propagating wave groups (source: Mei et al., 2005).

These bound waves increase in amplitude as water depth decreases and short waves shoal and transfer energy to the sub-harmonic regime (Battjes et al., 2004), although the amount of growth has been found less than Eqn. 2-34 would indicate for shoaling water (Janssen et al., 2003). These waves remain bound to the group until break point, after which Longuet-Higgins and Stewart (1964) theorised their release as freely propagating long waves which may subsequently reflect from the shoreline. This increase in long wave amplitude prior to their release explains the larger reflected waves observed by

Munk (1949) and Tucker (1950), although the exact mechanisms for their release at break point was still not fully explained by the theory. Support for this theory has been shown within field (Guza et al., 1984), laboratory (Janssen et al., 2003) and numerical (List, 1992) results. However, according to bound wave theory, a quadratic relationship between short and long amplitudes should exist (Battjes, 1988), whereas a linear relationship is more frequently observed (Baldock et al., 2004).

This discrepancy is partially addressed by the alternative generation mechanism of a time and space-varying break point (Symonds et al., 1982; Schaffer, 1993). This varying break point changes the surf zone width, radiation stress distribution and associated water levels and thus acts as a pseudo-wavemaker, releasing long wave energy in both an onshore and offshore direction. This variable break point-induced long wave is evidenced as a dynamic set-up at the shoreline where a long wave crest precedes the wave group (i.e. Baldock, 2004) before being subsequently reflected offshore.

The theory of Symonds et al. (1982) makes the assumption of complete saturation within the surf zone. This assumption implies that at every depth, wave height is at a maximum based on a defined breaker index. This assumption also implies complete destruction of groupiness within the surf zone. However, comparison of this varying break point-generated long wave model with laboratory data (Kostense, 1984) showed discrepancy (Roelvink, 1993). Relaxing the assumption of complete saturation within the surf zone, and thus allowing some persistence of groupiness, as has been observed within the field (i.e. List, 1991), considerably improved model fit to laboratory data (Schaffer and Jonsson, 1990). This persistence of groupiness within the surf zone provides the third mechanism for long wave generation, although is likely to be related to the varying break point mechanism (Schaffer, 1993).

The relative importance of the various generation mechanisms has been the subject of debate, with Guza et al. (1984), Masselink (1995), Battjes et al. (2004) and others finding the release of bound waves to be the major contributor with only marginal proof for the existence of break point forced long waves, yet Schaffer (1993) found the generation of long wave energy by bound-wave release and by varying break point to be roughly comparable, with the persistence of groupiness within the surf zone only a minor contributor.

Baldock (2004) suggests differences in the relative importance of generation mechanisms and the resultant characteristics of long waves on beaches of differing slope. On steeper slopes, long waves generated by the varying break point were found to be more prominent due to larger variations in radiation stress gradients and limited enhancement of bound long waves prior to breaking. Baldock (2004), shows results of a laboratory study on a plane, steep faced (1:10) beach where the reflection of the dynamic set-up crest was found to be much more efficient, with only marginal reflection of the released bound wave trough from the shoreline (Figure 2-11). On gentler slopes (and typically most field sites), the effectiveness of the varying break point is reduced due to phase cancellation (Baldock and Huntley, 2002) and bound waves become enhanced to a greater degree during shoaling (Battjes et al., 2004); therefore the release of these bound waves is more dominant.

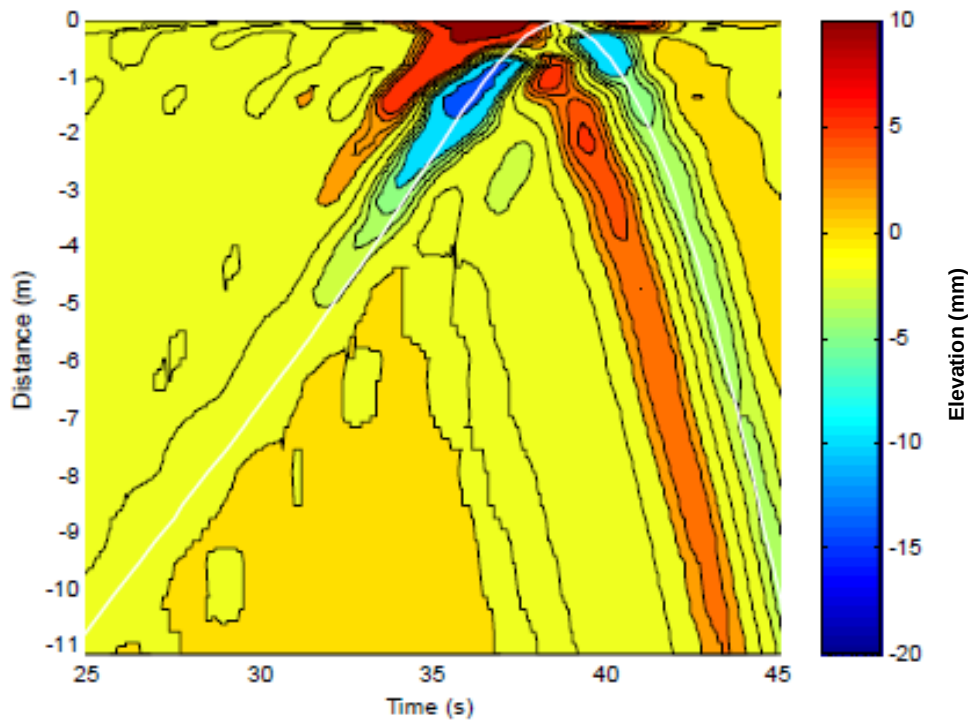


Figure 2-11 Space-time plot of the low-frequency surface elevation associated with wave group propagation on a steep (1:10), plane sloped beach. Solid white line indicates the propagation path of the wave group velocity (source: Baldock, 2004).

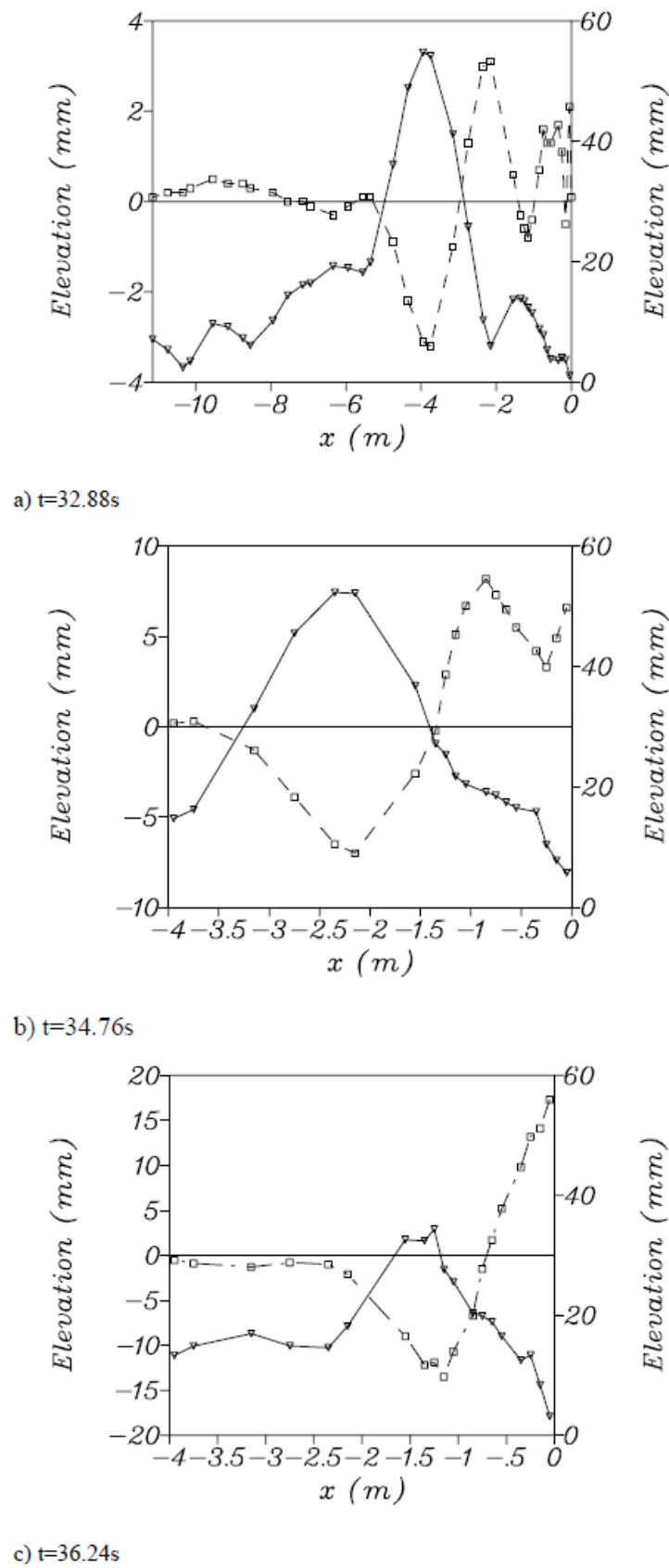


Figure 2-12 Spatial variation in surface elevation envelope (∇ - right hand axis) and low frequency elevation ($\square$ - left hand axis) at different times (source: Baldock, 2004).

As discussed earlier, changes in mean water level must be incorporated into models of irregular wave transformation within the surf zone. However, temporal changes in water level within the surf zone due to long wave propagation are less well incorporated. Goda (2000) offers an empirical expression for estimating the amplitude of surf beat within the surf zone (Eqn. 2-35):

$$\zeta_{rms} = \frac{0.01H'_0}{\sqrt{\frac{H'_0}{L_0} \left(1 + \frac{d}{H'_0}\right)}} \quad 2-35$$

where ζ_{rms} is the root-mean-squared value of the surf beat profile, H'_0 is the offshore significant wave height and d is the depth, presumably based on still water level.

2.5 Engineering design within the surf zone

A variety of structures including groynes, breakwaters, seawalls, piers and pipelines are placed on a regular basis within the coastal nearshore. These structures serve a variety of purposes including providing protection from wave energy (i.e. breakwaters, seawalls, etc), modification to coastal processes, such as longshore sediment movement (i.e. groynes), mooring for boats and recreational uses (i.e. piers and jetties).

The successful design of nearshore structures is greatly dependent on our ability to accurately define the wave loading which could be placed on the structure. This is evidenced for example by the wave height cubed relationship to the unit mass of rock riprap or armour units within design codes (USACE, 2003), thus inferring that a small increase in wave height will result in a large increase in required unit mass. Additionally, the maximum run-up elevation that a wave may achieve is critical to safety and structural aspects of design and is directly related to the energy which may reach the structure. It is therefore imperative that we have good predictors of wave height, without being overly conservative, to facilitate reliable yet economic design.

Design guidelines such as the *Coastal Engineering Manual* (USACE, 2003), *Rock Manual* (CIRIA, 2007) and the *EurOtop Wave Overtopping of Sea Defences and Related Structures: Assessment Manual* (Pullen et al., 2007) are used extensively worldwide to provide practitioner guidance on nearshore wave assessment and structural design. While methods and techniques for assessment of run-up and material

sizing differ somewhat dependent on structure type and publication, they all remain acutely dependent on accurate definition of a design wave height. This aspect in particular, and factors affecting its use and precision are discussed.

2.5.1 Structural considerations

Design of nearshore structures typically includes the major components of:

1. determining the hydraulic response of waves to such a structure, including anticipated run-up elevation and, in the case of likely overtopping, the expected overtopping volumes;
2. sizing of structural members or armour rock/units to withstand anticipated wave forces.

In both cases, the wave height able to reach, and affect, the structure is of primary concern.

2.5.1.1 Run-up and overtopping

The run-up heights and overtopping volumes of incident waves influence both the stability and sizing of material at the crest and back slope of a structure and also safety considerations. Run-up on nearshore structures such as breakwaters and revetments differs from that on a natural beach slope due to the generally steeper slope of structures, their rougher and more highly permeable composition and their placement in often deep water. This placement potentially exposes the structure to significantly greater wave energy than is experienced within the back-shore swash zone. Therefore, while the calculation of run-up on a beach is generally based on deep water wave properties, i.e. Hunt (1959), Guza and Thornton (1982), Holman (1986), wave run-up on a structure is dependent on the wave energy which may reach that structure.

While a number of equations and formulae exist to predict run-up on a structure, values typically predict the run-up elevation exceeded by a percentage, i , of swash incidences. The run-up ($R_{ui\%}$) is generally given as a function of wave height at the structure toe (H), breaker type as defined by the Irribarren number (ξ , Section 2.3.2.2), wave angle of incidence and structure characteristics including profile slope, roughness and permeability. Run-up of irregular waves on an impermeable slope is given by USACE (2003), after Battjes (1974), as:

$$\frac{R_{ui\%}}{H_s} = (A\xi + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \quad (2-36)$$

Where A and C are coefficients dependent on ξ and i but assuming Rayleigh distributed waves and γ_r , γ_b , γ_h , γ_β are reduction factors for slope roughness, geometry, deviation from the assumed Rayleigh distribution due to shallow water influences and oblique wave angle. Wave height is generally defined as the statistical (USACE, 2003) or zeroth-moment (Pullen et al., 2007; Hughes, 2005) significant wave height at the structure toe.

Studies by Mase (1985) demonstrated a relationship between bed slope and run-up characteristics, with beds at steeper slope exhibiting higher frequency run-up, correlating more closely to individual wave events, and beds at lower slope exhibiting lower frequency run-up, more closely related to wave groups and long waves. This implies that coastal structures, more generally falling into the steeper slope category, are highly dependent on maximum incident wave height.

Once the run-up elevation exceeds the crest elevation of a structure, a flow of water over the crest, or overtopping results. The volume of overtopping critically affects the integrity of the structure at and behind the crest and also safety considerations for pedestrians, vehicles and buildings in the vicinity. While a wide variety of formulae are used to predict run-up and overtopping for structures of varying type and material (i.e. USACE, 2003; CIRIA, 2007; Pullen et al., 2007), a general formula employed within the EurOtop manual (Pullen et al., 2007) is given as:

$$\frac{q}{\sqrt{gH_{m0}^3}} = a \exp(-bR_c / H_{m0}) \quad (2-37)$$

Where q is the mean discharge rate, H_{m0} the zeroth moment significant wave height at the structure toe, a and b are coefficients dependent on structure type and R_c is the freeboard between still water level and the structure crest elevation. The $(gH_{m0}^3)^{0.5}$ term on the left hand side of the equation indicates a nonlinear relationship between wave height and overtopping volume, again implying that small increases in wave height will result in relatively larger increases in overtopping volume.

Mean overtopping discharge, measured in l/s/m, is most typically used to define tolerable overtopping volumes (Table 2-2). While mean discharges may be relatively

low, the individual overtopping events that contribute may be much larger (up to 100 times - USACE, 2006) due to the inherently random nature of overtopping, both in time and in volume (Pullen et al., 2007). This, combined with the findings of Mase (1985), emphasises the importance of accurate determination of the maximum nearshore wave height.

2.5.1.2 Wave loads

The most critical loading generally placed on nearshore structures is that of wave impact. Some structures such as caisson breakwaters and cantilevered sheet piling are rigid and must withstand the design wave impact or suffer catastrophic failure through member buckling, collapse or overturning. Others, such as rubble mound breakwaters and revetments are less rigid and may absorb some wave energy through percolation movement and reshaping via displacement of individual units without failure of the structure as a whole (CERC, 1984).

In all cases, the stability of coastal structures is critically dependent on the height and breaking type of the impacting incident waves. For example, the unit mass of rock armour specified by design codes based on the Hudson formula (CIRIA, 2007) exhibits a wave height cubed dependency (Eqn. 2-38).

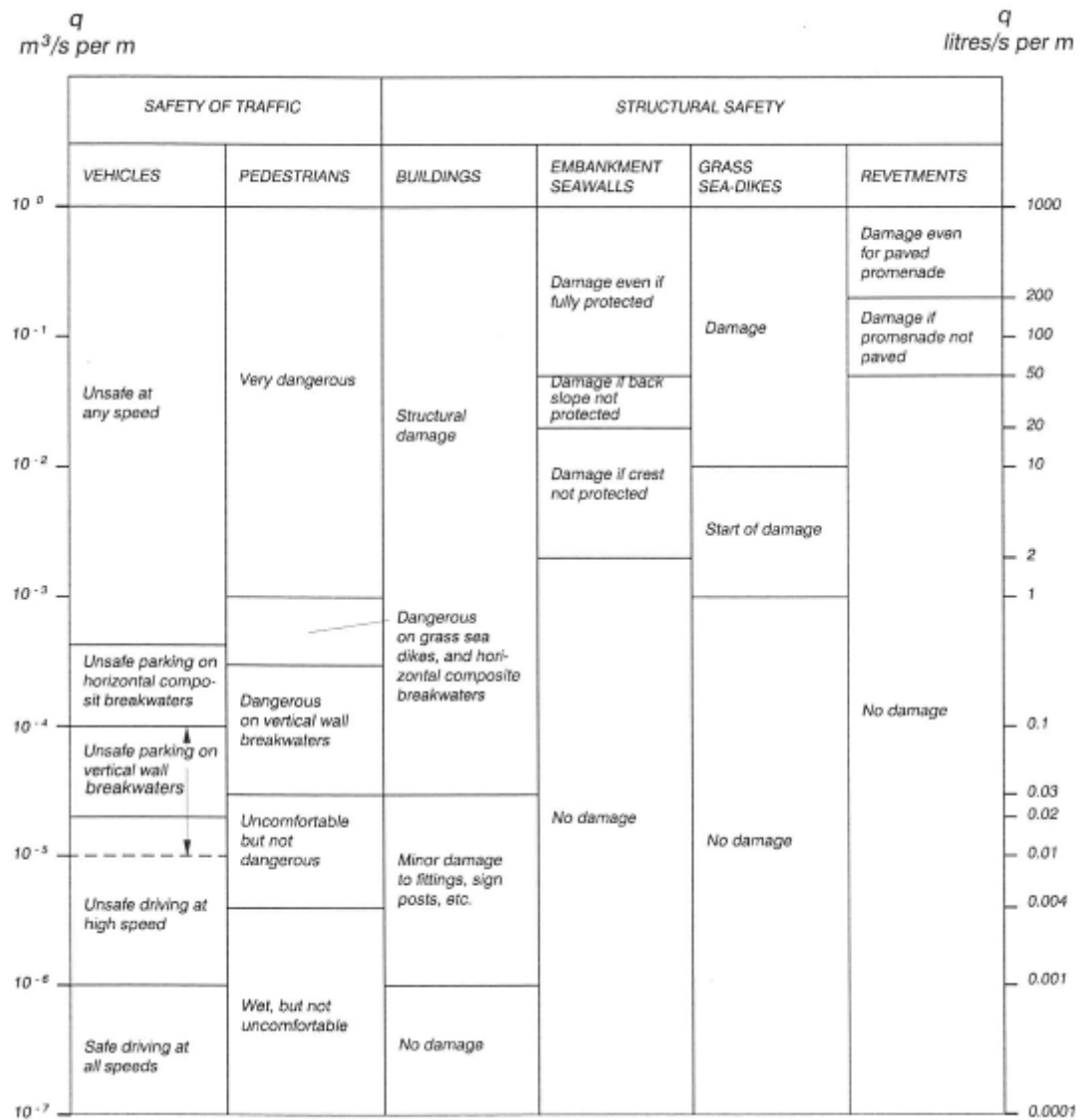
$$W_{50} = \frac{\rho_r g H^3}{K_D \Delta^3 \cot \alpha} \quad (2-38)$$

Where W_{50} is the median weight of the armour stone in Newtons, ρ_r is the apparent rock density, H is the wave height at the structure toe, K_D is a stability parameter dependent on the type of armour stone or unit employed and whether the calculation is applying to the breakwater head or trunk, Δ is the relative buoyant density of the armour stone and α is the structure slope angle. Although derived by regular wave testing, the formula is used for irregular wave design with the 1977 Shore Protection Manual (CERC, 1977) recommending the use of H_{sig} for irregular waves, although this was subsequently revised to $H_{10\%}$ within the 1984 edition (CERC, 1984). The CEM (USACE, 2003) provides stability coefficient for both H_{sig} and $H_{10\%}$.

While the Hudson formula incorporates cases of direct breaking wave impact, other approaches employ differing formula for deep and shallow water conditions (Van der Meer, 1988; Van Gent et al., 2004). Additionally, the formula of Van der Meer (1988)

and Van Gent et al. (2004) make the distinction between plunging and surging waves using a critical Irribarren number threshold, therefore recognising the difference in loading characteristics of these cases. Both the deep and shallow water formulae use the significant wave height at the structure toe and exhibit a wave height cubed dependency in determining rock armour unit mass.

Table 2-2 Critical values of average overtopping discharge (source: USACE, 2003).



2.5.2 Design wave height

Determination of a design wave height first requires establishment of whether the design environment is depth limited. A depth limited environment infers that the design wave height is determined by water depth rather than the offshore wave conditions. While this has the potential advantage of requiring design for a lesser wave height due

to energy losses occurring in larger waves breaking offshore, actively breaking waves generate greater local forces than non-breaking, surging type waves. Additionally, if little information is available on characteristic offshore wave conditions, the assumption of a depth-limited environment provides a first approximation of maximum possible wave height for desktop studies.

To calculate the maximum possible wave in a depth-limited environment, it is necessary to revert to the concept of a breaker index. This was discussed within Section 2.3.2.3 and defined in its simplest form by Eqn 2-39.

$$H_{\max} = \gamma d_b \quad (2-39)$$

Where γ is a breaker index varying between 0.78 and 1.6 (Weggel, 1972) dependent on bed slope (m) and offshore wave steepness (s_0) and d_b is the depth at break point. As discussed within the earlier Section, a multitude of differing breaker indices and criteria exist for both regular and irregular waves, yet the fundamental definitions of primary terms remain ill defined (Black and Rosenberg, 1992).

Design and assessment manuals such as the EurOtop Manual (Pullen et al., 2007) recommend the following process for determining depth-limited wave height for use in design:

1. calculate deep water wave steepness ($s_{op} = H_{so}/L_{op}$);
2. determine the local water depth (d/L_{op});
3. determine foreshore slope (m);
4. evaluate the breaker index using charts or equation (see Section 2.3.2.3);
5. calculate breaking wave height, H_b , based on the established breaker index and water depth (d).

While this process seems relatively straightforward, a number of aspects must be considered. These include:

- the appropriate definition of break point;

- the selection of appropriate wave height, i.e. breaking (H_b) or characteristic (H_{max} , H_{sig} , etc) height and the appropriate breaker index to derive that height (i.e. regular or irregular based);
- the definition of depth employed to calculate wave height once the breaker index is established.

2.5.2.1 Break point position

The majority of existing literature and guidelines such as the Coastal Engineering Manual (USACE, 2003), Hughes (2005), the Rock Manual (CIRIA, 2007) and EurOtop (Pullen et al., 2007) recommend that for the purposes of evaluating wave height effective on the structure, depth should be defined at the structure toe. However, wave breaking is a transient process which, once initiated at break point, propagates shoreward. This implies that a wave impacting on a structure may be initiated some distance offshore. While this distance is likely to make little difference to the depth and consequently design wave height on gentler or lower slopes, more difference may result on steep slopes.

While excluded from the more modern literature such as the CEM (USACE, 2003), the Shore Protection Manual (CERC, 1984) discusses selection of a break point sufficiently offshore of a structure to allow for breaker travel distance (Figure 2-13). Initially, Hedar (1965) proposed that the breaking process extends over a distance roughly equal to one half of the shallow water wavelength and that breaking depth should therefore be evaluated at this distance from the structure.

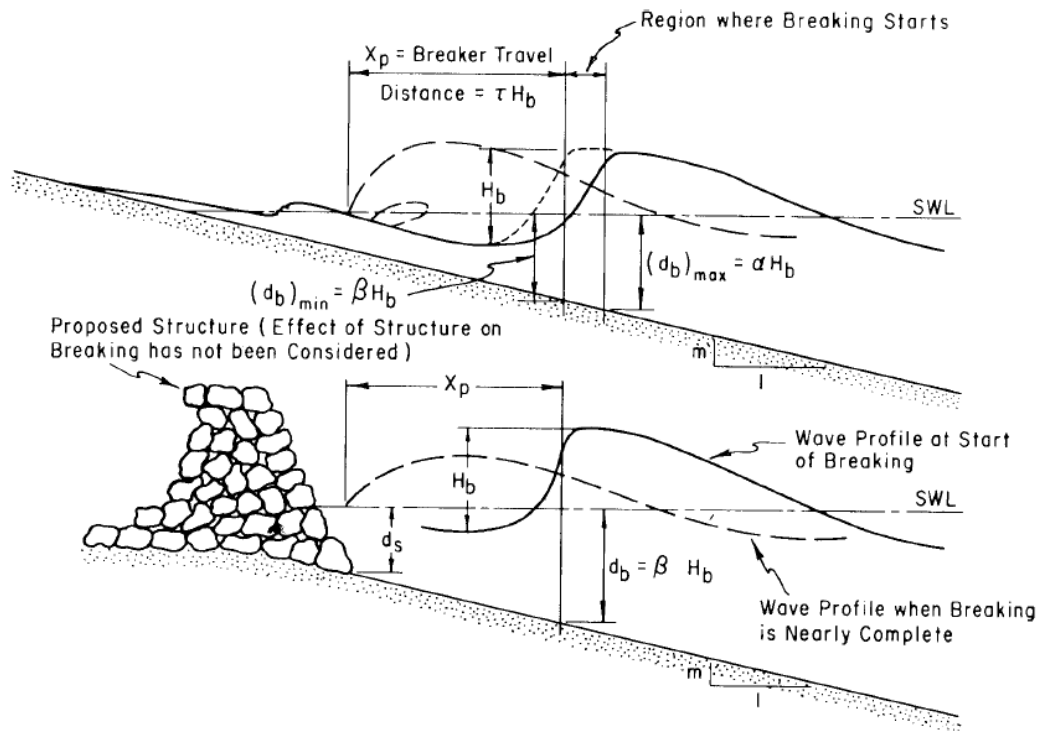


Figure 2-13 Definition of breaker geometry and travel distance (source: CERC, 1984).

Galvin (1969) subsequently suggested an expression for plunge distance (Eqn 2-20) as a function of the incident wave height and beach slope (Section 2.3.2.2). However, the definition by Galvin removed any dependence on wave period and therefore imposes no reliance on breaker type. Once a first estimate of breaking wave height is obtained, this expression may be used iteratively to refine breaking position and depth and to recalculate breaking height.

2.5.2.2 Breaker index and wave height considerations

While the derivation of the majority of breaker index data and associated formulae and charts is based on the wave height at break point (H_b) for regular waves, for design purposes irregular wave fields and characteristic or statistical heights such as maximum, $H_{10\%}$ or significant wave heights are important (Goda, 2007).

While breaker indices for regular waves typically vary with beach slope and wave steepness, a constant cross-shore value is assumed due to the singular break point nature of regular waves. However, Goda (2007) notes that, even for regular waves, scatter is inherently present within laboratory data and that a coefficient of variation (CoV) must be incorporated. This coefficient, attributed to the inherently stochastic nature of wave breaking, is measured at 6 to 7% for slopes less than 1:100, increasing to around 14%

for a slope of 10:1. This scatter is evident within Figure 1-1 and raises the question of whether upper envelope or best-fit curves should be employed within design. Smith and Kraus (1990) state that the average index from scattered data is more useful for most coastal-process studies while the maximum of the envelope is more appropriate for wave-force calculations.

The breaker indices derived for regular waves have been suggested as appropriate for deriving the highest wave in an irregular wave-train, i.e. $H_{\max(\text{irreg})}/d \approx H_{b(\text{reg})}/d$ (Goda, 1975, 2007; Seyama and Kimura, 1988; Kamphuis, 1991; Durand, 1997), but not for derivation of other characteristics waves. However, use of a regular wave-based index neglects possible cross-shore variations in breaker index due to changes in mean water level or other phenomena associated with irregular waves. While this may be partially addressed by independently assessing the contribution of set-up to water level at the design location, little guidance is provided within the standard texts (i.e. USACE, 2003; CIRIA, 2007).

Goda (1975) proposed that his earlier criterion for regular wave breaking (Eqn. 2-24) could be employed for determining significant wave height in irregular wave breaking with the coefficient A taken at 0.12 (c.f. 0.17 for regular waves) at incipient breaking. This implies that the breaker index for significant waves is around 30% lower at incipient breaking than for regular waves. This finding was supported by similar results by Seyama and Kimura (1988) and Kamphuis (1991). An increase in breaker indices with decreasing depth across the surf zone was however, noted by several researchers (Goda, 2007). Goda reanalysed these earlier experiments, assigning ranges of the coefficient A to each. A summary of these experiments and the cross-shore variation in the coefficient A is presented within Table 2-3.

Table 2-3 Summary of laboratory data sets of random wave breaking (source: Goda, 2007).

Authors	Beach slope	Nos. data	$T_{1/3}(\text{s})$	$H_0'(\text{cm})$	Depth h (cm)	Value of A
Tomi-Hashi (1972)	1/70	5	1.5	11.5	20.0 - 4.3	0.13 – 0.17
Goda (1975)	1/50	37	1.0 - 1.9	8.5 - 12.1	25.0 - 6.0	0.11 – 0.17
Ting (2001, 2002)	1/35	12	1.7, 2.0	14.2	27.0 - 6.3	0.12 – 0.16
Battjes (1972)	1/20	14	1.3, 2.2	12	15.9 - 4.1	0.08 – 0.2
Ting (2001, 2002)	1/35	12	1.7, 2.0	14.2	27.0 - 6.3	0.09 – 0.12

Larger values of A occurred within the surf zone suggesting that relatively larger waves may occur in shallower depths within the surf zone than outside. Values of A can be observed to approach (or exceed for Battjes, 1972, on the steepest slope) the value of $A = 0.17$ assumed for regular waves. As the irregular wave index relates to significant wave height, larger waves would be expected although no data is presented on the maximum wave height distribution across the surf zone. These larger waves would therefore exceed those predicted by the regular wave index. This is inconsistent with the assumption of the regular wave-breaking criterion providing an ‘upper envelope’ for irregular wave breaking. However, as no reference to the definition of water depth is provided, the increase noted in the irregular wave index could be a consequence of elevated water level due to set-up within the surf zone if breaker indices are being evaluated with respect to still water level. However, field studies, as reported within Goda (2007), found a similar increase in the coefficient A , with decreasing depth defined according to mean water level. This indicates that despite an increase in mean water level due to set-up, the breaker index exhibits a systematic increase with decreasing depth.

Observed variability in regular wave breaking is also apparent for irregular wave systems. Seyama and Kimura (1988), analysing individual irregular wave breaking for slopes between 1:10 and 1:50 found CoV between 18% and 23%. This was reduced to between 8% and 11% by introducing an artificial water depth based on the mean depth plus half the difference between crest and trough amplitude (the effect of alternative water level and depth definitions is further discussed in Section 2.5.2.3). Goda (1975) incorporated an inherent variability in his breaker index by assigning a linearly varying probability between 0 and 1 of the coefficient A varying between 0.12 and 0.18 at any location. He does not, however, allow for an increasing coefficient with decreasing water depth.

2.5.2.3 Breaker depth

While the maximum wave height prior to breaking may be measured relatively accurately as the distance from trough to crest, the depth at break point is subject to interpretation. Black and Rosenberg (1992) discuss the influence various definitions of water depth at break point may have on breaking ratios. These definitions are illustrated within Figure 2-14 and include;

- a. depth according to still water level without waves (d_{swl})
- b. the mean depth over the entire recorded time series (d_{mwl})
- c. the mean depth over the cycle of the wave being considered (d_w)
- d. the water depth below the trough of the wave being considered (d_t)
- e. the depth as defined by Seyama and Kimura (1988) as the mean depth plus half the difference between crest and trough amplitude (d_s).

Black and Rosenberg found that the effectiveness of various established breaking criteria depends on the definition of water depth used. The authors felt that there was a need for a consistent definition of the water depth.

The depth values using definitions of (a), (b) and (c) are similar, particularly if regular waves are considered and all waves are breaking in a single breaker line, with only a small variation in (b) and (c) likely due to set down around the break point. The depth according to (d) will be somewhat smaller, and the depth according to (e) will be larger.

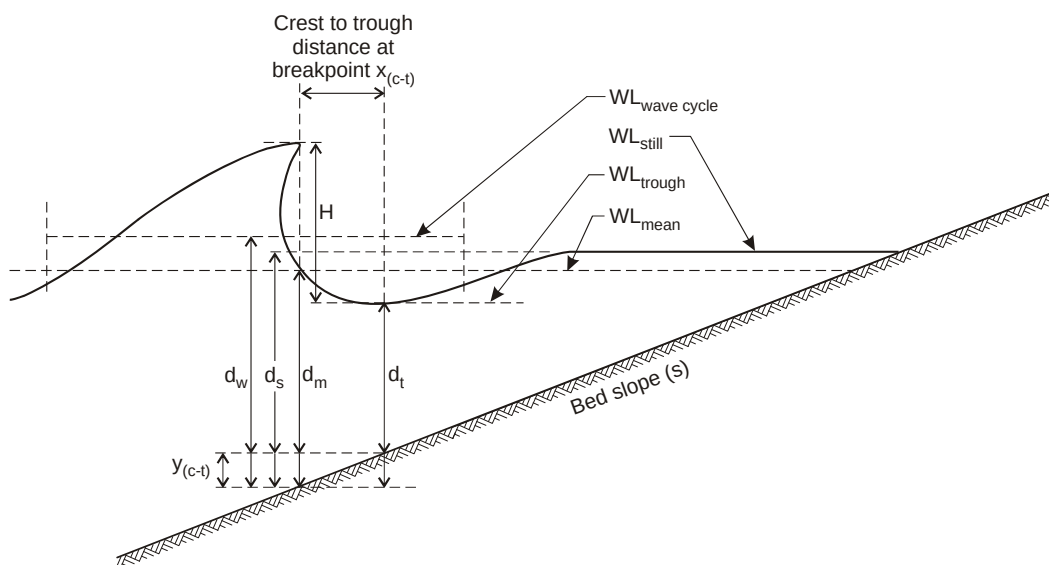


Figure 2-14 Example of a multitude of valid definitions of water level (WL) and depth (d) compared to wave height (H). The difference between the depth below the wave trough and crest is illustrated by y_{c-t} .

When irregular waves are considered, however, depths as determined by the various definitions becomes much less robust. While the water depth according to (a) varies only with the bed position relative to still water, regardless of wave conditions, definition (b) is subject to changes in mean water level due to set-up within the surf zone. Depths calculated according to (b) therefore become progressively relatively

larger than those calculated by (a). Temporal fluctuations in water level, such those associated with long waves and group effects, will not however be resolved using either definitions (a) or (b). Definitions (c) and (d) are better able to account for these longer period fluctuations however, there are very few direct measurements of these definitions and, at present, they are not suitable for engineering design. Furthermore, definition (c) is highly dependent on whether zero-up or down-crossing is used to define individual wave cycles.

Breaker index depends critically on the definition of depth. Care therefore needs to be taken by the practitioner when using breaker index guidelines that the original method of deriving depth and breaker indices is known. However, this is difficult, as these guidelines may be based on a combination of earlier investigations and are often unclear in their definition of depth. Regular wave testing generally uses a depth based on still water level to derive breaker indices (Rattanapitikon and Shibayama, 2000). This is expected due the small difference that set down makes at break point and the singular nature of the calculated index value. However, if a regular wave index, based on d_{swl} is employed to determine the upper limit H_{max} for an irregular wave field within the surf zone and set-up is not incorporated, the depth and therefore maximum wave height will be underestimated, particularly far within the surf zone where set-up includes become significant. Furthermore, certain definitions of depth such as (c) and (d) are unavailable to the designer, with a simple definition such as (a) preferred or, if required, set-up may be incorporated to derive (b).

2.5.3 Group effects

While the effect of wave groups on structural design and stability has been the subject of limited investigation, results have typically been inconsistent and there are relatively few guidelines available to assist the practitioner in quantifying group effects. While Johnson et al. (1978) found groups with long runs of high waves to be more damaging to wave exposed structures (Medina et al., 1990), experiments by Burcharth (1979) found waves of short run length to be more damaging. And while Bruun (1985) strongly emphasised the influence of wave groupiness on armour damage (in Medina et al., 1994), van der Meer (1988) found little effect and thus did not incorporate the effects of groupiness in subsequent design formulae.

Medina et al. (1994) undertook experiments at a large-scale wave flume ($104.0 \times 3.7 \times 4.6$ m) investigating the relationship of the groupiness characteristics (refer Section 2.2.2.) of spectral shape and envelope exceedance coefficient to armour damage. Testing incorporated random wave fields of identical significant wave height and peak period but varying groupiness characteristics. While correlation between armour damage levels and the envelope exceedance coefficient and groupiness factor (GF, as defined by Eqn. 2-39) was found, no such correlation was noted between damage and the spectral peakedness or mean run length of the wave field. The study concludes that the effects of groupiness on armour damage may be of the same significance as storm duration, which defined the number of waves impacting a structure during design conditions, and so should be considered in both preliminary designs and analysis of experimental data for armour damage.

$$GF = \frac{\sigma[H^2(t)]}{8m_0} \quad (2-40)$$

Where σ is the surface elevation variance and m_0 is the zeroth-order spectral moment or rms wave height. Galland and Manoha (1991) found a similar relationship between the groupiness factor of irregular wave-trains and damage to breakwater armour stone.

As described within Chapter 1, The Coastal Engineering Manual (USACE, 2003) suggests that higher waves associated with wave groups are generally thought responsible for armour layer damage and are of significant concern. While practitioners are recommended to be aware of groupiness and its implications, there is little guidance and few practical formulae available, although numerical and physical simulations are recommended. The use of physical modelling, where the effects of groupiness are explicitly included, remain prudent. The use of numerical models, where accurate simulation of the highly nonlinear and complex intra-wave group and breaking processes remain problematic, is questionable as a basis for detailed analysis or design.

While explanations for groupiness being responsible for armour layer damage are not explicitly given within the CEM, the literature identified the following factors:

- temporal variations in water level within the surf zone (Baldock, 2004 and others);

- variations in breaker index across the wave group as a function of increasing groupiness (Svendsen, 2001) and
- nonlinear intra-group processes stabilising energy convergences and delaying the initiation of breaking (Dack et al., 2005).

2.6 Summary

This chapter has reviewed existing literature on the subjects of wave group development, wave shoaling, breaking and surf zone processes and the development and evolution of long waves. Specifically, attention has focused on the derivation and characteristics of breaker indices used in current engineering design guidelines.

In summary, the stability of coastal structures is critically dependent on the height of the impacting incident waves. For example, the unit mass of rock armour specified by design codes based on the Hudson formula (USACE, 2003) exhibits a wave height cubed dependency. Conventionally, design is based on the maximum wave height which can be achieved in water of limited depth.

Defining this maximum wave height has been the subject of several significant investigations over the past century. Critical studies have been those of Goda (1970) and Weggel (1972). These studies form the basis of modern design guidelines (Goda, 2000 and USACE, 2003) which define the breaker index according to wave steepness (height and length or period) and bed slope (see also Rattanapitikon and Shibayama, 2000). The original raw measurement data however, exhibit significant scatter, with data points evident above the recommended design curves although, in the absence of turbulent perturbations, monochromatic waves should always break consistently in the same manner and in the same location. Goda (2007) identifies an inherent variability in wave breaking and breaker index of 6 to 14% depending on bed slope with steeper slopes exhibiting greater scatter.

Additionally, there are a number of intrinsic problems associated with the application of these guidelines. These include the definitions of both the wave break point and the depth at break point (Black and Rosenberg, 1992) and, perhaps most crucially, that these investigations have tended to focus on idealised wave trains consisting of regular, monochromatic waves therefore neglected possible group or irregular wave effects.

Effects of irregular waves are discussed by Goda (2007), with the indices for regular waves suggested as an upper limit or $H_b/d \approx H_{\max}/d$.

Monochromatic water waves are inherently unstable due to sideband frequency instabilities and wave trains must, over time and space, form groups (Benjamin and Feir, 1967; Hwung and Chiang, 2005). Recent studies have highlighted the role of wave groups in deepwater wave breaking due to convergences in wave energy density (Rapp and Melville, 1990; Song and Banner, 2002; Banner and Peirson, 2007). These convergences are inherent to the groups themselves rather than due to depth or subsurface current effects. Detailed work on the changes in these intra-group convergence processes as shallow water is encountered and their interaction with shoaling-induced energy convergences leading to breaking has not, to the author's knowledge, been undertaken previously.

Wave groups are also known to induce low-frequency waves as they propagate due to variations in radiation stress (Longuet-Higgins and Stewart, 1962). These waves, bound to the group in deep water, increase in amplitude during shoaling until being released at break point propagating as free long waves (Ruessink, 1998). Long waves within the surf zone may also be generated by alternative mechanisms such as a variable break point and persistence of groupiness within the surf zone as described by Ruessink (1998), Baldock et al. (1997), Baldock (2004).

The static elevation of water level, or set-up within the surf zone, is well discussed within the literature (Longuet-Higgins and Stewart, 1962, 1963, 1964) and its effect on water depth, breaker indices and the resultant breaking wave heights may be resolved. However, the time varying water-level fluctuations associated with group induced long waves propagating through the surf zone and their effect on engineering design and breaking wave heights is less well described.

Limited investigation of the influence of wave groupiness on breaking properties in shallow water has shown individual waves to exhibit differing indices dependent on their position within the group (Svendsen, 2001) and for spatial phasing of group energy convergences relative to the initiation of shoaling to influence breaking position, height and breaker indices (Dack et al., 2005, Peirson et al., 2007 and Shand et al., 2007).

Using idealised group spectra and judicious selection of the spatial phasing of group evolution relative to shoaling, breaker indices up to 10%, 27% and 35% higher than design guidelines were observed on horizontal, 10:1 and 5:1 sloped beds respectively.

The effects of groupiness on engineering design are recognised, with the Coastal Engineering Manual (USACE, 2003) stating that wave grouping and its consequences is of significant concern. However, there is little guidance and few practical formulae for use in practical engineering. The manual suggests that practitioners should be aware of the existence of groupiness and evaluate its importance in situations known to be sensitive to group-related phenomena, although procedures for this evaluation are stated as beyond the scope of the Manual.

Chapter 3 Experimental overview

3.1 Introduction and experimental aims

Laboratory based investigation of wave breaking enables the carefully controlled examination of complex physical processes. Such controlled examinations are not possible in field studies and beyond the capabilities of modern numerical models. Whilst scale limitations on physical modelling must be considered, judicious selection of scaling dimensions with respect to the physical aspects being examined allows experimental integrity to be maintained (refer Section 5.4.5).

The aims of this experimental study include:

1. observation and measurement of the deep water propagation and evolution of several different wave group types. These groups range in their generation mechanism, initial steepness and number of waves in the group;
2. observing the shoaling, breaking and surf zone characteristics of both monochromatic and group waves on plane slopes of varying steepness;
3. observing the variation in shoaling, breaking and surf zone processes as the spatial phase of group evolution (refer aim 1.) relative to the initiation of shoaling (refer aim 2.) is incrementally adjusted;
4. comparing breaker indices obtained during the present experimental study to those presented in existing literature and design guidelines such as the Coastal Engineering Manual (USACE, 2003, II-4-5) and Goda (2007).

A review and critique of previously used methods for surface detection within the laboratory is presented within this chapter and rationale for the development and employment of a relatively new technique of data collection and visualisation using digital video and laser induced fluorescence, referred hereafter as optical LIF is provided. This chapter then details the experimental facilities and data acquisition systems used to facilitate the present study.

3.2 Review of laboratory wave measurement techniques

Laboratory measurement techniques have developed over the last 50 years, yielding new insights into shoaling wave structure, kinetics and kinematics. While surface penetrating devices remain the most popular due their simplicity and low cost (Shepherd, 1997), less intrusive, absolute measurement devices such as ultrasonic sensors and optical methods are becoming increasingly popular.

A summary of laboratory measurement techniques and associated accuracies are presented in Table 3-1 and discussed in detail below. While techniques generally fit into the categories of subsurface, surface penetrating and above-surface methods, distinction should also be made between surface elevation and surface slope measurements and between spatially discrete point measurements, continuous profile measurements and complete three-dimensional surface measurements.

Table 3-1 Summary of laboratory measurement techniques.

Detection technique	Description	Measurement type	Frequency Response	Quantified accuracy	Selected references
Subsurface					
Subsurface pressure measurement	Pressure transducers or manometers	Point measurement of surface elevation	Determined by subsurface pressure response to wave frequency	5% wave height in deep water	Folsom (1947); Seiwall (1947); Grace (1970, 1978); Bishop and Donelan (1987)
Subsurface acoustic measurement	An upwardly directed acoustic pulse is reflected off the upper surface	Point measurement of surface elevation	Determined by digitisation frequency of signal output	N/A	Chatley (1939)
Surface penetrating					
Float-type systems	Buoy with vertical position recorded using transducers or integration of accelerometer output.	Point measurement of surface elevation	Determined by interaction between float, surface and tethering system.	N/A	Longuet-Higgins (1982)
Capacitance /resistance gauge	Surface piercing dielectric-coated wire or rod. Output voltage designed to vary linearly with immersion depth.	Point measurement of surface elevation	Determined by digitisation frequency of signal output. Meniscus effect unquantified.	0.4% of range (0.8 mm over 200 mm probe) given by manufacturers	Kinsman (1965); Popil and Curzon (1979); RBR (2007)
Above surface					
Acoustic transducers	Sensor emits an acoustic pulse and detects the time delay of the reflected signal	Point measurement of surface elevation (although obtained over a discrete area)	Determined by digitisation frequency of signal output	0.18 mm given by manufacturers	Gillespie et al. (1982); Wang et al. (1991); Microsonic (2007)
Optical (slope measurement)	Slope is measured using light reflection and photomultipliers	Either profile or surface measurement of slope	Dependent on camera imaging rate and frequency of optical signal	Resolution dependent on optics used	Cox and Munk (1954); Jahne and Riemer (1990); Barter et al. (1993)

Detection technique	Description	Measurement type	Frequency Response	Quantified accuracy	Selected references
Optical (sidewall observed)	A digital camera records the air-water interface against a glass flume sidewall	Profile measurement of surface elevation and slope along the glass sidewall	Dependent on camera imaging rate	Estimated at between 1.7 and 6.7% of wave height or around 1 mm	Govender et al. (1996, 2002); She and Canning (2007)
Optical (particle imaging)	Flow is seeded with fluorescent particles then illuminated with a laser sheet. Digital camera records position of particles and air-water interface inferred by lack of particles.	Profile measurement of particle position (interpreted as surface elevation)	Dependent on laser pulse frequency and camera imaging rate	Given at ± 0.1 mm (in deep water). RMS difference from resistance probes 10% in deep water, increasing to 60% during and immediately following breaking	Hannan et al. (1996); Okamoto et al. (1996); Peirson (1997); Misra et al. (2006); Mukto et al. (2007); Kimmoun and Branger (2007)
Optical (laser sheet/fluorescent dye)	Flow is seeded with a fluorescent dye then illuminated with a laser sheet. Digital camera records image and air-water interface detected as break in fluorescence.	Profile measurement of surface elevation and slope	Dependent on laser pulse frequency and camera imaging rate	Resolution given at ± 0.1 mm. No information on errors related to oblique view or lens distortions	Bonmarin et al. (1985); Perlin et al. (1993); Duncan et al. (1999); Law et al. (1999); Sue et al. (2006 and 2007)

3.2.1 Subsurface methods

The use of observed subsurface pressure to derive water surface elevation has been a subject of development since the late 1940's. Relating observed pressure to an absolute depth below water surface in the presence of waves is dependent on the theory employed, with both the hydrostatic and dynamic components of wave propagation contributing. Initial work by Folsom (1947) and Seiwall (1947) was continued by Homma et al. (1966), Esteva and Harris (1970), Grace (1970, 1978), Nielsen et al. (1988) and Nielsen and Dunn (1998). Bishop and Donelan (1987) summarise previous work and state that using linear theory, wave heights within 5% of absolute are achievable using pressure transducers. From a laboratory perspective, this represents a relatively large discrepancy and alternative methods became favoured. Additionally, the complexity of the vertical velocity structure under breaking waves makes these techniques unsuitable within the active breaking zone.

The use of acoustic devices directed in an upward direction from below the surface is described briefly within Shepherd (1997). Early attempts by Chatley (1939) used an upward directed echo sounder and were successful in obtaining static measurements of depth, although information on resolution and accuracy is unavailable. Their use is limited by acoustic beam angles and the need to obtain adequate reflection. While there is little record of their more recent use, the method is likely to encounter problems with detection of steep water profiles.

3.2.2 Surface penetrating methods

Capacitance and resistance type wave probes were developed largely between 1950 and 1980 with Campbell (1953), Tucker and Charnock (1954), Whittenbury et al. (1956, 1959) and Popil and Curzon (1979) all developing variations of the probes. The general principle of the instruments is that a dielectric-coated wire or rod is submerged within a flow. Voltage potential is measured, varying linearly as a function of immersion depth. This voltage is then converted to a water level using calibrated parameters.

Accuracy of these instruments is largely a function of signal noise, wire width and coating properties and calibration parameters. Inconsistencies or damage to coating along the wire length can result in the generation of high frequency components within

the true signal as can noise generated by the electrical circuitry (Whittenbury et al., 1959). Lange et al. (1982) found spatial resolution of the probe to be a function of the effective diameter of the wire, which governs meniscus structure. The 0.075 mm diameter wire used within their experiments was found to have a meniscus of 2.15 mm diameter for clean tap water (surface tension around 0.0735 Nm^{-1} at 15°C). Additionally, since calibration is generally undertaken during static conditions, the influence of a dynamic meniscus affects absolute wave height accuracy with the meniscus observed to reverse direction during the rising and falling phases of oscillation (Kinsman, 1965). Kinsman (1965) concluded that his capacitance gauges typically gave wave values *a few millimetres* too small and also introduced fictitious bumps on the front wave face resulting in spurious harmonics within a Fourier analysis of the data. Some capacitance gauges are however, capable of sub-millimetre measurements (Caulliez et al., 1998).

The probes are also sensitive to temperature and chemical composition of the fluid, with changes compromising the initial calibration parameters. Similarly, the measurement of flow within breaking regions is unreliable due to aeration and rapidly evolving flow fields. Finally, the probes are limited in shallow environments as they must maintain a sufficient portion of wire within the flow at all measurement times. Most significant, however, is that the probes provide a spatially discrete point measurement of surface elevation rather than surface profile descriptions. This may be partially addressed using series or arrays of probes, all of which then require calibration and are subject to the same limitations as outlined above.

Despite these limitations, capacitance and resistance wire gauges remain popular devices for measurement of water surface within laboratory environments due to their low cost, ease of calibration and use, high temporal resolution and generally sufficient accuracy (Shepherd, 1997). Manufacturers presently quote accuracies of 0.4% and linearity of 0.2% on their devices (RBR, 2007).

3.2.3 Above-surface methods

Above-surface devices such as acoustic sensors have become more popular for the measurement of surface elevation as they overcome a number of the deficiencies associated with the capacitance and resistance type gauges. Acoustic sensors operate by

emitting an acoustic pulse in the high frequency range ($> \sim 25\text{kHz}$) and detecting the reflected signal. While these instruments have their origins in the early 20th century with sonar being developed to detect solid objects within the ocean, devices were produced specifically to detect the air-water interface, firstly by intrusive (Gillespie et al., 1982) then by non-intrusive methods (Wang et al., 1991 and others).

Following these early developments, extensive ranges of acoustic measurement devices have been produced. Being non-intrusive, they do not influence the flow and also are not sensitive to the temperature and chemical composition of the fluid. However, measurement is undertaken over a finite detection area, with size depending on the distance from instrument to target. This detection area may constitute a significant portion of the wave-form for shorter wavelength spectra and appears to result in general under measurement of steep wave heights (refer Section 5.2). Additionally, high angle reflective surfaces, as frequently occur with steep wave-forms, have been observed to be problematic as can measurements within the aerated surf zone. These limitations significantly compromise the use of these types of probes in critical shallow water applications. Manufacturers presently quote resolution of 0.18 mm and accuracy of 2% for their acoustic devices intended for laboratory use (Microsonic, 2007).

3.2.3.1 Slope-based optical methods

Optical methods for determining water surface properties have their origins in field observation, with Hulbert (1934) using reflection by sunlight to assess maximum wave slope on a sea surface. Cox and Munk (1954) used aerial photography and similar sun-glint methods to gather data over large spatial areas and Schooley (1956) furthered these methods by employing flashbulbs as a light source. Extracting data from these photography based techniques was, however, extremely time and labour intensive. The techniques were further adapted by Wu et al. (1969, 1977), to determine surface slope and surface curvature distribution of wind waves within a laboratory flume. The use of a photomultiplier to measure the relative frequencies of various water surface slopes enabled much more rapid and effective data collection and analysis. While these methods may be coupled with other techniques for determining water surface elevation (such as capacitance wire probes) to produce comprehensive datasets, they cannot produce this water surface elevation data independently.

The use of wave refraction rather than reflection to determine water surface slopes was originally developed by Cox (1958) whereby a beam of light, produced by an incandescent filament source, was projected through the water from below. The refraction of light at the interface due to surface slope was measured using a photomultiplier, thereby giving a surface slope. Sturtevant (1966) used a similar experimental set-up but measured light attenuation as opposed to refracted light direction. By assessing the absorption of certain light wavelengths (particularly in the near infrared), depth was determined using a nonlinear depth-absorption relationship. While this technique allowed for relatively low water slopes, the depth beneath slopes greater than 49° were undetectable. Additionally, the ability of the technique in the aerated surf-zone is not discussed.

Tober et al. (1973) and Sorrell and Withers (1976) continued after Cox (1958) but, using improved optics, geometry and, in the case of Tober et al., a helium-neon (He-Ne) laser, obtained much greater accuracies and slopes in both the along- and cross-flume directions. This improvement in accuracy and measurable wavelength made the technique much better suited to the measurement of random wave fields such as generated by wind. This technique was continued by Jahne and Riemer (1990) using a submerged illumination system and an above-flume mounted CCD camera which detected intensity gradients along or across the flume. Using calibration images, wave slope was defined to along-wind and cross-wind resolution of 1.30 and 1.85 mm respectively and two-dimensional wave slope spectra obtained.

Lange et al. (1982) meanwhile, orientating a laser vertically downwards onto the water surface as opposed to from below. This had a number of advantages in keeping the area directly above the water surface free from obstacle and also allowing measurement of much larger slope angles. Lange et al. compared power spectra extracted from the optical laser technique with the spectra obtained from a wire resistance type gauge with good agreement up to 80 Hz. Finally, Barter et al. (1993) combined the refractive techniques of Tober et al. (1973), Jahne and Riemer (1990) and others with the intensity attenuation method of Sturtevant (1966) to give a method which allows simultaneous measurement of surface slopes and wave amplitude.

3.2.3.2 General elevation-based optical methods

With the availability of high resolution and affordable digital CCD cameras, new techniques of observation were developed making use of their ability to observe over much larger spatial domains and rapidly collect large volumes of data. Jahne and Waas (1989) used two CCD cameras and the earlier reflection techniques of Munk (1954) and Schooley (1956) to record stereo image sequences of the specular reflections off a water surface. Using a processing algorithm, wave height is calculated from the parallax of the two images. Height resolution of 6 mm was achieved on a laboratory spectral surface with underlying 0.9 m gravity waves and 8 m/s wind speed. Resultant variance of 42 mm² was slightly below that recorded by wire gauges of 46 mm².

Mulsow et al. (2006) used a similar reflection technique to reflect a laser light sheet off the water surface normal to the wave crest and on to a projection plane. A digital camera then recorded the projection image and the water surface was inferred through calibration. Static calibration yielded accuracies of around 0.2 mm. However, accuracy in recording actual wave forms is not presented. While this technique appears useful in large basin configurations, the method has several limitations for high accuracy and surf-zone application. The method would not cope with waves of very high steepness or within the aerated, broken zone as reflective properties are not constant. Similarly, across-plane slopes in the water surface would yield error.

Govender et al. (1996, 2002) made measurements of water level and the wave roller structure of breaking waves on a 20:1 laboratory slope by extracting time series from video images at given horizontal positions. A camera was installed at the still water level looking horizontally through the glass flume-wall and the flow surface illuminated from above, presumably with a white light sheet or source. A single pixel width of the video image was extracted from each frame and the resulting samples stacked side by side forming a new image coined a *keogram*. A black backing sheet within the flume provided an interface between the water surface and air which was detected giving a time series. Although data on temporal and spatial resolution and accuracies are not presented, results appear qualitatively reasonable. She and Canning (2007) used a similar, horizontally positioned video method to record a wide length of flume. The wave surface profile at the glass sidewall was extracted using a thresholding operation

and edge detection algorithm. Geometric properties including wave height and length at break point and depth were collected. Resolution is relatively coarse due to the large area of coverage and stated as 1 to 2 mm per pixel. Error is estimated at between 1.7 and 6.7% of wave height or up to around 1 mm, but details on error quantification are not provided.

These methods, however, are restricted in several aspects. By viewing the flow field from the side, the actual surface position becomes obscured by contaminative effects on the flume sidewall. These include the effect of dynamic meniscus on the glass sidewall obscuring view of the actual wavy surface, as well as boundary layer effects that can alter flow around the sidewall from that at the flume centre. In small scale experiments these discrepancies can be substantial. Finally, the use of uniform white light does not easily facilitate sharp images and well-defined interfaces owing to the rapid movement of the target surfaces.

3.2.3.3 Light sheet profiling techniques

The use of pulsed light velocimetry (PLV) as a method of flow visualisation has become increasingly popular in the laboratory study of wave processes due to its non-intrusive and versatile nature. PLV requires a flow to be seeded with markers which are illuminated and tracked over time. These markers may either be particulate markers producing Particle Image Velocimetry (PIV) images or fluorescent molecular markers producing Laser Induced Fluorescence (LIF) images. Merzkirch (1987), Gad-el-Hak (1989) and Adrian (1991) provide comprehensive descriptions and reviews of the origins and principles of these methods and Misra et al. (2006) provides a more recent review of its use for determining air-water interfaces.

Early studies by Bonmarin and Ramamonjjarisoa (1985) examining breaking waves in deep water used a powerful light source and lens arrangement to generate an along-flume light sheet. The flow was seeded with fluorescein, thus producing an illuminated profile once the agent was excited. A high speed camera captured the profile and manual digitization allowed quantification of the profile evolution through time. However, a discussion of calibration techniques, accuracies and errors associated with the method was neglected. Subsequent studies by Perlin et al. (1993) investigated capillary waves generated by steep gravity waves using an Argon-ion laser and digital

camera. A series of two cylindrical, convex lenses were used to exaggerate the image 7 to 8 times in the vertical direction, thus allowing more precise measurement of low-amplitude waves even when using a relatively coarse resolution camera. Resolutions achieved were around 0.4 mm/pixel horizontally and 0.05 mm/pixel vertically.

Duncan et al. (1999) examined the crest profile evolution of gentle spilling breakers within deep water using a light sheet produced by an Argon-ion laser, cylindrical optics and fluorescein seeded flow. Images captured with a high speed camera were digitized and the air-water interface detected using a computer-based analysis technique. The laser and camera units were mounted and traversed the flume at a similar speed to the wave of interest, thus producing a near stationary view of wave evolution. Resolution was given at 0.1 mm and spatial accuracy of the detected interface to 1-2 pixels or 0.1 to 0.2 mm.

Using particle tracking to detect the air-water interface was first attempted using floating particles (Hassan et al., 1996 and Okamoto et al., 1996). However, the methods often resulted in discontinuities in the detected surface, even when very high particle densities were used. Furthermore, these high particle densities contaminated the free surface, altering the surface tension and the flow development (Misra et al., 2006).

Peirson (1997) used internally seeded flow and a PIV technique to measure the position of a wavy air-water interface. A multiple camera configuration was employed with one camera located below the still water level capturing subsurface velocity fields and one above capturing wave phase. Light from the seeded particles is reflected off the water surface, producing a reflected image. Provided the surface is planar, the mid-point between the particle and reflected particle represents the air-water interface. Problems exist where significant curvature or cross-tank slope of the water surface occurs. Law et al. (1999) used fluorescent dye as a molecular marker and a gradient-based method to detect the interface of a submerged, confined jet. The method employed a single, horizontally located camera with an along-tank viewing plane and two mirrors. Accuracy was compared to conventional Eulerian-based methods by comparing the root-mean-squared velocity of the interface, with uncertainty of around ± 5 mm/s found. However, no comparisons of spatial accuracies were presented.

Mukto et al. (2007) and Kimmoun and Branger (2007) determined the air-water interface in a wavy flow using an upwardly directed laser sheet, seeded flow and a horizontally positioned CCD camera. Both used a variable threshold method to detect the air-water interface. Particulate seeded flow was used owing to the reduction in intensity that the upwardly directed laser experiences when passing through a fluorescent dye and also to allow velocity vectors within the flow to be resolved. Total errors were given by Mukto et al. (2007) at around ± 0.1 mm, although measurement was limited to deep water. Kimmoun and Branger (2007) compared their derived surface profiles with those obtained by resistive wave gauges. Close agreement ($<10\%$ rms difference) was found between the probes and PIV method in deep water, becoming progressively worse (up to 60% rms difference) during and immediately following breaking, but improving through the surf zone (around 12% rms difference at the still water position).

Sue et al. (2006 and 2007) used an overhead laser sheet to fluoresce a fluid medium which is captured by a side-mounted, slightly oblique digital camera. The top edge of the observed fluorescent medium within each frame is extracted as the air-water interface using a constant intensity threshold method (Nokes, 2007). Camera resolution was slightly better than 0.5 mm/pixel and accuracies given at 0.1 mm using sub-pixel methods. Point measurements obtained during relatively slow and small oscillations are compared to data from resistance wire gauges and found to be in good agreement.

3.2.4 Summary

Commonly used instrumentation for observing water level and wave forms within a laboratory environment, including acoustic instrumentation and intrusive capacitance and resistance-type wave probes, have limited use within the breaking and surf zone and are not able to capture spatially extensive profile data. This data is required to accurately study the shoaling and breaking process. Recent advances in optical-based data collection techniques have enabled non-intrusive visualization of water surfaces. However these methods are often limited by various factors including being constrained to deep water environments, being capable of observing only slowly evolving or surface profiles of low slope, or neglecting errors induced by laboratory set-up (i.e. meniscus effects at the flume sidewalls and light or laser-sheet irregularities at the water surface)

or processing of imagery (i.e. appropriate corrections for optical and perspective distortions).

3.3 *Experimental facilities*

Laboratory experimentation for this project has been carried out in a wave flume at the University of New South Wales (UNSW) Water Research Laboratory (WRL) in Manly Vale, Sydney.

3.3.1 Wave tank

The wave tank used has maximum dimensions of 30 m in length by 0.6 m in width and 0.9 m in height. A schematic diagram of the wave tank and general experimental layout is provided within Figure 3-1. The tank has a solid bed, glass sidewalls and an open top. The glass sidewalls minimise frictional coefficients along the flume boundary and maintain excellent two-dimensionality in propagating waves. Glass sidewalls also allow better observation of wave processes.

The wave tank incorporates a synthetic absorption beach comprised of a perforated sloping bed and horsehair beach. This beach is positioned at the far end of the flume and is intended to reduce wave reflection to negligible levels. This beach has been tested in previous studies (Banner and Peirson, 2007) and was found to be effective in damping reflected low-frequency waves to less than 1 - 2 mm (1.5 to 3% of incident wave amplitude).

A dissipation tank is also incorporated behind the wave paddle. This tank, which extends 15 m in length by 0.6 m in width and 1.5 m in depth is intended to reduce energy produced during the back stroke of the paddle and to ensure that this superfluous energy does not escape into the glass testing tank. This dissipation tank contains a number of synthetic absorption cylinders. These cylinders, like the absorption beach in the glass tank, are excellent reducers of wave energy in the high to mid frequencies.

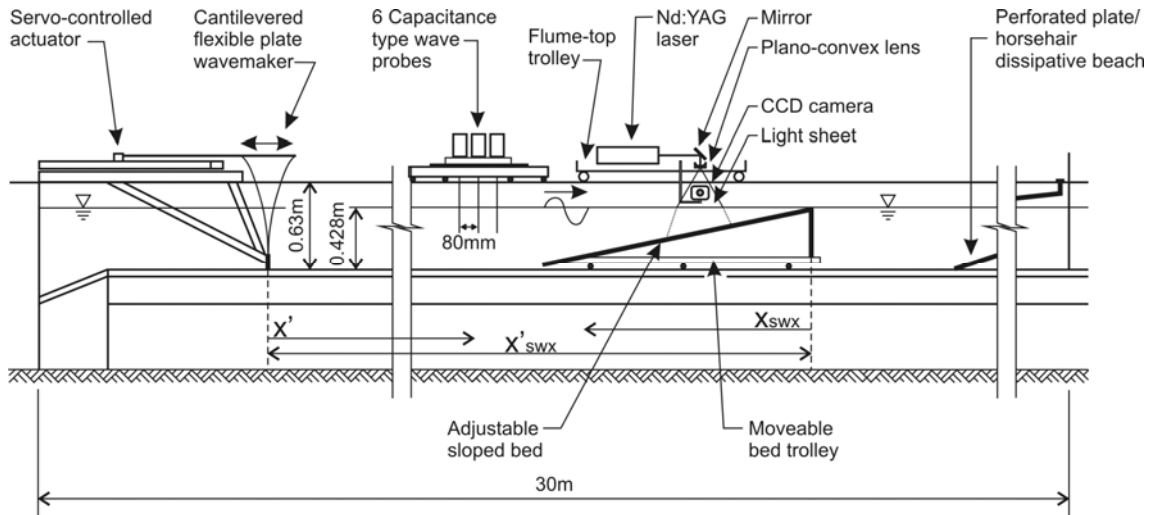


Figure 3-1 Schematic diagram of the laboratory wave tank and general experimental layout.

Water level within the flume is maintained at a depth of 428mm through use of a stilling well, circuit sensor and solenoid controlled valve. The accuracy of this method in maintaining water level is a function of surface tension, with the still water level (swl) able to drop by an elevation Δswl_{max} before contact with the circuit sensor (a steel point gauge in this case) is broken and refilling is initiated. This elevation Δswl_{max} is dependent on water properties and needle tip diameter and has been estimated during the present study at around 0.5 mm for water at 17°C.

3.3.2 Wave generation

A flexible, cantilevered plate, fixed near the base of the flume was used to generate waves (Figure 3-2). A flexible plate wavemaker induces orbital motions of water particles that decrease approximately exponentially with depth, matching those observed in deep water waves much more closely than when either piston or flap-type wavemakers are used. The paddle is driven by a servo-controlled actuator with a signal resolution of 500Hz. The actuator could be programmed from PC, with digital to analogue converter to generate monochromatic waves, wave groups or sequences of random waves by adjusting the wave height, period and phasing. The paddle is fronted by a synthetic damping mat to reduce high frequency secondary waves.

The use of a cantilevered, flexible plate excludes use of established wavemaker theory (developed for piston or flap type paddles) to predict the paddle stroke required to

generate waves of certain amplitude. A summary of linearized wavemaker theory and application is discussed within Dean and Dalrymple (1991).

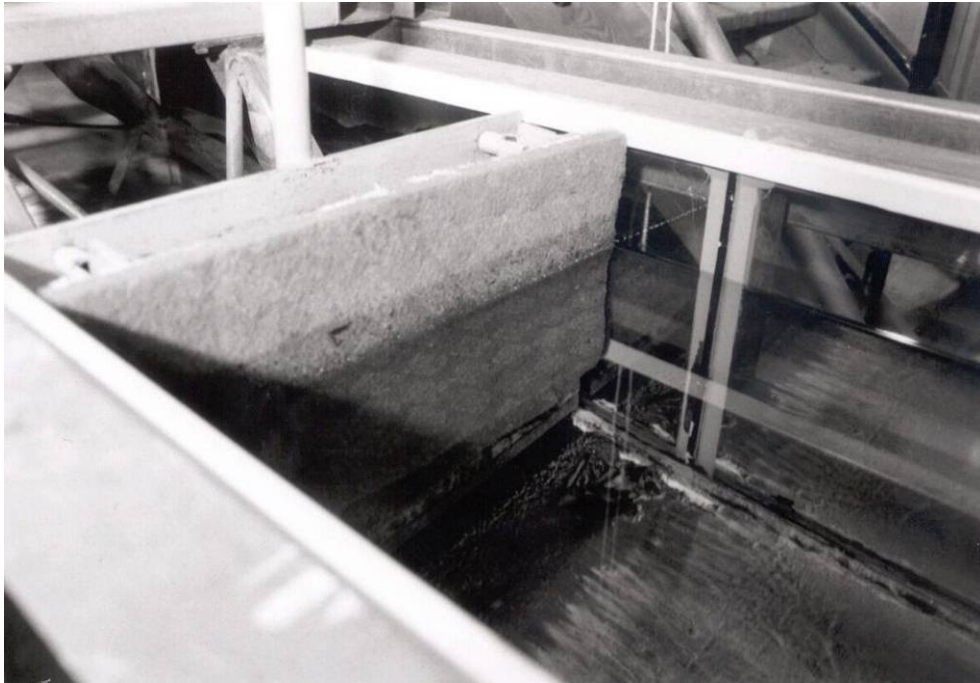


Figure 3-2 Flexible plate-type paddle fronted by a synthetic damping mat (source: Dack, 2004).

Wavemaker theory for flap-type paddles was therefore used for preliminary estimation of the required paddle stroke. Calibration of the generated signal using in-site wave probes (Section 3.3.1) was then used to refine the paddle stroke-amplitude relation. Whilst the use of a simple sinusoidal wavemaker signal could induce free secondary harmonic waves within the flume, their presence was found to be minimal under the majority of wave regimes tested during this study.

Long-term stability was tested by recording the structure of waves generated over a long period of time. Results showed that the structure of primary waves remains virtually unchanged over time. However, small amounts of background, low-frequency wave energy was observed. These background waves were also observed by Banner and Peirson (2007) with amplitudes of 1 - 2 mm (1.5 – 3% of incident wave) and were attributed to bound long-waves associated with generated wave groups. The occurrence of long waves within the flume is further discussed within Section 4.4.4.3.

3.3.3 Moveable bed

An adjustable plywood bed was constructed to facilitate wave shoaling and depth-limited breaking. The requirements for this bed were that it could be adjusted in the following ways:

- free-moving in the horizontal axis, along the wave flume;
- adjustable in slope between 5(H):1(V) and 20(H):1(V).

The first criterion was achieved through construction of submerged trolleys. These trolleys were comprised of steel angle-bar and plate and each measured 4.0 m in length by 0.6 m in width. In addition to wheels on the trolley base, wheels were attached along the trolley sides to maintain free movement along the flume side-walls.

On top of these trolleys, plywood slope boards of varying sizes were attached depending on the required slope. The boards were interchangeable and could produce slopes of 5:1, 10:1 and 20:1. On top of the slope boards, a plywood bed was attached (Figure 3-3). This bed extended from near the base of the flume to the intersection of still water level, referred forthwith as the still water intersection (swx). The sloping bed was terminated at this elevation to minimise the effects of sloshing and reflection due to wave run-up at the shoreline (refer Section 4.4.4). Although this will cause some difference from continuous impermeable plane slopes where run-up and rundown will occur, within the flume reflected waves may reflect again off the wavemaker and propagate with the incident waves as contaminative secondary waves. In this first study of wave groups it was essential that reflected and contaminative secondary waves be minimised and thus justified truncation of the sloping bed at the still water level.

Weights were added to the trolley units to minimise bed movement under wave action. Bed movement under incident waves of 0.1 m height and 0.75 s period was found to be negligible.



Figure 3-3 Sloping bed constructed on a moveable trolley within the flume.

The plywood bed had a physical roughness (ϵ) of approximately 0.6 mm (Mirajgaoker and Charlu, 1963). While the plywood bed could be moved horizontally along the flume, it was positioned sufficiently far from the wave generating paddle so as not to interfere with the generation process. It was important that the waves be generated entirely in deep water before propagating onto the slope and into shallow water. This minimised the generation of spurious secondary waves and ensured uniformity in the primary incident waves.

3.4 Data acquisition

3.4.1 Wave probes

Capacitance type wave probes were used for observing the free water surface at discrete spatial locations along the flume. The experimental set-up incorporated an array of six probes, configured in two in-line sets of three probes per line. This enabled simultaneous recordings at 3 locations, while giving the increased accuracy of two probes per location. The probes were spaced approximately 80 mm apart along the flume and 120 mm apart across the flume. The probes were located symmetrically across the longitudinal flume centreline with approximately 240 mm separating the probes from the sidewalls. The probes were placed on a flume-top trolley enabling repeat acquisition of wave data at any longitudinal location within the flume to an accuracy of approximately 5 mm.

The probes, manufactured by the Manly Hydraulics Laboratory in Sydney, were capacitance wire gauges incorporating a 200 mm looped wire ($\sim 200\ \mu\text{m}$ diameter) element suspended within a metal frame (Figure 3-4). When partially immersed in water, an analogue signal between ± 5 volts is produced corresponding to the position of the water surface. This signal is converted to a digital format and read and interpreted by a PC computer. The water surface position was recorded at a frequency of 125 Hz, corresponding to readings every 0.008 seconds. The PC computer interprets one volt of change as corresponding to a change in water level of $\sim 20\ \text{mm}$ ($\pm$ a calibration value).



Figure 3-4 Capacitance-type wave probes used to capture water surface elevation.

Probe resolution of $\pm 0.1\ \text{mm}$ was found by Banner and Peirson (2007), with linearity over the probe's 200 mm range of $\pm 1\%$. Manufacturers similarly give accuracy at 0.4% or $\pm 0.4\ \text{mm}$ over the probe's 200 mm range (RBR, 2007). Instrumentation noise of between ± 0.2 and $0.5\ \text{mm}$ was typically observed during the current study and removed through filtering as described within Section 4.2.

Wave probes were used to collect surface elevation data in depths greater than 150 mm. As such, they were generally used for deep water tests or to compliment the optical LIF data (Section 3.3.2) in deep and intermediate water depths.

3.4.2 LIF imagery

The type of data required to satisfy the aims of the present study could not be entirely achieved with conventional probes and necessitated the development of data collection and processing methods with a number of relatively stringent criteria. These criteria included: sub millimetre record of the air-water interface including of the peaks and troughs of relatively large amplitude waves; no dependence on temperature or chemical composition of the fluid; the ability to accurately record the air-water interface up to, throughout and beyond the breaking region in a spatially continuous record and to obtain the above in a time efficient manner (i.e. largely automated). A number of potential sources of error including the geometry of the laser sheet, camera horizon, optical and perspective distortion and statistical requirements for determining absolute wave height were also considered. The following sub-sections and Section 4.3 detail the method developed to fulfil these criteria.

3.4.2.1 Laser system

A TWINS Ultra-30 pulse, Nd:YAG laser system and dispersive plano-concave lens was used to produce a pulsed light sheet. The light sheet was orientated along and in the centre of the wave flume, approximately 340 mm from either glass sidewall (Figure 3-5). Due to the originally circular nature of the 3 mm diameter laser beam, the resultant light sheet was elliptical in shape. The sheet had dimensions on the still water surface of approximately 105 mm in length and 3 mm in width at its centre. The laser was pulsed at a frequency of 20 Hz, with each impulse having duration of 7 ns.

The experimental apparatus was mounted on a flume-top trolley enabling rapid and easy relocation along the flume length to an accuracy of 5 mm.

Rhodamine WT dye with peak excitation and emission wavelengths of 558 and 582 nm respectively (French and McCutcheon, 1989) was used as the fluorescent tracer within

the flume. This provided a well defined illumination of the fluid medium when excited by the laser sheet.

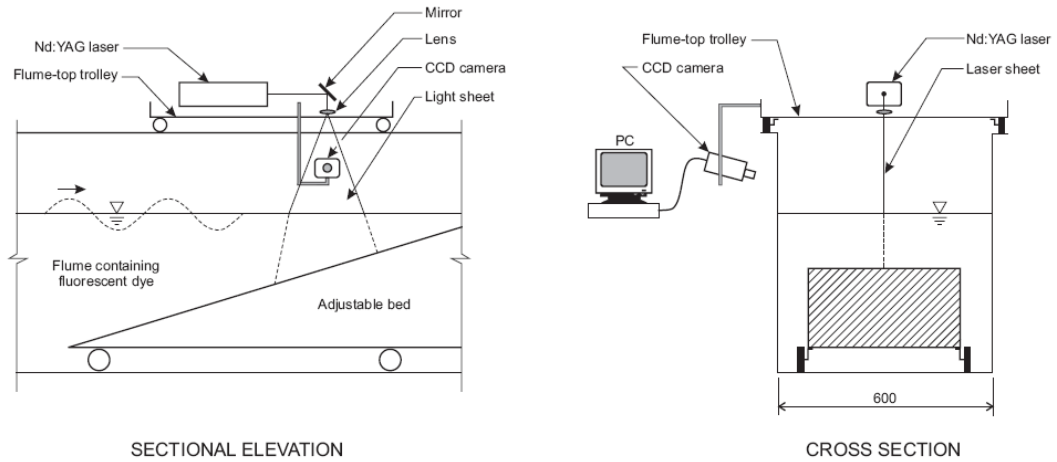


Figure 3-5 Schematic diagram of the optical LIF experimental set-up.

3.4.2.2 Image capture

Image capture was undertaken using a PixelFly 270XD CCD camera with Switar 25mm f/1.4 standard focal lens (Figure 3-6). The camera was capable of recording up to 23 fps at 696(H) by 512(V) pixel resolution or 12.5 fps at 1392(H) by 1024(V) resolution, both with a progressive scan rate of 20 MHz. Due to the maximum laser pulse rate of 20 Hz, images were recorded at 20 fps to ensure successive frames contained exactly one image. The extremely short and highly powerful laser pulse (30 mJ energy at 7 ns) enabled very clear and sharp images to be obtained (Figure 3-7). This is not possible with a continuous white light sheet. Blurring, to some degree inevitably occurs in the images due to the relatively slow camera shutter speed.

The camera was mounted to the same sliding trolley as the laser unit, and positioned outside the glass sidewall and above the level of maximum surface elevation. This ensured that the sidewall meniscus did not obscure view of the laser sheet during any phase of the wave cycle.



Figure 3-6 PixelFly 270XD CCD camera with Switar 25mm f/1.4 standard focal lens mounted to a flume-top trolley.

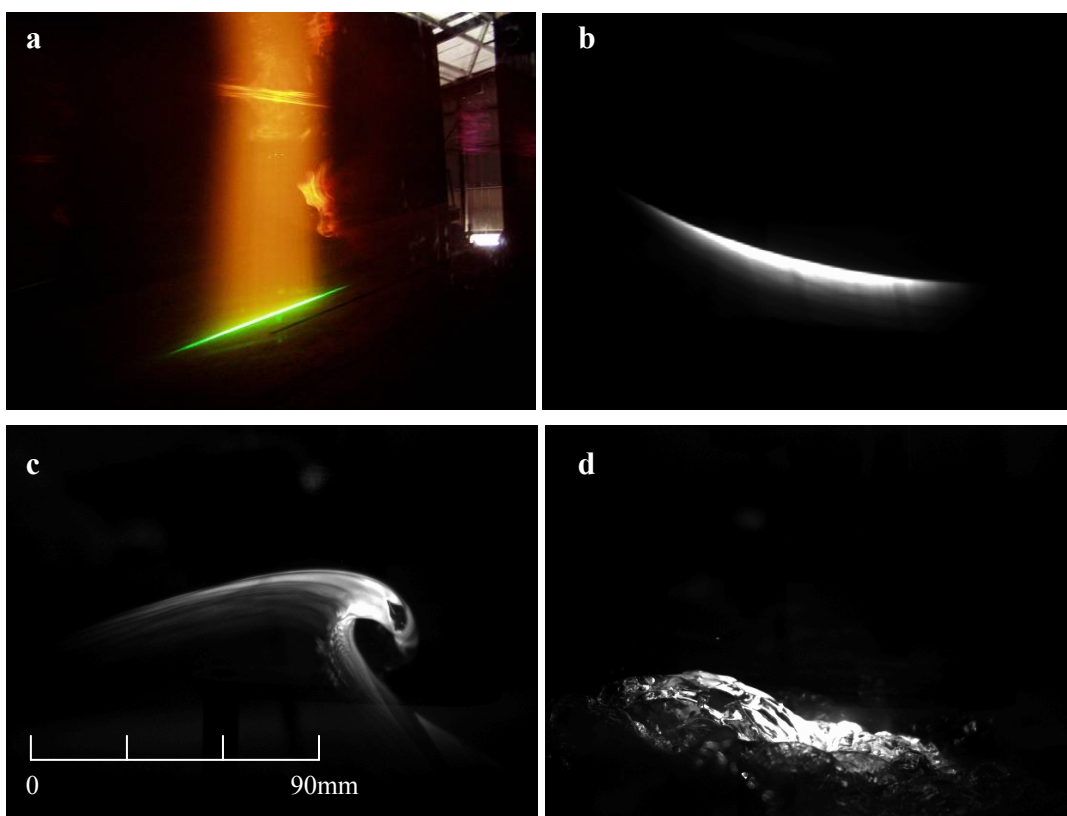


Figure 3-7 Example of the fluorescence emitted by the Rhodamine WT dye when excited with the laser (a), water surface within deep water (b), during breaking (c) and within the surf zone (d).

3.4.3 Wave breaking video

Whilst the optical LIF method enables very precise measurement of the surface elevation, the data collection process is manually time consuming and the spatial length of surface elevation examined within each video frame is small. Whilst some methods are employed to circumvent this second limitation (Section 4.3.1), additional, more expansive video is captured to complement the optical LIF data. This video gives a broad overview of the breaking process and is much more time-efficient to capture.

The Pixelfly camera as described within the previous section is also used for this image capture. The camera was positioned 1.8 m from the flume sidewall and again, a little above the still water level to ensure a lack of obstruction when viewing the surface. The flume was then illuminated with a 300 watt lamp. The viewing length along the flume centreline was approximately 700 mm. A ruler was used to provide scale and distance relative to the swx location (Figure 3-8). In cases where one or more waves broke seaward of this distance, a second video of the deeper flume section was captured.



Figure 3-8 Example of breaking wave video capture area.

3.5 *Experimental parameters*

3.5.1 Monochromatic

The laboratory program was primarily concerned with assessing the shoaling and breaking processes of wave groups. However, to preserve a control situation, and

confirm results of previous regular-wave based investigations (refer Section 2.5), a number of simulations were carried out with monochromatic wave structures.

Monochromatic waves were created by defining an initial wave amplitude and wave number. This wave number corresponds to the wavelength in deep water. At first order, the spatial structure of these simple monochromatic waves is described by Eqn. 3-1.

$$\eta = a_o \cos(k_o x) \quad (3-1)$$

Where

η = the surface water level deviation from still water level

a_o = wave amplitude

k_o = initial wave number = $2\pi/L_o$

L_o = initial wavelength = $T^2 g / 2\pi$

T = wave period

x = distance in the direction of propagation

In total, three different monochromatic wave structures were tested. These corresponded to a steep monochromatic wave, a wave of moderate steepness and a lower steepness wave. The parameters used to generate each of these wave types are presented in Table 3-2.

Table 3-2 Monochromatic wave properties.

Wave steepness	Deep water wave number (k_o)	Deep water wave amplitude, a_o (m)	Deep water wave steepness	
			$(ak)_o$	$(H/L)_o$
Low	7.4178	0.0149	0.11	0.035
Moderate	7.4178	0.0301	0.22	0.071
Steep	7.4178	0.0392	0.29	0.093

3.5.2 Wave groups

Wave groups comprise sequences of larger waves or higher energy density interspersed by smaller waves or lower energy density and occur by the modulation of a regular wave-train (Figure 3-9) as discussed within Section 2.2. Groups have an associated group wavelength (L_g) and a group velocity (c_g) different to the individual wave celerity (c).

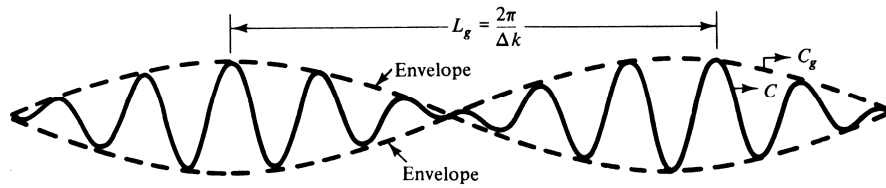


Figure 3-9 Schematic diagram of a simple wave group (source: Dean and Dalrymple , 1991).

Song and Banner (2002) describe three differing wave group classes dependent on initial geometry and generation mechanism. The first class, referred to as Case I, comprises an initial carrier wave at frequency f , and two small and symmetric sideband frequency waves. The second class, Case II, comprise two wave components of slightly differing frequency and phase. The third class, Case III, better known as a chirped wave packet (Rapp and Melville, 1990), comprises a wave sequence of increasing frequency, formed such that the low and high frequency components coalesce as the waves propagate away from their point of generation.

In deep water, the maximum and minimum ($z_{\max}$ and $z_{\min}$) elevations of the group envelope can be observed to fluctuate between alternative positions of local maxima or minima (Figure 3-10). These occur during dispersive propagation when a wave crest or trough is coincident with the group envelope maximum. These represent positions of maximum local energy density convergence. The amplitude of these local maxima and minima can be observed to increase with distance along the tank as the nonlinearity within the envelope increases until either a point of recurrence or breaking is reached, after which local maxima and minima amplitudes decrease.

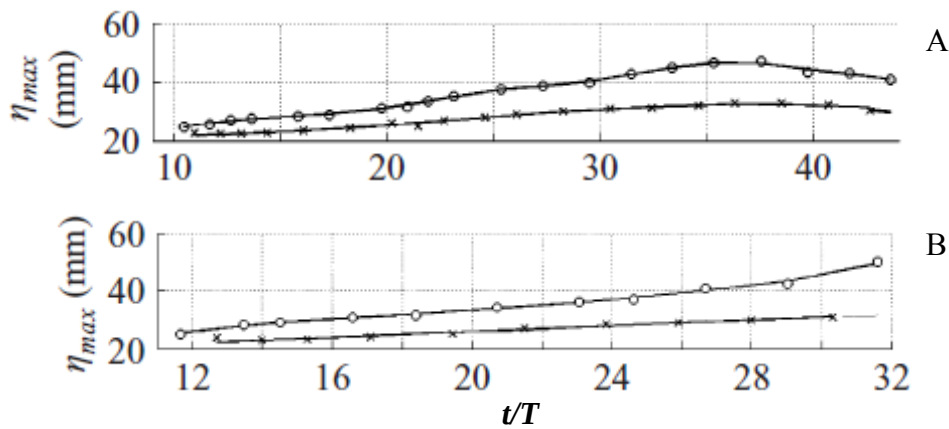


Figure 3-10 Measured group envelope maxima (o) and minima (x) of a Case II type wave for recurrent (A) and breaking (B) cases (Banner and Peirson, 2007).

Due to the relatively long evolutionary development of the Case I type waves compared to the Case II and III type and the limited flume length available, testing within this study was focussed on the latter group classes. The Case II groups had an initial spectrum as defined by Eqn. 3-2 and the Case III, or chirped type wave groups, were produced by a paddle motion as described by Eqn. 3-3.

$$\eta = a_o \cos(k_o x) + \varepsilon a_o \cos\left(\frac{N+1}{N} k_o x - \frac{\pi}{18}\right) \quad (3-2)$$

Where

η = the surface water level deviation

a_o = wave amplitude

k_o = initial wave number = $2\pi/L_o$

L_o = initial wavelength = $T^2 g / 2\pi$

T = wave period

x = distance in the direction of propagation from the wave paddle

ε = parameter to control group structure

N = an integer controlling the number of waves produced by the wave paddle

$$x_p = -0.25 A_p \left(\tanh \frac{4\omega_p t}{N\pi} + 1 \right) \left(1 - \tanh \frac{4(\omega_p t - 2N\pi)}{N\pi} \right) \sin \left[\omega_p \left(t - 0.018 t^2 / 2 \right) \right] \quad (3-3)$$

Where:

x_p = paddle position

t = time

N = number of carrier waves in group

A_p = piston amplitude

ω_p = angular frequency = $(g/(2\pi/\lambda) \tanh((2\pi/\lambda)h))^{1/2}$

λ = wavelength

h = still water depth

Four different wave group configurations were tested during this study (Table 3-3) corresponding to wave groups tested within Banner and Peirson (2007). These wave groups varied based on their generation mechanism (*Case*), the number of waves within the group (N), and the initial wave steepness (s_o) leading to either recurrent or breaking conditions. It should be noted that N relates to the number of waves which would be

observed spatially at any one time. The number of waves observed temporally at a fixed point in space differs dependent on the ratio of group velocity to wave celerity (Section 2.3) but in deep water is approximately twice the number of waves observed spatially.

Table 3-3 Four group configurations tested during this study.

Group configuration	Case	N	$s_0 = (ak)_0$	Evolutionary manner
C2N5r	2	5	0.143	Recurrent
C2N5sb	2	5	0.16	Strong Breaking ¹
C2N3r	2	3	0.168	Recurrent
C3N5	3	5	-	Recurrent

¹Refers to type of breaking in deep water (Song and Banner, 2002; Banner and Peirson, 2007)

3.5.3 Bed configuration

This study examined shoaling and breaking patterns using three different sloped beds: 5:1, 10:1 and 20:1. Four group classes and the three monochromatic structures were testing with various slopes. Testing of the wave groups was focused around the positions of maximum energy density convergence (eg Figure 3-10 [A], $t/T = 35$) to ensure maximum non-linearity in the incident wave field. The sloping bed was incrementally shifted along the flume allowing the spatial phase of the group evolution relative to the initiation of shoaling to be adjusted. This placement of the sloping bed for the various test cases is discussed more fully within Section 4.2.3.

3.6 Summary of experiments

Experiments undertaken using three monochromatic wave structures and four wave group configurations were first observed in deep water, without the presence of a sloping bed, then with three different bed slopes: 5:1, 10:1 and 20:1. For each of these combinations, a differing number of bed positions were tested dependent on the data collection technique employed. For example, the spatially extensive breaking wave imagery is able to be acquired more rapidly, thus, a greater number of bed positions were able to be tested within the time available using these methods. Other techniques, such as optical LIF were much slower and fewer bed positions were tested. A summary of experiments are presented within Table 3-4.

Table 3-4 A summary of experiments. Quantity refers to the number of sloping bed positions tested except where otherwise indicated.

Slope	Class	Breaking Video	Probes	Optical LIF
No slope	C2N5r	-	1	-
No slope	C2N5sb	-	1	-
No slope	C2N3r	-	1	-
No slope	C3N5	-	1	-
5:1	Mono (x3)	1	1	1
5:1	C2N5r	14	14	14
5:1	C2N5sb	12	-	-
5:1	C2N3r	11	-	-
5:1	C3N5	8	-	-
10:1	Mono (x3)	1	1	1
10:1	C2N5r	22	11	11
10:1	C2N5sb	10	6	6
10:1	C2N3r	6	6	6
10:1	C3N5	6	6	6
20:1	Mono (x3)	1	1	1
20:1	C2N5r	15	5	5

Chapter 4 Experimental data processing

This chapter details the data processing methods and techniques required to fulfill the study objectives described within Chapter 1. Data was obtained from capacitance-type wave probes, optical LIF imagery and more spatially extensive, wide-angle imagery. Each of these data sources required calibration, and the development of techniques for extraction, processing and visualization of data. The development of these techniques, relative accuracies and limitations are described within.

4.1 Capacitance-type wave probes

Calibration was undertaken at two levels to ensure accuracy. Primary calibration of the probes was achieved by manually adjusting the ‘zero’ and ‘gain’ within the hardware. This was accomplished by setting a zero point with the probe element approximately half immersed. The probe was then lowered 100 mm and the gain adjusted to -5 volts. The probe was then raised to 100 mm above the initial zero and the gain checked.

The secondary calibration occurred within the wave flume in still water. The probes were incrementally raised by known amounts and an output voltage recorded. The probes were then incrementally lowered by the same amounts and the corresponding voltages averaged. This removed any trend of internal drift from the probes. The values were plotted and a linear regression line fitted (Figure 4-1). The slope of the line corresponded to the relation between output volts and change in water surface level. A ideal fit corresponded to 20 mm per 1 volt change. Values in the range of 18 – 22 mm per 1 volt change were accepted, otherwise probes were recalibrated (Table 4-1). The regression value, R^2 , gives an indication of goodness of fit and probe linearity. Probes with R^2 values of less than 0.98 were rejected and recalibrated.

Resultant elevation time series were then filtered using a zero-phase digital filter to remove unwanted signal noise. This noise varied slightly from probe to probe but generally had amplitudes of ± 0.2 to ± 0.5 mm and was primarily concentrated at frequencies between 35 and 50 Hz. The filter employed was a 9th-order lowpass Chebyshev Type I filter with 0.05 dB of ripple in the passband and a passband edge frequency of 20Hz. This gave a relatively fast roll-off so that actual peaks and troughs suffered less truncation than for some other filters. Figure 4-2 shows an example of raw

surface elevation compared to the filtered elevation, with removal of high frequency noise, particularly at the wave crest evident. Filtering was found to reduce wave heights by an average of 2% below the raw non-filtered elevation records. However, as shown by Figure 4-2, the filtered signal more accurately captures the form of the wave crests. Zero-phase filtering processes the input data in both the forward and reverse direction, meaning that the resultant filtered series has zero phase distortion.

Table 4-1 Example calibration values for six wave probes.

Probe	Calibration slope (V/mm)	mm/V	R ²
1	0.0483	20.70	0.9992
2	0.0516	19.38	1.000
3	0.0523	19.12	0.9999
4	0.0517	19.34	1.000
5	0.0502	19.92	1.000
6	0.0508	19.69	1.000

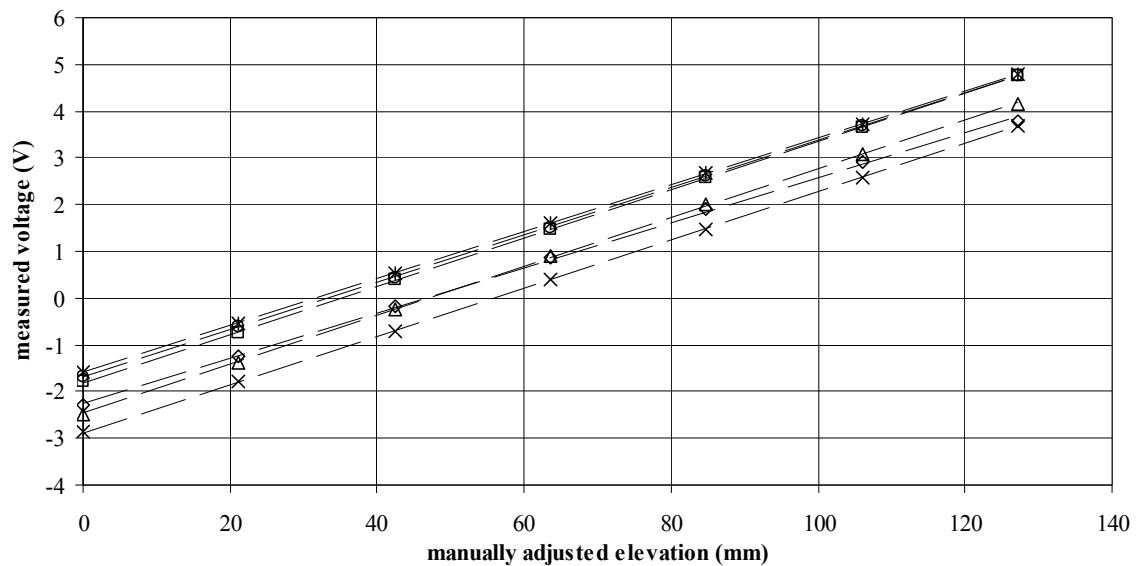


Figure 4-1 Example of a probe calibration series with symbols corresponding to observed voltages and dashed lines to linear regression lines.

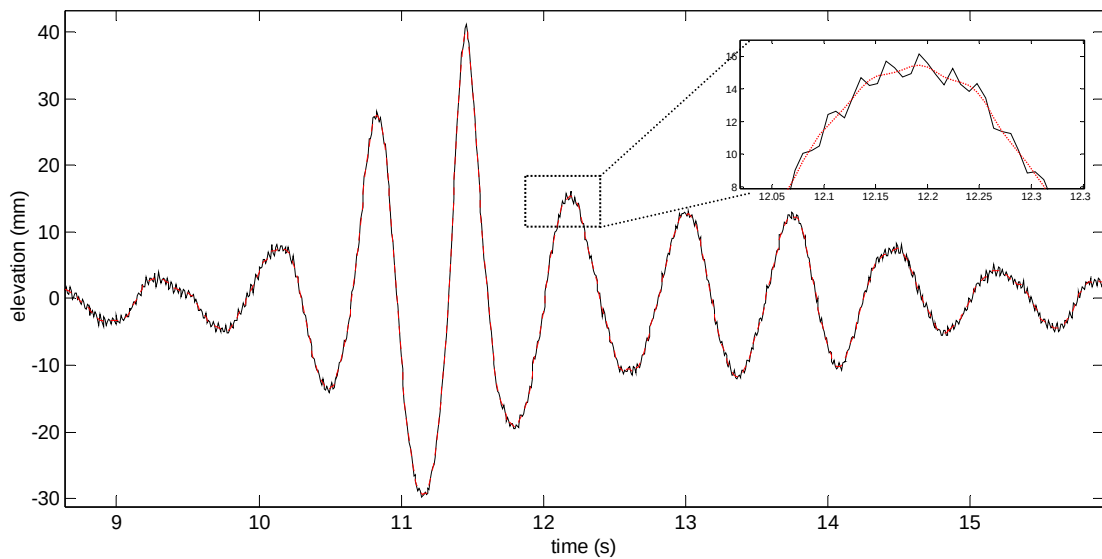


Figure 4-2 Example time series of raw surface elevation for a C2N5r type wave group (—) and surface elevation filtered using a lowpass Chebyshev Type I filter (---). The inset box depicts the region around a wave crest.

Capacitance probes typically exhibit a voltage drift over time due to temperature. However, calibration coefficients for these probes have been previously shown to have less than 2% drift when subject to temperature variations of 10°C (Peirson et al, 2001). This means that if data is collected over a long period of time, the series must be de-trended. This meant that the recorded surface position could not be determined with respect to a fixed datum. Therefore, data from the capacitance probes was corrected to the mean level of the series prior to processing.

4.2 LIF imagery

4.2.1 Processing methodology

Data from the Pixelfly camera was recorded as either bitmap still images or as uncompressed AVI video files. During an experiment, videos were collected sequentially along the flume from deep water to beyond the intersection of still water and the sloping bed. Several routines were created to merge, digitise and extract data from these videos. These routines are summarized and discussed in the following sections and presented in full within Appendix B.

4.2.1.1 Extracting air-water interface

Each video file was sequentially read frame by frame and converted from 16-bit Red-Green-Blue (RGB) images to 8 bit (256 intensity shades) greyscale images. A boundary-tracing algorithm was first employed to define an approximate region of interest (ROI). This was necessary to exclude spurious splashes, reflections and intensity dropout (salt and pepper) noise from the extraction. This was particularly important within the breaking region and surf zone where the air-water surface was considerably irregular (Figure 4-3). Based on this ROI, the derivative of the intensity was assessed along each column within the frame. The air-water interface was defined at the position of the maximum change in image intensity (Figure 4-4) as described by Law et al., 1999. This method was employed, rather than the simpler fixed threshold method due to the variation in intensity along the laser sheet caused by the laser spreading method and the variation in intensity between videos as a result of fluctuations in the fluorescent dye concentration and differences in wave properties (i.e. deep water, breaking, surf zone). The position of maximum intensity change was determined to sub-pixel accuracy using a spline function (Figure 4-5). This function enabled measurement resolution of the air-water interface to 0.2 pixels or around 0.05 mm.

The resultant series gave the upper air-water interface at every along-flume position. This meant that the complete geometric description of overturning crests, involving multiple interfaces at specific along-flume locations were not described, but rather only the upper edge. The complete geometry, i.e. multiple interfaces, were able to be obtained through the first ROI boundary tracing algorithm but to a lower resolution. Subsequent processing of this type of data was also considerably more difficult and, as this breaking description was not expressly required for this present study, multiple interfaces were ignored.

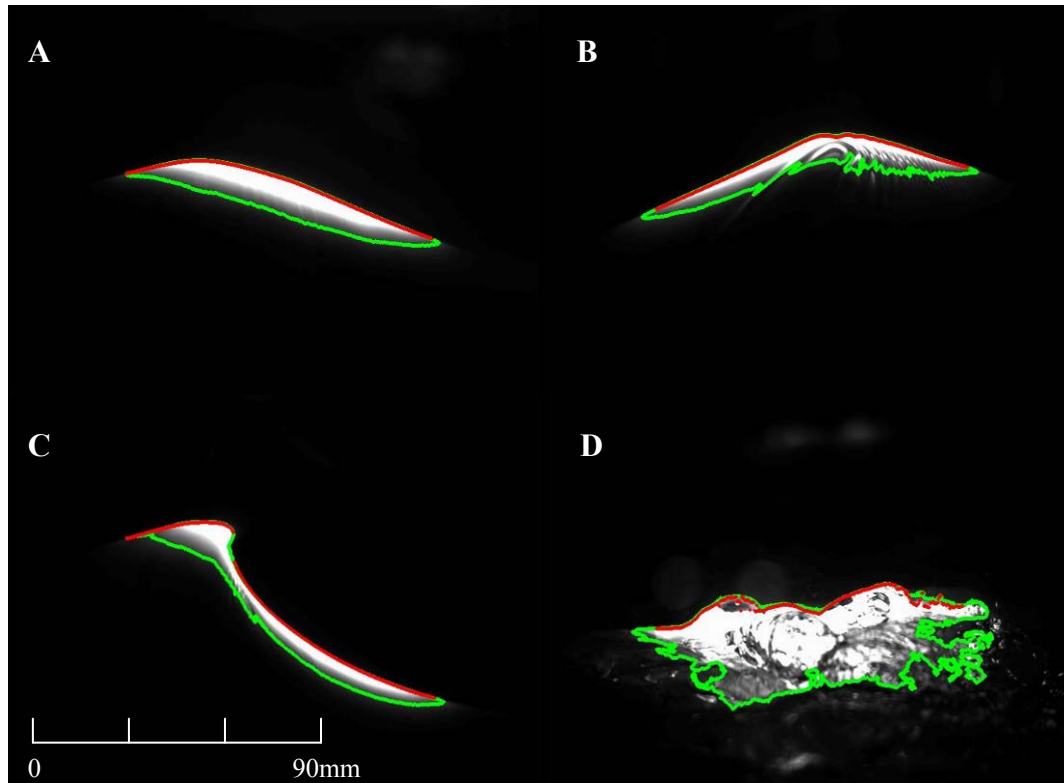


Figure 4-3 A region of interest (green line) is first defined by automated tracing of the boundary of the high intensity area (thus omitting splashes and other noise not connected to the main fluid body). The upper surface of the region corresponding to the air-water interface is then defined to sub-pixel resolution (red line). Examples show deep water (A), incipient breaking (B), breaking by weak plunge (C) and within the turbulent surf-zone (D). Note the concave portion of the overturning crest in (C) while detected by the ROI, is not used for subsequent processing.

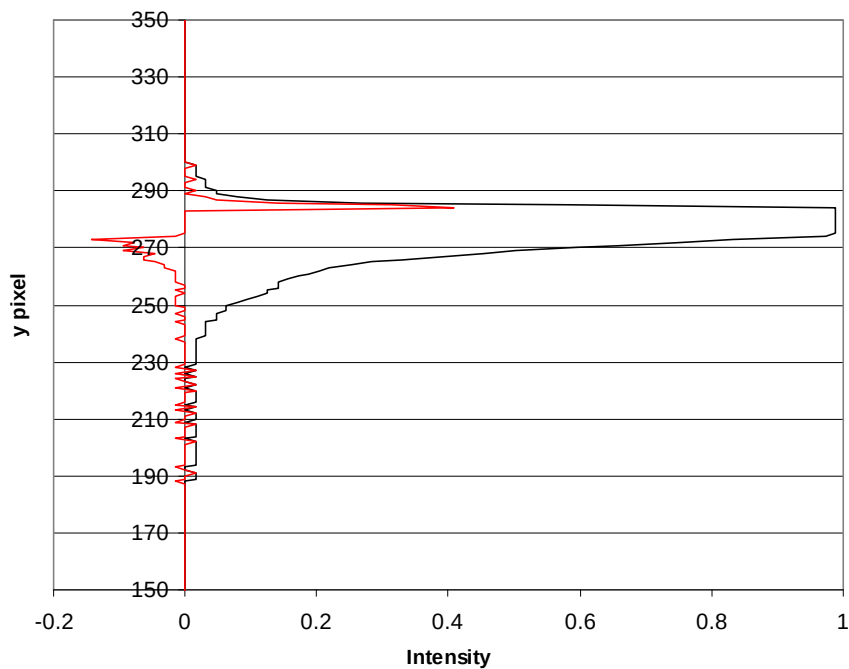


Figure 4-4 Intensity (—) and its derivative (—) along a single column (spatial location) of a frame of video imagery.

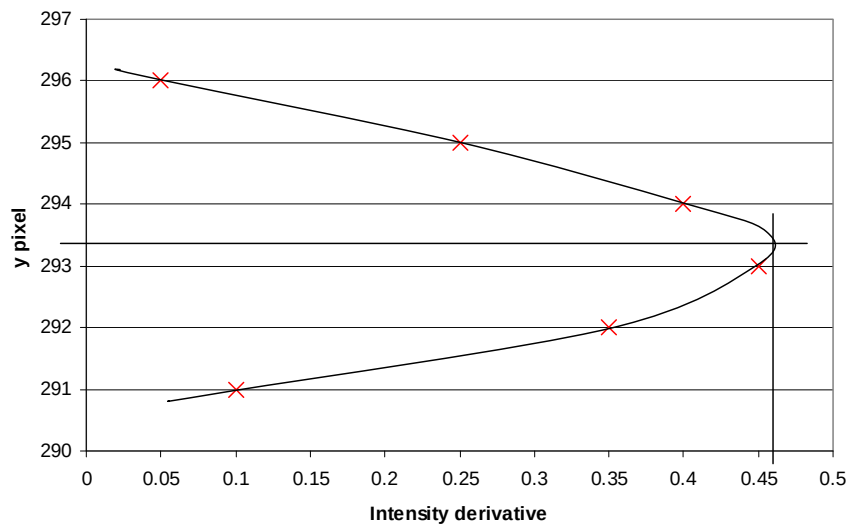


Figure 4-5 Example of determining maximum intensity change to sub-pixel accuracy using a spline function.

4.2.1.2 Image correction

The series of surface profiles obtained by the preceding routine, referred to hereafter as a *video-series*, were then processed to correct for optical and perspective distortion within the image, rectified to horizontal and adjusted to physical co-ordinates with respect to still water level at the bed crest position. The resultant values were then amended to allow for the spatially varying laser-sheet geometry and finally despiked to remove any remaining erroneous values. This was achieved in a number of steps as outlined within the flow diagram illustrated in Figure 4-6.

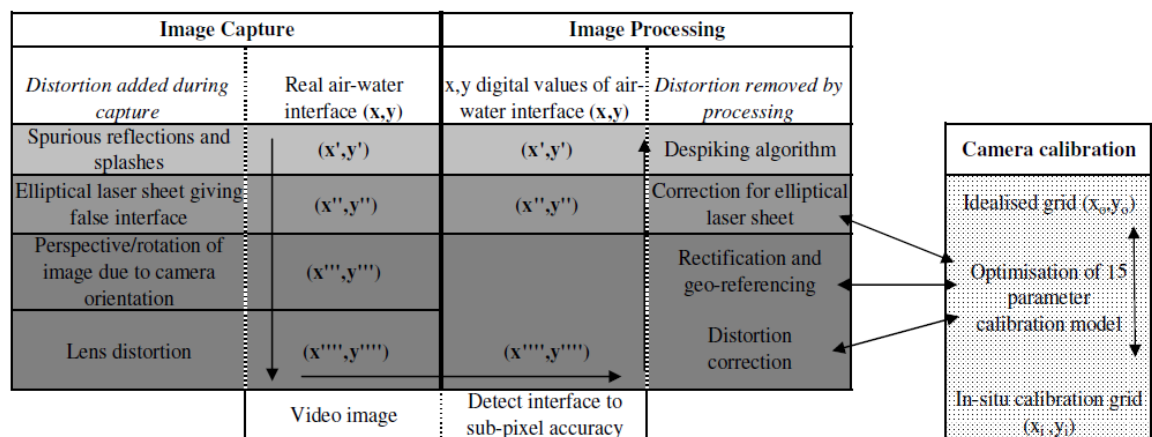


Figure 4-6 Flow chart illustrating the introduction of distortion and error into the image (left hand side) and subsequent removal and image registration (right hand side).

4.2.1.2.1 Calibration model

A 15 parameter model was optimized according to the Lavenberg-Marquardt method (Press, 1992) by digitising an image of an in-situ calibration grid (Figure 4-7). This model included parameters to correct the image centre and aspect ratio, the radial warp associated with barrel and pincushion distortion and perspective distortion associated with the slightly oblique orientation of the camera.

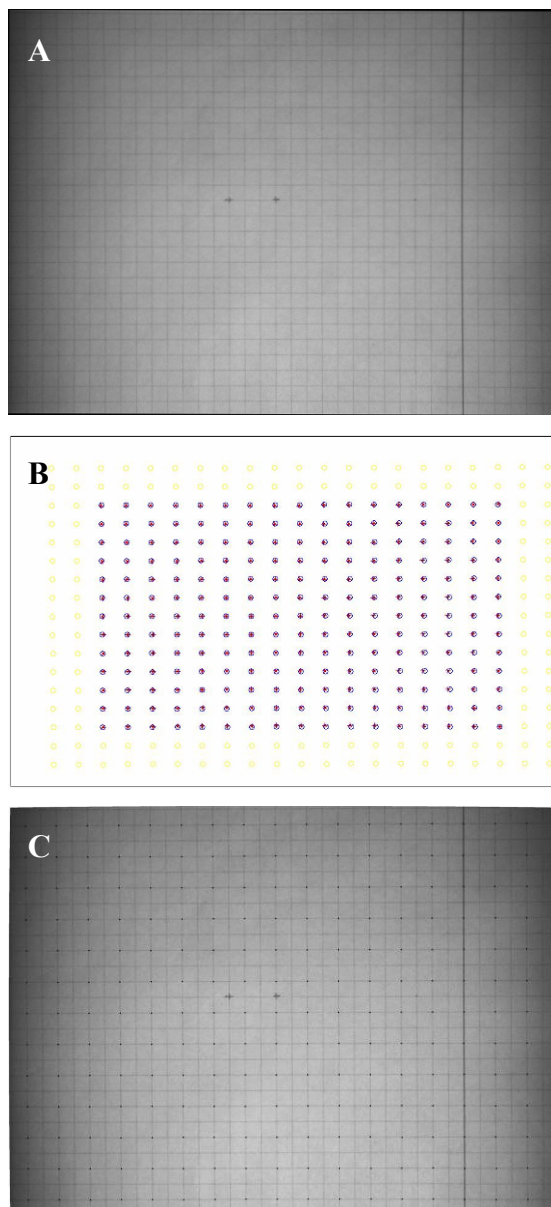


Figure 4-7 A calibration sheet is captured in-situ (A) exhibiting perspective and optical distortion. The observed grid intersections are defined and a correction model calibrated to transform the observed grid into an idealised grid (B). The original image is then corrected using the calibrated correction model (C).

Radial warp result from aberrations in the light rays passing through the camera lens. Wide angle lenses tend to result in barrel distortion where parallel lines are distorted with a bulge, while telephoto lenses result in pincushion distortion where straight lines are pinched in at the sides (Figure 4-8). In reality, a combination of both of these effects at different distances from the image centre can occur. Corrections for radial warp are achieved by the pair of equations given in (Eqn. 4-1).

$$\begin{aligned} x' &= x(1 + \kappa_1 r^2 + \kappa_2 r^4) + 2\tau_1 xy + \tau_2 (r^2 + 2x^2) \\ y' &= y(1 + \kappa_1 r^2 + \kappa_2 r^4) + \tau_1 (r^2 + 2y^2) + 2\tau_2 xy \end{aligned} \quad (4-1)$$

Where:

- x', y' = corrected x and y positions relative to the centre of distortion
- x, y = original x and y positions relative to the centre of distortion
- κ_{1-2}, τ_{1-2} = parameters solved through optimisation of a calibration grid
- r = the input radius from the centre of distortion

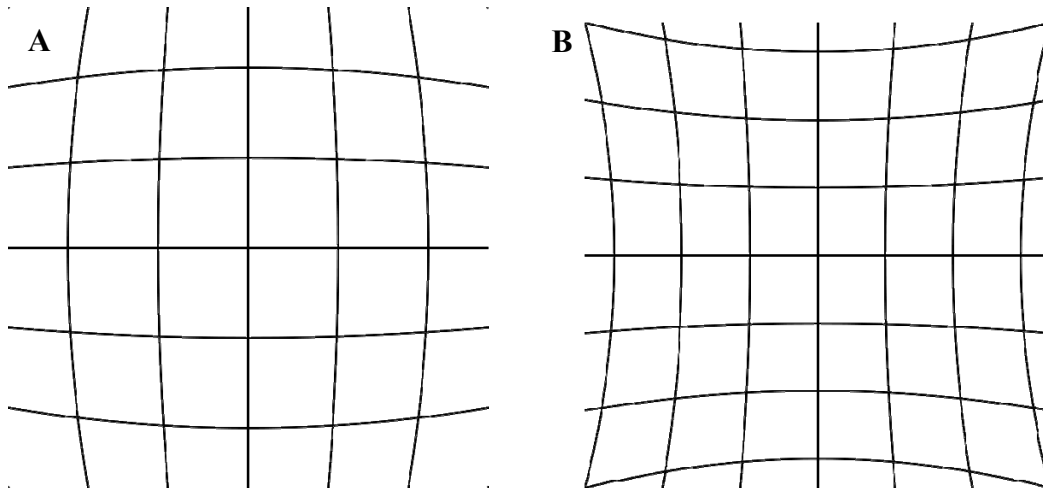


Figure 4-8 Barrel (A) and pincushion (B) distortion.

Rectification and referencing of the image to a physical tank position must take into account perspective distortion which results from the viewpoint of the camera in relation to the subject. As the imagery was captured looking slightly down on the subject (to avoid the meniscus obscuring the light sheet), some perspective distortion was expected.

This distortion is quantified using 9 parameters within the model described by the pair of equations (Eqn. 4-2).

$$\begin{aligned} y' &= \frac{\varepsilon_1 x + \varepsilon_2 y + \varepsilon_3}{\varepsilon_7 x + \varepsilon_8 y + \varepsilon_9} \\ x' &= \frac{\varepsilon_4 x + \varepsilon_5 y + \varepsilon_6}{\varepsilon_7 x + \varepsilon_8 y + \varepsilon_9} \end{aligned} \quad (4-2)$$

Where:

ε_1 to ε_9 = parameters solved through optimisation of a calibration grid

As all parameters interact to some extent, they must be optimised simultaneously. This was carried out using the Lavenberg-Marquardt method for nonlinear parameter optimisation (Press et al., 1992) to match the known position of the target grid with the detected gridline intersection of the calibration image. Errors in the optimised correction model may be quantified according to the root mean squared (RMS) difference between the digitised and corrected grids. Quantification of this error yielded an RMS error of 0.05 mm.

It was difficult to precisely orientate the camera horizontal to the still water surface (i.e. horizon parallel to the water surface). While great care is taken mounting the camera as close to horizontal as possible, a slight slope (less than 1° and variable from set-up to set-up) may be present. Consequently, an additional rotation function was incorporated that allowed manual fine-tuning of the image rotation post calibration.

If uncorrected, these features are also evident in the detected mean water level. An example (Figure 4-9) where along-flume variation in mean water level should be small clearly shows this along-flume slope. Matching successive image segments along the tank yields steps or jumps in the derived along-tank mean water level. By comparing the detected mean water level across a number of sequential deep water video-series, an appropriate rotation is determined and incorporated into the correction routine.

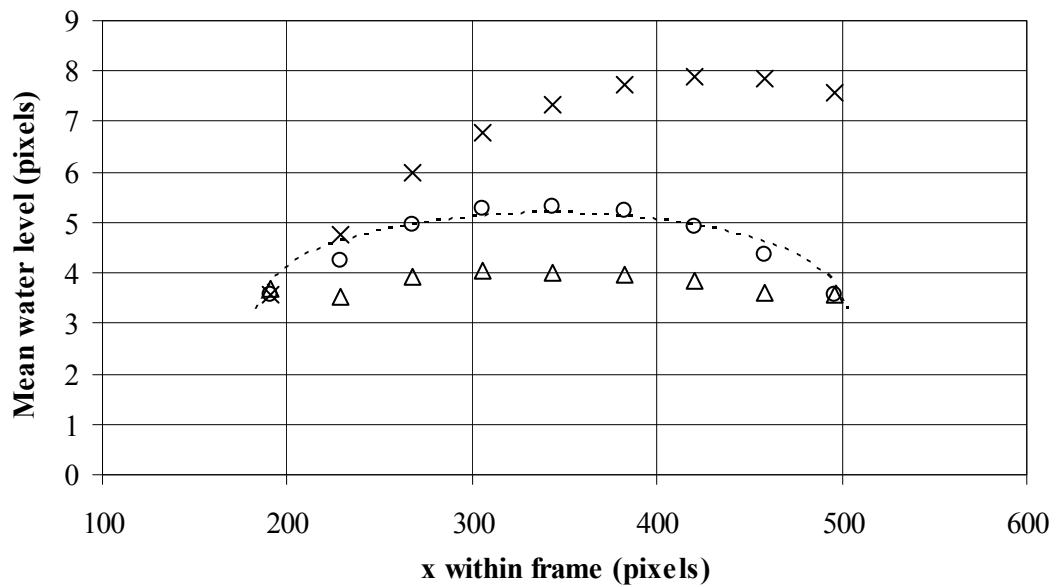


Figure 4-9 Example of distortion corrections applied to raw surface elevation data. Crosses indicate the mean water level (mwl) obtained from an uncorrected image; circles, the mwl once optical distortions and horizon effects have been removed; the dashed line depicts an idealised ellipse used to correct for the elliptical shape of the light sheet and the triangles show the mwl once this elliptical correction has been applied.

To obtain absolute maximum and minimum levels along the tank required referencing of the imagery to the absolute position of the camera in space. The horizontal position was determined by the location of the flume-top trolley. The vertical position was dependent on the distance between trolley and still water level, which was found to vary slightly with horizontal location. This along-flume variation was found using a point gauge mounted to the flume-top trolley. Once the location of the swx was defined within the first video series (in terms of $[x,y]$ pixels), corrections to the vertical offset of adjacent series were applied according to Figure 4-10.

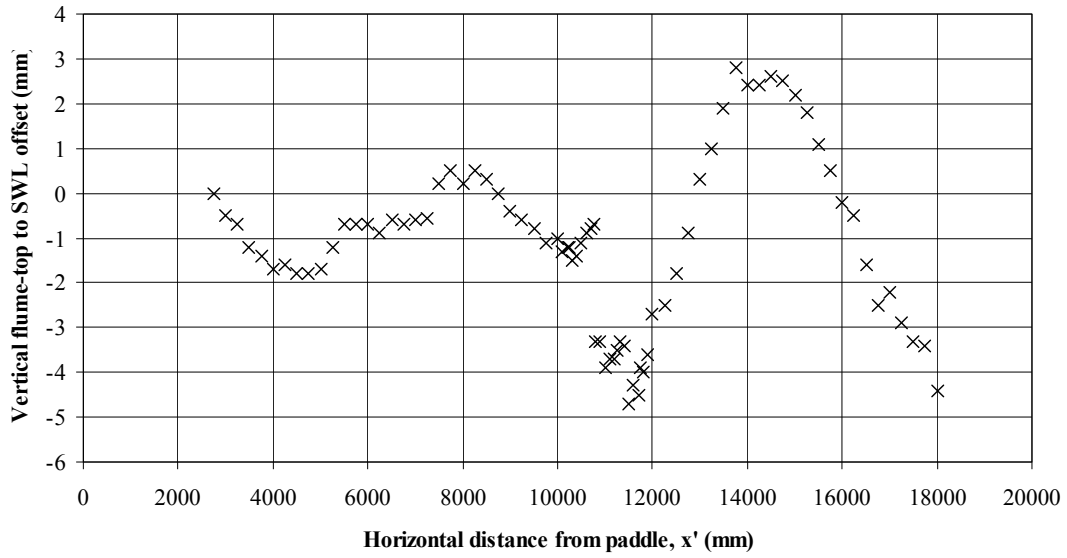


Figure 4-10 Vertical offset (from an arbitrary datum) between flume-top trolley and SWL as a function of horizontal distance from paddle (x').

4.2.1.2.2 Laser sheet geometric errors

After the profile series have been corrected for distortion, some convex appearance is evident along the upper edge (Figure 4-9). This convex effect is a result of minor ellipticity of the laser sheet induced by the plano-concave lens spreading the circular laser beam in one dimension. When viewed from above, the laser sheet appears as an ellipse. The ‘top’ (farthest) edge of the ellipse may thus be described by Eqn. 4-3.

$$y = \sqrt{b^2 \left(1 - \frac{x^2}{a^2}\right)} \quad (4-3)$$

Where a and b are the radii along the major (along-flume) and minor (cross-flume) axis respectively and x and y are the distance from the ellipse centre in the along-flume and cross-flume directions respectively. The radii a and b were measured manually during still water conditions and had dimensions of 110 and 3 mm respectively.

As the perspective view becomes progressively more oblique, the ellipse is observed to become progressively flatter (i.e. b reduces in width) until sides converge to a line as the view nears horizontal and b reduces to near 0.

By determining the angle of view from the observation point (the CCD camera) to the target (the ellipse major axis), a reduction in the apparent length of the minor axis (b) from that observed vertically can be calculated according to $b' = \sin \alpha(b)$, where b' is the perspective adjusted minor axis radius and α is the angle of observation. This may be substituted into the description of the elliptical 'top' edge according to Eqn. 4-4.

$$y = \sqrt{\sin \alpha(b^2) \left(1 - \frac{x^2}{a^2}\right)} \quad (4-4)$$

The angle of observation during an oscillatory wave-form changes constantly as a function of water surface level and must therefore be recalculated at every time-step for each observed position along the laser-sheet. The resultant y values are then subtracted from the observed interface to ensure that the air-water interface is defined along the centreline (major axis) of the ellipse and not the edge. During the measurements taken here, α varied between 4.5° and 15° . Consequently, the required adjustment due to the effect of elliptical laser sheet shape ranged between 0 pixels (0 mm) at the ends of the laser sheet, to 5.8 pixels (~ 1.5 mm) at the longitudinal centre of the laser sheet with α at 15° .

The mean water levels obtained from several tested video series after elliptical correction are compared to the raw and distortion/perspective corrected mean water levels. Results, presented in Table 4-2 show improvement (average along-profile difference of 0.75 pixels $\approx \pm 0.1$ mm) over the distortion/perspective corrected levels (1.3 pixels $\approx \pm 0.16$ mm) and substantial improvement over the raw levels (4.75 pixels $\approx \pm 0.65$ mm). Values found for the specific video series shown in Figure 4-9 are included for comparison.

Table 4-2 Resultant average variations of mean water level along a single laser sheet series.

	Raw (pixels)	With distortion/ perspective correction (pixels)	With elliptical correction (pixels)
Mean	4.76	1.32	0.72
Minimum	3.32	0.76	0.56
Maximum	8.52	1.84	0.8
Figure 4.8	4.24	1.66	0.56

4.2.1.2.3 Despiking

Erroneous data not removed by setting the region of interest required signal processing. A number of methods were examined including lowpass filters and series steepness thresholding. However, lowpass filters tend to truncate actual peaks as well as those in error and setting a threshold on series steepness is unreliable in detecting spikes comprising multiple points and may exclude real data during very steep phases of waveform evolution. A despiking routine based on the phase-space method proposed by Goring and Nikora (2002) for despiking acoustic doppler velocimeter (ADV) data and described in Mori (2007) was employed to detect remaining irregularities in the series. This routine is advantageous as it is more efficient than Fourier-based filters and requires no empirical coefficients. Time series data (y) and its first and second derivative (y' and y'') are plotted against each other. The variance of plotted data is used to construct an ellipse of expected maximum values (Goring and Nikora, 2002). Data points lying outside this ellipse are designated as outliers and removed. The ellipse is then recalculated and replotted and further outliers removed with iterations continuing until no further outliers remain (Figure 4-11).

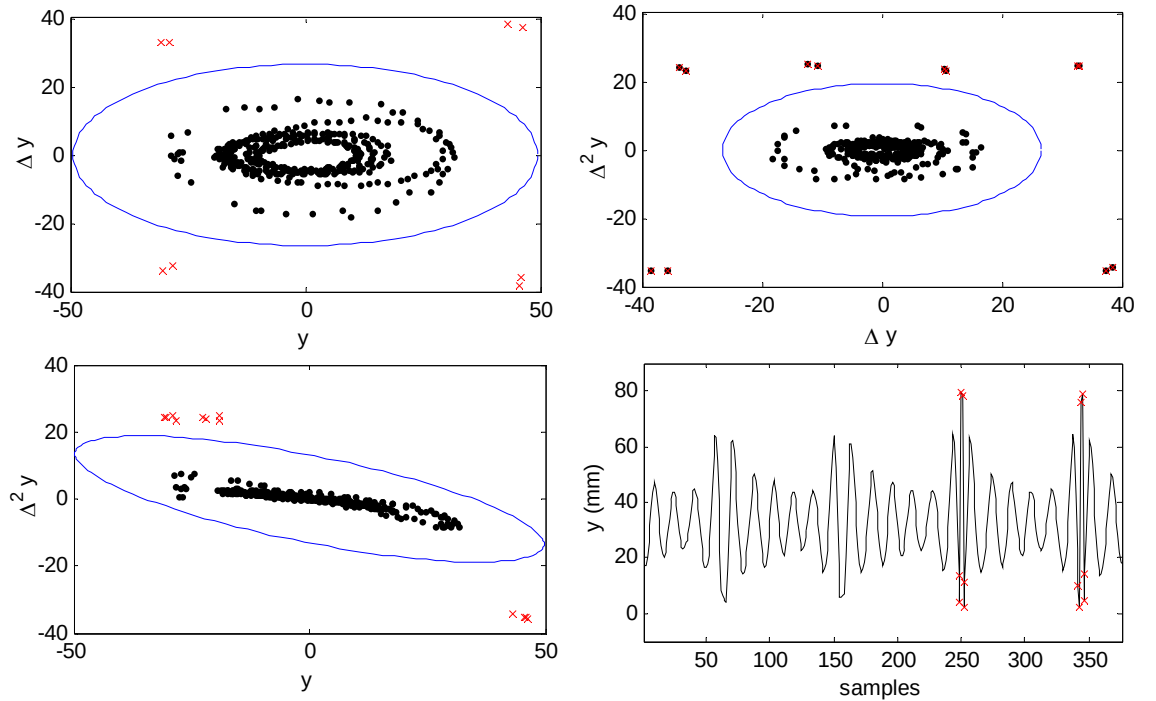


Figure 4-11 Example of despiking using the phase-space method of Goring and Nikora (2002). A raw time series is shown in the bottom right plot with the elevation values and the first and second derivatives of elevation plotted against each other in the remaining plots. Data points lying outside the idealised ellipse as discussed in the text are designated as outliers (indicated by red crosses) and removed.

4.2.1.3 Resultant series

Examples of water surface profiles extracted from the frames shown in Figure 4-3; spatially referenced, undistorted and corrected are presented within Figure 4-12. A discussion of the resolution and accuracies of the method are presented in Section 4.3.4.

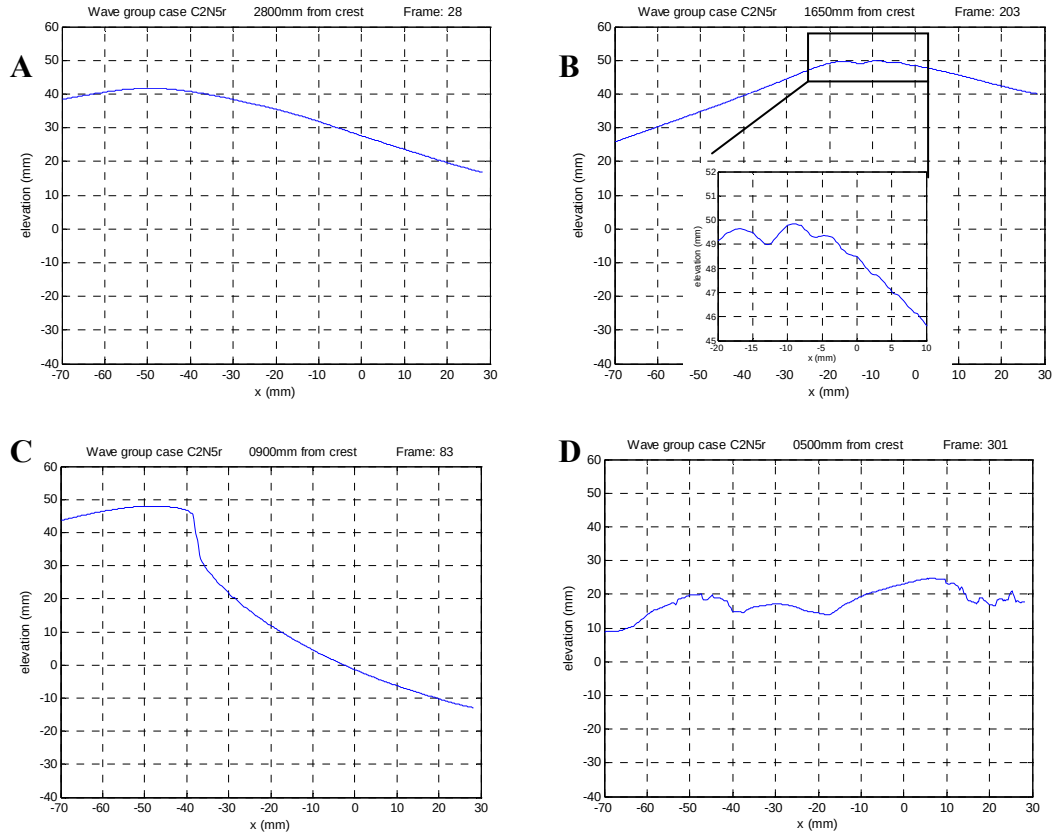


Figure 4-12 Resultant water surface profile examples from a deep water wave (A), incipient breaking (B), breaking by weak plunge (C) and within the turbulent surf-zone (D). The additional frame within (B) shows an enlargement of the incipient breaking region illustrating detection of waves less than 1 mm in height and 5 mm in wavelength. Note the concave portion of the overturning crest in (C) is truncated to near vertical.

4.2.1.4 Data merging

The resulting arrays, produced for each video-series, may then be viewed in one dimension as spatially varying profile-series either fixed in time, or in the second dimension as temporally varying time series fixed in space.

Analysis of spatially discrete time series (Figure 4-13) allows rapid assessment of wave properties comparable to traditional wave probe outputs.

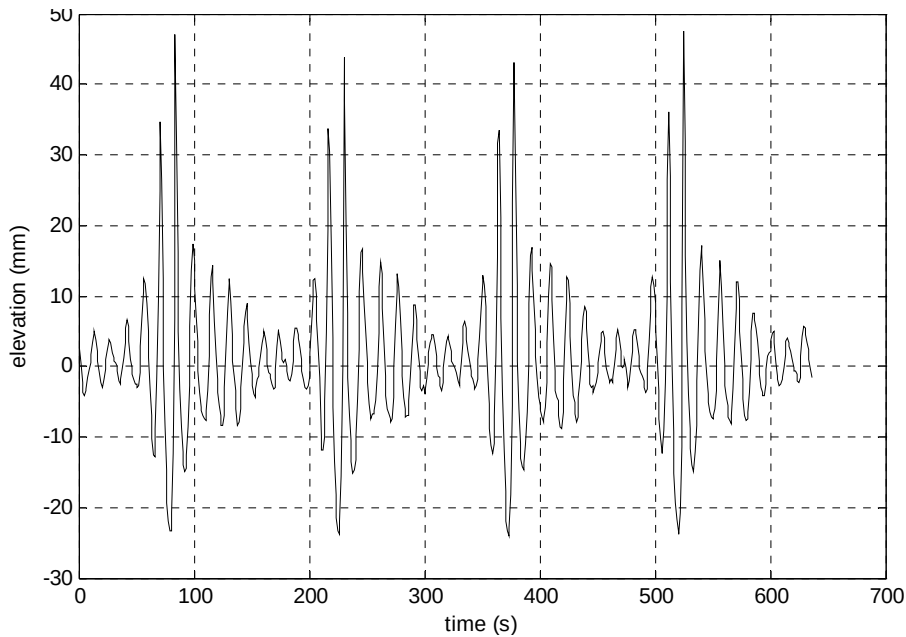


Figure 4-13 Extracted time series for a C2N5r class wave group 1475 mm from the intersection of a 10:1 sloping bed and still water level (swx).

As adjacent video series were independent of each other with respect to wave phase, an algorithm was implemented to allow sequential series to be synchronised and merged. This was achieved by incrementally adjusting one time series in the time domain with respect to the adjacent time series and evaluating the RMS difference of the elevations. The phase difference is identified by the minimum RMS difference. While a spatial overlap between adjacent video series is ideal, this was not achieved by the present study. Wave celerity was therefore determined for adjacent series and used to correct the phase difference. Some error associated with rapidly changing celerity in the shoaling and breaking zone is therefore expected. Using this corrected phase difference, adjacent video series can be synchronised and merged.

An example of a space-time plot comprising 20 individual video series, synchronised and merged is presented in Figure 4-14. Note the white spaces depicting discontinuities as a result of a non-overlap between adjacent series. This space-time plot gives time along the x axis, position within the flume with respect to the swx in the y axis and water level in the z axis. Dispersion within the group can be easily observed, as can reduction in wave celerity within the surf zone. While outputs have previously been presented in this form (Baldock et al., 2004), they have been derived from interpolation between spatially discrete resistance and capacitance probe measurements. This

introduces intrinsic error and uncertainty across the interpolated surface, especially within the rapidly changing breaking and surf zone.

While this space-time series (Figure 4-14) may still be observed at a specific spatial location as a time series, it may also be observed at a discrete temporal position as a spatial series (Figure 4-15). Data in this form allows detailed observation and measurement of wave kinematics including wave shape, steepness and velocities of individual wave portions such as the wave crest.

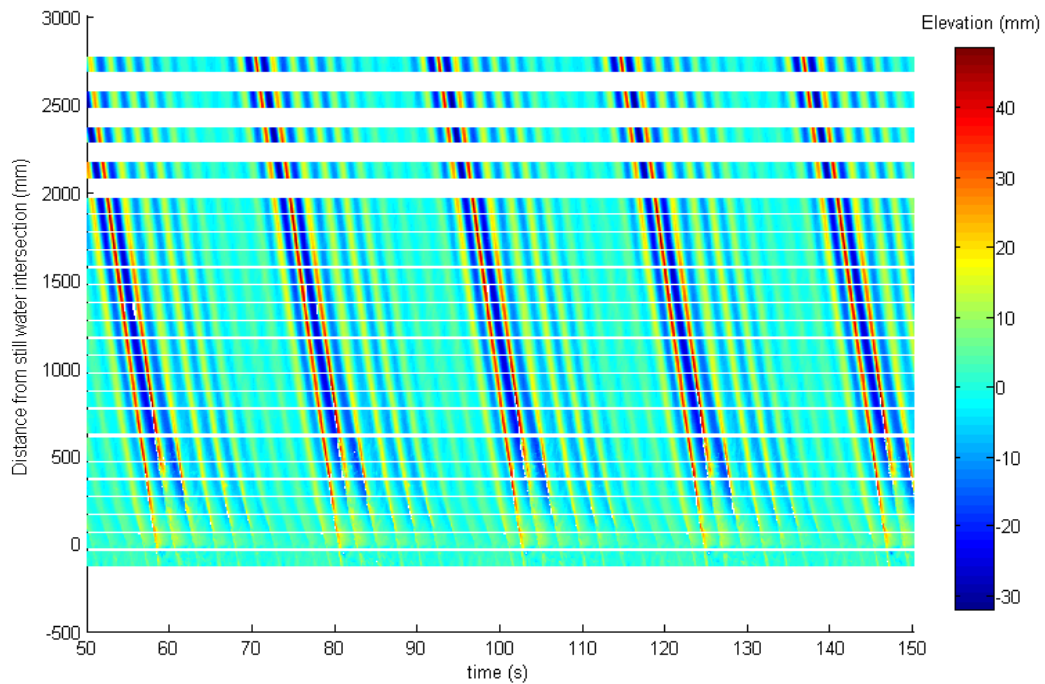


Figure 4-14 Space-time plot showing wave group evolution during shoaling, breaking and progression through the surf zone. Discontinuities indicate missing data between adjacent video series.

Due to the relatively slow temporal resolution of the camera system employed (20 Hz) compared to the wave period (0.7365 s), a phase error in synchronization may remain between merged video series. Given a phase shift between successive images of $360^\circ / (20 \text{ Hz} * 0.7365 \text{ s}) = 24.4^\circ$, the maximum error in synchronisation between adjacent video series may be as high as $24.4^\circ / 2 = 12.2^\circ$. This is observed within Figure 4-16 where the n th wave in a series of idealised groups is shown. Differences in elevation at either end of the profile are evident and may be substantial. Methods for improvement of this disparity are discussed within Section 4.2.2 and quantification of the errors in Section 4.3.4.

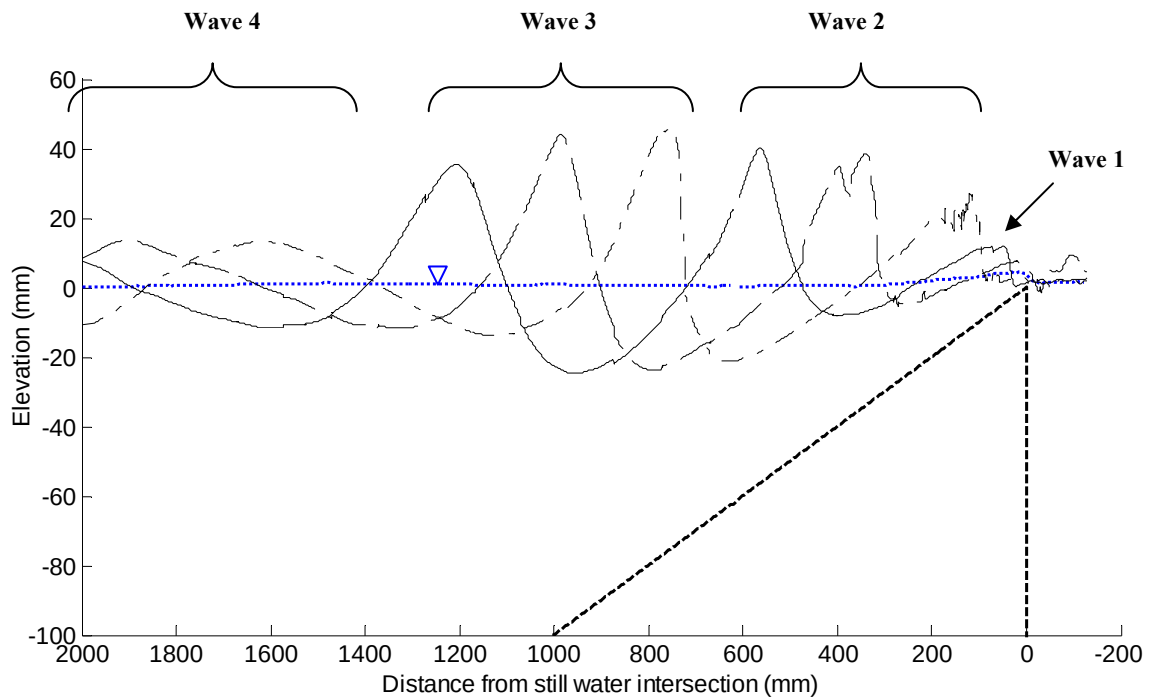


Figure 4-15 Examples of combined profile-series with the solid line (—) depicting the profile at an arbitrary time of $t = 4.20$ s, the dashed line (---) $t = 4.85$ s and the dash-dotted line (— ·) at $t = 5.50$ s. The blue dotted line (···) depicts the mean water level and the bold dashed line (—), the sloping bed. At $t = 4.20$ s a leading wave (wave 1) has broken and is almost at the swx, travelling as a broken bore. A second, larger wave (wave 2) is steepening as it approaches break point and the third and fourth waves (waves 3 and 4) are steepening as they shoal. At $t = 4.85$ s wave 2 has broken and the initial splash-up following breaking is evident. Waves 3 and 4 continue to steepen as they shoal. At $t = 5.50$ s wave 2 is progressing across the surf zone as a turbulent bore, wave 3 is at break point with the front face near vertical and wave 4 continues to shoal.

4.2.2 Discussion

4.2.2.1 Improving temporal resolution

The limited temporal resolution (20 Hz) of the optical LIF method affects results in two ways.

1. While monochromatic and grouped waves generated within this flume have been shown in this and previous studies (Banner and Peirson, 2007) to be highly repeatable, a small range in wave heights between successive groups is evident within the optical LIF data (i.e. in Figure 4-13);
2. When adjacent video series are merged, adjacent surface elevations often contain discontinuities as shown evident within Figure 4-15.

The reason for this, as described in Section 4.2.1.4, is that successive images have a 24.4° wave phase shift and so adjacent images may have up to 12.2° phase difference. This means that defining the absolute peak of every wave crest is not possible. It also means that the correlation of adjacent video series is somewhat random.

Figure 4-16 depicts the n th wave within a group just prior to breaking for 3 consecutive groups and the $n+1$ frame from the first group. This figure indicates that the wave-forms and data capture apparatus are close to synchronised within 4 groups. If the groups are repeatable, as previous studies have shown (i.e. Banner and Peirson, 2007), then 3 sequential groups may be combined together. The temporal resolution of the series is thus improved and the phase shift between successive images and adjacent series reduced to 8.2 and 4.1° respectively.

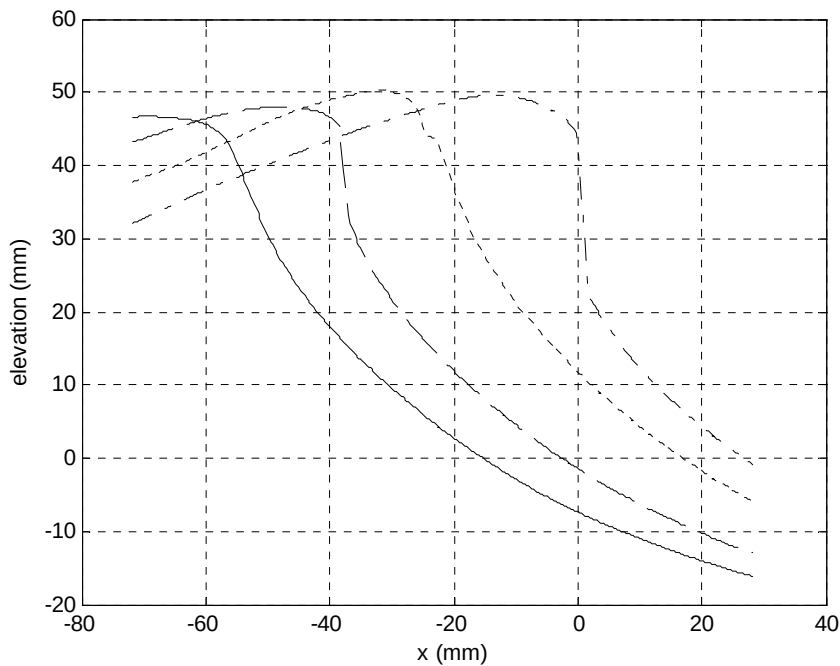


Figure 4-16 Example of the n th wave of 3 consecutive wave groups with group 1 depicted by (—), group 2 by (---), group 3 by (· ·) and the $(n+1)$ frame of group 1 (—·).

4.2.2.2 Observation limits

Due to the observed portion (laser sheet) of the wave-form being near the centre of the flume, the part of the wave-form between the sheet and the flume has the ability to occasionally obscure portions of view. Most typically, this portion is the steep front face of the wave once it has passed the camera's central axis (Figure 4-17).

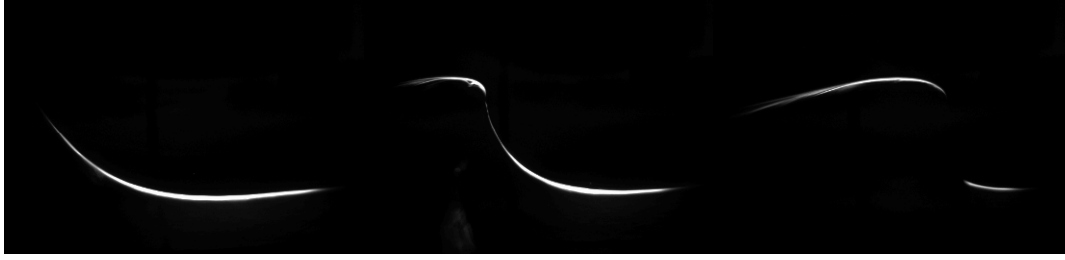


Figure 4-17 Exemplary wave-forms with the steep wave face becoming obscured away from the camera's central axis.

Preliminary quantification of these effects indicate that the front face of the wave is only observable when the following relation (Eqn. 4-5) is true:

$$\tan \theta \geq \frac{d\eta}{dx} \sin \alpha + \frac{d\eta}{dy} \cos \alpha \quad (4-5)$$

Where:

θ = the angle of observation between the horizontal and the water surface

$\eta(x,y)$ = the water surface position

α = the angle along the flume (down-flume) from the camera midpoint to the observed water surface position

While the actual portion of wave face obstructed is also a function of the wave face shape and the distance between the flume sidewall and observation point, the expression in Eqn. 4-5 gives a useful indication of when obstruction of the wave front is likely to occur.

The above expression, shows that obstruction is minimised by increasing θ , the vertical camera viewpoint, by minimising α , the along-flume angle of view, or by limiting the steepness of the front waveface. As the present study involves observing wave forms throughout the breaking process, limiting the steepness of the front waveface is impractical. The camera elevation may be increased; however, this occurs at the expense of accuracy in defining the water surface position as errors in correcting for this perspective distortion during image rectification increase. Finally, α may be minimised by either limiting observation down-flume of the camera midpoint or by orientating the camera in a slightly up-flume direction, thus ensuring the entire image is up-flume of the camera midpoint. The disadvantage in the former is a reduction in useable captured image and, in the latter, increased error during subsequent image rectification. In

practice, a balance may be achieved by elevating the camera slightly and orientating the camera slightly up-flume to minimise obstruction.

4.2.2.3 *Method accuracies*

Error analysis of the optical LIF method comprises several components including both systematic (constant between measurements) and random errors. These are:

1. resolution in determining the raw air-water interface: 0.2 pixels (0.05 mm) using the sub-pixel detection technique;
2. residual optimization error in calibration of ± 0.05 mm;

While the above errors influence the actual definition of the air-water interface, when accurately referencing the interface to spatial coordinates an additional error is introduced:

3. Residual error in correcting for horizon and laser sheet geometry of ± 0.1 mm.

The combined effect of these errors may be assessed using the property of variance addition as described within statistical texts (eg. Larson and Marx, 1986). As these sources of error are independent, they may be combined by the root sum of the squares. This gives a combined error in determining the absolute position of the air-water interface of ± 0.12 mm.

Merging of adjacent images introduces an additional random phase error dependent on the phase difference between adjacent sequences. This phase error may reach a maximum of 12.2° and up to 17% of the wave-form height if the adjacent series meet at the point of maximum change in elevation. These two variables are independent of one another and thus this maximum elevation error has low probability of occurring. However, the discontinuities evident in Figure 4-15 show that some disparity does regularly occur. Additionally, steeper wave-forms, such as the front face immediately prior to breaking may exhibit larger differences. Improving the temporal resolution on the series through merging of sequential group forms reduced this maximum phase error to 4.1° or 6% of the maximum change in elevation.

Similar to the phasing of adjacent series, definition of the absolute crest and troughs of observed wave forms may contain some uncertainty due to the phase difference of 24.4° between successive images. This is further discussed within the following Section but

for relatively uniform waves within deep water, observed wave height may be up to 2.2% below actual height. The error becomes greater near a steep crest.

4.2.2.4 Comparison to probes

Optical methods are yet to be proven as a reliable tool for measuring surface elevations. While conventional techniques such as capacitance-type probes and acoustic sensors have been found to be unreliable in measuring steep wave forms or wave forms within the surf zone, these techniques are recognized as providing reasonable measurement of deep water waves of low steepness. Two capacitance-type wave probes (referred hereafter simply as probes) and one acoustic sensor were thus used alongside the optical-laser assembly to simultaneously record a number of wave-train series (Figure 4-18).

Both the probes and acoustic sensor were calibrated using a static water calibration. Capacitance probe resolution was found by Banner and Peirson (2007) to be better than 0.1 mm and linearity better than $\pm 1\%$ over their 200 mm length. Manufacturers similarly give accuracy at 0.4% or 0.8 mm over the probe's 200 mm range (RBR, 2007). Acoustic sensor resolution is given by manufacturers at 0.18 mm and accuracy at 2 % (Microsonic, 2007). However, the detection area of the acoustic sensors is a less well defined. At a 200 mm distance, Microsonic (2007) defines the detection area as a spot of approximately 100 mm diameter.

To undertake controlled benchmarking of the optical method developed, the probes and ultrasonic sensor were aligned with the latitudinal centre of the laser sheet and were spaced at intervals of 80 mm across the flume.

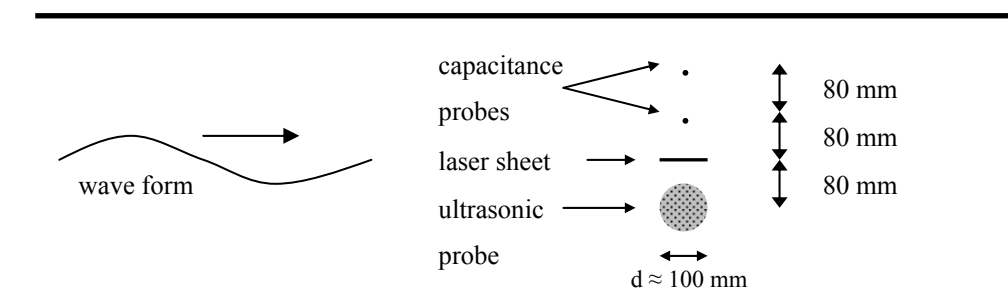


Figure 4-18 Schematic plan view diagram of the laser sheet alongside two capacitance wire probes and an acoustic sensor (sensor area at 200 mm range indicated).

While ensemble averaged comparisons between optical and conventional methods can be undertaken, a more accurate method is by direct comparison. The capacitance and acoustic sensor are digitally synchronised; however, the optical time series required manual synchronisation. An example is presented in Figure 4-19, with the optical series synchronised with the capacitance probe wave crests. Comparison of the resultant wave properties for the various probes is presented in Table 4-3. As discussed within the previous section, the accuracy of the optical LIF method in defining the air-water interface has been calculated at ± 0.12 mm and the uncertainty in defining wave height at 0 to 2.2%. Based on the observed wave heights, this translates to 0 to 0.79 mm and, when combined with the accuracy of surface detection using the root sum of the squares method (eg. Larson and Marx, 1986), gives a total uncertainty of $H_{\text{actual}} = H_{\text{observed}} + 0.80$ to -0.12 mm.

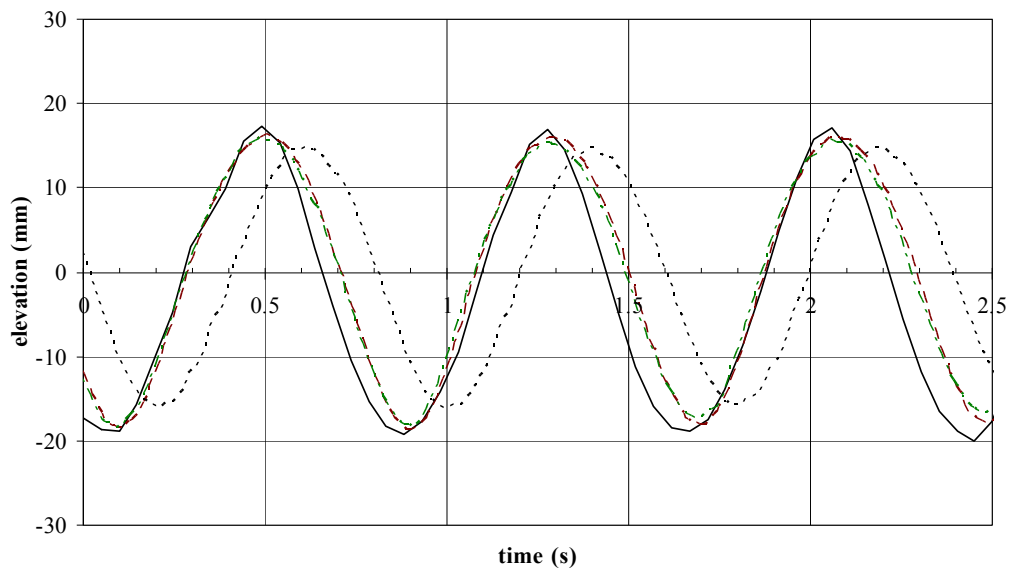


Figure 4-19 Exemplary comparison of capacitance (— — and — ·) and acoustic (··) probes and an extracted optical time series (—).

Table 4-3 Comparison of mean wave heights for a 1.2 Hz monochromatic wave measured by various detection methods.

	Optical	Probe 1	Probe 2	Acoustic
H_{mean} (mm)	35.91 (+0.8/-0.12 mm)	35.14 (± 0.4 mm)	34.67 (± 0.4 mm)	31.85 (± 0.18 mm)
difference from optical method (mm)	-	-0.76	-1.24	-4.05
% difference from optical method	-	-2.1	-3.5	-11.3

From Figure 4-19 and Table 4-3 it can be seen that for this single case, the waves observed using optical method typically have slightly larger amplitude than the wave probes (2 and 3.5%) and significantly larger than the ultrasonic sensor (11%). Manual frame by frame examination of the wave-form in the raw video record (note: without distortion correction applied) gives values within ± 2 pixels or ± 0.5 mm to those obtained using the automated optical method. These are within the uncertainty limits described above and shows robustness in the optical extraction technique. This leaves inaccuracies in the probes or probe calibration or spurious cross-flume waves as the potential sources of difference. In spite of careful spatial alignment of the ultrasonic sensor area, measurements exhibited a uniform phase lag of 0.1 to 0.12 s relative to the capacitance probes.

The difference between the capacitance wire probes and the optical heights slightly exceeds the quoted 0.4 % (± 0.4 mm) probe accuracy. However, lowpass (Chebyshev type) filtering was applied to the probe records to remove high frequency noise. This filtering reduced the average wave heights by around 2% (3% for the ultrasonic) which could partially explain part of the difference. A further potential source of error in the wave probes themselves is the dynamic meniscus effect whereby there is a slight delay in the time taken for the layer of water, which is attracted to the wire by surface tension, to drain away as the water drops into a trough (Kinsman, 1965). It may be suggested that if this layer is not completely drained by the time the wave-form starts to move up the wire again, the probe will not measure the full extent of the trough and the resultant wave height will be smaller than in reality. There is some evidence of this within the above figure (also noted in Kinsman, 1965), with the capacitance probes appearing to display a slight lag after the crest has passed and the surface drops. The larger elevation difference and observed delay between the ultrasonic sensor and the probes/optical is thought to be due to large detection area of the ultrasonic probes. This results in spatial averaging by the sensor at the crest and trough peaks, thus truncating the extremes.

4.2.3 Summary

An improved method for detecting the air-water interface using optical techniques is presented within this section. The method offers improvements in the vertical resolution (0.2 pixels) and accuracies (± 0.45 pixels) at which the air-water interface

may be detected and results in values which are maintained throughout the shoaling, breaking and inner surf-zone regions where conventional probes often become unreliable. The method quantifies and corrects optical and perspective distortions acquired during capture, the geometric shape of the laser sheet and the camera horizon. Additionally, filters including setting a region of interest and despiking are employed to automatically detect and remove erroneous data with limited manual input. In addition to detecting the primary carrier waves, resultant surface profiles show detection of capillary wave-forms less than 1 mm in height and 5 mm in wavelength.

Results were compared with more conventional capacitance wire wave probes and ultrasonic sensors. The optical method is found to predict a wave height between 2 and 3.5% larger than the capacitance probes and approximately 11% higher than the ultrasonic. These results were also compared to careful manual measurement of crest and trough positions using raw video with resultant wave heights found to be within ± 0.5 mm or 1.4% of optical measurements. This indicates that under these short and steep wave conditions, optical methods may offer improvement in detecting the air-water interface over conventional measurement techniques.

Adjacent video series may be combined through best-fit phase matching giving more complete spatial records of surface elevation. Due to the limited temporal resolution of the available imaging device, a maximum phase difference between adjacent series of up to 12.2° may exist. This introduces discontinuities between adjacent series of up to 17 % of wave height (based on an assumed sine wave-form). However, this may be improved by combining multiple wave series, thereby generating a pseudo high-temporal resolution record. In this case, discontinuities may be reduced to a maximum of 4.1° or 6 % of wave height. The limited temporal resolution and resultant phase shift between successive images similarly introduces an uncertainty in defining the absolute crest and trough of a wave-form. This has been estimated at $< 2.2\%$ of wave height in deep water. The error becomes greater near a steep crest prior to breaking and must be evaluated empirically. This is further discussed within Section 6.4.3.

While the use of higher frequency laser and video apparatus would substantially improve the use of optical methods as water surface detectors, increased capture rate in video systems is generally at the expense of resolution. Additionally, the use of a

constant laser or light sheet as opposed to the pulsed sheet would result in substantial blurring of the air-water interface compared to current images. The use of a more extended light sheet in the along-flume direction, with larger overlap between video images and an oblique orientation looking up-flume into the wave-forms would, however, provide improved continuity and verification of records and is suggested in future experimental configurations.

4.3 Time series data

Data in the form of a spatially discrete time series was obtained using both capacitance probes and optical LIF methods. Extraction and processing of these series were largely automated owing to the large number of series to be processed. Several manual check steps were implemented within the processing to ensure integrity of outputs. Numerical routines used to process time series data is provided within Appendix B.

Time series obtained by either of these methods were processed in a similar fashion with the only differences being rate of data capture (capacitance probe series at 125 Hz and optical LIF series at 20 Hz, or 60 Hz if multiple series combined) and the use of vertical reference datums for each method. While the optical LIF series was geo-referenced according to the still water level, capacitance probes exhibit a voltage drift which prohibited the use of such a fixed datum and therefore the series mean was used as a reference datum.

The first step in processing the time series was to determine the horizontal position of the time series within the flume with respect to the intersection of the sloping bed and still water level (x_{swx}) and with respect to the wave paddle (x'). The time series were then statistically processed to obtain maximum (z_{max}) and minimum (z_{min}) elevations and standard deviation.

4.3.1 Water levels and depth

Water levels, and the depth according to these levels, were determined for each time series. Once the x_{swx} position of the series was defined, the depth with respect to still water level (d_{swl}) could be determined from the bed slope (θ) using the relation:

$$d_{swl} = x_{swx} \tan\theta \quad (4-6)$$

Due to the voltage drift inherent in capacitance probes as discussed above, determination of an absolute mean water level (relative to still water level) was only possible for the optical LIF records. Wave group records were first truncated to an integer of the wave group period. This ensured consistency in mean level from series to series. Mean water level was then obtained by finding the arithmetic mean of the truncated series. The depth with respect to mean water level (d_{mwl}) is obtained by similar relationship:

$$d_{mwl} = x_{swx} \tan\theta \quad (4-7)$$

Alternative definitions of water level, and therefore depth, are described within Black and Rosenberg (1992) and summarised in Section 2.5.2.3 and Figure 2-14.

4.3.1.1 Time-varying water level

While the mean water level describes the average surface position over the entire time series, low frequency variations in water level associated with wave groups (Section 2.4) occur throughout the measurement period. These fluctuations are extracted using a Chebyshev Type I lowpass filter. The filter removed frequencies greater than 1 Hz, where the peak wave period is 1.36 Hz. This left only the *infragravity* component of surface fluctuation. While averaging over a mean wave cycle (as discussed by Black and Rosenberg, 1992) can give a similar measurement, this method was not adopted because it is highly dependent on whether zero up or zero down-crossing analysis is used to define the wave cycle.

Examples of lowpass filtered water levels for a wave group propagating onto a sloping bed are included within Appendix C and their relevance to the maximum wave height possible in shallow water discussed within Chapter 7.

4.3.2 Wave properties

While individual values within a time series provide an instantaneous water elevation, this differs from a wave height, which is the difference between the maximum and minimum elevation in a single *wave form*. This wave form is defined by two traverses of an arbitrary threshold. This threshold may be the mean water level of a series, or

alternatively, a still water level or other arbitrary datum. The time between these two traverses is the wave period. The start of a wave-form may be indicated by either a crossing of the datum in an upward direction (zero up-crossing) or in a downward direction (zero down-crossing). While these alternative definitions should return the identical results for Gaussian wave-forms, in reality, results will differ somewhat. While the zero up-crossing method is more widely used in wave analysis, only zero down-crossing has been used within this study. This is because zero down-crossing defines a wave crest relative to the preceding trough which is a more practically applicable definition when considering wave breaking.

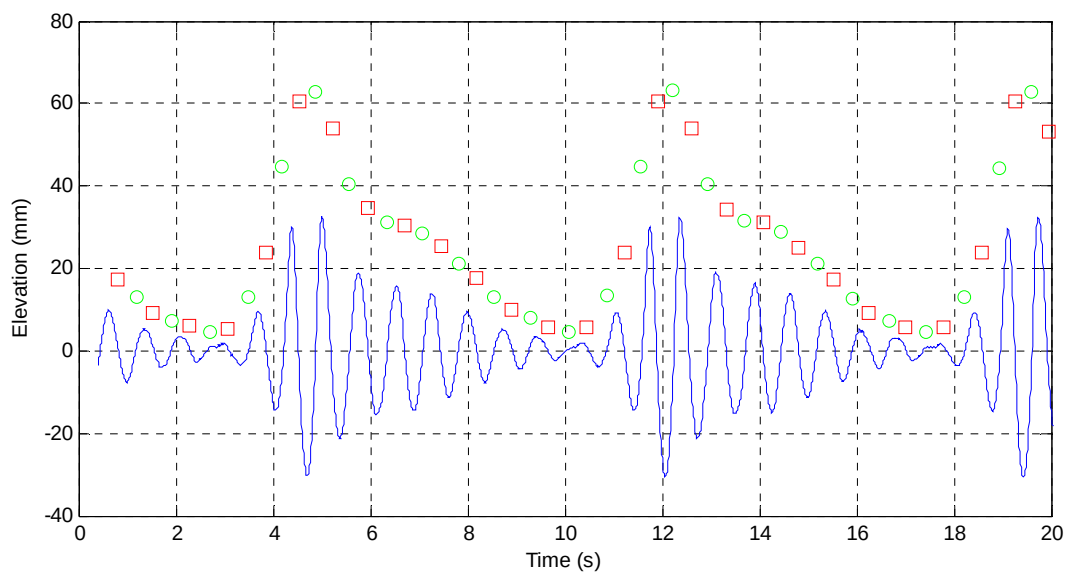


Figure 4-20 A comparison of wave heights computed by zero-up (□) and zero-down (●) crossing for a C2N5r type group in deep water (—).

Waves are defined by zero down-crossing of the mean water level as discussed above, with a lower wave period threshold between sequential down-crossings implemented. This threshold is used to correct erroneous waves which may occur due to higher frequency down-crossings (Figure 4-21). The alternative option is to filter the series so that these irregularities do not occur. However, this tends to severely truncate wave crests and troughs, resulting in under prediction of wave height. When an erroneous wave is detected, the wave-form is combined with adjacent wave forms to produce a corrected wave (Figure 4-22).

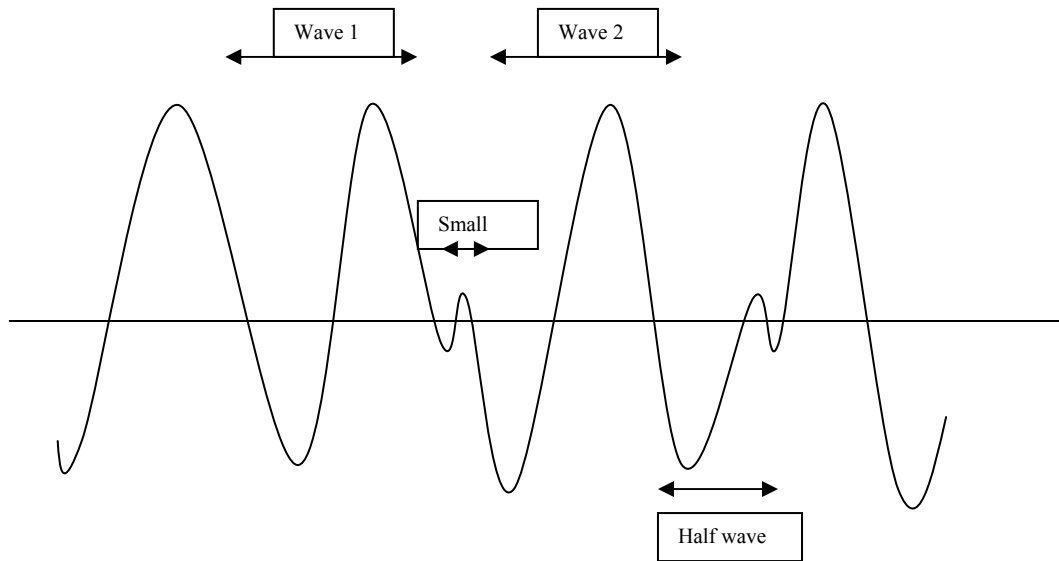


Figure 4-21 Example of possible errors in determining waves by zero down-crossing.

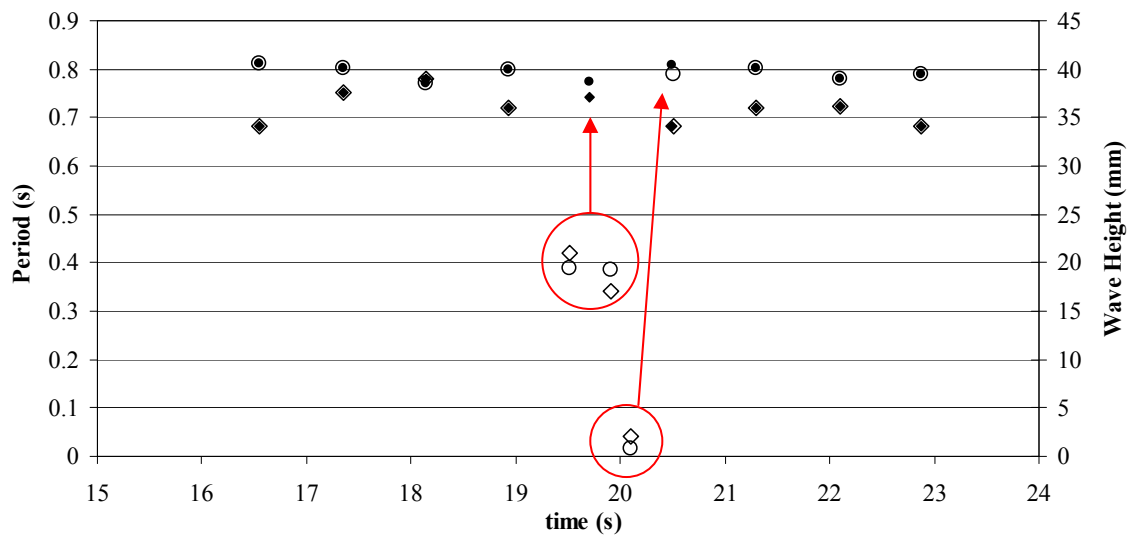


Figure 4-22 Corrections to a wave series extracted by zero down-crossing by using a period threshold. The raw extracted wave height and period are depicted by $\diamond$ and $\circ$ respectively and the threshold corrected height and period by $\blacklozenge$ and $\bullet$.

For each time series, a series of waves is extracted with each wave possessing a:

- *wave time*: defined as the average time (w.r.t. the beginning of the series) between the down-crossings which define that wave;
- *wave period*: defined as the time between successive down-crossings;
- *wave crest and trough elevations*: defined as the maximum and minimum elevations of that wave form respectively;

- *wave crest and trough times*: defined as the time at which the maximum and minimum elevations occurred;
- *wave height*: defined as the difference between maximum (crest) and minimum (trough) elevations within that wave-form.

From the wave height series, a probability distribution of heights is calculated and compared to an idealised Rayleigh distribution. A range of characteristic wave parameters are additionally extracted including:

- *mean wave height*: defined as the arithmetic mean wave height of the series;
- *rms wave height*: defined as the root mean squared wave height of the series;
- *maximum wave height*: as given;
- $H_{10\%}$: defined as the average of the largest 10% of waves;
- H_{sig} : defined as the average of the largest one third (33%) of waves;
- T_{mean} : defined as the arithmetic mean wave period.

Additionally, the energy based significant wave height (H_0) is extracted from the raw time series according to ($H_0 = 4\sqrt{\eta^2}$) where η is the instantaneous surface elevation.

The difference in wave crossings between adjacent time series allows calculation of wave celerity. These may be time series from adjacent capacitance probes, or time series extracted from the same optical LIF profile series, both separated by a distance x . The averaged *wave time* is used giving average wave celerity rather than a crest or trough-specific celerity.

4.3.2.1 Wave height index

The ratio of wave height to depth is simple to calculate once the exact definition of each is defined. Wave height may be either characteristic wave height such as H_{max} , H_{sig} or H_{m0} or a breaking wave height, H_{br} . The precise definition of breaking (Section 2.3.2) affects the location at which breaking wave height is measured. The depth may be similarly defined according to a number of definitions (Section 2.5.2). The resultant breaker indices using these differing definitions are assessed and compared to design guideline values.

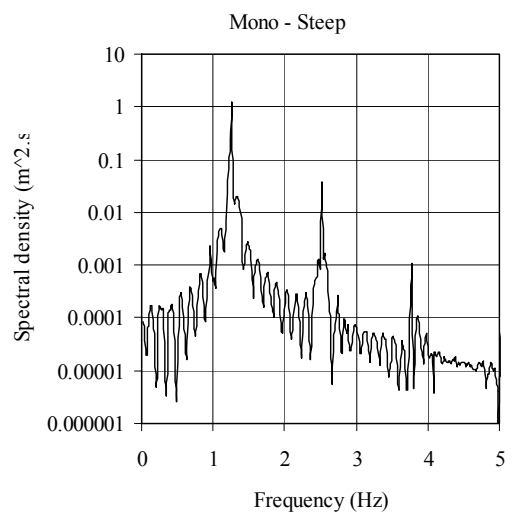
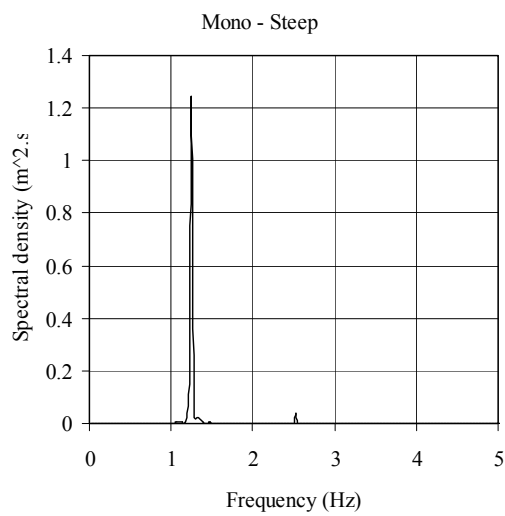
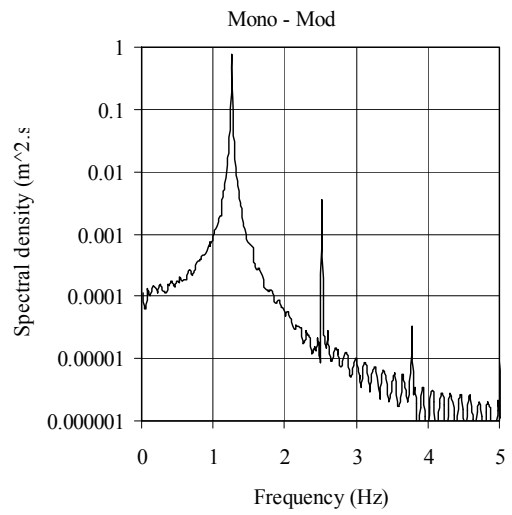
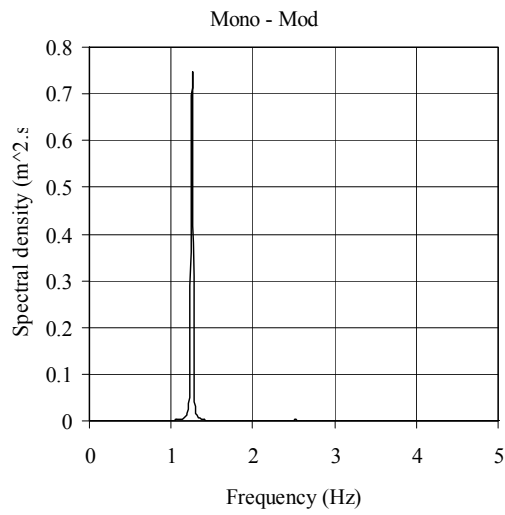
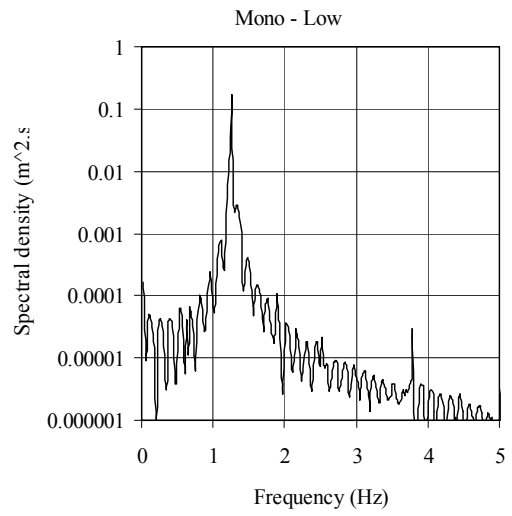
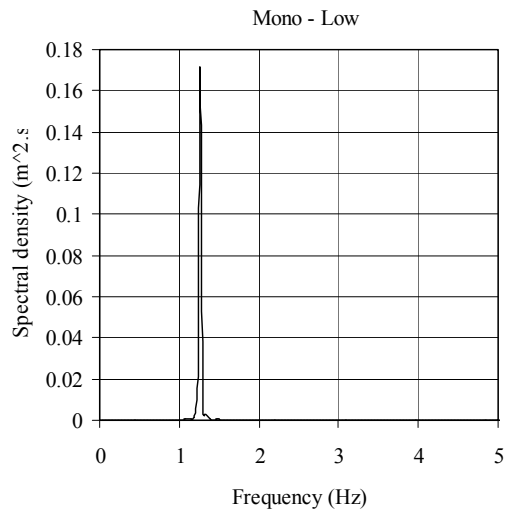
4.3.3 Spectral analysis

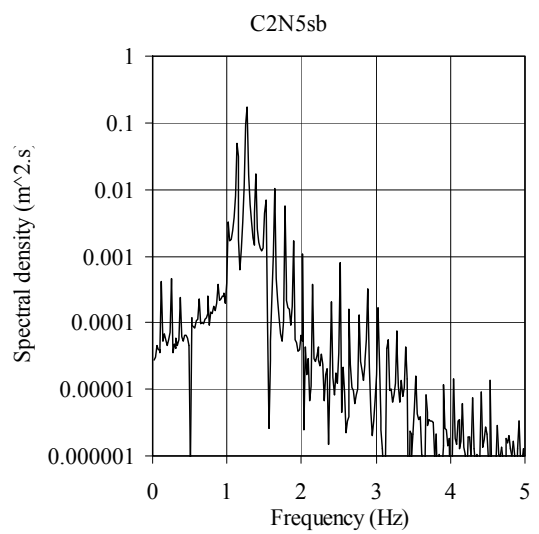
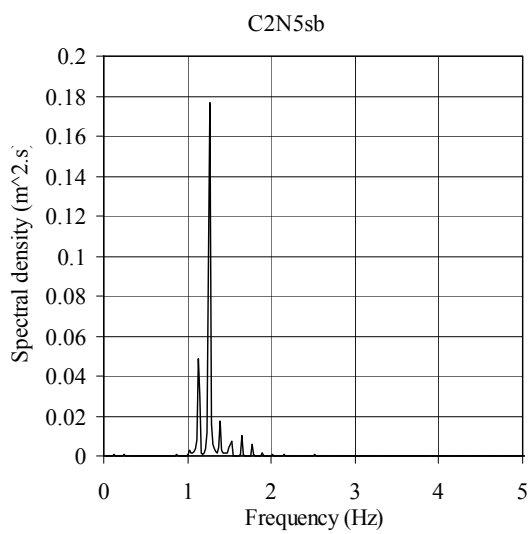
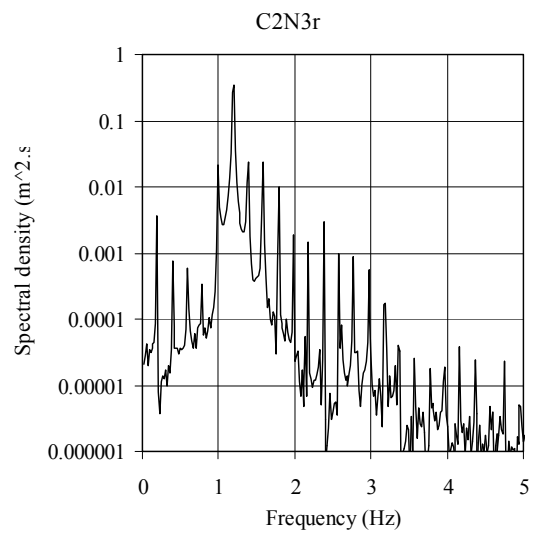
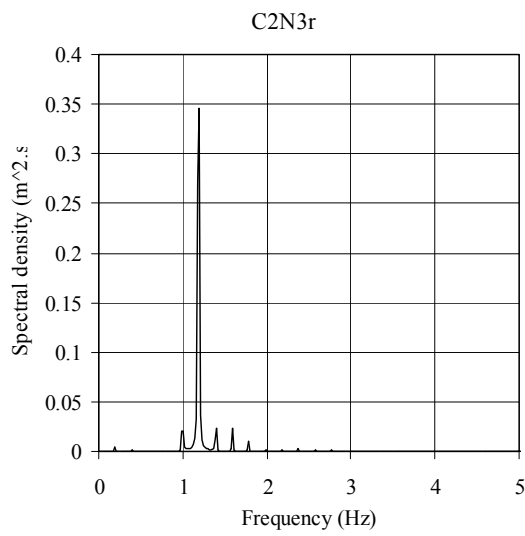
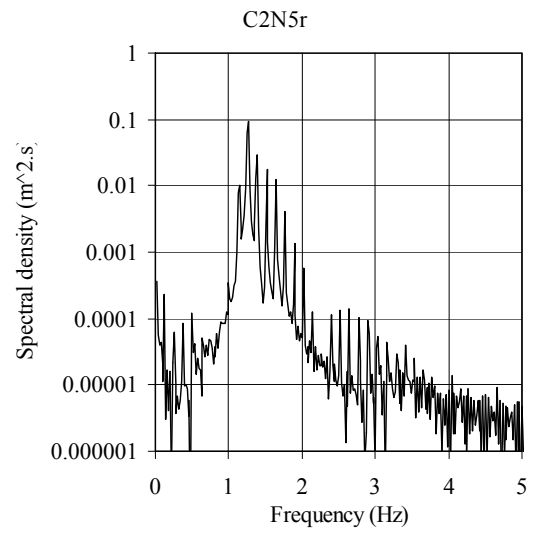
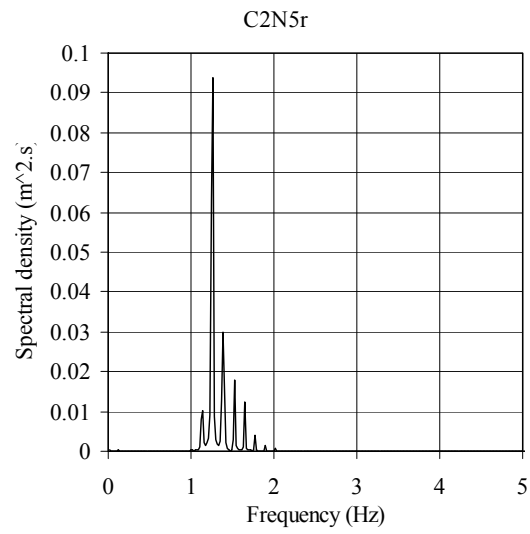
The use of spectral analysis is a powerful tool to quantify the amount of energy present at differing frequencies within a series. The time series is transformed to the frequency domain by approximating the series as the sum of a series of sinusoidal waves at varying frequency. An assumption in doing so is that the original time series is Gaussian distributed. While this assumption is violated, particularly during the shoaling and breaking processes, the analysis none-the-less provides useful guidance.

The time series is converted to a frequency-energy spectrum using a Fourier transformation. This spectrum presents the energy density contained within the series for each frequency interval (Figure 4-23). The Fourier analysis is undertaken using a Matlab in-build fast Fourier transform (fft) algorithm (Duhamel and Vetterli, 1990). When using a Fourier transform, results must agree with Parseval's theorem which states that the energy in the time domain (a) equals the energy in the frequency domain (f). This is evaluated by the relationship (Kaplan, 1992):

$$\frac{1}{\pi} \int_{-\pi}^{\pi} [f(x)]^2 dx = \frac{1}{2} a_0^2 + \sum_{n=1}^{\infty} (a_n^2 + b_n^2) \quad (4-8)$$

Comparisons of the sum of energy in the time and frequency domains for three monochromatic and four group classes are presented in Table 4-4. These comparisons show the calculated energies to be near identical indicating the series agree with Parseval's theorem and reliability in the FFT fit.





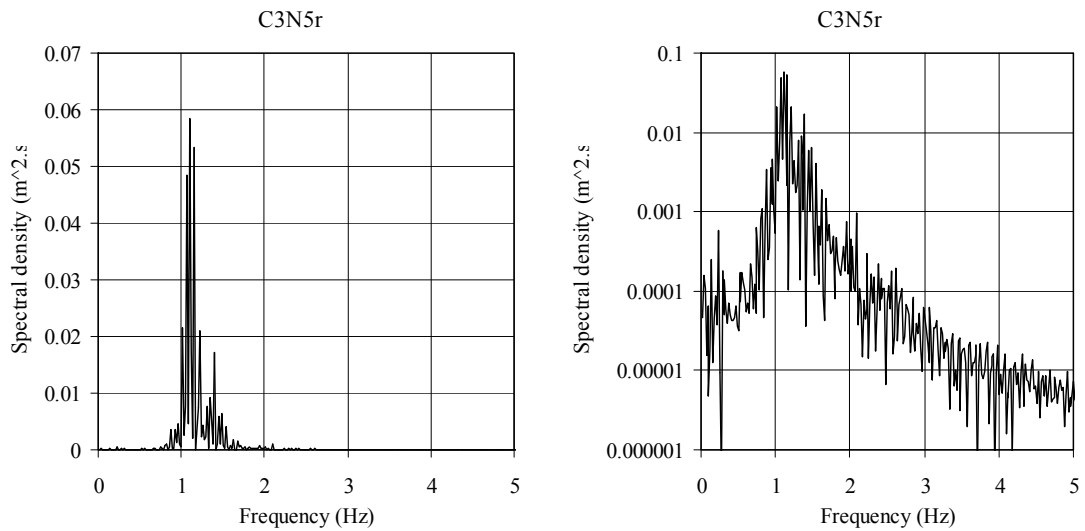


Figure 4-23 Energy spectra for low, medium and steep monochromatic waves and C2N5r, C2N3r, C2N5sb and C3N5r class wave groups plotted on both linear and log axis.

Table 4-4 Comparison of the sum of energy in the time and frequency domains for the series plotted within Figure 4-23.

Wave class	C2N5r	C2N3r	C2N5sb	C3N5r	Mono - low	Mono - mod	Mono - steep
Frequency domain	0.000125	0.000265	0.000161	0.000085	0.000097	0.000404	0.000679
Time domain	0.000126	0.000265	0.000161	0.000085	0.000097	0.000404	0.000680

4.3.4 Experimental errors and uncertainties

4.3.4.1 Incident wave reflection

Reflection of incident wave energy may occur because of or owing to bathymetric changes (Whalin, 1972), the breaking process (Rapp and Melville, 1990), or reflection off the shoreline itself (Seelig and Ahrens, 1981). In the most extreme case of shoreline reflection - a vertical wall, the reflected wave (H_r) is near 100% of the incident wave (H_i). Battjes (1974) found reflection on flatter slopes to depend on the surf similarity parameter (Section 2.3.2.2), which is a function of beach slope and offshore wave steepness. Seelig and Ahrens (1981) developed curves to predict wave reflection off a smooth slope as a function of these above properties and found reflection to decrease as the wave steepness increased or beach slope decreased.

Wave reflection due to a bathymetry step was quantified by Rosseau (1952) but is negligible for more subdued changes in bathymetry. While reflection associated directly with wave breaking is poorly described within the literature, secondary waves of small amplitude and higher frequency can often be observed propagating in both the landward and seaward directions following breaking in both the laboratory (Rapp and Melville, 1990) and field. This energy is typically very small compared to the incident wave energy and in most cases is negligible.

Quantification of reflected waves requires separation of the incident and reflected wave components. A number of methods are available to achieve this including the direct measurement method of Healy (1953), or spectral techniques of Goda and Suzuki (1976) and Mansard and Funke (1980). These methods are discussed in detail within Tajziehchi (2006) and are presented briefly below:

- Healy (1953): Healy uses a simple comparison of the maximum ($H_{\max}$) and minimum ($H_{\min}$) wave heights recorded at a location offshore of the reflective structure. The coefficient of reflection, K_r , is determined as

$$K_r = \frac{H_r}{H_i} = \frac{H_{\max} - H_{\min}}{H_{\max} + H_{\min}} \quad (4-11)$$

However, the method has been developed specifically for regular waves and is therefore not applicable to the wave group component of this study.

- Goda and Suzuki (1976): The signal is resolved into the reflected and incident components using Fourier analysis of two time series separated by a distance Δl . The distance Δl is selected so that $\Delta l / L_0 \neq n / 2$, where $n = 1, 2, \dots$ and L_0 is the deep water wavelength. The distances $0.45 < \Delta l / L_0 < 0.6$ are specifically excluded and the distance $0.05 < \Delta l / L_0 < 0.45$ recommended as providing the best estimate. Amplitudes at the peak energy frequency are evaluated, and higher harmonics are excluded.
- Mansard and Funke (1980): This method has become the preferred method of industry, with many developers of wave generation and measurement software incorporating this technique of reflection analysis into their software packages.

Three time series at spacing Δl_{12} and Δl_{23} are required, with the authors making spacing recommendations for improved accuracy as follows:

$$\Delta l_{12} = L_0/10$$

$$L_0/6 < \Delta l_{13} < L_0/3$$

$$\Delta l_{13} \neq L_0 / 5$$

$$\Delta l_{13} \neq 3L_0 / 10$$

Energy spectra are obtained for each series and a least squares method used to separate incident from reflected energy.

Tajziehchi (2006) compared the methods using statistical measures of bias, RMS error and regression coefficient and found both the Goda and Suzuki (1976) and Mansard and Funke (1980) methods to provide a better indication of incident wave height than the Healy (1953) method. Additionally, the Mansard and Funke (1980) method was found to be less sensitive to higher harmonic waves present within the flume.

Measurements from an array of capacitance probes with spacing Δl_{12} and Δl_{23} of 75 and 80 mm respectively were used to quantify the reflection of the three monochromatic and four group classes for the 5:1, 10:1 and 20:1 sloping beds. All three methods of quantifying reflection as described above were used, with results compared to the empirical based reflection coefficients of Seelig and Ahrens (1981). Results are presented in Table 4-5.

Due to the small size of the reflected waves (1 to 9 mm with average of 2.6 mm) compared to the ± 0.5 mm accuracy given for the capacitance probes (RBR, 2007), results are subject to maximum uncertainty of ± 5 to 50%, with a mean 25%. Results indicate firstly that the spectral methods of Goda and Suzuki (1976) and Mansard and Funke (1980) both predict higher reflection coefficients for the monochromatic waves than the method of Healy (1953). For both the monochromatic and grouped waves, the calculated reflection coefficient for the Goda and Suzuki (1976) method is also higher than the Mansard and Funke (1980) coefficient. Tajziehchi (2006) found the method of Mansard and Funke (1980) to provide a better estimate of reflection than either of the other methods, particularly when spectra contain secondary harmonic waves. These harmonic waves become increasingly pronounced in steeper wave-trains, as discussed

in the following section. Table 4-5 shows the difference between the reflection coefficients calculated by the Goda and Suzuki (1976) and Mansard and Funke (1980) to be larger for steeper wave-trains, thus supporting the findings of Tajziehchi (2006).

Table 4-5 Measured wave reflection coefficients ($K_r = H_r/H_i$), using three theories, compared to empirical values for differing wave characteristics on a 5:1, 10:1 and 20:1 slope bed.

Case	Measured			Empirical
	Healy (1953)	Goda & Suzuki (1976)	Mansard & Funke (1980)	Seelig and Ahrens (1981)
5:1 Slope				
Mono Low	0.029	0.069	0.066	0.109
Mono Mod	0.019	0.072	0.050	0.055
Mono Steep	0.052	0.081	0.067	0.042
C2N5r	-	0.079	0.068	0.108
10:1 Slope				
Mono Low	0.024	0.026	0.033	0.029
Mono Mod	0.026	0.095	0.055	0.016
Mono Steep	0.036	0.165	0.075	0.012
C2N5r	-	0.032	0.030	0.026
C2N3r	-	0.082	0.076	0.019
C2N5sb	-	0.059	0.047	0.023
C3N5r	-	0.078	0.053	0.019
20:1 Slope				
Mono Low	0.027	0.051	0.037	0.007
Mono Mod	0.022	0.078	0.060	0.002
Mono Steep	0.043	0.117	0.089	0.000
C2N5r	-	0.053	0.037	0.008

Results were compared to the empirical based reflection coefficients according to Seelig and Ahrens (1981). However, the relationship of lower bed slope and higher wave steepness inducing lower reflection coefficients as predicted by Battjes (1974), was not evidenced within the laboratory data, with steeper monochromatic waves tending to exhibit higher coefficients regardless of method. Additionally, while the reflection coefficients for the 10:1 slope were lower than for the 5:1 slope - in agreement with Battjes (1974), they were also lower than for the 20:1 slope.

4.3.4.2 Secondary harmonic waves

The presence of free secondary harmonic waves within a flume may be caused by inexact wave generation at the paddle. This occurs due to the sinusoidal motion of a wave paddle, which results in orbital motions that are not an exact representation of an actual wave form. Therefore, secondary harmonic waves may be generated. These

waves appear superimposed on the primary wave-form, yet propagate freely and at a different velocity to the primary waves. Accordingly, the combined surface of primary and secondary waves is transitional, varying in time and space along the flume (Hughes, 1993). The generation of unwanted free, secondary harmonic waves is reduced using a wavemaker which produces orbital velocities more closely approximating those of real waves, such as the flexible, cantilevered type wavemaker described in Section 3.3.2.

Energy at harmonic frequencies to the spectral peak (around 2.5 Hz) is evident for all monochromatic wave structures tested due to non-linearity inherent in propagating wave-forms (Figure 4-23), although more pronounced in the high steepness series. The energy contained within secondary harmonic waves for the low, moderate and steep monochromatic cases are 0.03%, 1.02% and 6.44% of that at the peak frequency respectively indicating that less nonlinearity in the lower steepness series. Energy at harmonic frequencies for the wave group structures tested was found to be 0.49%, 1.87% and 0.98% of the spectral peak energy respectively for C2N5r, C2N5sb, C2N3r groups. The class three type group is more complex due to the frequency modulation within the group. The secondary harmonic of the peak frequency contains 3.02% the energy of the spectral peak.

The growth of higher order harmonics within the wave spectrum becomes more pronounced during the shoaling process (Flick et al., 1981) as the development on nonlinearity increases. Examples of the growth of these harmonics for a wave group propagating up a uniform slope are given in Appendix C-1.

4.3.4.3 Low frequency energy

In addition to the free secondary harmonic waves induced by wavemaker motion, as described above, spurious energy may also be present at low frequency ranges. While group-bound long waves are expected to be present (Section 2.4), reflection of long waves at the beach slope may contaminate the experiment as they cannot propagate ‘offshore’, as in the natural environment, but rather, are reflected from the paddle and return as secondary waves.

This energy may be examined and quantified according to the energy spectra for the various wave conditions (Figure 4-23). Long wave energy associated with the Case 2

type groups occurs at frequencies of 0.125 to 0.2 Hz (or $T_g = 5$ to 7.5 s) depending on exact group class. Based on the earlier calculated spectra, energy at this frequency is 0.05%, 0.07% and 0.14% of the peak spectral energy for the C2N5r, C2N5sb and C2N3r class groups respectively and 0.15 % for the C3N5r group (Table 4-6). This energy can be converted to an equivalent amplitude by calculating the root of the integrated area beneath the spectrum (Eqn. 4-15).

$$a = \sqrt{\int_{f_{\min}}^{f_{\max}} S(f)} \quad 4-15$$

Wave amplitudes at the group frequency are 2.3, 2.7 and 3.8% of the peak spectral amplitude for the C2N5r, C2N5sb and C2N3r class groups and 3.8 % for the C3N5r group. Energy at lower frequencies, as may occur due to seiching within the flume, are one to two orders of magnitude below that at the group frequency.

The expected group-bound long wave amplitudes due to the gradient in radiation stress associated with the groups may be calculated according to Longuet-Higgins and Stewart, 1964 (L-H&S64) as discussed within Section 2.4 and presented in Eqn. 2-31. The calculated bound long waves are compared to incident waves (Table 4-6) with the resultant bound wave amplitudes varying between 0.8 and 1.5% of the incident. These magnitudes are roughly 30 to 40 % of those calculated according to the energy spectra. While the long wave amplitudes are approaching the instrumentation limits, and thus caution should be placed on their evaluation, the difference may indicate some reflection from the beach is occurring.

Table 4-6 Comparison of group-bound long wave to incident wave amplitude according to Longuet-Higgins and Stewart (1964) and comparison of peak spectral to long wave energy.

Group	Peak energy (m2.s)	Equiv amp (mm)	Energy at long wave freq (m2.s)	Equiv amp (mm)	long wave energy - % of peak	long wave amp - % of peak	Group Hmax (mm)	L-H&S64 theoretical bound wave amp (mm)	Bound long wave amp - % of Hmax
C2N5r	0.2174	62.4	0.00012	1.4	0.05	2.32	54.5	0.45	0.83
C2N3r	0.6042	104.0	0.00085	3.9	0.14	3.76	71.5	0.77	1.08
C2N5sb	0.3265	76.4	0.00023	2.0	0.07	2.67	63	0.70	1.12
C3N5r	0.2779	70.5	0.00041	2.7	0.15	3.82	90.05	1.35	1.50

Measured deep-water long-wave amplitudes of 2 to 4 % of incident wave height agree with the 2 to 5 % as noted within Banner and Peirson (2007). The lack of energy at

frequencies lower than the group frequency indicates effects of wind-induced or other seiching within the flume are negligible.

4.3.4.4 *Lateral sloshing*

Comparison of the wave heights as determined from two probes spaced at 80 mm and aligned perpendicularly to the direction of incident wave propagation (across-flume – refer Section 4.2.2.4) indicate a difference of 0.4 to 1.1 mm and RMS difference of 0.87 mm. This agrees with the 1 to 2 mm noted within Banner and Peirson (2007) for the same wave flume, although modifications to the glass sidewall joints (smoothing using a plastic tape) may be responsible for the improvement.

4.3.4.5 *Frictional Losses*

Frictional losses occur due to viscous forces induced by the flume side-walls on propagating wave-forms. The total mean energy density averaged over a wave packet, $\langle E \rangle$, has been used to quantify this attenuation divided by $\rho_w g$, may be estimated in deep water by the relationship:

$$\langle E \rangle = \sum \frac{\eta^2}{n} \quad (4-16)$$

where η is the root mean squared surface elevation, ρ_w and g are the water density and gravitational acceleration.

While mean energy losses in propagating deep water wave groups can be attributed to both the frictional effects of the sidewall and wave breaking effects, losses associated with recurrent-type wave cases, where breaking does not occur, are a function of viscosity coupled with surface and sidewall effects (Banner and Peirson, 2007). These cases were used within their study to identify background ‘reference’ levels of frictional attenuation. Of the four wave groups tested within the current study, only the C2N5sb case exhibited wave breaking in deep water. Energy losses induced by the flume sidewall interactions may therefore be quantified for the other three group cases (Figure 4-24).

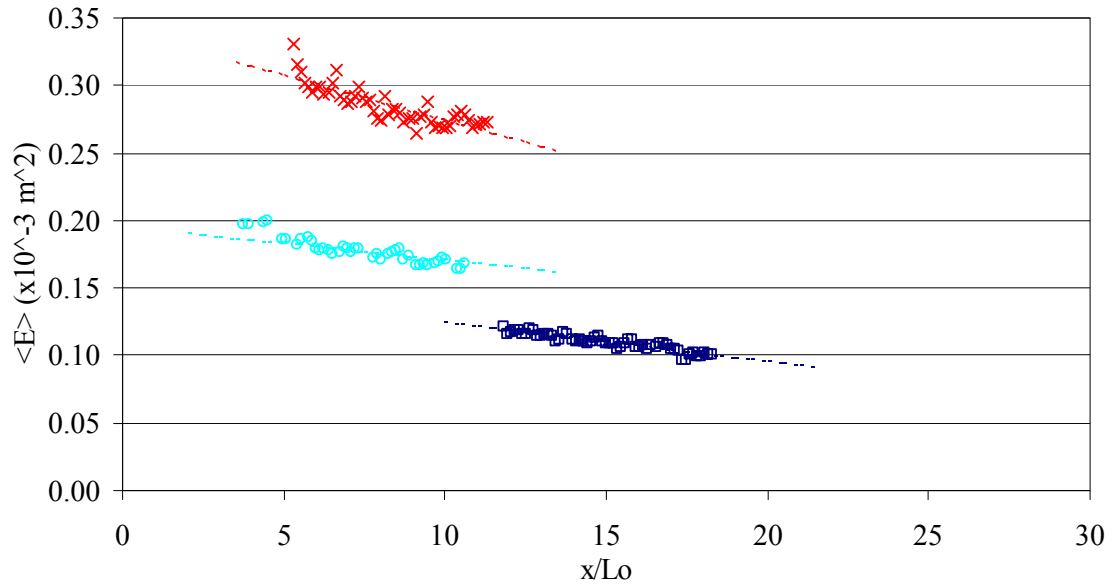


Figure 4-24 Evaluation of background viscous energy losses by recurrent type wave groups in deep water. Symbols represent the mean measured energy $\langle E \rangle$ and dashed lines the least-squares best fit with C2N5r = $\square$ and $--$, C2N3r = $\times$ and $--$ and C3N5r = $\diamond$ and $--$.

Banner and Peirson (2007) found the background viscous losses as a function of fetch were best estimated using a log-normal, least-squares fit. Applying this methodology, viscous losses for each of the three recurrent group classes may be quantified according to

$$\Delta \langle E \rangle = e^{a \left(\frac{x_1}{L_0} \right) + b} - e^{a \left(\frac{x_2}{L_0} \right) + b} \quad (4-17)$$

Where $\Delta \langle E \rangle$ is the viscous energy loss between positions x_1/L_0 and x_2/L_0 and a and b are coefficients dependent on the group class. These coefficients are presented for each of the recurrent group classes within Table 4-7 along with the 90% confidence interval (90% CI) and standard error of fit (S.E).

Table 4-7 Coefficients describing changes in energy with fetch for the recurrent-type groups

Group	C2N5r	C2N3r	C3N5r
a	-0.0268	-0.023	-0.0138
b	-8.7173	-7.9737	-8.5403
90% CI	0.001	0.001	6.32E-04
S.E	0.0291	0.027	0.02

Viscous losses for the C2N5r class groups are in good agreement with those found by Banner and Peirson (2007) using a similar group structure and comparable to the decay predicted by Ursell's (1952) linearized theory (Tulin and Waseda, 1999). The shorter

wave packet of the $N = 3$ class appears to experience slightly higher viscous losses, while the Case 3 group experiences slightly lower.

Banner and Peirson (2007) found viscous energy losses of about 1.08% per wavelength or around 0.5% of wave height per wavelength. By comparison the present study found energy losses between 1.3 and 1.95% per wavelength and wave height losses between 0.8 and 1.5% per wavelength. Reasons for this difference are not immediately apparent but may be related to degradation of the glass sidewall seals with time or the accumulation of deposits and films on the glass sidewall surface itself.

4.3.5 Output summary

Data extracted from time series analysis include the following characteristics

Table 4-8 Characteristics extracted for individual time series.

Derived from time series position	Time series analysis	Statistical analysis of wave data	Spectral analysis values
Position from swx (x_{swx})	Surface elevation series (η)	Maximum wave height (H_{max})	Spectral significant wave height (H_{mo})
Position from wave paddle (x')	Lowpass filtered surface elevation ($\eta_{lowpass}$)	Crest elevation of maximum wave (C_{max})	Equivalent monochromatic wave height (H_e)
Depth from SWL (d_{swl})	Mean water level from SWL (η_{mean})	Trough elevation of maximum wave (T_{max})	Spectral significant period (T_s)
	Maximum series elevation (η_{max})	Mean wave height (H_{mean})	Spectral width (ν)
	Minimum series elevation (η_{min})	RMS wave height (H_{rms})	Spectral width (κ)
	Depth from MWL (d_{mwl})	3% wave height ($H_{3\%}$)	
	Kurtosis	10% wave height ($H_{10\%}$)	
	Skewness	Mean period (T_{mean})	
	Standard deviation (σ)	Median wave celerity based on adjacent series (c) ¹	
	Variance (σ^2)	Maximum wave-cycle averaged mwl (mwl_{wc-max})	
	Energy based significant wave height (H_0)	Minimum wave-cycle averaged mwl (mwl_{wc-min})	
		H_{max}/d_{swl}	
		H_{max}/d_{mwl}	
		H_{BP}/d_{swl}^2	
		H_{BP}/d_{mwl}^2	

¹ Determined based on phase difference of waves extracted from adjacent series

² Only applicable for the series at positions of break point

The length-based characteristics are non-dimensionalised by the deep water peak spectral wavelength (L'_0) calculated according to linear theory.

The development of these characteristics with distance along the wave flume is observed by creating either a video series of sequential characteristic data or by plotting the characteristics as a function of horizontal distance. Additionally, the merged series of surface elevations and lowpass surface elevations are combined into single space-time plots as described within Section 2.3.1.4. The energy spectrum is combined similarly as a function of both space and frequency. Examples of these outputs for a Case 2, $N = 5$, recurrent wave group propagating on a plane 10:1 slope bed with x_0 at $x'/L_0 = 16.65$ from the paddle are given in Appendix C.

Comparisons of the evolution of specific characteristics for differing slope locations (i.e. altering the spatial phasing of wave group evolution and shoaling) may be subsequently undertaken. These comparisons are provided and discussed within Chapter 6.

4.4 Wave breaking

4.4.1 Optical LIF imagery

Profile data is obtained along small (~ 100 mm) sections using the optical LIF method. These sections, while relatively short compared to wavelength, can provide highly accurate spatial data such as wave crest shape, break point location, wave face steepness at break point and wave breaking type. Examples of these profile sections are given in Figure 4-12. However, the complete geometric description of overturning crests was not described, but rather only the upper edge.

Wave break point is defined manually for each wave within the group. While there are numerous definitions of break point (Singamsetti and Wind, 1980), the point at which the front wave face first becomes vertical has been used as a practical definition as recommended by Smith and Kraus (1990). Due to the $24^\circ.4$ wave phase shift between sequential frames, the exact break point is not captured every wave. However, due to the repeatable nature of the generated wave groups, multiple images of the 'same' breaking wave can be observed and the break point defined through either a singular

image that best defines the break point, or interpolation between two or more near-break point images.

4.4.2 Spatially extensive imagery

To avoid the synchronization problems associated with data merging of the optical LIF images, more spatially expansive video of the breaking process was also captured. Collection of this video data, referred hereafter as *wave breaking video*, is discussed within Section 3.4.4. Due to its relatively rapid acquisition time, data could also be collected for a greater number of test configurations than was possible with the optical LIF method.

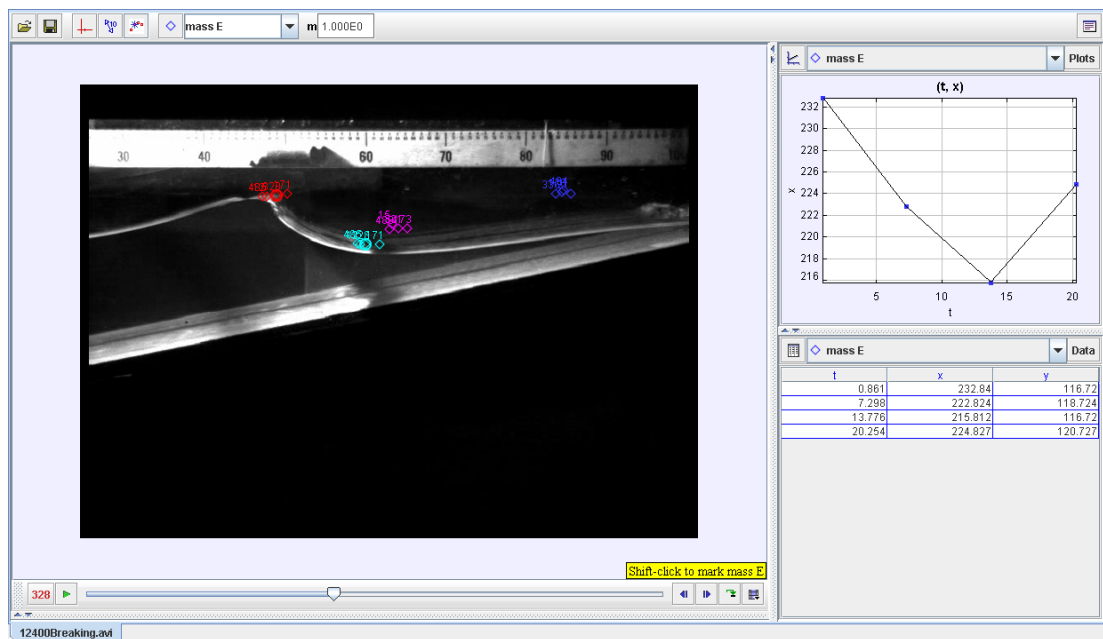
Calibration of the *wave breaking video* is less precise than for the optical LIF images, with an in-situ ruler placed in mid-flume providing image scale. No image processing to correct for optical or perspective distortion is applied. For each series of images, known positions along the scale ruler (with respect to the swx) are manually digitized. This provides scale and a georeference to the images. Resolution is approximately 1.1 mm/pixel.

Break points, as identified by the point at which the wave front becomes vertical, are defined manually using frame stepping and digitizing software (Brown, 2007). An example of defining the various features of the breaking process, including the maximum crest and minimum trough position at break point ($x_{BP_{crest}}$, $y_{BP_{crest}}$) and ($x_{BP_{trough}}$, $y_{BP_{trough}}$), the lip impact position ($x_{BP_{impact}}$, $y_{BP_{impact}}$) and the maximum splash-up position, ($x_{BP_{splash}}$, $y_{BP_{splash}}$) is shown in Figure 4-25.

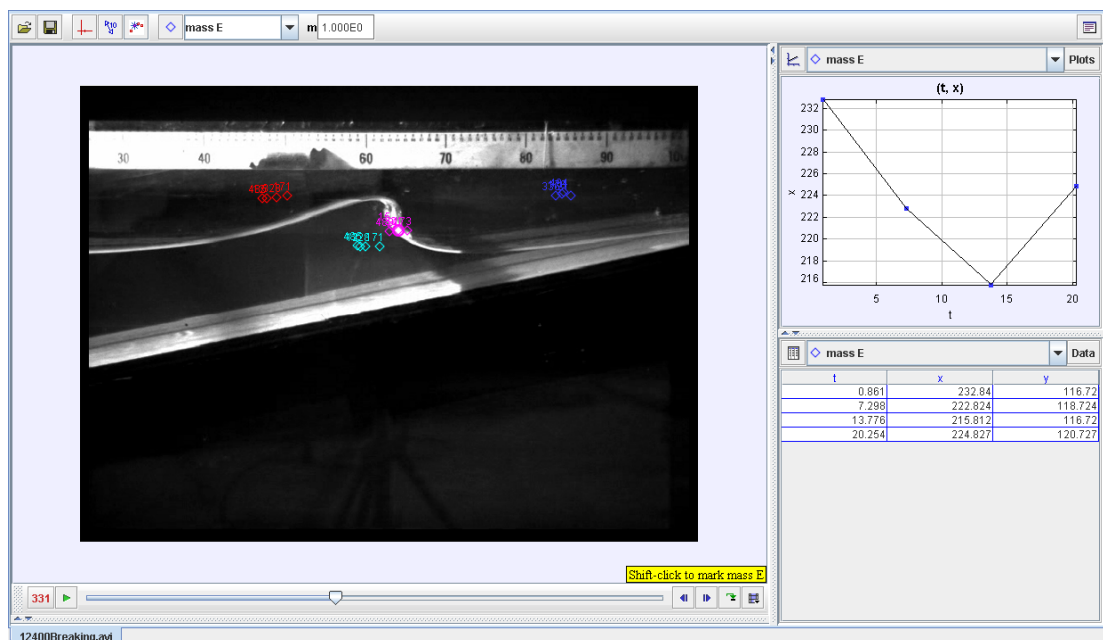
Additionally, a qualitative breaker type (λ_{br}) is assigned to each breaking wave. The standard classifications of spilling, plunging, collapsing and surging (Galvin, 1968) were found to be inadequate during the initial stages of investigation with a more continuous spectrum of breaker types noted. A similar conclusion was reached by Weishar and Byrne (1978) during their field comparisons of breaker type to established predictive formula and also by Sayce (1997) in a study of plunging waves. A 1 – 10 scale of breaking type was therefore established based loosely around the Galvin (1968) classification of spilling, plunging, collapsing and surging breaker types. A value of 1 corresponded to a very gentle, incipient-type spilling wave and a value of 10 to breaking

by a surge from the wave base with a continuum of breaking types between. Descriptions of these types 1 – 10 are presented within Table 4-9. Waves which also break with a lip projected forward from the wave face (Types 4 to 8) form a vortex (Figure 4-26) which may be quantified according to the vortex length to width ratio (Mead and Black, 2001). While this vortex ratio was not directly measured when assigning breaker type, the designated breaker types tend to exhibit a strong relationship.

A



B



C

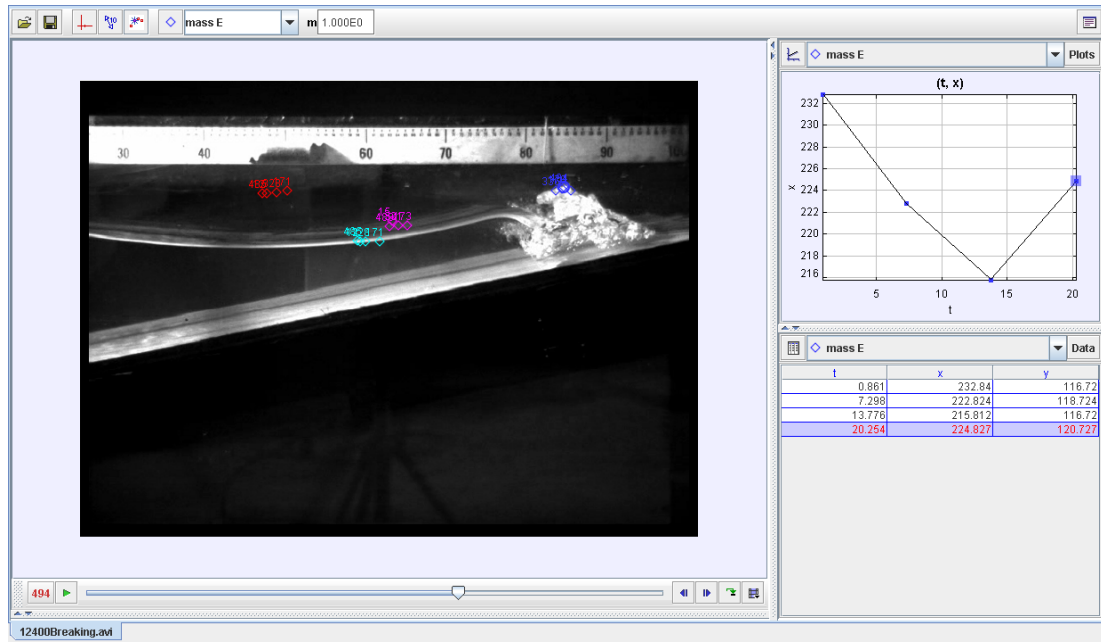


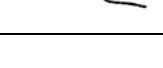
Figure 4-25 Digitization of the wave crest (red) and trough (aqua) at break point (A), the plunging jet impact point (B – purple) and maximum splash up position (C – blue).

Problems with defining exact break point due to the wave phase shift between sequential frames (10.5°) as described within the previous section remain. A similar solution is employed with multiple images of the ‘same’ breaking wave observed and the break point defined for each. If the captured frame does not depict the exact break point, the point was interpolated from sequential images and the results averaged. Additionally, while efforts are made to select the portion of the wave-form corresponding to the centre of this flume (away from sidewall effects and in line with the in-situ ruler), this is approximate and some error may be introduced. Overall, the accuracy of defining these data is estimated at ± 5 mm. Data obtained from each wave include:

- horizontal wave breaking position from the swx ($x_{BP_{crest}}$)
- a spatial measure of wave height at break point ($H_{br-s} = y_{crest} - y_{trough}$)
- wave breaker index based on swl at $x_{BP_{crest}}$ (H_{br-s}/d_{swl})
- trough to crest steepness at break point ($S_{br-s} = y_{crest} - y_{trough}/x_{crest} - x_{trough}$)
- a plunge distance ($X_p = x_{crest} - x_{impact}$)
- splashup height above trough elevation ($Y_s = y_{splash} - y_{trough}$)
- breaking type (λ_{br})

These data enable a variety of relationships to be tested and also allow comparison with the optical LIF results.

Table 4-9 Scale of breaker types.

No.	Name	Description	Cross-section	Vortex length to width ratio ¹
1	Incipient	- Smooth water surface disturbed, - Leaves small 'surface wake' waves - Wave-form largely unchanged		-
2	Weak spill	- Breaking by spill initiated at the very top of wave and progressing slowly down face		-
3	Spill	- May be slight lip impacting on top 1/4 of wave face and spill progressed down face more rapidly		> 3.5
4	Hard spill	- Initial lip/spill may impact up to 1/2 way down wave face and progresses quickly to trough - Lack of defined vortex during break		3.25 – 3.5
5	Weak plunge	- Thin lip throws and impacts between 1/2 and 3/4 way down face - Small, elliptical vortex - Splashup below breaking wave H		3.0 – 3.25
6	Plunge	- Lip impacts between 3/4 way down face and wave base - Splash-up as high as breaking H		2.5 – 3.0
7	Strong Plunge	- Substantial lip impacts well out from wave base - Splash-up higher than breaking H - Breaking vortex near circular		2.0 – 2.5
8	Very strong plunge	- Thick lip (substantial in comparison to H) impacts further than the breaking wave H - Possibly deformations (steps) in wave face		< 2.0
9	Collapse	- Breaking initiated on the wave face and wave collapses		-
10	Surge	- Wave shoals but then surges forward from near the base of the wave face		-

¹ after Mead and Black (2001)

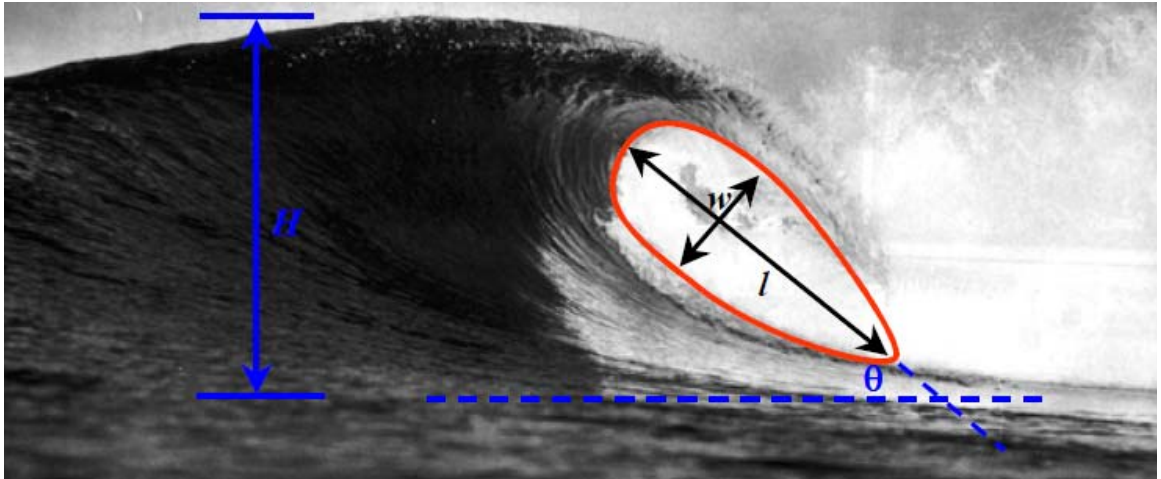


Figure 4-26 A breaking wave of hight H , with vortex length (l), width (w) and angle (θ) (source: Mead and Black, 2001).

PART 1: WAVES ON PLANE SLOPES

This part of the thesis presents the results and discussion of experiments undertaken on a plane sloping bed.

Within this part, Chapter Five presents results and discussion associated with monochromatic experiments, Chapter Six presents the fluid dynamic aspects of the group influence on shoaling and breaking water waves and Chapter Seven provides results and discussion on the implications of wave groupiness on structural design in the nearshore.

Chapter 5 Shoaling and breaking of monochromatic waves

5.1 Introduction

While the primary motivation behind the present study has been the analysis of wave groups and their influence on the shoaling and breaking processes, preliminary testing using monochromatic waves has been undertaken. Monochromatic wave trains are characterized by continuous, homogeneous waveforms. Testing using monochromatic waves enables significant simplification compared to irregular or grouped testing. This is particularly useful when using laboratory testing to verify theoretical work.

Monochromatic testing has, in this case, been used primarily to compare with empirical expressions for the prediction of breaker index presented within modern design guidelines. Such guidelines include the *Coastal Engineering Manual* (USACE, 2003) and *Random Seas and Design of Maritime Structures* (Goda, 2000). These guidelines use design curves derived from original studies by Weggel (1972) and Goda (1970) respectively which were primarily based on regular or monochromatic wave testing.

This chapter presents results and discussion on the propagation, shoaling, breaking and surf zone characteristics of three monochromatic wave structures of differing steepness propagating onto 5:1, 10:1 and 20:1 plane sloping beds. The positions and heights of breaking waves have been recorded and breaker depth indices calculated and compared to those predicted by design guidelines. Additionally, use of an optical LIF technique allowed precise quantification of changes in the wave crest and trough amplitudes during shoaling and at break point. The maximum water level attained by the wave crest at break point, an important parameter for engineering design, was therefore extracted and compared to the above-mentioned expressions for predicting wave height. Modifications to this expression to allow calculation of maximum water level are presented.

5.2 Experimental parameters

Three different monochromatic wave structures were tested propagating onto plane sloping bathymetries of between 5:1 and 20:1. The monochromatic waves varied in

steepness (s_0) based on their deep water amplitude (a_0) and wave number (k_0), with a steep, moderate and low steepness wave structure tested. A summary of the parameters used to generate each of these wave *configurations* is presented in Figure 5-1.

Table 5-1 Parameters of the three monochromatic wave structure tested during this study

Wave configuration	Initial wave number (k_0)	Initial mean wave amplitude, a_0 (m)	Initial wave steepness $s_0 = (ak)_0$
Mono - Low	7.4178	0.0149	0.11
Mono - Moderate	7.4178	0.0301	0.22
Mono - Steep	7.4178	0.0392	0.29

5.3 Deep water wave train evolution

The difficulty of maintaining a regular wave-train within a long wave tank is well recognised and was first explained by Benjamin and Feir (1967). They suggested that travelling nonlinear waves are unstable due to growth of a modulational perturbation, known as sideband instabilities. The development of this modulation has been shown for initially regular, monochromatic waves travelling in a controlled laboratory environment by Hwung and Chiang (2005). The development of modulation in an initially regular wave-train tested by Hwung and Chiang (2005) of initial steepness $(ak)_0 = 0.22$ is shown within Figure 5-1.

The surface profiles, wave heights and power spectra for low steepness ($s_0 = 0.11$), moderate steepness ($s_0 = 0.22$) and steep ($s_0=0.29$) monochromatic structures tested within the present study are shown at an along-flume distance of $k_0x = 65$ to 85 (Figure 5-2). Results show that while harmonics of the primary carrier frequency associated with the development of nonlinearity in the propagating wave-forms are evident for the low and moderate steepness structures, there is little evidence of modulation and the development of energy at sideband frequencies. However, the development of amplitude modulation in the steep monochromatic series is evident. Furthermore, this modulation increases with distance along the flume and its growth is accompanied by the development of energy at sideband frequencies, also termed Benjamin-Feir instabilities. While these sideband instabilities are relatively subdued at $k_0x = 65$, they increase rapidly and become pronounced by $k_0x = 85$. While a uniform and repeated group structure is not evident, an increase in energy at $f = 0.25$ Hz indicates the emergence of a low frequency wave with a period of 5 to 6 T_p .

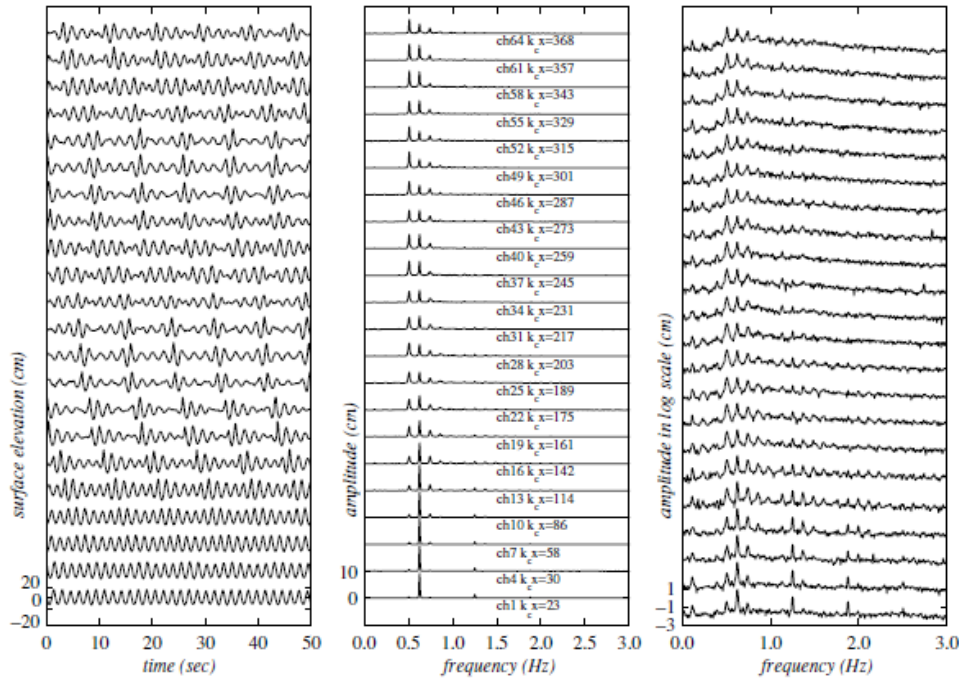


Figure 5-1 Records of water surface elevation and spectral analysis of an initial regular wave-train with steepness $(ak)_0 = 0.22$ and relative water depth $d/L_0 = 0.88$ (source: Hwung and Chiang, 2005).

While the $(ak)_0 = 0.22$ steepness monochromatic wave presented within Hwung and Chiang (2005) develops sideband energy and modulation by $k_0x = 58$ to 86 , such modulation is not observed in the comparable steepness wave tested within the present study. This may relate to the generation mechanism, with the Hwung and Chiang (2005) study using a piston-type wavemaker and the present study using a flexible-plate wavemaker which more accurately produces the orbital motions of water particles that decrease exponentially with depth, thus delaying the development of modulation.

Svendsen and Veeramony (2001) defined a groupiness parameter, G , as the ratio of the variation in wave height (ΔH) to mean wave height (H_{mean}). This parameter is calculated for the three monochromatic groups at $k_0x = 65$ and 85 (Table 5-2). While the low and moderate steepness monochromatic structures show some variation in wave height and a small increase with along-flume distance, the steep monochromatic structure shows the development of more pronounced modulation by $k_0x = 65$ and a significant (three-fold) increase by $k_0x = 85$. This finding supports Benjamin and Feir's theory of exponential growth in sideband instabilities (Benjamin and Feir, 1967).

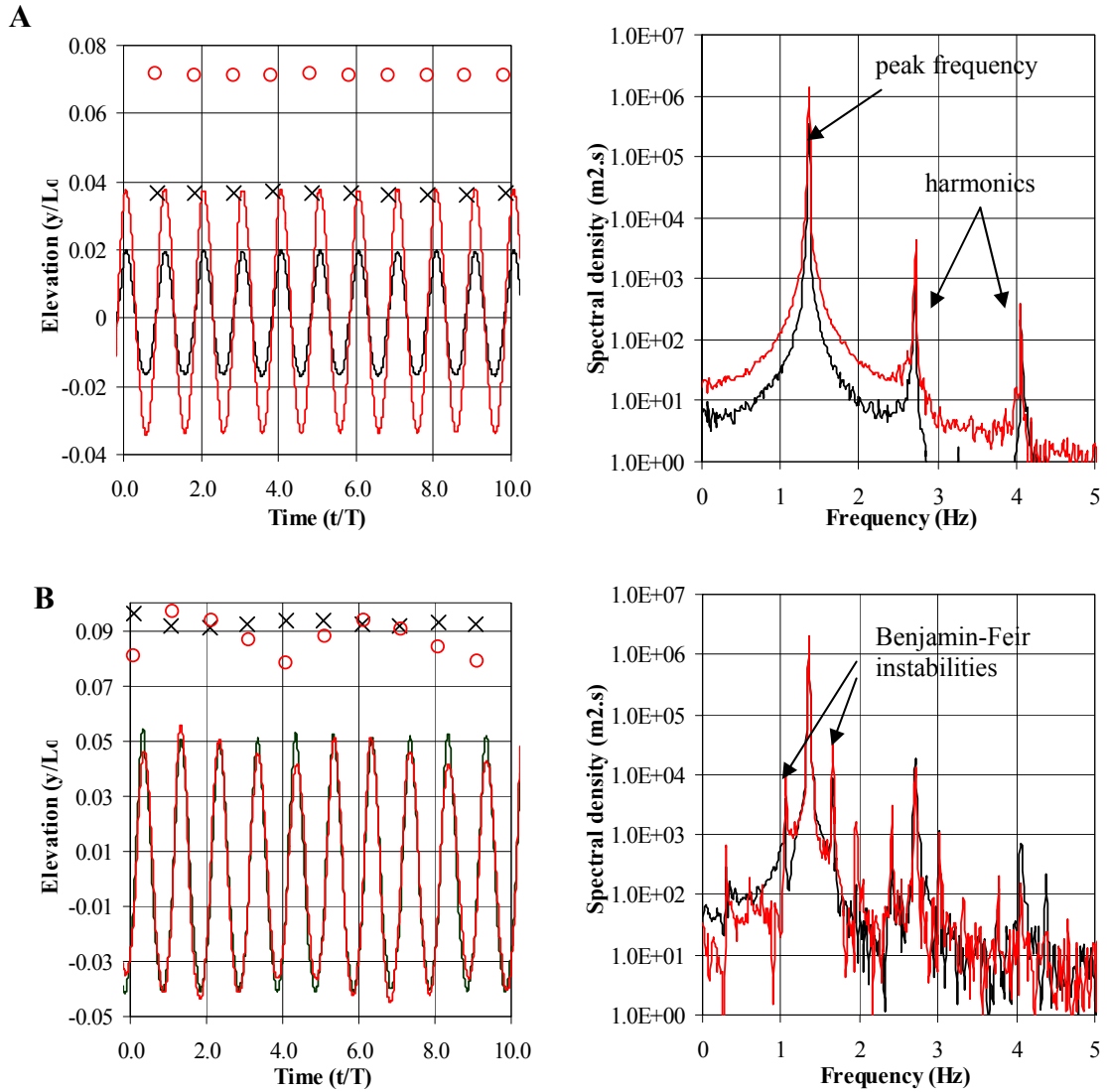


Figure 5-2 Surface profile, wave height and power spectra for the low steepness ($s_0 = 0.11$) ($y = \text{—}$; $H = \times$; spectra = —) and moderate steepness ($s_0 = 0.22$) ($y = \text{—}$; $H = \circ$; spectra = —) monochromatic structures (A) at $k_0x = 65$. and for the steep ($s_0 = 0.29$) monochromatic structure (B) at $k_0x = 65$ ($y = \text{—}$; $H = \times$; spectra = —) and 85 ($y = \text{—}$; $H = \circ$; spectra = —).

Table 5-2 Groupiness parameter G , defined by the ratio of the variation in wave height (ΔH) to mean wave height (H_{mean}) for the three monochromatic structures.

Monochromatic structure	$k_0x = 65$	$k_0x = 85$
Mono - Low ($s_0 = 0.11$)	0.043	0.045
Mono - Mod ($s_0 = 0.22$)	0.038	0.047
Mono - Steep ($s_0 = 0.29$)	0.105	0.341

5.4 Results and discussion

5.4.1 Shoaling transformations

The wave envelope ($z_{\max}$ and $-z_{\min}$), wave height (H) and mean water level (z_{mean}) were recorded during shoaling and breaking using a combination of wave probes in deeper water ($> \sim L_0/4$) and the optical LIF method in shallow water. Figure 5-3 to Figure 5-5 present the above-listed wave and water level data, non-dimensionalised by deep water wavelength (L_0) for the 5:1, 10:1, and 20:1 sloping beds respectively. Mean wave breaking position (Section 5.4.2) are indicated on each of the figures by dashed vertical lines.

For the most part, the two methods of data collection overlap well, with wave heights always within 5%, and more typically within 1 to 2%. These values are in agreement with direct comparison of the two methods discussed within Section 4.3.2.1. Elevations for lower steepness waves appear in better agreement than for steeper wave forms and minimum (trough) elevations are in better agreement than for the maximum (crest) elevations. This is attributed to the modulation that occurs in the steeper wave-forms which may induce temporal variations in observed wave height, regardless of the instrumentation used. Although the present investigation does not focus on monochromatic wave transformation and the applicability or accuracy of various methods and models for calculating the shoaling and surf zone transformations or changes in the mean water level, a limited discussion of the evolution of monochromatic characteristics and their comparison to existing theory is provided within the following sections.

Shoaling transformations may be identified by changes in the water-level envelope, defined by the maximum and minimum surface elevations, and by changes in the wave height as the wave forms approach break point. The maximum and minimum surface elevations correspond to the wave crest and trough amplitudes and so may be used to identify the development of vertical asymmetry during shoaling.

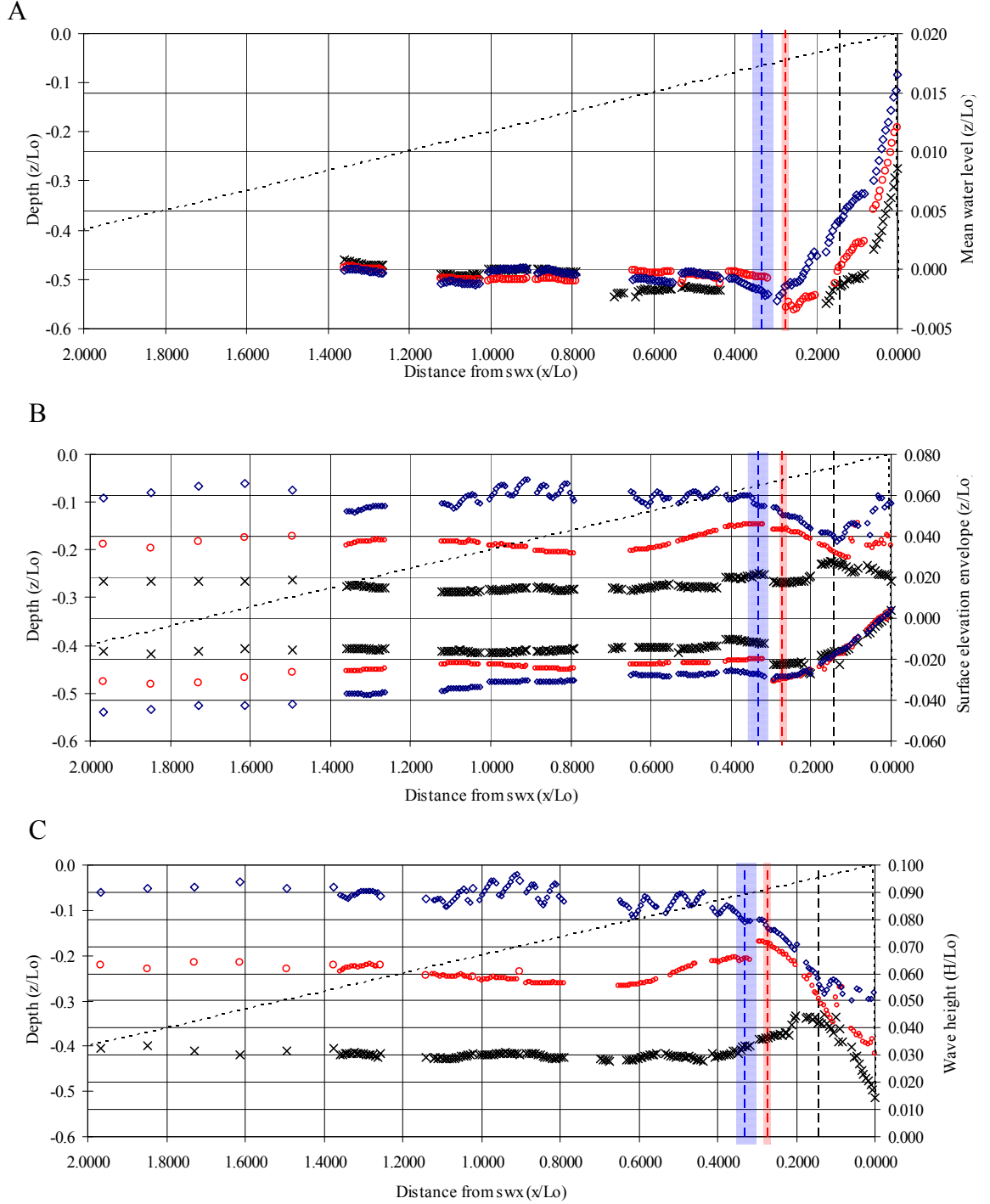


Figure 5-3 Propagation of low ($ak_0 = 0.11$; $\times$), moderate ($ak_0 = 0.22$; $\circ$) and steep ($ak_0 = 0.29$; $\diamond$) monochromatic waves onto a 5:1 sloped bed ($\cdots$) with the evolution of mean water level (z_{mean}) presented within plot A, maximum and minimum water levels (z_{max} , $-z_{\text{min}}$) within plot B and observed wave height (H) together with the wave height predicted by linear wave theory within plot C. Mean break points (BP) for each wave structure are illustrated by dashed vertical lines ($ak_{0-0.11}$: $---$; $ak_{0-0.22}$: $---$; $ak_{0-0.29}$: $---$) with the range in breaking position, indicated by shading.

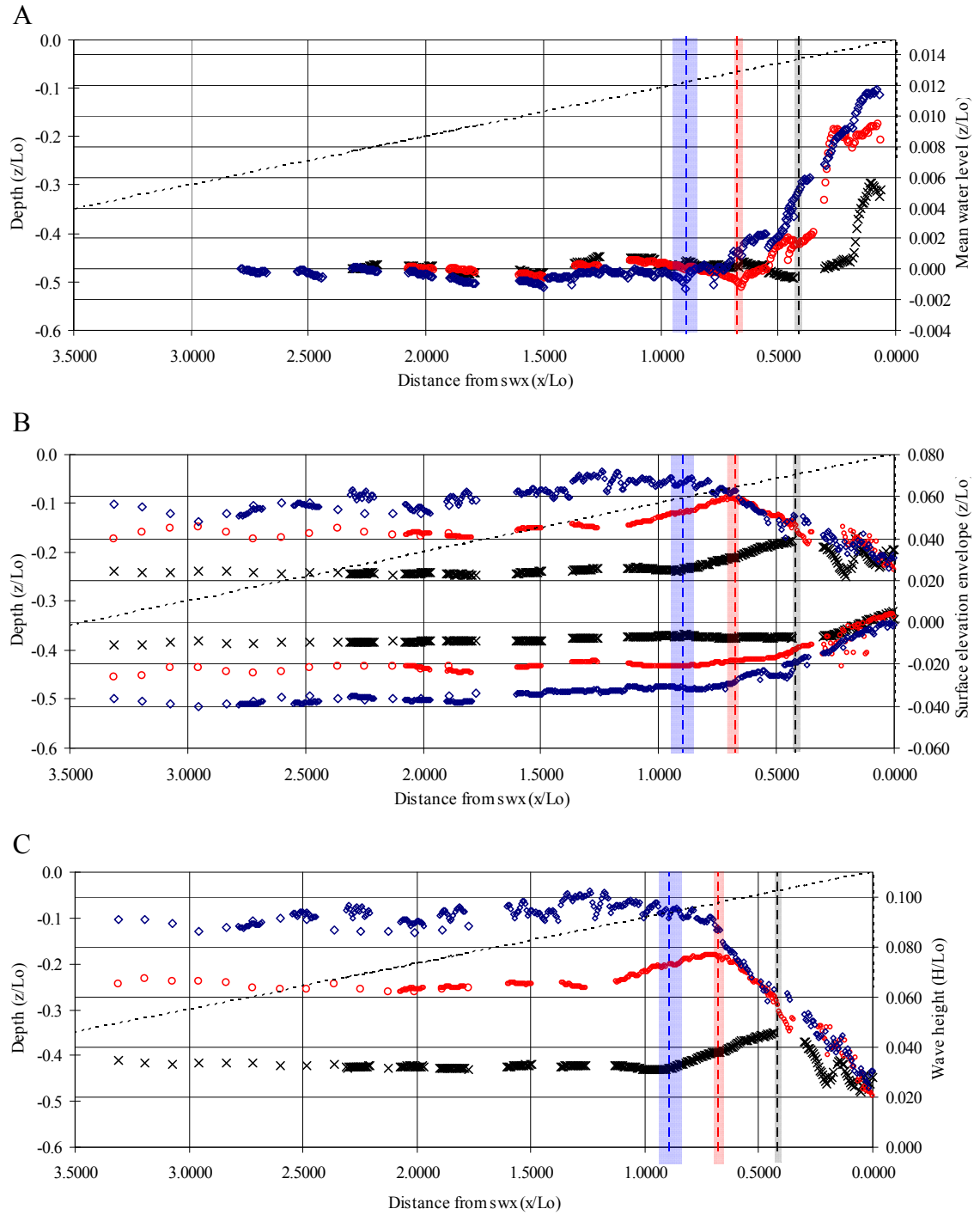


Figure 5-4 Propagation of low ($ak_0 = 0.11$; $\times$), moderate ($ak_0 = 0.22$; $\circ$) and steep ($ak_0 = 0.29$; $\diamond$) monochromatic waves onto a 10:1 sloped bed (---) with the evolution of mean water level (z_{mean}) presented within plot A, maximum and minimum water levels (z_{max} , $-z_{\text{min}}$) within plot B and observed wave height (H) together with the wave height predicted by linear wave theory within plot C. Mean break points (BP) for each wave structure are illustrated by dashed vertical lines ($ak_{0-0.11}$: ---; $ak_{0-0.22}$: - - -; $ak_{0-0.29}$: - - -) with the range in breaking position, indicated by shading.

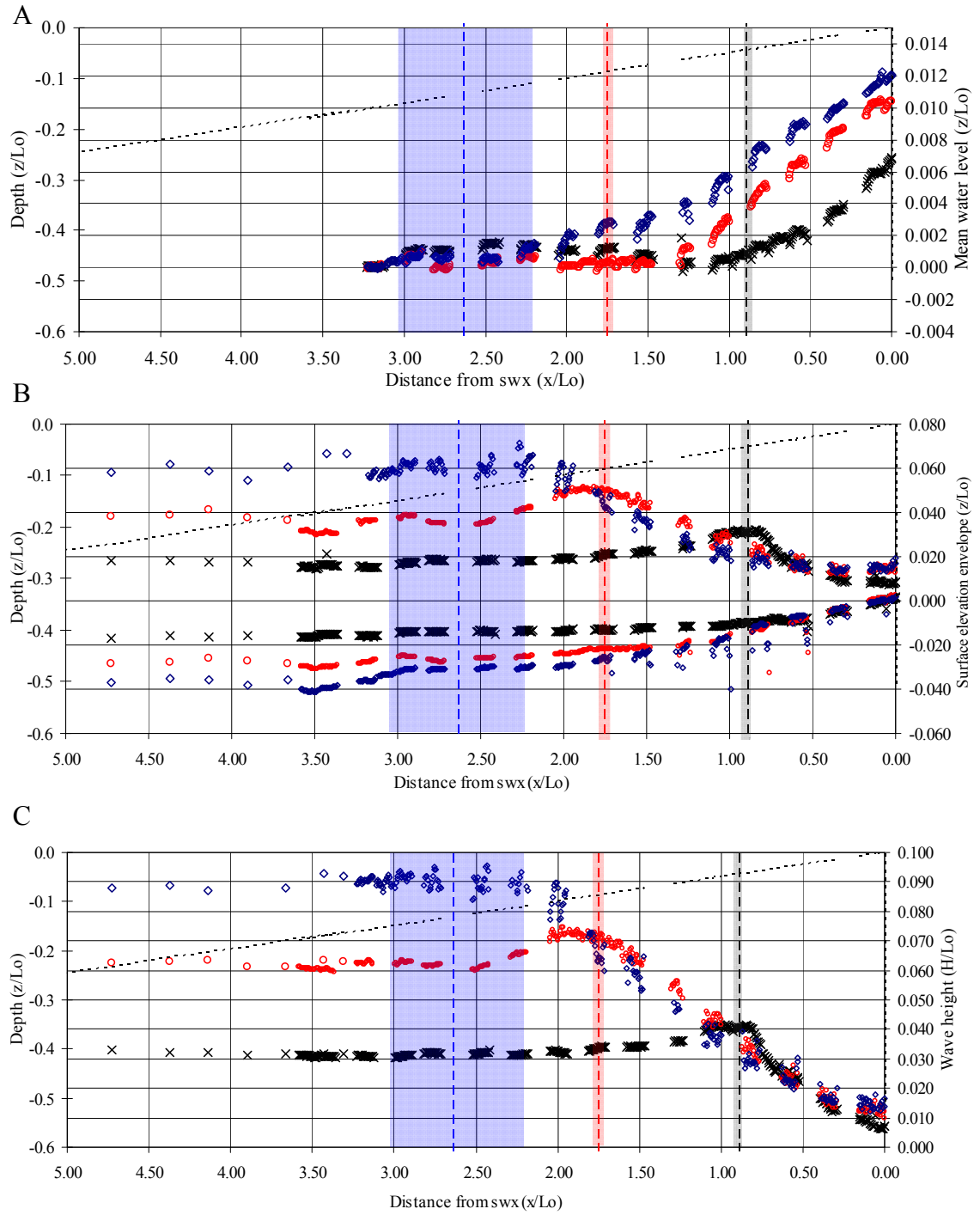


Figure 5-5 Propagation of low ($ak_0 = 0.11$; ×), moderate ($ak_0 = 0.22$; ○) and steep ($ak_0 = 0.29$; ◇) monochromatic waves onto a 20:1 sloped bed (---) with the evolution of mean water level (z_{mean}) presented within plot A, maximum and minimum water levels (z_{max} , $-z_{\text{min}}$) within plot B and observed wave height (H) together with the wave height predicted by linear wave theory within plot C. Mean break points (BP) for each wave structure are illustrated by dashed vertical lines ($ak_{0-0.11}$: ---; $ak_{0-0.22}$: - - -; $ak_{0-0.29}$: - · -) with the range in breaking position, indicated by shading.

Apart from Benjamin-Fier effects, the maximum surface elevation (crest amplitude) appears relatively stable during the initial stages of shoaling. This is followed by a rapid increase in elevation during the later stages of shoaling prior to break point. This increase appears more pronounced on steeper bed slopes and for lower wave steepnesses. The minimum elevation (trough amplitude), in contrast, displays a more consistent reduction in elevation throughout the shoaling process. This implies that shoaling increases in wave height are dependent on the increase in crest elevation exceeding the simultaneous decrease in trough elevation.

The relatively stable crest amplitude during the early stages of shoaling suggests that the reduction in wave height that occurs during initial stages of wave shoaling is more related to reduction in the trough amplitude than in the crest amplitude. The subsequent increases in wave height are more closely associated with increases in crest amplitude, with trough amplitude continuing to decrease. While the crest amplitude is increasing and trough amplitude is decreasing as the wave shoals, no increase in mean water level is observed, but rather a slight decrease associated with radiation stresses as discussed within Section 5.4.3.2. The trough width of the wave-forms must increase and crest width decrease to maintain this mean water level. This indicates development of the vertical asymmetry generally evident during shoaling (Weggel, 1960).

Changes in the ratio of crest to trough amplitude ($z_{\max}$ to $z_{\min}$) are presented within Figure 5-6. Ratios are observed to increase during shoaling to a maximum near break point. The ratio then decreases, presumably as energy is lost during breaking, before a rapid increase in the ratio within the inner surf zone. This is attributed to the increase in mean water-level due to set-up resulting in relatively higher crest amplitudes.

Higher ratios at break point (further discussed within Section 5.4.2.2) are observed for the lower steepness waves and for the lower bed slopes. This is likely due to the lower bed slopes providing an extended fetch, over which non-linearity may develop prior to breaking. The lowest steepness wave on the 5:1 slope differs in that it decreases during initial shoaling before only increasing slightly prior to break point. Reasons for this discrepancy are not, at present, clear.

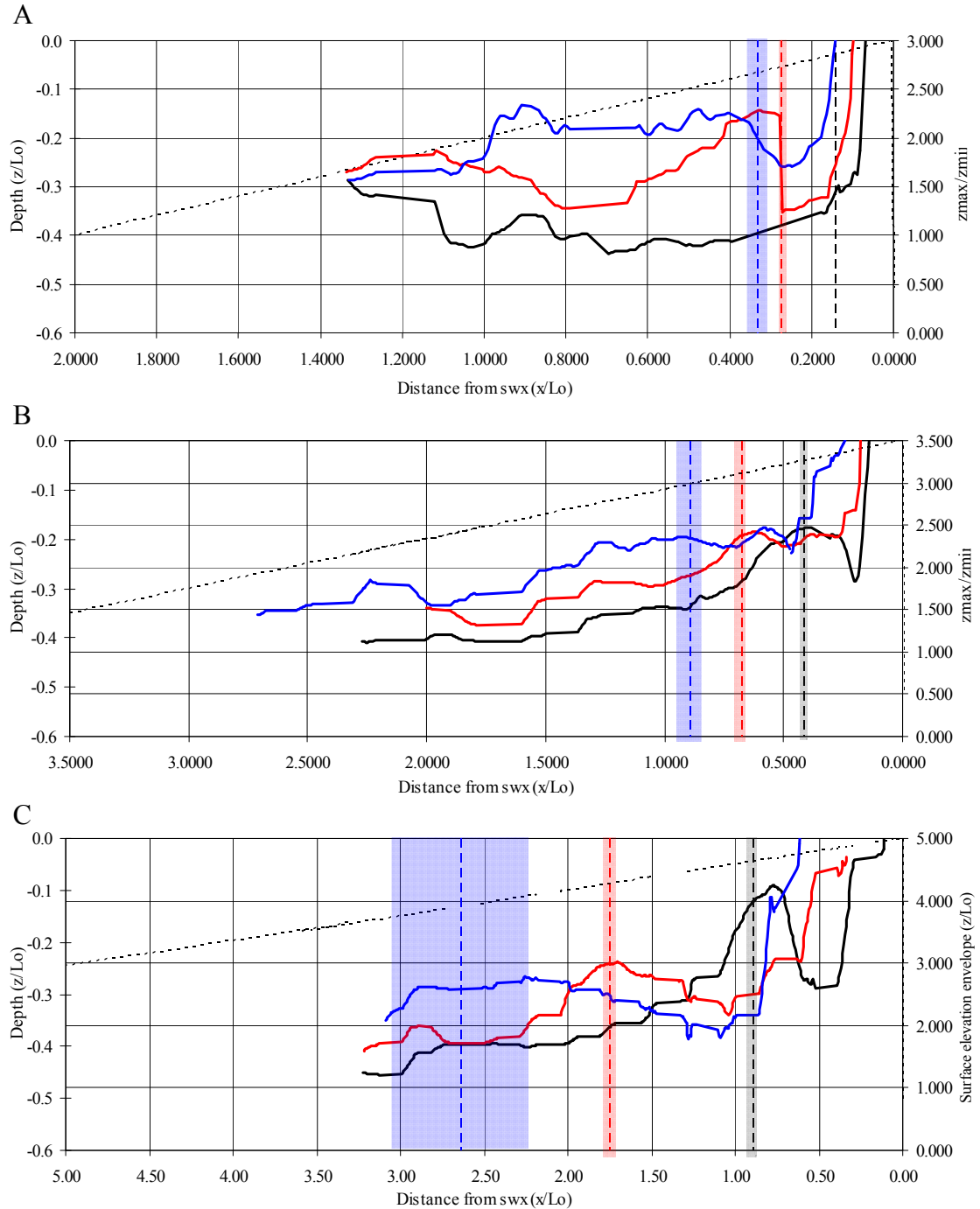


Figure 5-6 Changes in the maximum to minimum surface elevation ($z_{\max}/z_{\min}$) during shoaling and within the surf zone for a low ($ak_0 = 0.11$; —), moderate ($ak_0 = 0.22$; —) and steep ($ak_0 = 0.29$; —) monochromatic waves propagating onto a 5:1 (A), 10:1 (B) and 20:1 (C) sloped bed. Mean break points (BP) for each wave structure are illustrated by dashed vertical lines ($ak_{0-0.11}$: - - - ; $ak_{0-0.22}$: - - - ; $ak_{0-0.29}$: - - -) with the range in breaking position indicated by shading.

Wave height exhibits the expected slight reduction in height during the initial stages of shoaling, followed by a rapid increase prior to breaking. The increase is most pronounced for the lower wave steepness cases and on steeper bed slopes. The position

of maximum shoaling height was generally in agreement with the break point location(s) as defined by the front wave-face becoming vertical (Section 5.4.2). However, waves breaking in more of a plunging manner (i.e. lower steepness wave forms on steeper bed slopes) appear to reach a maximum, or maintain a near maximum height slightly shoreward of the break point. Waves breaking in more of a gentle or spilling manner (i.e. steeper wave forms on lower bed slopes) appear to reach a maximum on the seaward side of the break point range. Observation shows that this is likely due to the kinematics of the respective breaking processes. For the plunging case, a free jet is ejected forward from the initially defined break point (Figure 5-7). The horizontal trajectory of this jet enables wave height to be maintained shoreward of the break point.

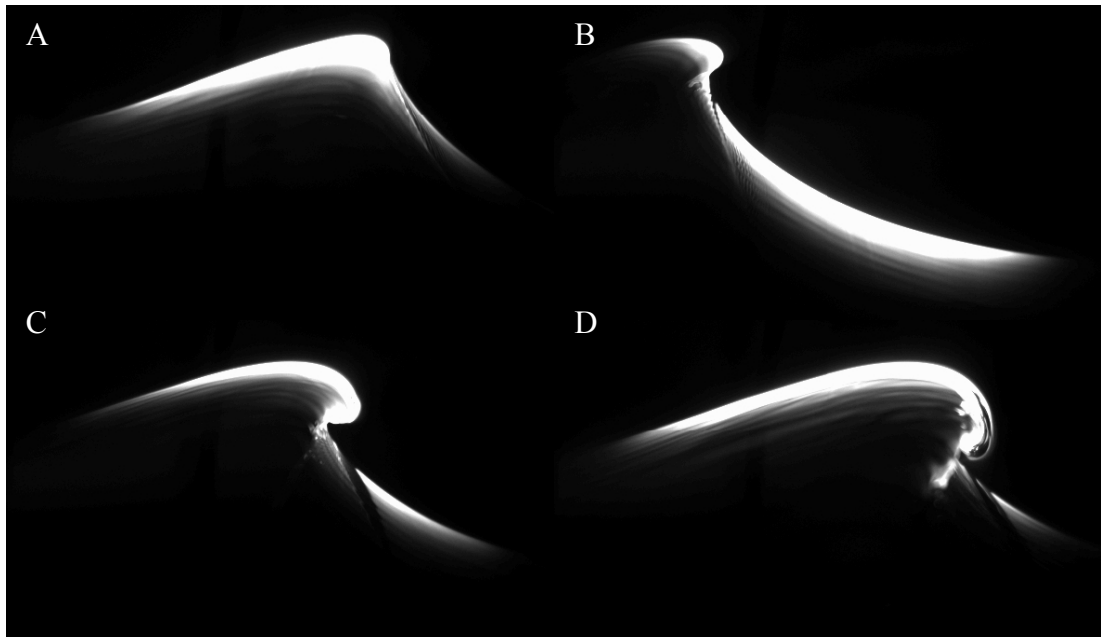


Figure 5-7 Breaking kinematics of a plunging wave. The initial break point, as defined by the front wave face becoming vertical is shown in (A), with (B – D) showing subsequent evolution. Note the images in (B, C and D) are offset from (A) by $0.12L_o$.

For the spilling case, two reasons are noted. Firstly, breaking often begins as small capillary waves on the front face, followed by the crest becoming vertical and more substantial spilling (Figure 5-8). While the break point is defined as the front face becoming vertical, some small scale breaking and thus energy loss may be occurring prior. Secondly, the break point range of the spilling waves is typically wider for reasons discussed within Section 5.4.2. The larger waves break in a more seaward location and height is decreased due to dissipation. The heights of the more landward

breaking waves are smaller and thus the maximum height occurs at the seaward side of the breaking region.

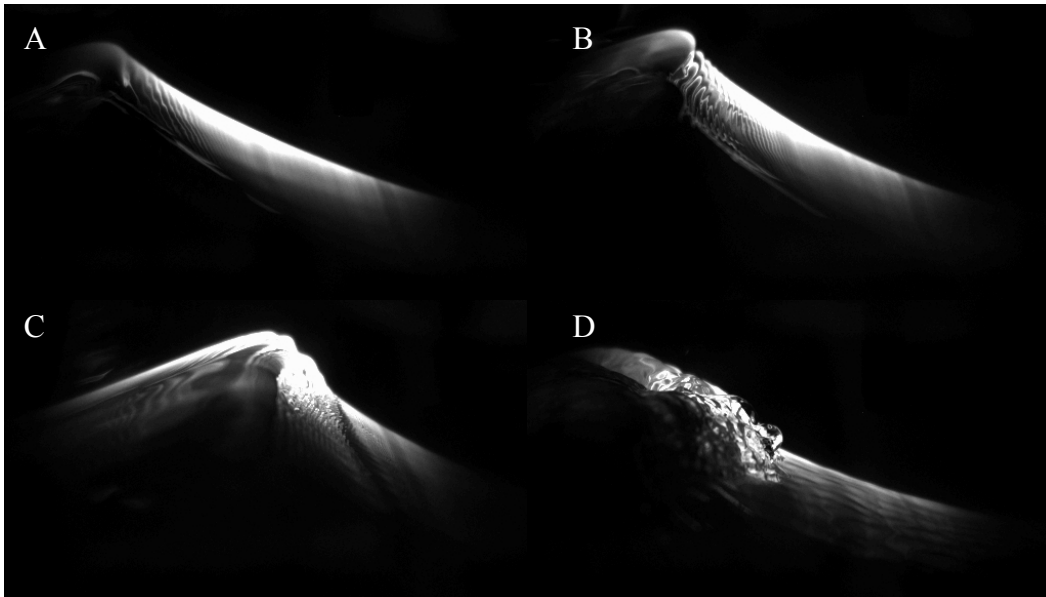


Figure 5-8 Breaking kinematics of a spilling wave. Small capillary waves form on the front wave face (A), followed by the face becoming vertical (B) and thus defining the break point. Spilling then progresses down the front face (C and D). Note each image is offset from the previous by $0.12L_o$.

A reduction in mean water level, or set down, is typically evident during the shoaling process, most notably for the steeper bed slopes and lower wave types. This process is discussed in more detail, within the discussion on surf zone processes (Section 5.4.3).

5.4.2 Wave breaking

The break point has been used to define the partition between shoaling processes and surf zone processes. Break point locations have been defined by the front wave face first becoming vertical. As discussed previously, some variability in wave height was observed in each of the monochromatic wave-trains within deep water. This variation in deep water wave height results in a variation in the position of breaking and in breaking wave height. Similar variation in the breaking position and height of monochromatic waves has been noted by Smith and Kraus (1991) and Goda (2007) and such variation was considered inherent to the breaking process of realistic waves. The seaward-most and landward-most positions of breaking for each of the wave types and bed slopes are presented within Table 5-3 along with the mean breaking position. Due to the very similar breaking heights of the monochromatic waves (compared to well defined wave groups where individual waves are more easily distinguished), the height of individual

waves could not be directly linked to individual breaking location using the optical LIF method. The maximum wave heights corresponding to the seaward-most and landward-most breaking positions are therefore used. This approach provides a conservative upper bound on breaking wave height and on breaker depth indices.

Additionally, the breaker type has been visually defined according to the scale described within Section 4.6 with low values correlating to weakly spilling breaking and high values to intense plunging or collapsing/surging breaker types. Lower steepness wave forms, and waves breaking on steeper bed slopes are observed to break in more of a plunging manner and steeper wave forms or waves breaking on lower bed slopes in more of a spilling manner. Exemplary images of wave breaking on 5:1, 10:1 and 20:1 slopes, corresponding to visual breaker types of 8.0, 6.5 and 3.0 respectively are presented within Figure 5-9. This again, is consistent with literature (i.e. Galvin, 1969).

Table 5-3 Wave breaking characteristics point for each of the monochromatic wave types and bed slopes. The visual breaker type index is defined within Table 4.9.

Wave steepness (s_0)	Slope	Breaking Location (x_{swx}/L_0)				Breaking height (H_b/L_0)				Visual breaker type index
		Landward	Seaward	Mean	Range	Landward	Seaward	Mean	Range	
0.11	5:1	0.138	0.145	0.141	0.007	0.0419	0.0419	0.0419	0.0000	8.5
	10:1	0.399	0.427	0.413	0.028	0.0456	0.0432	0.0444	0.0024	8.0
	20:1	0.870	0.909	0.890	0.039	0.0406	0.0403	0.0404	0.0003	4.5
0.22	5:1	0.263	0.281	0.272	0.018	0.0707	0.0714	0.0711	0.0007	8.0
	10:1	0.655	0.691	0.673	0.036	0.0747	0.0765	0.0756	0.0018	7.0
	20:1	1.711	1.785	1.748	0.074	0.0699	0.0725	0.0712	0.0026	3.0
0.29	5:1	0.309	0.352	0.330	0.043	0.0796	0.0826	0.0811	0.0030	7.0
	10:1	0.838	0.945	0.891	0.107	0.0928	0.0978	0.0953	0.0050	6.5
	20:1	2.224	3.034	2.629	0.81	0.0944	0.0925	0.0934	0.0019	3.5

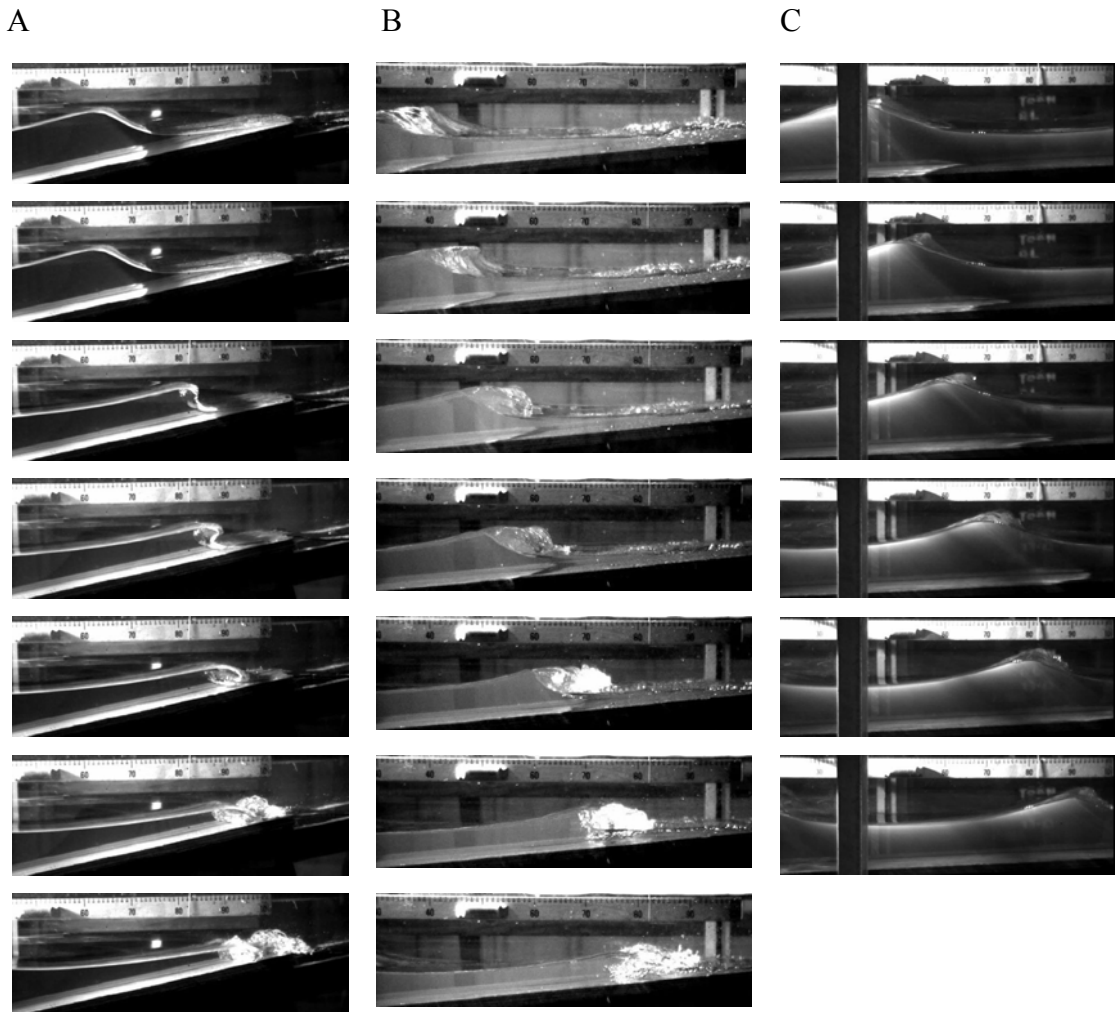


Figure 5-9 Exemplary imagery for a moderate steepness ($s_0 = 0.22$) monochromatic wave breaking on a 5:1 sloping bed (A), and a 10:1 sloping bed (B) and steep ($s_0 = 0.29$) wave breaking on a 20:1 sloping bed (C). The time interval between frames is 0.1 s.

Both the height and position of individual breaking waves was, however, able to be recorded using manual digitization of the less accurate, wide-view breaking imagery. These heights and positions for each of the wave types and bed slopes are presented within Figure 5-10. The figure shows limited scatter in breaking wave height and position for the low ($s_0 = 0.11$) and moderate ($s_0 = 0.22$) steepness waves breaking on the steeper 5:1 and 10:1 slopes, and greater scatter on the 20:1 slope. The steeper ($s_0 = 0.29$) group shows more scatter, especially on the 20:1 slope where a number of larger outliers both in height and breaking position are attributed to the largest waves within the modulated wave-train.

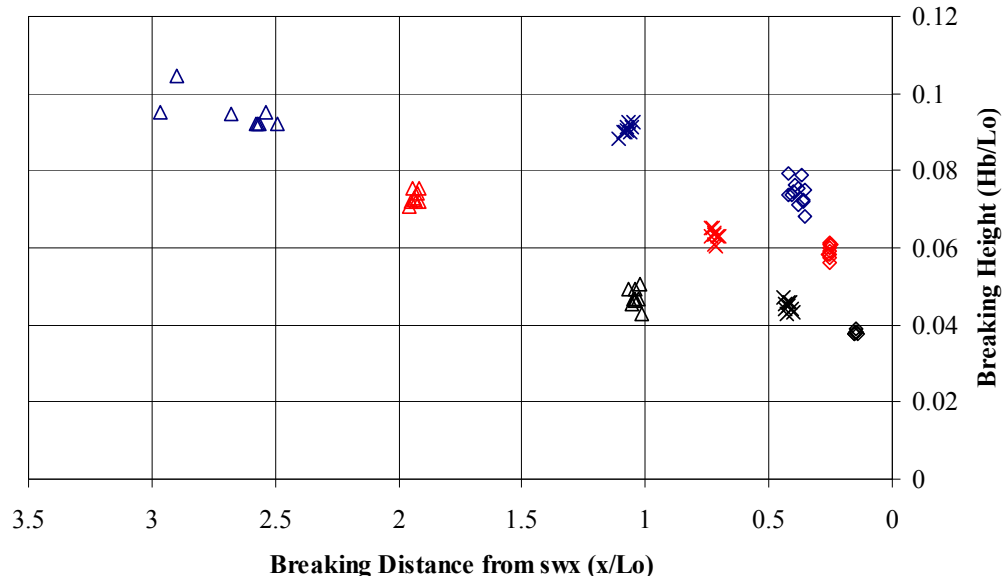


Figure 5-10 Breaking wave position and height for the three monochromatic structures on a 5:1 ($ak_{0-0.11}$: $\diamond$; $ak_{0-0.22}$: $\times$; $ak_{0-0.29}$: Δ), 10:1 ($ak_{0-0.11}$: $\diamond$; $ak_{0-0.22}$: $\times$; $ak_{0-0.29}$: Δ) and 20:1 ($ak_{0-0.11}$: $\diamond$; $ak_{0-0.22}$: $\times$; $ak_{0-0.29}$: Δ) sloping bed as observed using wide-angled imagery and manual digitization. Accuracy and errors associated with this data abstraction technique of are discussed within Section 4.6.

Some difference is observed between the breaking position and height using the wide-view breaking imagery and the optical LIF imagery. Mean wave height when measured from the wide-view breaking imagery is smaller (5 to 20%) on the steeper, 5:1 and 10:1 slopes than measured with the optical LIF technique and larger on the flatter 20:1 slope (1 to 13%). Conversely, mean breaking position when observed in the wide-view breaking imagery is nearly always slightly seaward (2 to 17%) of that observed in the optical LIF imagery.

These differences in observed heights are attributed to the difference in definition of height between the two methods. Breaking wave height is defined in the wide-view breaking imagery as the instantaneous trough to crest height at the instant of breaking whereas optical LIF and wave probe data have been processed to obtain trough and crest elevations at a single point in a time series. As trough amplitude is observed to constantly decrease during shoaling (Figure 5-3 to Figure 5-5), measurement in a more shoreward location will result in a smaller trough amplitude and therefore smaller total wave height.

The difference in observed breaking position appears due to the sidewall influence on the propagating wave-form. This sidewall appears to slightly slow the adjacent wave-form, meaning that as the crest in the central part of the flume is observed to become vertical, the position of the crest against the sidewall, where measurement is taken, is located slightly seaward. This was observed to be most pronounced for the steeper waves on gentler bed slopes, as is reflected in the differences between the two sets of observations. This sidewall interference was one of the primary motivations for development of the optical LIF method and, on the basis of these comparisons, well founded.

As these differences in breaking height and position between the two methods were not found to be entirely consistent with bed slope or wave steepness, a *correction factor* or similar may not be employed. Furthermore, as the differences may be relatively large, the measurements obtained using the wide-view breaking imagery must be treated with prudence, although may be used for quasi-quantitative examination of scatter in data or of general trends.

5.4.2.1 Breaker depth index

To define the largest wave which may occur in water of finite depth, the engineering practitioner uses established design guidelines such as the *Coastal Engineering Manual* (USACE, 2003) and *Random Seas and Design of Maritime Structures* (Goda, 2000). These guidelines use design curves derived from original studies by Weggel (1972) and Goda (1970) respectively. Equations defining their curves are given by Eqns. 5-1 and 5-2 respectively.

$$\frac{H_b}{d_b} = b - a \frac{H_b}{gT^2} \quad (5-1)$$

$$\frac{H_b}{d_b} = \frac{A}{d_b / L_0} \left\{ 1 - \exp \left[-1.5\pi \frac{d_b}{L_0} \left(1 + 11s^{4/3} \right) \right] \right\} \quad (5-2)$$

where a and b for Weggel's Eqn. 5-1 is given by $a = 43.8(1 - e^{-19\tan\beta})$ and $b = 1.56/(1 + e^{-19.5\tan\beta})$ and A for Goda's Eqn. 5-2 is 0.17 for regular waves. Within the guidelines, d_b is defined as the *depth at breaking* (USACE, 2003; Weggel, 1972) or *water depth at breaking points* (Goda, 1970). In a review of previous investigations and breaker index

formulae, Rattanapitikon and Vivattanasirisak (2003) suggest that a depth based on still water level is appropriate.

As discussed earlier, some variation in breaking location and height was noted for all monochromatic wave cases tested. This attributed to amplitude modulation within the wave train which developed during deep water propagation. The resultant breaker depth indices, with depth defined according to still water level (H_b/d_{swl}) are presented within Table 5-4 and compared to values predicted by the USACE (2003) and Goda (2000).

Breaker index trends are as expected according to established theory (i.e. Goda, 1970; Weggel, 1972), with lower waves and steeper slopes exhibiting higher indices and steeper waves and flatter slopes exhibiting lower indices. The only exceptions are the low and moderate steepness cases on the 10:1 slope where the mean values are near identical at 1.11 and 1.12 respectively. Table 5-3 showed variation in breaking position to increase with increasing wave steepness and with decreasing bed slope. The variation in breaker depth index differs with the variation typically being greater on the steeper slopes. This is attributed to small differences in breaking position on the steeper slopes leading to relatively larger variation in depth and therefore breaker index. The observed breaker indices are compared to those predicted by the respective design guidelines within Figure 5-11.

Table 5-4 Observed breaker depth index and visual breaker type index compared with breaker depth index values predicted by USACE (2003) and Goda (2000). Visual breaker type index is defined within Table 4.9.

Wave steepness (s_0)	Slope	Observed breaker depth index (H_b/d_{swl})				Visual breaker type index	Design guidelines	
		Maximum	Minimum	Mean	Range		USACE (2003) H_b/d_{swl}	Goda (2000) H_b/d_{swl}
0.11	5:1	1.52	1.46	1.49	0.06	8.5	1.24	1.57
	10:1	1.14	1.07	1.11	0.07	8.0	1.09	1.05
	20:1	0.93	0.89	0.91	0.04	4.5	0.96	0.85
0.22	5:1	1.34	1.27	1.30	0.07	8.0	1.05	1.38
	10:1	1.14	1.11	1.12	0.03	7.0	0.91	0.96
	20:1	0.82	0.79	0.80	0.03	3.0	0.83	0.76
0.29	5:1	1.30	1.17	1.24	0.13	7.0	0.95	1.27
	10:1	1.11	1.03	1.07	0.08	6.5	0.81	0.90
	20:1	0.83	0.61	0.72	0.22	3.5	0.75	0.71

While both guidelines provide reasonable prediction of the observed breaker depth index trends, the USACE (2003) guidelines under-predicted breaker indices on steeper 5:1 and 10:1 slopes by 17% to 25%, although the lower wave-steepness case on 10:1 slope was in good agreement with predictions. Predictions were also in better agreement on the lower 20:1 slope, with an over-prediction by guidelines of 2% to 5%. The values predicted by Goda (2000) are in better general agreement, although slightly under-predict the 10:1 and 20:1 slope values (5% to 15%) and over-predict the steeper, 5:1 values (average of 6%). However, it should be noted that only very limited data for slopes steeper than 10:1 was included in the study of Weggel (1972), upon which the USACE (2003) guidelines are based. While the USACE (2003) guidelines define Eqn. 5-1 as being applicable for slopes of less than 10:1, the accompanying figure gives a design curve for a 5:1 slope (Figure 2-7). Similarly, while limits of slope applicability are not presented explicitly within Goda (2000), the original study of Goda (1970), upon which the more recent guidelines are based, did not appear to include data from slopes steeper than 10:1. Given this lack of data, the agreement of the observed to predicted values for the steeper bed slope is very good.

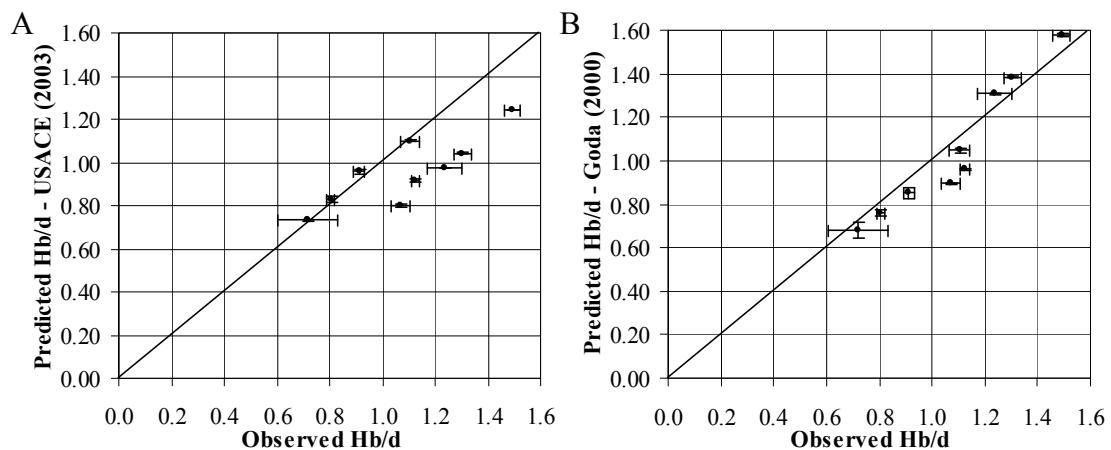


Figure 5-11 Observed breaker depth indices compared to that predicted by USACE (2003) (A) and Goda (2000) (B). Range in both the observed and predicted values are depicted by error bars in the both the x and y directions respectively and the solid line depicts an idealised relationship.

As discussed earlier, scatter in breaking position and height and consequently in breaker depth index was noted in all monochromatic tests. Goda (2007) notes that, even for regular waves, scatter is inherently present within acquired laboratory data. This scatter is attributed to the inherently stochastic nature of wave breaking and a coefficient of

variation (CoV) is defined. This CoV is quantified at 6% to 7% for slopes less than 1:100, increasing to around 14% for a slope of 10:1. While a CoV varying uniformly with depth was not noted within the present study, the low and moderate wave steepness cases gave CoV values of 4% to 5.5% on the 5:1 slope, 2.7% to 6.3% on the 10:1 slope and 3.8% to 4.4% on the 20:1 slope. These variations are comparative to the wave height variation observed within deep water propagation ($H_{\text{range}} < 4\% H_{\text{mean}}$). The steeper monochromatic case exhibited greater variation at 10%, 7.5% and 30% for the 5:1, 10:1 and 20:1 slopes respectively. Again, these high levels of variation are comparable to the substantial height modulation ($H_{\text{range}} \approx 30\% H_{\text{mean}}$) which developed during propagation. Overall, the variation observed within the current testing supports the observations by Smith and Kraus (1991) and Goda (2007) of inherent scatter in H_b/d values of regular breaking waves. However, the current study has shown that a portion of this scatter is likely attributable to modulation of regular wave-trains by sideband instabilities.

5.4.2.2 Maximum surface elevation at break point

Along with maximum breaking wave height, information on maximum likely water levels is important for engineering design as it influences run-up and required crest elevations on structures and hazard levels. Figure 5-3 to Figure 5-5 show the changes in maximum water level (z_{max}) during shoaling and at break point. The ratio of maximum surface elevation to depth at break point ($z_{\text{max-BP}}/d$), and the ratio of maximum surface elevation to breaking wave height ($z_{\text{max-BP}}/H_b$) are presented within Table 5-5. As for the breaking index, some variation in breaking position occurred and therefore in the observed $z_{\text{max-BP}}/d$ ratios. Ratio trends were in general agreement with the previously discussed breaker height indices, with lower waves and steeper slopes exhibiting higher ratios and steeper waves and flatter slopes exhibiting lower ratios.

While ratio of maximum surface elevation to wave height in deep water, where waves are nearly sinusoidal, should approach 0.5, greater values were noted in all cases at break point. This is related to the development of the earlier described asymmetry, with the crest becoming higher and more peaked and the trough lower and flatter. Ratios were generally higher on flatter slopes and for lower wave trains. This is likely related to the fetch length over which non-linearity may develop prior to breaking with the flatter slopes providing a greater fetch and the lower steepness wave trains interacting

with the bed in deeper water and thus developing greater non-linearity and asymmetry. These results imply that on flat slopes and for low wave trains, the maximum surface elevation relative to still water level at break point may comprise a significant (up to 85%) portion of the wave height. This is an important concept for engineering design.

Resultant $z_{\max}/d$ ratios are also compared to guideline predicted breaker indices (Goda, 2000; Eqn 5-2) in Table 5-5 and Figure 5-12(A). Guidelines predicted values according to USACE (2003) were not used as the expression contains breaking wave height on both sides of the expression (Eqn. 5-1) and thus was deemed unsuitable for calculating the maximum water level.

Table 5-5 Observed $z_{\max-BP}/d$ ratios, ratios of $H_b/z_{\max-BP}$ and breaker depth index values predicted by Goda (2000).

Wave steepness (s_0)	Slope	Observed $z_{\max-BP}/d_{swl}$				Ratio of $z_{\max-BP}/H_b$	Design guidelines
		Maximum	Minimum	Mean	Range		Goda (2000) H_b/d_{swl}
0.11	5:1	0.97	0.94	0.96	0.03	0.64	1.57
	10:1	0.96	0.9	0.93	0.06	0.84	1.05
	20:1	0.76	0.73	0.75	0.03	0.82	0.85
0.22	5:1	0.82	0.78	0.80	0.04	0.62	1.38
	10:1	0.81	0.8	0.81	0.01	0.72	0.96
	20:1	0.63	0.6	0.62	0.03	0.78	0.76
0.29	5:1	0.86	0.85	0.86	0.01	0.69	1.27
	10:1	0.79	0.74	0.77	0.05	0.72	0.90
	20:1	0.61	0.43	0.52	0.18	0.72	0.71

Results show Goda's expression to overpredict $z_{\max-BP}/d$. This is expected as the expression is derived for wave height, not maximum surface elevation. However, the expression only slightly overpredicts the ratio for the 10:1 and 20:1 slopes, again indicating the significant asymmetry in the wave forms at break point. More significant overprediction occurs for the steeper 5:1 sloping bed cases where asymmetry is less pronounced. Figure 5-12(B) shows the same comparison, with the coefficient, A , within Goda's Eqn. 5-2 modified to provide an improved fit. Rather than the $A = 0.17$ assumed for breaking wave height, A is reduced to 0.15 for the 10:1 and 20:1 slopes and to 0.11 for the 5:1 slope. Variation in $z_{\max-BP}/d$ for the differing wave steepnesses appears to be well predicted by the expression. Overall, while slopes flatter than 20:1 remain untested,

it would be expected that as the bed flattens and a greater shoaling fetch length is available for the development on non-linearity, crest asymmetry will become more pronounced and the coefficient, A will tend towards 0.17. Further testing is required to confirm this.

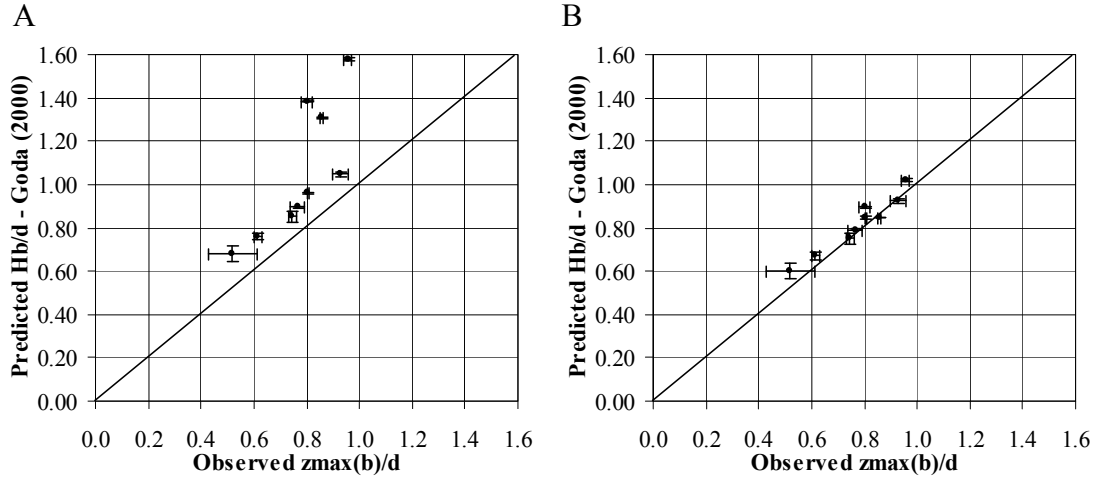


Figure 5-12 Observed ratio of maximum wave height to depth ($z_{\max-BP}/d$) is compared to predicted H_b/d according to Goda (2000) (A) and to a modified expression (B), with the coefficient, A , adjusted for improved model fit.

5.4.3 Surf zone processes

5.4.3.1 Wave height transformations

Wave heights begin to diminish from around the break point, although as discussed within the previous section, some variation is observed in the exact break point location and in the position of maximum shoaling wave height, compared to break point. In general, waves breaking on steeper slopes are observed to decay more rapidly than waves on lower slopes. This is in agreement with the similarity decay model of Longuet-Higgins and Stewart (1963), which assumes a constant wave height-to-depth ratio between break point and the shore (Eqn. 5-3). Thus, wave decay on steeper slopes, where depth is decreasing more rapidly, must exhibit a faster rate of decay. In contrast, waves breaking on flatter slopes, where water depth is decreasing more gradually, decay less rapidly.

$$H = \gamma_b d \quad (5-3)$$

Where:
 γ_b = breaker index at break point
 d = depth

Wave height is observed to decay in a generally linear manner, again, in agreement with the findings and model of Longuet-Higgins and Stewart (1963). However, on the flatter 20:1 slope, decay appears to begin more rapidly and is followed by a reduction in decay rate through the surf zone. This is in agreement with the findings of Smith and Kraus (1990), who found that wave decay on flatter slopes followed a power law relationship. This is explained by rapid dissipation of energy immediately following breaking, which subsequently reduces within the surf zone as a turbulent bore is formed and waves tend toward a stable wave height (Dally et al., 1985). On steeper slopes, wave forms do not have time to transition towards this stable height.

Some irregularity is also noted in the decay of waves which break in a plunging manner (i.e. lower steepness wave forms on steeper bed slopes). These irregularities are identified as being related to splash-up events inshore of the initial break point.

Dissipation rates are typically evaluated using either the similarity methods of Longuet-Higgins and Stewart (1963) and Smith and Kraus (1990), which assume a constant or power law wave height to depth ratio between break point and the shore, or energy flux methods of LeMehaute (1962) and Battjes and Janssen (1978) as used by Dally, Dean and Dalrymple (1985), where the change in energy across the surf zone is evaluated using semi-empirical dissipation approximations. Both are described in more detail within Section 2.3.3. The similarity method assumes wave propagation over a plane slope and that the depth at the intersection of the bed and still water is zero with wave height correspondingly reaching zero at this point. The method has been found to overestimate wave height on slopes greater than around 1:30 and underestimate height on flatter slopes (USACE, 2003).

As wave height is observed to decay in a generally linear manner for these breaking cases, decay rates have been quantified using a linear best-fit curve to find the average change in wave height with horizontal distance across the surf zone. Results are presented within Table 5-6. Additionally, the predicted decay rates according to the similarity method of Longuet-Higgins and Stewart (1963) are calculated using the breaker depth indices calculated according to within Table 5-4 and assuming these indices are constant across the surf zone. The predicted decay rates are between 7 % and

120 % higher than observed, with the steepest slope exhibiting greater over-prediction. However, the similarity method assumes that water depth and wave height both reduce to zero at the still water intersection (swx). The following section will show that the water level is elevated at the swx and consequently, wave height does not reduce to zero. The predicted decay rates have therefore been recalculated taking into account the positive water level elevation at the swx due to set-up (as discussed within Section 5.4.3.2). Better agreement is found using this model variation with predicted decay rates differing from observed by between -4 % and 70%. Again, greater over-prediction occurred for the steeper slopes.

Table 5-6 Average rates of measured wave height decay (dH/dx) compared to decay rates predicted using the similarity method of Longuet-Higgins and Stewart (1963).

Wave case	Slope	Observed average dH/dx	Predicted dH/dx	Predicted dH/dx (including set-up)
Mono - Low ($s_0 = 0.11$)	5:1	0.2035	0.291	0.201
	10:1	0.0513	0.110	0.091
	20:1	0.0345	0.046	0.039
Mono – Mod ($s_0 = 0.22$)	5:1	0.1309	0.258	0.202
	10:1	0.0737	0.112	0.094
	20:1	0.0348	0.041	0.036
Mono – Steep ($s_0 = 0.29$)	5:1	0.1046	0.230	0.178
	10:1	0.0786	0.105	0.090
	20:1	0.0364	0.039	0.035

5.4.3.2 Mean water levels

The commonly observed reduction in mean water elevation prior to and increase following breaking are known as wave set-down and set-up. These changes are associated with momentum flux being directed in an offshore and onshore direction prior to and following breaking respectively. Both are evident within the monochromatic data sets shown in Figure 5-3 to Figure 5-5.

Set down is more pronounced on steeper slopes, with a rapid drop in water level correlating to the rapid increase in wave height evident immediately prior to breaking. This relation is in agreement with radiation stress theory (Longuet-Higgins and Stewart, 1962, 1963, 1964), where the strength of shoaling influences momentum flux and the

corresponding reduction in water level. Set down is generally observed to reach a maximum at around break point, after which point the mean water level begins to rise as momentum flux becomes directed in an onshore direction. However, temporal variation in breaking location means that within the breaking region, shoaling, breaking and decaying waves may exist over time. Mean momentum flux only becomes directed in an onshore direction when the onshore flux associated with dissipation exceeds the offshore flux associated with shoaling. The transition position between set down and set-up is therefore often located landward of the mean breaking position, especially for steeper waves on lower slopes, where the variation in breaking position is greatest. However, the opposite is observed for the lower wave case, which typically breaks in a plunging manner. The transition point appears located slightly seaward of the mean break point. This discrepancy is not addressed within the present study, although the result infers that momentum flux becomes directed in an onshore direction before the plunging jet impacts the surface. This disparity in the position of maximum set down and momentum flux transition has been well noted in the literature (i.e. Cokelet, 1977) and generally attributed to the manner of wave breaking. However, height modulations within the wave-train and variations in breaking position also appear to have influence.

Set-up gradient is evaluated using a linear best-fit curve between the transition point of set down and set-up and the swx. Gradient is observed to be related to both bed slope and wave steepness, with larger gradients being induced by waves breaking on steeper slope and by wave forms of lower offshore steepness (Table 5-7). However, both these cases result in waves breaking relatively close to the shore, thus limiting the horizontal distance over which the set-up gradient can act and limiting the resultant water elevation at the swx. In contrast, while waves of higher steepness and waves breaking on lower slopes tend to exhibit lower set-up gradients, they break further offshore, thus allowing greater horizontal distance for water level to become elevated. The resultant water elevation at the swx is a function of both these variables and is presented within Table 5-7. Set-up for the 10:1 bed slope has been extrapolated to the swx as the record was missing data through a portion of the inner surf zone. While set-up gradient can be observed to very greatly dependent on wave steepness and bed slope, maximum set-up elevations at the swx are more uniform, varying between 13% of breaking wave height for the steeper wave case on a 20:1 slope and 21% for the lower wave case on a 5:1 bed

slope. Slightly higher elevations are observed on steeper slopes and for lower waves; this trend is in agreement with existing literature (i.e. Holman and Sallenger, 1985)

Both the set-up gradients and water level at the still water intersection are compared to those predicted according to radiation stress theory (Longuet-Higgins and Stewart, 1963) based on the similarity wave decay model (Eqn. 5-3) and described within USACE (2003) (Eqn. 5-4 and 5-5). Predicted set-up gradients were found to exceed observation by between 30 % and 125 % and mean water level at the swx was found to exceed observation by between 5 and 70 %. Similar findings were reported by Stive and Wind (1982), with both the set-up gradient and elevation at the shoreline predicted by linear theory substantially exceeding that observed.

$$\frac{d\bar{\eta}}{dx} = \frac{1}{1 + \frac{8}{3\gamma_b^2}} \tan \beta \quad (5-4)$$

$$\bar{\eta}_s = \bar{\eta}_b + \left[\frac{1}{1 + \frac{8}{3\gamma_b^2}} \right] d_b \quad (5-5)$$

Where:

$d\eta$ = set-up gradient

η_s = set-up at the still water line

η_b = set down at break point

γ_b = breaking index (H_b/d_b) at break point

$\tan\beta$ = bed slope

While both the change in wave height across the surf zone and set-up may be evaluated separately from each other, in reality they are interrelated, with the rate of dissipation or change in wave height affecting momentum flux or radiation stress and therefore water level, and changes in water level affecting depth and therefore achievable wave height and rates of decay. Rate of wave height decay and set-up gradient are plotted against each other for both the observed and predicted cases (Figure 5-13). In both cases, set-up gradient is well correlated to wave height decay, with more rapid decay resulting in steeper set-up gradients. It should be noted that as both the expressions for predicting wave decay and set-up are based on similar theoretical assumptions, very high correlation is expected.

Table 5-7 Observed and predicted set-up gradients and elevations at the still water intersection as a function of breaking wave height.

Wave case	Slope	Observed			Predicted	
		Breaking Location (x_{swx}/L_o)	Set-up gradient dy/dx	Set-up $_{swx}/H_b$	Set-up gradient dy/dx	Set-up $_{swx}/H_b$
Mono - Low ($s_0 = 0.11$)	5:1	0.144	0.0678	0.21	0.0882	0.22
	10:1	0.414	0.0181	0.16 ¹	0.0312	0.23
	20:1	0.880	0.0061	0.17	0.0120	0.21
Mono – Mod ($s_0 = 0.22$)	5:1	0.277	0.0344	0.17	0.0769	0.24
	10:1	0.687	0.0182	0.14 ¹	0.0320	0.24
	20:1	1.718	0.0079	0.15	0.0101	0.22
Mono – Steep ($s_0 = 0.29$)	5:1	0.367	0.0416	0.20	0.0663	0.24
	10:1	0.889	0.0153	0.14 ¹	0.0293	0.24
	20:1	2.287	0.0046	0.13	0.0093	0.21

¹ Set-up extrapolated to still water intersection

Both series are found best fitted by a linear relationship, with the predicted values having a linear regression (correlation) coefficient of $r^2 = 0.9992$ and the observed values having a coefficient of $r^2 = 0.9307$. Using Pearson correlation analysis for 9 observations ($n = 9$), the correlation becomes significant at the 99.9% level when $r > 0.8982$, making the observed relationships are statistically significant.

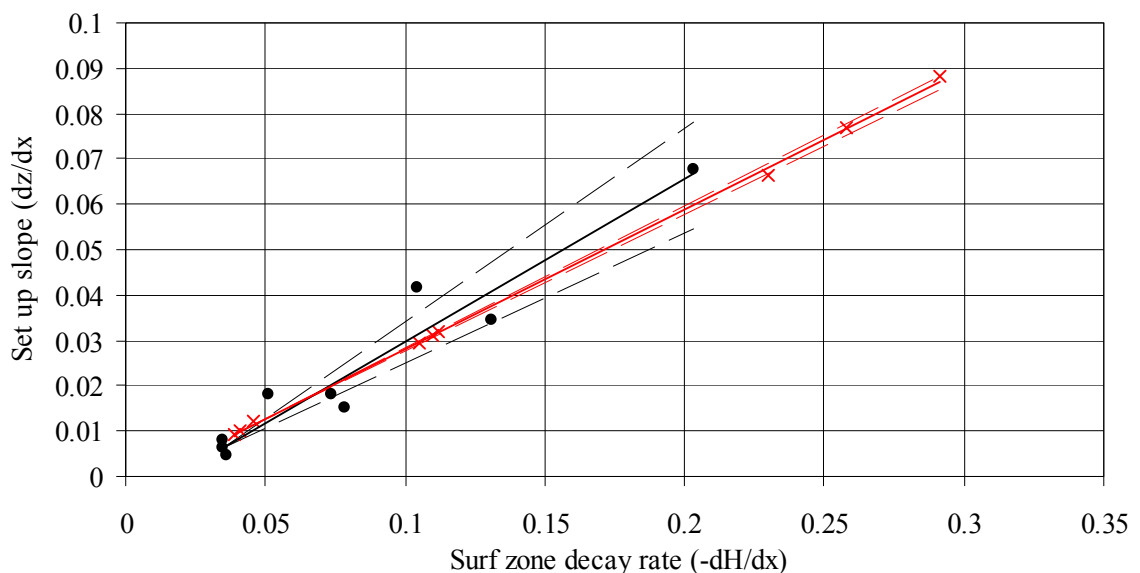


Figure 5-13 Relation between wave height decay rate within the surf zone (dH/dx) and set-up gradient (dz/dx) for both the observed (•) and predicted (×) values, with linear regression lines (—) and 90% confidence intervals (- - -) shown.

5.5 Summary and conclusions

This chapter has investigated the deep water propagation, shoaling, breaking and surf zone characteristics of three monochromatic wave structures of differing steepness propagating onto 5:1, 10:1 and 20:1 plane sloping beds. The primary objective of this monochromatic testing was to confirm the empirical expressions for the prediction of breaker index presented within modern design guidelines (Goda, 2000; USACE, 2003) and to investigate possible causes of the scatter apparent in raw breaker index data upon which these guidelines are based.

Some variation in observed breaking position and resultant breaker indices of the low and moderate steepness monochromatic wave trains was noted, in agreement with previous studies of regular wave breaking by Smith and Kraus (1990) and Goda (2007). Greater variation was noted however for the steepest wave-train. This greater variation appears largely due to the development of wave height modulation (up to 30%) during propagation. Such modulation has previously been noted in regular wave-trains (Melville, 1982; Hwung and Chiang, 2005) and is attributed to the development of instabilities in sideband frequencies (Benjamin and Feir, 1967). It is therefore proposed that a portion of the observed scatter in breaker index diagrams (i.e. Weggel, 1972; Goda, 1970), upon which modern design guidelines are based, is likely due to the development of modulation within propagating wave-trains.

Observed breaker indices were compared to those predicted by design guidelines. The USACE (2003) guidelines under-predicted breaker indices on steeper 5:1 and 10:1 slopes (by -17 to -25%) but were in better agreement on the lower 20:1 slope, with an over-prediction by guidelines of 2 to 5%. Values predicted by the Goda (2000) guidelines were in better general agreement, slightly under-predicting the 10:1 and 20:1 slope values (by -5 to -15%) and over-predicting the steeper, 5:1 values (+6%). Given the observed tendency for initially regular wave trains to become modulated and to develop groupiness during propagation, and given that current design guidelines are based upon such wave trains, a more thorough investigation of the effects of groupiness on breaker indices is justified.

The use of the optical LIF technique allowed precise quantification of absolute changes in the wave crest and trough amplitudes during shoaling and at break point, rather than just wave height but do not appear to have been quantified previously. These characteristics are important as they provide useful information on the geometric transformations that wave forms undergo during shoaling and they provide information on the maximum water level attained by the wave crest during shoaling and at break point. This is important for engineering design where maximum water levels must be known along with wave height.

The observation of crest and trough amplitudes during shoaling suggest that the initial reduction in wave height observed during shoaling is more a function of the decrease in trough amplitude, while the subsequent increase in height is due to the increase in crest amplitude exceeding the simultaneous decrease in trough amplitude. As water level is held largely constant during these later stages of shoaling (apart from a small amount of set down related to the offshore momentum flux), the width of the trough and crest must increase and decrease respectively to balance this mean elevation. This is supported by the observed increase in the ratio of maximum to minimum water levels ($z_{\max}/z_{\min}$) during shoaling. This ratio increases during shoaling, reaching a local maximum at break point, followed by a rapid decrease owing to breaking-induced energy losses within the crest. Similarly, ratios of maximum surface elevation to wave height at break point were found to be significantly higher than the 0.5 which would be expected in deep water where waves are nearly sinusoidal. Ratios were generally higher on flatter slopes and for lower steepness wave trains. This is likely related to the fetch length over which non-linearity may develop prior to breaking. The flatter slopes provide a greater fetch and the lower steepness wave trains begin to interact with the bed earlier and thus develop greater non-linearity and asymmetry. These results imply that on flat slopes and for low steepness wave trains, the maximum surface elevation relative to still water level at break point may comprise a significant (up to 85%) portion of the wave height.

The ratio of maximum water level to depth at break point $z_{\max-BP}/d$ was found to be in general agreement to breaker height index trends, with lower steepness waves and steeper slopes exhibiting higher ratios and steeper waves and flatter slopes exhibiting lower ratios. These ratios were compared to values predicted by the Goda (2000) expression for breaker height indices. As expected, the expression over-predicted

values, particularly for the steeper 5:1 slope where less non-linearity limits the crest amplitude. The coefficient, A was adjusted to provide an improved fit, with A reduced to 0.15 for the 10:1 and 20:1 slopes and to 0.11 for the 5:1 slope. Variation in $z_{\text{max-BP}}/d$ for the differing wave steepnesses appears to be well predicted by the expression. While further testing on slopes flatter than 20:1 is required, it is likely that as the bed flattens, crest asymmetry will become more pronounced and the coefficient, A will tend towards 0.17. Obtaining an accurate definition of this maximum water level is important for engineering design.

Chapter 6 Group influence on breaking of shoaling water waves

6.1 Introduction

While energy-fluxes associated with wave group processes have been shown to play a critical role in the incidence and strength of wave breaking in deep water (Banner and Peirson, 2007), our understanding of their influence on shallow-water wave breaking is limited. In shallow water, where depth-limited breaking occurs, dispersion becomes less pronounced and therefore intra-group processes could be assumed negligible and non-influential to wave breaking.

Preliminary studies (Shand et al., 2007) showed that the interaction of horizontal (intra-group) and vertical (depth-induced) energy fluxes played a key role in the shoaling and breaking processes of waves within a group. The study indicated a critical depth, where group structure at that point influenced the subsequent shoaling and breaking dynamics. This early study was undertaken using one group structure, C2N5r, and on a 10:1 slope bed. The study was also limited to water depths greater than 50 mm due to instrumentation limitations, meaning that only the one or two largest waves at break point were typically captured. These shortcomings have been addressed within the current, more comprehensive study, through the use of an optical LIF method for data acquisition (refer Section 3.3.2).

This chapter first describes the observed evolution of local energy density in the four tested wave groups propagating within deep water. A method is then presented for altering the spatial phasing of this group evolution with the initiation of shoaling by adjusting the along-flume position of a sloping bed. The wave group evolution in shoaling water is then described including wave and surface envelope evolution, breaking positions and heights and changes to the mean and temporally varying water levels within the surf zone. The threshold depth at which the state of group evolution determines future shoaling patterns and breaking position is discussed and a linear model for predicting the locations of local energy density convergence points is presented. Finally, errors and study limitations are addressed. The implications of these

group effects for engineering design are found to be important and are further discussed within the following chapter.

6.2 Experimental parameters

6.2.1 Wave parameters

As discussed within Section 3.4, four wave group structures were tested. The wave groups varied in their generation mechanism (*Case*), the number of waves within the group (N), and the initial wave steepness ($s_0 = a_0 k_0$) leading to either recurrent or breaking conditions. A summary of the parameters used to generate each of these wave *configurations* is presented in Table 6-1. Note the value N is a spatial measure of the number of waves within a group. A wave group observed at a fixed point in space over time exhibits around twice this number of waves in deep water (i.e. 10 waves for an $N = 5$ case) due to wave dispersion.

Table 6-1 Parameters of the three monochromatic and four wave group configurations tested during this study

Wave configuration	Group Case	Number of waves in group (N)	Initial wave number (k_0)	Initial mean wave amplitude, a_0 (m)	Initial wave steepness $S_0 = (ak)_0$	Evolutionary manner
C2N5r	2	5	7.4178	0.0193	0.143	Recurrent
C2N5sb	2	5	7.4178	0.0216	0.16	Strong Breaking ¹
C2N3r	2	3	7.4178	0.0226	0.168	Recurrent
C3N5	3	5	6.8068	0.0184	0.125	Recurrent

¹ Refers to type of breaking in deep water (Song and Banner, 2002)

6.2.2 Deep water wave group evolution

As discussed within Section 3.4, the group envelope ($z_{\max}$ and $z_{\min}$) can be observed to fluctuate between alternative positions of local crest maxima and trough minima. These occur during dispersive propagation when a wave crest or trough is coincident with the group envelope maximum and represent positions of maximum or minimum local energy density convergence. Figure 6-1 shows a conceptual diagram of this mechanism.

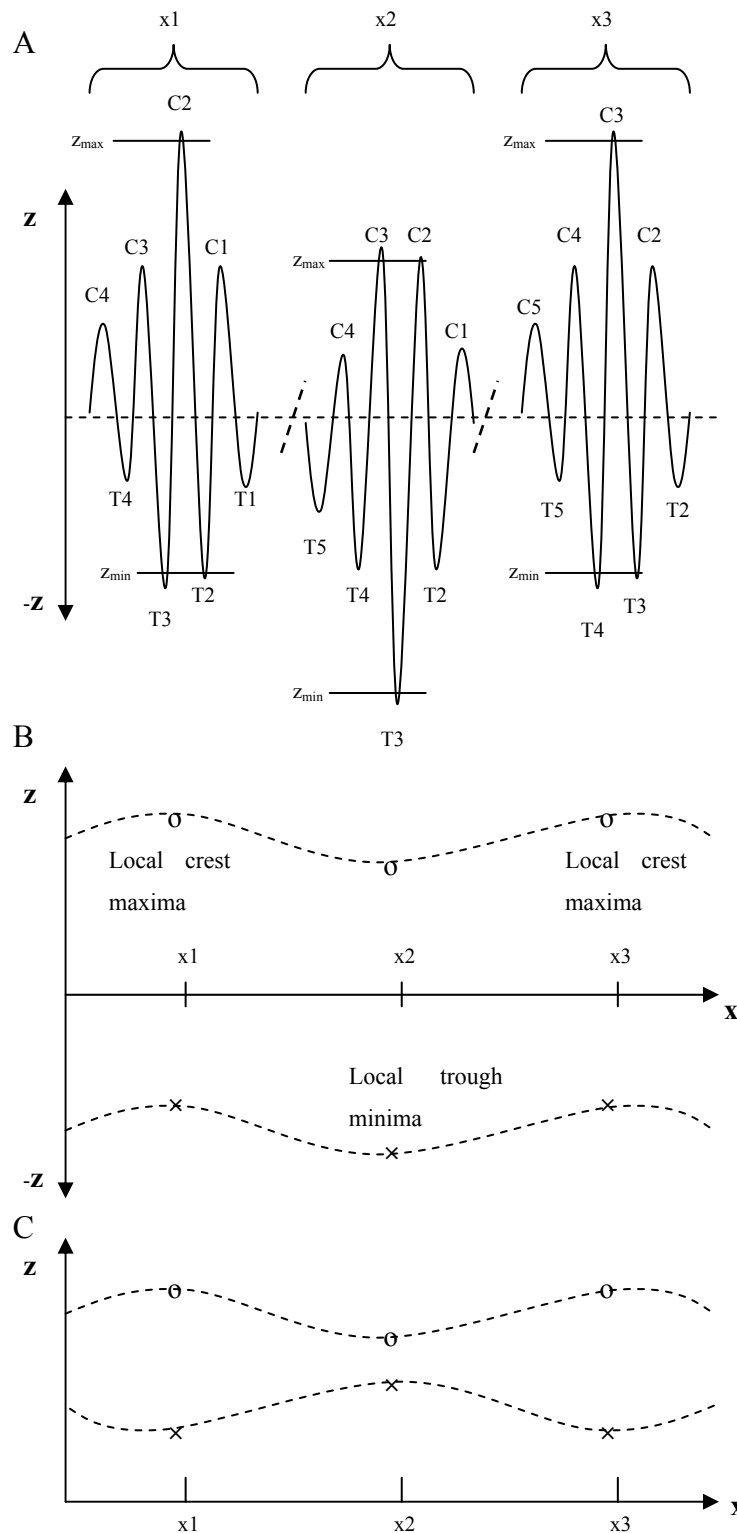
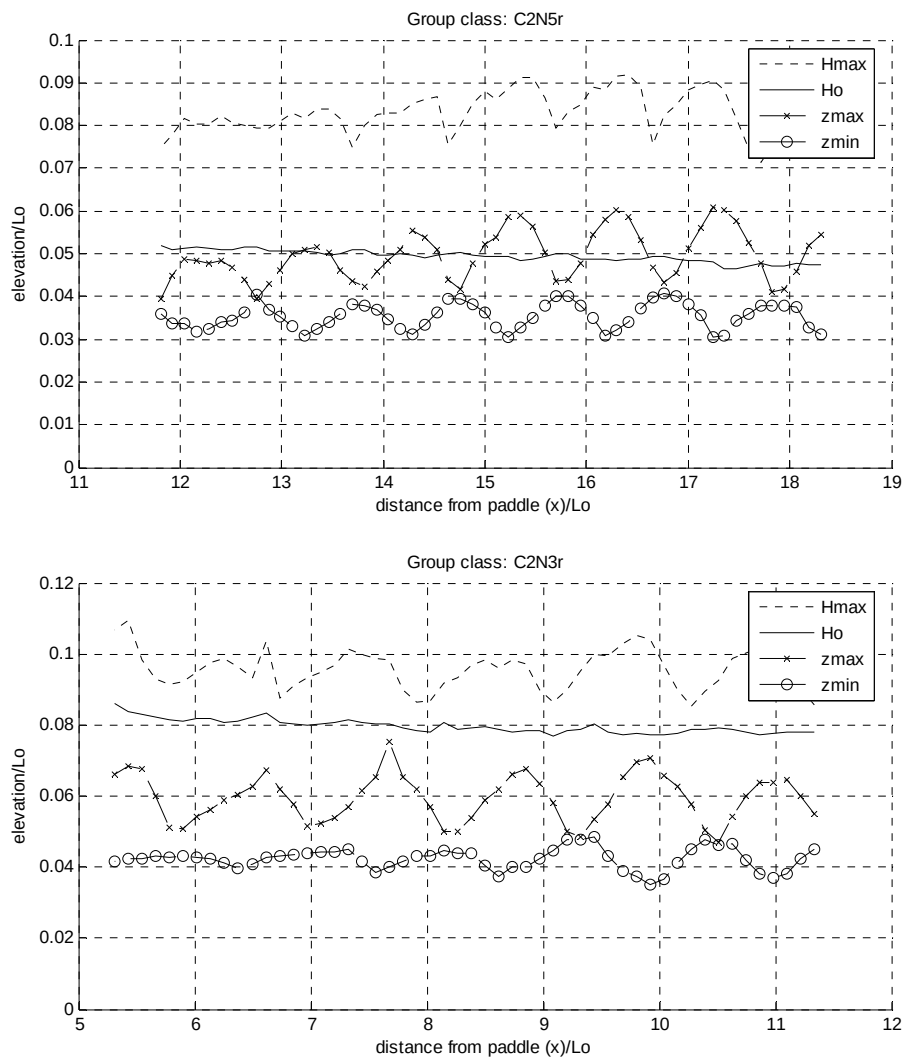


Figure 6-1 Conceptual diagram of wave position at three sequential spatial locations (A) with waves observed to form at the rear of the group, move through the group increasing in size until reaching a maximum before reducing towards the front of the group. The envelope maximum and minimum is extracted at each spatial location and positions of local maxima and minima are observed corresponding with the maximum wave crest and trough amplitudes (B). Absolute values of $z_{\min}$ are then obtained (C).

The maximum and minimum ($z_{\max}$ and $z_{\min}$) elevations of the group envelopes during propagation in deep water, along with the $H_{\max}$ and H_{m0} values, as defined in Section 4.4.2, are non-dimensionalised by the deep water wavelength (L_0) and presented within Figure 6-2. For each group configuration, the magnitude of these local maxima and minima, as defined by the elevation of $z_{\max}$ and $z_{\min}$ respectively, can be observed to increase with distance along the tank as the groups evolve to either breaking onset or recurrence, where a maximum is reached without breaking (refer Section 2.2.3). The crest elevation ($z_{\max}$) typically displays greater variance relative to the mean water level than the trough elevation. This illustrates the non-Gaussian characteristics of the propagating wave-forms.



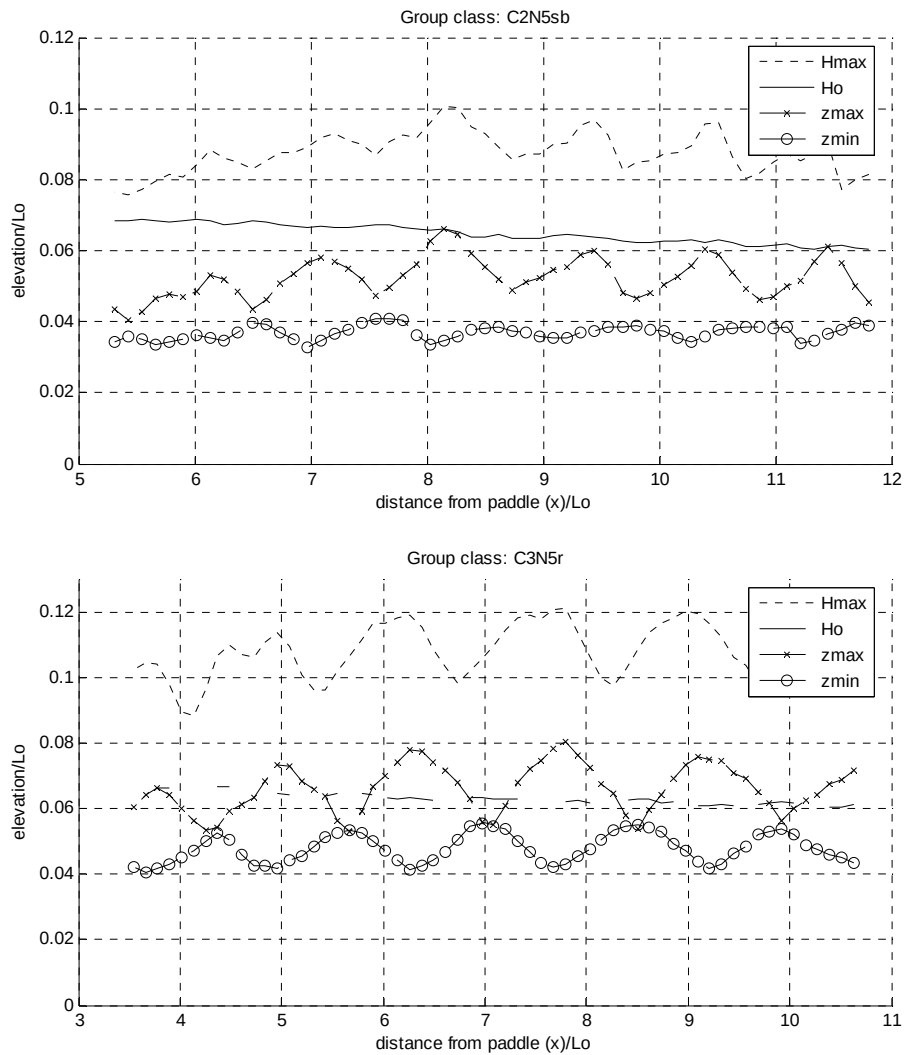


Figure 6-2 Deep water evolution of maximum and energy-based significant wave heights ($H_{\max}$ and H_0), maximum surface elevation ($z_{\max}$) and the absolute minimum surface elevation ($z_{\min}$) for the four group configurations as a function of deep water wavelength (L_0). Measurements are obtained using an array of capacitance wave probes, incrementally shifted along the wave tank.

The position of these maximum energy convergence points (local maxima) and positions of recurrence or breaking are given in Table 6-2. Case 2 type groups can be observed to evolve more rapidly when the initial group is steeper (C2N5sb) or the numbers of waves within the groups are lower (C2N3r). Case 3 type groups (C3N5r) evolve more rapidly due to their focussed generation mechanism. Following this point of either recurrence or breaking onset, the magnitude of local maxima and minima begin to decrease. The magnitude of maxima/minima in the only breaking case, C2N5sb, can be observed to decrease more rapidly following breaking. This is assumed due to the higher energy losses sustained during breaking (refer Banner and Peirson, 2007).

Maximum wave height follows a similar fluctuation with distance, generally reaching positions of local maxima near the $z_{\max}$ local maxima positions. This is due to the wave height being a function of both $z_{\max}$ and $z_{\min}$, with $z_{\max}$ typically the more dominant fluctuation. The $H_{\max}$ similarly increases in amplitude as the group evolves and approaches recurrence or breaking onset before subsequently decreasing.

The energy-based significant wave height (H_0), however, behaves differently, with minimal fluctuation occurring as the wave group evolves. This indicates that while the local energy density of the group envelope fluctuates, the mean energy density across the group remains near constant. A small, near-linear reduction in H_0 is evident with distance along the flume. This equates to a change in energy density with distance along the flume. Banner and Peirson (2007), undertaking experiments on energy losses associated with deep water wave breaking, noted both the frictional effect of the flume sidewall (Rapp and Melville, 1990) and breaking related losses as contributing to overall changes in potential energy. Frictional losses as a result of side wall interaction are further discussed in Section 6.4.3.5.

Table 6-2 Positions of local minima and maxima, as defined by the difference between $z_{\max}$ and $z_{\min}$ shown within Figure 6-2 with the point of recurrence (r) or breaking (b) indicated.

Group	C2N5r	C2N3r	C2N5sb	C3N5r
Local	x'/L_0	x'/L_0	x'/L_0	x'/L_0
Maxima	12.12	5.41		3.76
Minima	12.71	5.88	5.41	4.35
Maxima	13.18	6.59	6.12	5.06
Minima	13.76	6.94	6.47	5.65
Maxima	14.24	7.65	7.06	6.24
Minima	14.71	8.24	7.53	6.94
Maxima	15.18	8.82	8.12(b)	7.76(r)
Minima	15.76	9.29	8.71(b)	8.47
Maxima	16.24(r)	9.88(r)	9.41(b)	9.18
Minima	16.71	10.47	9.76	9.88
Maxima	17.18	10.94	10.35	
Minima	17.76		10.82	
Maxima			11.41	
Minima			11.76	

Examples depicting the surface elevation in time near positions of local trough minima and crest maxima, along with wave height and period obtained by zero down-crossing are presented within Figure 6-3. Note, the pronounced trough at the trough minima location (Figure 6-3A), and the pronounced crest at the position of crest maxima (Figure

6-3B). Along with amplitude modulation, frequency modulation is also observed, with lower wave periods observed for the largest waves within the group. This implies that the amplitude and frequency modulation are 180° out of phase with one another, in agreement with findings of Melville (1982), who used a Hilbert-transform technique to investigate amplitude and frequency modulation. Also in agreement with findings of Melville (1982) is the pronounced asymmetry evident both the envelopes of amplitude and frequency modulation, with steeper front faces and more gently sloping backs. This implies larger waves occur at the front of highly nonlinear groups.

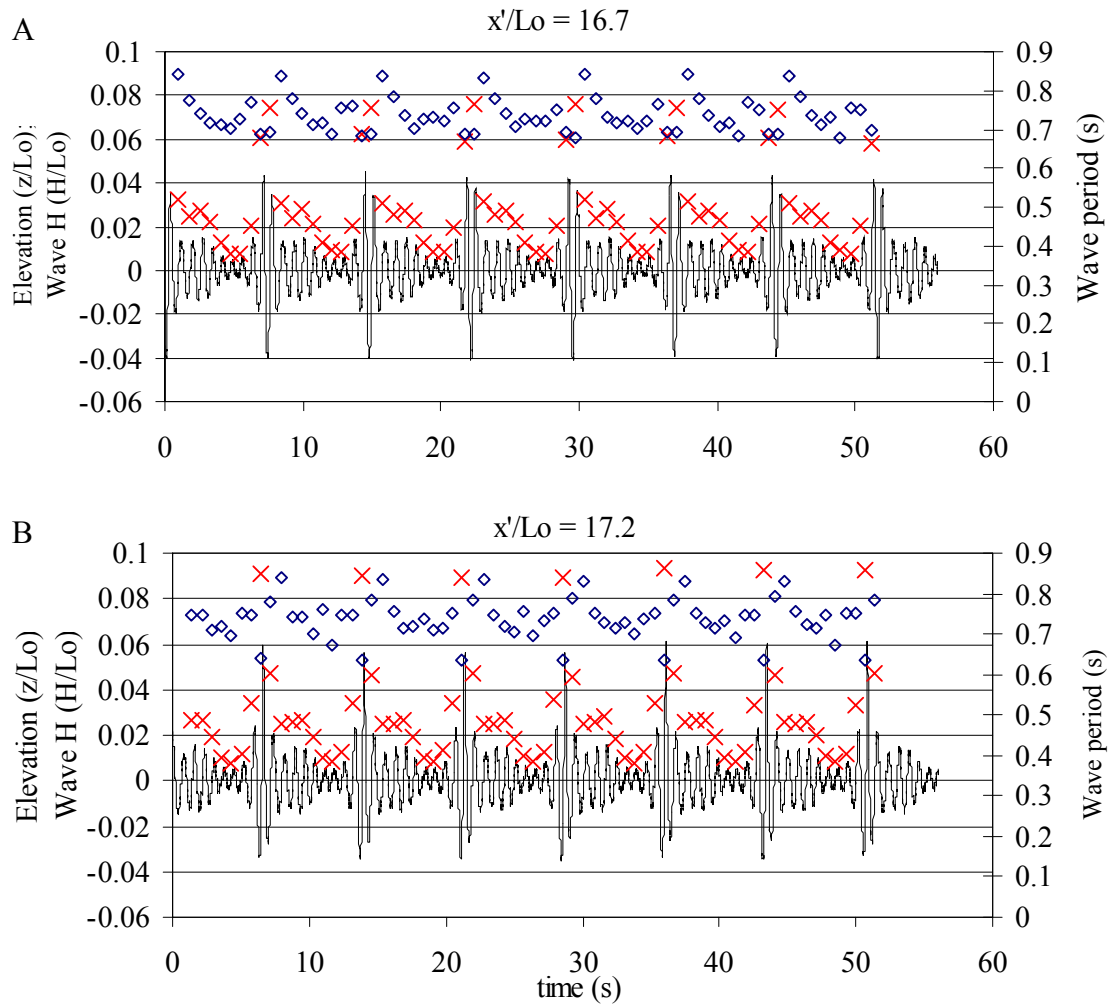


Figure 6-3 Comparisons of water surface elevation (z ; —) and wave height (H ; $\times$) and period (T ; $\diamond$) obtained from zero-downcrossing for a C2N5r type group at positions of local trough minima at $x/L_0 = 16.7$ (A) and local crest maxima at $x/L_0 = 17.2$ (B). Measurements are obtained using an array of capacitance wave probes.

While the phase and asymmetric trends of frequency modulation is consistent regardless of the group evolutionary state, the amplitude of frequency modulation varies with group

evolutional state, i.e. the period exhibited by the largest wave during a crest maxima position is lower than the period which occurs during the largest waves at a trough minima position. In both evolutional states, and for all other locations within the flume, the mean wave period is consistent. This implies that local energy convergences not only result in higher waves within the group, but also waves of shorter wavelength, thus enhancing the local steepness of these waves.

6.2.3 Group phasing positions

Preliminary studies (Shand et al., 2007) showed that the spatial phasing of the wave group evolutional cycle, (i.e. whether the group is in a state of crest maxima or trough minima), at a depth between $L_0/4$ and $L_0/7$ determines the subsequent shoaling and breaking characteristics. These differences are expected to become more pronounced with increasing group nonlinearity, i.e. immediately prior to recurrence or the onset of breaking. Testing has therefore been focussed around these positions.

For each of the bed slopes and wave group types tested, the sloping bed at a depth of $L_0/4$ is aligned with a position of deep water local crest maxima. The bed is then incrementally adjusted and testing repeated until the $L_0/4$ depth corresponds to the following crest maxima. This encompasses a complete crest maxima - trough minima - crest maxima cycle, ensuring that *differences* in shoaling and breaking characteristics related to the spatial phasing of group evolution and shoaling are effectively captured. The positions of the swx for each bed slope and wave group types tested are given in Table 6-3.

Table 6-3 For each of the wave group structures and bed slopes tested, the range of bed positions, denoted by the swx location from the wavemaker (x_{swx}/L_0) is indicated along with the number of bed positions tested for each case.

Slope	Wave case	SWX (x_{swx}/L_0)		Spatial steps
		From	To	
5:1	C2N5r	15.25	16.78	14
10:1	C2N5r	16.47	17.65	11
	C2N3r	11.29	12.24	3
	C2N5sb	10.59	11.76	6
	C3N5r	8.94	10.12	6
20:1	C2N5r	17.29	18.35	5

6.3 Wave group evolution in shallow water

In order to assess differences in the wave shoaling and breaking properties caused by altering the spatial phasing of the group, a number of characteristics are examined. Firstly, changes to the group envelope, wave heights and water levels as the group transitions from deep to shallow water and into the surf zone are examined, using a combination of capacitance probe and the optical LIF methods. Secondly, the locations and types of wave breaking within the groups are observed using video imagery.

6.3.1 Wave and group envelope transformations

A preliminary study, undertaken prior to extensive testing using the optical-LIF method, was carried out for a single wave group configuration using capacitance-type wave probes. This study is described in greater detail within Shand et al. (2007) but utilized a 10:1 plane sloping bed and a C2N5r wave configuration as described within Table 6-1.

Testing was focused around positions of maximum energy density convergence occurring during deep water wave-train propagation (i.e. near the position of recurrence), with the bed first being positioned so that breaking occurred at a location of local crest maxima. The bed was then incrementally moved forward within the wave tank and testing repeated until breaking occurred at the previous location of local crest maxima, thus encompassing a complete crest maxima-trough minima cycle.

Water surface elevation was collected for a number of positions between deep water, offshore of the sloping bed toe, and shallow water to a depth of around $L_0/15$, this being the depth which the probes were physical limited to. This data was analysed to extract maximum crest and minimum trough elevation series along the tank with respect to the mean water level. The minimum trough elevation was inverted giving an absolute value.

Maximum and minimum surface elevations ($z_{\max}$ and $z_{\min}$), non-dimensionalised by deep water wavelength, are presented in Figure 6-4 with respect to absolute position within the wave tank as well as relative to the sloping bed-still water intersection (swx). For each series, the position of the sloping bed swx within the flume is given in terms of x'/L_0 within the figure legend. Additionally, the $z_{\max}$ and $z_{\min}$ series without the presence of a sloping bed are included for comparison.

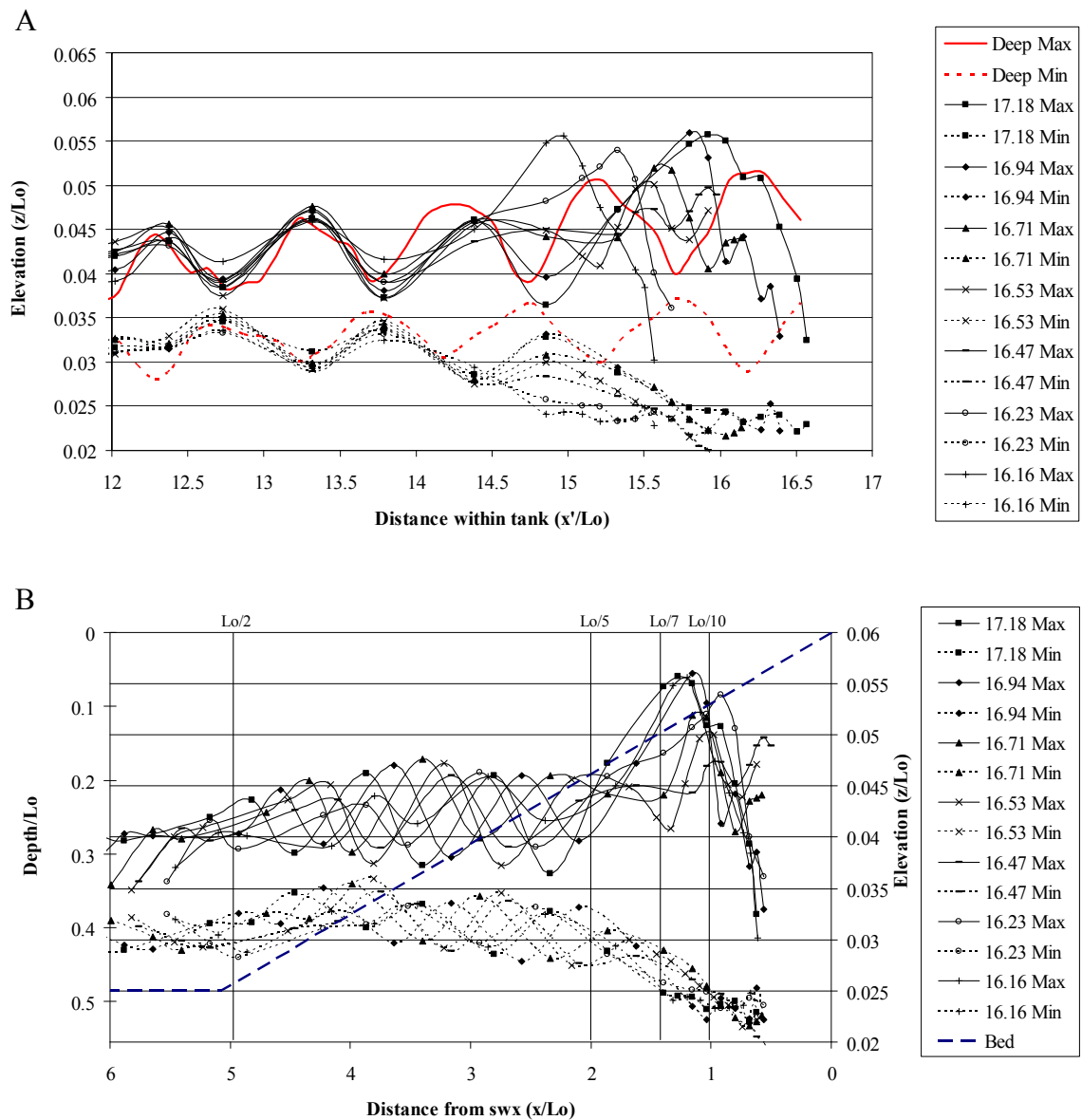


Figure 6-4 Non-dimensionalised maximum (—) and minimum (---) surface elevation for a C2N5r type group propagating onto a 10:1 slope as a function of distance from the paddle (A) and distance from the bed-still water intersection (B). The equivalent deep water maximum and minimum surface elevations without the presence of a sloping bed are shown within (A) and sloping bed location and depth within (B) (source: adapted from Shand et al., 2007).

Figure 6-4A shows that in deep water ($x'/L_0 < \sim 13.5$), the cyclic trends of all the wave groups tested within the study were in good agreement with purely deep water trends. This is important as it demonstrated good repeatability and robustness of the experimental method and also that reflections that may be caused by the sloping bed are not having significant effect. However, once the groups enter shallow water and begin

to shoal ($x/L_0 > \sim 13.5$ within Figure 6-4A), deviations from the deep water evolutionary cycle begin to occur. Primarily, the amplitudes of $z_{\max}$ and $z_{\min}$ begin to reduce and the position of the local crest and trough maxima and minima become delayed.

While the amplitudes of the local crest and trough maxima and minima reduce by differing amounts dependent on local water depth, the same general pattern is maintained. However, inshore of this location, where shoaling begins to become predominant, significant variation from the deep water evolutionary cycle becomes apparent. This can be best observed in Figure 6-4B, where in water deeper than $L_0/5$ waves, although out of phase with each other due to the horizontal datum offset between the respective series, follow the general deep water cycle. Inshore of this depth, the group cycle becomes less predominant. However, the subsequent evolution of the groups differs markedly, dependent on the evolutionary state of the group, i.e. whether the group is in a state of crest maxima, trough minima, or in between these extremes, as it reaches around this depth.

Between the depths of $L_0/5$ and $L_0/7$, groups approaching a position of local crest maxima (i.e. groups with bed positions at $x_{\text{swx}}'/L_0 = 17.18, 16.97$ and 16.16) depart from the deep water cycle and $z_{\max}$ continues to increase in elevation, while $z_{\min}$ continues to decrease in absolute elevation. This appears to be the location where shoaling transformations replace intra-group interaction as the dominating factor influencing energy fluxes. Groups approaching a local trough minima position between $L_0/5$ and $L_0/7$ (i.e. bed positions at $x_{\text{swx}}'/L_0 = 17.71, 16.53, 16.47$ and 16.23) continue towards this local minima position before $z_{\max}$ increases in elevation very rapidly prior to breaking. In these cases, the energy fluxes within the group appears to remain dominated by intra-group interactions for longer before the shoaling process takes over. Thus, the relative phasing of the group (i.e. whether the group is approaching a local crest maxima or trough minima position) as it reaches approximately this depth appears important in determining the future evolution of the wave group. These differing types of shoaling were referred to as *early* and *delayed shoaling* groups respectively.

However, the limited number of probe measurement positions and the limited depth up to which measurements could be obtained, restricted more detailed characterisation of these apparently differing types of shoaling. A more thorough investigation, utilizing a

greater number of wave group types propagating onto beds of differing slopes was undertaken using a more comprehensive optical-LIF technique for data collection. This allowed near-continuous observation of the shoaling characteristics from deep water, to break point and throughout the surf zone.

For each of the wave group types, bed slope and group spatial phasing combinations as described within Table 6-3, the evolution of the maximum and significant wave heights, maximum and minimum surface elevations and the mean and low frequency water level characteristics are extracted and are presented within Appendix D. Due to the greater number of bed position-group spatial phasing cases examined during this more comprehensive testing programme, and the substantially greater amount of data collected for each case, it is not convenient or beneficial to display the data in entirety in a similar manner to that presented within Figure 6-4. Instead, the ratio of maximum wave height to water depth ($H_{\max}/d$) is examined at specific depths. Based on the results of this, critical cases may be selected and their shoaling patterns compared in their entirety. These ratios are presented for each of the wave group type and bed slope combinations within Appendix D.

An example of the variation in $H_{\max}/d$ for the C2N5r type group propagating onto a 10:1 slope using the optical-LIF method is presented within Figure 6-5. This group type and bed slope combination, referred to hereafter as the *example case*, has been selected as it is comparable to that used during the preliminary study, thus allowing verification of results obtained by both methods and it was tested using the greatest number of different bed positions. The following paragraphs describe the selection of extreme or critical group-phasing series for the example case (denoted by colour bands on Figure 6-5 and subsequent related figures) and the differences in the shoaling and breaking characteristics of these critical series. Similar figures and tables are presented for all other wave group type and bed slope combinations within Appendix D, with an examination of the differences and variation between these cases and the example case given within the discussion section.

A group exhibiting none of the group evolutionary characteristics described in Section 6.2.2 and propagating onto the sloping bed in the same manner regardless of the spatial phasing should exhibit ratios increasing with decreasing depth, but uniform across all

bed positions. This is clearly not the case, with variation evident for differing bed positions. The results are presented numerically within Table 6-4 and show the maximum variation or difference in the $H_{\max}/d$ ratios between different group phase positions to increase with decreasing depth. This implies that the effects of the initial group spatial phasing become most pronounced within shallow water.

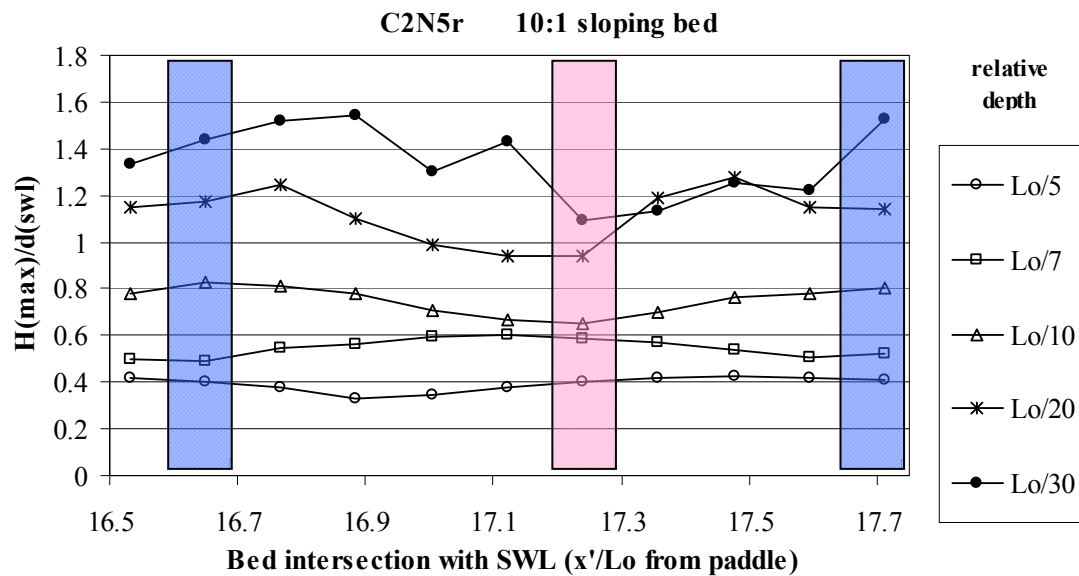


Figure 6-5 $H_{\max}/d$ ratios for varying bed positions for a C2N5r type wave group propagating onto a 10:1 sloped bed as a function of water depth. Selected example cases corresponding to approximate extremes at the $L_0/7$ to $L_0/10$ depths are depicted in red (crest maxima) and blue (trough minima) shading.

At $d=L_0/7$, groups nearer a position of local crest maxima (those nearer the centre of the plot) have a higher maximum wave height and therefore a higher $H_{\max}/d$ ratio compared to those nearer a local trough minima position (those nearer the edge of the plot). However, the groups nearer a local crest maxima position at this point tend to exhibit less growth in $H_{\max}/d$ as they continue to shoal compared to those groups nearer a trough minima position, whose $H_{\max}/d$ ratios increase more markedly in shallower water. Extreme examples of these differences have been highlighted with red corresponding to groups near a local crest maxima position at $d=L_0/7$ which experience a gradual increase in $H_{\max}/d$ and blue corresponding to groups near a local trough minima position at $d=L_0/7$ and experiencing more pronounced increase. These extreme examples occur with the bed swx at $x'_{\text{swx}}/L_0 = 16.65$ (rapid increase), 17.24 (gradual increase) and 17.71 (rapid increase) from the paddle. The characteristics of these cases,

referred to hereafter as the 16.25, 17.24 and 17.71 series, will be examined in greater detail

Table 6-4 Hmax/d ratios for varying bed positions for a C2N5r type wave group propagating onto a 10:1 sloped bed as a function of water depth.

Non-dimensional depth	$L_0/5$	$L_0/7$	$L_0/10$	$L_0/15$	$L_0/20$	$L_0/30$	$L_0/40$
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
16.53	0.42	0.49	0.78	1.00	1.15	1.34	1.51
16.65	0.40	0.49	0.82	0.94	1.17	1.44	1.55
16.77	0.38	0.54	0.81	0.86	1.24	1.52	1.66
16.89	0.33	0.57	0.78	0.94	1.10	1.54	1.64
17.00	0.35	0.60	0.70	0.98	0.99	1.30	1.62
17.12	0.38	0.61	0.67	0.98	0.94	1.43	1.59
17.24	0.40	0.59	0.65	0.96	0.94	1.09	1.37
17.36	0.42	0.57	0.70	0.92	1.19	1.13	1.27
17.48	0.42	0.54	0.76	0.91	1.27	1.25	1.34
17.59	0.42	0.51	0.78	0.93	1.15	1.23	1.93
17.71	0.41	0.52	0.81	0.95	1.14	1.53	1.73
Max difference	0.09	0.11	0.17	0.14	0.34	0.45	0.66

Figure 6-6 depicts the evolution of the maximum and minimum surface elevations ($z_{\max}$ and $z_{\min}$) for the C2N5r group type propagating onto a 10:1 slope bed with the bed located in the three selected positions as described earlier. The equivalent deep water characteristics without the presence of a sloping bed are shown for comparison. The toe of the sloping bed for the three example cases is located at $x'/L_0 = 12.0$, 12.58 and 12.05 respectively.

Within the figure, a local trough minima point is reached at $x'/L_0 = 13.75$ by all bed positions series in good agreement with the deep water series, although the 16.65 series does exhibit slight delay as it has begun to encounter the sloping bed to a greater extent than the other cases. As they move into shallower water, their evolution patterns deviate from that observed in deep water as they begin to interact more significantly with the bed and the dispersive processes slow. This relationship is explored further within the discussion section 6.4.2. The subsequent local crest maxima position is delayed in all

bed position series and the following trough minima position is further delayed, although again, most noticeably for the 16.65 series, which encounters the sloping bed first. The maximum surface elevation of this series then increases rapidly, reaching a peak maximum elevation at $x'/L_0 = 15.7$ before decreasing to a minimum at the swx position. The 17.24 and 17.71 series also reach this crest maxima position at around $x'/L_0 = 15.7$, with the 17.24 series continuing to a peak elevation at $x'/L_0 = 15.8$ after which point the elevation decreases. The 17.71 series decreases approaching a further position of trough minima before the maximum surface elevation increases rapidly to a maximum at $x'/L_0 = 16.6$. The series then decreases rapidly.

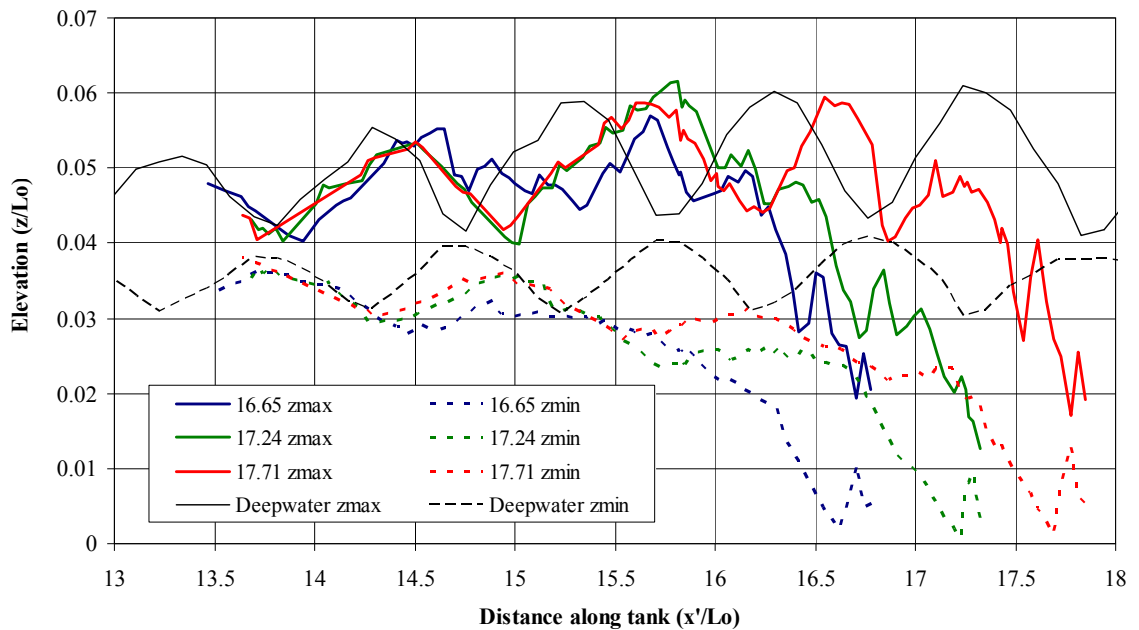


Figure 6-6 The evolution of the minimum and maximum surface elevation ($z_{\min}$ and $z_{\max}$) with distance along the wave tank for a C2N5r-type wave group propagating onto a 10:1 sloped bed located at three differing along-flume positions denoted by the x'_{swx}/L_0 within the legend. Also, the equivalent deep water characteristics without the presence of a sloping bed are included for comparison.

The x-axis is adjusted to give all distances relative to the respective swx locations, with the evolution of the maximum and minimum surface elevations ($z_{\max}$ and $z_{\min}$) presented within Figure 6-7A and the maximum and significant wave heights ($H_{\max}$ and H_0) within Figure 6-7B. Within deeper water, the surface elevation patterns of the 16.65 and 17.71 series are near in phase with each other, while the 17.24 series is near 180° out of phase. While the 17.24 series can be observed to peak at depth = $L_0/7$ before decreasing towards the swx, the 16.65 and 17.71 series peak later, closer to depth = $L_0/10$. This

delay shows a larger wave height being maintained into shallower water, as illustrated by the $H_{\max}/d$ ratios within Figure 6-5. Decrease in wave height immediately following the maximum height is notably sharper for the 16.65 and 17.71 series than for the 17.24 series. Smaller peaks in maximum surface elevation following the largest peak are evident in all cases, although occur further offshore for the 17.24 series.

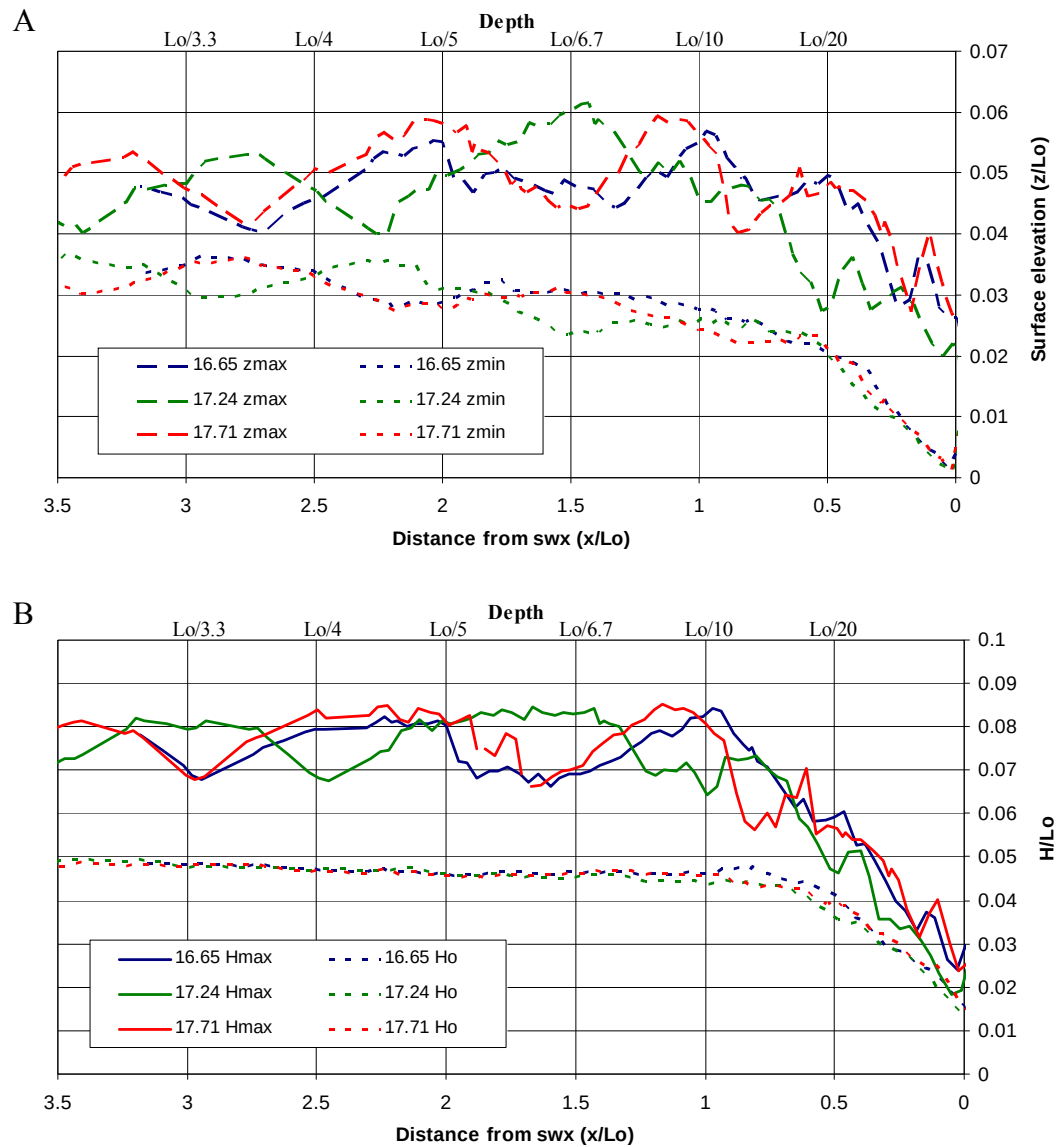


Figure 6-7 The evolution of the minimum ($z_{\min}$) and maximum ($z_{\max}$) surface elevation (A) and the maximum ($H_{\max}$) and significant (H_0) wave heights (B) relative to the swx position for a C2N5r-type wave group propagating onto a 10:1 sloped bed located at three differing along-flume positions denoted by the x'_{swx}/L_0 within the legend.

Differences in minimum surface elevations are more subdued once they depart the deep water pattern, with all series tending to decrease inshore of around $x/L_0 = 2.5$ or depth = $L_0/4$. This indicates that, similar to the monochromatic series, the wave trough decreases

in amplitude during shoaling, with the increase in crest elevation during shoaling being responsible for maintaining or increasing wave height. From around depth = $L_0/20$, the reduction in trough amplitude occurs more rapidly until it reaches near zero at the swx in contrast to the maximum surface elevation which maintains a finite amplitude at the swx. Similar $z_{\min}$ characteristics occur for all cases, although the 17.24 series displays more initial amplitude reduction than the other series.

Maximum wave height follows the trend of the maximum surface elevation with the 17.24 series reaching a maximum earlier than the other series and subsequently decreasing more gradually with additional small peaks at depth = $L_0/12$ and $L_0/25$. The 16.65 and 17.71 series peak at around depth = $L_0/10$ with less pronounced inshore peaks. Maximum wave height at the swx remains at around $H_{\max}/L_0 = 0.026$ (~30 % of peak $H_{\max}$) for the 16.65 and 17.71 series while the 17.24 series reduces to around $H_{\max}/L_0 = 0.02$ (~24% of peak $H_{\max}$).

The change in maximum wave height ($dH_{\max}/dx$) and surface elevation ($dz_{\max}/dx$) approaching peak respective values, corresponding roughly to the seaward-most break points as discussed within the following section, is presented within Table 6-5. These values show evolution of the maximum surface elevation to occur more rapidly than maximum wave height in all cases. This infers that as $z_{\max}$ increases, $z_{\min}$ is simultaneously decreasing, thus moderating the increase in $H_{\max}$. The 16.65 and 17.71 series show steeper increases in both $H_{\max}$ and $z_{\max}$ than the 17.24 series.

Table 6-5 Evolution of the maximum surface elevation and wave height approaching peak respective values.

Position of (x_{swx}'/L_0)	$dz_{\max}/dx$	$dH_{\max}/dx$
16.65	0.040	0.031
17.24	0.023	0.020 ¹
16.77	0.054	0.037

¹ note: change in $H_{\max}$ slope *immediately* prior to peak value for this series is near horizontal. Gradient represents increase over a distance of approximately $x/L_0 = 1$.

The evolution of the significant wave height is more subdued than the maximum wave height or surface elevation, with little offshore signature of group evolution evident.

This is, as discussed earlier, due to the energy-based method of determining the height. The measure is therefore unaffected by changes in local energy density. Significant wave height decreases gradually and relatively uniformly from deep water to around the peak $H_{\max}$ (around $L_0/7$ for the 17.24 series and $L_0/10$ for the 16.65 and 17.71 series). This initial decrease is explained largely due to sidewall friction and other forms of energy loss as described within Section 6.4.3. From these peak $H_{\max}$ positions, the significant height begins to decrease more notably until around depth = $L_0/20$. After this point, the height decreases more rapidly and near linearly to the swx. As the significant height of the 17.24 series begins to decrease earlier than the other series, it has a slightly lower H_0 value throughout, although its final value at the swx of $H_0/L_0 = 0.015$ (~32% of H_0 at peak $H_{\max}$) is only marginally lower than the other series which reach a value at the swx of $H_0/L_0 = 0.016$ (~34% of H_0 at peak $H_{\max}$).

The ratio of maximum to significant wave height reduces through the surf zone from around 1.8 at peak $H_{\max}$ to between 1.3 and 1.6 at the swx dependent on group phasing. This illustrates firstly the change in wave height distribution which occurs through the surf zone and secondly, that this change in distribution may also be affected by group phasing. Changes in the wave height distribution through the surf zone and group effects on these distributions are further discussed within Chapter 7

6.3.2 Wave breaking

A preliminary study, discussed within the previous section, tested a C2N5r type wave group propagating onto a 10:1 sloped bed. While examination of breaking position and type was limited to visual observation, a number of inferences were possible. In all cases, the leading wave of the group peaked offshore at a position of local crest maxima, then decreased in amplitude before plunging in shallow water well inside the surf zone. The degree to which this wave decreased prior to breaking was dependent on the position of the last local crest maxima point with respect to the bed. The breaking behaviour of the second wave of the group changed more significantly depending on the shoaling case approaching break point. For the early shoaling cases, i.e. those breaking coincident with, or soon after, a local crest maxima position, waves appeared to break in a spilling manner. For delayed shoaling groups, where breaking was coincident with, or occurred soon after, a trough minima position, the second wave appeared to break in a plunging manner. For almost all cases, this second wave in the group was the largest

breaking wave. The third wave in the groups propagated into shallower water than the second and appeared to break more consistently by plunging in all cases. Again, the size of the third wave compared to the second wave is a function of the relative phasing of the group and bed, with the two waves being more similar in height for the delayed shoaling cases and more substantially different for early shoaling cases.

The results of the preliminary study inferred that the location and type of breaking is influenced by more complex variables than simply the offshore wave steepness and bed slope. Shand et al. (2007) hypothesised that this may go some way towards explaining the considerable scatter observed in breaker-type diagrams.

The primary investigation observed and recorded wave breaking characteristics including break point location, trough to crest distance and height at break point, plunge and splash-up distance and breaker type using the calibrated optical LIF imagery and supplementary wider angle video imagery as described within Section 4.6. The three largest waves within the group were identified and classified as wave 1, 2 and 3 according to their position within the group. The break points were defined accordingly as break point (BP) 1, 2 and 3 dependent on the wave order, rather than the breaking order or position.

Similar to the previous section, the C2N5r-type wave group propagating onto a 10:1 sloped bed has been selected as an example case and results and analysis for this case will be presented within the following sections. Similar figures for the other wave group-bed slope combinations are provided within Appendix D and differences from the example case considered within the discussion.

6.3.2.1 Location

The breaking position may be assessed with respect to either distance from the bed-still water intersection (x_{swx}) or absolute distance within the wave tank (x'). A true monochromatic or solitary wave would be expected to break in the same depth or location relative to the swx regardless of bed position. Similarly, a group of waves undergoing no group evolution would be expected to break in specific depths or locations relative to the swx regardless of group-bed phasing. In contrast, a wave breaking due to convergent group processes, such as shown in Figure 6-2, would be

expected to break in a constant absolute position, dependent only on the group evolution characteristics.

Figure 6-8 depicts the breaking position of the three largest waves within the group as defined, according to their order in the wave group, by BP1, BP2 and BP3. These breaking locations, along with breaking wave height are also presented within Table 6-6. With the bed swx at $x'_{swl}/L_0 = 16.53$, wave one breaks furthest inshore, wave two furthest offshore and wave three breaks inshore, closer to BP1. Similarly, wave one is the smallest of the three waves, wave two is the largest and wave three is in between.

As the bed is incrementally moved away from the paddle, the break point of wave one remains constant with respect to the swx (Figure 6-8A) for the first three incremental bed movements, before the wave begins to break in progressively shallower water, closer to the swx in subsequent bed movements. Wave height follows a similar pattern, with height maintained during the first three movements, and decreasing concurrently with the shoreward movement of the break point in subsequent movements. This indicates that the influence of the dispersive group processes remains, with energy being progressively removed from the leading wave with distance along the wave tank.

Wave two breaks progressively further offshore of the swx (Figure 6-8A) as the bed is incrementally moved. Initially, the wave breaking location moves slightly along-flume with incremental bed shifts, though not at the same rate that the bed is moved, as depicted by the dashed line within Figure 6-8B. Between $x'_{swx}/L_0 = 17.00$ and 17.24 , breaking position remains relatively stationary in absolute along-flume position. Wave height slightly increases initially then remains near constant as the breaking position remains constant. At the bed position of $x'_{swx}/L_0 = 17.36$, offshore breaking ceases at a distance of $x/L_0 = 1.66$ from the swx. This wave then continues into shallower water before breaking at a substantially smaller height at a distance of $x/L_0 = 0.59$ from the swx. As this energy is not lost by breaking or dissipation, it may be assumed lost as part of the group dispersive process. From this point, the breaking location moves slightly closer to the swx, before stabilising in distance, similar to the initial behaviour of wave one.

The breaking position of wave three moves offshore of the swx slowly at first then more rapidly as it increases in height and becomes the largest wave after wave two ceases to break offshore. Between $x'_{swx}/L_0 = 17.59$ and 17.71 , the breaking location of wave three becomes more constant in absolute along-flume position, similar to the previous progression of wave two. This implies wave three is assuming the evolutionary trends of wave two.

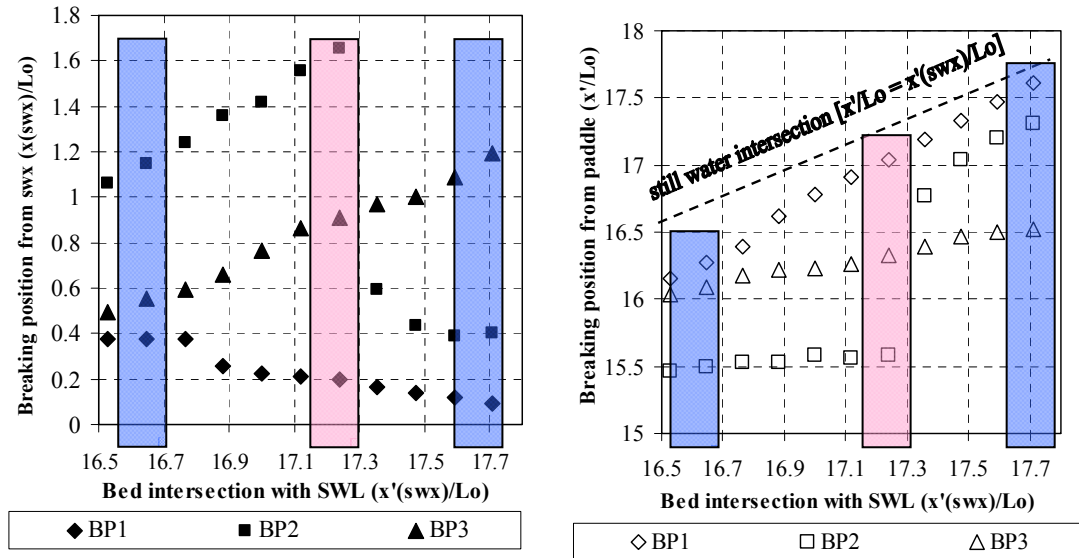


Figure 6-8 Wave breaking location for a C2N5r-type wave group breaking on an incrementally shifted 10:1 sloping bed with reference to the bed-still water intersection (A) or absolute position within the tank (B).

The behaviour implies that both shallow water, depth-limited (constant break point with respect to swx) processes and deep water, intra-group processes (constant breaking location in terms of absolute along-flume position) are occurring simultaneously. Each process affects certain waves within the group to a different extent dependent on their position within the wave group and the spatial phasing of the group with sloping bed. The wave co-incident with the group envelope maximum (i.e. typically the largest wave within the group) becomes progressively more influenced by the intra-group processes, with breaking determined by the location of a local crest maxima position, until a threshold depth is reached. Offshore of this depth, the criteria required for breaking is not reached and breaking does not occur. This wave then diminishes in height as its energy is transferred back within the group and the following wave increases in height. This, now diminished, wave breaks inshore in a depth-limited fashion. A discussion on this threshold depth is provided within Section 6.4.1.

Table 6-6 Wave breaking locations and height for a C2N5r-type wave group breaking on an incrementally shifted 10:1 sloping bed.

Position of swx (x'/L_0)	Break point 1			Break point 2			Break point 3		
	x/L_0	x'/L_0	Hb/L_0	x/L_0	x'/L_0	Hb/L_0	x/L_0	x'/L_0	Hb/L_0
16.53	0.38	16.15	0.049	1.06	15.47	0.075	0.50	16.03	0.057
16.65	0.38	16.27	0.052	1.15	15.50	0.078	0.55	16.09	0.058
16.77	0.38	16.39	0.051	1.24	15.53	0.083	0.59	16.17	0.063
16.89	0.26	16.62	0.042	1.36	15.53	0.084	0.66	16.22	0.063
17.00	0.22	16.78	0.040	1.42	15.58	0.085	0.77	16.23	0.059
17.12	0.21	16.91	0.039	1.56	15.56	0.082	0.86	16.26	0.067
17.24	0.20	17.04	0.033	1.65	15.58	0.084	0.91	16.33	0.073
17.36	0.17	17.19	0.029	0.59	16.77	0.050	0.97	16.39	0.071
17.48	0.14	17.33	0.027	0.44	17.04	0.050	1.00	16.47	0.076
17.59	0.12	17.47	0.029	0.39	17.20	0.049	1.09	16.51	0.078
17.71	0.09	17.62	0.039	0.40	17.31	0.052	1.19	16.52	0.084

Wave breaking was observed for a greater number of bed positions using the wider angle video imagery as described within Section 4.6. Although accuracy in defining breaking location was inferior to that achieved using the optical LIF video imagery, and some of the subtleties in the change of break point location could not be observed, general trends remain evident. Figure 6-9 includes the same spatial phasing positions as in Figure 6-8A (indicated by red rectangle), although additionally includes phasing positions either side. Note in certain locations multiple break points are indicated for the same wave. This occurs when offshore breaking occurs but stops prior to the shoreline and a secondary break point occurs. Overall, a cyclic pattern is noted with the break point of waves behind the group envelope maxima, i.e. waves gaining energy due to the group dispersive process, moving offshore until they coincide with the group envelope maxima. They continue to breaking in this location until depth increases to a point where the breaking criteria are not achieved. After this point, as waves move toward the front of the group and lose energy, their break point moves progressively onshore.

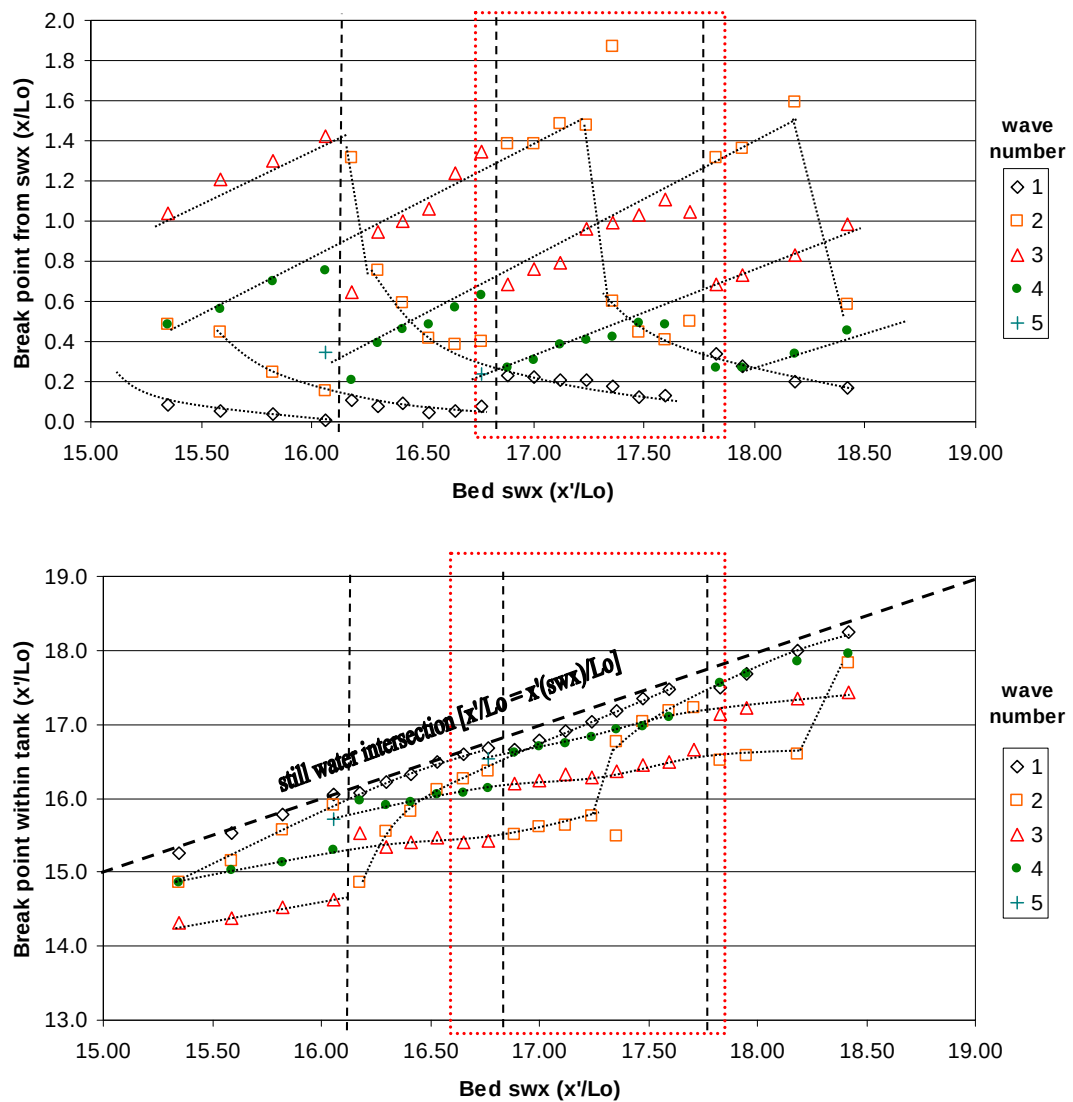


Figure 6-9 Break point locations for the 5 primary waves in a C2N5r-type wave group propagating onto a 10:1 sloping bed positioned as noted in the x-axis. Change points in wave numbering are shown by dashed lines and general trends in break point location are identified by the black dotted lines. The red rectangle shows the bed locations included in Figure 6-8.

6.3.2.2 Type

At each break point, a visually determined breaker type index, as described within Section 4.6, is assigned. This index is principally based on the manner in which breaking is initiated, the size and trajectory of the plunging jet (if any) and the subsequent splash up. Breaking type indices for the three largest waves of the example C2N5r-type wave group propagating onto a 10:1 sloped bed are presented within Table 6-7 along with the breaker location and height. Additionally, the breaker parameter B_b (Galvin, 1972) and Irribarren number (Battjes, 1974) are calculated and breaker types,

as determined the empirical transition values according to Galvin (1972), Battjes (1974) and Smith and Kraus (1990) are included.

Table 6-7 Wave breaking locations, height and visually determined breaker type for a C2N5r-type wave group breaking on an incrementally shifted 10:1 sloping bed. The Irribarren number and breaker parameter B_b , are also determined along with the breaker type predicted by these parameters.

Position of SWX (x'/L_0)	x_{SWX}/L_0	H_b/L_0	Visual breaker type index	Irrib. No ξ_b	Breaker type (Battjes, 1974)	Breaker type (Smith and Kraus, 1991)	Bb	Breaker type (Galvin, 1972)
Wave 1								
16.53	0.38	0.049	6.0	0.45	Plunging	Plunging	0.08	Spilling
16.65	0.38	0.052	6.5	0.44	Plunging	Plunging	0.08	Spilling
16.77	0.38	0.051	5.0	0.44	Plunging	Plunging	0.08	Spilling
16.89	0.26	0.042	6.5	0.49	Plunging	Plunging	0.07	Plunging
17.00	0.22	0.04	8.0	0.50	Plunging	Plunging	0.06	Plunging
17.12	0.21	0.039	8.0	0.51	Plunging	Plunging	0.06	Plunging
17.24	0.2	0.033	8.0	0.55	Plunging	Plunging	0.05	Plunging
17.36	0.17	0.029	9.0	0.59	Plunging	Plunging	0.05	Plunging
17.48	0.14	0.027	9.0	0.61	Plunging	Plunging	0.04	Plunging
17.59	0.12	0.029	9.0	0.59	Plunging	Plunging	0.05	Plunging
17.71	0.09	0.039	-	0.51	Plunging	Plunging	0.06	Plunging
Wave 2								
16.53	1.06	0.075	4.0	0.37	Spilling	Spilling	0.12	Spilling
16.65	1.15	0.078	4.0	0.36	Spilling	Spilling	0.12	Spilling
16.77	1.24	0.083	2.5	0.35	Spilling	Spilling	0.13	Spilling
16.89	1.36	0.084	2.5	0.35	Spilling	Spilling	0.13	Spilling
17.00	1.42	0.085	2.0	0.34	Spilling	Spilling	0.14	Spilling
17.12	1.56	0.082	2.0	0.35	Spilling	Spilling	0.13	Spilling
17.24	1.65	0.084	2.0	0.35	Spilling	Spilling	0.13	Spilling
17.36	0.59	0.05	5.0	0.45	Plunging	Plunging	0.08	Spilling
17.48	0.44	0.05	5.0	0.45	Plunging	Plunging	0.08	Spilling
17.59	0.39	0.049	5.0	0.45	Plunging	Plunging	0.08	Spilling
17.71	0.4	0.052	-	0.44	Plunging	Plunging	0.08	Spilling
Wave 3								
16.53	0.5	0.057	7.5	0.42	Plunging	Plunging	0.09	Spilling
16.65	0.55	0.058	7.0	0.42	Plunging	Plunging	0.09	Spilling
16.77	0.59	0.063	7.0	0.40	Spilling	Spilling	0.10	Spilling
16.89	0.66	0.063	7.0	0.40	Spilling	Spilling	0.10	Spilling
17.00	0.77	0.059	7.0	0.41	Plunging	Plunging	0.09	Spilling
17.12	0.86	0.067	7.0	0.39	Spilling	Spilling	0.11	Spilling
17.24	0.91	0.073	7.0	0.37	Spilling	Spilling	0.12	Spilling
17.36	0.97	0.071	7.0	0.38	Spilling	Spilling	0.11	Spilling
17.48	1	0.076	6.0	0.36	Spilling	Spilling	0.12	Spilling
17.59	1.09	0.078	6.0	0.36	Spilling	Spilling	0.12	Spilling
17.71	1.19	0.084	6.0	0.35	Spilling	Spilling	0.13	Spilling

As the breaking position relative to the bed-swl intersection changes, the breaking type, as defined by the visual breaking type index, is also observed to vary (Figure 6-10). This figure is observed to be the near inverse of Figure 6-8A. As the break point progresses offshore and shoaling occurs more gradually, waves tend toward breaking in a more gentle, spilling manner. This is observed as waves 2 and 3 break with consistently lower visual breaker type indices as their breaking position progresses offshore. Waves which shoal more rapidly and break in shallower water, break in a more plunging-type manner.

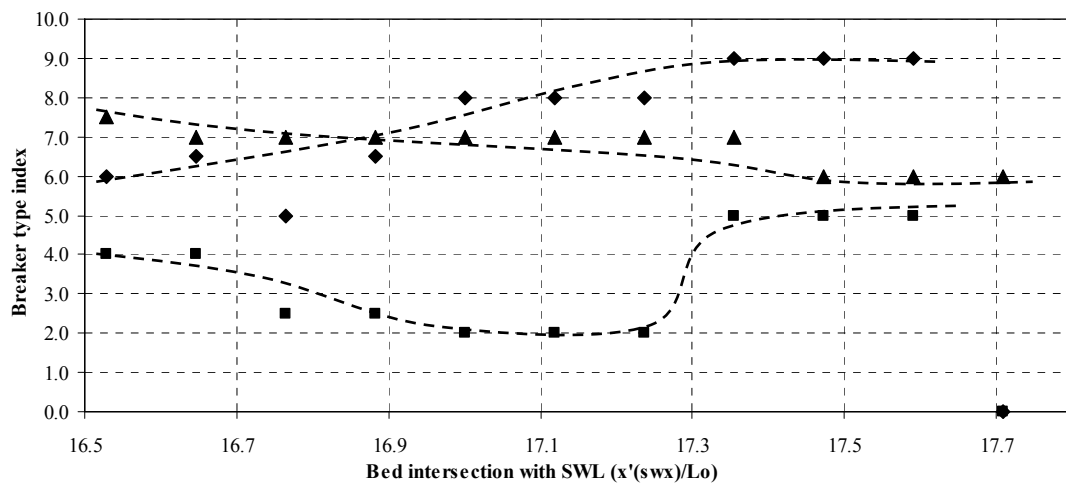


Figure 6-10 Breaking type index for a C2N5r-type wave group breaking on an incrementally shifted 10:1 sloping bed with break point 1 = ♦, break point 2 = ■ and break point 3 = ▲.

Differences in the breaking characteristics of the largest wave within a group dependent on the group-bed phasing can be observed for a C2N5r-type group propagating onto a 5:1 sloping bed within Figure 6-11. The largest wave for the $x'_{swx}/L_0 = 15.5$ series (A) has a visually determined breaker type index of 6, or a plunging breaker type. The breaking position of this wave progresses offshore as the bed is incrementally moved along the flume. With the bed position at $x'_{swx}/L_0 = 16.36$ (B), the wave breaks with a visually determined breaker type index of 2, or in a weakly spilling manner. The turbulent front on the spilling wave may be observed to progress relatively slowly down the wave face, before the wave breaks more strongly in shallow water in the last frame. This indicates that even if the initial breaking type conforms to certain criteria, the manner of breaking may subsequently change over time. Once the bed is moved slightly further than $x'_{swx}/L_0 = 16.36$, breaking by spilling, as shown within Figure 6-11(B) ceases altogether and the wave progressed into shallow water and breaks by plunging. The C2N5r-type wave group breaking on a 5:1 slope has been used in preference to the

10:1 slope as the difference between breaking types is more pronounced on the steeper slope.

These findings imply that significant variation in the breaking type is possible, even with identical offshore wave steepness and bed slope and may partially explain the considerable scatter observed in breaker-type diagrams and difficulty in obtaining an accurate predictor of breaking type.

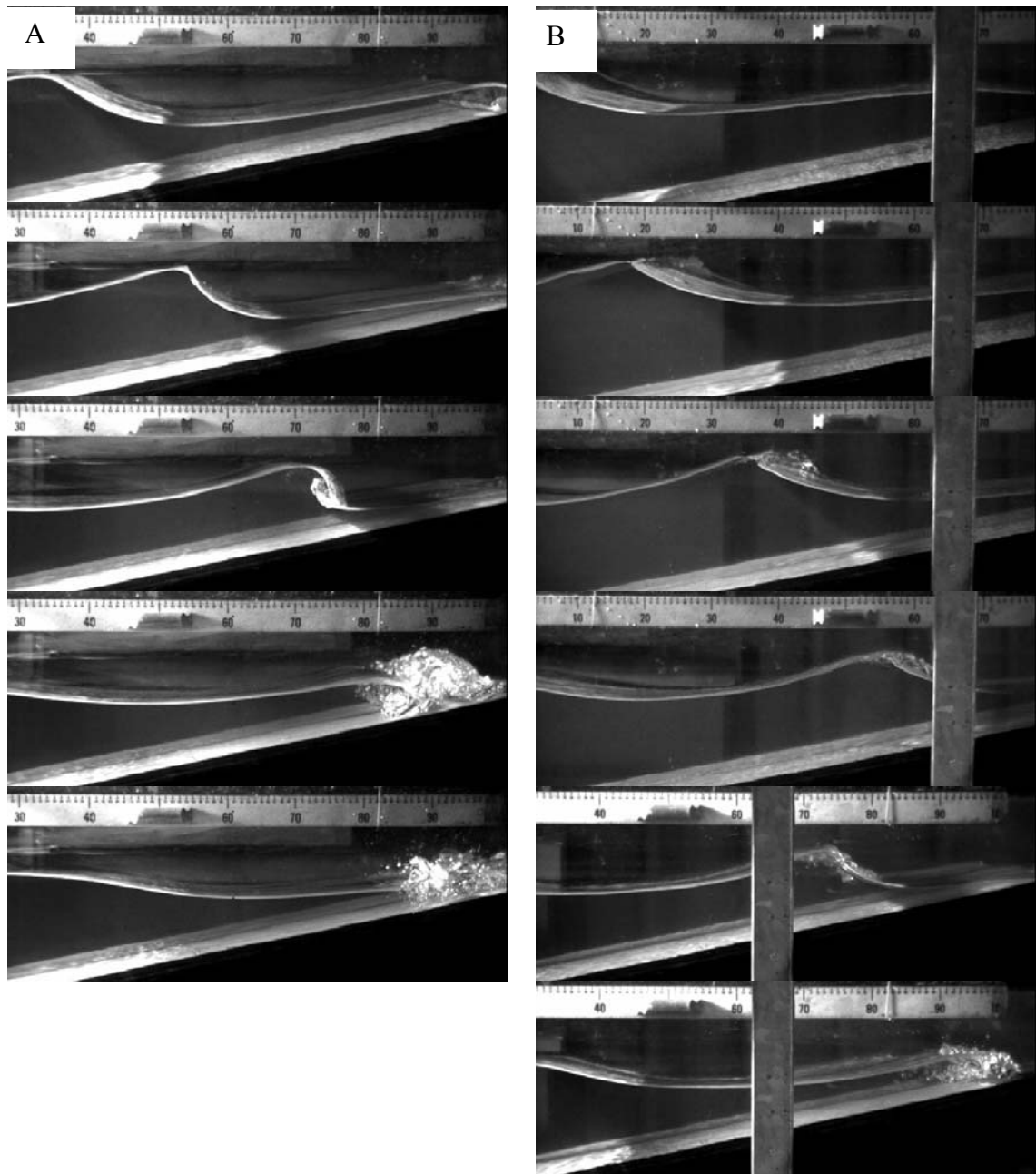


Figure 6-11 Breaking pattern for a C2N5r-type group with 5:1 sloping bed located at $x'_{swx}/L_0 = 15.5$ (A) and $x'_{swx}/L_0 = 16.36$ (B). Images are captured 0.2 s apart.

6.3.3 Mean water levels

The increase of the mean water level within the surf zone occurs to create an offshore gradient to balance the onshore momentum flux associated with the dissipation of wave energy following breaking (Longuet-Higgins and Stewart, 1962; 1964). Definition of the point following breaking at which water level begins to increase has been the subject of debate (Cokelet, 1977) and was found to vary somewhat even for monochromatic waves (Section 5.4.3). For irregular and grouped waves the problem becomes complicated by the spatial and temporal variation in the breaking position. This means that at any point between the furthestmost and innermost break points, waves may be shoaling, causing set down or propagating as a turbulent bore following breaking, dissipating energy and inducing set-up. The point at which the dissipative process dominates over the shoaling defines the transition point.

The three example cases for the C2N5r-type wave group propagating onto a 10:1 sloped bed as described earlier are examined in detail, with the change in mean water level, compared to maximum and significant wave height, presented within Figure 6-12.

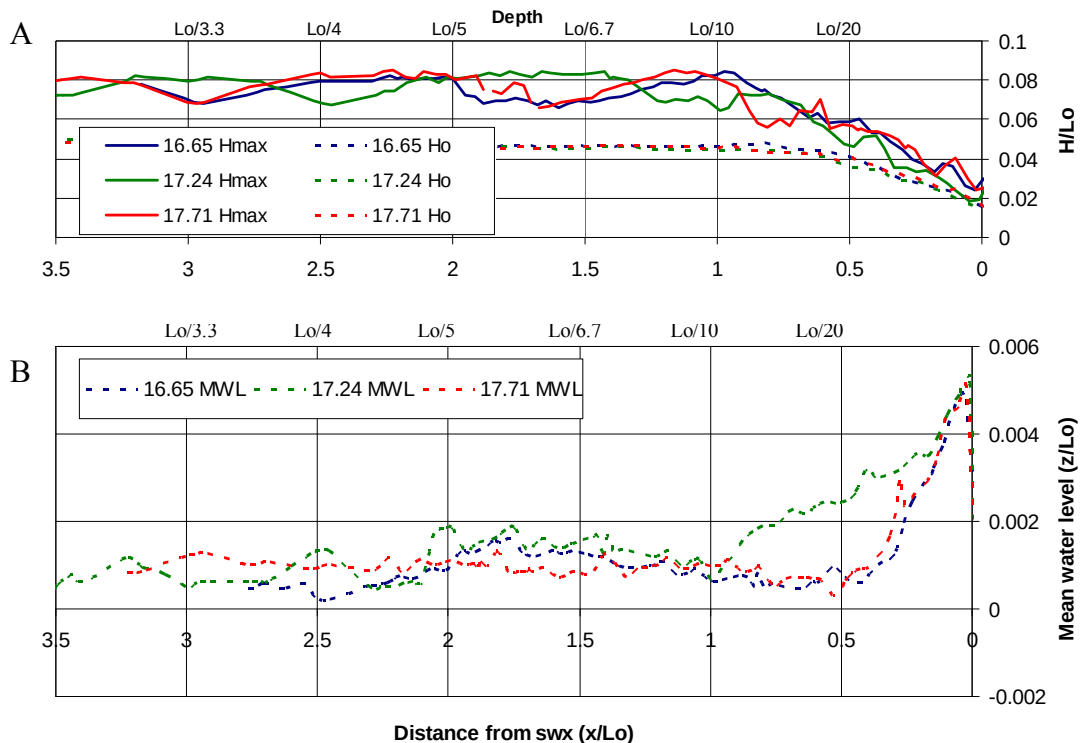


Figure 6-12 Evolution of the maximum and significant wave heights (H_{max} and H_0) (A) compared to mean water level (z_{MWL}) (B) relative to the swx position for a C2N5r-type wave group propagating onto a 10:1 sloped bed located at three differing along-flume positions denoted by the x'_{swx}/L_0 within the legend.

Offshore of the shoaling region, some scatter in the mean level is evident. This corresponds to around $z/L_0 \sim 0.001$, or around 1 mm. This is expected due to slight variation in the trolley elevation with respect to the still water level with distance along the flume. This variation has been quantified and a correction applied; however, residual error in this correction remains around 1 mm.

In all cases, the transition from a decreasing to increasing mean water level is delayed until inside the surf zone. This indicates that the offshore momentum flux associated with the shoaling of waves smaller than the maximum wave within the group balances the onshore momentum flux induced by the breaking and energy dissipation in the largest wave for some distance.

The change in the mean water level for the 17.24 series differs from the 16.65 and 17.71 series in that the transition point is further seaward at around depth $\approx L_0/10$ and the gradient is lower ($dz/dx = 0.003$) until close to the swx. From this point, the mean water level gradient increases steeply. The mean water level attained by the 17.24 series at the swx is $z/L_0 = 0.005$, corresponding to 6.0% of $H_{\max}$ or 10.9% of H_{sig} at break point. The transition point for the 16.65 and 17.71 series occurs further landward at a depth $\approx L_0/20$. The mean gradient is also steeper at $dz/dx = 0.010$. Water level at the swx is comparable to the 17.36 series at $z/L_0 = 0.005$, corresponding to 6.0% of $H_{\max}$ or 11% of H_{sig} at break point. This indicates that despite the difference in breaking type and position, set-up level at the swx remains relatively constant, indicating that a similar amount of energy is being dissipated in all cases, as is evident by the wave height at swx in Figure 6-12(A).

6.3.4 Temporally varying water levels

While calculation of the mean water level provides the average water depth at a particular position, it is not particularly relevant in the presence of wave groups. Wave groups, as are frequently present within the open coast environment, have been shown to induce temporally varying water levels outside and within the surf zone.

Figure 6-13 shows the lowpass filtered maximum and minimum surface elevation ($LP_{\max}$ and $LP_{\min}$). Offshore of significant shoaling transformations by the incident waves when intra-wave group processes remain dominant, the $LP_{\min}$ is offset from the

mean water level further than the $LP_{\max}$. The mean water level itself is slightly elevated above zero; however, this is thought to be an experimental error associated with the flume-top trolley position, as there is no physical reason for the mean level to be elevated at this location. The larger $LP_{\min}$ value compared to mean water level is in agreement with bound wave theory (Longuet-Higgins and Stewart, 1964) where long waves, induced by variations in radiation stress beneath wave groups (Section 2.2.5) generally possess narrower and deeper troughs in the vicinity of maximum incident waves and wider and lower crests in the vicinity of smaller incident waves. This may be observed in Figure 6-14 where the space-time plots of the lowpass filtered surface elevation is presented for the three example cases. In all three cases, the most notable offshore feature is the deep and narrow troughs associated with the group maximum.

Both $LP_{\max}$ and $LP_{\min}$ are observed to increase in amplitude relatively uniformly and equally inshore of around $x_{\text{swx}}/L_0 = 2.0$ or depth $= L_0/5$. At a certain location, however, the $LP_{\max}$ begins to increase more sharply, while $LP_{\min}$ decreases. This occurs at slightly different locations dependent on the group phasing, with the transition point for the 17.24 series occurring slightly inshore of depth $= L_0/10$, or roughly where the mean water level is observed to increase. The transition point for the 16.65 and 17.71 series occurs in shallower water at around a depth of $L_0/20$. The gradient of the $LP_{\max}$ increase and the $LP_{\min}$ decrease is slightly greater for these series. Again, both observations are consistent with the changes observed in the mean water level.

Similar to the mean water level, the elevations of $LP_{\max}$ and $LP_{\min}$ at the swx are similar regardless of group-phasing and shoaling and breaking type. While the mean water level at the swx has been shown to reach around 6.0% of $H_{\max}$ or 11% of H_{sig} at break point, $LP_{\max}$ at the swx is around 17% of $H_{\max}$ or 30% of H_{sig} at break point. This variation in water level at the backshore indicates the presence of a time-varying or dynamic set-up, as described by Symonds et al. (1982), Baldock (2004) and others. An empirical-based root-mean-squared value for surf beat amplitude (Eqn. 2-34) given by Goda (2000) is also compared to the observed low-frequency amplitudes. This root-mean-squared value is, as expected, substantially lower than the observed maximum values.

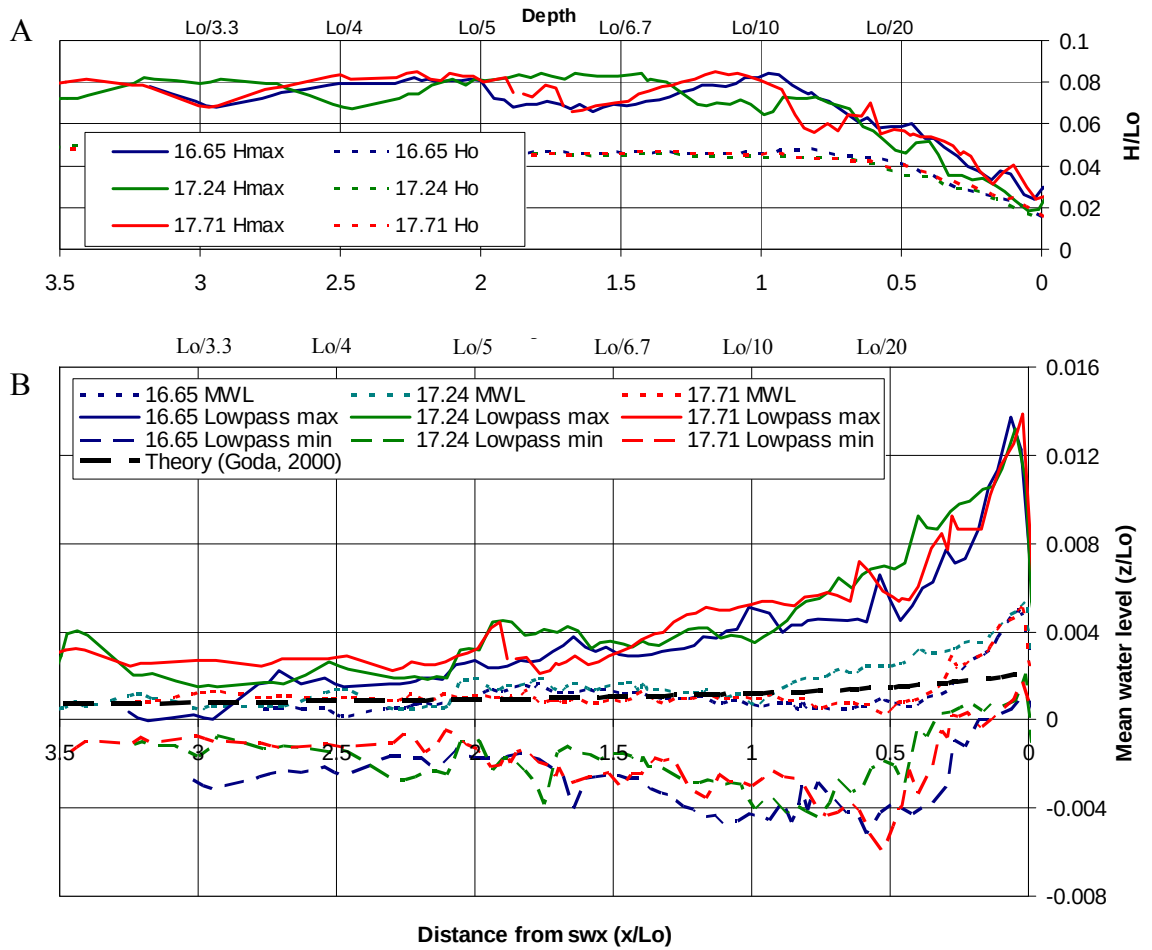


Figure 6-13 Evolution of the maximum and significant wave heights ($H_{\max}$ and H_0) (A) compared to mean water level (z_{mwl}) and the lowpass filtered maximum and minimum water levels ($LP_{\max}$, $LP_{\min}$) (B) relative to the swx position for a C2N5r-type wave group propagating onto a 10:1 sloped bed located at three differing along-flume positions denoted by the x'_{swx}/L_0 within the legend.

While observing changes in the time-averaged maximum and minimum lowpass surface elevation provides some insight into the range of low-frequency motion, they do not offer insight into the temporal changes in these low-frequency motions. Figure 6-14 shows the space-time plots of the lowpass filtered surface elevation for the three example group-phasing cases. In all cases, as discussed earlier, a deep narrow trough is evident in deeper water associated with the group maximum. This trough deepens approaching the initial break point and within the outer surf zone before becoming abruptly truncated within the inner surf zone. This truncation occurs at a distance from the swx of $x_{\text{swx}}/L_0 \approx 0.5$ for the 17.24 series. For the 16.65 and 17.71 series, the trough appears to propagate slightly further landward and reach slightly greater amplitude before being truncated at around $x_{\text{swx}}/L_0 \approx 0.3$.

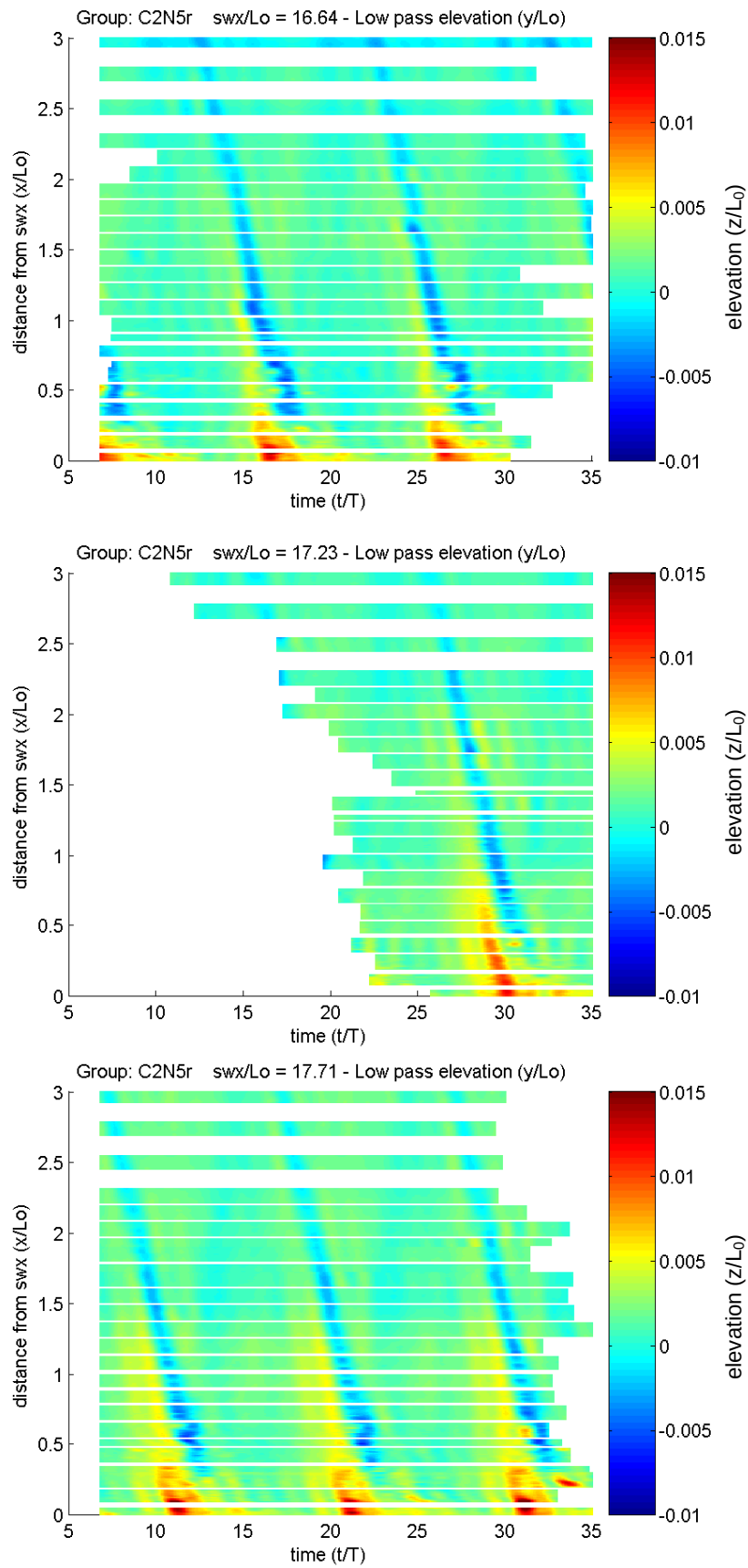


Figure 6-14 Space-time plots of the lowpass filtered surface elevation for a C2N5r-type wave group propagating onto a 10:1 sloped bed located at three differing along-flume positions.

A low frequency crest is present only as a relatively low, wide feature in deep water. However, within shallower water a sharper and higher crest, preceding the trough, develops. This crest is likely associated within the onshore directed momentum flux or increased radiation stress caused by energy dissipation following wave breaking (Longuet-Higgins and Stewart, 1962; 1964). While a crest appears to develop from around $x_{swx}/L_0 = 1.5$ for the 16.65 and 17.71 series, development is initially slow until around $x_{swx}/L_0 = 0.2$ when a rapid increase in elevation occurs. In the absence of a trough in the inner surf zone, the crest becomes temporally long and appears to persist as the majority of the group reaches the backshore, although decreasing in amplitude over time. This is consistent with the increase in radiation stress occurring as the largest waves dissipate and the concept of dynamic set-up. In contrast, the low-frequency crest of the 17.24 series is initiated slightly further landward at around $x_{swx}/L_0 = 1 - 1.2$, and increases in amplitude more rapidly. The crest also remains narrower, not persisting at the backshore in the same manner as the 16.65 and 17.71 series. This indicates that more wave energy is likely dissipated in the outer surf zone for the 17.24 series.

Although significant reflection of the long wave does not occur, as was noted by other researchers such as Baldock (2004), this is attributed to truncation of the bed at the swx with a portion of the low frequency wave propagating over the swx and not reflecting. However, as the crest precedes the trough, the water level at the swx will still be relatively higher as the crest reaches this position and therefore a partial reflection may occur. This reflection would effectively cancel the trough in the inner surf zone, as observed. However, minimal evidence of long wave reflection into the offshore region is evident.

6.4 Discussion

6.4.1 Threshold breaking depth

As discussed above, a cyclic pattern emerges as the spatial phasing of the wave group and sloping bed is altered. As the sloping bed is moved incrementally along the wave tank, a wave increases in size and its breaking position shifts offshore as it approaches coincidence with the group envelope maximum. Once coincident with the group envelope maximum, breaking occurs at or near a position of local crest maxima. As the sloping bed is moved further, breaking continues at the position of local crest maxima

and so the break point moves relatively offshore. This trend continues until a threshold depth is reached where breaking is unable to occur and the break point shifts onshore, with the following wave in the group assuming the most offshore breaking position. This indicates that while intra-group processes (horizontal energy convergences) are becoming dominant in dictating breaking location, some shoaling influence (vertical energy convergences) is required to increase total local energy convergences to a point where breaking is possible. A conceptual diagram of this hypothesis is presented within Figure 6-15.

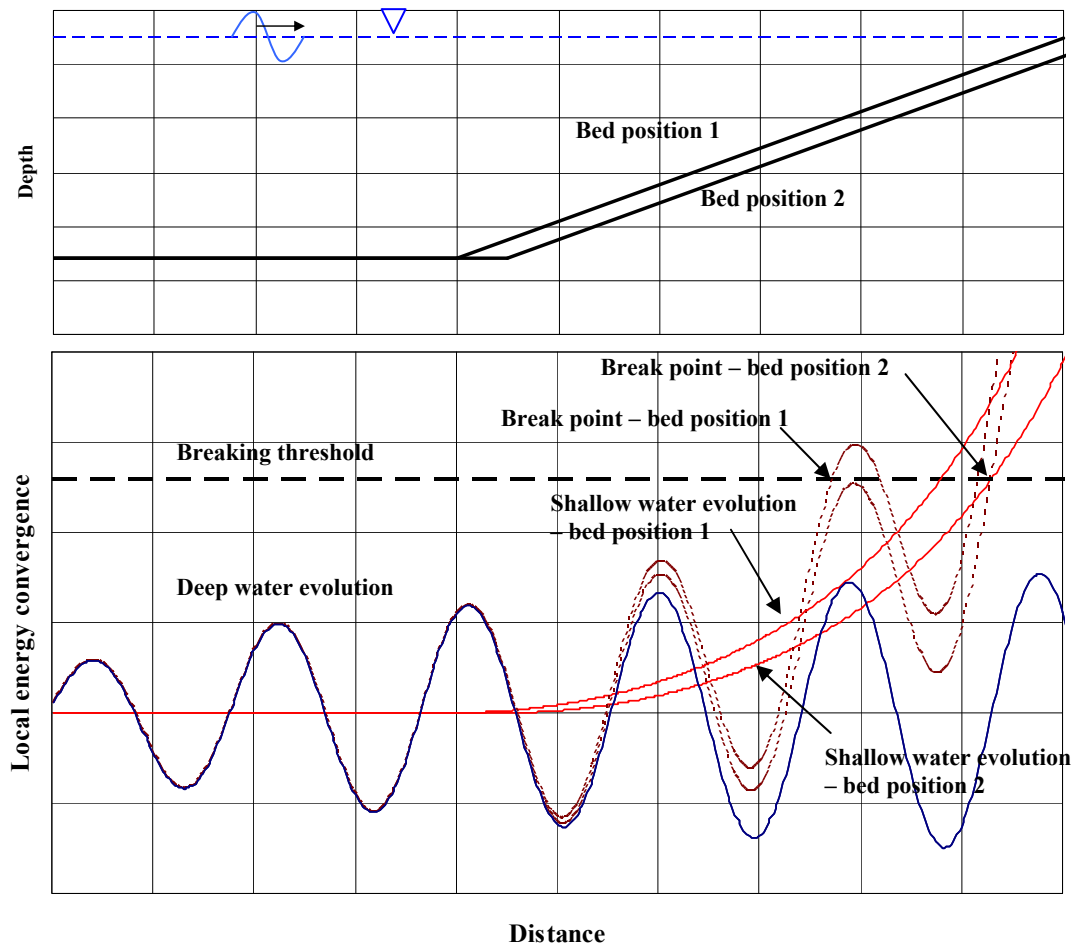


Figure 6-15 Conceptual diagram of the combination of deep water local energy convergences (—) with shallow water, depth induced convergences (—) resulting in total convergences (—). Position at which combined convergences exceed a threshold (—) define breaking. During certain spatial phasing, the combination of deep water and shallow water convergences exceeds the threshold at an offshore position of local crest maxima (early shoaling) and in others, the threshold is not reached until shallow water influences dominate (delayed shoaling). Note that this conceptual diagram does not incorporate the increase in distance between successive positions of local crest maxima which occurs in shallow water as discussed within Section 6.3.1.

Defining this threshold point at which breaking ceases could be investigated from a depth-limited perspective, typical of shallow water studies (Goda, 1970; Weggel, 1972; Smith and Kraus, 1991), or from a limiting wave steepness perspective, typical of deep water studies (Michell, 1893; Miche, 1944; Munk, 1949). As the breaking process at this point appears to include influences from both shallow and deep water processes, neither of these approaches considered singularly are likely to provide complete and definitive solutions. However, shallow-water, depth-limited methods may better predict the point at which breaking resumes when breaking ceases offshore. Furthermore, the limiting wave steepness method of predicting breaking was found unsatisfactory in predicting purely deep water breaking (Banner and Peirson, 2007). While the threshold energy convergence method (Song and Banner, 2002, 2004; Banner and Peirson, 2007) has been shown to remain applicable for prediction of breaking in water of intermediate depth, the method relies on quantification of total energy. In deep water, this may be calculated based on observed surface elevation as potential energy is approximately equivalent to kinetic energy. This assumption is however, inappropriate within shallow water.

Figure 6-16A shows the breaking wave steepness, H_b/L_b , with L_b predicted according to linear theory, for a C2N5r type group propagating onto a 10:1 slope at differing group spatial phasing positions. Breaking wave steepness is observed to initially increase as the wave increases in height approaching coincidence with the group envelope maxima. Steepness then decreases as the break point moves offshore and wavelength at break point is longer. These observed steepnesses are compared to the threshold wave steepnesses before initiation of breaking as predicted by Miche (1944), Kamphuis (1991) and Rattanapitikon and Vivattanasirisak (2003). These formula, presented within Appendix A, are selected on the basis of Miche (1944) being the theoretical limiting steepness formula upon which a large number of subsequent formula are based, Kamphuis (1991) including a parameter for bed slope and calibrated using laboratory data and Rattanapitikon and Vivattanasirisak (2003) being calibrated using 26 sources of empirical data. A vertical dashed line indicated the position at which offshore breaking ceases and the breaking position moves inshore.

While the formula of Rattanapitikon and Vivattanasirisak (2003) is close to the observed breaking steepness prior to cessation of breaking, the observed breaking

steepness is relatively uniform prior to cessation. Additionally, breaking steepness increases once offshore breaking ceases (after $x'_{swx} = 17.24$). This indicates that this parameter alone may not provide a robust predictor of whether breaking will occur. The formula of Kamphuis (1991) collapses to predict equivalent values to that of Miche (1944) on a 10:1 slope and both over-predict the observed steepness. Use of a nonlinear theory, such as a Stokes 5th order theory results in lower wave steepnesses due to the longer predicted wavelengths. However, as this change in steepness is constant in both the observed and predicted values, discrepancy between the two remains unchanged.

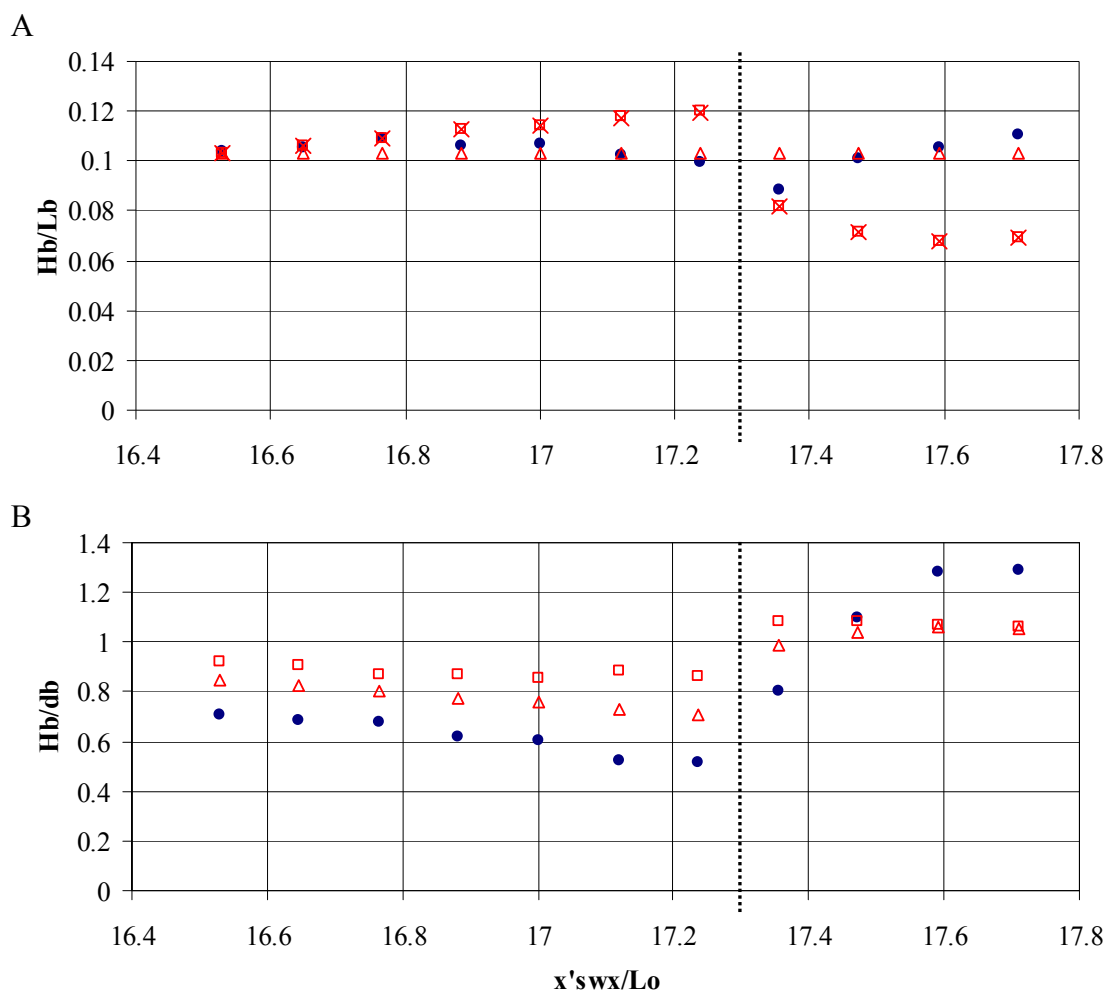


Figure 6-16 Observed breaking steepness (H_b/L_b) ($\bullet$) for varying group spatial phasing positions are depicted within (A) for a C2N5r type group propagating onto a 10:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) ($\square$), Kamphuis (1991) ($\times$) and Rattanapitikon and Vivattanasirisak (2003) ($\triangle$). Within (B), observed breaker depth index ($\bullet$) is compared to indices predicted by USACE (2003) ($\square$) and Goda (2000) ($\triangle$). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

Figure 6-16B depicts the breaker depth index, H_b/d_b together with the breaker index predicted by the USACE (2003) and Goda (2000) design guidelines. The use of these guidelines is further discussed within Chapter 7, however, this figure illustrates the reduction of the observed breaker depth index below values predicted by either guideline as breaking position shifts offshore. Chapter 5 compares guideline-predicted values to those observed for monochromatic waves, with generally good agreement. This indicates that intra-group processes are likely the cause of the discrepancies between observed and predicted indices. Additionally, the guideline values do not well-predict the inshore break point following cessation of offshore breaking. A further discussion on the difference between observed and predicted breaker indices, particularly within the surf zone is presented within Chapter 7.

While results presented and discussed in preceding sections have used an example case of a C2N5r-type wave group propagating onto a 10:1 sloped bed, similar information has been extracted and compiled for other wave group-bed slope combinations as detailed in Section 6.2.1 and is presented within Appendix D. Similar cyclic variation in shoaling and breaking patterns dependent on group spatial-phasing remained evident for each of these wave group-bed slope combinations, with breaking position of the wave coincident with the group envelope maximum moving offshore and breaking wave steepness decreasing until a threshold depth is reached and offshore breaking ceases. Similar figures to those depicted within Figure 6-16 are presented for each case within Appendix D. The threshold at which breaking ceases and breaking position shifts inshore are examined for each bed slope and wave group type combination within Table 6-8. Note that significantly more spatial positions (bed locations) were tested for some cases than others, meaning that the critical threshold point is likely to be more accurately defined for these groups.

While all group cases and bed slopes exhibit similar reduction in breaking wave steepness as breaking position shifts offshore, the wave steepness at cessation of breaking varies with differing bed slope and group type (Table 6-8). This indicates that a generic limiting wave steepness, as suggested by Stokes (1880) or Michell (1893), is not appropriate but rather that both the bed slope and intra-group processes have influence.

Table 6-8 Comparison of wave properties at the most seaward breaking location and at the subsequent shoreward break point.

Bed slope	Group type	Number of spatial testing positions	Offshore breaking limit				Inshore breaking resumes		Distance x_b shifts
			$x_{b(swx)}/L_0$	H_b/d	db/L_0	H_b/L_b	$x_{b(swx)}/L_0$	H_b/d_b	
5:1	C2N5r	9	1.14	0.35	0.228	0.085	0.18	1.54	0.96
10:1	C2N5r	11	1.66	0.52	0.167	0.098	0.59	0.81	1.07
	C2N3r	3	1.67	0.56	0.167	0.11	0.59	1.16	1.08
	C2N5sb	6	1.94	0.45	0.194	0.098	0.27	1.58	1.67
	C3N5r	6	2.77	0.37	0.227	0.105	0.52	1.14	2.25
20:1	C2N5r	5	3.09	0.51	0.155	0.095	0.98	0.51	2.11

Trends evident within Table 6-8 include, as expected, a more offshore cessation of breaking point (x/L_0) for lower bed slopes, although relative depth follows an inverse trend, with breaking ceasing in relatively deeper water on steeper slopes. This implies that shallow water energy convergences may influence offshore breaking to a greater extent on steeper bed slopes. This could be explained by the trough of wave-forms approaching steeper slopes being subject to shallower relative depths than on a flatter slope. Other possible explanations include that waves which marginally begin to break on the 5:1 slope, continue breaking into shore, whereas a wave marginally breaking offshore on the 10:1 and 20:1 slopes may cease before re-breaking inshore. These events where breaking initiates offshore but does not continue to the shoreline have not been defined as the wave break point within the present study. A difference in offshore and nearshore breaking types of a continually breaking wave were also observed in some cases, with offshore breaking occurring in a relatively gentle, spilling manner, transitioning to a harder plunge in shallow water. These results imply that both deep water and shallow water breaking may overlap and the theory that the original breaking type of a wave dictates the subsequent evolution of that wave-form on a plane slope may be incorrect.

The position at which inshore breaking resumes appears to be due primarily to shallow water convergences, with location depending on bed slope and wave steepness. Breaker index at the point of inshore breaking for the C2N5r types groups are, as expected, higher for the steeper slopes. Distance between the cessation of offshore breaking and inshore breaking varied, although, apart from the C2N5sb case, appeared to be grouped around movements of either $1L_0$ or $2L_0$, with the steeper C3N5r case and the C2N5r

group breaking on a 20:1 slope having shifts of around $2L_0$. This may imply that some intra-group influence remains even through to inshore breaking.

6.4.2 Predictive model for positions of local maxima/minima

The time or distance interval between sequential crest maxima is dependent on the group dispersive properties, or more specifically the ratio of group velocity to wave celerity. This interval may be defined according to time, $T_{interval}$ (Eqn. 6-1) or distance, $D_{interval}$ (Eqn. 6-2).

$$T_{interval} = \frac{L}{(c - c_g)} \quad (6-1)$$

$$D_{interval} = c_g \left(\frac{L}{(c - c_g)} \right) \quad (6-2)$$

Where L , c and c_g are the wavelength, wave celerity and group velocity respectively, calculated at the position of group envelope maxima. Within deep water, where $c_g \approx 0.5c$, these intervals between successive crest maxima reduce to $D_{interval} \approx L$ and to $T_{interval} \approx L/0.5c$. Within shallow water, wave celerity reduces and approaches the group velocity. This increases the time and distance between successive crest maxima as individual wave forms propagate relatively more slowly through the group. In very shallow water as wave celerity and group velocity approach unity, i.e. $c_g \approx c$, the interval tends to infinity.

The above described model is calculated according to linear theory using the method of Hunt (1979) to approximate wave number and celerity. This method of approximation has been shown to have small errors ($<0.01\%$) for all water depths (Hunt, 1979). The model is used to predict positions of local maxima and minima for the various wave cases within deep water and results are compared to observed values of maximum and minimum surface elevations (Figure 6-17). The positions of a crest coincident with the group envelope maximum, defined previously as a position of *local crest maxima*, are designated by a dispersive phase of 180° and positions of a trough coincident with the group envelope maximum, defined previously as a position of *local trough minima*, are designated by -180° .

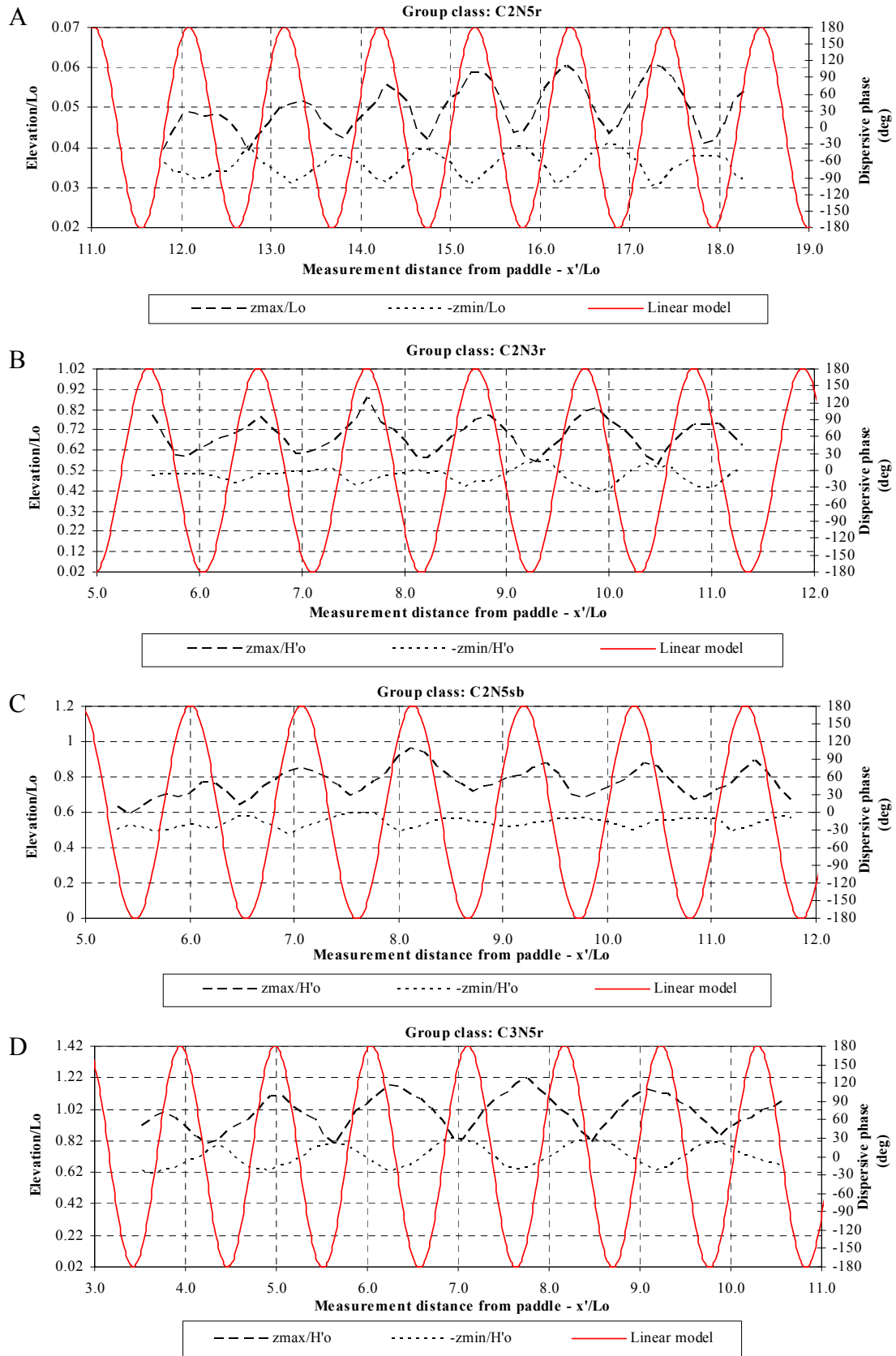


Figure 6-17 Comparison of the deep water envelope cycle for four wave group configurations compared to the derived linear model. Good agreement is found between the model and bichromatic waves groups (A-C), with less good agreement found for the Case 3 wave group (D).

Reasonable agreement is found between the model and observed results, although a small amount of along-flume variation in the measured interval between positions of crest maxima is evident. This could be related to the slight differences in wave period observed within the group at differing stages of group evolution (Figure 6-3). The C3N5r *chirped*-type wave group fits the model less well. However, the structure of this group, with higher frequency waves generated at the front and lower frequency at the rear, means that the mean period should change with along-flume distance. This may be observed, with Figure 6-18(A) depicting the along-flume change in mean period and Figure 6-18(B) the revised model predictions compared to observation incorporating the spatially varying period values. An improved model fit is noted.

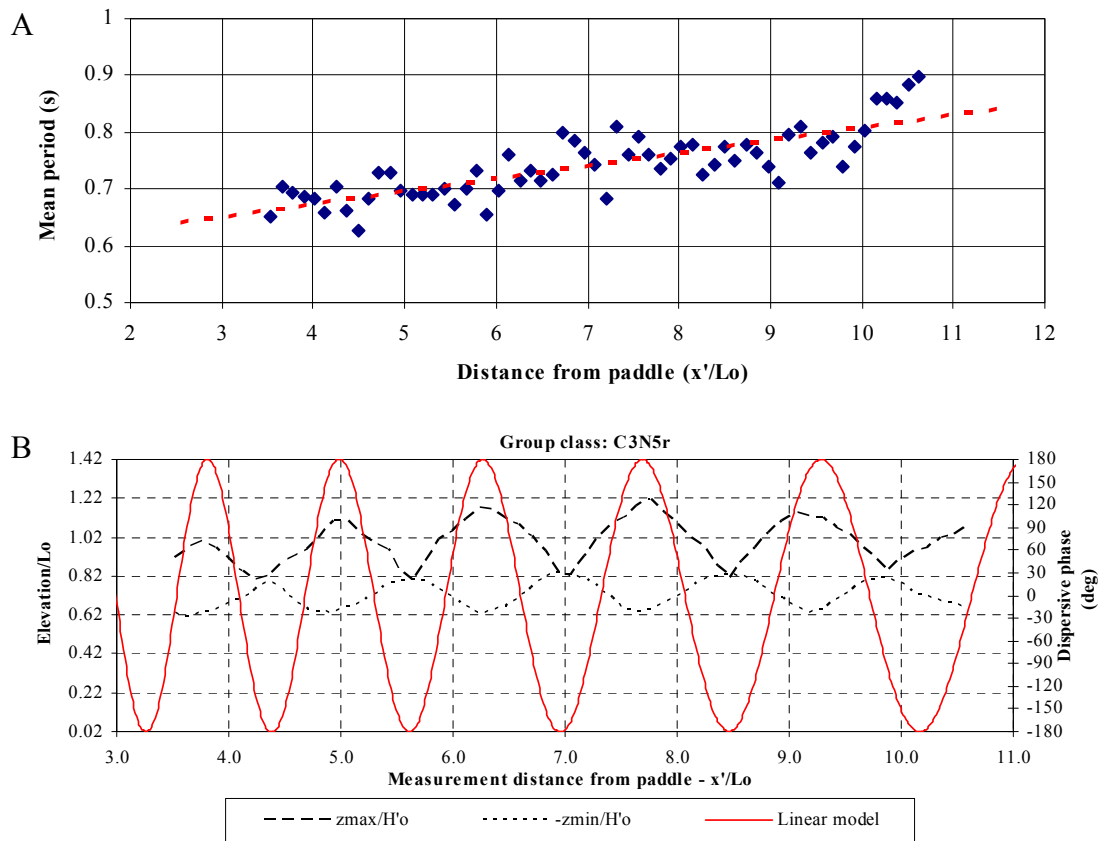


Figure 6-18 Mean period with distance along wave flume for Case 3, chirped-type wave group (A) and revised model prediction (B).

The predicted locations of local maxima and minima are then compared to observations of maximum and minimum surface elevation on a sloping bed (Figure 6-19). The models clearly show an increased distance between sequential crest maxima within progressively shallower water. Three example cases are given corresponding to a C2N5r-type wave group on 10:1 sloped bed with the swx located at $x'_{swx}/L_0 = 16.65$

(A), $x'_{swx}/L_0 = 17.36$ (B) and $x'_{swx}/L_0 = 17.71$ (C). A and C correspond to *delayed shoaling*-type cases and B to an *early shoaling*-type case. For the delayed shoaling-type cases, the maximum surface elevation can be observed to increase to a local crest maxima position, in good agreement with model predictions, then begin to decrease, again in agreement with model predictions. However, as the depth continues to decrease, a point is reached where the group process diverges from the model and the maximum surface elevation increases rapidly to break point. In contrast, the early shoaling group follows the model prediction, shoaling slowly as the group approaches a position of local crest maxima and breaking near this position.

The early shoaling groups, with waves approaching break point as the group approaches a position of local crest maxima, break in deeper water in more of a gentle, spilling manner. The delayed shoaling groups, not breaking at a position of crest maxima but rather being forced to shoal rapidly inshore due to depth limitations, break in shallower water in more of a plunging manner. These series may be combined (Figure 6-20) and presented in terms of depth (z/L_0). The group near a position of local trough minima between $L_0/4.5$ and $L_0/5$ ($z/L_0 = 0.22$ to 0.20) evolves to early shoaling, while groups near a position of local crest maxima at this depth evolve to delayed shoaling.

While the premise that dispersion is negligible in shallow water and therefore group effects do not influence wave breaking is common, the break point of the largest wave within the group, appears to be affected by the group evolution and the position of local crest maxima. This finding implies that accurate predictions of breaking position may require knowledge of the offshore evolutionary characteristics of a wave group. This is in contrast to the existing methods of using critical depth indices or steepness thresholds to predict breaking, both of which have been shown to result in scattered data and imperfect predictions.

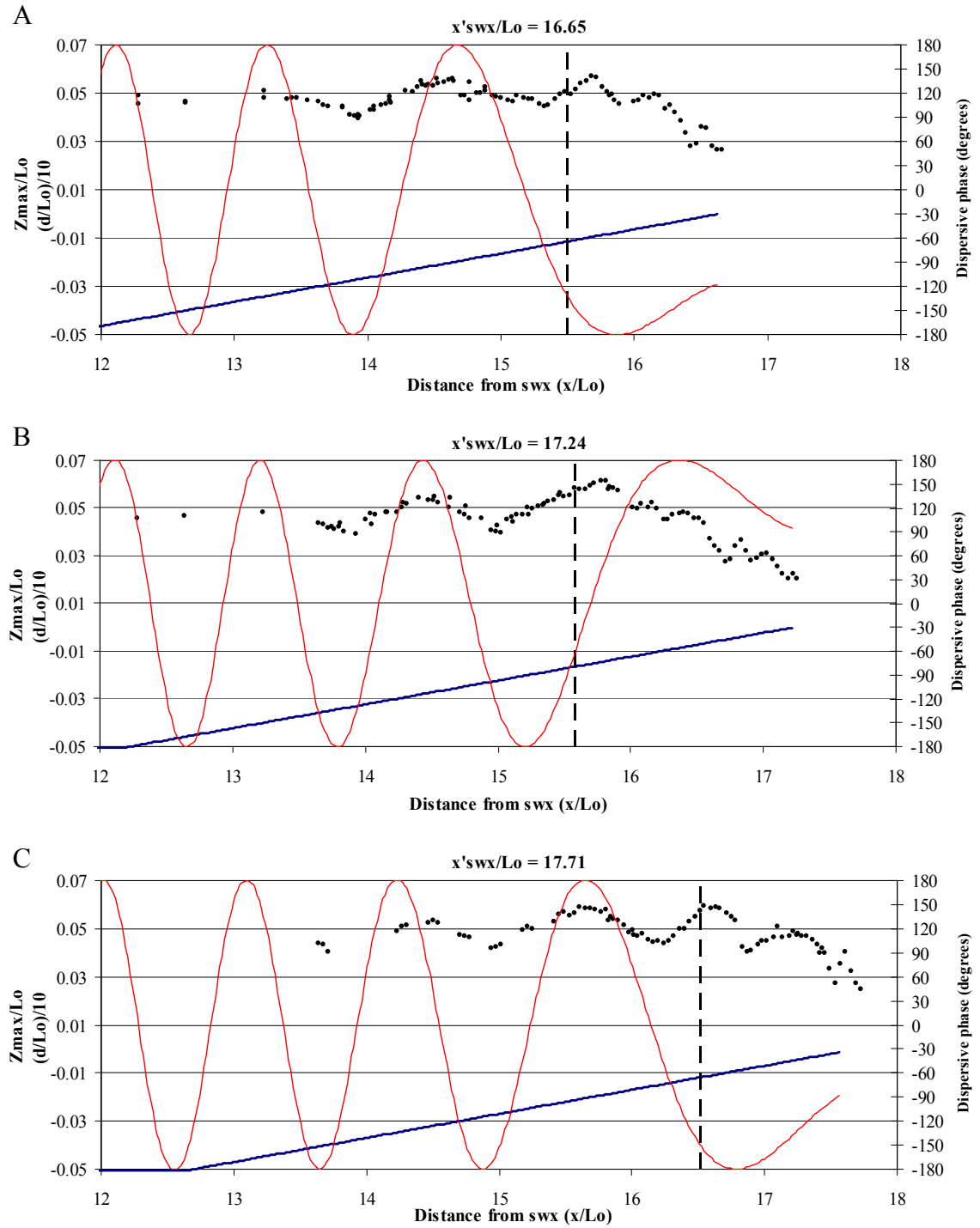


Figure 6-19 Comparison of $z_{\max}$ (•) for a C2N5r-type wave group on a 10:1 sloped bed compared to group phase predicted by linear model (+180° = local maxima; -180° = local minima) (—). Dashed lines indicate break point. Three sloping bed positions are shown with the swx located at $x'_{\text{swx}}/L_0 = 16.65$ (A), $x'_{\text{swx}}/L_0 = 17.36$ (B) and $x'_{\text{swx}}/L_0 = 17.71$ (C). A and C correspond to *delayed shoaling*-type cases and B to an *early shoaling*-type case. Depth (d/L₀) is given /10 to better fit the plot axis.

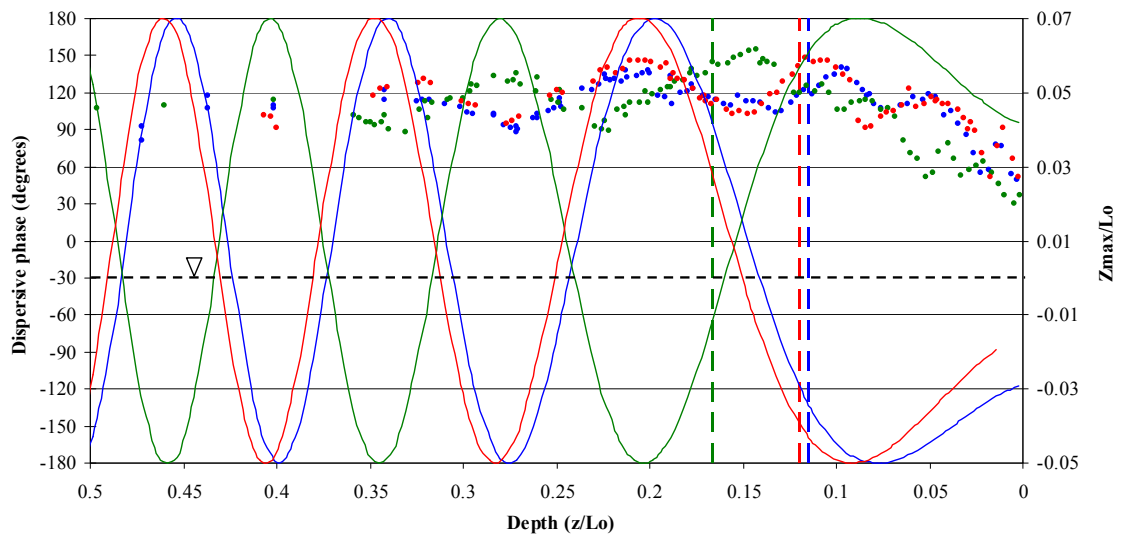


Figure 6-20 Data as shown within Figure 6-19 is presented as a function of depth (z/L_0). Observed z_{max} is depicted by dots for the delayed cases: $x'_{swx}/L_0 = 16.65$ (●) and $x'_{swx}/L_0 = 17.71$ (●) and the early shoaling case: $x'_{swx}/L_0 = 17.36$ (●). Equivalent coloured solid lines depict group phase predicted by model ($+180^\circ$ = local maxima; -180° = local minima) and dashed lines indicate break point.

This linear model well predicts group evolution during shoaling for C2N5r group on 5:1 slope and the C2N3r and C2N5sb groups on 10:1 slope although performs less well for C2N5r group on the 20:1 slope and C3N5r group on a 10:1 slope. Similar patterns of critical depth dictating early and delayed shoaling held for all cases and are presented within Table 6-9. The group near a position of local trough minima between $L_0/4.5$ and $L_0/5$ ($z/L_0 = 0.22$ to 0.20) evolves to early shoaling, while groups near a position of local crest maxima at this depth evolve to delayed shoaling.

Table 6-9 Critical depth at which state of group evolution (near a position of local trough minima or local crest maxima) dictates future shoaling type (early or delayed shoaling respectively) for each group type and bed slope tested.

Group type	Slope	Initial average steepness $(ak)_0$	Critical depth indicating future evolution
C2N5r	0.2	0.143	$L_0/3.5$
C2N5r	0.1	0.143	$L_0/4.7$
C2N5r	0.05	0.143	$L_0/5.25$
C2N3r	0.1	0.168	$L_0/4.3$
C2N5sb	0.1	0.160	$L_0/3.95$
C3N5r	0.1	0.125^1	$L_0/3.15$

¹ steepness increases during propagation due to focusing group structure

Critical depth as a function of bed slope is examined for a C2N5r type group within Figure 6-21. The critical depth is observed to increase (i.e. occurs in deeper water) as bed slope increases. A linear best-fit is found and indicated on the plot with 90% confidence intervals similarly indicated. An F-ratio is used to test the hypothesis that the slope of the regression line = 0 (Wilkinson, 1998). This enables establishment of whether the line is positively or negatively significant. An F ratio > 100000 is found, corresponding to very low (≤ 0.002) probability that the slope of the regression line = 0. This indicates that despite the very small number of samples (i.e. 3), a statistically significant relationship relating bed slope to critical depth exists.

Relationships defining critical depth for differing group types is less well defined than for the bed slope. Further testing is required to quantitatively determine the influence of group steepness and structure on critical depth but overall, critical depth defining early or delayed shoaling appears to occur in deeper water for groups of steeper initial structure and for groups comprising less waves (i.e. $N = 3$). ***Overall, the critical depth at which the state of group evolution determines future shoaling and breaking patterns occurred at $L_0/4.2 \pm 1$ for the group types and bed slopes tested within the current study.***

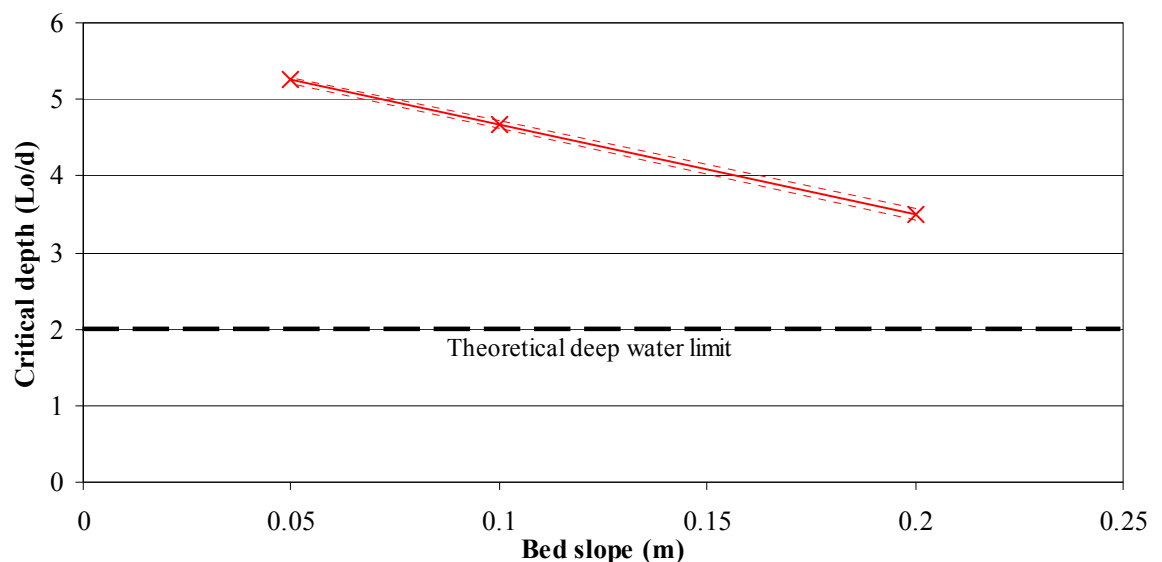


Figure 6-21 Critical depth at which group evolutionary state determines future shoaling patterns for a C2N5r type group (x) as a function of bed slope (m). The linear best-fit is indicated by a solid line and 90% confidence intervals by dashed lines.

6.4.3 Errors and limitations

6.4.3.1 Accuracy in defining wave height and breaking position

While the uncertainty in defining the crest to trough height for an individual wave is discussed within Section 4.2.2.3, the probability of identifying the absolute peak of a wave crest is improved by observing a higher number of *identical* waves. These may be sequential waves in a monochromatic series or the equivalent wave in a series of wave groups. This assumption also relies on independence between the wave and data capture frequency. While the wave phase difference between successive measurements for the capacitance probes is only ($360^\circ/120\text{Hz} \times 0.7365\text{s} = 4.1^\circ$), the difference is 24.4° for data obtained by the optical LIF method. While a phase difference of 24.4° results in a relatively small error in defining the crest for an idealised sine wave ($< 2.2\%$ of wave height), the error becomes greater near a steep crest prior to breaking. An analysis of waves at varying steepness indicates that at least 4 waves in deep water and 12 waves near break point are required to guarantee observation of a wave crest or trough within one pixel of absolute (Figure 6-22).

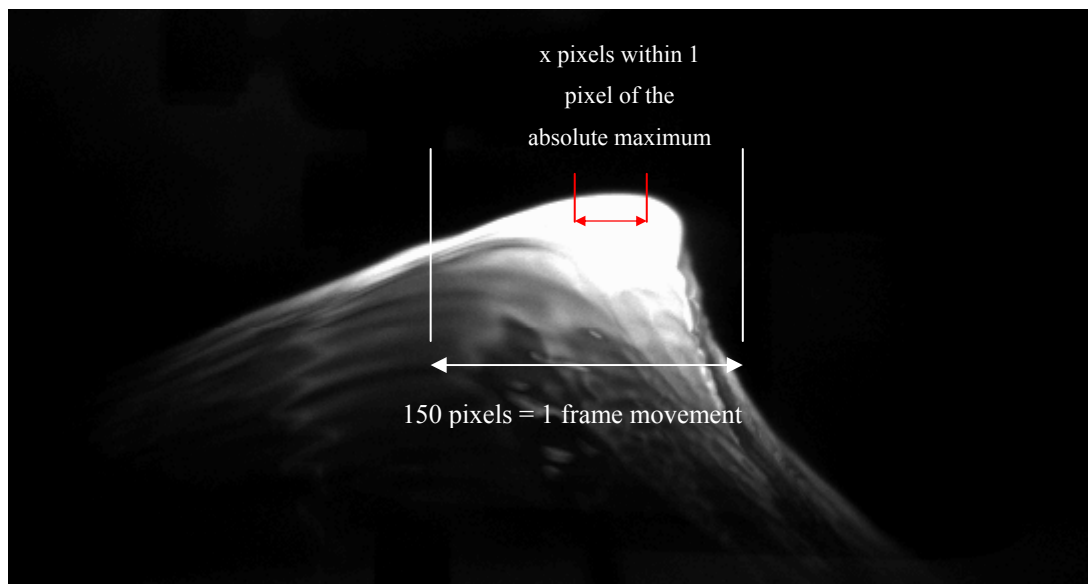


Figure 6-22 Example of length of surface within one pixel of absolute crest maximum.

Based on an average of 4 wave groups observed at each spatial location, the maximum error in defining actual wave height using time series obtained by the optical LIF method increases from around 1 pixel (0.25 mm) in deep water to 1.5 pixels (0.4 mm) near break point. When combined with the uncertainties in defining the air-water interface of ± 0.12 mm, the total error ranges between $H_{\text{actual}} = H_{\text{observed}} + 0.28$ to -0.12

mm in deep water and $H_{\text{actual}} = H_{\text{observed}} + 0.42$ to -0.12 mm near break point. This is an improvement over $H_{\text{actual}} = H_{\text{observed}} + 0.80$ to -0.12 mm for each individual wave as calculated within Section 4.2.2.

The break point of individual waves, as defined by the wave face first becoming vertical, is defined manually using the optical LIF imagery. Accuracy is therefore similarly related to the temporal resolution of the imagery and the horizontal distance travelled by a wave form between successive frames. Near break point, the crest may travel 35 to 40 mm between successive frames (Figure 6-22). However, manual interpolation between successive frames and observation of a higher number of *identical* waves allows better approximation of the exact break point. Horizontal accuracy in defining the break point is estimated at ± 5 mm based on the typical range in manually defined breaking positions. The subsequent accuracy in defining breaking depth therefore becomes a function of bed slope with accuracy for the 5:1, 10:1 and 20:1 slopes of ± 1 mm, ± 0.5 mm and ± 0.25 mm respectively.

6.4.3.2 Wave group limitations

Wave groups examined within the present study were selected based on their evolutionary characteristics. As the primary research motivation required testing of groups exhibiting high nonlinearity, groups were selected such that recurrence or breaking occurred within a specific spatial range constrained by the size of the testing facility. The frequency and steepness of the fundamental carrier wave within tested groups was determined empirically during previous investigations (Banner and Peirson, 2007; Dack and Peirson, 2005) to satisfy these requirements. Groups of lower initial steepness, or containing a greater number of waves were unable to be tested due to their long evolution time and the limited size of testing facility.

6.4.3.3 Bathymetric limitations

The use of simple, plane sloping bathymetry, while useful for the investigation of complex interactions of multiple processes does not account for parabolic, equilibrium or barred profiles as are often observed in reality. However, due to the largely unknown relationships between deep water intra-group processes and shoaling processes, such simplification of bathymetry was appropriate. Once these processes are sufficiently

understood, future experimental or numerical investigations may extend the studies into more realistic natural profiles.

Additionally, the slopes of 5:10, 10:1 and 20:1 are very steep for natural beach slopes, although applicable for cobble or reef profiles. This limitation was imposed by the size of testing facility and the requirement for rapid (i.e. without draining and refilling of the wave tank) relocation of the sloping bed within the tank. Additionally, the interaction of these deep and shallow water processes was found to be most pronounced on steeper slopes due to the rapid transition in depth. The use of steeper sloping beds was therefore most useful in the examination and comprehension of these interactions.

6.4.3.4 Scale effects

Scale effects are introduced when the model fails to accurately represent prototype scale physical processes. The effects of surface tension (i.e. Miller, 1972; Banner and Phillips, 1974; Duncan 1999, 2001; Perlin and Schultz, 2000) have been shown to influence small scale wave breaking, particularly the entrainment of air during breaking. As surface tension becomes dominant, breaking characteristics are altered with the formation of a plunging jet replaced by a bulge-capillary structure at the crest (Duncan, 2001) and the entrainment of air diminished (Peregrine, 1983). Surface tension was found to affect the validity of the dispersion equation at depths less than 20 mm and for wave periods less than 0.35 s (Le Mehaute, 1976).

Peak period of wave tested within the present study were > 0.7 s. This satisfied the requirements for applicability of the dispersion equation (Le Mehaute, 1976). A bulge-capillary structure at the crest was noted during breaking of the smallest observed waves within the group, however this did not appear to influence either the breaking position or breaking height of these waves, but rather the breaking manner and was thus not deemed contaminative to results of the present study.

6.4.3.5 Frictional Losses

Quantification of frictional losses in propagating wave-forms due to viscous forces induced by the wave tank side-walls is discussed within Section 3.4.5. Viscous losses were slightly higher than those found by Banner and Peirson (2007) with energy losses found at between 1.3 and 1.95% per wavelength and wave height losses between 0.8

and 1.5% per wavelength (c.f. viscous energy losses of about 1.08% per wavelength or around 0.5% of wave height per wavelength found by Banner and Peirson, 2007).

Banner and Peirson (2007) compared wave breaking processes within a wave basin, subject to minimal viscous losses compared to those observed within the wave tank. Results were very similar and they concluded that frictional losses were not critical for the breaking initiation phase of their study. Similarly, frictional losses are not anticipated to significantly influence the results of the present study as the primary focus is on variation in shoaling and breaking patterns due to changes in the group spatial phasing. To assess these differences, the sloping bed is incrementally adjusted, with a maximum difference in position of around 1 wavelength. Based on the above, this corresponds to a wave height difference of 0.8 to 1.5% or 0.3 to 0.6 mm relative to H_0 . This difference is within the error assumed in defining wave height and thus may be considered negligible with respect to the study findings.

6.5 Summary and conclusions

This chapter has principally investigated:

1. the influence of shoaling water on the intra-wave group evolutionary processes which are characteristic to deep-water group propagation, and also
2. the influence of these intra-group effects on the shoaling, breaking and surf zone processes of the wave groups.

To achieve this, the deep water, shoaling, breaking and surf zone characteristics of four wave group structures propagating onto 5:1, 10:1 and 20:1 sloping beds were observed. Changes in the surface envelope, maximum and significant wave heights, mean and temporally varying water levels have been examined along with the positions and type of wave breaking.

During propagation within deep water, the group envelope was observed to fluctuate between alternative positions of local maxima and minima coincident with changes in local energy density. The amplitude of these fluctuations was observed to increase with distance along the wave tank until either a point of recurrence or breaking occurred, dependent on the initial group steepness. This evolution was consistent across all group structures tested and with previous studies (Song and Banner, 2002; Banner and Peirson, 2006). Preliminary studies (Shand et al., 2007) have shown that the spatial

phasing of the wave group evolutionary cycle at a critical shoaling depth influenced the subsequent shoaling and breaking processes. Testing was focussed around locations of greatest wave-train nonlinearity, i.e. near the point of recurrence or breaking, to maximise differences in shoaling trends. The position of the sloping bed was incrementally adjusted to alter the spatial phasing of group evolution and shoaling and differences in the shoaling and breaking patterns observed.

Changes to the group evolutionary trends were noted during propagation onto a sloping bed. The spacing between successive positions of local maxima or minima increased, i.e. local group maxima and minima positions occurred further along the wave tank than observed in deep water. This was shown, using simple linear relationships, to be associated with changes in the wave celerity compared to group velocity as depths decreased. An expression for the distance between successive local maxima and minima with decreasing depth was derived and is shown below where $D_{interval}$ is the distance between sequential crest maxima and L , c and c_g are the wavelength, wave celerity and group velocity respectively. This expression showed good agreement with observation.

$$D_{interval} = c_g \left(\frac{L}{(c - c_g)} \right)$$

Differences in the shoaling characteristics leading to breaking, the breaking locations and types, and subsequent surf zone processes were noted when altering the spatial phasing of group evolution. While these differences were categorized into two regimes, *early* and *delayed shoaling* within a preliminary study, the subsequent, more extensive investigation has shown that these changes are more cyclic, with the early and delayed shoaling regimes describing the extrema of the cycle.

As spatial phasing of group evolution is incrementally adjusted, a wave approaching coincidence with the group envelope maximum, increases in height and its breaking position moves progressively offshore. Once coincident with the group envelope maximum, breaking occurs at a position of local crest maxima. As the spatial phasing of group evolution is further adjusted, i.e. the bed is moved progressively along the wave tank, the position of local maxima and breaking position remains constant and therefore moves relatively offshore compared to the shoreline position. However, at a critical

depth, breaking ceases and that wave continues inshore, decreasing in height as its energy is transferred back within the group and eventually breaks inshore in a depth-limited fashion.

This depth at which breaking ceases appears to represent a position where the total energy convergences required to initiate breaking are reduced to a point where breaking cannot occur. As the horizontal convergences associated with intra-group processes are insufficient to promote breaking by themselves in all but the steepest wave group tested, vertical energy convergences associated with shoaling processes must provide the additional energy convergence. Once these shoaling-induced convergences reduce to a point, breaking ceases. Steeper initial wave groups appear to allow propagation of the break point to continue further offshore due to the increased horizontal energy convergences associated with these steeper cases. The following wave within the group then progressively increases in height and its break point moves offshore as it receives energy by the dispersive process and approaches coincidence with the group envelope. In this manner, the cyclic pattern of group dispersion remains evident and influential in shallow water breaking.

Along with these cyclic differences in breaking position, differences in the shoaling patterns leading to breaking also occur. Waves which do not break at an offshore position of local maxima, shoal rapidly in shallow water prior to breaking. This type of shoaling was termed delayed shoaling. On the other hand, where the local crest maxima position occurs sufficiently close enough to the shore and energy convergences appear to reach a threshold value, shoaling continues until breaking. The shoaling of these waves is more gradual and was termed early shoaling. These differences in shoaling patterns dependent on group phasing were noted for all group types tested and on all bed slopes, although were more pronounced on steeper bed slopes due to the more rapid shallow water wave transformation.

Determining the manner in which the wave coincident with the group envelope maximum shoals and breaks can therefore be approached from two viewpoints:

1. a threshold convergence rate of local energy density dictates whether breaking may occur at a position of local crest maxima, or whether the wave will continue inshore before breaking.

2. the state of group evolution at a certain depth determines the future evolutionary pattern of that group;

In deep water, if the total kinetic energy density can be related to potential energy density, a threshold value of energy convergence rates leading to breaking can be obtained (Banner and Peirson, 2007). In shallow water, both the intra-group and depth induced energy convergences are important in determining the initiation of wave breaking. While proxy measures such as wave steepness thresholds and wave height on depth ratios are often employed to determine when and whether breaking may occur, neither has been found a robust predictor of breaking threshold within this or previous studies (Banner and Peirson, 2007). Significant work therefore remains to quantify accurately the energy convergence rates required in shallow water to induce breaking, including possible quantification of both the potential and kinetic energies associated with shallow water shoaling and evolving wave groups.

By determining the evolutionary state of a group at specific offshore depths, the future shoaling and breaking characteristics of that group were able to be predicted. Groups near of position of local trough minima at this critical depth were found to shoal gradually, with breaking occurring offshore near a position of local crest maxima, while groups near a position of local crest maxima at this depth, were found to shoal more rapidly within shallower water and breaking occurred in shallower water near a position of local trough minima. This essentially provides an empirical method of determining future evolutionary trends, with less emphasis on determining the above-mentioned specific energy convergence thresholds.

This critical depth was found to vary with bed slope and group steepness. Overall, the critical depth at which the state of group evolution determines future shoaling and breaking patterns occurred at $L_0/4.2 \pm 1$ for the group types and bed slopes tested within the current study. The critical depth for groups propagating onto steeper bed slopes and with steeper wave structures was generally deeper, between $L_0/3$ and $L_0/4.3$ than for groups propagating onto flatter slopes or groups of lower initial steepness where the critical depth was found to range between $L_0/4.7$ and $L_0/5.25$.

The pattern of water level elevation within the surf zone also differed with group phasing position. In all cases, the transition between set down and set-up was delayed until a point within the surf zone where the onshore momentum flux associated with energy dissipation in the larger waves exceeded the offshore directed flux associated with shoaling of the smaller waves. However, elevation of the mean water level began to become elevated further offshore for the early shoaling type cases and the subsequent increase in mean water level was more gradual. Mean water level for delayed shoaling cases, where waves broke further inshore, became elevated later, with a steeper gradient observed. These findings were in agreement with those of the monochromatic study where waves breaking in shallower water must dissipate their energy more rapidly, leading to greater momentum flux and set-up gradients. Of note, mean water level elevation at the swx was similar for both early and delayed shoaling cases at around 6% of H_{sig} or 11% of H_{max} at initial break point. This indicates that although wave dissipation patterns between the cases differed, total energy dissipation was similar.

Chapter 7 Implications of wave groupiness for coastal structural design

7.1 Introduction

While the effects of groupiness on engineering design are recognised within standard engineering design literature, with the *Coastal Engineering Manual* (USACE, 2003) stating that *wave grouping and its consequences is of significant concern*, guidance on design in the presence of wave groups is lacking with the manual suggesting that *practitioners should be aware of the existence of groupiness and evaluate its importance in situations known to be sensitive to group-related phenomena, although procedures for this evaluation are stated as beyond the scope of the Manual*.

Determining the largest wave height, H , which can occur in water of depth, d , without breaking, is a fundamental reference quantity for the design of coastal structures. Design guidelines such as the *Coastal Engineering Manual* (USACE, 2003) and *Random Seas and Design of Maritime Structures* (Goda, 2000) are based on original studies by Weggel (1972) and Goda (1970) respectively, which were primarily based on regular or monochromatic wave testing.

Chapter 5 tested these empirical expressions using monochromatic waves and found reasonable agreement for both expressions, although the expression within Goda (2000) provided slightly better results on steeper bed slopes. However, amplitude modulation or the emergence of groupiness in the initially regular wave train was also noted. Groupiness in wave-trains is an inherent characteristic of freely propagating waves in deep water and has been shown to cause variation in derived breaker indices (Svendsen, 2001; Dack and Peirson, 2005). Hence this chapter investigates the applicability of standard design guideline formula in the presence of grouped wave trains.

This chapter focuses on changes in the breaker depth index as a function of the spatial phasing of group evolution and the initiation of shoaling. As groups are known to induce temporal variations in water level within the surf zone due to fluctuations in radiation stress, the influence of differing definitions of depth is examined. These observed indices are compared to those predicted by existing design guidelines and

discussion as to the relative importance of wave groupiness provided. Empirical expressions describing characteristic wave height indices ($H_{\max}/d$ and H_{sig}/d) rather than the breaking height (Goda, 2000) are then compared to observed data and a modification is suggested to better encapsulate the results of the present study. The maximum surface elevation, an important characteristic in engineering design, is similarly assessed and a further modification to existing empirical expressions is suggested to allow calculation of this value in the presence of wave groups. The implications of groupiness for coastal engineering design are discussed, with specific reference to the sizing of armour rock within the surf zone using both existing and the proposed modified methods for calculating breaker index and wave height.

7.2 Experimental parameters

As discussed within the previous chapter, four wave group structures were tested. The wave groups varied in their generation mechanism (*Case*), the number of waves within the group (N), and the initial wave steepness ($s_0 = a_0 k_0$), leading to either recurrent or breaking conditions. A summary of the parameters used to generate each of these wave *configurations* is presented in Table 6-1.

7.2.1 Group height distributions

The distribution of wave heights within a narrow-banded, random sea have been shown to closely approximate a Rayleigh distribution (Longuet-Higgins, 1952). The wave height distributions of the idealised groups used within the present laboratory experiments are compared to this idealised distribution, with Figure 7-1 showing a wave height probability plot for the C2N5r type observed wave group compared to an idealised Rayleigh distribution. The idealised Rayleigh distribution is calculated using an inbuilt Matlab® cumulative distribution function.

As shown in Figure 7-1, the wave groups yield a reasonable approximation of a Rayleigh distribution for the central wave heights. Both extreme upper and lower wave heights are under-represented, however, with the largest observed wave corresponding to around the 1% exceedance wave height for an idealised distribution.

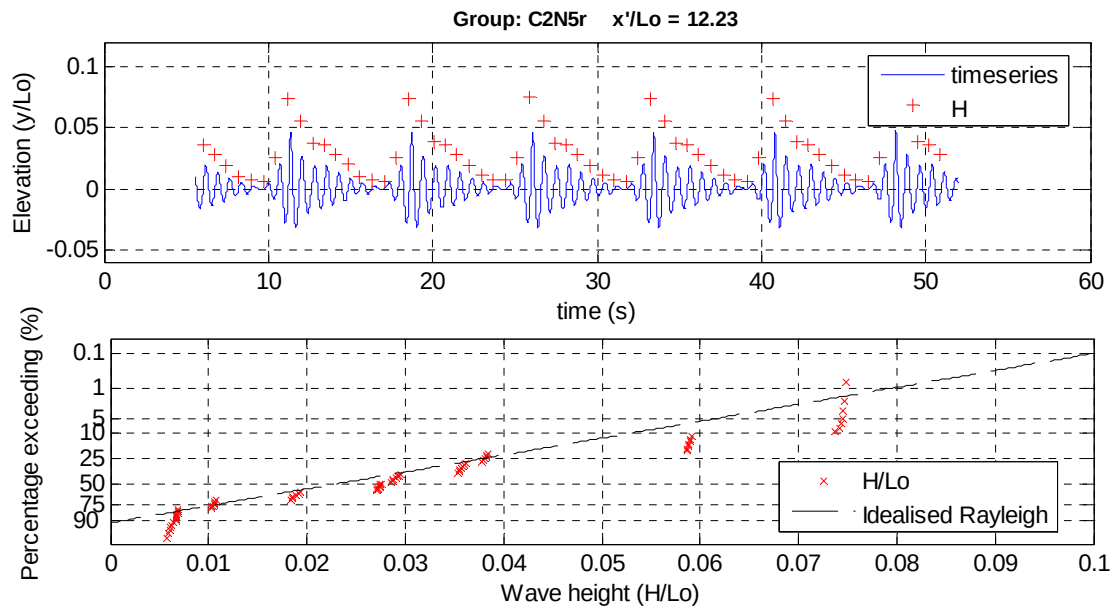


Figure 7-1 Water surface elevation and wave heights for a C2N5r type group (upper panel), with the probability of wave height occurrence for observed spectra compared to an idealised Rayleigh distribution (lower panel).

Wave heights within a Rayleigh distribution may be related to one another statistically (Longuet-Higgins, 1952) through the relationship:

$$P(H) = 1 - \exp\left[-\frac{H^2}{H_{rms}^2}\right] \quad (7-1)$$

where $P(H)$ represents the probability of a wave H occurring and H_{rms} is the root mean squared wave height. From this equation, relationships between characteristic wave heights may be approximated. While a realistic wave height distribution may vary depending on the site-specific conditions, a Rayleigh-type distribution is widely used and is considered a robust first approximation (USACE, 2003).

The relationship between observed H_{max} and H_o and between H_{max} and H_{rms} are examined for an evolving deep water series (Figure 7-2). From these relationships, a slight dependence on group evolutionary phase is observed, with the larger waves associated with a local maxima representing slightly more extreme waves in the tail of an idealized spectrum.

If an idealized Rayleigh distribution is assumed, the probability of the largest observed wave is determined using Eqn. 7-1 by comparison to H_{rms} . This yields a probability of occurrence for the largest wave in the C2N5r series between 0.2% and 2%, with an average of 0.6%. For the C2N3r series, the probability is between 3.7% and 11.9%, with an average of 7.4% and for the C2N5sb series, between 1.4% and 8.4%, with an average of 3.3%. These differences are attributed to the lower number of waves within the $N = 3$ group, therefore the largest observed waves are relatively smaller compared to the group mean. For the strong breaking type, all waves are initially larger due to the steeper structure and the maximum wave is therefore relatively smaller.

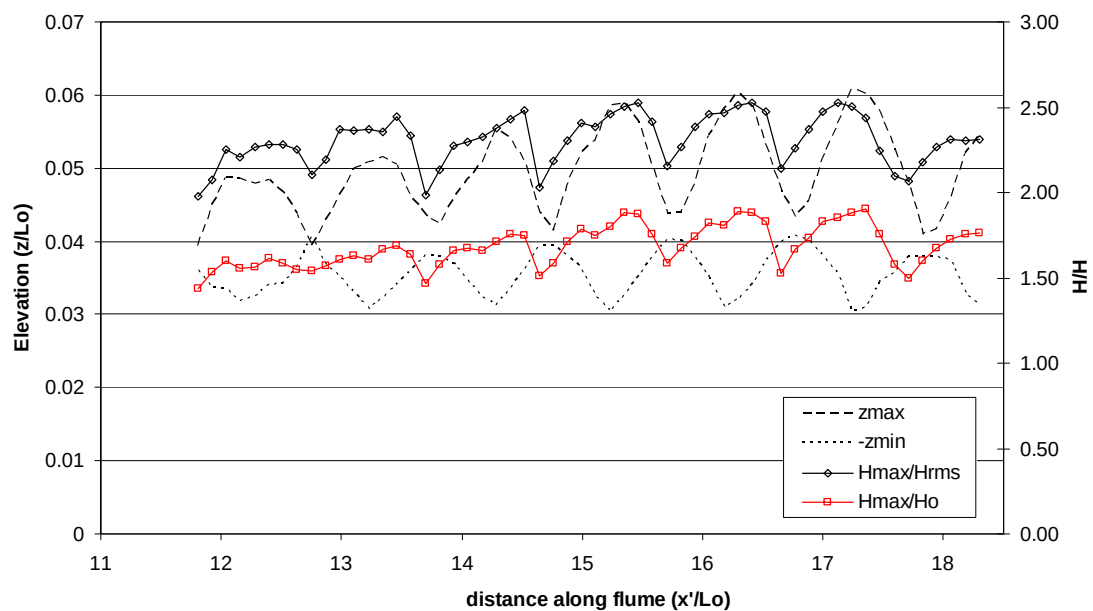


Figure 7-2 The ratio of maximum (H_{max}) to significant (H_0) wave heights and maximum to root mean squared (H_{rms}) wave heights is presented for an evolving wave group depicted by the maximum (z_{max}) and minimum (z_{min}) surface elevations.

7.2.2 Group phasing

As discussed within the previous chapter, the spatial phasing of the wave group evolutionary cycle at a depth between $L_0/4$ and $L_0/7$, influences the subsequent shoaling and breaking characteristics. These differences in shoaling characteristics are expected to become more pronounced with increasing group nonlinearity, i.e. immediately prior to recurrence or the onset of breaking (Table 6-2). Testing has therefore been focussed around these positions.

The sloping bed is incrementally adjusted and testing repeated until a complete crest maxima - trough minima - crest maxima cycle is completed. This ensures that *differences* in shoaling and breaking characteristics related to the spatial phasing of group evolution and shoaling are effectively captured. The positions of the swx for each bed slope and wave group configuration tested are given in Table 6-3.

7.3 Results and discussion

This chapter focuses on the largest wave which may occur prior to breaking in water of finite depth and the effects that wave groupiness and spatial phasing of group evolution may have on this limiting wave height. Observed breaker indices are compared to those predicted by current design guides. These guidelines, based primarily on regular wave testing are recommended as appropriate upper limits for irregular waves and should therefore remain applicable for grouped waves. The concept of effective water depth and its consequence on breaker indices are similarly assessed and conclusions drawn as to the most appropriate definition in the presence of wave groupiness.

The concept of limiting characteristic wave heights is also assessed. Characteristic waves such as $H_{\max}$, H_{sig} and H_{rms} are often required for engineering design and the concept that $H_{\max}$ is roughly equivalent to breaking wave height is examined. Within deep water, wave heights are typically distributed according to certain distributions. Based on these distributions, characteristic wave heights may be related to one another statistically (Longuet-Higgins, 1952).

7.3.1 Comparison of breaker indices to existing guideline predictions

As described within Section 5.4.2.1, the engineering practitioner uses established design guidelines such as the *Coastal Engineering Manual* (USACE, 2003) and *Random Seas and Design of Maritime Structures* (Goda, 2000) to define the largest wave which may occur in water of finite depth. These guidelines use design curves derived from original studies by Weggel (1972) and Goda (1970) respectively with equations defining their curves given by Eqns. 5-1 and 5-2 respectively.

Definition of the breaker depth index for individual waves within a group relies on accurately defining the breaking wave height. In almost all cases, the breaking wave

height corresponded to the maximum wave height at break point. This was manually checked on a wave-by-wave basis using the time series of surface elevation and wave heights at the defined break points (Figure 7-3). The relationship of $H_b = H_{\max}$ is not, however, preserved when a larger wave breaks by plunging directly offshore of the break point of a smaller wave. In this case, the splash-up of the broken wave may reach a higher elevation than the breaking wave height of the smaller wave (i.e. Figure 7-4) and manual reselection of the breaking wave height is required. Note in Figure 7-3 that at the break point of the third wave in the group, while the elevation of the first wave crest is higher, the trough elevation is also higher and thus the overall height of wave 3 is larger.

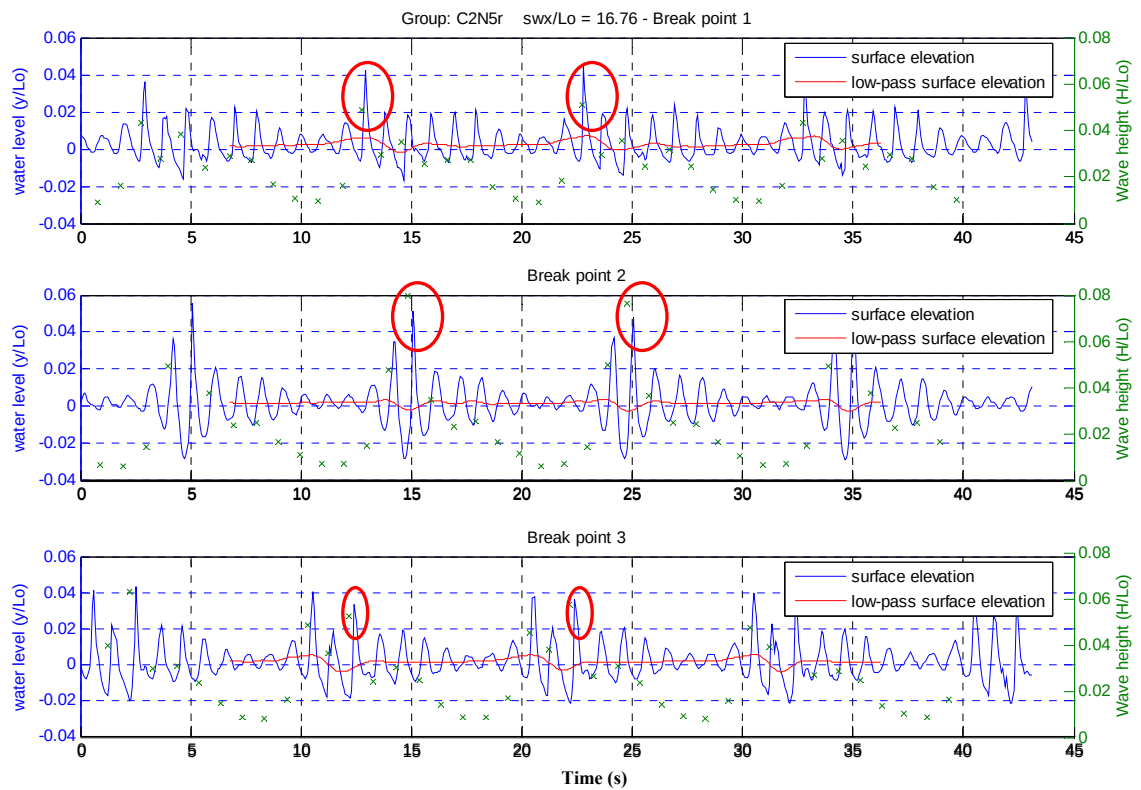


Figure 7-3 Surface elevation and individual wave height at three break point locations for a C2N5r-type group breaking on a 10:1 slope bed. At the break point of the first wave in the group, two originally larger waves have broken offshore and become reduced in height, so that the largest wave in the series ($H_{\max}$) is wave 1 (circled). At break point 2, the largest wave is wave 2 (circled) and at break point 3, the larger wave 2 has broken, its height reduced and wave 3 (circled) is the largest.

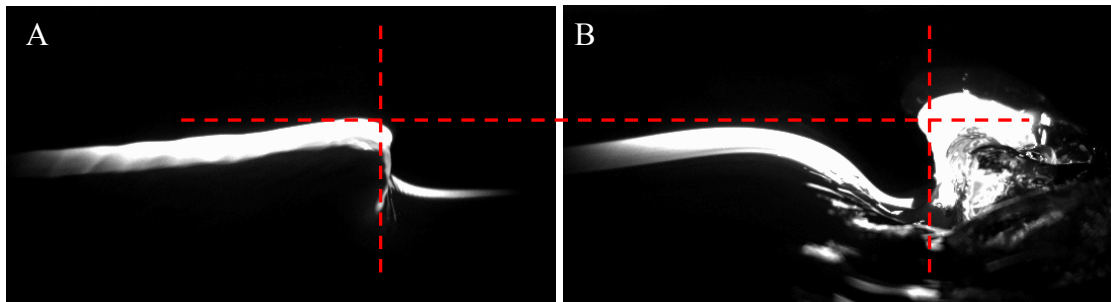


Figure 7-4 The breaking wave elevation of a smaller first wave (A) is exceeded by the splash-up of a following larger wave (B). The breaking height of the smaller, first wave is therefore not equal to the $H_{\max}$ of the record at that location, and H_b must be manually selected from the wave height record.

7.3.1.1 Depth based on still water level

The breaking wave heights, locations and resultant H_b/d_{swl} ratios, where d_{swl} is the depth based on still water level, are presented in Figure 7-5 for the C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed. Table 7-1 provides these values compared to those predicted by the *Coastal Engineering Manual* (USACE, 2003) and *Random Seas and the Design of Maritime Structures* (Goda, 2000) design guidelines. The figure and table show the first, smallest wave of the group to have the largest H_b/d_{swl} value with the sloping bed at $x'_{swx}/L_0 = 16.53$. The second, largest wave of the group has smallest H_b/d_{swl} and the third wave has a slightly larger breaking wave height than the first wave and slightly smaller H_b/d_{swl} value. The first and third waves exceed values predicted by either the Coastal Engineering Manual (USACE, 2003) or Goda (2000) by 8 to 15% while the second wave has a value below guideline predictions.

As the bed is shifted and the first wave decreases in size, breaking closer to the swx, its corresponding H_b/d_{swl} value increases, substantially exceeding the predicted guideline values. At $x'_{swx}/L_0 = 17.71$, the H_b/d_{swl} for the first wave exceeds the guideline values by up to 100%. Possible explanations for this include the super-elevation of mean water level within the surf zone due to set-up. This means that the true water depth at the breaking location is greater than the depth based on still water level and the actual H_b/d ratio should be smaller than calculated. This is further discussed within Section 6.3.2.

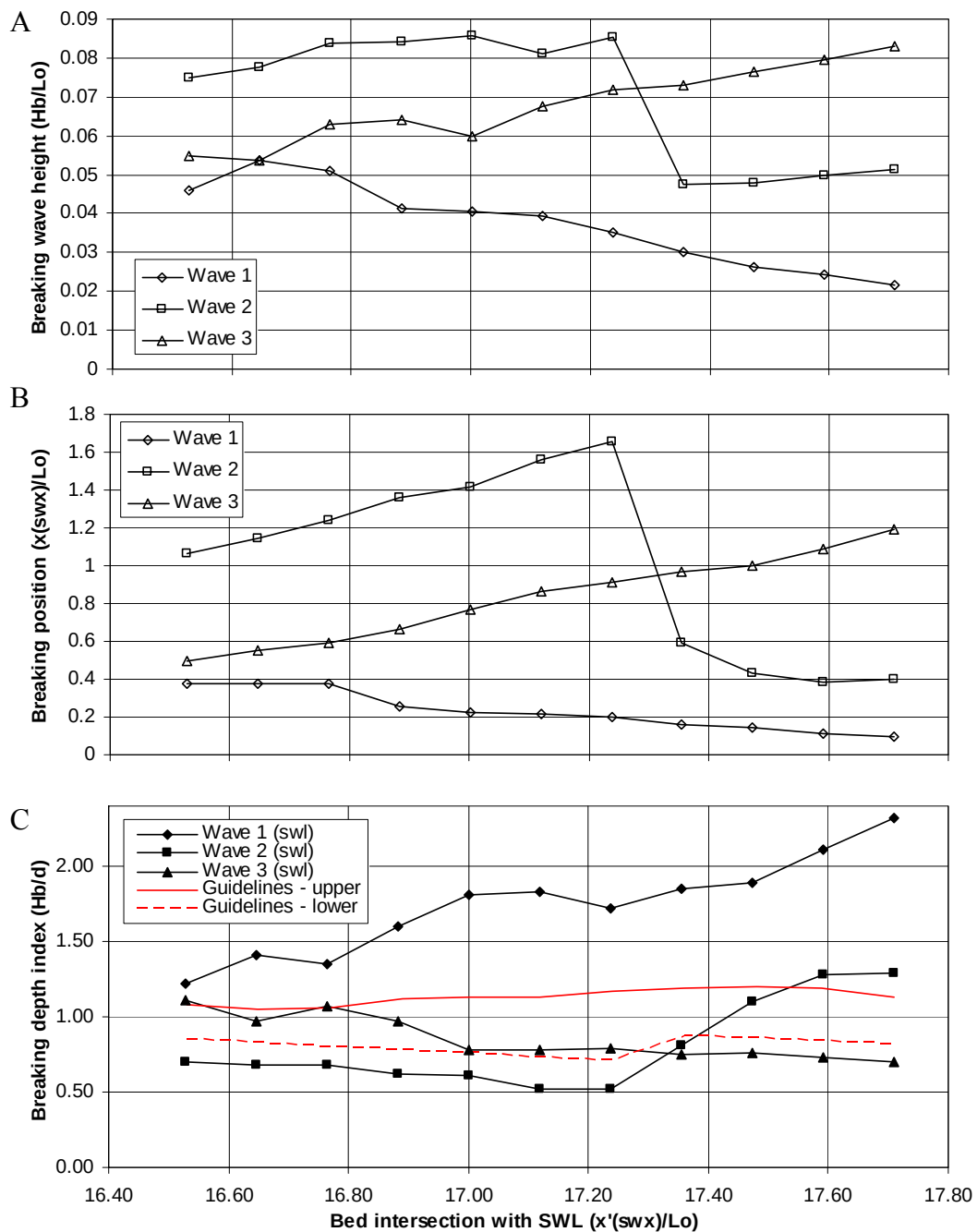


Figure 7-5 Breaking wave height (A), location (B) and breaker depth index, with depth based on still water level (H_b/d_{swl}) (C) for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed. The upper and lower envelope of values predicted by design guidelines (USACE, 2003; Goda, 2000) are also provided for comparison.

For engineering design purposes, the larger waves within the group are generally of more relevance. These waves, breaking at, or near the outer edge of the surf zone, are subject to less set-up influence and so the observed H_b/d_{swl} values should be more comparable to guideline predicted values. The second wave generally maintains a similar wave height as the bed is incrementally shifted, while the breaking location

moves offshore (as discussed within Chapter 6). The H_b/d_{swl} value therefore decreases, and is below that predicted by the guidelines. At a certain point, however, breaking is not able to occur ($x'_{swl}/L_0 = 17.36$ in this case), the break point shifts onshore and wave height is reduced as energy is redistributed to the following wave within the group. The H_b/d_{swl} value of this now smaller wave, breaking in shallower water, increases. This value increases to exceed that predicted by guidelines by up to 20% as the wave break point moves shoreward, and the wave assumes the *position* of the former wave one. The third wave increases in height and its break point moves offshore. The H_b/d_{swl} value of this wave, originally exceeding the guidelines, decreases to below those predicted as this wave assumes the *position* of the former wave two.

Table 7-1 H_b/d_{swl} values for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed. Values predicted by the Coastal Engineering Manual (USACE, 2003) and Goda (2000) design guidelines are also provided for comparison.

Bed Position (x'_{swl}/L_0)	Breaking Wave 1			Breaking Wave 2			Breaking Wave 3		
	Exp	CEM	Goda	Exp	CEM	Goda	Exp	CEM	Goda
16.53	1.22	1.08	1.06	0.70	0.92	0.85	1.11	1.03	1.02
16.65	1.41	1.05	1.06	0.68	0.90	0.83	0.97	1.02	1.00
16.77	1.35	1.06	1.06	0.68	0.87	0.80	1.07	0.99	0.99
16.89	1.60	1.12	1.10	0.62	0.87	0.78	0.97	0.99	0.96
17.00	1.81	1.13	1.12	0.61	0.86	0.76	0.78	1.01	0.93
17.12	1.83	1.13	1.12	0.52	0.88	0.73	0.78	0.97	0.90
17.24	1.72	1.17	1.13	0.52	0.87	0.71	0.79	0.93	0.89
17.36	1.85	1.19	1.14	0.81	1.07	0.99	0.75	0.94	0.87
17.48	1.89	1.20	1.15	1.10	1.07	1.04	0.76	0.92	0.86
17.59	2.11	1.19	1.16	1.28	1.07	1.06	0.73	0.90	0.84
17.71	2.32	1.13	1.17	1.29	1.06	1.07	0.70	0.87	0.82

Figure 7-6 presents observed breaker index as a function of relative depth compared to guidelines predictions. Overall, for the C2N5r-type group propagating onto a 10:1 sloped bed, guideline values were always exceeded by waves within the inner surf zone ($d/L_0 < \sim 0.02$). These waves exceeded guidelines by up to 100% in some cases. Waves within the mid surf zone ($0.02 < d/L_0 < 0.06$) exceeded guideline values by up to 20%, while waves breaking near the seaward edge, or in the outer surf zone ($d/L_0 > \sim 0.06$), typically had values below those predicted by guidelines. The spatial phasing of group evolution was important, with the previously discussed *delayed shoaling groups* tending to exhibit indices within the mid surf zone exceeding guidelines while those waves in the early shoaling groups remained below guidelines values.

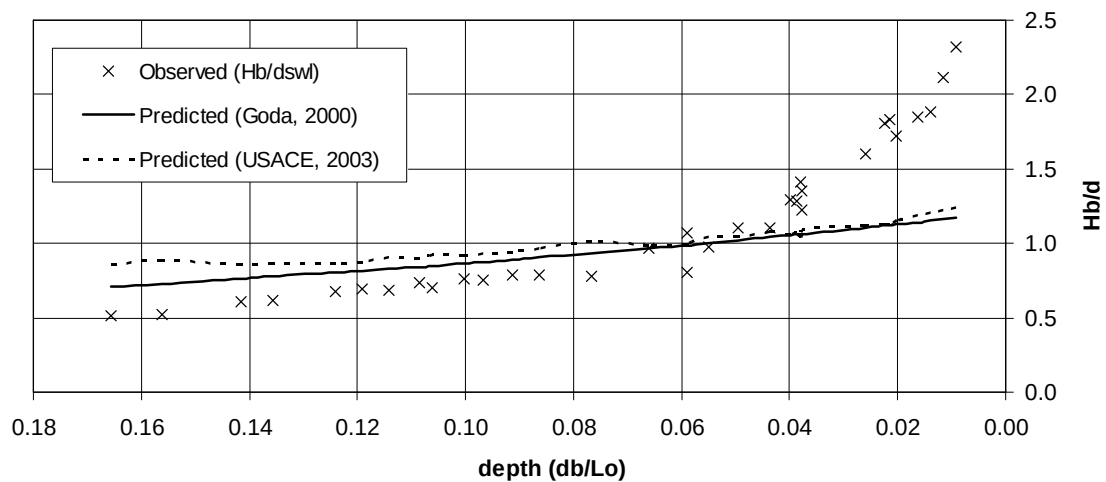


Figure 7-6 Breaker depth index, with depth based on still water level (H_b/d_{swl}) for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed with values predicted by design guidelines (USACE, 2003; Goda, 2000) also provided for comparison.

Similar cyclic trends are observed for each of the group types propagating onto differing bed slopes. Comparison of the observed upper and lower breaker depth indices across all bed positions for the different wave group types and bed slopes are presented within Table 7-2. Note that for consistency wave 1 is defined as the smaller wave at the front of the group which decreases in height as the sloping bed is moved incrementally along the tank, while wave 2 increases in height and its breaking position moves offshore until breaking ceases, after which its breaking position shifts shoreward and its height reduces. Wave three increases in height as it receives energy and its breaking position moves progressively offshore. These values are compared to those predicted by design guidelines (USACE, 2003; Goda, 2000) and the minimum and maximum percentage differences are provided.

For all wave group types, on all bed slopes, each wave exceeds values predicted by design guidelines during some phase of evolution (i.e. during at least one bed position) (Figure 7-7). Similar to that found for only the C2N5r type group propagating onto the 10:1 slope, observed values tended to exceed at least one of the guideline predictions (i.e. either the USACE, 2003 prediction or Goda, 2000 prediction) for all slopes in water shallower than around $d_b/L_0 = 0.06$. In depths shallower than around $d_b/L_0 = 0.02$ (inner surf zone), observed values on the 10:1 and 5:1 slope exceeded both guidelines, although for the 20:1 slope, all waves were observed to break outside this depth and thus the guidelines were not exceeded to the same extent.

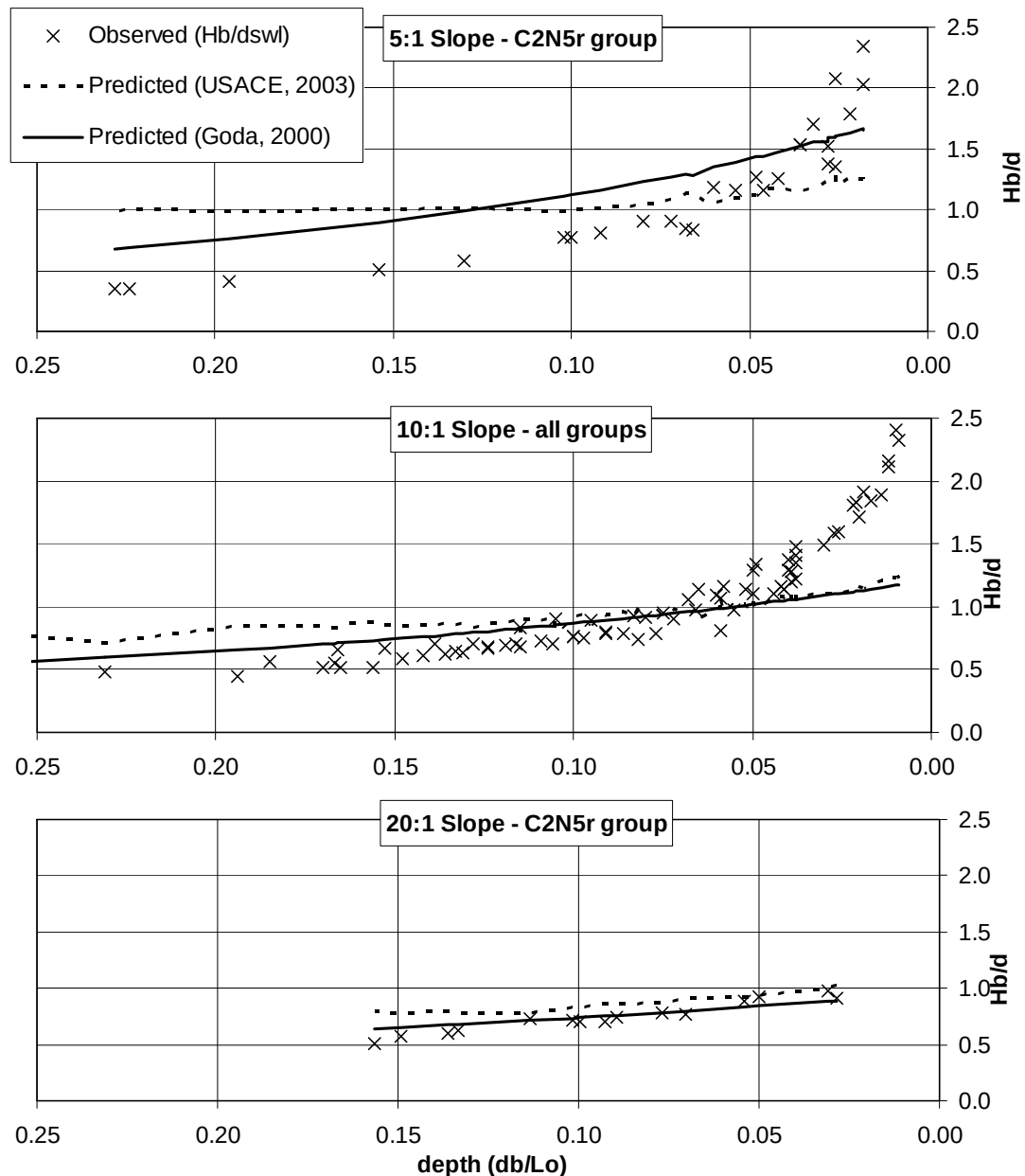


Figure 7-7 Breaker depth index, with depth based on still water level (H_b/d_{swl}) for the C2N5r type group breaking onto 5:1 and 20:1 slopes and all groups breaking on the 10:1 slope. Values predicted by design guidelines (USACE, 2003; Goda, 2000) are provided for comparison.

These results suggest that the current design guidelines, based on monochromatic wave testing and using a depth defined according to still water level, remain applicable and conservative for the largest waves that break at the outer edge of the surf zone ($d_b/L_0 > 0.06$). However, these methods are not suitable for prediction of breaking wave heights which may occur within the mid to inner surf zone. Furthermore, the spatial phasing of group evolution was found to be influential, with *delayed shoaling groups* exhibiting indices exceeding guidelines within the mid surf zone, while the *early shoaling groups*

tended to remain below design guides. In both cases, however, larger waves breaking in the outer surf zone remained below guideline values while smaller waves within the inner surf zone exceeded guideline values.

Table 7-2 Comparison of the observed upper and lower breaker depth indices across all bed positions for the different wave group types and bed slopes and the minimum and maximum difference (%) from predicted design guide values (USACE, 2003; Goda, 2000).

Bed slope	Group type	Wave 1				Wave 2				Wave 3			
		Observed		Diff from guidelines (%)		Observed		Diff from guidelines (%)		Observed		Diff from guidelines (%)	
		Lower	Upper	Min	Max	Lower	Upper	Min	Max	Lower	Upper	Min	Max
5:1	C2N5r	-	-	-	-	0.35	2.08	-65	78	0.78	1.11	-35	12
10:1	C2N5r	1.22	2.32	13	105	0.52	1.29	-41	22	0.7	1.11	-23	9
	C2N3r	1.33	1.47	30	42	0.56	1.16	-32	20	0.9	1.29	1	32
	C2N5sb	1.2	2.4	10	106	0.45	1.58	-47	44	0.63	1.06	-27	13
	C3N5r	-	-	-	-	0.37	1.49	-51	37	0.56	0.89	-24	4
20:1	C2N5r	-	-	-	-	0.51	0.97	-35	11	0.57	0.74	-27	3

7.3.1.2 Alternative definitions of depth

As evident from the above discussion, breaker indices derived from regular wave testing provide a reasonable approximation of larger waves breaking at the outer edge of the surf zone but provide a poor prediction of breaker indices within the surf zone. Breaker indices substantially higher than predicted are observed within the inner surf zone.

Goda (2000) recommends that for irregular wave breaking, where wave breaking may occur over a wide area within the surf zone, various contributors to water level must be allowed for. These include changes to the mean water level due to set down and set-up and temporal changes to the water level through *surf beat*, or low-frequency fluctuations. While these differences in definition of depth would not significantly influence results in regular wave testing where wave breaking occurs in a relatively consistent location, they may cause significant variation in irregular or particularly grouped wave testing. Alternative definitions of depth are therefore considered in order to examine the discrepancy between observed and predicted values. These alternative definitions are used to recalculate observed breaker indices and these recalculated values compared to predicted values.

Mean water level

As discussed within Section 5.4.3, mean water level is first depressed as monochromatic shoaling waves approach break point. This mean water level then becomes elevated following wave breaking as momentum flux becomes directed in an onshore direction. For wave groups (Section 6.3.3), where the break point varies in space, an increase in mean water level does not occur until a point within the surf zone where the momentum flux in the onshore direction due to the dissipation of broken waves exceeds the momentum flux in the offshore direction associated with wave shoaling. Furthermore, as the breaking and dissipative processes vary, depending on the spatial phasing of wave groups, mean water level through the surf zone similarly varies (Figure 6-12).

Depth may thus be evaluated based on the mean rather than still water level and resultant breaker indices (H_b/d_{mwl}) calculated accordingly. Figure 7-8 presents H_b/d_{mwl} ratios for a C2N5r type wave group breaking on a 10:1 sloped bed. The breaker index values are based on still water depth (H_b/d_{swl}) and the upper and lower envelope of design guideline predicted values are included for comparison.

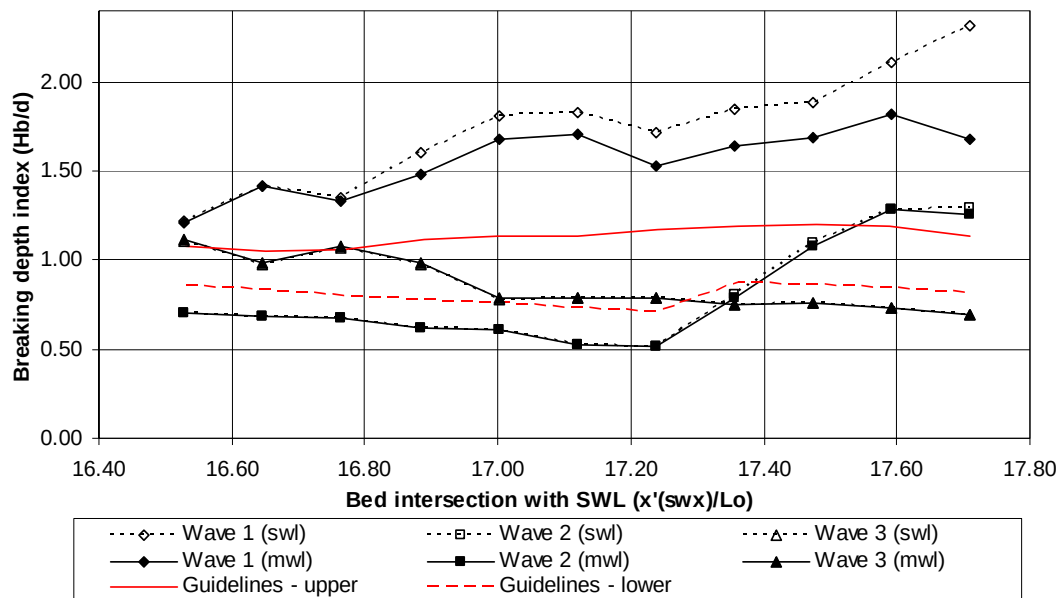


Figure 7-8 Wave breaker depth index, with depth based on mean water level (H_b/d_{mwl}) compared to H_b/d_{swl} for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed. The upper and lower envelope of values predicted by design guidelines (USACE, 2003; Goda, 2000) are also provided for comparison.

Little difference is observed for the larger waves breaking in the outer surf zone as set down is typically small in magnitude compared to set-up (Longuet-Higgins and Stewart,

1963, 1964). However, more substantial difference is noted for the smaller waves in the inner surf zone where set-up becomes substantial.

Effective water level

While the mean water level provides indication on changes in the time-averaged water depth, wave groups also induce temporal variation in water level as discussed in Section 6.3.4. In deep water and at the outer edge of the surf zone, a low frequency, or bound wave trough is most prominent. This depression in water level is due to radiation stresses associated with the largest waves within the group. Within the surf zone, a low frequency crest, preceding the group envelope maximum, develops, increasing in amplitude toward the shoreline. These patterns are evident within space-time plots of lowpass filtered surface elevation (Figure 6-14) and are in agreement with previous investigations (Baldock et al., 2000; Baldock 2004).

Nelson and Gonslaves (1992) discuss the concept of *effective depth*, where the depth at the time of wave passage is used to define limiting wave height and wave decay. This is similar to the depth proposed by Black and Rosenberg (1992) based on the mean depth over the cycle of the wave being considered. However, this later definition is highly dependent on whether zero-up or zero-down crossing analysis is used to define the wave cycle. To avoid this, the water level at the break point of an individual wave is obtained from the lowpass filtered surface elevation at the time of zero-upcrossing (Nelson and Gonslaves, 1992). This *effective depth* is referred to henceforth as d_{eff} .

Figure 7-9 presents H_b/d_{eff} ratios for a C2N5r type wave group breaking on a 10:1 sloped bed. The breaker index values based on still and mean water depth (H_b/d_{swl} and H_b/d_{mwl}) and the upper and lower envelope of design guidelines predicted values are included for comparison. Lower indices are observed for the smaller, preceding wave within the group than are obtained using either the still or mean water levels. This is attributed to the elevated water level (or low frequency crest) which precedes the group envelope maximum, as is evident within Figure 6-14 and discussed by Baldock (2004). These waves, which move towards the front of the group, may therefore ‘ride’ on the low frequency crest and thus propagate further into shallower water before breaking than would generally be possible.

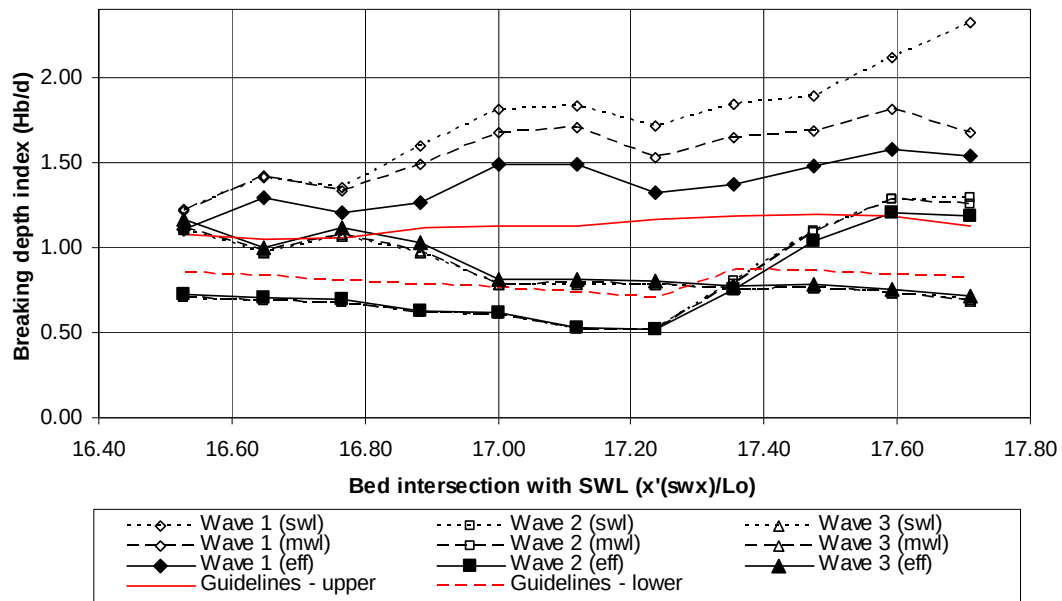


Figure 7-9 Wave breaker depth index, based on the effective depth (H_b/d_{eff}) compared to H_b/d_{swl} and H_b/d_{mwl} for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed. The upper and lower envelope of values predicted by design guidelines (USACE, 2003; Goda, 2000) are also provided for comparison.

Larger waves, breaking near the outer surf zone, are largely unaffected, or have slightly increased observed indices as set-down below the larger waves (or a low frequency trough) temporarily decreases water depth. However, waves breaking within the mid surf zone may be affected either way dependent on their position within the group. Although similar in breaking height and position, waves towards the back of the group (wave 3 between $x'_{swx}/L_0 = 16.53$ and 17.24) exhibit slightly increased indices as they occur in conjunction with a low frequency trough and waves towards the front of the group (wave 2 between $x'_{swx}/L_0 = 17.31$ and 17.71) exhibit decreased indices as they occur in conjunction with a low frequency crest. This is in partial agreement with findings of Svendsen (2001), where individual waves within a modulated wave-train were found to have differing breaker index values. Although Svendsen found the larger waves within the group to have higher index values, which was not found within this study, waves towards the front of his groups tended to exhibit slightly higher indices.

Maximum water level

Alternatively, and for the sake of simplicity, depth at a particular cross-shore location may be defined with respect to the upper envelope of the temporally varying water level or dynamic set-up (Figure 6-13). This depth, based on the maximum lowpass filtered water level represents a maximum effective depth ($d_{\max}$). Figure 7-10 depicts the observed breaker depth indices $H_b/d_{\max}$ compared to those predicted using the still, mean, or effective water levels and to values predicted by the Coastal Engineering Manual (USACE, 2003) and Goda (2000) design guidelines.

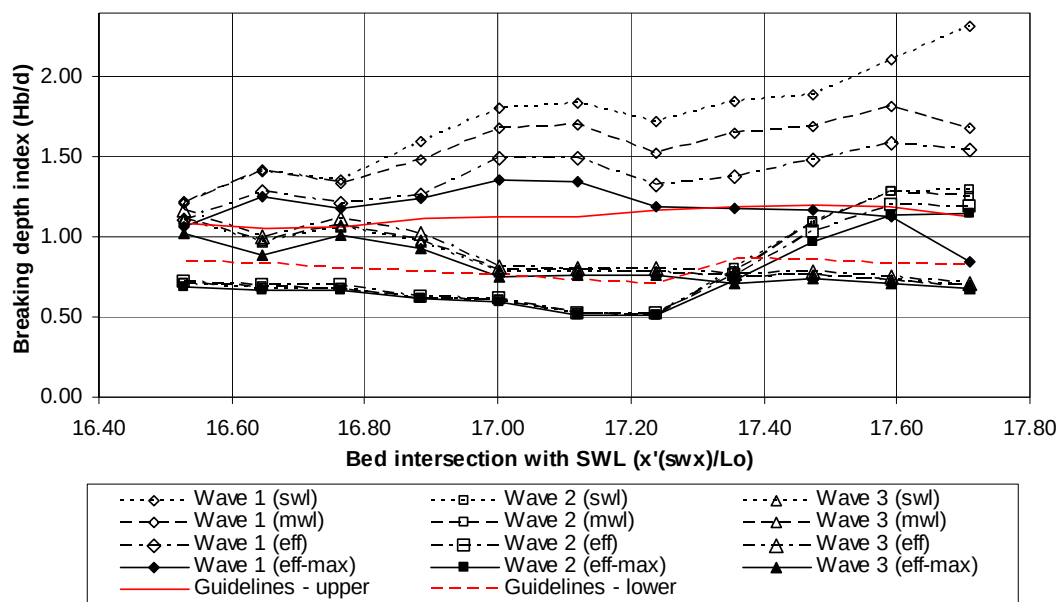


Figure 7-10 Wave breaker depth index, based on the maximum lowpass surface elevation, ($H_b/d_{\max}$) compared to H_b/d_{eff} , H_b/d_{swl} and H_b/d_{mwl} for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed. The upper and lower envelope of values predicted by design guidelines (USACE, 2003; Goda, 2000) are also provided for comparison.

Values at the outer surf zone are only slightly reduced from those obtained using the previous definitions of depth due to the small crest amplitude of low-frequency variations in the outer surf zone. Values within the mid surf zone are reduced further and values within the inner surf zone are substantially reduced. As an upper envelope depth is assumed, many observed values are over predicted using this approach. However, it would be expected to provide a conservative upper envelope on possible wave height. While select observed values in the inner surf zone remain up to 21% above predicted values, this is a significant improvement over the 100% difference found using a depth based on still water level.

7.3.1.3 Summary

Overall, breaker index values were observed to increase with distance into the surf zone. Such a variation in breaker index as a function of depth has been suggested by several investigators for random wave fields (Raubenheimer and Guza, 1996; Goda 2007). These variations were attributed to elevated water levels within the inner surf zone enabling the propagation of relatively larger waves into these areas before breaking occurred. A comparison of observed breaker indices using differing definitions of depth for a C2N5r type wave group propagating onto a 10:1 sloped bed, with the spatial phasing of group evolution relative to shoaling incrementally adjusted, is presented as a function of depth within Figure 7-11.

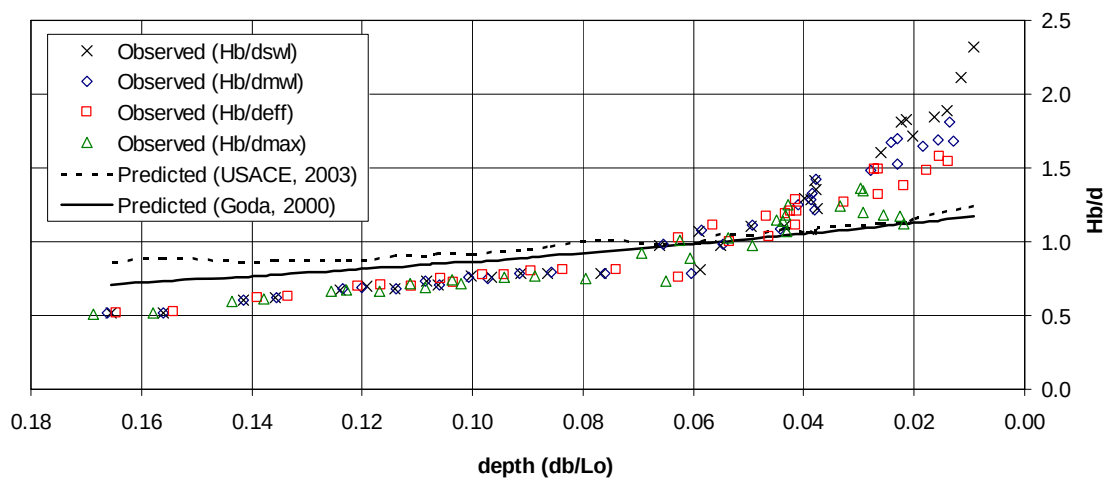


Figure 7-11 Observed wave breaker index values based on the maximum lowpass surface elevation, ($H_b/d_{\max}$), the effective surface elevation at the time of wave passage (H_b/d_{eff}), the mean water level (H_b/d_{mwl}) and the still water level (H_b/d_{swl}) for the three largest waves of a C2N5r-type wave group breaking on an incrementally adjusted 10:1 sloping bed compared to values predicted by design guidelines (Goda, 2000; USACE, 2003).

Observed values were found most similar to those predicted by guidelines when depth is defined according to the maximum envelope of lowpass surface elevation. However, in certain cases within the mid to inner surf zone, observed values remained up to 20% above those predicted by guidelines. Additionally, this method has the disadvantage that by providing a conservative, upper bound on depth, over-prediction of the majority of breaking waves is likely. Similarly, while this low-frequency surface elevation value has been available within the present study due to the nature of the data collected, the prediction of such values for specific physical sites with variable bed morphologies and sea conditions is problematic. Goda (2000) offers an empirical expression for estimating

the amplitude of surf beat within the surf zone (Eqn. 2-32). However, as this root-mean-squared value does not provide information on the maximum amplitude (Figure 6-13), it similarly does not allow description of the maximum possible depth or wave height.

Definition of an upper envelope of dynamic set-up, applicable over a multitude of bed slopes, morphologies and real-sea environments is beyond the scope of this investigation. However, this study has shown that breaker indices derived from regular wave testing may underestimate breaker indices within the mid to inner surf zone regardless of the definition of depth employed. The spatial phasing of group evolution also appears to be of importance, with delayed shoaling groups tending to exhibit waves within the mid surf zone that exceed design guideline values, while these waves are not present within the early shoaling groups.

To accurately resolve breaker depth index, either:

1. the maximum anticipated depth, including changes in the dynamic water level, must be determined, after which the breaker index may be calculated using design guidelines derived from regular wave testing (to a confidence of 20% based on these present results);
2. or a revised expression for breaker index should be derived which implicitly includes the effects of low frequency water level variation or dynamic set-up.

7.3.2 Revising index predictors

Goda (1975) proposed that his earlier criterion for regular wave breaking (Eqn. 5-2) could be employed for determining characteristic wave heights in irregular wave breaking by adjusting the coefficient A . The characteristic wave approach defines a maximum characteristic wave (i.e. $H_{\max}$, H_{sig} , H_{rms}) possible in a finite water depth as opposed to a breaker height and is typically used for irregular waves where the breaking position of individual waves is not known.

This coefficient was assumed at 0.17 for regular waves at break point. As the breaker indices derived for regular waves have been suggested as appropriate for defining the highest wave in an irregular wave-train (Goda, 1975, 2007; Seyama and Kimura, 1988; Kamphuis, 1991; Durand, 1997), this coefficient value is suggested as appropriate for

the maximum wave height. For the significant wave height, this coefficient was determined at 0.12 at incipient breaking (Goda, 2000).

Goda (2007) reanalysed data sets of several earlier investigations and found A to vary across the surf zone (Section 2.5.2.2). While these studies have used beach slopes between 1:20 and 1:70 which are lower than the present 1:10 beach slope, the study on a 1:20 slope (Battjes, 1972) noted an increase in A to 0.2 near the shoreline. However, this maximum value of A found during Goda's reanalysis remains insufficient to encapsulate the data from the present laboratory study using a depth based on still water level. The present data has therefore been analysed to determine the required coefficient A to match observed H_{sig}/d , H_{max}/d , z_{max}/d and H_{br}/d with predicted assuming a depth based on still water level.

7.3.2.1 Significant wave height

The significant wave height (H_{sig}), is a characteristic wave height used often in engineering design (USACE, 2003; CIRIA, 2007). This height may be defined either statistically as the arithmetic mean of the largest 1/3 of waves, or as an energy-based value, H_{m0} , as four times the surface elevation variance. While in deep water these values are similar, with $H_{1/3}$ generally assumed $\approx 0.95H_{m0}$, during wave shoaling, increasing nonlinearity causes $H_{1/3}$ to increase, exceeding H_{m0} (Goda, 2007). The value of H_{m0} , is therefore used instead as the nonlinear enhancement of $H_{1/3}$ means this value is not representative of breaking-dissipated wave energy levels (Goda, 2007).

As the break point in an irregular wave field occurs over a wide distance, definition of a breaking location cannot be employed in the same manner as for regular waves. The concept of an incipient break point is therefore used (Kamphuis, 1991). As discussed within Section 2.3.2, to define the incipient break point, the significant wave height is defined from a distance offshore and through the breaking and surf zone. Curves are fitted to the shoaling portion of the measurements and to the wave height decay portion. The intersection of these curves defines the incipient break point (Goda, 2007).

The significant wave height (H_{sig}) of three example cases for a C2N5r type wave group propagating onto a 10:1 sloped bed are examined (Figure 7-12A). These three example cases are described within Section 5.4 and correspond to an *early shoaling* case and two *delayed shoaling* cases. The incipient break point was found to occur at around x_{swx}/L_0

$= 0.8$ for all cases. The H_{sig}/d ratios for the observed data are compared to that predicted by Eqn. 5-2 with $A = 0.12$ with depth based on still water level (Figure 7-12B). Results show the equation to slightly over-predict breaker index near the incipient break point and to substantially under-predict breaker index within the inner surf zone when depth is based on still water level.

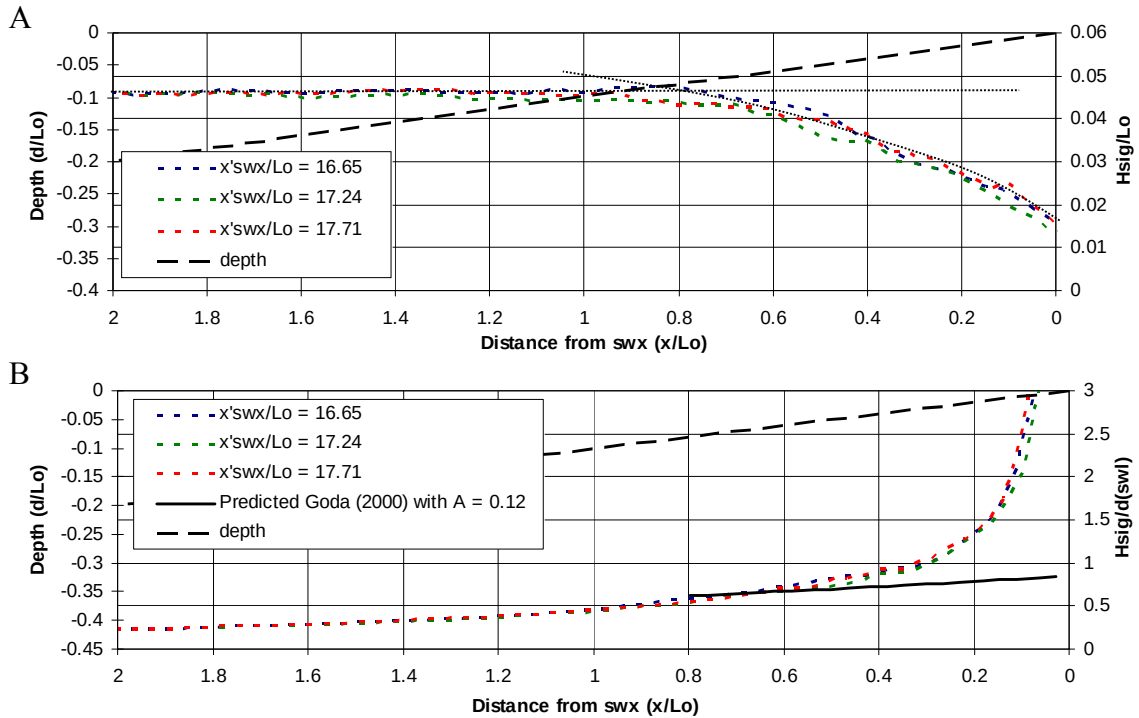


Figure 7-12 Significant wave height (H_{m0}) for a C2N5r type wave group propagating onto a 10:1 sloped bed for three different group spatial-phasing cases with incipient break point denoted by the intersection of dotted lines (A). The resultant breaker depth indices based on d_{swl} are given within (B) along with values predicted by Eqn. 5-2 with $A = 0.12$.

The value of coefficient A is adjusted through the surf zone to match predicted to observed breaker indices. These values are presented within Figure 7-13 as a function of relative depth (d_{swl}/H_{sig}'), where H_{sig}' is the energy-based deep water significant wave height. Depth with respect to deep water wave height is used in preference to deep water wave length as the use of hight better characterizes the relative position within the surf zone and thus the relative contribution of group-induced water level variation. Best-fit curves are fitted to the resultant values, thus providing an expression for the coefficient A as a function of relative depth. Slight variation is evident dependent on the shoaling type (early or delayed). This was similarly observed in the raw data sets with wave height found to remain slightly higher in the delayed shoaling case as breaking occurred closer to the shoreline.

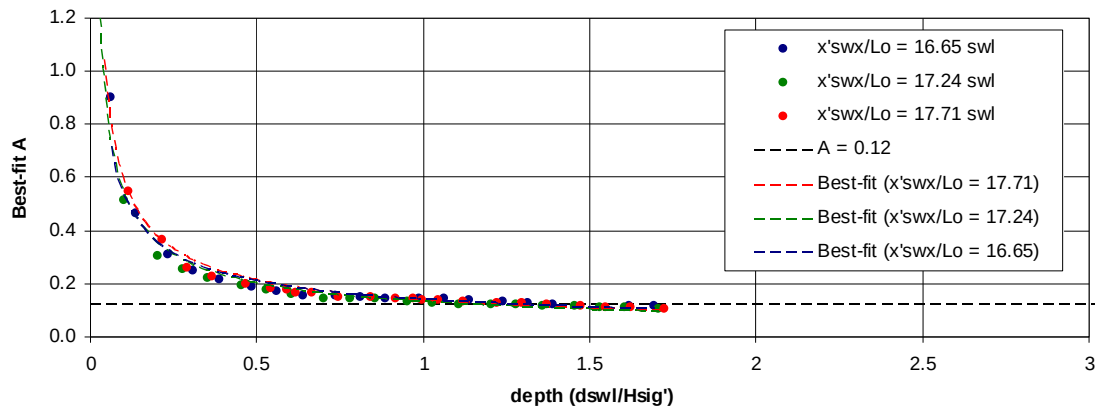


Figure 7-13 Required coefficient A for breaker index values predicted by Eqn. 5-2 to match observed H_{sig}/d_{swl} for three example cases of a C2N5r type wave group propagating onto a 10:1 sloped bed.

Waves propagating on to slopes of 5:1 and 20:1 are similarly evaluated and expressions for the A coefficients determined. Although slight differences between the delayed and early shoaling cases are evident, the expressions for the delayed cases are used for conservatism. These expressions are combined through incorporation of the bed slope parameter giving the following combined expression:

$$A_{sig} = (-0.24m + 0.17) \left(\frac{d_{swl}}{H'_{sig}} \right)^{(-1.65m - 0.42)} \quad (7-2)$$

However, this equation also illustrates a problem with using depth based on still water level to define breaker index and wave height. Depth based on still water level tends to zero at the intersection of a sloping bed and still water. In practice, water depth remains finite due to set-up effects and so the observed breaker index tends to infinity using the above equation. The equation is therefore assumed valid only for the limits of the data collected, i.e. $d_{swl}/H'_{sig} > 0.02$.

7.3.2.2 Maximum wave height

As discussed previously, the breaker indices derived for regular waves have been suggested as appropriate for deriving the highest wave in an irregular wave-train, i.e. $H_{max}/d \approx H_b/d$. Observed values of H_{max}/d_{swl} for three example cases of a C2N5r type wave group propagating onto a 10:1 sloped bed are therefore compared to those predicted by Eqn. 5-2, with the coefficient A adjusted to provide a best-fit. These values are presented within Figure 7-14.

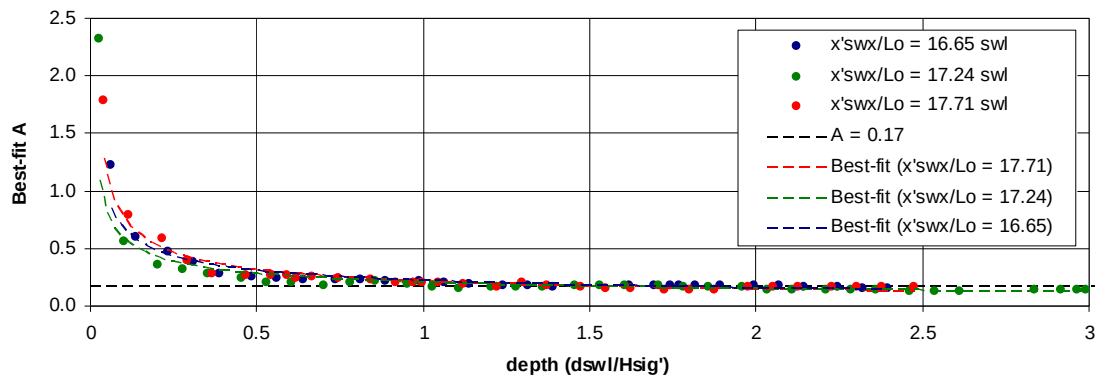


Figure 7-14 Required coefficient A for breaker index values predicted by Eqn. 5-2 to match observed $H_{\max}/d_{swl}$ for three example cases of a C2N5r type wave group propagating onto a 10:1 sloped bed.

Waves propagating onto slopes of 5:1 and 20:1 are similarly evaluated and expressions for the A coefficients determined. Again, differences between the delayed and early shoaling cases were evident but the coefficients derived for the delayed cases are used for conservatism. These coefficients are combined through incorporation of a bed slope parameter giving the following combined expression:

$$A_{\max} = (-0.12m + 0.25) \left(\frac{d_{swl}}{H'_{m0}} \right)^{(-1.65m - 0.35)} \quad (7-3)$$

Comparing observed breaker indices

Predicted H_b/d_{swl} values for the individual breaking waves of the C2N5r type wave group propagating on to a 10:1 sloping bed are recalculated using Eqn. 5-2 and the modified coefficient, A (Eqn. 7-3). These values are compared to observed values (Figure 7-15), with the predicted values in good agreement, or slightly higher than observed in all cases. Figure 7-16 compares the same data as a function of depth and shows the modified expression to provide a suitable upper envelope to the data.

In very shallow water (wave 1 between $x'_{swx}/Lo = 17.24$ and 17.71), the model appears to somewhat over-predict H_b/d . This implies that the maximum wave height in very shallow water is larger than observed breaking wave heights. This was shown in Figure 7-4 where splash-up and other occurrences from larger breaking waves may exceed the breaking wave height of small waves within the inner surf zone.

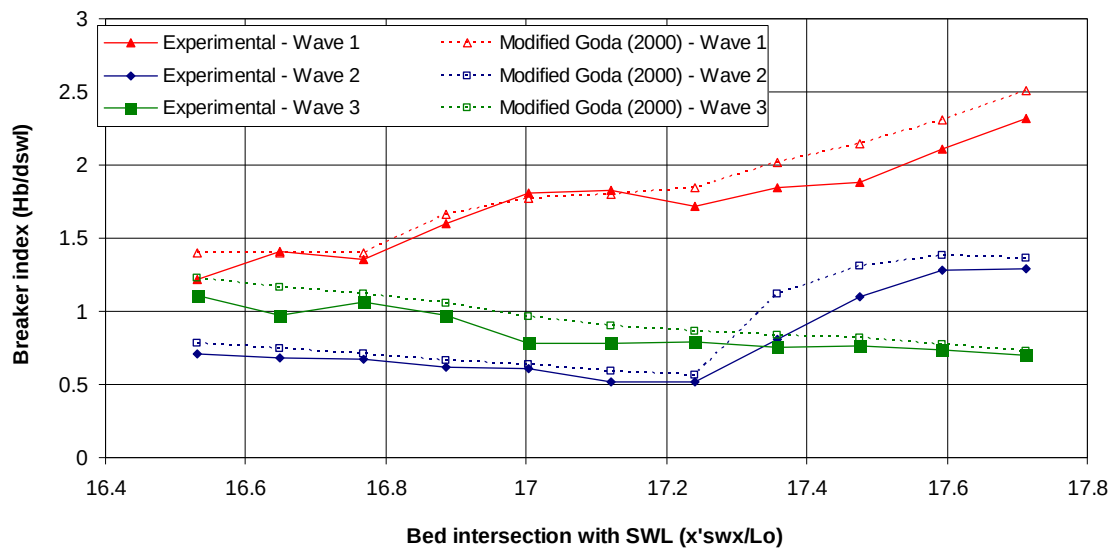


Figure 7-15 Observed H_b/d_{swl} for a C2N5r type wave group propagating onto a 10:1 sloped bed compared to values predicted by Goda (2000) using a variable coefficient A (Eqn. 7-3).

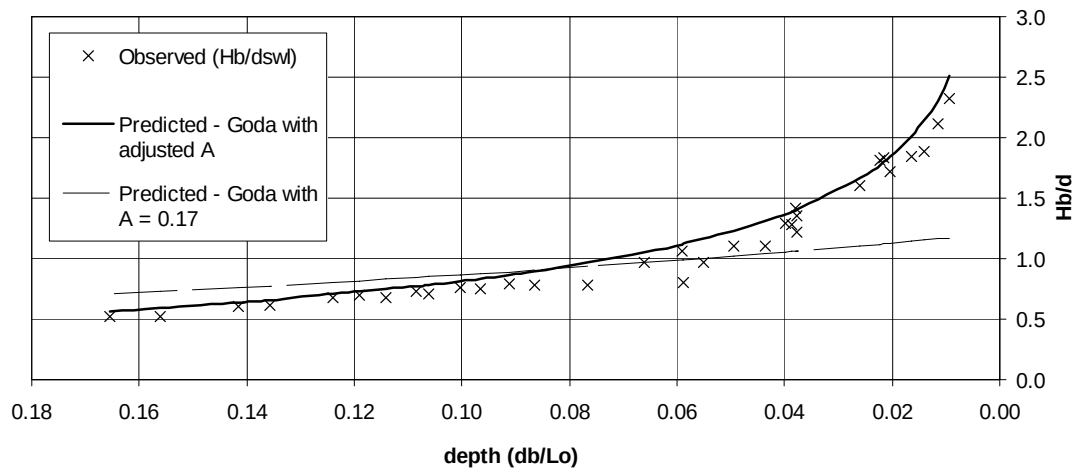


Figure 7-16 Observed H_b/d_{swl} as a function of depth for a C2N5r type wave group propagating onto a 10:1 sloped bed compared to values predicted by Goda (2000) using a fixed ($A = 0.17$) and variable coefficient A (Eqn 7-3).

While Eqn. 7-3 was derived for a single wave group structure on bed slopes between 5:1 and 20:1, the method may be compared to other wave group structures and across the range of bed slopes tested. Results are presented within Table 7-3 and show that for each case, breaking wave heights may be substantially (between 10 and 105% depending on group and bed slope) under-predicted using a constant value of A . Using the variable coefficient A , this under-prediction is reduced to less than 2.5% in all groups. Over-prediction of wave height was improved or remained constant (within 1%) for all cases except the flatter 20:1 bed slope, where over-prediction increased from

20% to 29%. These larger over-predictions however, occur in very shallow water and the absolute difference in height is relatively small. The standard deviation of differences between observed and predicted is also substantially reduced.

Table 7-3 Difference (%) between predicted maximum wave height and observed breaking height.

C2N5r 5:1					
<i>A</i>	0.17	0.17	0.17	0.17	Variable (Eqn. 7-3)
Depth	SWL	MWL	Effective	Maximum	SWL
Maximum	-49.8	-49.8	-	-	-44.0
Minimum	40.9	22.5	-	-	1.2
Mean	-16.2	-18.9	-	-	-20.2
Std. dev.	24.0	20.7	-	-	11.7

C2N5r 10:1					
<i>A</i>	0.17	0.17	0.17	0.17	Variable (Eqn. 7-3)
Depth	SWL	MWL	Effective	Maximum	SWL
Maximum	-28.7	-28.7	-27.8	-29.5	-28.1
Minimum	98.2	56.4	36.3	21.3	2.1
Mean	12.4	7.2	2.3	-7.2	-8.4
Std. dev.	35.3	26.8	19.2	14.9	5.9

C2N5r 20:1					
<i>A</i>	0.17	0.17	0.17	0.17	Variable (Eqn. 7-3)
Depth	SWL	MWL	Effective	Maximum	SWL
Maximum	-20.1	-19.3	-	-	-29.0
Minimum	10.7	11.7	-	-	2.5
Mean	-2.6	-1.8	-	-	-10.9
Std. dev.	8.6	8.5	-	-	7.4

C2N3r 10:1					
<i>A</i>	0.17	0.17	0.17	0.17	Variable (Eqn. 7-3)
Depth	SWL	MWL	Effective	Maximum	SWL
Maximum	-21.5	-20.4	-	-	-13.8
Minimum	38.9	43.3	-	-	-0.4
Mean	14.6	17.2	-	-	-8.4
Std. dev.	19.4	19.5	-	-	4.7

C2N5sb 10:1					
<i>A</i>	0.17	0.17	0.17	0.17	Variable (Eqn. 7-3)
Depth	SWL	MWL	Effective	Maximum	SWL
Maximum	-31.8	-31.9	-	-	-19.1
Minimum	105.5	45.9	-	-	-5.5
Mean	10.7	2.4	-	-	-11.6
Std. dev.	42.6	26.4	-	-	3.3

C3N5r 10:1					
<i>A</i>	0.17	0.17	0.17	0.17	Variable (Eqn. 7-3)
Depth	SWL	MWL	Effective	Maximum	SWL
Maximum	-29.8	-29.5	-	-	-30.8
Minimum	36.7	31.2	-	-	1.1
Mean	-3.8	-4.8	-	-	-12.8
Std. dev.	17.2	16.1	-	-	9.3

7.3.2.3 Maximum elevation

Having accurate information on maximum water levels ($z_{\max}$) at breaking and within the surfzone is, as discussed within Section 5.4.2.2, important for engineering design. A similar approach to that used to derive expressions for the coefficient, A as a function of relative depth for maximum and significant wave height is therefore used for the maximum water level $z_{\max}$.

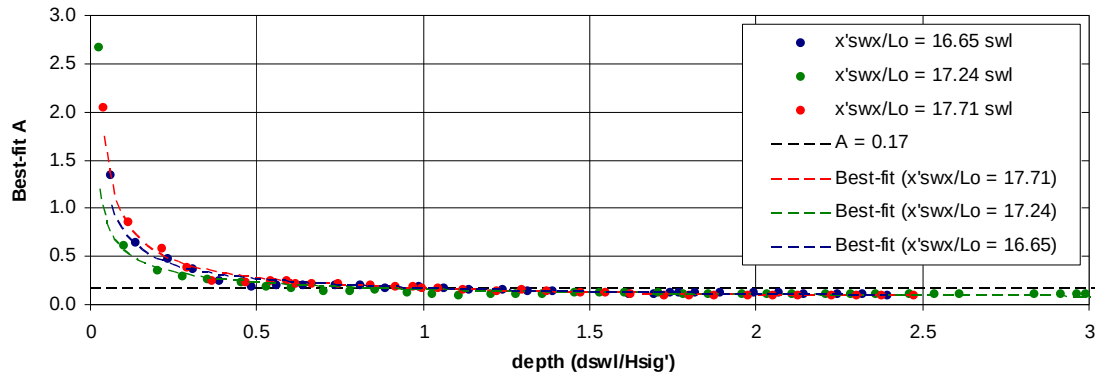


Figure 7-17 Required coefficient A for breaker index values predicted by Eqn. 5-2 to match observed $z_{\max}/d_{\text{swl}}$ for three example cases of a C2N5r type wave group propagating onto a 10:1 sloped bed.

Waves propagating onto slopes of 5:1 and 20:1 are similarly evaluated and an expression for the coefficient A is determined incorporating a bed slope parameter:

$$A_{Z_{\max}} = (-0.09m + 0.2) \left(\frac{d_{\text{swl}}}{H'_{m0}} \right)^{(-1.65m - 0.5)} \quad (7-4)$$

Of interest, the required value of A for $z_{\max}$ are below those required for $H_{\max}$ in deep water but increase to exceed those for $H_{\max}$ in the inner surf zone ($d/H_{\text{sig}}' < 0.3$). This implies that the crest amplitude (above still water level) exceeds $H_{\max}$ in the inner surf zone. This is explained by the contribution of set up, which increases the trough level above still water level, therefore giving a lower relative maximum wave height than $z_{\max}$. This is important as it illustrates the inappropriateness of simply using a proportion of the maximum wave height to determine maximum possible water level within the inner surf zone.

7.3.3 Implications of groupiness for coastal structural design

The design of nearshore structures such breakwaters and rock revetments are undertaken using standard design methods based on available material properties, structure geometry, tolerable damage levels and wave characteristics. Two such methods, presented within the Coastal Engineering Manual (USACE, 2003) and the Rock Manual (CIRIA, 2007), are based on the work of Hudson (1974) and van der Meer (1988). Summaries of these methods, used to determine rock diameter or mass for non-overtopped slopes (USACE, 2003), are presented within Appendix F.

Groupiness within the wave-train is of relevance to the design of nearshore structures. Several mechanisms found during this and previous studies are generated by groupiness. These include variation in breaking location (Section 6.3.2), breaker index (Section 7.3.1) and breaking type (Section 6.3.2) which are a function of the spatial phasing of group evolution as shoaling water is approached and of temporal changes in water level through the surf zone (Sections 6.3.4 and 7.3.1). The latter changes are induced by low frequency fluctuations in radiation stresses associated with wave groupiness. These mechanisms and their likely implications for the sizing of structural materials, as calculated according to the above design approaches, are investigated below.

7.3.3.1 *Breaking location*

Wave break point was found to vary depending on the spatial phasing of group evolution as shoaling water is approached (Section 6.3.2). This resulted, during certain scenarios, in waves breaking significantly seaward of guideline predictions. Examples of this are evident within Figure 6-16(B) and Figure 7-5(C), where the observed breaker depth indices of the most seaward-breaking waves are up to 30% below guidelines predicted values. This has implication in the selection of appropriate damage coefficient (K_d) values using the approach of Hudson (1974), with values differing, dependent on whether breaking or non-breaking conditions apply (Appendix F-1). K_d values for breaking waves given by the Shore Protection Manual (1984) (in USACE, 2003) are 50% of values assumed for non-breaking waves. Due to the inverse relation between K_d and rock mass, this results in median armour-rock mass for non-breaking waves being 50% of that required for breaking waves. Thus, if a breaking wave is defined as non-breaking, derived armour rock values may not be conservative.

7.3.3.2 Breaker index and definition of depth

Successful design of coastal structures within a depth-limited environment requires the accurate definition of the largest wave possible within water of a finite depth. The practitioner defines this height by first determining an appropriate breaker depth index (H_b/d_b) from design guidelines (i.e. USACE, 2003; Goda, 2000; Pullen et al., 2007). This breaker index is based on deep water wave steepness, bed slope and local water depth. The limiting wave height is then calculated according to the local water depth. Thus, appropriate definition of this local water depth becomes critical in accurate definition of maximum possible wave height.

Existing guidelines for breaker index derived from regular wave testing have typically used a depth based on still water level to derive values. This assumed depth is slightly greater than the actual *effective* depth due to the effect of set down around the break point for regular waves. This means that the derived breaker indices tend to slightly over-predict wave height resulting in slightly conservative values. However, irregular waves and wave groups contain waves of differing height and may break across a wide surf zone. Thus the local water depth becomes affected by factors such as set-up and temporal variations in water level induced by wave grouping (Section 6.3.4).

Present testing has shown that while observed breaker indices at the outer edge of the surf zone are well-or over-predicted by design guideline values, waves within the mid to inner surf zone, particularly those associated with *delayed-type shoaling* groups, exhibit breaker indices larger than those in the outer surf zone and larger than guideline-predicted values. The extent to which observed values exceed guideline predictions is dependent on group type, bed slope, position within the surf zone and the definition of depth. Observed values exceeded guideline predicted values by up to 100% within the inner surf zone if depth was assumed relative to still water level. While this discrepancy reduces if a more accurate, effective measure of depth is used, including a static or dynamic change in water level through the surf zone, observed values remain above guideline predicted values by up to 20%, indicating that factors other than effective water depth may influence the maximum wave height possible.

Components which may contribute to water level within the surf zone include both non-wave affected components such as tide and storm surge and wave-induced components such as set-up and temporal variations due to group-induced low-frequency motions. While some guidance is provided on calculation of a mean water level due to set-up (Eqn. 2-30, USACE, 2003) and in estimating the root-mean-squared value of surf-beat (Eqn. 2-34, Goda, 2000), incorporation of these factors still does not provide the maximum possible effective depth or the maximum possible wave height.

An alternative approach was suggested within Section 7.3.2 where, instead of the practitioner having to first calculate individual components of water level from empirical-based expressions and then using another empirical-based expression to calculate wave height, depth is assumed relative to still water level and all water level components are incorporated into a variable coefficient, A , which increases as a function of relative depth (or distance into the surf zone) and bed slope. This modified coefficient A is used within the Goda (2000) expression to define the breaker index for either significant or maximum wave height (Eqns. 7-2 and 7-3 respectively) with good agreement found with observed values (Figure 7-15). This differs from the standard approach of using a constant value or range of values for A , dependent on the characteristic wave height sought, but not adjusting A with relative depth, or distance into the surf zone.

Significant wave height index (H_{sig}/d) and the resultant wave height (H_{sig}) for a wave-train of steepness ($ak_0 = 0.15$) propagating onto a 10:1 sloped bed is calculated across the surf zone according to (i) Goda (2000), with depth based on still water level and $A = 0.12$ and (ii) with the coefficient, A , modified to incorporate group effects (Figure 7-18A and B). Differences in size of required armour rock across the surf zone using the different breaker index methods are determined the equations of Hudson (1974) (in USACE, 2003) (Figure 7-18C).

In both cases, required armour rock near the incipient break point and within the outer surf zone is similar in size. Within the surf zone, however, larger armour rock is required where wave height has been derived incorporating these group effects. Maximum difference in armour rock sizing occurs within the mid surf zone. This maximum difference is around 25% of the armour rock size required at incipient break

point. Maximum difference on a steeper, 5:1 bed slope is between 35% of armour rock size required at incipient break point and between 22% on a 20:1 slope. Based on this small difference between the 10:1 and 20:1 slope, flatter slopes than 20:1 are likely to exhibit similar differences. Differences in armour rock sizing appear to increase with lower steepness wave-trains and decrease with higher steepness wave-trains, although testing within the present study was limited to relatively steep wave-trains.

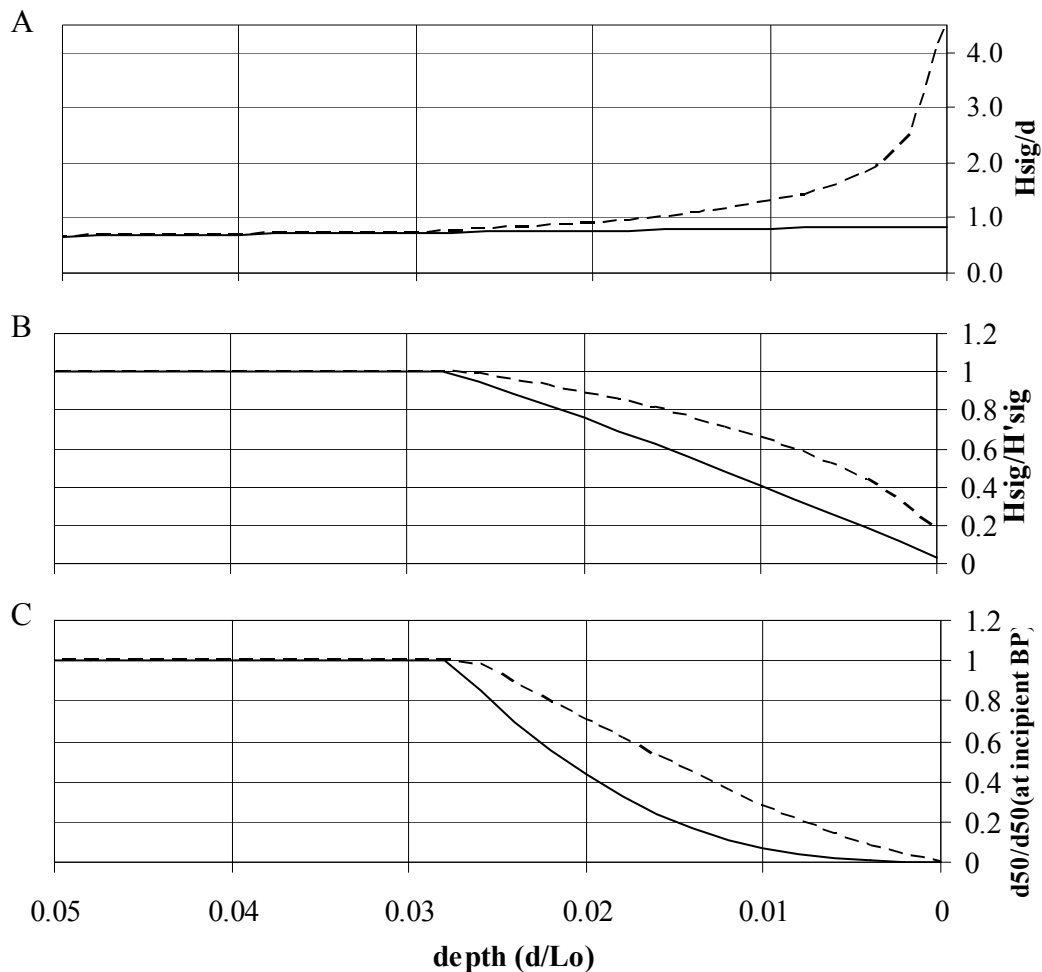


Figure 7-18 Breaker depth indices, significant wave height and diameter of required armour stone for a C2N5r type wave group propagating on to a 10:1 plane sloped bed. (A) compares breaker depth indices (H_{sig}/b) according to Goda (2000) with a fixed coefficient, $A = 0.12$ (—) and the Goda (2000) equation with modified coefficient to account for changes in water level within the surf zone (---). (B) shows the change in significant wave height compared as a function of deep water significant wave height according to both fixed and varying coefficient A (—) and (---). And (C) compares the nominal diameter (d_{n50}) of armour rock as a function of armour rock required at incipient break point ($d_{n50-incipient}$) using the equations of Hudson (USACE, 2003) based on wave heights with both fixed and varying coefficient A (—) and (---).

It is noted that using a depth based on mean water level to derive wave height using the original Goda (2000) formula with constant A, would result in larger heights and armour rock sizing and therefore lower differences between the two methods. However, as shown within Section 7.3.1.2, resultant heights still under-predict those observed.

While this expression must be tested more widely, using a greater variety of wave groups and bed slopes, before use for practical engineering application, it highlights possible flaws in our present design processes.

7.3.4 Errors and limitations

7.3.4.1 Wave height and breaker index

As discussed within Section 6.4.3, accuracy in defining wave height and the break point of individual waves is dependent on the temporal resolution of the imagery and the horizontal distance travelled by a wave form between successive frames. Errors in defining wave height near break point were calculated at $H_{\text{actual}} = H_{\text{observed}} + 0.42$ to -0.12 mm. Accuracy in defining break point is estimated at ± 5 mm with subsequent accuracy in defining breaking depth at ± 1 mm, ± 0.5 mm and ± 0.25 mm for the 5:1, 10:1 and 20:1 slopes respectively.

Error in defining the breaker index (H_b/d_b) is dependent on wave height, with breaker indices of smaller waves influenced to a greater extent by errors in wave height and depth. Based on the above error analysis, errors in H_b/d_b are ± 0.02 to 0.07 for the 5:1 sloped bed, ± 0.015 to 0.05 for the 10:1 sloped bed and ± 0.01 to 0.035 for the 20:1 sloped bed.

7.4 Summary and conclusions

This chapter has principally investigated the influence of groupiness on design of nearshore structures. Specifically, the applicability of standard design guideline formula used to predict breaker indices and wave height are assessed by comparison with data collected within the present study.

7.4.1 Comparing existing guideline predictions

Assuming a depth based on still water level, as typical for regular wave testing (Rattanapitikon and Vivattanasirisak, 2003), the largest waves breaking within the outer surf zone ($d/L_o > \sim 0.06$) were typically below design guidelines values, waves within the mid surf zone ($\sim 0.02 < d/L_o < \sim 0.06$) were close to, or occasionally exceeded guidelines values (by up to 20%) and waves breaking within the inner surf zone ($d/L_o < \sim 0.02$) substantially exceeded guideline values (by up to 100%). For all wave group types, on all bed slopes, each wave exceeds values predicted by design guides during some phase of evolution (i.e. during at least one bed position). Groups propagating on to steeper slopes exhibited larger variation in breaker index and difference from guidelines predicted values. This was similarly evident within the monochromatic testing with values on the lower 20:1 slope better predicted by guidelines.

These results suggest that:

- the current design guidelines, based on monochromatic wave testing and using a depth defined according to still water level, remain applicable and conservative for the largest waves that break at the outer edge of the surf zone.

However,

- these methods are not suitable for prediction of breaking wave heights which may occur within the mid to inner surf zone.

Furthermore, the spatial phasing of group evolution was found influential, with *delayed shoaling groups* exhibiting indices exceeding guidelines within the mid surf zone, while the *early shoaling groups* tended to remain below design guides. In both cases however, smaller waves within the inner surf zone exceeded guideline values.

However, the definition of depth, which is of minor importance in regular wave testing due to the near singular breaking position, becomes critical for waves breaking over a wide surf zone. This is due to the spatially and temporally varying water levels which occur over the width of a surf zone. While the definition of depth makes little difference in the derived breaker index for the largest wave, breaking in the outer surf zone, it causes significant variation in smaller waves breaking within the mid to inner surf zone. Similarly, while the guidelines predict reasonably the breaker index in the outer surf

zone, they remain non-conservative, at least during some phase of group evolution, within the inner surf zone, regardless of definition of depth (Table 7-4).

Table 7-4 Maximum percentage difference in observed and predicted breaker index using different definitions of depth.

Observed index depth based on:	Maximum % difference (Goda, 2000)	Maximum % difference (USACE, 2003)
Still water level	98	87
Mean water level	57	48
Effective depth	37	27
Maximum depth	25	20

This implies that for design in the presence of wave groups, even if the contribution of set up and/or temporal variation associated with long waves are calculated and included within the assumed depth, under-prediction of breaking wave height by up to 20% may occur. Furthermore, while characteristics of temporal variations in water level were available within this study, standard methods for prediction of such fluctuations at specific locations within the surf zone are lacking with the Coastal Engineering Manual (USACE, 2003) stating that *practical questions of how to predict infragravity waves and design for their effects remain un-answered.*

7.4.2 Revised index predictors

A revised breaker index model is therefore proposed which encapsulates observed data using a depth based on still water level. This revision is achieved by modification of the coefficient A , used within the Goda (2000) breaker index formula (Eqn. 5-2). For the maximum and significant wave height, and the maximum surface elevation, new expressions were found for A to match observed values using a depth based on still water level. These values therefore implicitly include variations in water surface elevation within the surf zone, such as set-up and low frequency fluctuations, and provide an upper envelope of possible wave height. An upper envelope value is essential for engineering or structural design within the nearshore, where the maximum possible force to be placed on structures must be known.

Overall, the modified coefficient A may be described by the following expression:

$$A_x = (C_1 m + C_2) \left(\frac{d_{swl}}{H'_{m0}} \right)^{(C_3 m + C_4)} \quad (7-5)$$

Where C_1 - C_4 are coefficient dependent on the wave characteristic sought (Table 7-5).

Table 7-5 Coefficients used to determine the coefficient A as a function of relative water depth for the maximum and significant wave heights ($H_{\max}$, H_{sig}) and the maximum surface elevation ($z_{\max}$).

x	H_{sig}	$H_{\max}$	$z_{\max}$
C_1	-0.24	-0.12	-0.09
C_2	0.17	0.25	0.2
C_2	-1.65	-1.65	-1.65
C_4	-0.42	-0.35	-0.5

Figure 7-19 depicts the variation in the coefficient A as a function of relative depth on a plane 10:1 slope for the maximum and significant wave heights ($H_{\max}$, H_{sig}) and the maximum surface elevation ($z_{\max}$). The required value of A for $z_{\max}$, while originally below those required for $H_{\max}$ in deep water, increases to exceed those for $H_{\max}$ in the inner surf zone ($d/H_{\text{sig}}' < 0.3$). This occurs as set up increases the trough level above still water level, giving a lower relative $H_{\max}$ than $z_{\max}$. This is important as it illustrates the non-suitability of simply using proportion of the maximum wave height to determine maximum possible water level within the inner surf zone.

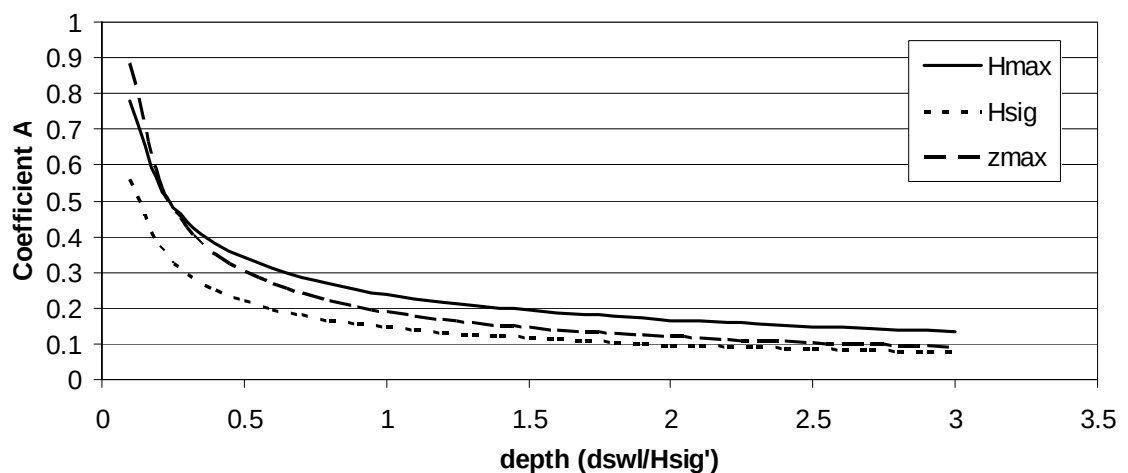


Figure 7-19 Example of the coefficient A as a function of relative depth for the maximum and significant wave heights ($H_{\max}$, H_{sig}) and the maximum surface elevation ($z_{\max}$) on a plane 10:1 slope.

While the revised expressions for A were derived for a single wave group structure on bed slopes between 5:1 and 20:1, predicted breaking wave height using the both the constant A value with varying definitions of depth and the modified value were compared to observed height for other wave group structures and across the range of bed slopes tested. Results showed that for each case, wave heights may be substantially (between 10 and 105% depending on group type, bed slope and definition of depth) under-predicted using a constant value of A , although the use of more accurate definitions of depth incorporating mean or temporal changes in water level reduced under-prediction. Using the variable coefficient A , this under-prediction was reduced to less than 2.5% for all cases. Over-prediction of wave height was improved or remained constant (within 1%) for all cases except the flatter 20:1 bed slope, where over-prediction increased from 20% to 29%. However, these larger over-predictions occur in very shallow water and the absolute difference in height is relatively small. The standard deviation of differences between observed and predicted was also substantially reduced.

While these expressions are useful in that they implicitly incorporate group effects and influences on water level the use of depth based on still water level tends to zero at the intersection of a sloping bed and still water. In reality, water depth and therefore wave height remain finite due to set-up effects. The above expressions therefore tend to infinity in very shallow depth which is clearly unfeasible. The expressions are therefore assumed valid only for the limits of the data collected, i.e. $d_{swl}/H'_{sig} > 0.02$.

7.4.3 Implications of groupiness for coastal structural design

Groupiness within a wave-train has been shown within the present study to result in variation in breaking location, breaker index and breaking type as the spatial phasing of group evolution is altered.

These have implications for the design of nearshore structures which rely on accurate measures of breaking wave height and position. The methods of Hudson (1978), for example, contains a wave height cubed relation to armour rock sizing indicating that small discrepancies in wave height may result in large discrepancy in required armour rock sizing. Additionally, defining whether a structure is subject to breaking or non-

breaking waves affects the selection of an appropriate damage coefficient, resulting in armour rock sizing differences of up to 50%.

Groupiness has been shown to induce temporal variation in water level through the surf zone, induced by low frequency fluctuations in radiation stresses. This results in larger effective water depths occurring periodically and therefore the occurrence of larger breaking waves than predicted using either still or mean water level. The magnitude of these group-associated water-level fluctuations increases with distance into the surf zone, resulting in larger discrepancies within shallow water.

The implications of these temporal changes in water level on design are negligible near incipient break point or within the outer surf zone, but become more manifest within the mid to inner surf zone. Required rock sizing decreases with distance from the incipient break point, due to dissipation of wave energy regardless of method used to calculate wave height or rock sizing. However, incorporation of group-induced variations in water level within breaker index formulae means that design wave height decreases more slowly than predicted using depth based on still or mean water level. This results in larger required armour rock with the discrepancy ranging from 35% of rock size at incipient break point on a 5:1 slope to 25% on flatter 10:1 and 20:1 slopes.

So while results show that existing methods for wave height assessment and armour rock sizing near the outer surf zone remain valid and possibly conservative, rock sizing should not be decreased within the surf zone as rapidly as existing guidelines may predict. Caution should therefore be used in the design of structures wholly located within the mid to inner surf zone as water levels and consequently wave height may exceed heights predicted by design guides.

Revision of existing design guidelines is recommended to account for the effects of wave groupiness, particularly within the mid to inner surf zone. Resultant guideline values should use a depth based on still water level for simplicity and robustness, with breaker index values adjusted as a function of position within the surf zone to account for higher water levels, depth and wave heights possible under grouped conditions. Design wave conditions must therefore be known, even for structures within depth-

limited environments, to enable the structures' relative depth, or position within the surf zone, to be determined.

While this study presents such expressions for $H_{\max}$, H_{sig} and $z_{\max}$, derivation of the expressions was limited to a group of specific structure and steepness and to relatively steep ($\geq 1:20$), plane bed slopes and as such is not recommended for use in design. More extensive testing should be undertaken to confirm or modify the presented expressions.

PART 2: WAVES ON EMERGED PLATFORMS

Rock platforms are a common coastal feature (Stephenson and Thornton, 2005) and therefore are the second coastal landform considered during this study.

The following chapter provides an abbreviated description of work undertaken to quantify the hazard of wave overtopping of emerged platforms, particularly the contribution of wave groupiness to hazard. In order to quantify group effects, a method of prediction of hazardous waves was first derived based on scale laboratory testing of typical emerged rock platforms within the Sydney Region. Detail on the development of this method is abbreviated for the purposes of this thesis, with a more complete description of the project provided within Shand et al. (2009).

Chapter 8 Wave group behavior at emerged platforms

8.1 Introduction

Waves breaking on emerged platforms differ to those breaking on a plane slope in that a very rapid transformation is forced on the wave-form by the presence of these near vertical-faced structures. The platforms may exist in water of intermediate water depth, where only shoaling transformations are occurring, or within the breaker or surf zone, where a host of other processes are occurring. Due to the rapid change in depth at an emerged platform, much higher energy may reach the backshore due to the minimal dissipation by breaking and turbulence. This higher energy is important as it increases the hazard to users of these environments, including fishermen and general pedestrians. This is demonstrated by the high numbers of rock fishing fatalities which occur annually in New South Wales, Australia, principally due to unexpected wave impact and subsequent drowning. While the interaction of waves and emerged platforms have received far less attention within the literature than waves acting on plane or gentle slopes, nonlinear interactions and local energy convergences within wave groups may be similarly expected to influence overtopping processes.

This chapter details the background to the study problem including both physical and human aspects, describes a fieldwork program undertaken to allow representative platform geometries to be produced, and describes the laboratory study, including laboratory set-up, model construction, data acquisition and analysis techniques and the testing program. Results of the laboratory study are presented and analysed to produce prediction schemes for the instability hazard using environmental and physical site conditions. The influences of distance from the platform edge and groupiness are addressed and a site-specific example is provided to demonstrate the practical useage of these prediction schemes.

8.1.1 Rock platform form and origin

Stephenson and Thornton (2005) review the morphology and processes of Australian rock coasts. They summarise past literature which has tended to group rock coasts into morphologies based on either platform elevation (above high water level, between high

and low water level and below low water) or slope (gently sloping, near horizontal or plunging directly into the sea). The actual development mechanism of platforms remains the subject of debate with arguments for marine (wave and water level), biological and sub-aerial (weathering) processes dominating development (Stephenson, 2000; Twidale et al., 2005 and Trenhaile and Porter, 2007).

Shore platforms on the Sydney and Illawarra coasts tend to exhibit elevations around the high water level and are generally backed by steep, or in some cases, overhanging cliffs of low to moderate height. While in some locations cliff-fall debris has been washed off the platforms or completely broken down, in other areas moderate to very large boulders lie at the base of the backing cliffs.

8.1.2 Wave – platform interaction

Studies into the relationships between waves and rock platforms have so far been limited. One of the few studies that investigated wave processes on top of rock platforms was a 2005 review of Australian rock coasts by Stephenson and Thornton. As part of this study, wave recorders, presumably water depth gauges, were placed at the front crest and rear of a 35 m wide rock platform at Marengo, Victoria. Results showed very little dissipation of wave energy occurring across the platform with wave energy at the back of the shelf being typically 67-90% of that near the front. This implies that once a rock platform is overtopped, energy within the wave bore is conserved for some distance.

While investigations of wave overtopping of natural rock platforms have so far been limited, analogies may be drawn between these natural processes and waves impacting and overtopping vertical seawalls, breakwaters and sea-dikes, for which many more studies have been undertaken. The US Federal Emergency Management Agency (FEMA) overtopping guidelines (Broker et al., 2005) divides overtopping at coastal structures into: *wave-form overtopping* where the entire form overtops as green water; *run-up wedge* where the upper portion of run-up overtops the structure (presumably still as green water); *wave splash* where overtopping occurs as aerated (white water) splash and *wave spray* where highly aerated water (or water-filled air) overtop.

The EurOtop Wave Overtopping Manual (Pullen et al., 2007) divides the overtopping regime at vertical and near vertical seawalls into non-impulsive (surging) or impulsive (plunging) conditions. Non-impulsive conditions are generally achieved when wave height is relatively small compared to local water depth and the wave steepness is low. Waves run-up and over the crest producing green water overtopping. Impulsive conditions generally occur when waves are larger compared to the local water depth. The waves may shoal on the approach and then break onto or against the structure. Overtopping under these conditions is characterised by a violent uprush jet of (often aerated) water.

To distinguish between these two overtopping regimes an impulsiveness parameter, h^* , is defined (Eqn. 8-1) based on depth at the toe of the wall (h_s) and incident wave conditions (H_{m0} and $T_{m-1,0}$). Parameter values of $h^* > 0.3$ indicate non-impulsive conditions dominating, with impulsive conditions occurring when $h^* < 0.2$. Between these values, both conditions could occur and so should be evaluated separately. Bruce et al., (2001) relates this impulsive parameter to the vertical ‘throw’ speed of overtopping jets at a vertical seawall crest. Velocities for non-impulsive (green water) overtopping conditions were found to be 2 to 2.5 times the inshore wave celerity, c_i , and 5 to 7 times c_i for impulsive conditions. No further guidance, however, is given for horizontal flow velocities or depths on a flat plane behind such a seawall.

$$h^* = 1.35 \frac{d_t}{H_{m0}} \frac{2\pi h_s}{g T_{m-1,0}^2} \quad (8-1)$$

The majority of studies, including recent investigations by Grune et al. (2004) and Bay et al. (2004), have largely focussed on mean overtopping rates ($m^3/s/m$). This is the average amount of overtopping flow per linear meter in a defined time-period and is a much simpler parameter to measure than flow velocity and depth, particularly in the field. These studies and others have been incorporated into guidelines such as the *Coastal Engineering Manual* (USACE, 2006), the *Overtopping of seawalls, design and assessment manual* (HR Wallingford, 1999) and the *EurOtop Manual* (Pullen et al., 2007). These guidelines relate environmental parameters such as offshore wave height and water level to structural properties such as crest height and face slope, to give overtopping rates. These rates are typically presented in terms of mean discharge per linear meter ($m^3/s/m$).

However, while mean discharges may be relatively low, the individual overtopping events may be much larger (up to 100 times - USACE, 2006) due to the inherently random nature of overtopping, both in time and in volume (Pullen et al., 2007). This is especially relevant in deep water environments where no depth-limited breaking has occurred. Studies by Cox and Machemehl (1996), Bruce et al. (2002), van Gent (2002) and Schuttrumpf et al. (2002) have focussed on the volumes, depths and velocities of individual overtopping events, a measurement much more relevant in defining the safety of persons on or near the crest of such structures. These studies, however, have focussed almost exclusively on overtopping of sea dikes with much flatter front slopes than rock platforms.

8.1.3 Wave group effects

Tsai et al., (2004) carried out a detailed analysis of the mechanisms responsible for generating ‘mad-dog’ waves in Taiwan. These waves are so called because they apparently strike without forewarning and are responsible for the drownings of several recreational fishermen per year from rocky shores or breakwaters. Tsai et al. collected field data, including water level records using an ultrasonic wave gauge and video recordings, during five incidences of large overtopping events that swept fishermen off rock platforms and breakwaters. By analysing the water-level time series, recorded just offshore from the subject sites. They found that the wave record during two of the incidences had high groupiness characteristics and that another incident, which had moderate groupiness characteristics within the record, was caused by a group of three waves of increasing height with nearly equal period. This implied that despite the record showing no mean trend of groupiness, the actual waves responsible for the overtopping were grouped. His findings supported those of Chin (2002) who suggested that nonlinear evolution of wave groups is the most likely mechanism for generating these large and dangerous overtopping events.

Tsai et al., produced a set of possible conditions contributing to the occurrence of abnormally large overtopping events. These included:

- 2) that the prevailing incoming waves are significantly high;
- 3) that the water depth fronting the structure is deep enough not to significantly effect the incident waves and that the bottom slope is steep;

- 4) that wave overtopping occurs in the form of either green water/pulsating or impulse/impacting;
- 5) the groupiness of the incoming large waves may be evident; or, with the right wave direction, a group of large waves with equal wave periods occur, even though the mean groupiness properties of the wave record may not be high;
- 6) the wave group responsible for forming a mad-dog wave may contain a large wave, close to its breaking stage, prior to striking the coastal structure/cliff;
- 7) a near normal wave direction to the structure/cliff is a favourable condition for the incident.

The large, steep waves described by condition (5) appear similar to the locally high and steep waves observed by Song and Banner (2002) and Banner and Peirson (2007) which occur during certain stages of nonlinear wave group evolution.

8.1.4 Human factors

8.1.4.1 Personal stability under flow and wave impact

Pioneering work by Foster and Cox (1973) investigated the stability of a group of children aged between 9½ and 13 when exposed to flows within a controlled laboratory environment. From this work, a set of curves were produced illustrating the limits of safety as a function of water depth and velocity for both standing and seated positions. Results, while likely to be substantially lower than for adults, provide a useful lower boundary for stability. General conclusions were that relatively low flow depths (<0.3 m) may be unsafe at high velocities (i.e. greater than around 1.5m/s) and that stability is lower in a seated position than a standing position. This last conclusion is important as it infers that once footing has been lost, stability is further reduced and the likelihood of a person recovering footing is low.

Since this early report, a number of additional studies have been carried out both in Australia and overseas investigating the stability of a range of people, including trained safety workers, under a range of flow conditions. These are summarised and additional test results presented within Cox et al., (2004). Cox et al. find that the safety of healthy adults is near assured at depth * velocity ($D*V$) values less than $0.6 \text{ m}^2/\text{s}$ and that many

can withstand flow conditions up to $1.0 \text{ m}^2/\text{s}$. However, children, particularly those with height * mass indices below 50, have lower tolerances; with some unable to withstand flows less than $0.4 \text{ m}^2/\text{s}$. This is the critical flow value presented in flood flow guidelines (O'Loughlin and Robinson, 1997).

Other studies such as Ramsbottom et al., (2004), investigating flood risks during relatively steady flows, reported greater velocity-depth limits before safety is compromised (Figure 8-1). This suggests that $D*V$ values of between $0.25 \text{ m}^2/\text{s}$ and $0.6 \text{ m}^2/\text{s}$ for all, $0.6 \text{ m}^2/\text{s}$ to $1.25 \text{ m}^2/\text{s}$ for most and $1 \text{ m}^2/\text{s}$ to $2.5 \text{ m}^2/\text{s}$ for some such as experienced and well equipped persons. While these limits were not established for pulsating flows, as occur during wave overtopping, they suggest that in some situations, higher tolerance limits may apply.

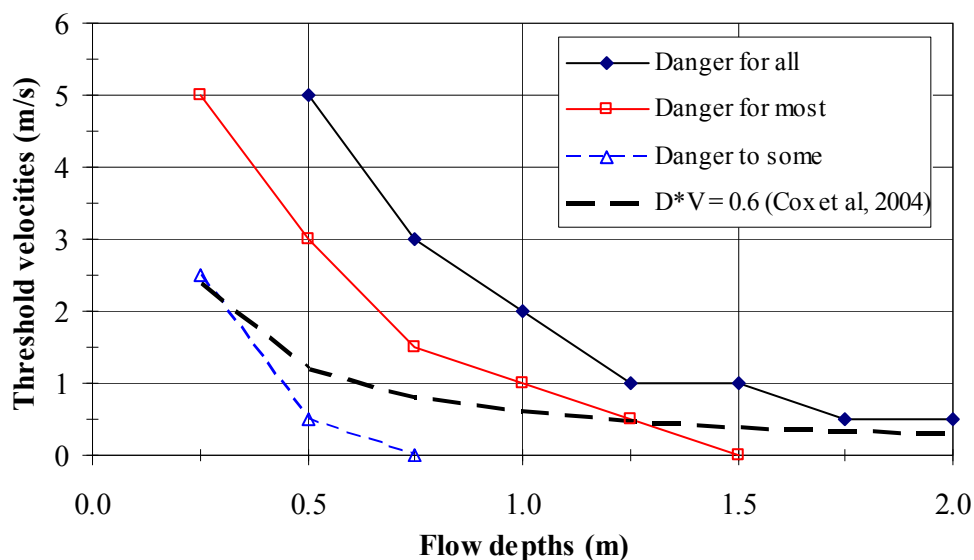


Figure 8-1 Recommended velocity-depth limits adapted from Ramsbottom et al. (2004) and the $D*V = 0.6$ threshold value of Cox et al. (2004).

Other studies, particularly in coastal environments where waves are the primary cause of flow, have produced safety guidelines based on overtopping volumes. Such guidelines are based either on tolerable mean discharges per linear metre ($\text{m}^3/\text{m/s}$), i.e. USACE, 2003 (refer Table 2-2) or tolerable peak overtopping volumes per linear metre (l/m , where l = total overtopping volume of a singular event in litres) for a range of safety criteria (i.e. Pullen et al., 2007). These peak volume guidelines suggest that maximum volumes of up to 1000-2000 l/m may be tolerable for trained and safety

equipped staff on a wide crested dyke or equivalent, though maximum volumes of only 100 l/m are tolerable for overtopping at a vertical wall. Clearly, tolerable overtopping rates for rock fishermen on a natural rock platform lie somewhere between these criteria.

Field studies of overtopping at the Zeebrugge Breakwater by Geeraerts et al. (2003) showed that during even moderate wave conditions (significant offshore wave height of 3.6 m with mean overtopping rates of 0.6 l/s/m), forces on instrumented dummies of up to 8.8 kN were measured during the impact of singular, low-probability overtopping events. Endoh and Takahashi (1994) considered an impact as low as 140 N leading to loss of stability. The values recorded by Geeraerts et al. (2003) are significantly in excess of this and indicate that even during moderate overtopping conditions, very large singular events may periodically occur.

8.1.4.2 Fatality assessment

Reports by Jones (2003) and the Interdepartmental Committee on Water Safety (IDCWS, 1994) provide comprehensive assessments of rock fishing related fatalities in NSW and the Greater Sydney coastline between 1969 and 2000. Fatalities over this period average 7-8 deaths per year with the NSW coroner identifying rock fishing as having the highest fatality rate of any sport in the state. These reports are supplemented by others including a 1996 report by Davis on the hazards and socio-economic impacts of rock fishing on New Zealand's Auckland West Coast (Davis, 1996) and Tsai et al. (2004) who gives a similar number of yearly fatalities by recreational rock fishermen in Taiwan.

The Jones report, in particular, includes a range of statistics relating recent fatalities to various environmental parameters including offshore wave and tidal conditions at the time of incident. The statistics showed that, in almost all cases, the cause of the victims entering the water was through unanticipated wave impact (94%). Additionally, while 55% of fatalities occurred during offshore wave conditions greater than 2 m, 45% occurred during 1-2 m offshore wave conditions, indicating that even during relatively benign conditions there remains a risk of large waves overtopping rock platforms.

While the largest proportion of fatalities occurred during the more frequent mid-tidal levels, a bias of fatalities during higher water levels was also evident. This relationship supports the premise of wave impact being the primary cause of fishermen entering the water as a higher water level allows larger waves to reach the platform and reduces the relative freeboard of the platform above the still water level, allowing greater wave overtopping.

While the Jones (2003) report does not include wave period statistics for the time of fatality, a separate paper by Evans (2004) investigates the effect of period on fatalities. The study finds that the risk of fatality is doubled when peak period (TP1) is raised from below 8s to 10-12s and doubled again when raised to above 12s. These results imply significantly increased risk in the presence of long period swells over shorter period local seas.

8.2 Study approach

The study has been undertaken in three phases allowing evaluation of the critical wave and water level conditions that lead to significant, and potentially hazardous, overtopping of rock platforms:

1. *Characterisation of rock shelves in the Sydney Region*

While it has been impractical to complete detailed hazard studies for every fishing site along the NSW coast within a short time frame, a set of representative characteristics for NSW rock shelves have been determined.

Jones (2003) found that fatalities from rock fishing accidents were widespread along the NSW coast. The initial phase of the investigation has been to undertake limited surveying of popular rock fishing sites where fatalities have occurred. For this study, available bathymetric information has been used to characterise offshore conditions and rock platform geometries determined by GPS, laser level and satellite and aerial photography images at representative rock fishing sites.

The outcome of this phase of the investigation has been a limited set of representative geometries which have been used in the detailed physical model investigation.

2. *Physical model investigation*

The behaviour of waves breaking at the coast is extremely complex and present numerical approaches for determining group breaking behaviour are not effective. Field studies are subject to vandalism and the vagaries of weather and very long project time frames would be required to obtain definitive results. Therefore, a physical laboratory investigation has been undertaken using well-established coastal engineering scaling principles.

The novel aspect of the physical investigation is the use of characteristically repetitive wave groups for laboratory testing. This enables careful examination of the effects of wave group development as the group approaches the shore.

For the representative rock shelf geometries (as determined during the first phase of this study), testing is undertaken for a range of tide levels and wave group structures. From these sets of results, a general characterisation of the depths and speeds of water flow across the rock shelves is developed for subsequent analysis and reporting.

3. *Analysis and reporting*

Pioneering work at the Water Research Laboratory 40 years ago determined the depth and water speed conditions that would lead to people being swept off causeways during floods (Foster and Cox, 1973). The results of these studies are now incorporated within international flood risk assessment procedures.

The results of our physical investigation have been assessed within this context to identify those conditions on rock shelves that are likely to lead to people losing their footing. At this point in the investigation, conditions during which people are at risk of being washed from rocks can be related to the offshore wave conditions and the influence of wave groupiness characterised.

8.2.1 Study limitations

Several limitations exist within the present study. These relate to components of both the field and laboratory investigations and include:

- Rock platforms in the Greater Sydney Region surveyed during the study were selected on the basis of having high usage for recreational fishing and being

sited in dangerous locations, either due to high numbers of fatalities in recent years or extreme exposure to wave energy. While a representative geometry was determined from these surveys and used for model investigations, it is not specific to any one site and as such, deviations from model results are likely as platform geometries change from location to location.

- Only two-dimensional aspects of wave breaking and overtopping were examined during model investigations. Within the field, waves may approach the shoreline at oblique angles or may contain three-dimensional components due to either nearshore seabed conditions or wave-wave interactions. These features have not been taken into account during the present study and may significantly change overtopping properties.
- Scale effects within the laboratory resulting from incorrect model simulation of the wave-platform interaction compared to full prototype scale. This may be due to heightened viscous and surface tension forces at model scale. These effects are discussed more fully within the report and quantified where possible. Overall, they are not expected to significantly influence results.
- Model errors including correct representation the seabed and platform geometry, correct simulation of incident waves and effects of the flume sidewall and wave paddle. Additionally, errors within the various instrumentation types themselves must be quantified. These, like scale effects, are further discussed within the text and overall are not considered to have significantly influenced results.
- Wind effects, which have been shown to contribute significantly to wave breaking characteristics and to overtopping flows at low volumes, have been omitted. This has been for a variety of reasons including the relatively low influence of wind on the more dangerous green water overtopping events, the likelihood that fishermen aren't on platforms during very strong onshore wind conditions and the difficulty of modelling this component.

8.3 Field investigations

A field investigation was initiated to assess the geometry of rock platforms in the greater Sydney region. These allowed a typical geometry to be determined for laboratory investigation. The field investigation was undertaken between 21 and 22 April 2008 to gather data on rock platform geometry along the Sydney coastline. Sites at 11 locations between Avalon and Cape Banks in Sydney were visited (Figure 8-2 and

Figure 8-3) at mid to low tide and a representative cross-section of the platform and backshore at each site obtained. Further details of the field study including specifications and accuracies of survey equipment are provided within Shand et al. (2009).



Figure 8-2 Platform survey sites on Sydney's Northern Beaches. Note: AR denotes the existence of an Angel Ring. Image: Google Earth, 2008.

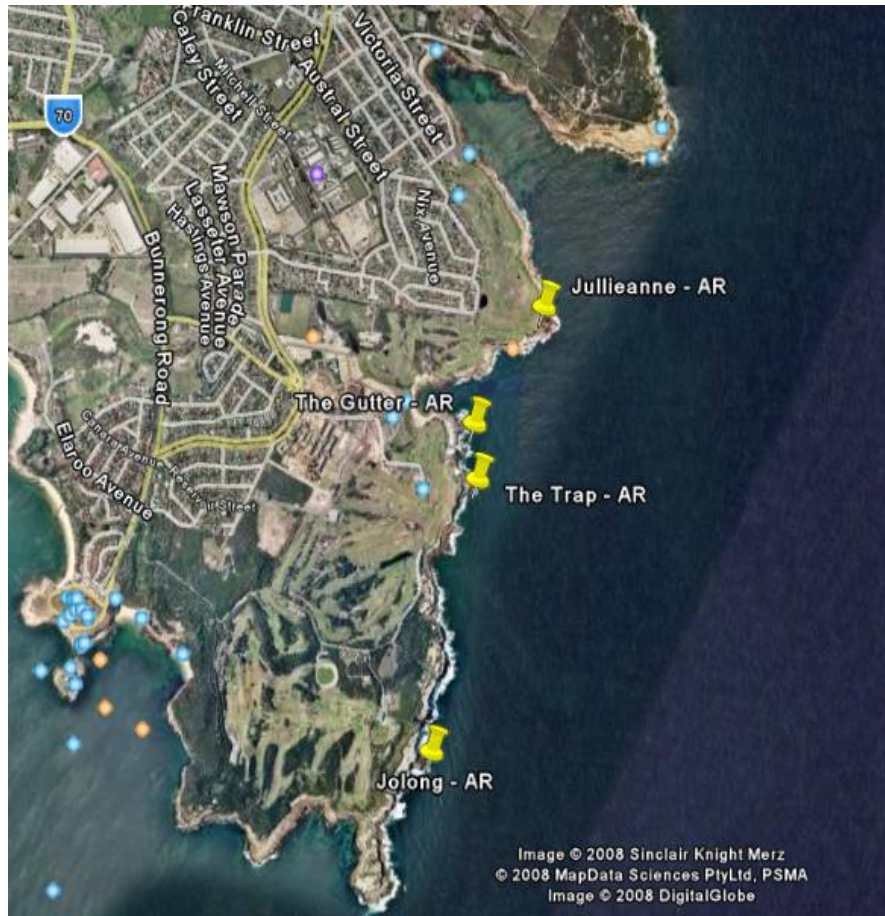


Figure 8-3 Platform survey sites on Sydney's Sydney's Eastern Suburbs. Note: AR denotes the existence of an Angel Ring. Image: Google Earth, 2008.

A number of photographs illustrating observations of the field study are included within Appendix E-1. These photographs illustrate both green water and white water overtopping at popular fishing locations during relatively benign offshore wave conditions (significant wave height at the Sydney offshore wave buoy of 1.9 – 2.0 m, with significant wave period of 7.5 – 8 s and peak swell period of 10.3 s).

8.3.1 Results

Surveyed geometries have been horizontally adjusted to the seaward platform extent (crest) and are presented in Figure 8-4. The vertical datum is Australian Height Datum (AHD71), which approximates mean sea level. Frequently used tidal levels on the Sydney coastline are presented with respect to NSW Maritime Services Board hydrographic datum (0 m Fort Dension tidal gauge 1954) and AHD71 within Table 8-1. Figure 8-4 shows platform widths typically in the order of 20 to 50 m, with some wider platforms occurring due to specific site conditions. Backing cliffs range from near

vertical to around 1(H):1(V) and from 5 to 50 m high. The platforms are typically near horizontal and range in elevation from 1.1 to 2.3 m AHD71 (0.62 to 1.82 m relative to mean high water level). Two platforms, which had lower seaward crest levels of 0.3 to 0.5 m AHD71, exhibited significantly more height variation across the shelf. The less horizontal nature of these platforms and lower seaward crest level is thought to be reflective of local, site-specific conditions such as geological faulting and bedding.

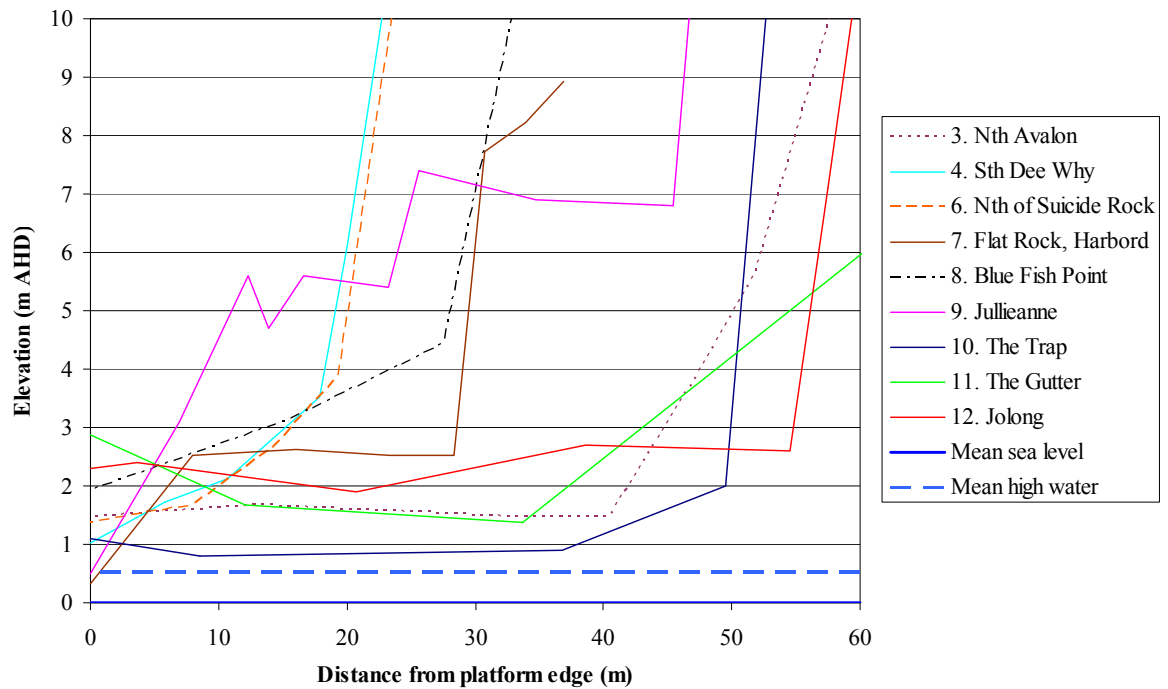


Figure 8-4 Summary of surveyed rock platform geometries.

Table 8-1 Frequently used tidal levels at Sydney, Australia.

Tidal level description	Level (m Ft Denison Datum)	Level (m AHD)
Highest Astronomical Tide	2.0	1.075
Mean High Water Spring	1.56	0.635
Mean High Water Level	1.44	0.515
Mean High Water Neap	1.32	0.395
Mean Sea Level Standard	0.895	-0.003
Mean Low Water Neap	0.45	-0.475
Mean Low Water Spring	0.245	-0.68
Indian Springs Low Water	0	-0.925

8.3.2 Recommendations

From these surveys and existing literature on general NSW shore platform morphology together with existing bathymetric data, an idealised platform for model testing was designed based on averaging critical geometric properties. These include offshore slope (s1), platform face slope (s2), platform toe depth below mean sea level (h), platform elevation above mean sea level (Rc) and platform width (w).

Whilst the toe depth and front face slope of the platforms could not be measured directly, the front face slope was inferred from the slope of the backing cliffs. This makes the assumption that the cliffs were originally formed in an emerged environment, with the portion of slope now beneath sea level subjected to the same processes as the emergent cliff is today. The toe depth is found by extrapolating the offshore slope from the most landward contour shown on the bathymetric charts and the slope of the front platform face until an intersection is found. Figure 8-5 illustrates the relevant geometric parameters with the averaged values presented in Table 8-2.

Table 8-2 Parameter values for generating representative model bathymetry.

Parameter	Units	Minimum	Maximum	Averaged
s1	degrees	1.5	11	6
h ¹	m	2	5	3
Rc ¹	m	0.4	2.5	1.15
s2	degrees	27	70	58
w	m	15	55	30

¹ with respect to AHD71

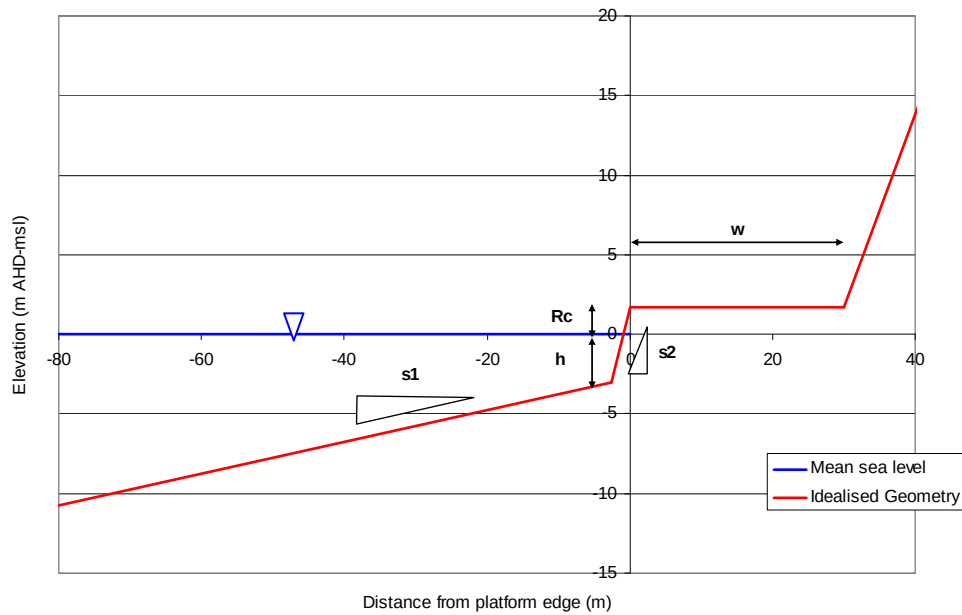


Figure 8-5 Relevant geometric parameters for an idealised rock platform.

8.4 Laboratory study

8.4.1 Aims

Aims of the laboratory study are to provide depths and velocities of overtopping flow over a characteristic rock platform under predetermined water level and wave conditions including wave group effects.

8.4.2 Testing Facilities

Laboratory experimentation was carried out in the 0.6 m wide wave flume detailed within Section 3.2, although differently from the earlier study, a piston-type wavemaker is used to generate waves. This allows generation of larger and longer period waves than is possible with the cantilevered plate used previously, although accuracy in the generated wave form is reduced. This was deemed necessary due to scale requirements of the current study, described further within the following section. The paddle is fronted by a synthetic damping mat to reduce high frequency secondary waves and turbulence and is driven by a servo-controlled actuator. This set-up allows precise waves and wave group structures to be generated with excellent short and long-term stability. A schematic diagram of the laboratory set-up is presented within Figure 8-6.

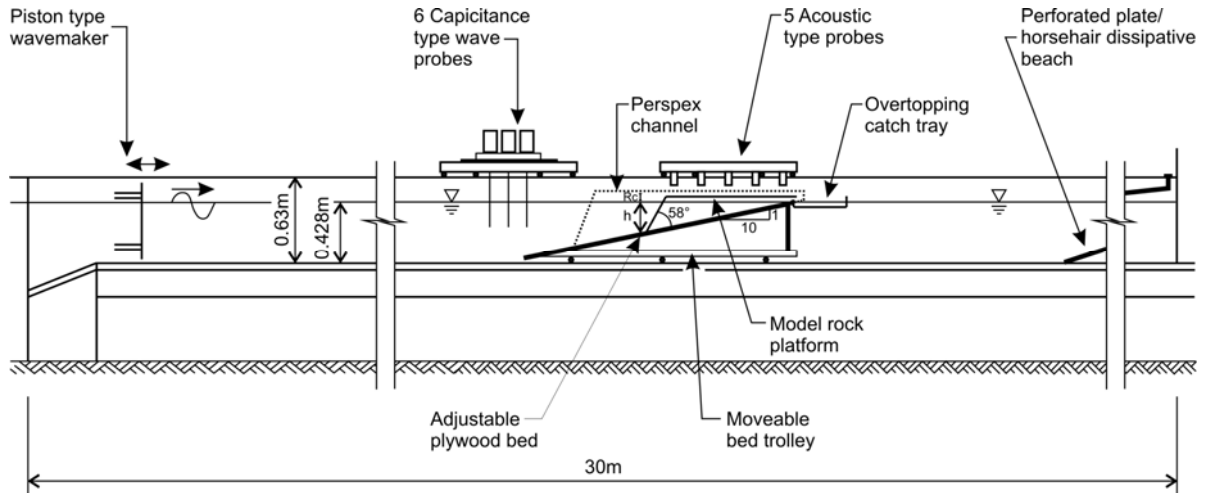


Figure 8-6 Laboratory facilities including wave tank, piston-type wavemaker, model bathymetry and platform and instrumentation.

8.4.3 Model design and scaling

Scale effects are introduced when the model fails to accurately represent the physical characteristics of prototype scale interaction; in this case, the interaction of the wave-form and rock platform. The various scaling relationships and their relevant similitude are as follows:

- Inertial to gravitational forces – Froude scaling
- Inertial to viscosity forces – Reynolds scaling
- Inertial to surface tension forces – Weber scaling.

Exact scaling requires the simultaneous fulfilment of both Froude and Reynolds scaling. This is not, in practice, possible. As gravity, pressure and inertial forces are the key forces for wave motion, Reynolds scaling is appropriate provided that the Reynolds number is sufficiently low.

Initial model scaling was based on geometric similarity with an undistorted scale of 1:40 used. The scaling relationship between length and time was determined by Froudian similitude, with the following scale factors being adopted:

- Length scale $L_m = 1/40 L_p$
- Time scale $T_m = 1/(40^{0.5}) T_p$
- Velocity scale $V_m = 1/(40^{0.5}) V_p$
- Overtopping scale $Q_m = 1/(40^{1.5}) Q_p$

Recent studies of overtopping (Pullen, 2007) have produced some generic rules for physical model studies to minimise scale effects associated with surface tension. These include using a model wave height greater than 50mm and period greater than 0.35s. Scale effects associated with the actual overtopping process of near vertical walls in a small-scale laboratory were found by de Rouck et al., (2005) to be negligible, however, Peirson and Cameron (2006) found that flows should exceed $D*V = 0.07 \text{ m}^2/\text{s}$ for air void ratios to scale correctly.

8.4.4 Model construction

8.4.4.1 Moveable bed

An average offshore seabed slope of 6 degrees or approximately 1:10 was found during field investigations (refer Section 8.3). As Froude scaling requires no correction to a uniform slope, a 10:1 constant slope bed was constructed from marine plywood to facilitate wave shoaling leading to the model platform

This sloping bed was constructed resting on a free-moving trolley in the horizontal axis, along the wave flume. This allows variable positioning of the model to an accuracy of 5 mm with respect to generated waves; thus allowing the effect of wave group evolution on overtopping to be characterised in full.

8.4.4.2 Platform structure

The test structure has been constructed from plywood to simulate the smooth and near impermeable nature of the rock platforms. The platform has been constructed to the representative geometry as determined in Section 8.3 and scaled according to the factors outlined in Section 8.4. The backing cliff has been omitted to enable capture and quantification of the overtopping flow volume using a catch tray.

To minimise reflection from the steep front face of the structure, the platform has been constructed to an across-flume width of 150 mm (of an available 600 mm width). This reduced the reflected incident (short) wave energy from 15.8% when a full width platform is used to between 2 and 9% depending on period and water level. Longer period waves generally induced greater reflection, as did lower water levels.

Perspex walls are used outside the model platform to channel the incoming wave and allowing the correct incident-reflected wave processes to occur offshore of the platform. Additionally, the sidewalls maintain the overtopping flow on the platform itself. Some sidewall effects are likely with along-flume video and photographs showing higher flows adjacent the perspex sidewalls than at the platform centre. Data collection is therefore limited to the platform centre line to minimise error. To facilitate adjustments in the water level, the platform is able to be moved forward and backwards on the sloping bed before being fixed in position (Figure 8-7). This allows desired water level to be obtained to a vertical accuracy of ± 1 mm.



Figure 8-7 Photograph showing the model platform with perspex sidewalls sitting on the sloping bed. The instrumentation trolley can be seen in the background.

8.4.5 Data collection and analysis

8.4.5.1 Offshore wave data

Offshore wave data is collected using capacitance type wave probes mounted on a flume-top trolley as described within Section 3.3.1. Probe records were obtained from

beyond the sloping bed toe (within deep water) to this physical probe depth limit at intervals of between 0.4 and 2.0 m.

Wave properties including maximum and minimum surface elevation, mean, maximum and significant wave heights, peak period and wave speed are obtained by zero-downcrossing as discussed within Section 4.3.

8.4.5.2 Overtopping data

Acoustic sensors

The characteristics of overtopping wave-forms on the platform surface are evaluated using an array of Microsonic mic+ acoustic sensors. These sensors use the delay between transmitting and receiving acoustic pulses to determine the distance to the reflecting surface. Sensor resolution is given by manufacturers at 0.18 mm and accuracy at 2 % (Microsonic, 2007). Probes were spaced at 120 mm intervals at 5 locations from the platform edge. A sampling rate of 125Hz ensured the sharp fronts of the overtopping waves were captured.

Data was processed using a peak detection algorithm. This enabled the evaluation of individual overtopping wave-forms including

- Time of overtopping wave-form arrival
- Time of wave-form peak
- Elevation of wave-form peak.

Use of the sensor array allowed changes in the wave-form with distance from the platform edge to be assessed as well as allowing wave-form velocity between adjacent probes to be assessed. An example of raw wave-form and overtopping data is presented in Figure 8-8. A wave group is first detected by a wave probe offshore from the modelled platform, then by 6 acoustic sensors extending across the platform. The largest overtopping flow is by the second wave in the group with flow depth decreasing with distance across the platform.

A previous comparison of acoustic sensors to capacitance wave probes (Section 4.2.2) found a delay in response time for acoustic sensors. However, in this context where an array of sensors is used, this delay becomes inconsequential as all sensors exhibit the same relative delay. The previous comparison also found the acoustic sensors to slightly under-estimate wave height, particularly near sharp wave crests, due to the discrete observation area of the sensor. However, an overtopping bore of relatively uniform flow depth will return the same elevation across the sensor area, this eliminating this under-estimation.

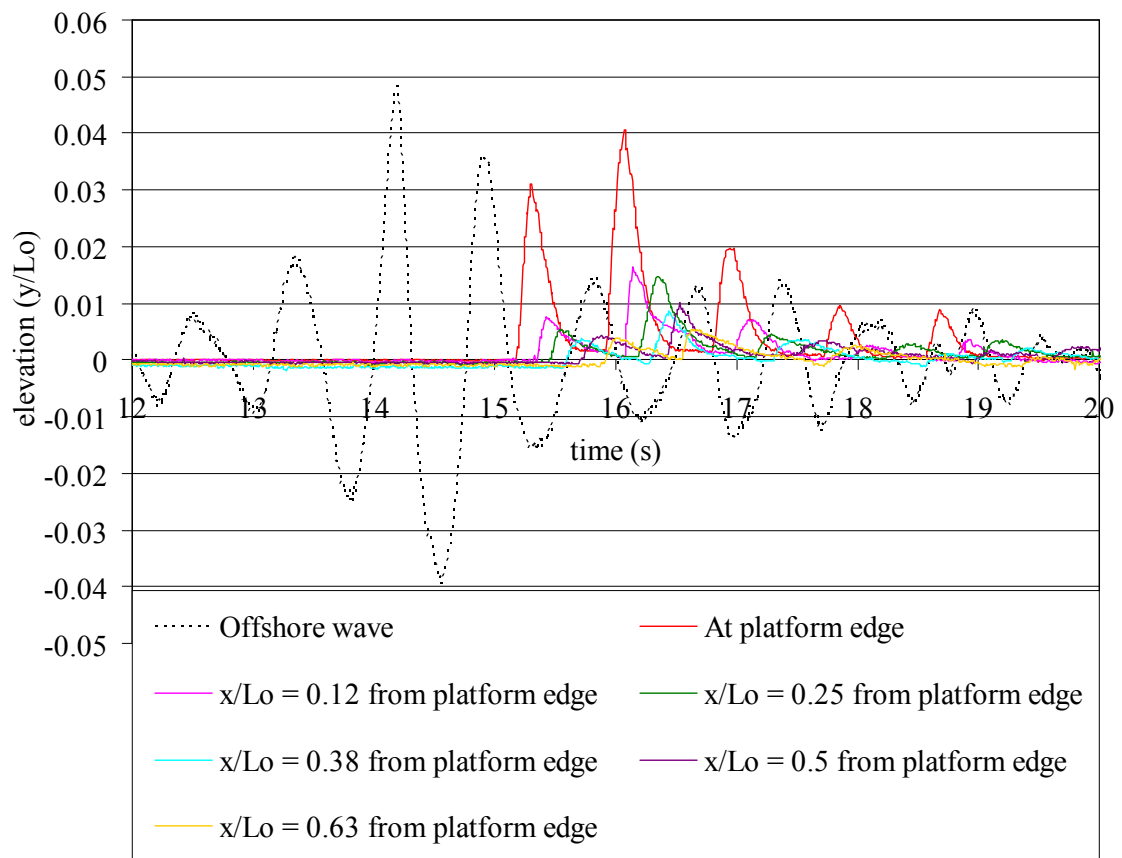


Figure 8-8 Recorded timeseries of offshore and overtopping wave-form elevations.

Digital video

A Casio Exilim EX-F1 camera using a high-speed CMOS type imaging sensor was used to obtain imagery of the overtopping process at a very temporal resolution of 300 Hz. This enabled very detailed qualitative analysis of precise overtopping characteristics such as incident wave impact, overtopping type (green water or aerated jet) and trajectory. Examples of non-impulsive (green water) overtopping and an impulsive (aerated white water) jet are shown in Figure 8-9.

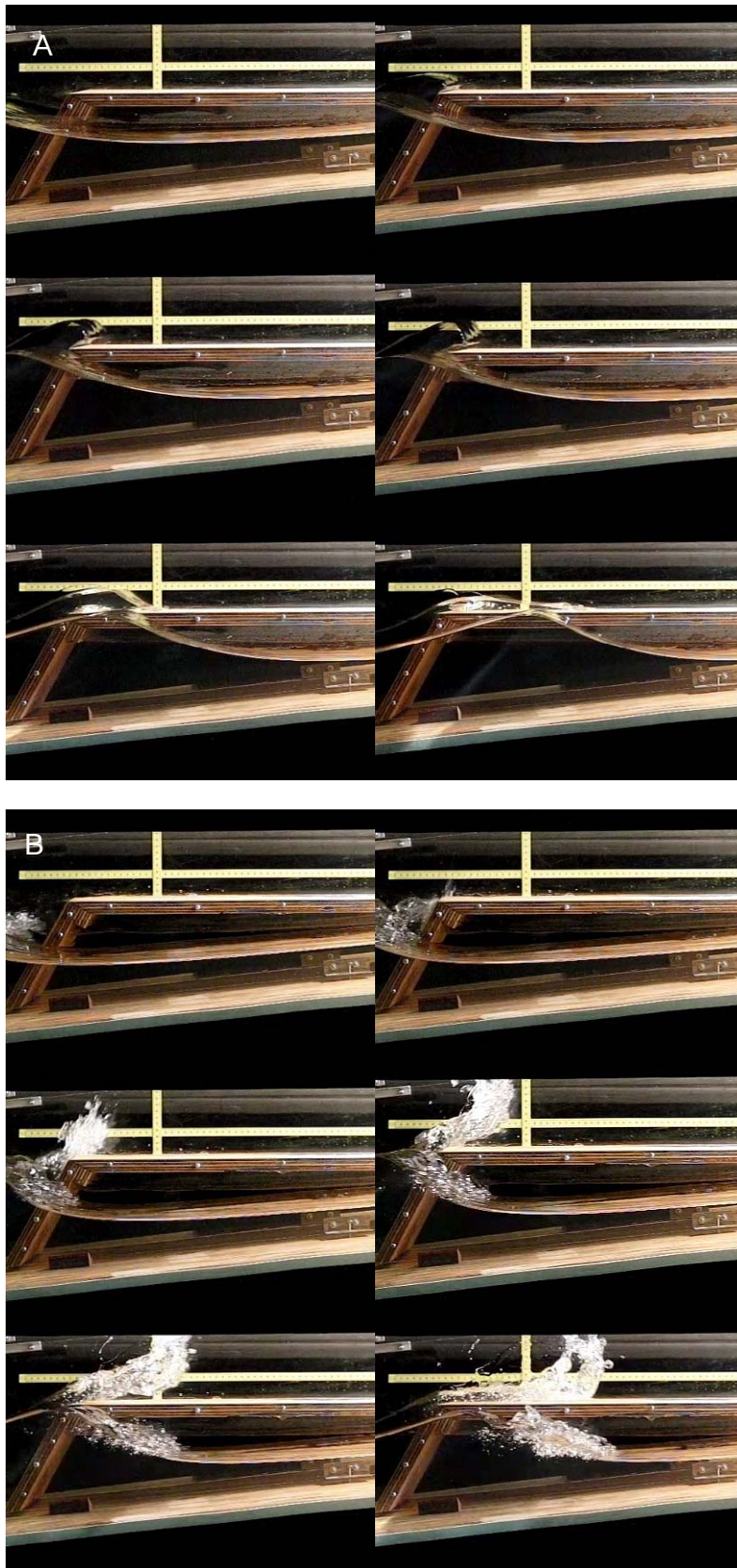


Figure 8-9 Examples of non-impulsive (green water - A) and impulsive (aerated white water - B) overtopping.

Overtopping catch tray

A catch tray was employed at the back of the model platform to obtain overtopping volumes. This catch tray was 0.15 m wide, extending across the entire model platform width and could store up to 5000 mL before removal and manual emptying was required. Overtopping volumes were group-averaged, and volumes for individual waves were not measured using this method.

8.4.6 Testing program

8.4.6.1 Wave conditions

Modulated wave-trains or wave groups have primarily been used in this study due to: 1. the unacceptable levels of reflection generally encountered using regular waves; 2. the tendency for waves within the field to exhibit grouped characteristics and 3. literature which suggests that episodes of abnormally large overtopping generally occur during conditions exhibiting high groupiness characteristics (Tsai et al., 2004). Additionally, the use of pre-defined wave groups substantially reduces the long testing regimes required to characterise group behaviour within irregular wave-trains and allows ‘worst-case’ scenarios to be tested directly.

A chirped type wave group, referred in Banner and Peirson (2007) as a Case 3 group has been used for model testing. This group, defined by Eqn. 8-2, is formed such that low and high frequency components coalesce as the waves propagate away from their point of generation. This allows waves of greater amplitude to be tested than may otherwise be generated.

$$x_p = -0.25A_p \left(\tanh \frac{4\omega_p t}{N\pi} + 1 \right) \left(1 - \tanh \frac{4(\omega_p t - 2N\pi)}{N\pi} \right) \sin \left[\omega_p \left(t - 0.018t^2 / 2 \right) \right] \quad (8-2)$$

Where:

x_p = waveboard position

t = time

N = number of carrier waves in group

A_p = piston amplitude

ω_p = angular frequency = $(g/(2\pi/\lambda)\tanh((2\pi/\lambda)h))^{1/2}$

λ = wavelength

h = still water depth

Wave combinations were tested as shown in Table 8-3 with wave height corresponding to the maximum height within the group and period to peak period.

Table 8-3 Tested wave conditions.

Prototype			Model			Wave steepness ($H_{\max}/L_o$)
Wave height ($H_{\max}$ - m)	Wave period (T_p - s)	Wavelength (L_o - m)	Wave height ($H_{\max}$ - m)	Wave period (T_p - s)	Wavelength (L_o - m)	
3.4	4.9	37	0.085	0.77	0.93	0.092
1.0	8.0	100	0.025	1.26	2.50	0.010
1.5	8.0	100	0.038	1.26	2.50	0.015
2.0	8.0	100	0.050	1.26	2.50	0.020
2.5	8.0	100	0.063	1.26	2.50	0.025
3.0	8.0	100	0.075	1.26	2.50	0.030
1.0	11.0	189	0.025	1.74	4.73	0.005
1.5	11.0	189	0.038	1.74	4.73	0.008
2.0	11.0	189	0.050	1.74	4.73	0.011
2.5	11.0	189	0.063	1.74	4.73	0.013
3.0	11.0	189	0.075	1.74	4.73	0.016
1.0	14.0	306	0.025	2.21	7.65	0.003
1.5	14.0	306	0.038	2.21	7.65	0.005
2.0	14.0	306	0.050	2.21	7.65	0.007
2.5	14.0	306	0.063	2.21	7.65	0.008
3.0	14.0	306	0.075	2.21	7.65	0.010

8.4.6.2 Water level and freeboard conditions

Water levels were modelled between Indian springs low water (ISLW) and highest astronomical tide (HAT). Tested water levels are given in Table 8-4 with respect to Fort Denison Datum in prototype and model scale.

Table 8-4 Tested water levels.

Modeled water level description	Prototype level (m Fort Denison Datum)
Highest Astronomical Tide	2.0
Mean High Water Spring	1.56
Mean High Water Level	1.44
Mean Sea Level Standard	0.895
Mean Low Water Neap	0.45
Indian Springs Low Water - approximately Lowest Astronomical Tide (LAT)	0

8.4.6.3 Group evolution effects

The wave group structures used correspond to the ‘Case III’ type waves as described by Song and Banner (2002). As discussed within Section 6.2.2, the maximum and minimum ($z_{\max}$ and $z_{\min}$) elevations of the group envelope can be observed to fluctuate between alternative positions of local maxima or minima within deep water (Figure 8-10). These occur when a wave crest or trough is coincident with the group envelope maximum and represent positions of maximum local energy density convergence. The amplitude of these local maxima and minima can also be observed to increase with distance along the tank as the nonlinearity within the envelope increases until either a point of recurrence or breaking is reached, after which local maxima and minima amplitudes decrease. This point is reached at around $x/L_0 = 8.3$ m from the paddle within the example group depicted in Figure 8-10. Wave steepness was selected to produce a recurrent type group, with testing focused around the positions of maximum energy density convergence.

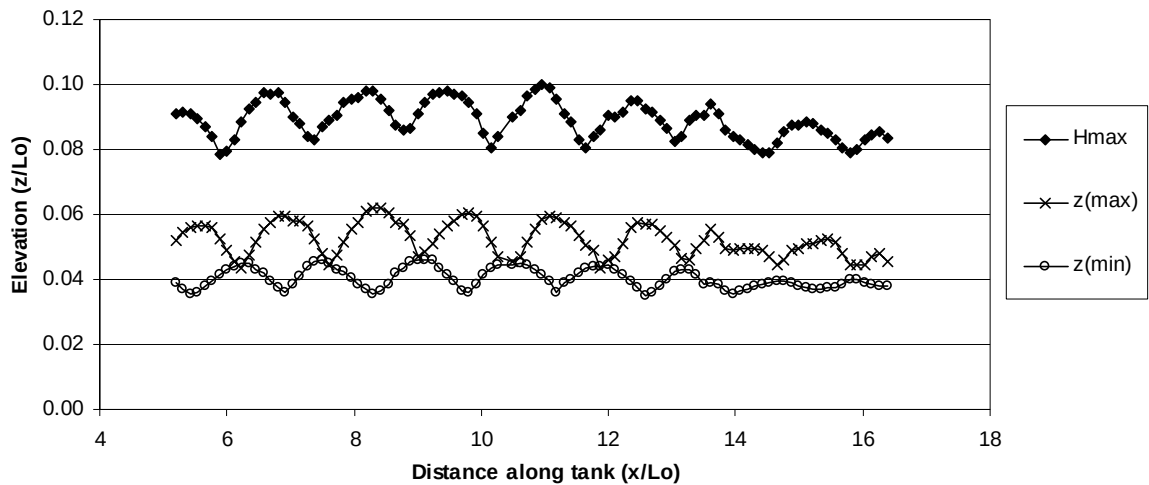


Figure 8-10 Evolution of maximum wave height ($H_{\max}$), maximum and minimum surface elevation ($z_{\max}$, $-z_{\min}$) in deep water for a Case 3, $N=5$, recurrent-type group. The group can be observed to reach a peak convergence at around $x/L_0 = 8.3$ m from the paddle, after which convergences decrease as the group becomes recurrent.

The position of the sloping bed is first selected with the platform edge located just in front of a local trough minima point (as observed in deep water) at $x/L_0 = 9.0$. Following data acquisition, the sloping bed is moved incrementally along the tank. This enables the spatial phase of the group evolution relative to the initiation of shoaling to be incrementally adjusted and differences in shoaling and breaking patterns observed.

The bed was moved in increments of 0.2 m or approximately $0.25L_0$ with a total of 10 positions over 1.8 m (or $\sim 2.0L_0$) captured. As this range captured more than a complete minima to minima cycle around the location of most highly grouped behaviour, it is likely to capture the most important scenario.

8.4.6.4 Summary of tests

A summary of the 67 detailed measurement tests undertaken is presented within Table 8-5. Results and analysis are provided in Section 8.5.

Table 8-5 Summary of tests. Blank values indicate same value as above.

			Prototype Scale					Model Scale				
	Testing influence of	Test	H _{max} (m)	T _p (s)	WL (m ISLW)	h _{toe} (m)	R _c (m)	H _{max} (m)	T _p (s)	WL (m ISLW)	h _{toe} (m)	Free board (m)
1	Group Phasing	1.1	3.4	4.87	1.500	4.000	1.350	0.085	0.77	0.038	0.100	0.034
		1.2										
		1.3										
		1.4										
		1.5										
		1.6										
		1.7										
		1.8										
		1.9										
		1.10										
3	Water Level / Steepness (Height, Period)	3.1	1	11	0.000	2.500	2.850	0.025	1.74	0.000	0.063	0.071
		3.2	2	11				0.050	1.74			
		3.3	3	11				0.075	1.74			
4		4.1	1	11	0.500	3.000	2.350	0.025	1.74	0.013	0.075	0.059
		4.2	1.5	11				0.038	1.74			
		4.3	2	11				0.050	1.74			
		4.4	2.5	11				0.063	1.74			
		4.5	3	11				0.075	1.74			
		4.6	1	14				0.025	2.21			
		4.7	1.5	14				0.038	2.21			
		4.8	2	14				0.050	2.21			
		4.9	2.5	14				0.063	2.21			
		4.10	3	14				0.075	2.21			
		4.11	1	8				0.025	1.26			
		4.12	1.5	8				0.038	1.26			
		4.13	2	8				0.050	1.26			
		4.14	2.5	8				0.063	1.26			
		4.15	3	8				0.075	1.26			
4.16		3.5	4.9				0.088	0.77				
5		5.1	1	11	1.000	3.500	1.850	0.025	1.74	0.025	0.088	0.046
		5.2	1.5	11				0.038	1.74			

		5.3	2	11				0.050	1.74			
		5.4	2.5	11				0.063	1.74			
		5.5	3	11				0.075	1.74			
		5.6	1	14				0.025	2.21			
		5.7	1.5	14				0.038	2.21			
		5.8	2	14				0.050	2.21			
		5.9	2.5	14				0.063	2.21			
		5.10	3	14				0.075	2.21			
		5.11	1	8				0.025	1.26			
		5.12	1.5	8				0.038	1.26			
		5.13	2	8				0.050	1.26			
		5.14	2.5	8				0.063	1.26			
		5.15	3	8				0.075	1.26			
		5.16	3.5	4.9				0.088	0.77			
		6.1	1	11	1.500	4.000	1.350	0.025	1.74	0.038	0.100	0.034
6		6.2	2	11				0.050	1.74			
		6.3	3	11				0.075	1.74			
7		7.1	1	11	1.800	4.300	1.050	0.025	1.74	0.045	0.108	0.026
		7.2	1.5	11				0.038	1.74			
		7.3	2	11				0.050	1.74			
		7.4	2.5	11				0.063	1.74			
		7.5	3	11				0.075	1.74			
		7.6	1	14				0.025	2.21			
		7.7	1.5	14				0.038	2.21			
		7.8	2	14				0.050	2.21			
		7.9	2.5	14				0.063	2.21			
		7.10	3	14				0.075	2.21			
		7.11	1	8				0.025	1.26			
		7.12	1.5	8				0.038	1.26			
		7.13	2	8				0.050	1.26			
		7.14	2.5	8				0.063	1.26			
		7.15	3	8				0.075	1.26			
		7.16	3.5	4.9				0.088	0.77			
8		8.1	1	11	2.200	4.700	0.650	0.025	1.74	0.055	0.118	0.016
		8.2	2	11				0.050	1.74			
		8.3	3	11				0.075	1.74			

8.5 Results and discussion

Offshore wave data including wave period, height distribution and evolution with distance along the flume was recorded using capacitance wave gauges and flow data across the platform including flow depth and velocity was recorded using acoustic sensors and digital video. Additionally, overtopping volumes reaching the back of the platform were recorded by a catch tray.

8.5.1 Incident wave data

As discussed within Section 7.2.1, the distribution of wave heights within a narrow-banded, random sea have been shown to closely approximate a Rayleigh distribution (Longuet-Higgins, 1952). The wave height distributions of the idealised groups used within the present laboratory experiments (Figure 8-11) may be compared to this idealised Rayleigh distribution. Figure 8-12 shows wave height probability plots for the observed waves compared to idealised Rayleigh distributions for each of the wave periods cases tested. The Rayleigh distribution is calculated using an inbuilt Matlab® cumulative distribution function.

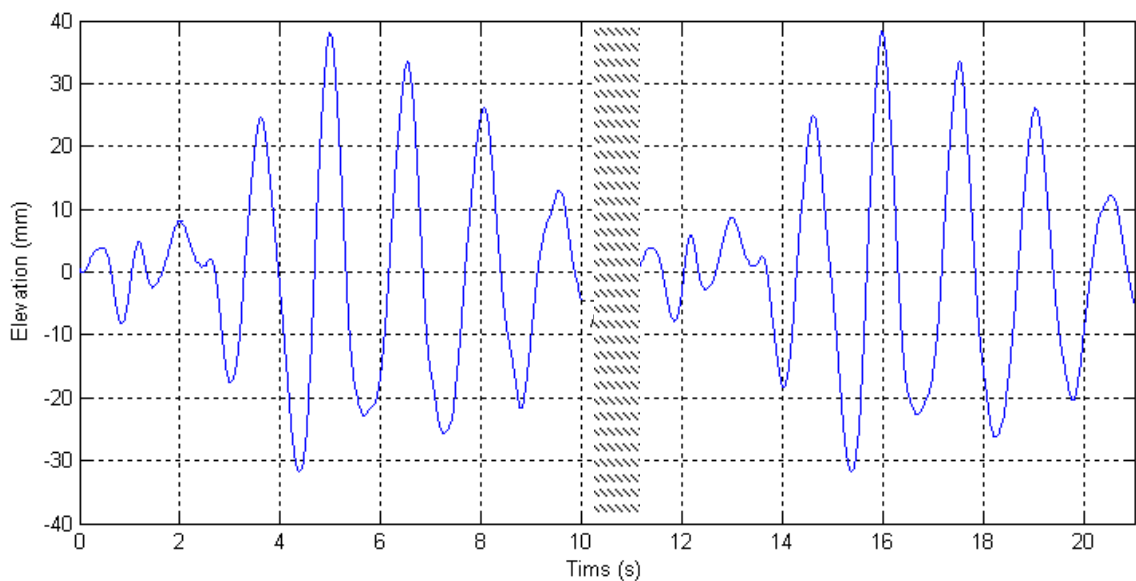


Figure 8-11 An example elevation time-series of idealised wave groups. Note that periods of no wave activity have been removed as indicated by the hatched area.

As shown in Figure 8-12, the wave groups yield a reasonable approximation of a Rayleigh distribution of wave heights. However, the maximum wave observed corresponds to a probability of exceedance between 6 and 12% for a Rayleigh distribution. In later sections of this report, the likelihood of levels of hazard occurring on rock shelves will be defined. To enable a consistent level of likelihood to be defined for different conditions, we will assign a probability of exceedance of 8% to the maximum waves for all test cases. This has the disadvantage of making likelihood of hazardous conditions at longer wave periods slightly less frequent than Figure 8-12 would imply. We can address this problem by observing that a small bias in probability can also be regarded as a bias in wave height. The difference between the wave height exceeded 8 and 12% of the time is 10%. As we will see later in the report, the spatial

phasing of the group propagation relative to the physical position of the rock shelf generates variations of up to 30%. Consequently, the approximate representation of the ocean wave distributions is a limitation of this investigation but this limitation is not expected to crucially affect the study findings.

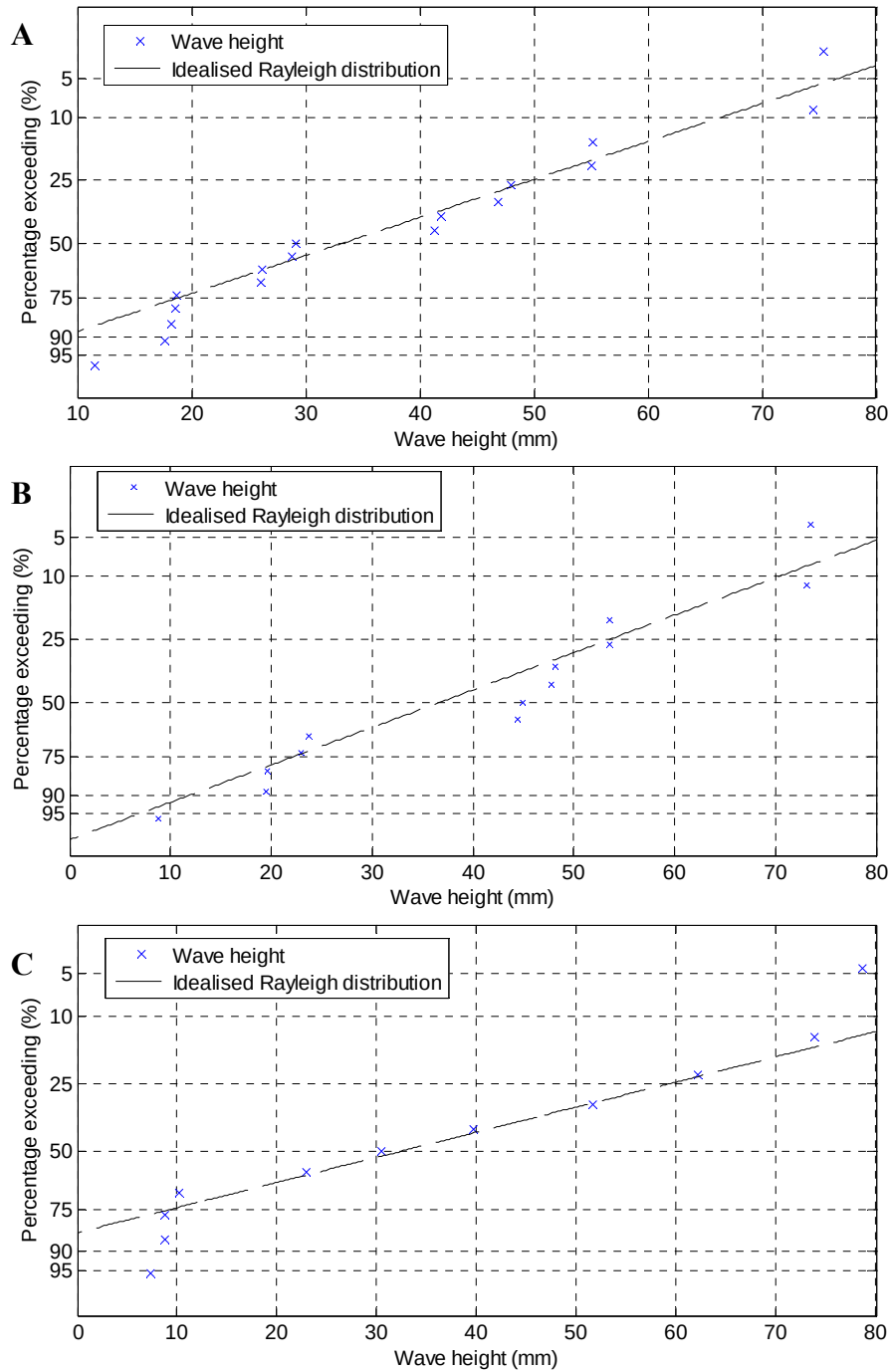


Figure 8-12 Wave height distribution for idealised group spectra with $T_{\text{prototype}} = 8\text{s}$ (A), $T_{\text{prototype}} = 11\text{s}$ (B), $T_{\text{prototype}} = 14\text{s}$ (C), compared to an idealised Rayleigh distribution. Maximum observed wave height corresponds to $H_{6\%}$ for $T_{\text{prototype}} = 8\text{s}$ (A), $H_{8\%}$ for $T_{\text{prototype}} = 11\text{s}$, and $H_{12\%}$ for $T_{\text{prototype}} = 14\text{s}$.

Wave heights within a Rayleigh distribution may be related to one another statistically (Longuet-Higgins, 1952) through the relationship given by Eqn. 7-1. From this equation, relationships between characteristic wave heights may be approximated. The most common characteristic wave height used in reporting and predicting sea conditions is the significant or wave height (H_{sig}). Study outputs should therefore be given according to this characteristic height. In order to ensure the waves tested correspond to the more extreme waves within a distribution, the recorded maximum wave height in each experiment was defined as being equivalent to $H_{8\%}$ as discussed earlier. A corresponding significant wave height could then be determined according to Eqn. 7-1 as $H_{8\%} = 1.14 H_{sig}$. While the assumed distribution may vary depending on the site-specific conditions, a Rayleigh-type distribution widely used as a first approximation (USACE, 2003).

8.5.2 Overtopping flow results

The overtopping flow depth and velocity for each wave is found using a peak-detection algorithm (O'Haver, 2006) modified substantially for the present study. Due to the highly repeatable nature of the generated wave groups, ensemble averaged values are calculated for each wave within the group. Examples of flow depth (D) and velocity (V) for individual waves from Exp 7, Test 5 are shown in Figure 8-13 (A-B). This figure illustrates the variation in both flow depth and velocity with distance from the platform edge. While depth undergoes a significant reduction within a short distance from the edge, followed by relatively consistent depth, velocity varied less predictably with some increased velocities evident shoreward of the platform edge. In order to obtain a flow value comparable to safety guidelines as discussed within Section 2.3.2, depth and velocity are multiplied to provide a DV value. An example of the wave-to-wave and cross-platform variation of this value is illustrated within Figure 8-13 (C).

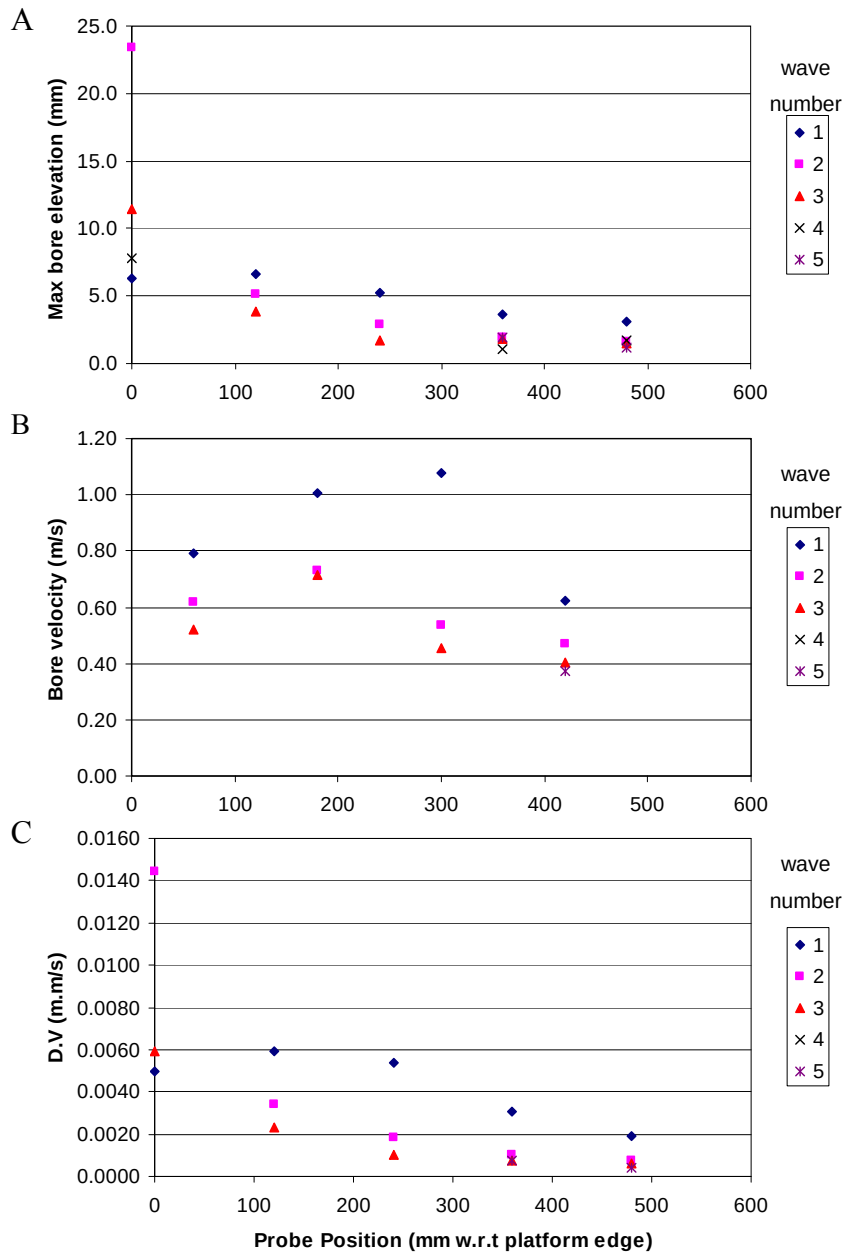


Figure 8-13 An example of overtopping flow depth (A), velocity (B) and flow $D \cdot V$ (C) for individual waves as a function of across-platform distance. Example from Test 7.5.

While the wave-to-wave distribution of overtopping flow is of scientific and engineering interest, from a hazard perspective, only the maximum values are relevant. These maximum depth, velocity and DV values are obtained for each wave height, period and freeboard combination.

8.5.3 Predicting hazardous conditions

In order to accurately predict the onset of hazardous conditions, the relative importance of various parameters on overtopping must be quantified. A correlation table (Table

8-6) allows the relative significance of linear relationships to be tested using a Pearson correlation coefficient. Based on a sample population of 44, giving 42 degrees of freedom, a correlation becomes significant to the 95th percentile at $r = 0.3044$. This implies that the overtopping flow depth, velocity and DV are influenced principally by wave height (H). However, overtopping volume is influenced by not only wave height, but also freeboard (Rc) and toe depth (h).

Table 8-6 Correlation values (r) for linear relationships.

	R_c (m)	h (m)	$H_{8\%}$ (m)	H_{sig} (m)	L_o (m)	T
$D_{8\%}$ (m)	-0.275	0.275	0.478	0.478	0.089	0.115
$V_{8\%}$ (m/s)	-0.095	0.095	0.630	0.630	0.051	0.016
$(D.V)_{8\%}$ (m.m/s)	-0.301	0.301	0.562	0.562	0.105	0.109
Overtopping q (l/m)	-0.415	0.415	0.460	0.460	0.280	0.283

While the correlations are statistically significant, the relationships are relatively weak and are not likely to provide meaningful predictors of hazardous conditions. Therefore, further analysis was undertaken.

8.5.3.1 Non-dimensional parameters

To obtain results that were scale independent, non-dimensional parameters corresponding to platform geometry and wave and flow conditions were examined. Examples of some non-dimensional parameters used include:

Wave steepness
$$\frac{H_{sig}}{L_0}$$

Where H_{sig} = deep water significant wave height (m)
 L_0 = deep water wavelength (m)

Relative freeboard
$$\frac{H_{sig}}{R_c}$$

Where R_c = Freeboard between still water level and the edge of platform (m)

Relative toe depth
$$\frac{H_{sig}}{h}$$

Where h = depth below still water level at platform toe (m)

$$\text{Distance from platform edge} \quad \frac{x_{edge}}{L_o}$$

Where x_{edge} = distance from platform edge (m)

$$\text{Flow depth} \quad \frac{D_{8\%}}{H_{sig}}$$

Where $D_{8\%}$ = flow depth for the 8th percentile largest wave in a Rayleigh distributed sea (m)

$$\text{Flow velocity} \quad \frac{V_{8\%}}{\sqrt{gH_{sig}}}$$

Where $V_{8\%}$ = flow velocity depth for the 8th percentile largest wave in a Rayleigh distributed sea (m/s)

$$g = \text{gravitational force} = 9.81 \text{ m/s}^2$$

$$\text{Flow rates for safety analysis} \quad \frac{DV_{8\%}}{\sqrt{gR_c}^{3/2}}, \frac{DV_{8\%}}{\sqrt{gR_c}L_0}, \frac{DV_{8\%}}{\sqrt{gL_0}^{3/2}}$$

Where $DV_{8\%}$ = flow depth * flow velocity for the 8th percentile largest wave in a Rayleigh distributed sea (m.m/s)

These non-dimensional parameters are tested using the same Pearson correlation test (Table 8-7). An r value greater than 0.3044 indicates significance to the 90th percentile and greater than $r = 0.4896$ indicates significance to the 99.9th percentile. This is exceeded by a number of relationships; however, the highest correlations are between $H_{sig}^2/R_c L_0$ and $(DV)_{8\%}/g^{0.5}L_0^{3/2}$ and between H_{sig}/R_c and $(DV)_{8\%}/(g.L_0)^{0.5}R_c$. These relationships are depicted graphically within Figure 8-14.

Table 8-7 Correlation values (r) for non-dimensional relationships with the two highest correlations shown in bold.

	H_{sig}/R_c	H_{sig}/h	H_{sig}/L_0	h/R_c	$H_{sig}^2/R_c L_0$	$H_{sig} R_c/L_0^2$	$R_c^2/H_{sig} L_0$	$R_c L_0/H_{sig}^2$	$H_{sig}^2/h L_0$
$D_{8\%}/H_{sig}$	0.216	-0.282	-0.375	0.402	-0.251	-0.364	-0.511	-0.158	-0.371
$D_{8\%}/R_c$	0.874	-0.021	-0.038	0.781	0.248	-0.141	-0.556	-0.414	-0.079
$V_{8\%}/g^{0.5} H_{sig}^{0.5}$	0.280	0.103	-0.054	0.123	0.049	-0.103	-0.406	-0.333	-0.075
$V_{8\%}/g^{0.5} R_c^{0.5}$	0.809	0.176	0.166	0.549	0.407	0.025	-0.509	-0.544	0.101
$(D.V)/((g^{0.5})(R_c^{3/2}))$	0.915	0.050	0.012	0.732	0.317	-0.101	-0.493	-0.381	-0.029
$(D.V)_{8\%}/((g^{0.5})(L_0^{3/2}))$	0.431	0.466	0.885	0.049	0.940	0.726	0.270	-0.381	0.787
$(D.V)_{8\%}/((g^{0.5})(R_c L_0^{1/2}))$	0.941	0.195	0.305	0.639	0.605	0.118	-0.381	-0.497	0.214

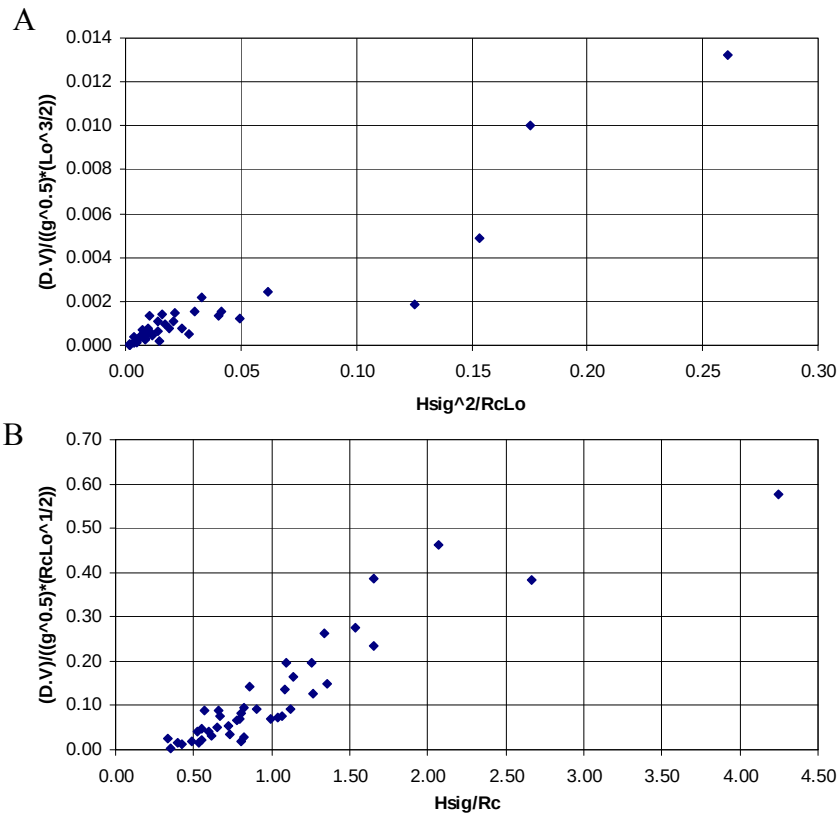


Figure 8-14 Relationships between the non-dimensional parameters $H_{sig}^2/R_c L_0$ and $(DV)_{8\%}/g^{0.5} L_0^{3/2}$ (A) and between H_{sig}/R_c and $(DV)_{8\%}/(g L_0)^{0.5} R_c$ (B)

While the correlation coefficients of the above relationships are similar ($r = 0.940$ and 0.941 respectively), the extreme values in the $H_{sig}^2/R_c L_0$ vs $(DV)_{8\%}/g^{0.5} L_0^{3/2}$ relationship exhibit a poor fit. This is evidenced by removing these high values and replotting (Figure 8-15). These extreme values are much better represented by the H_{sig}/R_c and $(DV)_{8\%}/(g L_0)^{0.5} R_c$ relationship.

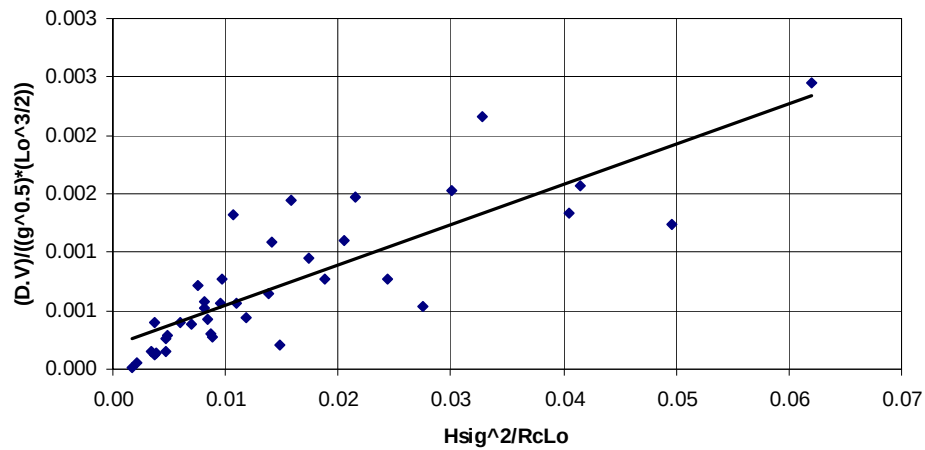


Figure 8-15 Relationships between the non-dimensional parameters $H_{sig}^2/R_c L_0$ and $(DV)_{8\%}/g_0^{0.5} L_0^{3/2}$ excluding the outlying values. Note that r has now reduced to 0.82.

8.5.3.2 Refining best-fit parameters

In the interest of comparison, both possible equations have been further refined by using parametric models and minimising the mean square error. For Eqn. 8-3a:

$$A \left(\frac{H_{sig}}{L_o} \right)^B \left(\frac{H_{sig}}{R_c} \right)^C + D = \frac{(D.V)_{8\%}}{\sqrt{g} L_0^{3/2}} \quad (8-3a)$$

Best fit parameters are:

Parameter	Value
A	0.245
B	1.824
C	1.358
D	0.000

For Eqn. 8-3b:

$$A \left(\frac{H_{sig}}{R_c} \right)^B + C = \frac{(D.V)_{8\%}}{\sqrt{g} L_0 R_c} \quad (8-3b)$$

Best fit parameters are:

Parameter	Value
A	0.227
B	0.680
C	-0.164

Comparing modelling and observed $(DV)_{8\%}$ values from the two equations (Figure 8-16) show Eqn 8-3b to have a superior visual fit. The mean residual difference for Eqn 8-3a is 0.026 with a std dev. of 0.016. The mean residual difference for Eqn 8-3b is 0.0006 with a std dev of 0.010. This implies Eqn 8-3a tends to substantially over predict $(DV)_{8\%}$ values while the mean prediction for Eqn 8-3b is in good agreement with observed with a std dev of around 15% of the higher $(DV)_{8\%}$ values. Visually, the agreement between modelled and observed $(DV)_{8\%}$ values is substantially better for Eqn 8-3b.

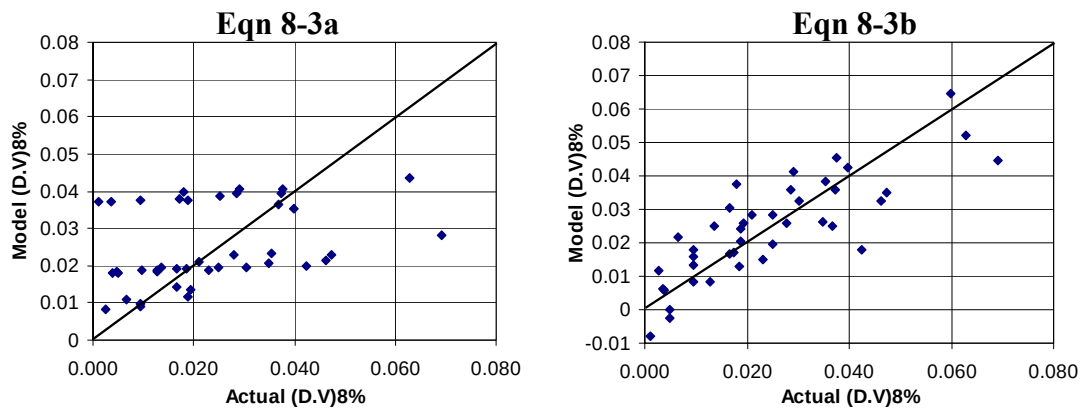


Figure 8-16 Differences between modeled and actual DV values for Eqns. 8-3a and 8-3b.

8.5.3.3 Effect of distance from platform edge

The change in $(DV)_{8\%}$ with distance from platform edge was assessed and is presented within Figure 8-17. While for the majority of cases, $(DV)_{8\%}$ is observed to decrease relatively uniformly to around $x/L_0 = 0.1$ before a steady-state is achieved, certain cases show a significant (up to 6 times) increase in $(DV)_{8\%}$ immediately adjacent to the platform edge before decreasing and also reaching a steady-state flow at around $x/L_0 = 0.1$. These differences are attributed to the type of overtopping, i.e. whether the wave is impacting a structure in an impulsive manner (generally leading to violent white water overtopping, i.e. Figure 8-9B) or non-impulsive (green water) manner (Figure 8-9A). White water overtopping is typically characterised by an upwardly directed flow jet and is therefore likely to reach maximum elevations landward of the platform edge. However, the limiting $(DV)_{8\%}$ guidelines, as discussed within Section 2.3.2, were derived for steady-type flow and their relevance to the highly aerated flows that characterise white water overtopping are unknown.

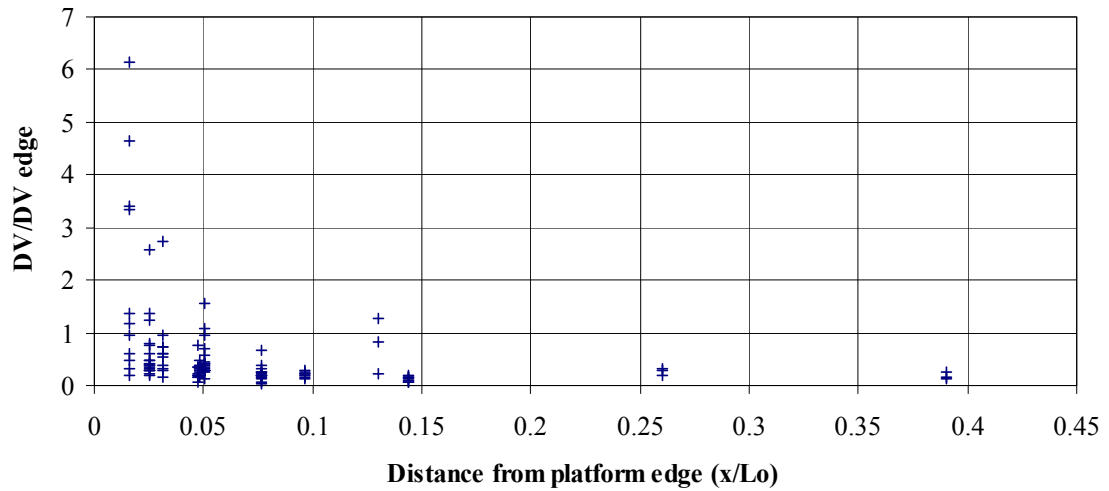


Figure 8-17 Change in $(DV)_{8\%}$ with distance from the platform edge.

The EurOtop guidelines (Pullen et al., 2007) give an impulsiveness parameter (Eqn. 8-1) as a function of the ratio of relative water depth at the structure toe (h) to both incident wave height (H_{sig}) and period ($T_{m-1,0}$). This impulsiveness parameter is plotted against the ratio of flow depth (d) immediately onshore of the platform crest to flow depth at the crest (Figure 8-18). Results show that for h^* values less than 0.075, depths onshore of the platform crest may exceed those at the crest, thus implying the existence of the upwardly directed jet which characterises white water overtopping. While the EurOtop guidelines defines $h^* < 0.2$ as impulsive, these guides were derived from testing at vertical walls which differs from the natural rock platforms and the present testing regime. Additionally, their criteria of impulsive, which is dependent only on the manner of wave impact on a vertical wall, differs from that required here which is dependent on overtopping jet trajectory inshore of the platform edge. In many cases, the elevation of overtopping flows classed by Pullen et al., (2007) as impulsive, with $h^* < 0.2$, do not attain higher inshore elevations than occur at the platform edge. These flows are therefore treated as non-impulsive or green water overtopping in the present study.

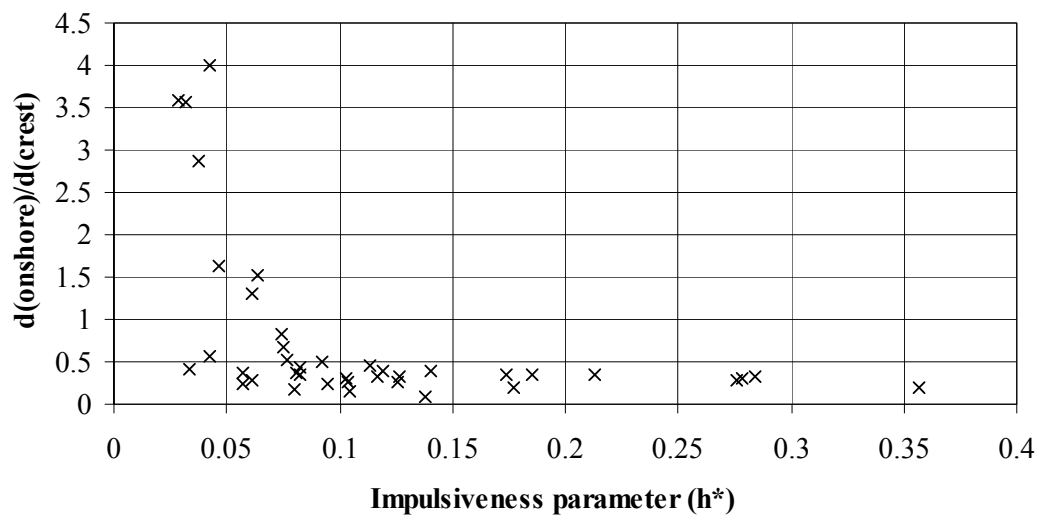


Figure 8-18 Relation between the EurOtop impulsiveness parameter (h^*) and the ratio of flow depth immediately onshore of the platform crest to flow depth at the crest.

This overtopping criteria according based on impulsiveness parameter is tested by comparing example overtopping cases with differing impulsiveness parameter to overtopping imagery (Figure 8-19). Results show good agreement, with lower h^* value cases exhibiting more violent and upwardly directed overtopping. While aeration, characteristic of white water overtopping, is limited in the $h^* = 0.032$ to 0.046 cases, scale effects are known to limit air entrainment in small scale models (Peirson and Cameron, 2006). However, the general hydrodynamic regime remains applicable. Thus, the $h^* = 0.064$ case, where significant white water or aeration is not evident, is none-the-less impulsive as the inshore overtopping flow elevation exceeds the elevation at the platform edge. For the green water cases ($h^* = 0.092$ to 0.213), more subdued overtopping is noted, with little or no upwardly directed jet motion.

Test	h^*	Overtopping based on criteria	Image
4.8	0.032	Impulsive/ white water	

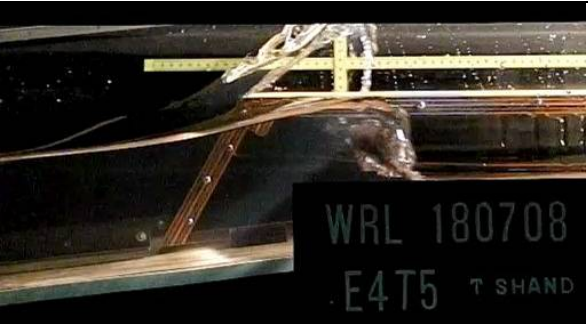
4.5	0.046	Impulsive/ white water	
5.5	0.064	Impulsive/ white water	
7.5	0.092	Non-impulsive/ green water	
5.14	0.125	Non-impulsive/ green water	
7.14	0.213	Non-impulsive/ green water	

Figure 8-19 Example cases with differing impulsiveness parameter value (h^*), the defined overtopping class according to the parameter and an overtopping image.

The upper envelope of the scattered data is shown within Figure 8-20. The green water case shows a relatively linear decrease from the platform edge to x/L_0 where the $(DV)_{8\%}$ is around $0.3(DV_{8\%})_{edge}$. This appears to be a steady-state value, remaining largely unchanged with further distance from the edge. The white water condition however, may reach a very high $(DV)_{8\%}/(DV_{8\%})_{edge}$ ratio immediately landward of the crest before exponentially decreasing to a similar steady-state value of around $0.3(DV_{8\%})_{edge}$ at $x/L_0 = 0.1$. Like the green water case, this value appears to hold with further distance from the edge.

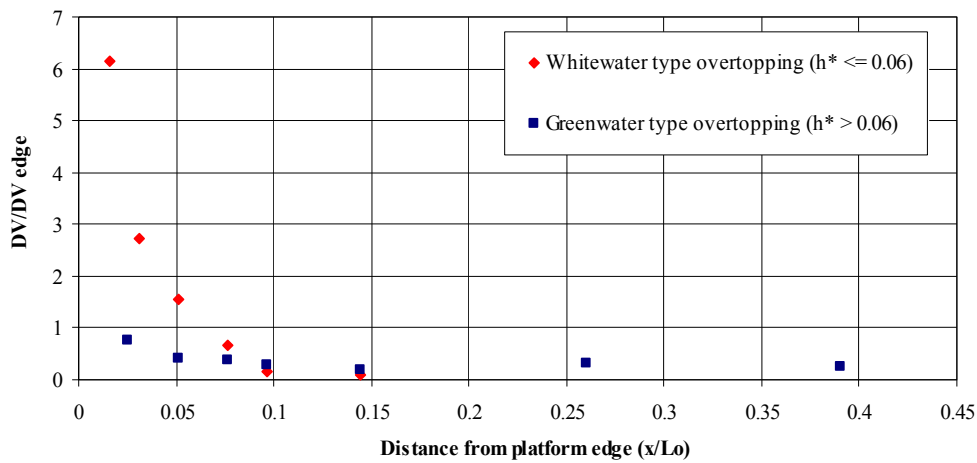


Figure 8-20 Maximum change in $(DV)_{8\%}$ with distance from the platform edge for white water ($h^* \leq 0.06$) and green water ($h^* > 0.06$) conditions.

Adjustment factors may be assumed as follows:

Green water type overtopping ($h^* > 0.06$)

$$\text{For } x/L_0 = 0 \text{ to } 0.1 \quad \gamma_x = 1 - 7x/L_0 \quad (8-4a)$$

$$\text{For } x/L_0 > 0.1 \quad \gamma_x = 0.3$$

White water type overtopping ($h^* \leq 0.06$)

$$\text{For } x/L_0 = 0 \text{ to } 0.015 \quad \gamma_x = 1 + 350x/L_0 \quad (8-4b)$$

$$\text{For } x/L_0 = 0.015 \text{ to } 0.1 \quad \gamma_x = 11e^{-38(x/L_0)}$$

$$\text{For } x/L_0 > 0.1 \quad \gamma_x = 0.3$$

An adjustment factor γ_x may thus be incorporated as a function of distance, as depicted within Figure 8-21, into the hazardous wave height equation.

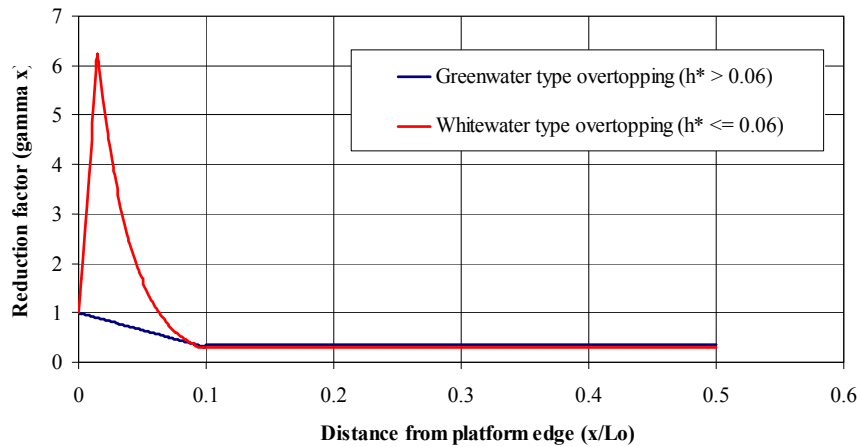


Figure 8-21 Reduction factor γ_x , as a function of distance from platform edge for white water ($h^* \leq 0.06$) and green water ($h^* > 0.06$) conditions.

8.5.3.4 Derivation of hazardous condition formula

Eqn 8-3B is combined with Eqn 8-4 to give a relationship which defines the limiting significant wave height before hazard from the 1% overtopping event is reached based on a threshold $(DV)_{8\%}$ value (Eqn. 8-5). The reduction factor for impulsive, white water-type overtopping between $x/L_0 = 0$ and 0.1 should be used with caution as safety guides are untested in this type of flow.

$$H_{sig} = R_c \left(\frac{(D.V_{8\%})_{Limit}}{0.227 \sqrt{gL_0} R_c \gamma_x} + 0.722 \right)^{1.47} \quad (8-5)$$

Where:

H_{sig} = significant wave height (m)

$(D.V_{8\%})_{Limit}$ = limiting $(DV)_{8\%}$ value before conditions are deemed hazardous

$D_{8\%}$ = overtopping depth (m) of the largest 8% of waves

$V_{8\%}$ = overtopping velocity (ms^{-1}) of the largest 8% of waves

L_0 = deep water wavelength (m)

γ_x = adjustment with distance from platform edge (refer Section 7.4.3.3)

x = distance from platform edge (m)

R_c = freeboard (m) ($R_c \geq 0.5$)

Examples of the limiting significant wave height before hazardous conditions are encountered as a function of freeboard, wave period and $(DV)_{8\%}$ are presented within Figure 8-22. Values are given for the platform edge and for $x/L_0 > 0.1$ from the edge, i.e. >15 m from the platform edge for a 10 s wave.

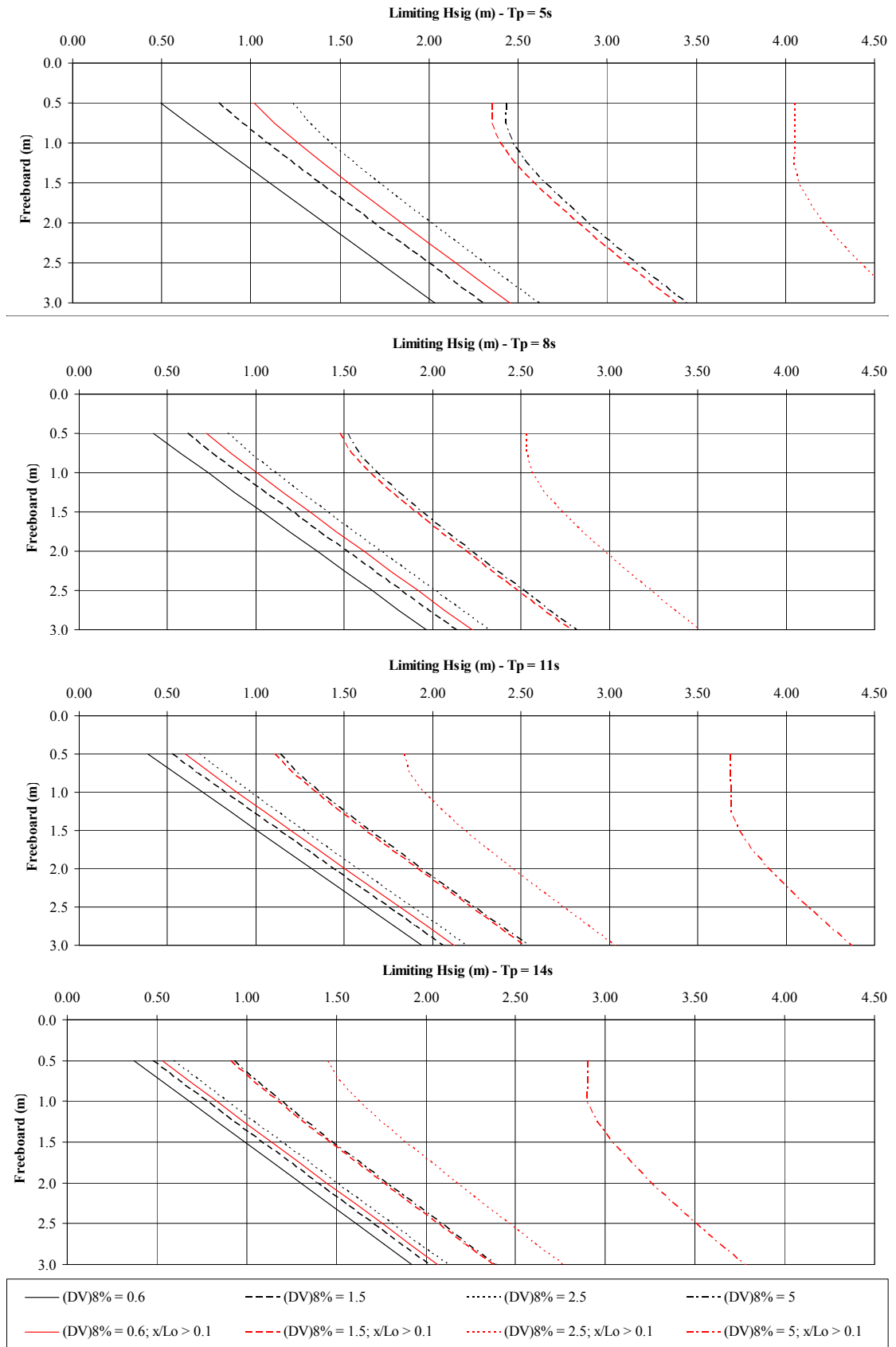


Figure 8-22 Examples of varying limiting significant wave heights before hazard from an 8% overtopping event is initiated as a function of freeboard (R_c) and peak wave period (T) at the platform edge and for $x/L_0 > 0.1$ from the edge for limiting $(DV)_{8\%}$ values of 0.6 to 5.

These results show that the DV_{Limit} value selected to indicate the initiation of hazardous conditions influences the resultant significant wave height. The values of $0.6 \text{ m}^2\text{s}^{-1}$ and $5 \text{ m}^2\text{s}^{-1}$ appear to provide lower and upper limits based on existing guidelines, with the values corresponding to $0.6 \text{ m}^2\text{s}^{-1}$ indicating safety for nearly all persons, values corresponding to $2.5 \text{ m}^2\text{s}^{-1}$ indicating safety for more experienced and better-equipped persons and values corresponding to $5 \text{ m}^2\text{s}^{-1}$ indicating possible safety limits for higher tolerable $(DV)_{8\%}$ values in highly aerated flows.

Moving back from platform by at least $x/L_0 = 0.1$ or 10% of wavelength allows between a 1.5 and 2 times increase in limiting wave height. For example, this would equal a distance of greater than 15 m for a $T_p = 10 \text{ s}$ wave. However, between the platform edge and this distance is an area of ambiguity for aerated white water type overtopping. Very high $(DV)_{8\%}$ values are possible in this region however, their influence on hazard due to the highly aerated nature of the flows is unknown. An example of this may be observed in the photograph of a rock fisherman being impacted at Flat Rock, Harbord (Figure 1-2). The flow depth appears 1.2 to 1.5 m and velocity, based on field photographs of overtopping and appropriately scaled laboratory rates may be approximated at 4-8 m/s. This gives a $(DV)_{8\%}$ value of between 4.8 and 12.0. This is significantly higher than the $(DV)_{8\%}$ of $2.5 \text{ m}^2\text{s}^{-1}$ assumed for experienced and well-equipped persons and the fisherman remained unimpeded in this case.

8.5.4 Site-specific examples

Site-specific assessment may be carried out using the platform elevation and toe depth relevant to specific locations. For example, this has been carried out for Flat Rock, at Harbord (Figure 8-23). However, site-specific features such as three-dimensional wave of geometric effects have not been taken into account and so the hazardous conditions predicted remain an indication only.

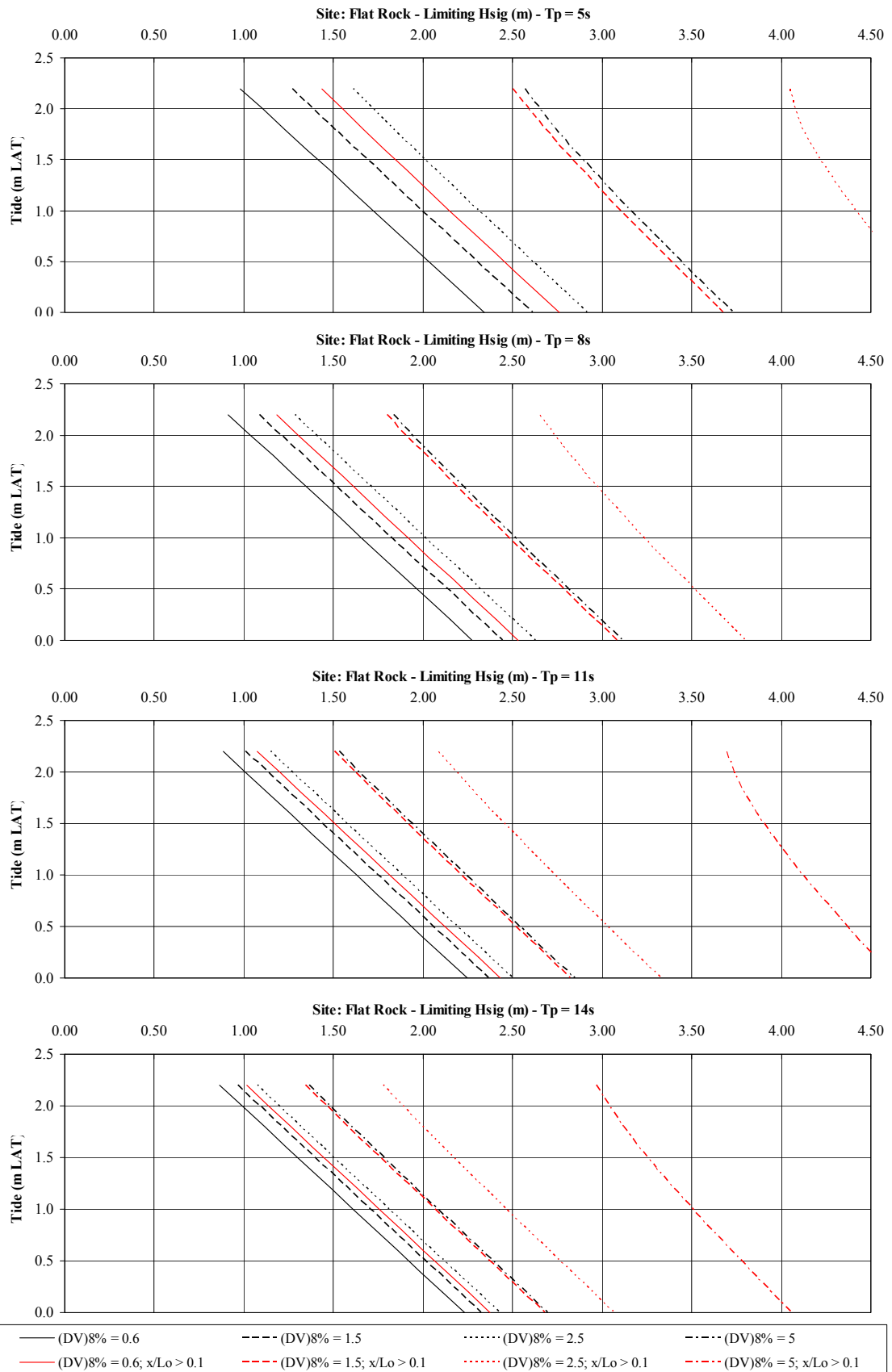


Figure 8-23 Examples of varying limiting significant wave heights before hazard from a 1% overtopping event is initiated at Flat Rock, Harbord as a function of tide (m LAT) and wave period (T) at the platform edge and for $x/L_0 > 0.1$ from the edge for limiting $(DV)_{8\%}$ values of 0.6 to 5.

8.5.5 Wave group phasing effects

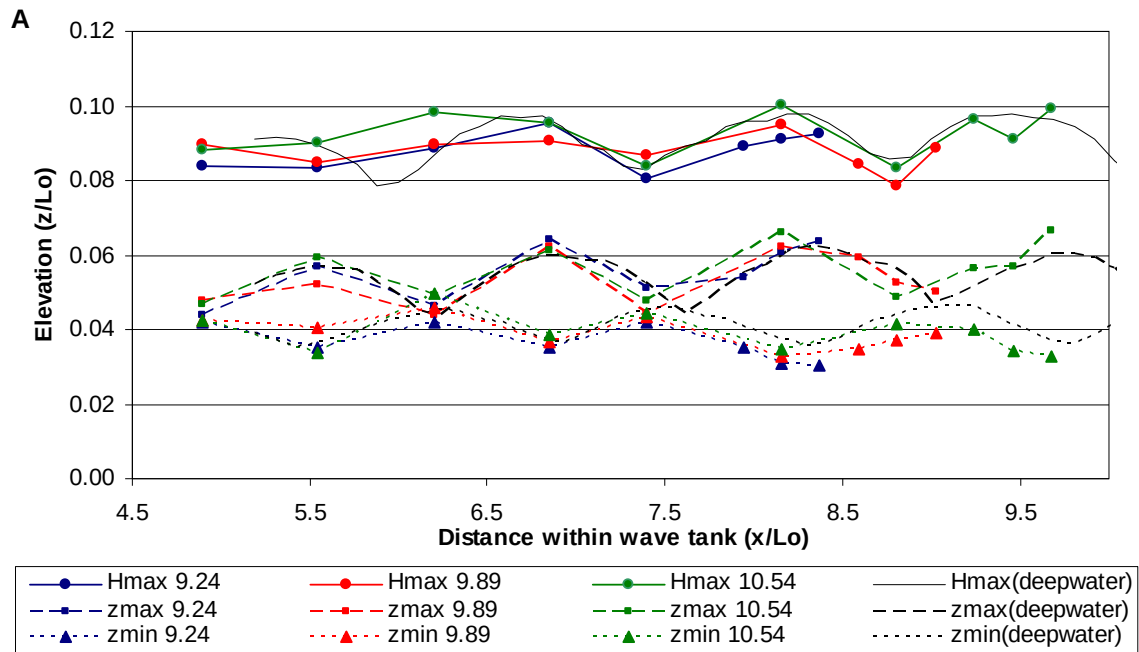
The spatial phasing of wave group evolutionary patterns has been shown to critically affect wave breaking processes on plane beaches by altering shoaling characteristics and breaking position and type (Shand et al., 2007). As detailed in Section 8.4.6.3, wave groups are observed to fluctuate between alternative positions of local maxima or minima dependent on whether a wave crest or trough is coincident with the group envelope maximum. Studies on plane beaches have shown that if, at a critical depth, a local maxima occurs and the group envelope maximum is coincident with a wave crest, then the subsequent shoaling occurs more gradually and breaking occurs more gently in a more offshore location. Whereas, if the group envelope maximum is coincident with a trough at this critical depth, shoaling is delayed until shallower water when it occurs more rapidly and waves break in a more plunging manner. This critical depth was found within the previous chapters to be between $\text{depth} = L_0/3$ and $L_0/5$ depending on bed slope and group steepness (Table 6-9). The present investigation aims to determine whether these differing shoaling and breaking patterns influences platform overtopping and if so, which of the shoaling types may lead to greater overtopping rates and more hazardous conditions and what the variation in overtopping rates may be.

A wave group of steepness $H_{\text{sig}}/L_0 = 0.05$ has been tested for group-induced overtopping effects. Nine combinations of group evolution and platform position were tested capturing more than a complete minima to minima cycle (Section 8.4.6.3). Probe results show a $\pm 3\%$ difference in initial, deep water maximum wave height. These differences may be attributed to the phasing of reflected waves with the instrument position and incident waves, long-waves present within the flume, or some other contaminative effect. Overall however, the wave groups offshore follow very similar evolutionary characteristics offshore.

By adjusting the position of the platform with respect to group evolution, the spatial phasing of the group cycle when the platform is encountered may be adjusted. Example cases are selected where the group reaches a local crest maxima immediately offshore of the platform ($x'_{\text{platform}}/L_0 = 9.24; 10.54$) and where the group reaches a local trough minima platform immediately offshore of the platform ($x'_{\text{platform}}/L_0 = 9.89$). Probe data has been collected up to $x/L_0 = 0.87$ (just under 1 wavelength) offshore of the platform.

Figure 8-24A depicts the evolution of the maximum wave height ($H_{\max}$), and maximum and minimum surface elevations ($z_{\max}$, $z_{\min}$) with distance along the flume. Note the location at which probe measurements cease is $x/L_0 = 0.87$ (just under 1 wavelength) offshore of the platform. The probe records cease for the example cases in differing locations and with the group structures in differing evolutionary states dependent on the platform position. Figure 8-24B depicts the same example cases with the spatial position given relative to the platform location. Here again, the evolutionary states of the wave groups differs between the $x'_{\text{platform}}/L_0 = 9.24$ and 10.54 cases which are approaching a local maxima with $z_{\max}$ increasing and $z_{\min}$ decreasing and the $x'_{\text{platform}}/L_0 = 9.89$ case which is approaching a local trough minima.

As the wave-train approaches the platform, the depth also decreases and the largest waves tend towards breaking. For the $x'_{\text{platform}}/L_0 = 9.24$ and 10.54 cases, breaking was found to occur at distances $x/L_0 = 0.76$ and 0.65 from the platform respectively. For the $x'_{\text{platform}}/L_0 = 9.89$ case, breaking did not occur until immediately offshore of the platform location, where the largest wave broke in a plunging manner on the platform face. This is in agreement with previous chapters investigating wave group behaviour and breaking on plane slopes which showed variability in the breaking position and type dependent on offshore group phasing.



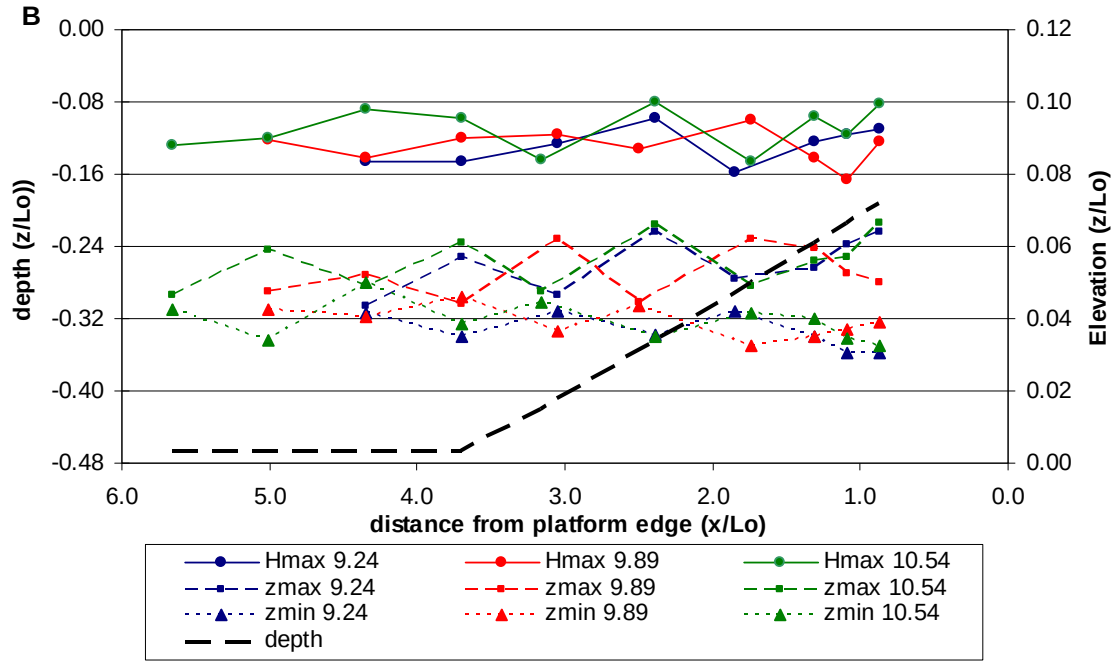


Figure 8-24 Wave height characteristics with respect to the paddle (A) and the platform edge (B) for three example group spatial phasing cases.

Using a Casio Exilim EX-F1 high-speed camera, as described within Section 8.3.5.2, the wave overtopping flows for each test case were recorded. Imagery of the overtopping flows of the three largest waves within each example case, at 0.05 s intervals is presented within Appendix E-2. Results show that overtopping by the first wave of the $x'_{\text{platform}}/L_0 = 9.24$ and 10.54 cases is small and occurs in a relatively benign manner. The overtopping by the first wave in the $x'_{\text{platform}}/L_0 = 9.89$ case however, occurs more substantially, with a jet projected slightly above the platform and some aeration evident. In all cases, the second wave overtops more violently, with a substantial white water jet is ejected above the platform level. The overtopping by the $x'_{\text{platform}}/L_0 = 9.89$ case however, is slightly more substantial, with a slightly larger jet attaining a higher elevation. The third wave of the $x'_{\text{platform}}/L_0 = 9.24$ and 10.54 cases overtops moderately, with a jet projected slightly above the platform and some aeration evident. Overtopping by the third wave of the $x'_{\text{platform}}/L_0 = 9.89$ case is more benign, with a small flow surging across the platform. These results show the offshore group phasing is important in determining the type of overtopping by various waves within the group.

Overtopping characteristics including overtopping flow depth and velocities are recorded for each overtopping flow event and non-dimensionalised by the idealised offshore significant wave height to remove any bias induced by slight variations in offshore wave properties. These depths and velocities and their combined $(DV)_{8\%}$ are presented within Appendix E-3. The maximum values for each case as a function of distance from the platform edge are presented within Figure 8-25.

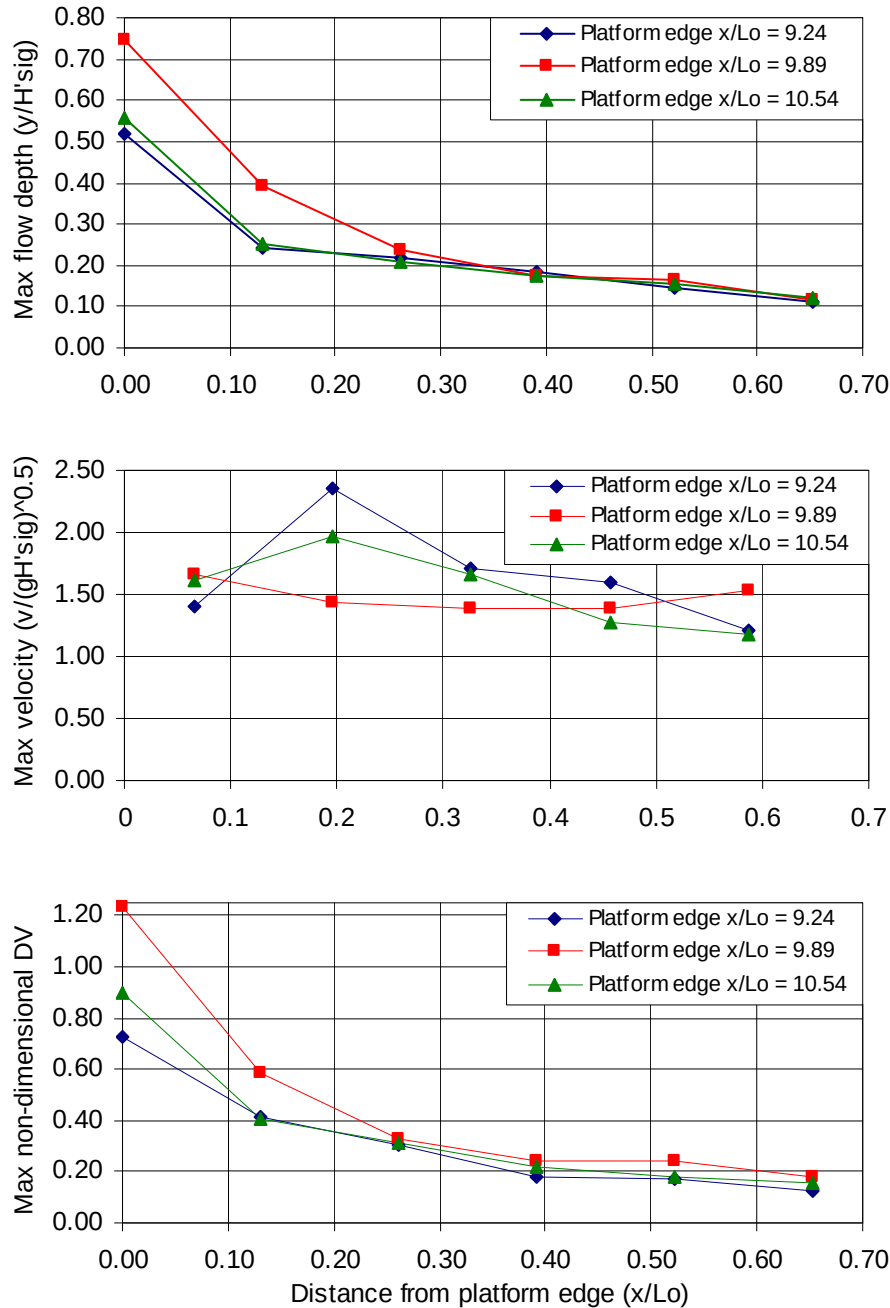


Figure 8-25 Overtopping flow characteristics for differing group evolution combinations where the non-dimensional $(DV)_{8\%}$ is given by $(DV)/(g.L_0)^{0.5}R_c$.

Overtopping flow depths for the $x'_{\text{platform}}/L_0 = 9.24$ and 10.54 cases are similar, reaching a maximum elevation at the platform edge and decreasing with distance across the platform. the $x'_{\text{platform}}/L_0 = 9.89$ case similarly reaches maximum elevation at the crest and decreases with distance across the platform, however, the maximum elevation attained is 30 to 40% larger than for the other cases until $x/L_0 = 0.25$ from the crest, where the elevations become similar. Velocity is somewhat more uniform across the platform, with highest velocities often reached inshore of the crest. This is attributed to the overtopping jet, initially projected above the platform level, increasing in velocity as it reaches the platform and flow depth decreases. For all cases, the combined $(DV)_{8\%}$ values are greatest platform edge, reducing with distance across the platform. However the $x'_{\text{platform}}/L_0 = 9.89$ case exhibits values 35 to 65% larger than the the $x'_{\text{platform}}/L_0 = 9.24$ and 10.54 cases.

The envelope of maximum overtopping flow depth, velocity and $(DV)_{8\%}$ including results for all tested group phasing positions is presented within Figure 8-26. The range in overtopping values are in near agreement with those presented for the three example cases. This is due to the extreme nature of the phasing cases which were originally selected. Similar reduction in flow depth and $(DV)_{8\%}$ with distance across the platform are noted for all cases. The percentage range in flow depth for all cases was 43% at the platform edge. This range remains near constant until $x/L_0 = 0.25$ from the platform edge when the range narrows, although percentage-wise remains at 30-50%. Similarly, the range for the $(DV)_{8\%}$ value at the platform edge is 70%, remaining constant until $x/L_0 = 0.25$ when the range narrows, although percentage-wise remains at 50-70%. Range in flow velocity ranges from 30 to 75%. It should be noted that one test was excluded due to errors in wave generation for that particular case.

In summary, differences in the group spatial phasing with respect to the platform position resulted in differing offshore breaking patterns and overtopping characteristics. Waves within groups reaching a position of local trough minima approximately one wavelength offshore of the platform edge tended remain unbroken until closer to the platform location and overtopping by waves at the beginning of the group was most substantial. Waves within groups reaching a position of local crest maxima offshore of the platform edge tended to break further offshore and reach the platform as a already spilling breaker, with more substantial overtopping occurring due to waves within the

middle of the group. Differences in the group spatial phasing resulted in ranges in depth and $(DV)_{8\%}$ at the platform edge of 43% and 70% respectively. These ranges generally remained constant until $x/L_0 = 0.25$ from the platform edge, after which they reduced. Larger maximum depth and DV values were noted for the groups associated with an offshore trough minima.

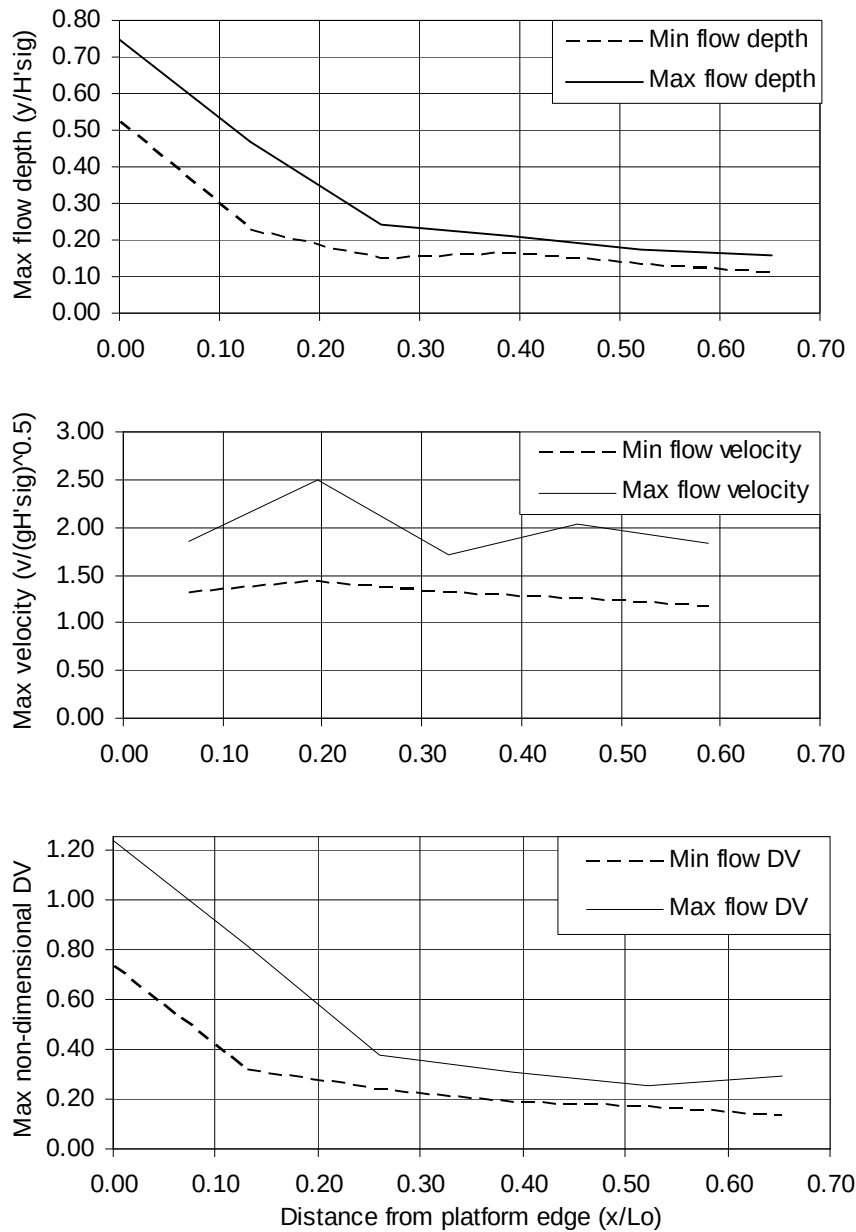


Figure 8-26 Envelope of maximum overtopping flow depth, velocity and $(DV)_{8\%}$ for varying spatial phasing of group evolution, where the non-dimensional $(DV)_{8\%}$ is given by $(DV)/(g.L_0)^{0.5}R_c$.

8.5.5.1 Effect of groupiness on predicting hazardous conditions

Based on these results, ranges in overtopping $(DV)_{8\%}$ values of up to $\pm 35\%$ could be expected during highly nonlinear wave group convergence events. Consequently,

changes in the threshold wave height before hazard is initiated of 10 to 30% could be expected dependent on freeboard, wave period and limiting $(DV)_{8\%}$ value.

In general, lower freeboard and wave period and higher limiting $(DV)_{8\%}$ values result in greater variability in the threshold wave height. However, for simplicity, a reduction in the threshold wave height before hazard is initiated of 30% is recommended when highly grouped events are likely. While research into the prediction of groupiness in wave-trains continues, spectral parameters such as spectral shape and the Benjamin-Feir index may, in the future, be of use.

8.6 Conclusions and recommendations

A laboratory-based study investigating the effect of hazardous waves, particularly those associated with wave groups, on the safety of rock fishing has been undertaken. This investigation has allowed evaluation of the critical wave and water level conditions that lead to significant, and potentially hazardous, overtopping of rock platforms

8.6.1 Rock fishing associated fatalities

While a high proportion of fatalities occur during larger than average wave conditions, fatalities have also occurred during smaller wave conditions indicating that even during relatively benign conditions, some hazard may exist. Other studies have shown that proportionally higher fatalities also occur during long wave period conditions and during incoming tides.

8.6.2 Rock platforms in the Sydney Region

A field investigation was undertaken between 21 and 22 April 2008 with 11 popular and/or dangerous rock fishing locations between Avalon and Cape Banks in Sydney surveyed using GPS and a laser level to obtain a representative cross-section of the platform and backshore at each site. Platform widths were typically in the order of 20 to 50 m, with backing cliffs ranging from near vertical to around 1(H):1(V) and from 5 to 50 m high. The platforms are typically near horizontal and range in elevation from 1.1 to 2.3 m AHD71 (-0.4 to +0.8 m relative to mean high water level). Offshore bathymetry was assessed through extrapolation of 2 m contour charts.

8.6.3 Key study findings

Key findings of the present study are as follows:

- The laboratory testing and analysis of wave groups has yielded prediction schemes for the 8% instability hazard as a function of platform freeboard, H_{sig} , T_p and distance from the edge of the platform. The 8% instability hazard is the level of hazard that would be anticipated to be exceeded every 12th wave;
- laboratory data collapses around the derived predictive schemes with a scatter of little more than $\pm 30\%$;
- an investigation of the sensitivity of the results to changed phasing of the wave group development relative to the shore shows systematic variation of $\pm 35\%$ and suggests that the bulk of the observed variability may be accounted for in terms of this process;
- significant increases in hazard can be observed as a function of increasing T_p for the same wave height – this supports recent fatality analysis that the relative risk is much higher for long period waves;
- very rapid changes in overtopping flow occur as a function of platform freeboard. This means that on rising tide levels, overtopping rapidly increases to a point at which potentially hazardous conditions are reached. Specifically, significant changes occur with only a change of 0.25 m in tide level. Under rising spring tide conditions, a change in 0.25 m tide level can occur in less than 30 minutes;
- large wave groups occur intermittently. Consequently, with rapid changes in tide level, group intermittency will tend to mask significant shifts in hazard that may occur;
- hazard rapidly changes with distance from the platform edge. This change in hazard depends on whether overtopping occurs in a white water or green water-type manner. A method for determining the type of overtopping expected and guidance for the decline in hazard for each have been quantified during this study;
- while very high DV values were noted for violent, white water-type overtopping events, little guidance is provided within existing literature on the tolerable DV values in the presence of pulsating or highly aerated flows. Due to the low density of these aerated flows, higher DV limits could be tolerable. However,

understanding of this process is, at present, insufficient to better refine guidance values;

- variation in maximum overtopping flows values of up to $\pm 35\%$ could be expected due to wave group effects for the same sea state conditions. For design, a reduction in the threshold wave height of 30% before hazard is recommended;
- while research into the prediction of groupiness in wave-trains continues, spectral parameters such as spectral shape and the Benjamin-Feir index may, in the future, be of use;
- during this study, reflections from a cliff-face behind the platform have not been incorporated but could be investigated during a subsequent study. It is noted that reflections may double the inundation depths on top of the platform, particularly during green water overtopping conditions. Large depths are likely to destabilise people on the platforms. Once destabilised, previous work on flood stability has shown that it is difficult for people to regain their footing.

8.6.4 Recommended future investigations

- A significant unknown within the present study is the influence of pulsating and aerated flows on DV safety limits. This is particularly true for white water type overtopping between the crest and the point at which steady-state type flow is reached. A controlled field or laboratory study may assist in better quantifying this value.
- The stability of fishermen wearing specialised equipment such as shoe cleats is not well known. This could be similarly field or laboratory tested. By refinement of these parameters, more accurate predictors of hazard can be given.
- Quantification of platform levels and offshore toe depth at more sites in the Sydney and NSW region would enable better site-specific hazard predictions. Platforms could be grouped according to geometry and hazard ranked according to wave and water level conditions.
- Field verification of overtopping flow depth and velocity predictions of this study would be useful.
- This study has not incorporated the effects of wave direction or wave convergence due to bathymetric effects. Studies specific to a number of hazardous sites would enable their effects to be incorporated.

Chapter 9 Conclusions and recommendations

This thesis has described a laboratory-based investigation into the effect of wave groupiness on wave shoaling and the breaking process. Specifically, the interaction of deep water intra-wave group processes and shallow water shoaling processes has been investigated and the applicability of existing methods for predicting breaking wave height and position evaluated. The implications of groupiness on the engineering design of nearshore structures is assessed, along with their effect upon overtopping and hazard on emerged rock platforms. The following conclusions are based on the results and analysis of the investigations, and recommendations for future study are described.

9.1 Conclusions

9.1.1 An optical LIF method for data collection

An improved technique, using an optical-based method for detecting the air-water interface, has been developed during this investigation. The method offers improvements in the vertical resolution (0.2 pixels) and accuracies (± 0.45 pixels) at which the air-water interface may be detected and results in accurate values throughout the shoaling, breaking and inner surf-zone regions (Figure 4-12) where conventional probes often become unreliable.

Results were compared with more conventional capacitance-wire wave probes and acoustic sensors (Figure 4-17). The optical method is found to predict a wave height between 2 and 3.5% larger than the capacitance probes and approximately 11% larger than the acoustic. These differences are attributed to the effects of a dynamic meniscus (Kinsman, 1965) on the wire capacitance probes and the finite sensor area of the acoustic sensors. The results were also compared to careful manual measurement from raw video, with resultant wave heights found to be within ± 0.5 mm or $\pm 1.4\%$ of optical measurements. This indicates that under these short and steep laboratory wave conditions, optical methods may offer improvement over conventional measurement techniques in detecting the air-water interface.

Adjacent along-flume video series are combined to give more complete spatial records of surface elevation (Figure 4-15). However, due to the limited temporal resolution of

the available imaging device (20 Hz), a maximum phase difference between adjacent series of up to 12.2° may exist introducing discontinuities between adjacent series. This may, however, be improved by combining multiple wave time-series, thereby generating a pseudo high-temporal resolution record. The limited temporal resolution and resultant phase shift between successive images similarly introduces an uncertainty in defining the absolute crest and trough of a wave-form. This has been estimated at $< 2.2\%$ of wave height in deep water, but increases near a steep crest prior to breaking and must be evaluated empirically. This error is reduced by observing a greater number of waves with observation of at least four wave forms required to guarantee measured wave height within one pixel of actual height. The error in determining height within the present investigation has been determined at between $H_{\text{actual}} = H_{\text{observed}} + 0.28$ to $- 0.12$ mm in deep water and $H_{\text{actual}} = H_{\text{observed}} + 0.42$ to $- 0.12$ mm near break point.

Improvements to this method could be achieved by:

- the use of higher frequency laser and imaging apparatus to improve temporal resolution;
- the use of higher resolution imaging devices;
- extending the light sheet in the along-flume direction, with larger overlap between video images;
- using an oblique camera orientation, looking up-flume into the wave-forms and thus eliminating obscured portions after the crest passes the image centreline.

These improvements were either unavailable to the present study or were recognised during the processing of imagery.

9.1.2 Monochromatic wave testing

Monochromatic waves of varying steepness were tested to confirm the empirical expressions for the prediction of breaker index as presented within modern design guidelines (Goda, 2000; USACE, 2003) and to investigate possible causes of the scatter apparent in raw breaker index data, upon which these guidelines are based

The development of height modulation was observed for these initially regular wave-trains with distance along the tank. Although waves were generated in a robust manner using a cantilevered, flexible plate-type wavemaker, capable of precisely reproducing deep water orbital wave motions, this modulation developed in all tested cases. It was

most notable for the steepest wave case, with modulation of over 30% wave height developing by $k_0x = 85$, where k_0 is the deep water wave number and x is the along-tank distance. These findings were in agreement with those of Benjamin and Feir (1967) and Melville (1982) where small instabilities in sideband frequencies were found to exponentially increase, inducing modulation in initially regular waves.

Variations in the observed breaking positions and height for individual test cases were noted, most evidently within the highly modulated, steep wave case, although also to a lesser extent in the lower steepness wave cases. This suggests that the scatter in observed wave breaking position noted by Smith and Kraus (1991) and Goda (2007) may be due, at least in part, to the development of these modulations. Variations in the derived breaker depth indices (H_b/d_b) were therefore also noted and although these tended to be greater for the steeper bed slopes, where small differences in breaking position resulted in larger differences in breaker depth index, they were also notably greater for the steep wave case where more substantial modulation was exhibited. These results suggest that Benjamin-Feir instabilities are intrinsic in propagating nonlinear wave-trains and the resulting height modulation may partially explain the scatter evident within breaker index plots (Weggel, 1972; Goda, 1970) upon which present design guidelines are based (USACE, 2003; Goda, 2000). Furthermore, other forms of instability, including three-dimensional types, may exist in finite-amplitude water waves (Melville, 1996), possibly introducing additional scatter in results.

Observed breaker indices were compared to those predicted by design guidelines, originally derived from regular wave testing. The USACE (2003) formula slightly over-predicted breaker index on the 20:1 slope (+2% to +5%), but under-predicted on the 10:1 and 5:1 slopes by 15% to 25% (Figure 9-1). The formula of Goda (2000) slightly over-predicted values on the steeper 5:1 slope (+6%) and under-predicted the 10:1 and 20:1 slope values by 5% to 15%. These results provided a background envelope of difference with which to compare wave group results.

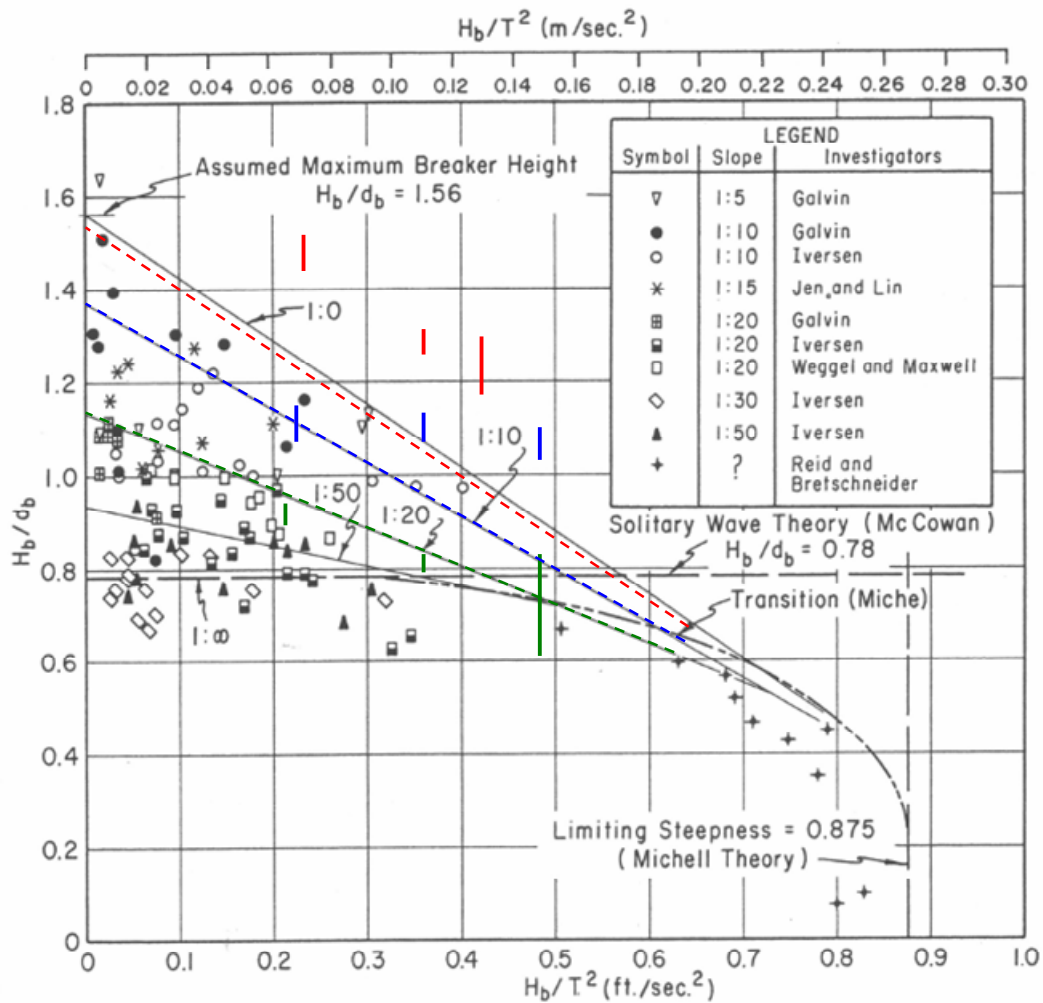


Figure 9-1 Breaker depth index data used by Weggel (1972) to derive generalised curves which form the basis of modern design guides such as USACE (2003). Guideline predicted values are indicated by dashed lines for 5:1 slope (—), 10:1 slope (—) and 20:1 slope (—) compared to the range of values observed within the present study for the 5:1 slope (|), 10:1 slope (|) and 20:1 slope (|). (source: Weggel, 1972).

The use of the optical LIF technique also allowed changes in the maximum and minimum surface elevations corresponding to the crest and trough amplitudes to be quantified during shoaling and at break point. The maximum surface elevation is an important quantity for engineering design. The ratio of maximum to minimum water levels ($z_{\max}/z_{\min}$) is observed to increase during shoaling, reaching a local maximum at break point, followed by a rapid decrease owing to breaking-induced energy losses. Ratios of maximum surface elevation to wave height at break point were found to comprise a significant (up to 85%) portion of the wave height. This is much higher than

would be expected in deep water where waves are nearly sinusoidal (ratio ≈ 0.5) and is attributed to the development of non-linearity during propagation and shoaling. Ratios were generally higher on flatter slopes and for lower wave trains which is attributed to the fetch length over which non-linearity may develop prior to breaking.

The ratio of maximum water level to depth at break point ($z_{\text{max-BP}}/d$) were compared to values predicted by the Goda (2000) expression for breaker height indices. As the expression is was developed for wave height rather than maximum water level, the expression over-predicted values, particularly for the steeper 5:1 slope where less non-linearity limits the crest amplitude. The coefficient A , which is assumed equal to 0.17 for regular waves (Goda, 2000), was adjusted to provide an improved fit. The coefficient A was reduced to 0.15 for the 10:1 and 20:1 slopes and to 0.11 for the 5:1 slope with good fit observed across the differing wave steepnesses. While further testing on slopes flatter than 20:1 is required, it is likely that as the bed flattens, crest asymmetry will become more pronounced and the coefficient, A will tend towards 0.17.

9.1.3 Wave group evolution during shoaling

Four wave group structures were tested. These groups varied in their generation mechanism (*Case*), the number of waves within the group (N), and the initial wave steepness ($s_0 = a_0 k_0$). During propagation within deep water, the group envelope was observed to fluctuate between alternative positions of local maxima and minima coincident with changes in local energy density. The amplitude of these fluctuations was observed to increase with distance along the wave tank until either a point of recurrence or breaking occurred, dependent on the initial group steepness and number of waves within the group. This evolutionary pattern was consistent across all group structures tested and with previous studies (Song and Banner, 2002; Banner and Peirson, 2007).

Testing was focussed around points of recurrence or breaking, where nonlinearity within the wave-train was greatest, to maximise differences in shoaling trends. The position of the sloping bed was incrementally adjusted to alter the spatial phasing of group evolution and differences in the shoaling and breaking patterns observed.

Changes to the group evolutionary trends were noted during propagation on to a sloping bed with the spacing between successive positions of local maxima or minima increasing in shallow water. This was shown, using simple linear relationships, to be associated with changes in the wave celerity compared to group velocity as depths decreased. An expression for the distance (D_{interval}) between successive local maxima or minima with decreasing depth was derived and showed good agreement with observation. This interval is given by the expression:

$$D_{\text{interval}} = c_g \left(\frac{L}{(c - c_g)} \right)$$

Where L , c and c_g are the wavelength, wave celerity and group velocity respectively, calculated at the position of group envelope maxima. Within deep water, where $c_g \approx 0.5c$, the interval between successive positions of local crest maxima reduces to $D_{\text{interval}} \approx L$. Within shallow water, wave celerity reduces and approaches the group velocity, thus increasing the distance between successive crest maxima as individual wave forms propagate relatively more slowly through the group. In very shallow water as wave celerity and group velocity approach unity, i.e. $c_g \approx c$, the interval tends to infinity.

A cyclic pattern emerges as the spatial phasing of the wave group and sloping bed is altered. A wave near the back of the group increases in size and its breaking position shifts offshore as it approaches coincidence with the group envelope maximum. Once coincident with the group envelope maximum, breaking occurs at or near a position of local crest maxima even as the sloping bed is shifted further. This trend continues until a threshold depth is reached where breaking is unable to occur and the break point shifts onshore. The following wave in the group then assumes the most offshore breaking position as it approaches coincidence with the following group envelope maximum.

This indicates that intra-group processes characterised by horizontal energy convergences remain dominant in dictating the breaking location of the wave coincident with the group envelope maximum. However, in all but one group case, breaking due solely to intra-group processes did not occur. Some shoaling influence (vertical energy convergences) is therefore required to increase total local energy convergences, as

described within Song and Benner (2002) and Banner and Peirson (2007), to a point where breaking is possible.

This is illustrated through the following conceptual diagram which qualitatively show the combination of deep water local energy convergences (—) with shallow water, depth induced convergences (—) resulting in total convergences (—). The position at which combined convergences exceed a threshold (—) define the initiation of breaking. During certain spatial phasing, the combination of deep water and shallow water convergences exceed the threshold at an offshore position of local crest maxima (early shoaling) and in others, the threshold is not reached until shallow water influences dominate (delayed shoaling). Note that this conceptual diagram does not incorporate the increase in distance between successive position of local crest maxima which occurs in shallow water as discussed within Section 6.2.2.

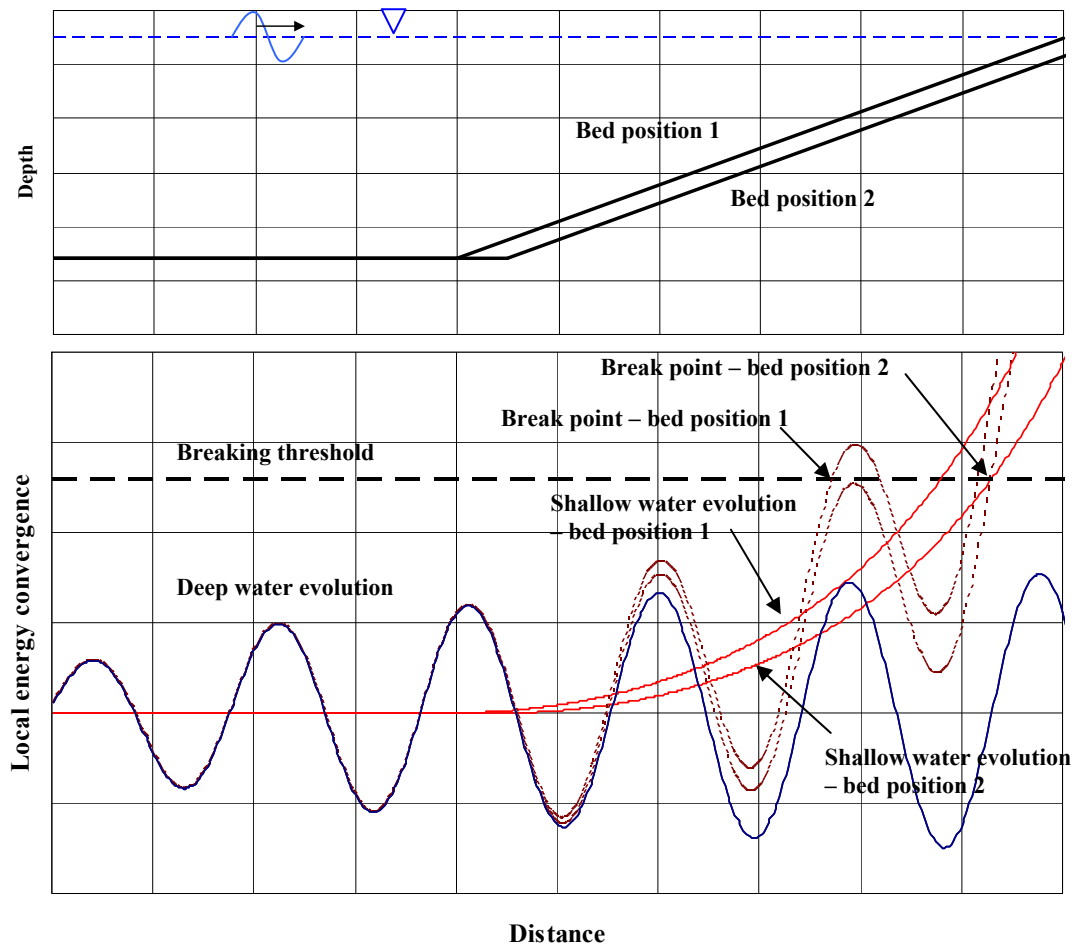


Figure 9-2 Conceptual diagram of the qualitative addition of horizontal (intra-group) and vertical (shoaling) energy convergences leading to wave breaking.

Differences in the shoaling, breaking and surf zone processes were noted dependent on spatial phasing of group evolution as shoaling commenced. Waves breaking offshore near a position of local crest maxima tended to shoal gradually and break in a gentle spilling manner. This type of group evolution was termed *early shoaling* and resulting in more gradual wave height dissipation across the surf zone and a more uniform increase in mean water level. Groups in which the largest wave did not break at a position of local crest maxima, continued into shallow water where shoaling occurred rapidly and breaking occurred in a more plunging manner. This was termed *delayed shoaling* and exhibited more rapid dissipation through the surf zone and non-uniform increase in mean water level. These differences in have implications for predicting maximum wave heights and for engineering design.

Knowledge of the evolutionary state of a group at specific offshore depths enabled prediction of the future shoaling and breaking characteristics of that group. Groups near a position of local trough minima at this critical depth were found to shoal gradually, with breaking occurring offshore near the following local crest maxima position. Groups near a position of local crest maxima at this depth, were found to shoal more rapidly within shallower water, with breaking occurring near a position of local trough minima. This critical depth was found to depend on bed slope and initial group steepness. Overall, the critical depth at which the state of group evolution determines future shoaling and breaking patterns occurred at $L_0/4.2 \pm 1$ for the group types and bed slopes tested within the current study. The critical depth for groups propagating onto steeper bed slopes and with steeper wave structures was generally deeper, between $L_0/3$ and $L_0/4.3$ than for groups propagating onto flatter slopes or groups of lower initial steepness where the critical depth was found to range between $L_0/4.7$ and $L_0/5.25$.

9.1.4 Maximum breaker indices and effect on engineering design

Successful design of nearshore structures relies critically on accurate definition of the maximum loading by wave forces that may act on that structure. For structures outside the surf zone, extreme wave statistics are required to determine this maximum wave. For structures within the surf zone, larger waves are expected to break offshore and the maximum wave height is limited by water depth. This maximum wave height to depth

ratio is defined by design guidelines (USACE, 2003; Goda, 2000) based on wave steepness and bed slope.

Existing guidelines for breaker index derived from regular wave testing have typically used a depth based on still water level to derive values (Rattanaipitikon and Vivattanasirisak, 2003). While this is appropriate for regular waves breaking in consistent location, irregular waves and wave groups contain waves of differing height and may break across a wide surf zone. Thus the local water depth becomes affected by factors such as set-up and temporal variations in water level induced by wave grouping (Section 6.3.4). This is frequently observed in field data sets, with Raubenheimer and Guza (1996) noting systematic increase in breaker index with decreasing relative depth. Goda (2000) suggests that the effect of both changes in the mean water level and changes due to surf beat should be considered when assessing random wave breaking.

Present testing has shown that while observed breaker indices at the outer edge of the surf zone are well or over-predicted by design guidelines, waves within the mid to inner surf zone, particular those associated with *delayed-type shoaling* groups, exhibit breaker indices higher than in the outer surf zone and higher than guideline-predicted values. The extent to which observed values exceed guideline predictions is dependent on group type, bed slope, position within the surf zone and the definition of depth. However, observed values exceeded guideline predicted values by up to 100% within the inner surf zone ($d/L_0 < \sim 0.02$) if depth was assumed relative to still water level. While this discrepancy reduces if more accurate measures of depth are used, including dynamic changes in water level through the surf zone, observed values remain above guideline predicted values by up to 20%. This indicates that for grouped wave trains, surf zone processes are complex and factors other than effective water depth may also influence the maximum wave height possible.

A revised breaker index model using the Goda (2000) expression (Eqn. 5-2) with a variable coefficient A , dependent on relative water depth and was found to better match observed data. By using a depth based on still water level, values implicitly include variations in water surface elevation within the surf zone induced by wave set-up and low frequency, group-induced fluctuations, and provide an upper envelope of possible wave height. An upper envelope value is essential for engineering or structural design

within the nearshore, where the maximum possible force to be placed on structures must be known. A first order assessment of the required adjustment of A yields the following expression:

$$A_x = (C_1 m + C_2) \left(\frac{d_{swl}}{H'_{m0}} \right)^{(C_3 m + C_4)}$$

Where C_1 - C_4 are coefficients dependent on the the wave characteristic sought including maximum ($H_{\max}$) or significant (H_{sig}) wave height and maximum surface elevation ($z_{\max}$):

x	H_{sig}	$H_{\max}$	$z_{\max}$
C_1	-0.24	-0.12	-0.09
C_2	0.17	0.25	0.2
C_2	-1.65	-1.65	-1.65
C_4	-0.42	-0.35	-0.5

This value has been derived for slopes ($m > 0.05$) and highly modulated groups. The values are likely to be conservative for less well modulated groups and on flatter slopes, as are likely to be encountered during practice with a minimum value of $m = 0.05$ used;

The required value of A for $z_{\max}$, while originally below those required for $H_{\max}$ in deep water, increases to exceed those for $H_{\max}$ in the inner surf zone ($d/H_{\text{sig}}' < 0.3$). This occurs as set up increases the trough level above still water level, giving a lower relative $H_{\max}$ than $z_{\max}$. This is important as it illustrates the non-suitability of simply using proportion of the maximum wave height to determine maximum possible water level within the inner surf zone.

Using the revised expression for A , predicted maximum wave heights were compared to observed heights for all wave group structures and bed slopes tested. Results showed that for each case, under-prediction was reduced to less than 2.5%. This offers substantial improvement over using a constant value of A , where under-prediction of up to 105% occurred depending on group type, bed slope and definition of depth.

Based on results of this thesis, the often assumed notion that wave climate information is not required when calculating maximum wave height within a depth-limited

environment is refuted. If not accounted for, the contribution of offshore wave breaking to water level and effective depth, particularly temporal variations induced by groupiness, may result in under-prediction of wave height by up to 100% within the inner surf zone.

This has implications for the design of nearshore structures which rely on accurate measures of breaking wave height and position. The methods of Hudson (1978), for example, contain a wave height cubed relation to armour rock sizing, indicating that small discrepancies in wave height may result in large discrepancy in required armour rock size.

The implications of the above-described temporal changes in water level for engineering design are negligible within the outer surf zone, but become more manifest within the mid to inner surf zone. Required rock sizing decreases inshore from the incipient break point due to dissipation of wave energy, regardless of the method used to calculate wave height. However, incorporation of group-induced variations in water level within breaker index formula means that design wave height decreases more slowly than predicted using a depth based on still or mean water level. This results in larger armour rock being required within the mid to inner surf zone. This discrepancy may range from 35% of rock size at incipient break point on a 5:1 slope to 25% on flatter 10:1 and 20:1 slopes.

So while results show that existing methods for wave height assessment and armour rock sizing near the outer surf zone remains valid and possibly conservative, rock sizing should not be decreased within the surf zone as rapidly as existing guidelines may predict. Caution should therefore be used in the design of structures wholly located within the mid to inner surf zone as water levels and consequently wave height may exceed those predicted by design guides.

9.1.5 Effects of wave groupiness on overtopping of emerged platforms

A detailed physical model study was undertaken based on field data collected at a number of popular or high-risk locations along the Greater Sydney Coastline. This study characterised the behaviour of wave groups impacting and overtopping coastal rock

platforms in order to derive predictive techniques for identifying potentially hazardous conditions. Hazard has been defined on the basis of on the anticipated maximum overtopping flow depth and velocity compared to existing guidelines for people exposed to floods. Prediction schemes are derived for the 8% instability hazard as a function of platform freeboard, wave height, period and distance from the edge of the platform. The 8% instability hazard is the level of hazard that would be anticipated to be exceeded (on average) by one in every 12 waves

Key findings of the study included:

- significant increases in hazard can be observed as a function of increasing wave period for the same wave height – this supports recent fatality analysis that the relative risk is much higher for long period waves;
- very rapid changes in overtopping flow occur as a function of platform freeboard. This means that on rising tide levels, overtopping rapidly increases to a point at which hazardous conditions are reached. Specifically, significant changes occur with only a change of 0.25 m in tide level. Under rising spring tide conditions on the NSW coastline, a change in 0.25 m tide level can occur in less than 30 minutes;
- hazard rapidly changes with distance from the platform edge. This change in hazard depends on whether overtopping occurs in a white water-or green water-type manner. Methods for determining the type of overtopping expected and guidance for the decline in hazard with distance from the platform edge for each have been quantified during this study;
- while very high overtopping values were noted for violent, white water-type overtopping events, little guidance is provided within existing literature on the tolerable overtopping flows in the presence of pulsating or highly aerated flows. Due to the low density of these aerated flows, higher threshold flows may be tolerable. However, understanding of this process is, at present, insufficient to better refine guidance values.

Specifically, wave groups were found to cause variation in maximum overtopping flow values of up to $\pm 35\%$ due to the spatial changes that occur within wave groups for the

same sea state conditions. A reduction in the threshold wave height of 30% for a specified level of hazard is therefore recommended in the presence of wave groups.

9.2 Recommendations for design guides

The Coastal Engineering Manual (USACE, 2003) states that that wave grouping and its consequences is of significant concern with armour layer damage generally attributed to higher waves associated with wave groups. However, the CEM acknowledges that there is little guidance and few practical formulae for use in practical engineering, with methods for evaluating the effects of groupiness stated to be beyond the scope of the Manual. This study has confirmed that wave groupiness may result in breaker indices which exceed those predicted by existing design guidelines within the mid to inner surf zone and which may result in under-design of coastal structures in these areas.

While the present study has not investigated sufficient group structures or bathymetric profiles to offer definitive recommendations for design in the presence of wave groups, certain trends have been noted, of which the practitioner should be aware during design. On steep bed slopes ($m > 0.05$), as were examined during this present study, these trends include:

1. groupiness within a wave-train has the ability to induce temporal fluctuations in water level within the surf zone exceeding the mean level, which incorporates set-up;
2. these fluctuations become more pronounced with distance shoreward across the surf zone;
3. by elevating water level, local water depths are similarly increased. This allows the propagation and breaking of larger waves than are predicted by design guidelines.

Recommendations are therefore that:

1. to conservatively quantify the largest wave possible within the surf zone, either the maximum possible depth, including the effects of temporally varying water level, must be assessed and used (with a possible under-prediction of up to 20% remaining), or an irregular wave approach (Goda, 1975, 2007) should be used with a coefficient A , suitably adjusted to include these effects across the surf zone;

2. resultant breaker index values should use a depth based on still water level for simplicity and robustness;
3. design wave conditions must therefore be known, even for structures within depth-limited environments, to enable the relative depth, or position within the surf zone to be determined.

Derivation of the above expressions was limited to wave groups of specific structure and steepness and to relatively steep ($\geq 1:20$), plane bed slopes and, as such, more extensive testing should be undertaken to confirm or modify the presented expressions before use in design.

9.3 Recommendations for future research

9.3.1 Experimental

Quantification of total energy convergence rates as wave groups in differing evolutionary states propagate onto sloping beds requires assessment of kinetic energy within wave forms together with potential energy. This may be achieved by measuring velocities within the water column along with surface elevation, using methods such as particle image velocimetry. These measurements may allow the prediction of breaking using the threshold convergence method (Song and Banner, 2002; Banner and Peirson, 2007) to be extended into shoaling water. This, combined with a probabilistic method for assessing the state of wave group evolution as the shoreline is approached, may allow for more robust predictors of breaking location and surf zone processes which depend on these breaking distributions.

9.3.2 Numerical

Numerical models such as Boussinesq and Navier-Stokes-based methods provide opportunity to assess the interaction of intra-group and shoaling processes across a wide range of group structures and bed profiles, including irregular bathymetries not achieved within the current investigation. Limited examples of this are provided within Song and Banner (2004) where a boundary element model (Drimer and Agnon, 1998) was used to model group evolution across a submerged sandbar, although model limitations prohibited examination of groups propagating in depths of less than $L_0/3$. More recently developed models such as volume of fluid type (i.e. Lin and Liu, 1998) or smooth

particle hydrodynamics models (i.e. Dalrymple and Rogers, 2006) may allow assessment of these interactive complex processes in shallower depths. Data acquired during the present investigation, or subsequent investigations quantifying the kinetic energy component, would allow calibration and verification of model performance and accuracy.

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**Appendix A. Table of published breaker index
criterion**

Reference	Type	Form	Function	Supplementary	Comment
Stokes (1880) ¹	Regular	Limiting steepness	Wave angle < 120°		Theoretically developed for oscillatory waves but also showed to hold for solitary
McCowan (1891) ¹	Regular	Breaker depth index	$\frac{H_b}{d_b} = 0.78$		Theoretically defined for solitary waves propagating over a horizontal bed
Michell (1893) ¹	Regular	Limiting steepness	$\left(\frac{H_0}{L_0}\right)_{\max} = 0.142$		Theoretically developed for deepwater oscillatory waves
Miche (1944) ¹	Regular	Limiting steepness	$\frac{H_b}{L_b} = 0.142 \tanh(kh)$		Theoretically developed for limiting steepness of waves propagating over a horizontal bottom in finite water depth
Munk (1949) ³	Regular	Breaker height index	$\frac{H_b}{H_0} = 0.3 \left(\frac{H_0}{L_0}\right)^{-1/3}$		Theoretically derived for solitary wave
Le Mehaute and Koh (1967) ²	Regular	Breaker height index	$\frac{H_b}{H_0} = 0.76 \left(\frac{H_0}{L_0}\right)^{-1/4} m^{1/7}$		Empirical, based on 3 sources of experimental data
Galvin (1969)	Regular	Breaker depth index	$\frac{H_b}{d_b} = 1.09$ $\frac{H_b}{d_b} = 0.71 - 1/(6.85m)$	$m > 0.07$ $m \leq 0.07$	Empirical based using new data with that of Iversen (1952)
Collins and Weir (1969) ²	Regular	Breaker depth index	$\frac{H_b}{d_b} = 0.72 + 5.6m$		Based on linear theory and empirically including slope effect from 3 sources of experimental data

Reference	Type	Form	Function	Supplementary	Comment
Goda (1970)	Regular		Best-fit curves produced to relate H_b/d_b to d_b/L_0 for 1:10, 20:1, 30:1, >50:1 slopes		Empirical curves derived from 3 laboratory sources
Weggel (1972)	Regular	Breaker depth index	$\frac{H_b}{d_b} = b - a \frac{H_b}{gT^2}$	$a = 43.8(1 - e^{-19 \tan \beta})$ $b = 1.56/(1 + e^{-19.5 \tan \beta})$ $\beta \leq 0.1$; $H_0/L_0 \leq 0.06$	Empirically derived from laboratory data for regular waves breaking on plane slope. Bounded by 0.78 on horizontal slope and twice this (1.56) as slope tends to infinity
Komar and Gaughan (1973)²	Regular	Breaker height index	$\frac{H_b}{H_0} = 0.56 \left(\frac{H_0}{L_0} \right)^{-1/5}$		Derived semi-empirically (coefficient 0.56 from field/laboratory data) from linear wave theory
Goda (1974, 2007)	Regular	Breaker height index	$\frac{H_b}{d_b} = \frac{A}{d_b/L_0} \left\{ 1 - \exp \left[-1.5\pi \frac{d_b}{L_0} (1 + 11s^{4/3}) \right] \right\}$	where $A = 17$	Conversion of graphical curves (Goda, 1970). Coefficient for bed slope revised by Goda (2007)
Battjes (1974)	Regular	Breaker height index	$\frac{H_b}{d_b} = 1.062 + 0.137 \log \xi_0$	for $0.02 \leq m < 0.2$	Empirical, based on previous laboratory data
Sunamura and Horikawa (1974)²	Regular	Breaker height index	$\frac{H_b}{H_0} = m^{0.2} \left(\frac{H_0}{L_0} \right)^{-1/4}$		Empirical, based on same data as Goda (1970)
Goda (1975)	Irregular	Breaker height index	$\left(\frac{H_{1/3,b}}{d_b} \right) = \frac{A}{d_b/L_0} \left\{ 1 - \exp \left[-1.5\pi \frac{(d_b)}{L_0} (1 + 11s^{4/3}) \right] \right\}$	$A = 0.12$ at incipient breaking, increasing to $A = 0.18$ as the relative water depth decreases	Empirical, based on previous laboratory data
Madsen (1976)²	Regular	Breaker depth index	$\frac{H_b}{d_b} = 0.72(1 + 6.4m)$		Combined formulas of Galvin (1969) and Collins and Weir (1969)

Reference	Type	Form	Function	Supplementary	Comment
Battjes and Janssen (1978) ²	Regular	Limiting steepness	$\frac{H_b}{L_b} = 0.14 \tanh\left(\frac{0.8}{0.88} \frac{2\pi d_b}{L_b}\right)$		Theoretical - modified Miche (1944)
Ostendorf and Madsen (1979)	Regular	Limiting steepness	$\frac{H_b}{L_b} = 0.14 \tanh\left[(0.8 + 5m) \frac{2\pi d_b}{L_b}\right]$ $\frac{H_b}{L_b} = 0.14 \tanh\left[(0.8 + 5(0.1)) \frac{2\pi d_b}{L_b}\right]$	for $m \leq 0.1$ for $m < 0.1$	Theoretical - modified Miche (1944) to include bed slope
Sakai and Battjes (1980) ⁵	Regular	Breaker height index	$H_b = H_0 \left[0.3839(H_0 / L_0)^{-0.3118}\right]$ $H_b = H_0 \left[0.6683(H_0 / L_0)^{-0.1686}\right]$ $H_b = H_0$	for $H_0 / L_0 < 0.0208$ for $0.0208 \leq H_0 / L_0 < 0.1$ for $0.1 \leq H_0 / L_0$	Based on theory of Cokelet (1977). Does not include effect of depth or bed slope
Sakai and Battjes (1980) ⁵	Regular	Breaker depth index	$H_b = H_0 \left[27429(d_b / L_0)^2 - 773.71(d_b / L_0) + 7.4343\right]$ $H_b = H_0 \left[0.3976(d_b / L_0)^{-0.3834}\right]$ $H_b = H_0 \left[21.867(d_b / L_0)^2 - 7.06(d_b / L_0) + 1.5573\right]$ $H_b = H_0$	for $d_b / L_0 < 0.011$ for $0.011 \leq d_b / L_0 < 0.049$ for $0.049 \leq d_b / L_0 < 0.6$ for $0.6 \leq d_b / L_0$	Based on theory of Cokelet (1977). Does not include effect of slope
Sunamura (1980) ²	Regular	Breaker depth index	$\frac{H_b}{d_b} = 1.1 \left(\frac{m}{\sqrt{H_0 / L_0}} \right)^{1/6}$		Based on existing laboratory data
Singamsetti and Wind (1980) ²	Regular	Breaker height and depth index	$\frac{H_b}{H_0} = 0.575m^{0.031} \left(\frac{H_0}{L_0} \right)^{-0.254}$ $\frac{H_b}{d_0} = 0.937m^{0.155} \left(\frac{H_0}{L_0} \right)^{-0.13}$	for $1/40 \leq m < 1/5$ for $0.02 \leq H_0 / L_0 < 0.065$	Empirical based

Reference	Type	Form	Function	Supplementary	Comment
Owen (1980)	Irregular	Breaker depth index	$H_{sb}/d_s = 0.58 - 2(d_s/gT_m^2)$ $H_{sb}/d_s = 0.66 - 10.583(d_s/gT_m^2) + 229.17(d_s/gT_m^2)^2$ $H_{sb}/d_s = 0.75 - 20.083(d_s/gT_m^2) + 479.17(d_s/gT_m^2)^2$ $H_{sb}/d_s = 0.95 - 38.417(d_s/gT_m^2) + 895.83(d_s/gT_m^2)^2$ $H_{sb}/d_s = 1.54 - 97.83(d_s/gT_m^2) + 2541.67(d_s/gT_m^2)^2$	for slope = 1:100 for slope = 1:50 for slope = 1:30 for slope = 1:20 for slope = 1:10	where H_{sb} is the upper limit of significant wave height in an depth of water
Thornton and Guza (1983)	Irregular	Breaker depth index	$\frac{H_{rms,b}}{d} = 0.43$ $\frac{H_{mo,b}}{d} = 0.6$		Applies within the saturated surfzone
Ogawa and Shuto (1984)²	Regular	Breaker height index	$\frac{H_b}{H_0} = 0.68m^{0.09} \left(\frac{H_0}{L_0} \right)^{-0.25}$	for $1/100 \leq m < 1/10$ and $0.003 \leq H_0 / L_0 < 0.065$	Empirical based on same data sets as Goda (1970)
Battjes and Stive (1985)²	Regular	Limiting steepness	$\frac{H_b}{L_b} = 0.14 \tanh \left\{ \left[0.5 + 0.4 \tanh \left(33 \frac{H_0}{L_0} \right) \right] \frac{2\pi d_b}{0.88 L_b} \right\}$		Modified Battjes and Janssen (1978)
Nelson (1985)	Regular	Breaker depth index	$\left(\frac{H}{d} \right)_{\max} = \frac{F_c}{22 + 1.82 F_c}$	where F_c is a non-linearity parameter given by $F_c = \left(\frac{H}{h} \right)^{1/2} \left(T \sqrt{\frac{g}{h}} \right)^{5/2}$	Based on field and laboratory data for a horizontal bed. Within shallow water $F_c > 500$ and $(H/d)_{\max}$ reduces to = 0.55
Massel (1996)	Regular	Breaker depth index	$\left(\frac{H}{d} \right)_{\max} < 0.6$	for $F_c < 1000$	Theoretically derived using Stokes 2 nd order theory for a horizontal bed

Reference	Type	Form	Function	Supplementary	Comment
Riedel and Byrne (1986)	Irregular	Breaker depth index	$\left(\frac{H}{d}\right)_{\max} < 0.55$		Empirically based on laboratory testing of irregular waves corresponding to a Pierson - Moskowitz spectrum
Seyama and Kimura (1988)²	Regular and irregular	Breaker depth index	$\frac{H_b}{d_b} = 0.16 \frac{L_0}{d_b} \left\{ 1 - \exp \left[-0.8\pi \frac{d_b}{L_0} (1 + 15m^{4/3}) \right] \right\} - 0.96m + 0.2$ $\frac{H_b}{d_b} = 0.2 \frac{L_0}{d_b} \left\{ 1 - \exp \left[-0.8\pi \frac{d_b}{L_0} (1 + 15m^{4/3}) \right] \right\} - 0.96m + 0.2$	for irregular waves for regular waves	Modified Goda (1974) to include irregular and regular wave data. Found breaker depth index for irregular waves around 20% lower than for regular
Larson and Kraus (1989)²	Regular	Breaker depth index	$\frac{H_b}{d_b} = 1.14 \left(\frac{m}{\sqrt{H_0 / L_0}} \right)^{0.21}$		Empirical based from laboratory data of Kajima et al (1982)
Hansen (1990)²	Regular	Breaker depth index	$\frac{H_b}{d_b} = 1.05 \left(m \frac{L_b}{d_b} \right)^{0.2}$		Empirical based from laboratory data of Van Dorn (1978)
Fenton and McKee (1990)⁵	Regular	Breaker depth index	$\frac{H_b}{d_b} = [0.141063(L_b / d_b) + 0.0095721(L_b / d_b)^2 + 0.0077829(L_b / d_b)^3] / [1 + 0.078834(L_b / d_b) + 0.0317567(L_b / d_b)^2 + 0.0093407(L_b / d_b)^3]$		Derived for horizontal bed. Slope is not considered
Smith and Kraus (1991)	Regular	Breaker depth and height index	$\frac{H_b}{d_b} = 1.12 / (1 + e^{-60m}) - 5.00(1 - e^{-43m}) \left(\frac{H_0}{L_0} \right)$	for $1/80 \leq m < 1/10$ and $0.0007 \leq H_0 / L_0 < 0.0921$ for $1/50 \leq m < 1/10$	Empirical based. Also derived formula for breaker indices over barred profiles which are outside the scope of the present study

Reference	Type	Form	Function	Supplementary	Comment
			$\frac{H_b}{H_0} = 0.34 + 2.47m \left(\frac{H_0}{L_0} \right)^{(0.30-0.88m)}$	and $0.0007 \leq H_0 / L_0 < 0.0921$	
Kamphuis (1991)²	Regular and irregular	Limiting steepness	$\frac{H_b}{L_b} = 0.095 \exp(4m) \tanh \left(\frac{2\pi d_b}{L_b} \right)$ $\frac{H_b}{L_b} = 0.127 \exp(4m) \tanh \left(\frac{2\pi d_b}{L_b} \right)$	for irregular waves for regular waves	Modified Miche (1944) to include bed steepness and calibrated for irregular and regular waves
Gourlay (1992)	Regular	Breaker height index	$\frac{H_b}{H_0} = 0.478 H_0 \left(\frac{H_0}{L_0} \right)^{-0.28}$	for $1/45 \leq m < 1/10$ and $0.001 \leq H_0 / L_0 < 0.066$	Empirical, based on several laboratory data sets
Komar (1998)⁵	Regular	Breaker height and depth index	$H_b = 0.39 g^{0.2} (T_w H_0^2)^{0.4}$ $d_b = H_b \left\{ 1.2 [S / (H_b / L_0)^{0.5}]^{0.27} \right\}$		
Song and Banner (2002, 2004)	Regular	Energy convergence threshold	$\delta(t) = 1.40 \pm 0.1 \times 10^{-3}$ where $\delta(t)$ is a non-dimensional mean growth rate parameter corresponding to the rate of wave energy convergence.	for waves in deep and intermediate ($>L_0/3$) depth.	Numerical based, confirmed by laboratory testing (Banner and Peirson, 2007). The presence of a sandbar found to increase threshold, possibility due to stabilisation by increased non-linearities. Testing in shallower water was not possible.
Rattanapitikon et al (2003)	Regular	Limiting steepness	$\frac{H_b}{L_b} = (-1.40m^2 + 0.57m + 0.23) \left(\frac{H_0}{L_0} \right)^{0.35}$		Empirical, based on fitting 26 sources of published laboratory data

Reference	Type	Form	Function	Supplementary	Comment
Le Roux (2007)	Regular	Breaker height and depth index	$H_b / L_b = 0.0625$ $d_b / L_b = 0.0749$ $H_b / d_b = 0.8333$		Iterative approach using 11 established formula used for fully developed sea on horizontal bed
Camenen and Larson (2007)	Regular	Breaker depth index	$\frac{H_b}{d_b} = \frac{0.284}{\sqrt{\lambda_0}} \tanh \left[f_*(m, \lambda_0) \pi \sqrt{\lambda_0} \right]$ where $f_*(m, \lambda_0) = A_1 + A_1 \sin \left[\frac{\pi}{2} \left(\frac{m}{0.1 + 1.6 \lambda_0} \right)^\alpha \right]$	where $\lambda_0 = H_0/L_0$ $A_1 = 0.87$ $A_2 = 0.32 + 14 \lambda_0$ $\alpha = 1 + 14 \lambda_0$ if $m \leq 0.1 + 1.6 \lambda_0$ $\alpha = -(1 + 20 \lambda_0)$ if $m > 0.1 + 1.6 \lambda_0$	Based on an empirical calibration using multiple laboratory data sets with shoaling according to 1 st order Stokes theory

Appendix B. Numerical routines for data processing

B-1 Optical LIF surface extraction code

B-2 Time series analysis code

B-1 Optical LIF surface extraction code

% BACKGROUND

%V1 Written by Tom Shand 12/06/07: time-stack, extracted series, found
%min, max, mean levels and plotted

%V2 03/07/07 added filter and zero downcrossing to determine wave properties

%V3 19/08/07 extracted information about the 1st - 6th waves including
%median height and height range & plotted these outputs.

%V3a 12/09/07 Added checks (i.e. steepness) to make sure actual values
%existed in series (i.e. were not obscured by column). Calculated steepness
%across columns by comparing elevations at same timesteps. Calculated velocity using twave.
%plotted these in additional figure. Removes filter and adds Goring and
%Nikora (2002) despiking routine

%V4 22/09/07 Adds ignore data file with xpositions obscured by column etc.

%06/09/07 Swapped wave median for wave maximum. Due to coarse temporal
%resolution really need to find max wave height to identify absolute max

%04/10/07 Swapped mmread for newer version which reads more vid files (i.e.
%mpg's etc)

%V4 14/10/07 Routine added for calculating wave speed and crest wave speed
%more accurately. Also data files on wave properties for each probe
%position saved.

%20/10/07 Added dynamic calibration to allow for perspective distortion and
%glass refractive effects

%26/10/07 Added correction (in stack2series) for pincushion distortion in lens

%11/02/08 V5 - new architecture to allow statial description of wave-form
% 1. extract x,y series of air-water interface from raw vid frame by
% frame using InterfaceExtract, mmread, processFrame
% 2. correct for perspective & optical distortion using 15 parameter
% distortion model within processFrame & resample to x integers

%18/02/08 V6 - separate programs for
%1. data extraction from video
%2. image correction
%3. data processing
%4. plotting

%March 2008 Added function to phaseshift sequential videos to extract
%complete profile information. This is presented as a x,t,y plot and may be
%viewed & saved as video using MakePhaseAdjSeriesVid

% EXTRACTING DATA FROM VIDEO FILES

%INITILISING

```

CrestPositions=['CrestPositions.dat'];
[crestposlist]=textread(CrestPositions,'%s',-1) %loads files 'CrestPositions.dat' %s indicates
white space between waves, -1 indicates read entire file
numcrestpos=length(crestposlist); %numfiles is number of entities in names
crestposnames=char(crestposlist); %convert names into character string (now '16
by 13' matrix)

for icrestpos = 1:numcrestpos %for 1 to however many strings have been read
crestfile=char(crestposlist(icrestpos)); %filename is 1st, 2nd etc character string from
names (1 by 13 matrix)
paramfilename=[crestfile,'Params.dat'];

global param;
param=paramfilename

[parameter, value] = textread(param, '%s %f', -1); %reads all input parameters from a dat file
[parameter, valuenam] = textread(param, '%s %s', -1);

global image_out; %Needs to be global so the stacker callback can save the data
global imframes; %Number of frames processed in the stack

CrestPosition=value(1); %crest position relative to paddle
CrestPositionName=valuenam(1);
CrestPositionName=char(CrestPositionName);
%CrestPositionName=mat2str(CrestPosition); %crest position name
slope=value(2); %bed slope
ColumnFrom=value(3); % Column in frame to stack from - usually 155
ColumnTo=value(4); % Column in frame to stack to - usually 555
IntensityThresh=value(5); % Intensity must be at least this in a column to record an
interface
DeriveThresh=value(6); % When this -ve derivative is reached, all following derivatives
become 0 - meant to stop false interfaces below actual being picked up
ROlupperdefault=value(7); %defining upper bound region of interest (pixels) if
boundary tracking method not used... - BE CAREFUL
ROlower=value(8); %defining lower bound region of interest (pixels) - BE CAREFUL
FrameFrom=value(9); % Frame to extract from - first is zero
FrameTo=value(10); % Frame to extract to - if <=Frame from, will automatically
extract all frames
resolution = value(11); %Approx resolution in mm/pixel
vid_frame_rate = value(12); %vid frame rate (Hz)
crestx100 = value(13); %x position in +100 vid
cresty100 = value(14); %y position in +100 vid
rotation = value(15); %Additional rotation required to get flat horizon (degrees: +ve is
clockwise)
crestposx=value(16); %x position of crest in pixels in 0.avi (use 0 unless otherwise)
laserellipse=value(17); %correction for laser convex shape (mm)
GroupT=value(18); %Group period in deep water
Tthresh=value(19); %defining minimum period for zero upcrossing
vidheight=value(20); %define height of vid in pixels
n = value(21); %n waves in group
umax = value(22); %threshold wave speed
umin=value(23); %lower threshold wave speed
Binthresh=value(24); %Threshold for turning intensity image into binary for
setting upper ROI
BP1=value(25); %breaking position (x from bed crest) of largest/critical wave
crest
WaveSpacing=value(27); %spacing (mm) between wave zero downcrossing
analyses
WaveT=value(28); %Wave period

```

```

BP2=value(29);
BP3=value(30);
sc=value(31);           %Additional scale adjustment
wavesingroup = n.*2;    %no of waves in each group temporally (i.e. 2 * n)

%READING FILES/COLUMNS

datafile=[CrestPositionName,'files.dat'];
[names]=textread(datafile,'%s',-1); %loads files 'vidpos.dat' %s indicates white space between
waves, -1 indicates read entire file
numfiles=length(names);           %numfiles is number of entities in names
fnames=char(names);               %convert names into character string (now '16 by 13'
matrix)

for ifile = 1:numfiles           %for 1 to however many strings have been read

filename=char(names(ifile));      %filename is 1st, 2nd etc character string from names (1 by
13 matrix)
filenameeavi=[filename,'.avi'];

Vidpos=cellstr(filename(7:10));    %read position of video rel to crest - 400mm, 800mm etc
Vidpos=str2double(Vidpos)

clear F
global F
F = struct();    %setting up a global variable for the getframe function if a video of extraction is
required

%Extracting data from vids & saving in data file
[VidSeries] = InterfaceExtract(filenameeavi);
seriesfilenamedat=[filename,'-series.dat'];
dlmwrite(seriesfilenamedat, VidSeries, 'delimiter', '\t', 'precision', 5);
% APPLYING IMAGE CORRECTIONS

%INITILISING
CrestPositions=['CrestPositions.dat'];
[crestposlist]=textread(CrestPositions,'%s',-1) %loads files 'CrestPositions.dat' %s indicates
white space between waves, -1 indicates read entire file
numcrestpos=length(crestposlist);           %numfiles is number of entities in names
crestposnames=char(crestposlist);           %convert names into character string (now '16
by 13' matrix)

%loading correction for trolley height data
trolleyheight=load('trolleyheight.dat');
flumexcorrection=(trolleyheight(1,1):0.25:trolleyheight(end,1));
flumeycorrection = interp1q(trolleyheight(:,1),trolleyheight(:,2),flumexcorrection);

for icrestpos = 1:numcrestpos           %for 1 to however many strings have been read
crestfile=char(crestposlist(icrestpos)); %filename is 1st, 2nd etc character string from
names (1 by 13 matrix)
%crestfilename=[crestfile,'files.dat']
paramfilename=[crestfile,'Params.dat']

global param;
param=paramfilename

[parameter, value] = textread(param, '%s %f', -1); %reads all input parameters from a dat file
[parameter, valuenam] = textread(param, '%s %s', -1);

```

```

CrestPosition=value(1);      %crest position relative to paddle
CrestPositionName=valuenam(1);
CrestPositionName=char(CrestPositionName);
slope=value(2);              %bed slope
ColumnFrom=value(3);         % Column in frame to stack from - usually 155
ColumnTo=value(4);           % Column in frame to stack to - usually 555
IntensityThresh=value(5);    % Intensity must be at least this in a column to record an
interface
DeriveThresh=value(6);       % When this -ve derivative is reached, all following derivatives
become 0 - meant to stop false interfaces below actual being picked up
ROIupperdefault=value(7);    %defining upper bound region of interest (pixels) if
boundary tracking method not used... - BE CAREFUL
ROIlower=value(8);           %defining lower bound region of interest (pixels) - BE CAREFUL
FrameFrom=value(9);          % Frame to extract from - first is zero
FrameTo=value(10);           % Frame to extract to - if <=Frame from, will automatically
extract all frames
resolution = value(11);      %Approx resolution in mm/pixel
vid_frame_rate = value(12);  %vid frame rate (Hz)
crestx100 = value(13);       %x position in +100 vid
cresty100 = value(14);       %y position in +100 vid
rotation = value(15);        %Additional rotation required to get flat horizon (degrees: +ve is
clockwise)
crestposx=value(16);         %x position of crest in pixels in 0.avi (use 0 unless otherwise)
laserellipse=value(17);      %correction for laser convex shape (mm)
GroupT=value(18);            %Group period in deep water
Tthresh=value(19);           %defining minimum period for zero upcrossing
vidheight=value(20);         %define height of vid in pixels
n = value(21);               %n waves in group
umax = value(22);            %threshold wave speed
umin=value(23);              %lower threshold wave speed
Bintthresh=value(24);        %Threshold for turning intensity image into binary for
setting upper ROI
BP1=value(25);               %breaking position (x from bed crest) oflargest/critical wave
crest
%BPtrough=value(26)          %trough position corresponding to above breaking wave
WaveSpacing=value(27);       %spacing (mm) between wave zero downcrossing
analyses
WaveT=value(28);             %Wave period
BP2=value(29);
BP3=value(30);
sc=value(31);                %Additional scale adjustment
wavesingroup = n.*2;         %no of waves in each group temporally (i.e. 2 * n)

%READING FILES/COLUMNS

datafile=[CrestPositionName,'files.dat'];
[names]=textread(datafile,'%s',-1); %loads files 'vidpos.dat' %s indicates white space between
waves, -1 indicates read entire file
numfiles=length(names);      %numfiles is number of entities in names
fnames=char(names);          %convert names into character string (now '16 by 13'
matrix)

for ifile = 1:numfiles        %for 1 to however many strings have been read

filename=char(names(ifile));  %filename is 1st, 2nd etc character string from names (1 by
13 matrix)
filenamedat=[filename,'-series.dat'];

```

```

Vidpos=cellstr(filename(7:10));      %read position of video rel to crest - 400mm, 800mm etc
Vidpos=str2double(Vidpos)

%Extracting data from vids & saving in data file
[VidSeries]=load(filename+dat);
xinframe=VidSeries(:,1);

[x_length, t_length] = size(VidSeries);
t_length=t_length-1;

datain=[crestx100 cresty100];
[modelx]=textread('modelx.dat');      %reads in 15 modelx parameters (defined by
earlier Impot script)
correctedzero = xform( datain, modelx, rotation, sc );      %corrects the frame 'zero'

approxresolution=datain(1)/correctedzero(1);      %calculates an approximate resolution
approxresolution=round(approxresolution);
approxresolution=1/approxresolution;      %adjusts to divisor of 1 (i.e. 0.25, 0.2, 0.33)

correctedxzero=correctedzero(1)/approxresolution;
correctedxzero=round(correctedxzero);
correctedxzero=correctedxzero*approxresolution;

resamplex=VidSeries(:,1).*approxresolution;      %calculates an idealised x
resamplex=resamplex-correctedxzero;      %according to the frame zero
correctedseries=[resamplex];

for t=1:t_length
    series=[VidSeries(:,1) VidSeries(:,(t+1))];
    testNaN=isnan(VidSeries(:,(t+1)));
    testNaN=sum(testNaN);
    if testNaN<=(length(VidSeries(:,(t+1)))-1)

%Correction model
seriesout = xform( series, modelx, rotation, sc );      %adjust x",y" to x',y' using model
seriesout(any(isnan(seriesout)),:)= [];

%Shift to '+100 zero'
correctedx=seriesout(:,1)-correctedzero(1);
correctedy=seriesout(:,2)-correctedzero(2);

%Despike using %despike using phasce-space method by Goring and Nikora(2002.
%Have corrected so it removes the spike's contribution to the mean (used to
%offset corrected data)
[correctedy_despike] = func_despike_phasespace( correctedy);
if sum(isnan(correctedy_despike))>=(length(correctedy_despike)/10)
else
correctedy_despike(isnan(correctedy_despike)) = interp1(find(~isnan(correctedy_despike)),
correctedy_despike(~isnan(correctedy_despike)), find(isnan(correctedy_despike)), 'linear');
%interpolating across NaN vales (From E. Rodriguez 05/08/2005)
correctedy=correctedy_despike;
end

elevation_series = interp1q(correctedx,correctedy,resamplex); %resamples to integer of x to
allow timestacking using linear interpolation

%Correction for elliptical lasersheet
laserlength=(abs(resamplex(end)-resamplex(1)));      %finding length of laser sheet

```

```

lasermidpos=resamplex(round(length(resamplex)./2));
for i=1:(length(resamplex))
    angle=atan(490/(94-(elevation_series(i))));
    ellipsecorrect=laserellipse.*(sin(pi.*((resamplex(i)-resamplex(1))./laserlength)));
    convexcorrection=(cos(angle).*ellipsecorrect)./2;
    elevation_series_ellipsecorrect(i)=elevation_series(i)-convexcorrection;
end

else
    elevation_series_ellipsecorrect=(VidSeries(:,(t+1)))';
end

crestx=Vidpos-100-resamplex;      %x in flume w.r.t. crest
absx=CrestPosition-crestx;      %abs x in flume
crestposk = dsearchn(flumexcorrection,CrestPosition);
CrestPosTrolleyCorrection=flumeycorrection(crestposk);

tempflumeycorrection=flumeycorrection-CrestPosTrolleyCorrection;
k = dsearchn(flumexcorrection,absx(1));
tempflumeycorrection=tempflumeycorrection(k:k+(length(absx)-1));
elevation_series_ellipsecorrect=elevation_series_ellipsecorrect-tempflumeycorrection';

figure(1)
maintitle = mat2str([FrameFrom+t-1]);
maintitle = ['FrameNo-',maintitle];
plot(resamplex,elevation_series_ellipsecorrect);
set(gca,'YLim',[-40 60]);
set(gca,'XLim',[resamplex(1) resamplex(end)]);
ylabel('elevation (mm)');
xlabel('x (mm)');
grid on
title(maintitle);
drawnow;
F(t) = getframe(gcf);

correctedseries=[correctedseries elevation_series_ellipsecorrect'];

end
correctedfilenamedat=[filename,'-correctedseries.dat']
dlmwrite(correctedfilenamedat, correctedseries, 'delimiter', '\t', 'precision', 5);
end
end

```

B-2 Time series analysis

```

%INITILISING
CrestPositions=['CrestPositions.dat'];
[crestposlist]=textread(CrestPositions,'%s',-1) %loads files 'CrestPositions.dat' %s indicates
white space between waves, -1 indicates read entire file
numcrestpos=length(crestposlist); %numfiles is number of entities in names
crestposnames=char(crestposlist); %convert names into character string (now '16
by 13' matrix)

%For each crest position
for icrestpos = 1:numcrestpos %for 1 to however many strings have been read
crestfile=char(crestposlist(icrestpos)); %filename is 1st, 2nd etc character string from
names (1 by 13 matrix)
%crestfilename=[crestfile,'files.dat']
paramfilename=[crestfile,'Params.dat']
outputvidfilename=[crestfile,'WaveVid.avi']

global param;
param=paramfilename

[parameter, value] = textread(param, '%s %f', -1); %reads all input parameters from a dat file
[parameter, valuenam] = textread(param, '%s %s', -1);

CrestPosition=value(1); %crest position relative to paddle
CrestPositionName=crestfile;
slope=value(2); %bed slope
ColumnFrom=value(3); % Column in frame to stack from - usually 155
ColumnTo=value(4); % Column in frame to stack to - usually 555
IntensityThresh=value(5); % Intensity must be at least this in a column to record an
interface
DeriveThresh=value(6); % When this -ve derivative is reached, all following derivatives
become 0 - meant to stop false interfaces below actual being picked up
ROIupperdefault=value(7); %defining upper bound region of interest (pixels) if
boundary tracking method not used... - BE CAREFUL
ROIlower=value(8); %defining lower bound region of interest (pixels) - BE CAREFUL
FrameFrom=value(9); % Frame to extract from - first is zero
FrameTo=value(10); % Frame to extract to - if <=Frame from, will automatically
extract all frames
resolution = value(11); %Approx resolution in mm/pixel
vid_frame_rate = value(12); %vid frame rate (Hz)
crestx100 = value(13); %x position in +100 vid
cresty100 = value(14); %y position in +100 vid
rotation = value(15); %Additional rotation required to get flat horizon (degrees: +ve is
clockwise)
crestposx=value(16); %x position of crest in pixels in 0.avi (use 0 unless otherwise)
laserellipse=value(17); %correction for laser convex shape (mm)
GroupT=value(18); %Group period in deep water
Tthresh=value(19); %defining minimum period for zero upcrossing
vidheight=value(20); %define height of vid in pixels
n = value(21); %n waves in group
umax = value(22); %threshold wave speed
umin=value(23); %lower threshold wave speed
Binthresh=value(24); %Threshold for turning intensity image into binary for
setting upper ROI
BP1=value(25); %breaking position (x from bed crest) oflargest/critical wave
crest

```

```

%BPtrough=value(26)           %trough position corresponding to above breaking wave
WaveSpacing=value(27);        %spacing (mm) between wave zero downcrossing
analyses
WaveT=value(28);              %Wave period
BP2=value(29);
BP3=value(30);
sc=value(31);                  %Additional scale adjustment
plotxfrom=value(32);
plotxto=value(33);
specdensupper=value(34);

ExcludeCrestEnd=1;            %timeseries to ignore at start of frame
ExcludeDeepEnd=1;             %timeseries to ignore at end of frame
outputspacing=30;             %average timeseries characteristics over...

wavesingroup = n.*2           %no of waves in each group temporally (i.e. 2 * n)

BP1char=num2str(BP1)BP2char=num2str(BP2)BP3char=num2str(BP3)BP1diffold=10000;
BP2diffold=10000;BP3diffold=10000;

%READING FILES/COLUMNS

datafile=[CrestPositionName,'files.dat'];
[names]=textread(datafile,'%s',-1) %loads files position of videos %s indicates white space
between waves, -1 indicates read entire file
a=(1:length(names))';
[a,tags]=sort(a,'descend');
names=names(tags)               %reverse order of vid processing - deep to shallow
numfiles=length(names);        %numfiles is number of entities in names
frames=char(names);            %convert names into character string (now '16 by 13'
matrix)

%establishing arrays
timeseriesdata=[];zerodowndata=[];fourierdata=[];maximumwaves=[];framephasingspeed=[];
yphaseshifted=[];yloppassphaseshifted=[];fouriershete=[];Hphaseshifted=[];Findex=1;

%Determining size of array to hold phase adjusted y data from all vids & initialising
testfile=char(names(1));       %filename is 1st, 2nd etc character string from names (1 by 13
matrix)
testfilenamedat=[testfile,'-correctedseries.dat'];
[testopen]=load(testfilenamedat);
testopen=size(testopen)
PhaseAdjSeries=zeros((testopen(1).*length(a)),testopen(2));
xposition=1;
interframemintoffsetadjlist=[];

%For each video
for ifile = 1:numfiles         %for 1 to however many strings have been read

    filename=char(names(ifile)) %filename is 1st, 2nd etc character string from names (1
by 13 matrix)
    correctedfilenamedat=[filename,'-correctedseries.dat']

    Vidpos=cellstr(filename(7:10)); %read position of video rel to crest - 400mm, 800mm etc
    Vidpos=str2double(Vidpos)

    %Extracting data from vids & saving in data file
    [VidSeries]=load(correctedfilenamedat);

```

```

xinframe=VidSeries(:,1);          %x in frame w.r.t. 'zero'
crestx=Vidpos-100-xinframe;       %x in flume w.r.t. crest
absx=CrestPosition-crestx;       %abs x in flume

[x_length, t_length] = size(VidSeries);
t_length=t_length-1
ColumnSpacing=(WaveSpacing./(abs(xinframe(2))-xinframe(1))));

timeseriesdatatemp=[];
zerodowndatatemp=[];
fourierdatatemp=[];
maximumwavestemp=[];

%Calculating phase offsets between videos
x1=(1+(ColumnSpacing.*ExcludeDeepEnd)):ColumnSpacing:(length(xinframe)-
(ColumnSpacing.*ExcludeCrestEnd));
quarterx=(floor(length(x1)/4));
xdeepindex=x1(quarterx);
xdeep=crestx(xdeepindex);
ydeep=[VidSeries(xdeepindex,2:end)]';
tseries=(0:(1./vid_frame_rate):((t_length./vid_frame_rate)-(1./vid_frame_rate)));
    if ifile > 1

[interframemindiff,interframemintoffset]=PhaseOffset(tseries,yshallow,tseries,ydeep,GroupT)
    interxdiff=xshallow-xdeep
    interframemintoffsetadj=interframemintoffsetadj+(
(intraframemintoffset./intraxdiff.*interxdiff)- interframemintoffset)
    else
    interframemintoffsetadj=0
    end
    interframemintoffsetadjlist=[interframemintoffsetadjlist;Vidpos,interframemintoffsetadj];
xshallowindex=x1(end-(quarterx-1));
xshallow=crestx(xshallowindex);
yshallow=[VidSeries(xshallowindex,2:end)]';
%Calculating phase difference within frame & average wave speed based
%on phase diff
[intraframemindiff,intraframemintoffset]=PhaseOffset(tseries,ydeep,tseries,yshallow,GroupT)
intraxdiff=xdeep-xshallow
phasingspeed=(intraxdiff./1000)/abs(intraframemintoffset);
framephasingspeed=[framephasingspeed;((xdeep+xshallow)./2),(CrestPosition-
(xdeep+xshallow)./2)),phasingspeed];

%Phase adjusting series & saving
for x=1:length(crestx)
    tseriesoff=tseries-interframemintoffsetadj;
    yi = interp1(tseriesoff,VidSeries(x,(2:end)),tseries,'linear');
    PhaseAdjSeries(xposition,:)= [crestx(x),yi]';
    xposition=xposition+1;
end

framemedspeeds=[];

%Extracting spatially discrete timeseries for analysis
for x=(1+(ColumnSpacing.*ExcludeDeepEnd)):ColumnSpacing:(length(xinframe)-
(ColumnSpacing.*ExcludeCrestEnd))
    orderedx=(floor(x/(WaveSpacing./(abs(xinframe(2))-xinframe(1))))) +1;
    yseries=[VidSeries(x,2:end)];
    tseries=0:(1./vid_frame_rate):((t_length./vid_frame_rate)-(1./vid_frame_rate));

```

```

testNaN=isnan(yseries);
testNaN=sum(testNaN);

%Making sure not more than 1/2 series is NaN
if testNaN<=(length(yseries)./4)
    GroupTsample = GroupT.*(vid_frame_rate);
    leny=length(yseries);
    NoGroups=leny/GroupTsample;
    NoGroupsRounded=floor(NoGroups);
    %NoGroupsRounded=3;

    %Making sure there is at least 1 group in sample (could be
    %prob in Case 3 vids!!)
    if NoGroupsRounded >=1
        skipatstart=5;

y_round=yseries(skipatstart:(floor(skipatstart+(NoGroupsRounded.*GroupTsample))));
t_round=tseries(skipatstart:(floor(skipatstart+(NoGroupsRounded.*GroupTsample))));

%Low-pass filtering timeseries
[b,a] = cheby1(9,0.05,1/10); %design filter passing freq <1Hz
y_lowpass=filtfilt(b,a,yseries);
y_lowpass(1:100)=[NaN];
y_lowpass(end-100:end)=[NaN];
y_lowpassmax=max(y_lowpass);
y_lowpassmin=min(y_lowpass);

[S]=allstats(y_round);
WLmean=S.mean; %finds mean surface elevation in series
discounting NaN
zmin=S.min; %finds min crest level
zmax=S.max; %finds max crest level
kurt=S.kurt;
skew=S.skew;
%mode=S.mode;
std=S.std;
var=S.var;
%More can be obtained from S

meany_round=nanmean(y_round);
y_round_zeroed=y_round-meany_round;
y_round_zeroed_sq=y_round_zeroed.^2;
mean_y_round_zeroed_sq=nanmean(y_round_zeroed_sq);
Ho=4.*(sqrt(mean_y_round_zeroed_sq));

twavemedianspeed=[NaN];

%ZERO DOWNCROSSING TO OBTAIN WAVE PROPERTIES
[H,twave,crest,trough,T, cresttime, troughtime]=zerodown(y_round',t_round',Tthresh);
%extracts individual wave heights by zero down-crossing relative to mean WL

%Calc wave stats
Hmean=mean(H); %finds mean wave height
Hrms=sqrt(sum(H.^2)/length(H)); %Calc RMS wave H
sH=sort(H);
Ny=length(sH);

```

```

N3=floor(2*Ny/3)+1;
N10=floor(.9*Ny)+1;
H3perc=mean(sH(N3:Ny));
H10perc=mean(sH(N10:Ny));
Hmax=max(H);
Tmean=mean(T);

%Calculating median wave speeds based on zero crossing positions
if crestx(x) < (crestx((1+(ColumnSpacing.*ExcludeDeepEnd))))
    try

[twavespeed,twavemaxspeed,twavemedianspeed]=CalcSpeeds(oldtwave,twave,oldxpos,crestx(
x),umax,umin);    %Calculating speed based around middle of crossing (twave)
    catch
        twavemedianspeed=[NaN];
    end
else
    twavemedianspeed=[NaN];
end

oldtwave=twave;
oldxpos=crestx(x);
waveoutput=[H,twave,crest,trough,T, cresttime, troughtime];

BP1diff=abs(crestx(x)-BP1);
BP2diff=abs(crestx(x)-BP2);
BP3diff=abs(crestx(x)-BP3);

if BP1diff<20
if BP1diff<BP1diffold
    BP1timeseries=[crestx(x),crestx(x);tseries',yseries'];
    BP1lowpassseries=[crestx(x),crestx(x);tseries',ylowpass'];
    BP1waves=waveoutput;
    BP1diffold=BP1diff;
end
end

if BP2diff<20
if BP2diff<BP2diffold
    BP2timeseries=[crestx(x),crestx(x);tseries',yseries'];
    BP2lowpassseries=[crestx(x),crestx(x);tseries',ylowpass'];
    BP2waves=waveoutput;
    BP2diffold=BP2diff;
end
end

if BP3diff<20
if BP3diff<BP3diffold
    BP3timeseries=[crestx(x),crestx(x);tseries',yseries'];
    BP3lowpassseries=[crestx(x),crestx(x);tseries',ylowpass'];
    BP3waves=waveoutput;
    BP3diffold=BP3diff;
end
end

%Carrying out Fourier Analysis on series, e is spectral width, Hmo & He are measures of
Hsig and k is spectral shape parameter
%[f,Sf,phay,Ts,band]=FourierAnalysis(t_round,y_round);

```

```

[f,Sf,Ts,e,Hmo,He,k]=FourierAnalysis(t_round,y_round);

if isnan(Sf)>=1
Sf=zeros(1,length(Sf));
Sf=Sf./Sf;
end

fouriersheet=[fouriersheet;crestx(x),Sf(2:end)];

    if isnan(Ts)>=1
        Ts=[NaN]
    end
    if isnan(e)>=1
        e=[NaN]
    end
    if isnan(Hmo)>=1
        Hmo=[NaN]
    end
    if isnan(He)>=1
        He=[NaN]
    end
    if isnan(k)>=1
        k=[NaN]
    end

%Finding depth from SWL (Slope needs to be correct for this to
%work properly!!!
    if crestx(x)<=-0.0001
        depth=428;
    elseif (crestx(x)./slope)>395
        depth=428;
    else depth=(crestx(x)./slope);
    end

%Calculating theoretical radiation stress/setdown - L-H&S64
w=2*pi/WaveT;
dw=1./vid_frame_rate;
h=depth./1000;
%determining wavenumber k according to Hunt 1979
%k=Hunt(T,h);
g=9.81;
twopi=2*pi;
X1=(twopi./WaveT).^2;
X2=(h.*X1)./g;

X3=1+X2.*(0.66667+X2.*(0.3555+X2.*(0.16084+X2.*(0.0632+X2.*0.02174+X2.*(0.00654+X2.*(
0.00171+X2.*(0.00039+X2.*0.00011))))));
X4=(1./X3)+X2;
C=sqrt(h.*g./X4);
k=twopi./(C.*WaveT);
nrad=0.5*(1+2*k*h/sinh(2*k*h));
factor=2*nrad-0.5;
Ht=H./1000;
E=(1025.*g.*Ht.^2)./8;
Sxx=E.*factor;
Cg=nrad*(g*WaveT/(2*pi))*tanh(k*h);
aa=-Sxx./(1025*(g*h-Cg^2));
aa=aa-mean(aa);

```

```

aa=aa.*1000;
aamwl=aa+WLmean;
aamax=max(aa);
aamin=min(aa);

Hdswl_all = H./depth;
Hdmwl_all = H./(depth+WLmean);           %H/d ratio w.r.t. mean WL

hold off

Lo=(1.56.*WaveT.^2).*1000;
xswx=crestx(x)./Lo;
xdashswx=CrestPosition./Lo;
depthLo=depth./Lo;

F(Findex) = getframe(gcf);
Findex=Findex+1;

%extracting maximum values for each wave in group
[wavemaxvalues] = wavemaximums(waveoutput,wavesingroup);

H1=wavemaxvalues(1,3);T1=wavemaxvalues(1,4);c1=wavemaxvalues(1,5);tr1=wavemaxvalues
(1,6);range1=wavemaxvalues(1,7);

H2=wavemaxvalues(2,3);T2=wavemaxvalues(2,4);c2=wavemaxvalues(2,5);tr2=wavemaxvalues
(2,6);range2=wavemaxvalues(2,7);

H3=wavemaxvalues(3,3);T3=wavemaxvalues(3,4);c3=wavemaxvalues(3,5);tr3=wavemaxvalues
(3,6);range3=wavemaxvalues(3,7);

H4=wavemaxvalues(4,3);T4=wavemaxvalues(4,4);c4=wavemaxvalues(4,5);tr4=wavemaxvalues
(4,6);range4=wavemaxvalues(4,7);

H5=wavemaxvalues(5,3);T5=wavemaxvalues(5,4);c5=wavemaxvalues(5,5);tr5=wavemaxvalues
(5,6);range5=wavemaxvalues(5,7);

H6=wavemaxvalues(6,3);T6=wavemaxvalues(6,4);c6=wavemaxvalues(6,5);tr6=wavemaxvalues
(6,6);range6=wavemaxvalues(6,7);

[H,tags] = sort(H);           %sorting H by size
Hlength=length(H);           %number of waves in sample
twave=twave(tags);crest=crest(tags);trough=trough(tags);T=T(tags);
Hmax = H(Hlength);           %finds max wave height in series
Cmax=crest(Hlength);         %finds crest elevation assoc with Hmax
Tmin=trough(Hlength);        %finds trough elevation assoc with
Hmax

Hrangeingroup = range(wavemaxvalues(:,3));

Hdswl = Hmax./depth;           %H/d ratio w.r.t SWL
Hdmean = Hmax./(depth+WLmean); %H/d ratio w.r.t. mean WL

timeseriesdatatemp=[timeseriesdatatemp;crestx(x),absx(x),depth,WLmean,zmax,zmin,kurt,ske
w,std,var,Ho,ylowpassmax,ylowpassmin];

zerodowndatatemp=[zerodowndatatemp;crestx(x),absx(x),Hmax,Cmax,Tmin,Hmean,Hrms,H3p
erc,H10perc,Tmean,twavemedianspeed,Hdswl,Hdmean,aamax,aamin];
fourierdatatemp=[fourierdatatemp;crestx(x),absx(x),Hmo,He,Ts,e,k];

```

```

        maximumwavestemp=[maximumwavestemp;crestx(x), depth, H1, T1, c1, tr1, range1,
H2, T2, c2, tr2, range2, H3, T3, c3, tr3, range3, H4, T4, c4, tr4, range4, H5, T5, c5, tr5, range5,
H6, T6, c6, tr6, range6];

```

```

        if crestx(x)<0
            depth=428;
        elseif (crestx(x)./slope)>395
            depth=428;
        else depth=(crestx(x)./slope);
        end

```

```

    end

```

```

    averageover=outputspacing./WaveSpacing;

```

```

    averagenumber=floor(length(x1)/averageover);

```

```

    for i=1:averagenumber

```

```

        timeseriesdataPart=timeseriesdatatemp((averageover.*i)-(averageover-1):averageover.*i,:);

```

```

        timeseriesdata=[timeseriesdata;nanmedian(timeseriesdataPart)];

```

```

        zerowndataPart=zerowndatatemp((averageover.*i)-(averageover-1):averageover.*i,:);

```

```

        zerowndata=[zerowndata;nanmedian(zerowndataPart)];

```

```

        fourierdataPart=fourierdatatemp((averageover.*i)-(averageover-1):averageover.*i,:);

```

```

        fourierdata=[fourierdata;nanmedian(fourierdataPart)];

```

```

        maximumwavesPart=maximumwavestemp((averageover.*i)-(averageover-
1):averageover.*i,:);

```

```

        maximumwaves=[maximumwaves;nanmedian(maximumwavesPart)];

```

```

    end

```

```

    timeseriesdatatemp=[];

```

```

    zerowndatatemp=[];

```

```

    fourierdatatemp=[];

```

```

    maximumwavestemp=[];

```

```

end

```

```

%Naming & writing dat files

```

```

timeseriesdatadat=[CrestPositionName,'-timeseriesdata.dat'];

```

```

dlmwrite(timeseriesdatadat, timeseriesdata, 'delimiter', '\t', 'precision', 7);

```

```

zerowndatadat=[CrestPositionName,'-zerowndata.dat'];

```

```

dlmwrite(zerowndatadat, zerowndata, 'delimiter', '\t', 'precision', 7);

```

```

fourierdatadat=[CrestPositionName,'-fourierdata.dat'];

```

```

dlmwrite(fourierdatadat, fourierdata, 'delimiter', '\t', 'precision', 7);

```

```

maximumwavesdat=[CrestPositionName,'-maximumwaves.dat'];

```

```

dlmwrite(maximumwavesdat, maximumwaves, 'delimiter', '\t', 'precision', 7);

```

```

framephasingspeeddat=[CrestPositionName,'-framephasingspeed.dat'];

```

```

dlmwrite(framephasingspeeddat, framephasingspeed, 'delimiter', '\t', 'precision', 7);

```

```

yphaseshifteddat=[CrestPositionName,'-yphaseshifted.dat'];

```

```

dlmwrite(yphaseshifteddat, yphaseshifted, 'delimiter', '\t', 'precision', 7);

```

```

ylopassphaseshifteddat=[CrestPositionName,'-ylopassphaseshifted.dat'];

```

```

dlmwrite(ylopassphaseshifteddat, ylopassphaseshifted, 'delimiter', '\t', 'precision', 7);

```

```

fouriersheetdat=[CrestPositionName,'-fouriersheet.dat'];

```

```

dlmwrite(fouriersheetdat, fouriersheet, 'delimiter', '\t', 'precision', 7);

```

```

PhaseAdjSeriesdat=[CrestPositionName,'-PhaseAdjSeries.dat'];

```

```

dlmwrite(PhaseAdjSeriesdat, PhaseAdjSeries, 'delimiter', '\t', 'precision', 7);

```

```

BP1timeseriesdat=[CrestPositionName,'-',BP1char,'-BP1timeseries.dat'];

```

```

dlmwrite(BP1timeseriesdat, BP1timeseries, 'delimiter', '\t', 'precision', 7);

```

```

BP1lopassseriesdat=[CrestPositionName,'-',BP1char,'-BP1lopassseries.dat'];

```

```

dlmwrite(BP1lopassseriesdat, BP1lopassseries, 'delimiter', '\t', 'precision', 7);

```

```
BP1wavesdat=[CrestPositionName,'-',BP1char,'-BP1waves.dat'];
dlmwrite(BP1wavesdat, BP1waves, 'delimiter', '\t', 'precision', 7);
BP2timeseriesdat=[CrestPositionName,'-',BP2char,'-BP2timeseries.dat'];
dlmwrite(BP2timeseriesdat, BP2timeseries, 'delimiter', '\t', 'precision', 7);
BP2lowpassseriesdat=[CrestPositionName,'-',BP2char,'-BP2lowpassseries.dat'];
dlmwrite(BP2lowpassseriesdat, BP2lowpassseries, 'delimiter', '\t', 'precision', 7);
BP2wavesdat=[CrestPositionName,'-',BP2char,'-BP2waves.dat'];
dlmwrite(BP2wavesdat, BP2waves, 'delimiter', '\t', 'precision', 7);
BP3timeseriesdat=[CrestPositionName,'-',BP3char,'-BP3timeseries.dat'];
dlmwrite(BP3timeseriesdat, BP3timeseries, 'delimiter', '\t', 'precision', 7);
BP3lowpassseriesdat=[CrestPositionName,'-',BP3char,'-BP3lowpassseries.dat'];
dlmwrite(BP3lowpassseriesdat, BP3lowpassseries, 'delimiter', '\t', 'precision', 7);
BP3wavesdat=[CrestPositionName,'-',BP3char,'-BP3waves.dat'];
dlmwrite(BP3wavesdat, BP3waves, 'delimiter', '\t', 'precision', 7);
```

```
moviename=[CrestPositionName,'-TimeseriesandH.avi'];
movie2avi(F,moviename,'fps',20)
```

```
end
```

%FIGURES

EXCLUDED FROM APPENDIX

Appendix C. Data extraction and visualisation examples for a C2N5r class wave group propagated on a plane 10:1 slope bed

- C-1 Exemplar outputs from a time series video sequence**
- C-2 Example of extracted characteristics**
- C-3 Example of the extracted space-time series for surface elevation and lowpass surface elevation**
- C-4 Example of the extracted frequency-space energy spectra**

Examples outputs for a Case 2, $N = 5$, recurrent wave group propagating on a plane 10:1 slope bed with x_0 at $x'/L_0 = 16.65$ from the paddle are presented within this Appendix. The development of surface elevation and wave characteristics (Section 4.3) with distance along the wave flume is observed by creating either a video series of sequential characteristic data (Figure C-1) or by plotting individual characteristics as a function of horizontal distance (Figure C-2). Additionally, the merged series of surface elevations and low-pass filtered surface elevations are combined into single space-time plots as described within Section 2.3.1.4 (Figure C-3). The energy spectrum is combined similarly as a function of both space and frequency (Figure C-4).

The breaking positions of the three largest waves within the group are indicated on Figure C-2 and occur, in their wave-train order, at $x'/L_0 = 16.27, 15.5$ and 16.1 or at $x_{swl}/L_0 = 0.38, 1.15$ and 0.55 from the still water intersection.

C-1 Exemplar outputs from a time series video sequence.

Figure C-1 shows examples of the evolution with distance along the flume of the:

- surface elevation, low-pass surface elevation and individual wave heights (upper panel);
- surface elevation and H/d ratios with respect to still and mean water level (middle panel);
- spectral density (lower left panel);
- observed wave height distribution compared to an idealised Rayleigh distribution (low right panel).

Examples have been selected at the following positions with respect to the bed-still water level intersection (x_{swx}/L_0) and depths relative to still water level (d_{swl}/L_0):

Wave groups can be observed to move between positions of crest maxima (e.g. $x_{swx}/L_0 = 3.27$) and trough minima (e.g. $x_{swx}/L_0 = 2.78$) within deep water. Individual wave forms can be observed to change in shape as they shoal and approach break point with increasingly vertical front faces. Following breaking, the front faces of wave forms are near vertical as they propagate across the surf zone. Groupiness is reduced within the

surf zone (e.g. $x_{swx}/L_0 = 0.18$) as larger waves break offshore, dissipating their energy earlier and diminishing in height. Far within the surf zone (e.g. $x_{swx}/L_0 = 0.00$) groupiness is increases slightly, possibly due to wave regeneration as suggested by Goda (2007).

Table C-1 **Locations of exemplar outputs from a time series video sequence for a Case 2, $N = 5$ type wave group propagating onto a 10:1 slope bed.**

Example number	x_{swx}/L_0	d/L_0	Notes
1	3.27	0.33	
2	2.78	0.28	
3	2.32	0.23	
4	1.85	0.19	
5	1.38	0.14	
6	1.14	0.14	Break point position of wave 2
7	1.00	0.10	
8	0.89	0.09	
9	0.70	0.07	
10	0.54	0.05	Break point position of wave 3
11	0.42	0.04	Break point position of wave 1
12	0.29	0.03	
13	0.18	0.02	
14	0.06	0.01	
15	0.00	0.00	

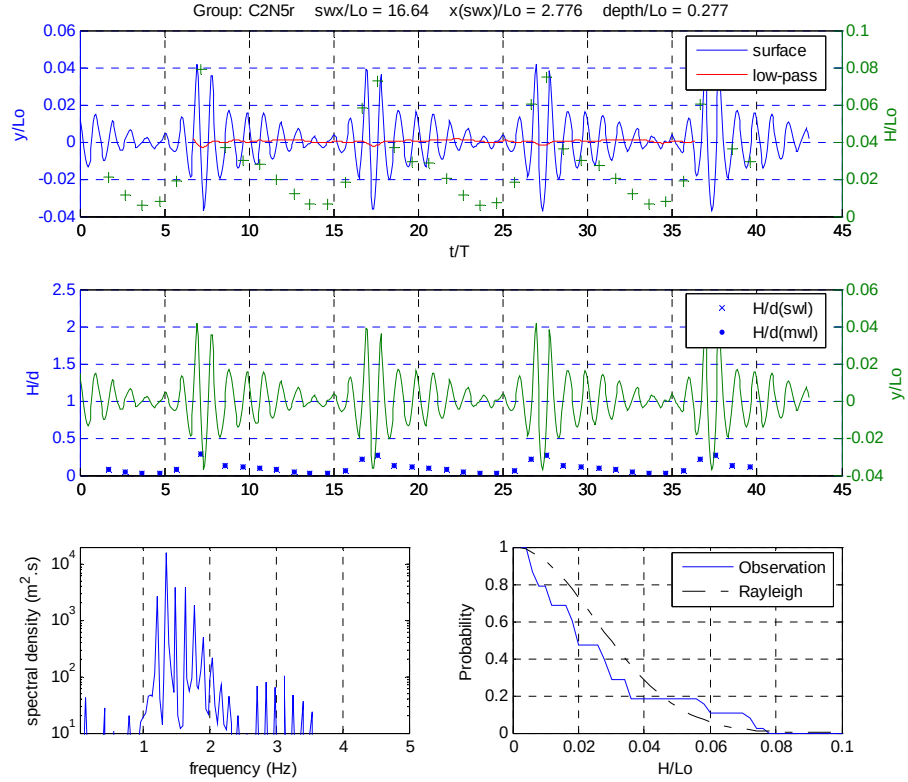
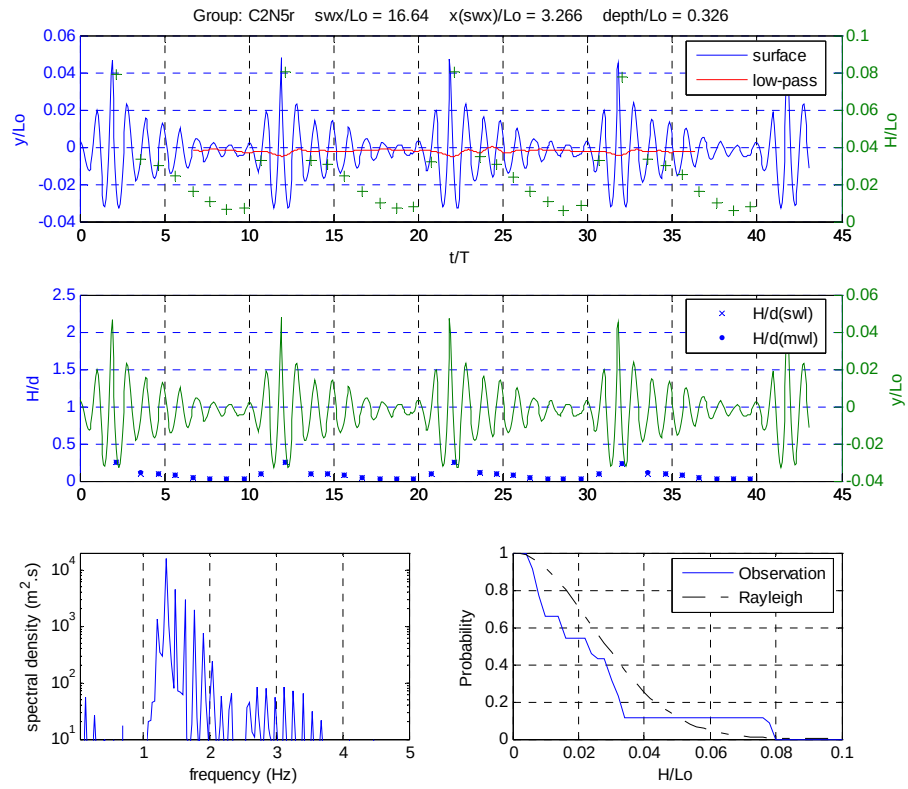
The long wave accompanying (and bound to) the wave groups is most evident as a trough in deep water, associated with the increased radiation stress of the largest waves. During shoaling the amplitude of the bound long wave increases, with a crest preceding the trough becoming evident at $x_{swx}/L_0 = 1.14$, or at around the break point of the largest wave (wave 2). The long wave amplitude continues to increase through the surf zone, particularly the crest height, with the trough becoming subdued, particularly from $x_{swx}/L_0 = 0.29$. From this point the long wave becomes increasingly sinusoidal, extending along the entire wave group length with the crest coincident with, or slightly preceding, the largest waves. The entire long wave is elevated above the still water level inside $x_{swx}/L_0 = 0.18$ illustrating the set up occurring within the surf zone, although the temporal nature of this set up in the presence of wave groups is apparent.

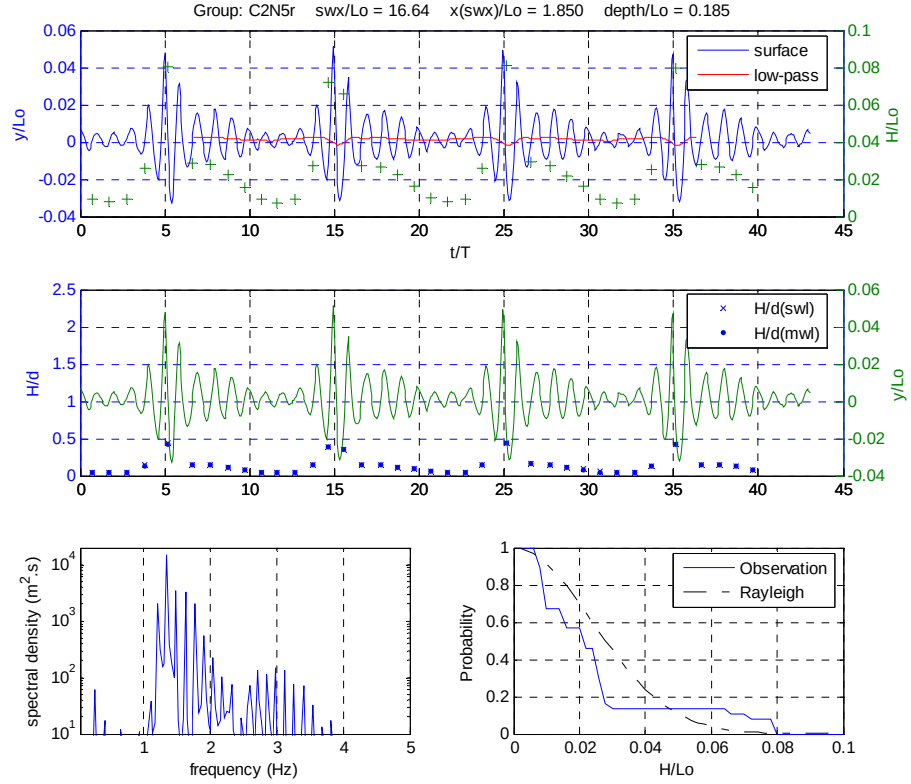
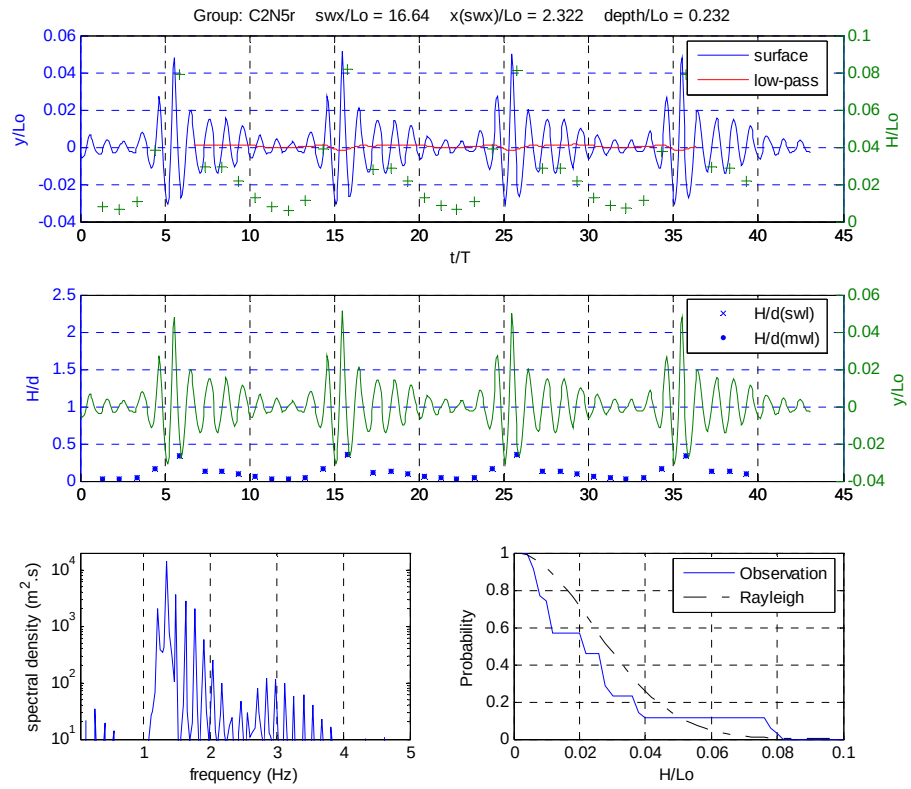
The H/d ratios according to still water level (swl) and mean water level (mwl) can be observed to increase as the wave groups shoal, break and propagate into the surf zone, with the largest ratios associated with the largest waves. Differences between the two

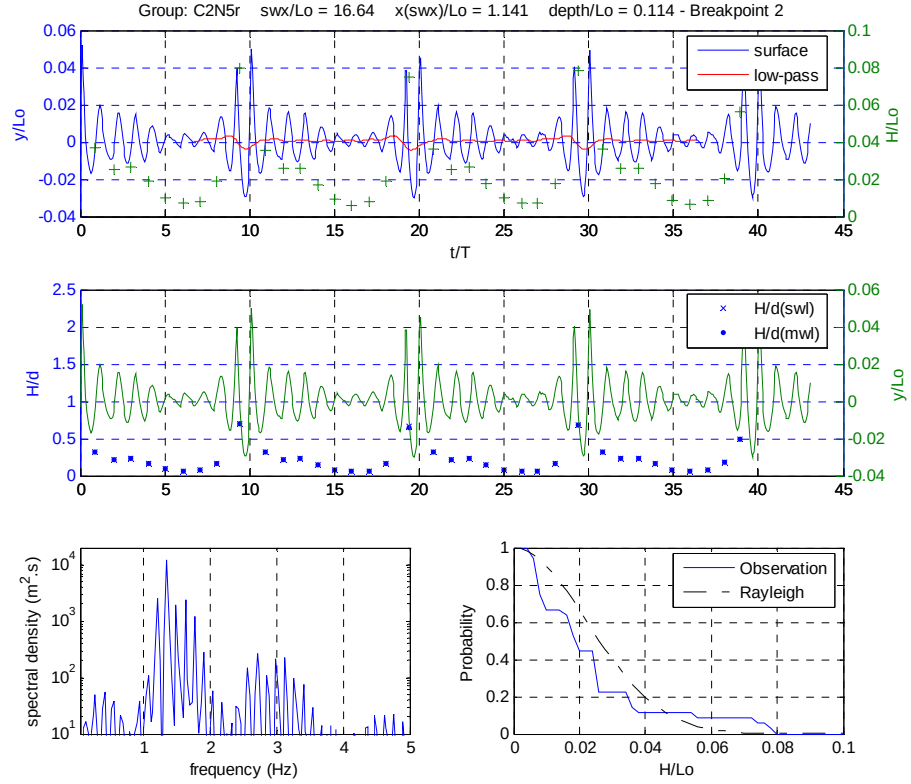
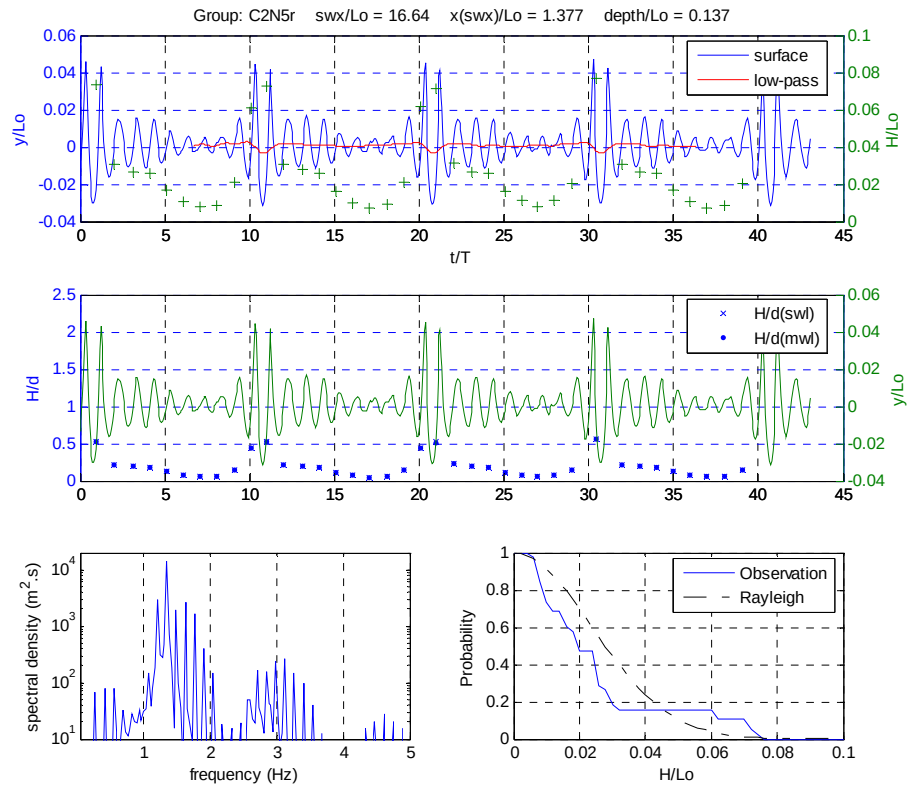
ratios are negligible until $x_{swx}/L_0 = 0.29$ when the H/d_{swl} begins to rise above the H/d_{mwl} . This appears to be due to increases in the water level due to radiation stresses which are not taken into account in the H/d_{swl} and thus, a shallower depth than actually occurs being used to define H/d . At the shallowest depth, the H/d_{swl} ratios of all waves are above the plot limits as the $depth_{swl}$ tends to zero.

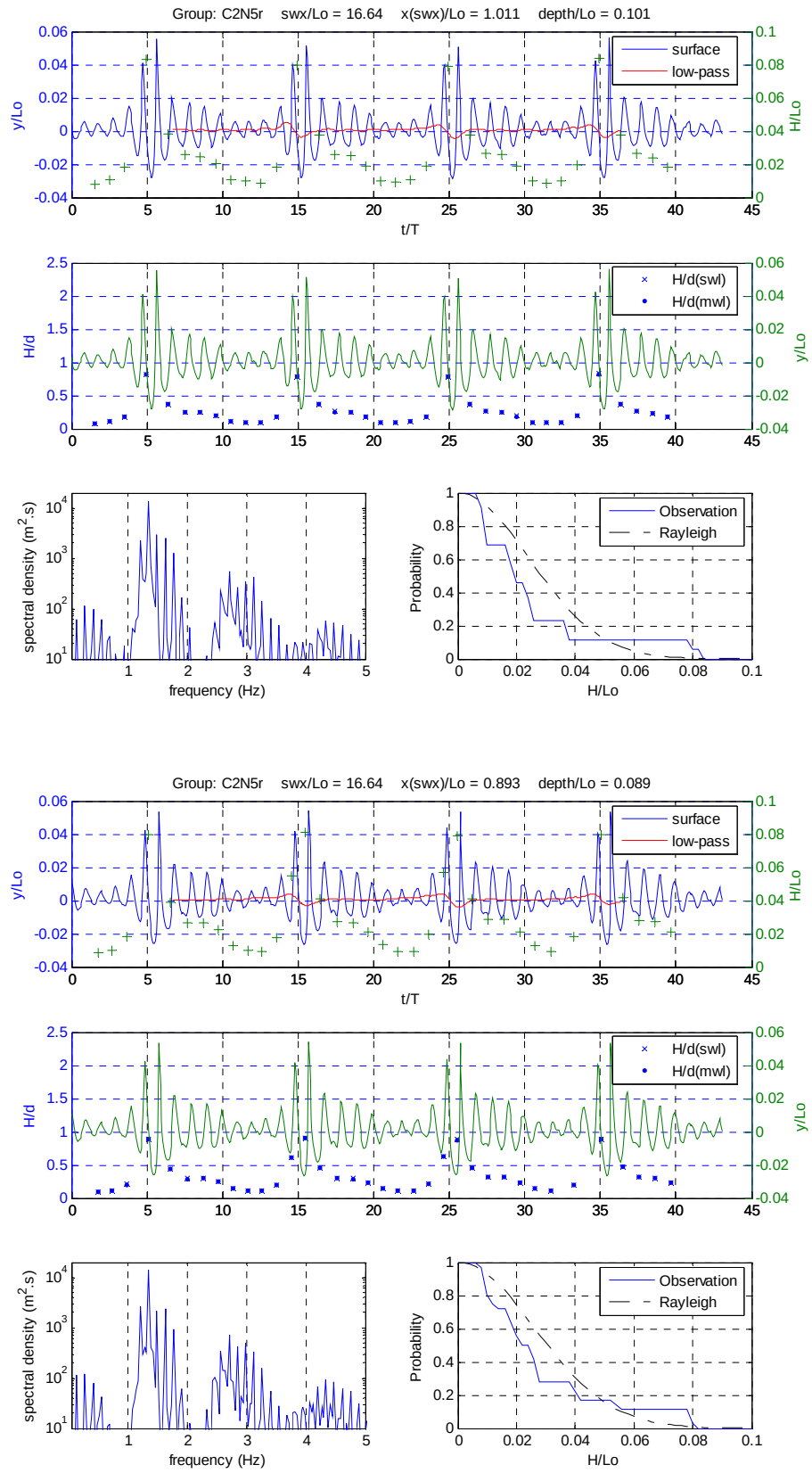
Peak spectral energy in deep water is occurs at around $f = 1.3$ Hz, although high energy is also contained within adjacent frequency bins. From $x_{swx}/L_0 = 1.85$, spectral energy within the low frequency component ($f < 0.5$ Hz) and first harmonic ($f = \sim 2.6$ to 3 Hz) is observed to increase, with development of a second harmonic ($f = \sim 4.5$ to 5 Hz) from $x_{swx}/L_0 = 1.38$. The spectral peak is observed to begin to decrease substantially from $x_{swx}/L_0 = 0.29$, with the low frequency energy exceeding the $f = 1.3$ Hz energy from $x_{swx}/L_0 = 0.06$.

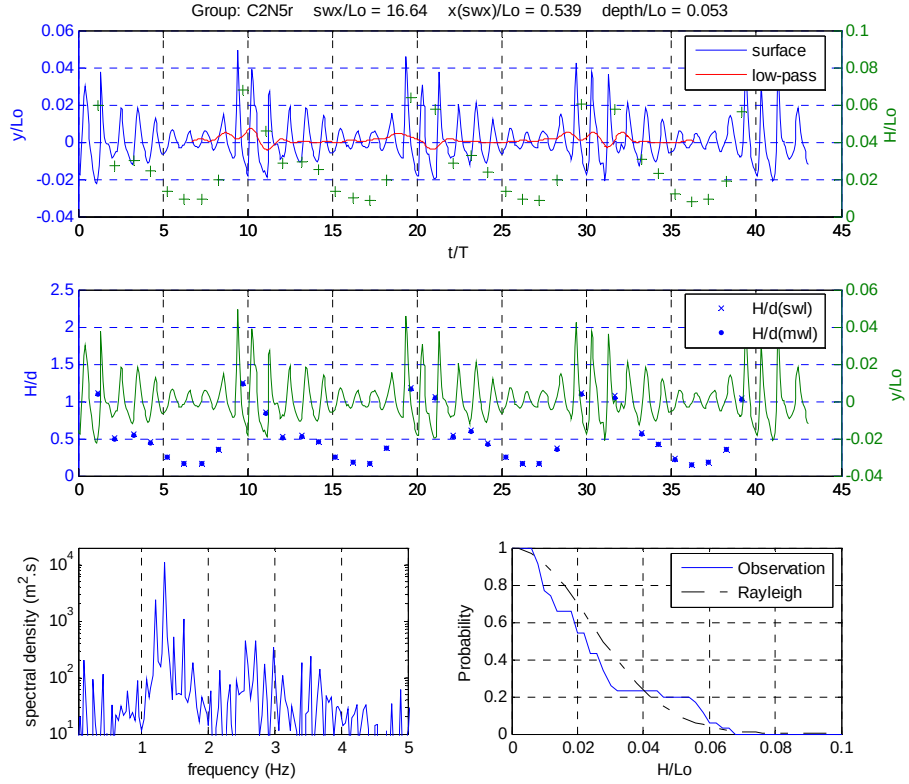
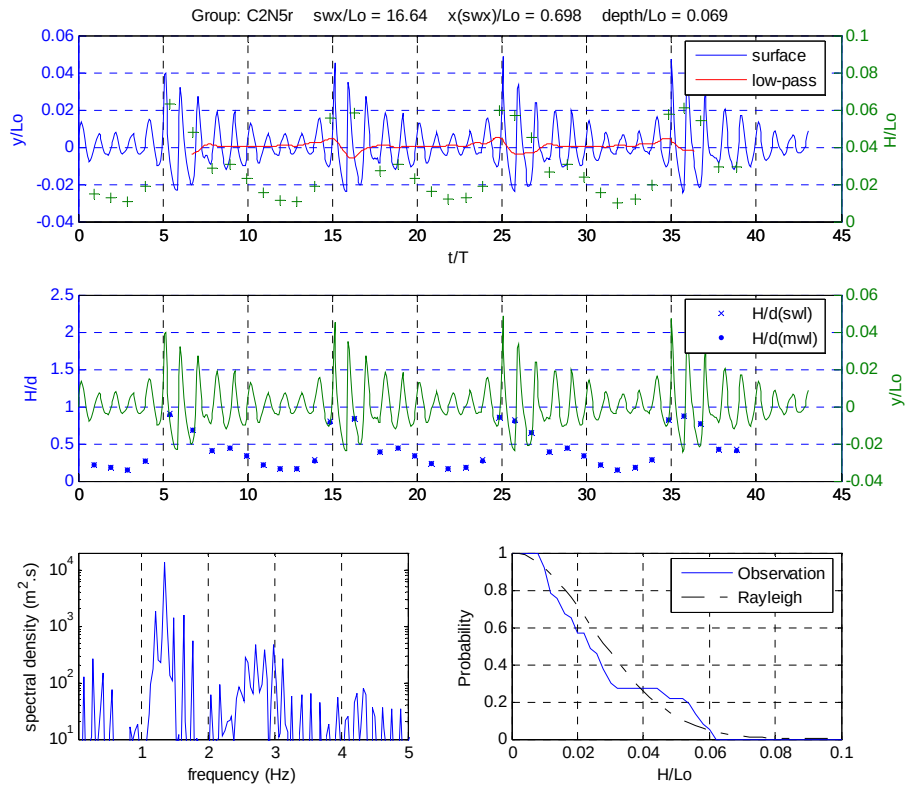
The wave height distribution is not well approximated by the Rayleigh distribution in deep water, with an under-representation of smaller waves and an over-representation in the tail. This general pattern is maintained until within the surf zone at around $x_{swx}/L_0 = 0.70$ when the largest waves become significantly truncated and the Rayleigh distribution becomes better approximated. From $x_{swx}/L_0 = 0.18$ however, the distribution begins to exhibit an under-representation of larger waves and an over-representation of smaller, indicating waves are tending towards a uniform height and a reduction in groupiness.

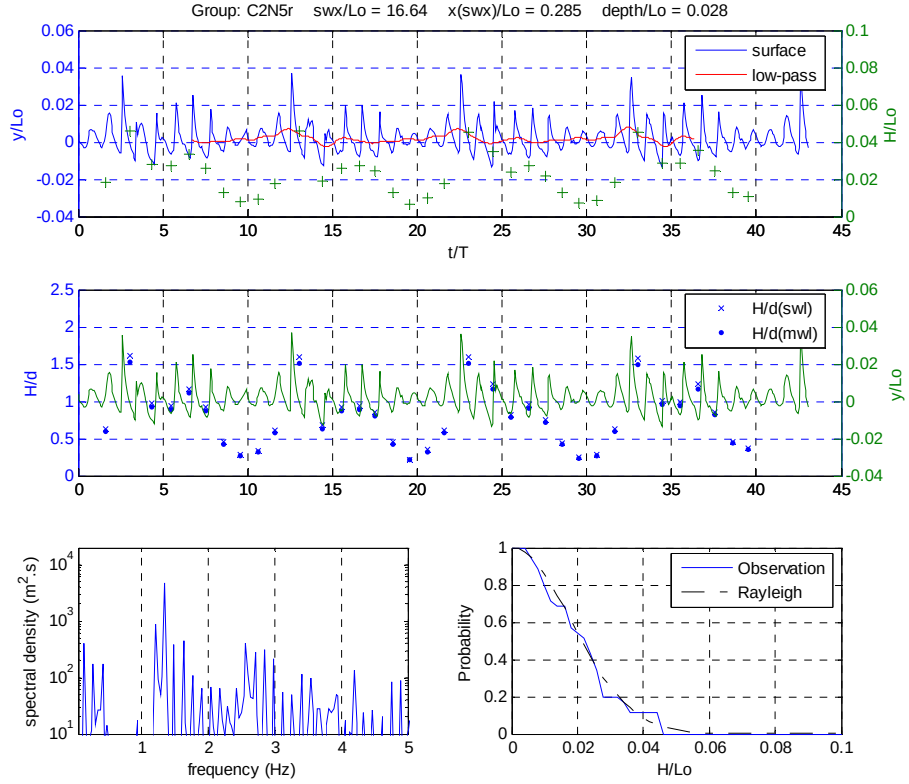
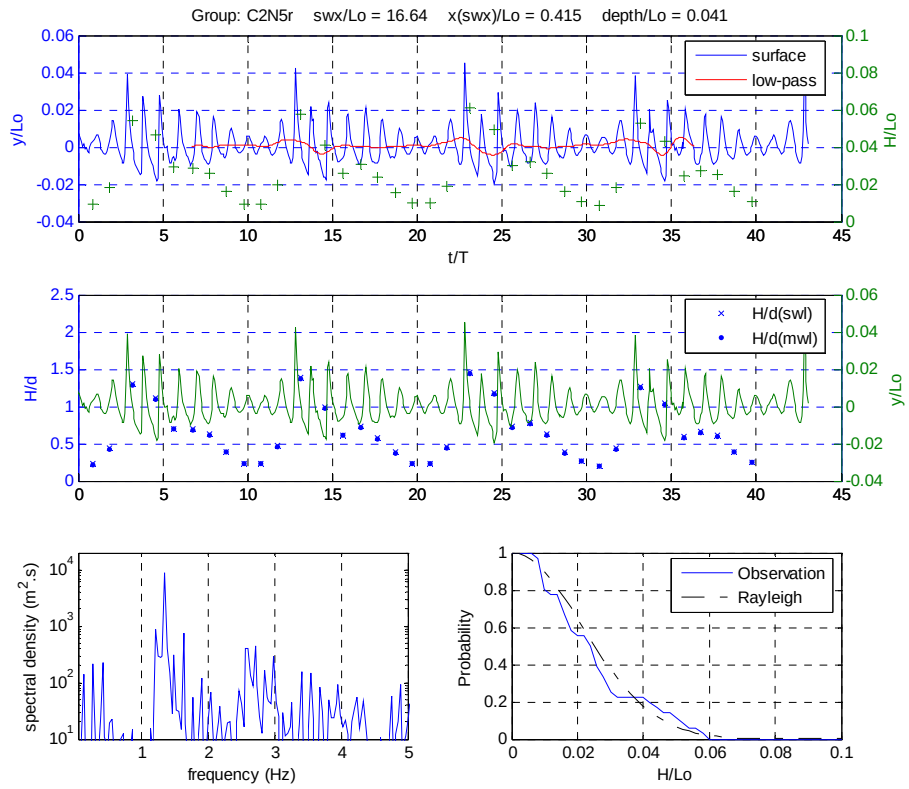


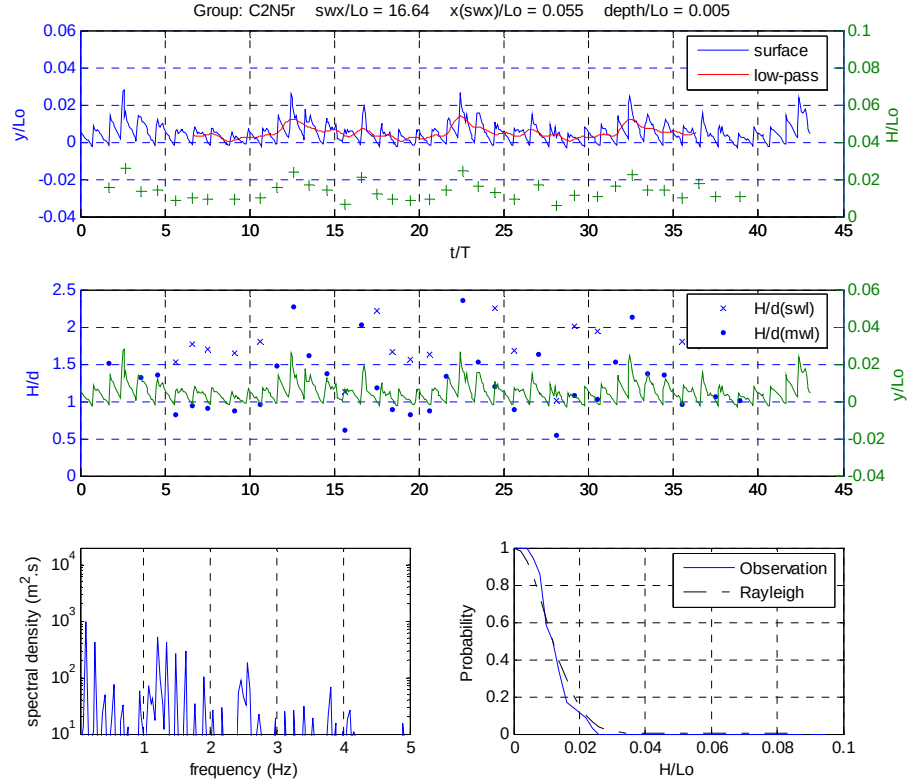
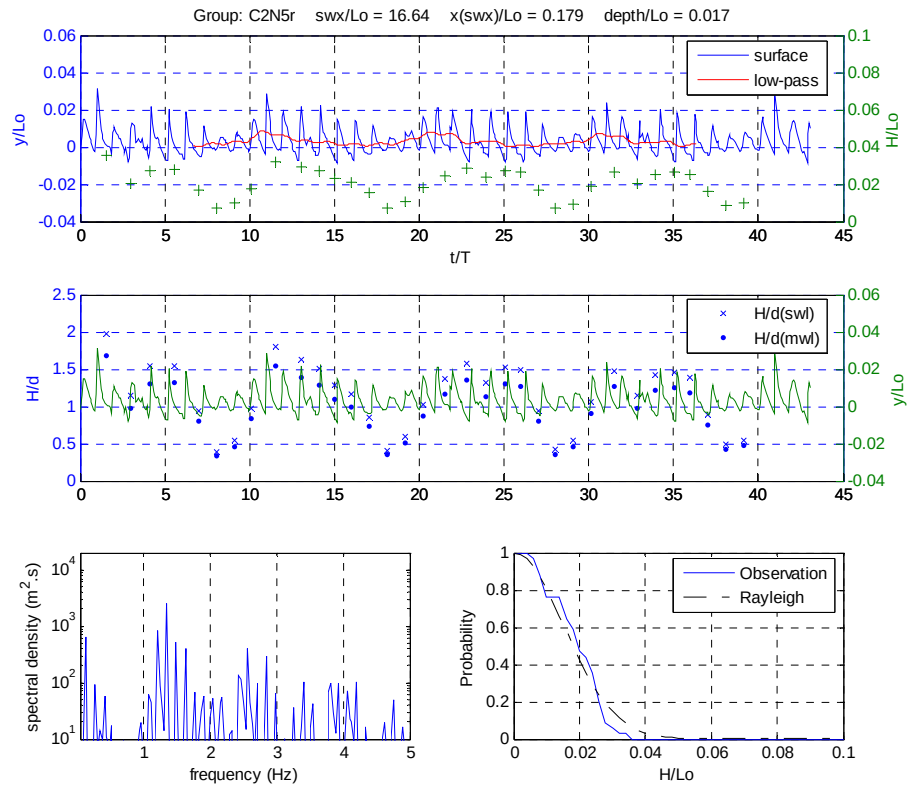












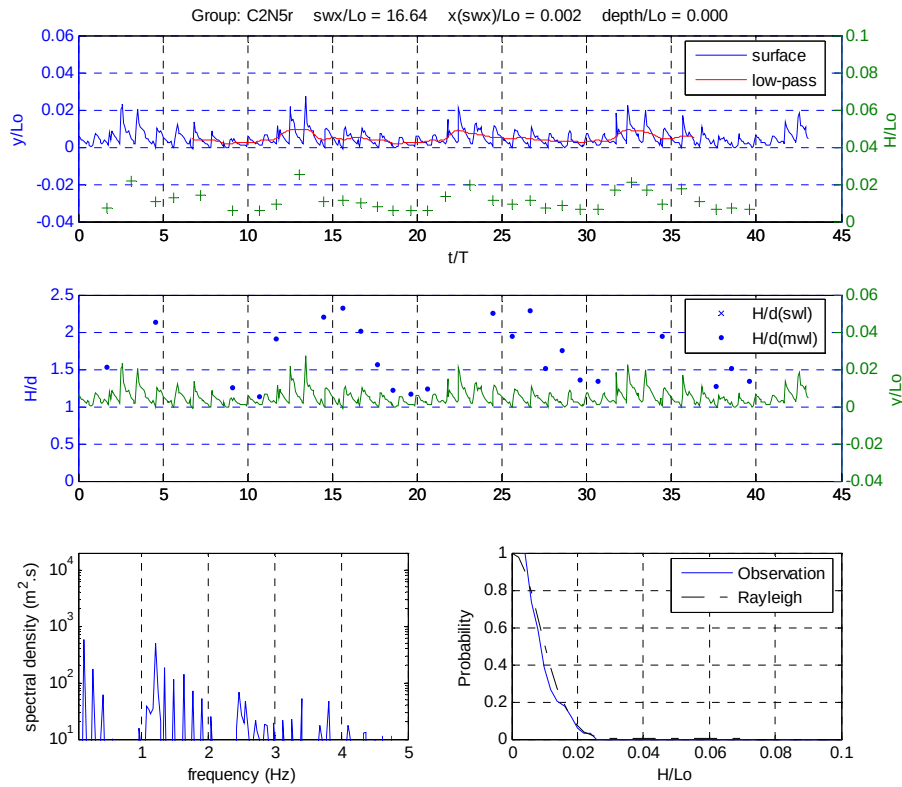


Figure C-1 Exemplar outputs from a time series video sequence for a Case 2, N = 5 type wave group propagating onto a 10:1 slope bed.

C-2 Example of extracted characteristics.

Figure C-2 depicts extracted water surface, wave and $H_{\max}/d$ characteristics for a Case 2, N = 5 type wave group propagating onto a 10:1 slope bed. The upper panel shows the change in mean water level, normalised by deep water wave length (L_0) compared to water depth. A reduction in mean water level evident from $x/L_0 = 14.75$ to 16.25 as the wave group shoals and momentum flux (radiation stress) is directed in an offshore direction (Section 2.3.3.1). From $x/L_0 = 16.25$ to the swx at $x/L_0 = 16.65$, mean water level increases more substantially to balance the momentum flux directed in an onshore direction. This decrease and subsequent increase in mean water level is commonly known as set down and set up (Longuet-Higgins and Stewart, 1962, 1964). Of interest, the mean water level continues to decrease following breaking of the largest wave at $x/L_0 = 15.5$ indicating that the overall momentum flux remains directed in an offshore direction even though some breaking and dissipation is occurring. The increase in mean water level offshore of $x/L_0 = 14.0$ is not explained by theory and is suspected due to

slight variance in the flume-top trolley elevation along the flume compared to still water level. While effort has been made to quantify and remove this variance (Section 4.2.1.2), some residual variance may remain.

The second panel shows the evolution of the maximum and energy based significant wave height ($H_{\max}$ and H_0 respectively) and the maximum and minimum surface elevation ($z_{\max}$ and $z_{\min}$ respectively), normalised by L_0 , during shoaling and breaking. Also, the breaking locations of the three largest waves within the group, as defined by the front face of the wave form becoming vertical and depicted by +, o and $\square$ (break point of waves 1, 2 and 3) are included. While some irregularities and drop-outs are evident related to side wall obstructions and similar interferences, for example the maximum wave height at $x/L_0 = 15.2$, the overall evolutionary patterns of the characteristics are well represented.

The water surface elevations $z_{\max}$ and $z_{\min}$ show patterns of local convergences and divergences offshore, although this pattern appears to become subdued or retarded inshore of around $x/L_0 = 15.0$. This is further discussed within Chapter 5. $z_{\max}$ rises sharply from $x/L_0 = 15.3$ to 15.7 before decreasing in a step-like fashion, possibly associated with the breaking locations of individual waves within the surf zone. The maximum surface elevation $z_{\min}$, decreases more uniformly from around $x/L_0 = 14.9$ indicating it may be less affected by individual wave shoaling and breaking. Maximum wave height generally reflects the patterns of $z_{\max}$ with an increase from $x/L_0 = 15.3$ to 15.7. Of interest, maximum wave height continues to increase after the break point (as defined by a vertical front wave face) of the largest wave has been reached. This is likely related to the post-breaking splash up causing further water level increase. The implications of this for defining breaker indices and engineering design are further discussed within Chapter 6. Energy based significant wave height is observed to decrease slightly from deep water to $x/L_0 = 15.8$, most likely related to sidewall and other frictional losses (Section 6.4.3). Furthermore, this indicates that the energy based significant wave height is largely unaffected by local energy converges and divergences. From $x/L_0 = 15.8$ to the swx at $x/L_0 = 16.65$, the energy based significant wave height exhibits a relatively linear decrease, similarly indicating little dependence on individual wave shoaling and breaking.

Panel three depicts the evolution of the $H_{\max}/d_{\text{swl}}$ and $H_{\max}/d_{\text{mwl}}$ ratios. An expected increase associated with decreasing water depth is observed from deep water until around $x/L_0 = 15.7$, when wave height sharply reduces following the breaking of the largest wave. At this point, the ratios stabilise temporarily before increasing rapidly from $x/L_0 = 15.95$. The ratios both continue to increase from this point until the swx, with a small decrease occurring at $x/L_0 = 16.4$ associated with a rapid decrease in maximum wave height. Ratios are generally identical until $x/L_0 = 16.25$, after which point the $H_{\max}/d_{\text{swl}}$ increases more rapidly than the $H_{\max}/d_{\text{mwl}}$ ratio. This is likely associated with the increase in mean water level or set up evident from this point inshore. The implications of this on engineering design are further discussed within Chapter 6.

Panel four depicts the evolution of the spectral significant period, normalised by the deep water significant period and the spectral bandwidth parameter. Spectral significant period can be observed to reduce slightly from around break point of the largest wave before increasing rapidly between $x/L_0 = 16.5$ and the swx. This increase may be related to the development and growth of low frequency waves far into the surf zone. The spectral bandwidth parameter relates the spectral zero up-crossing period to the crest period, with a value of 0 corresponding to a Rayleigh distribution of wave heights and a value of 1 to a Gaussian distribution (Huang et al, 1990). The bandwidth can be observed to increase from around 0.6 in deep water to almost 1 at the swx. This is somewhat in disagreement with the evolution of the wave height distribution presented in C-1, which shows the wave field attaining a more Rayleigh distribution inside the surf zone.

The bottom, fifth panel, depicts the change in wave celerity during shoaling and breaking. Two measures of celerity are presented, the first being based on the best-fit phase difference of time series extracted from adjacent video series around 100 mm apart (refer Section 4.3.2) and the second from the median difference between individual wave times extracted from adjacent time series around 10 mm apart. While the first method is anticipated to provide a more stable and robust estimate, as evidenced by the more uniform values of wave celerity within the figure, the method is reliant on minimal change in the wave field form in adjacent video series. While this is a reasonable estimate in deep water, the method is less reliable in shallow water where

rapid changes in wave form during shoaling and breaking may induce error. The second, individual wave method is likely to be more robust in rapidly evolving wave forms but is subject to error in defining identical waves within adjacent series. A number of thresholds and checks are however, implemented within the numerical code to minimise erroneous celerity values calculated by mistakenly comparing different wave forms. Additionally, the selection of the median wave celerity further minimises erroneous outliers. In general, both the methods predict a reduction in wave celerity as the group shoals and further reduction within the surf zone. This is as expected by theory.

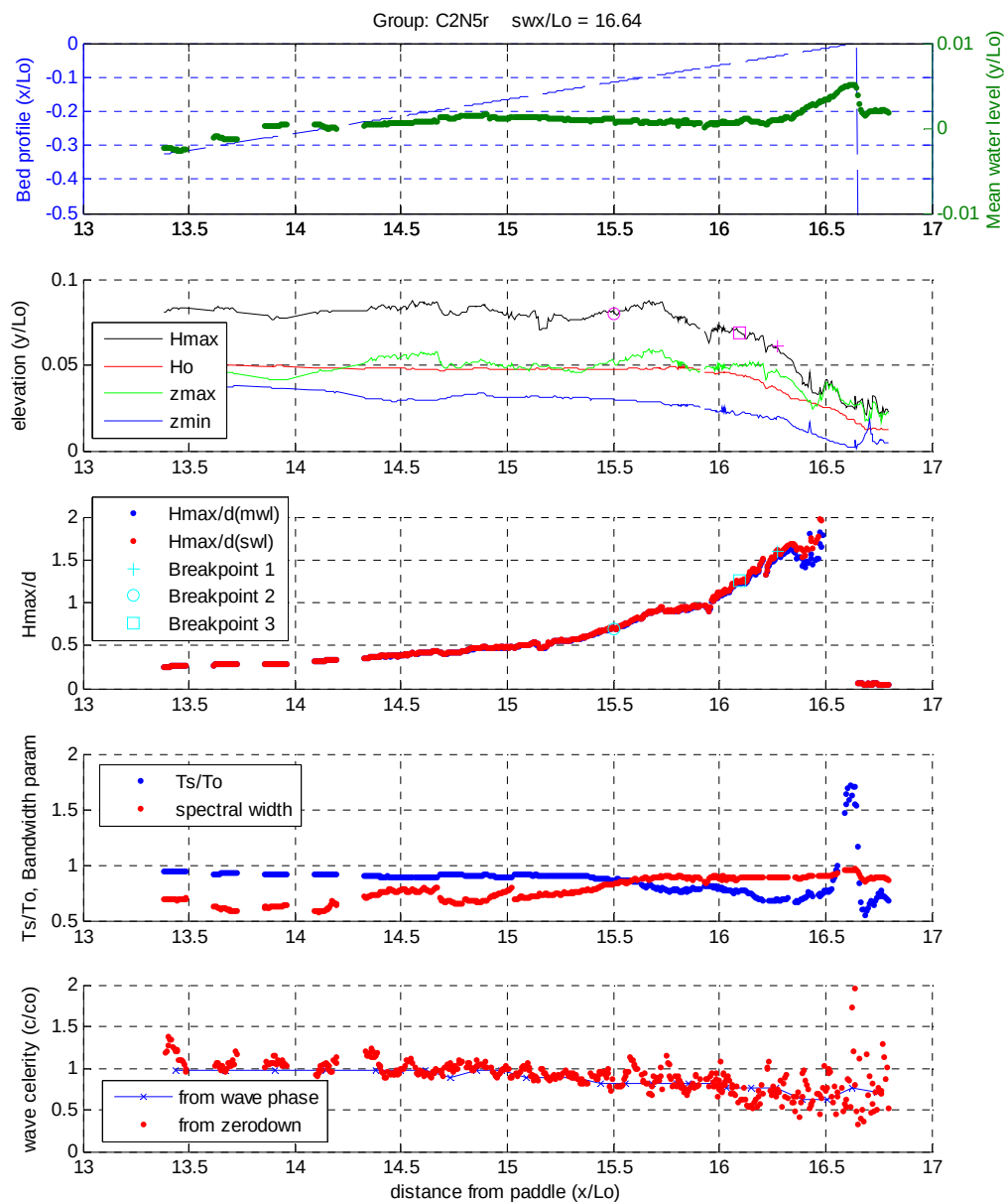


Figure C-2 Example of extracted water surface, wave and H_{max}/d characteristics for a Case 2, $N = 5$ type wave group propagating onto a 10:1 slope bed.

C-3 *Example of the extracted space-time series for surface elevation and low-pass surface elevation.*

Figure C-3(i) shows a space-time plot of the water surface profile with time along the x axis, position within the flume with respect to the swx in the y axis and water level in the z axis. Dispersion within the group can be easily observed with waves at the left hand side (front) of the groups observed to decrease in amplitude with distance along the flume and new waves emerging at the right hand side (back) of the groups. The wave form shape is also observed to change during propagation. Within deep water, wave crests and troughs are near uniform and sinusoidal, whereas during shoaling the wave crest can be observed to narrow and troughs widen. Furthermore, within the surf zone (e.g. inside $x/L_0 = 0.75$), troughs shift to the right, i.e. become deepest immediately in front of the wave face, with a more subdued slope evident on the wave back.

Dissipation is evident within the surf zone, with a reduction in wave form amplitude apparent. The largest wave form, near the front of the group however, appears to maintain its amplitude more effectively than other wave forms. This may be related to underlying water level elevations as discussed with respect to Figure C-3(ii). Wave celerity is defined by the angle (on the plot) of a wave form, with a more vertical propagation path indicating higher celerity and a more horizontal path, a lower celerity. A reduction in wave celerity is evident during shoaling and within the surf zone, as expected by theory.

Figure C-3(ii) shows the low-pass filtered surface elevation with group-associated long-waves evident. Due to the filtering methods employed, the first and last 7 s of record (corresponding to 140 samples) is excluded. Within deepwater, the long wave is evident primarily as a trough bound to the group and centered around the group maximum (as per Longuet-Higgins and Stewart, 1964). This trough deepens as the wave group shoals before being destroyed at around $x/L_0 = 0.3$ from the swx. A crest, preceding the wave group maximum, can be observed to develop initially slowly during shoaling, then rapidly from around $x/L_0 = 0.3$, and reaching a maximum at the swx.

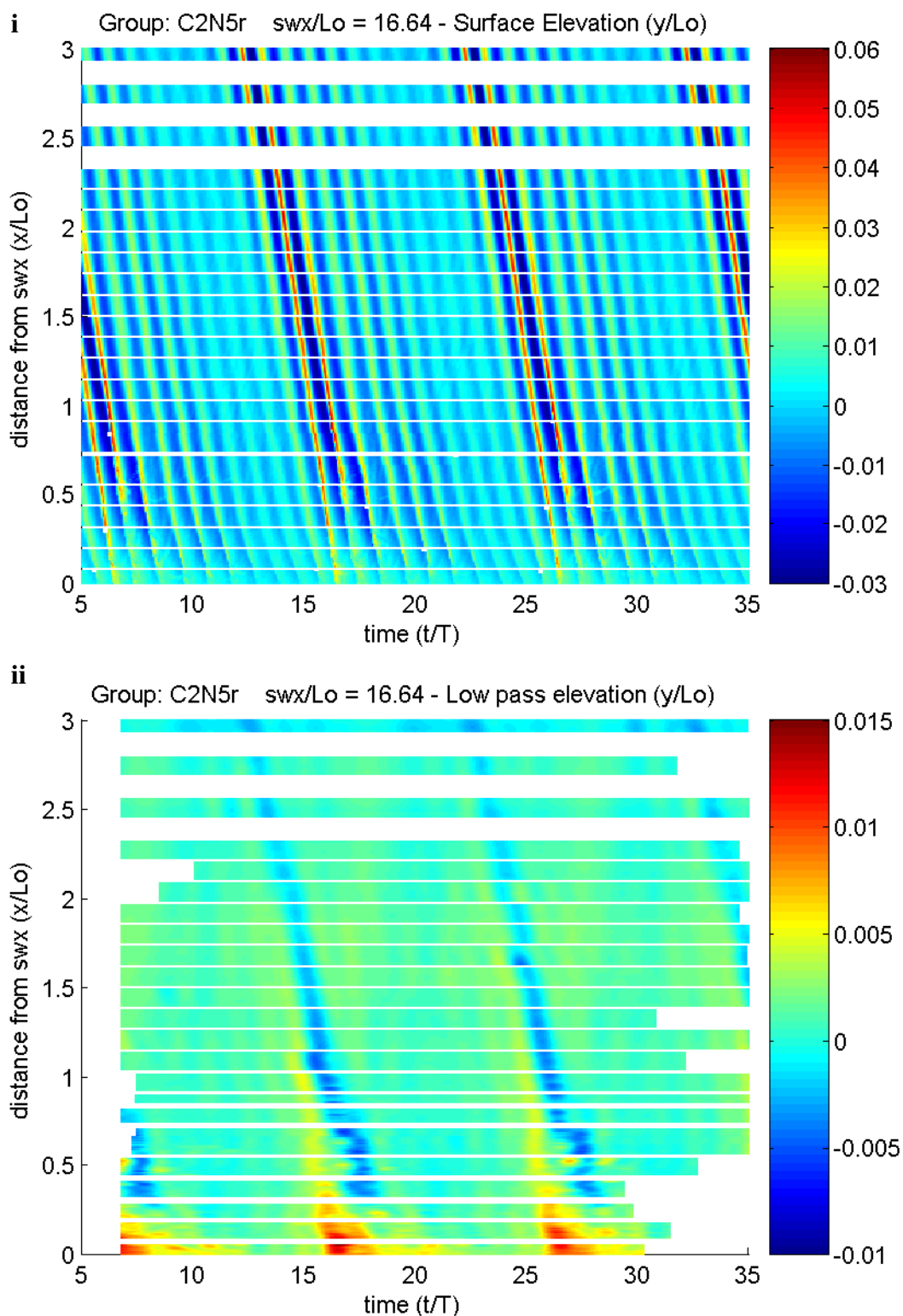


Figure C-3 Example of the extracted space-time series for surface elevation (i) and low-pass surface elevation (ii) for a Case 2, $N = 5$ type wave group propagating onto a 10:1 slope bed. White spaces depict gaps in the record due to either non-overlap of adjacent series or sidewall obstructions.

A small amount of reflection is evident with a low crest appearing to propagate seaward from around $x/L_0 = 1.0$. Based on the position in time and space of this long wave, it does not appear to be due to reflection off the beach. Long wave reflection from the beach appears impeded by the truncation of the sloping bed at the still water level (Section 3.3.3). Rather, the small offshore propagating long wave may be induced by the time-varying breakpoint position associated with wave height modulation within the group (i.e. Symonds et al, 1982). This overall behaviour is similar to that observed by Baldock (2004), although with less evidence of reflection, most likely due to bed truncation. The rapid destruction of the long wave trough at $x/L_0 = 0.3$ is not fully explained but may be associated with the preceding crest reaching the swx first and canceling the following trough through partial reflection.

C-4 Example of the extracted frequency-space energy spectra.

Figure C-4 depicts the evolution of the frequency-energy spectrum as a function of along-flume distance. The log-spectral density is used to better illustrate the changes in the non-spectral peak areas. The spectrum shows nearly all energy comprised around the spectral peak and sidebands during early stages of shoaling. Energy within the first harmonic (around 2.7 Hz) can be observed to increase however, at around $x/L_0 = 1.0$ from the swx. This energy reaches a maximum at around $x/L_0 = 0.8$ before diminishing further into the surf zone. Energy subsequently increases at the long-wave frequency of 0.13 Hz from $x/L_0 = 0.15$, reaching a maximum immediately before the swx.

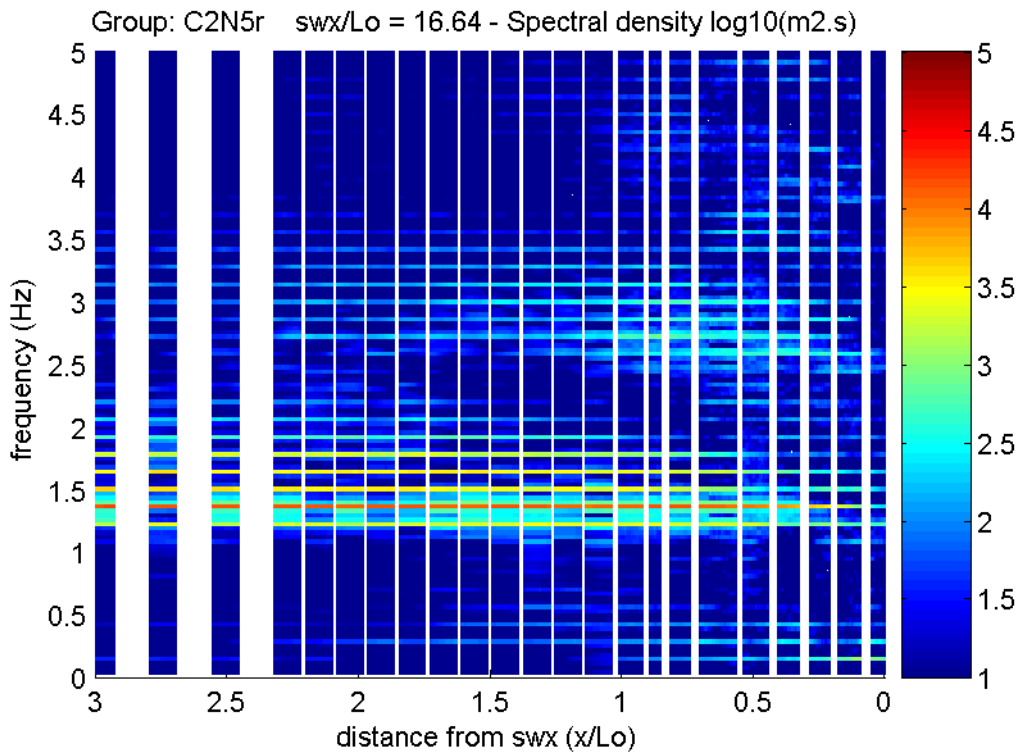


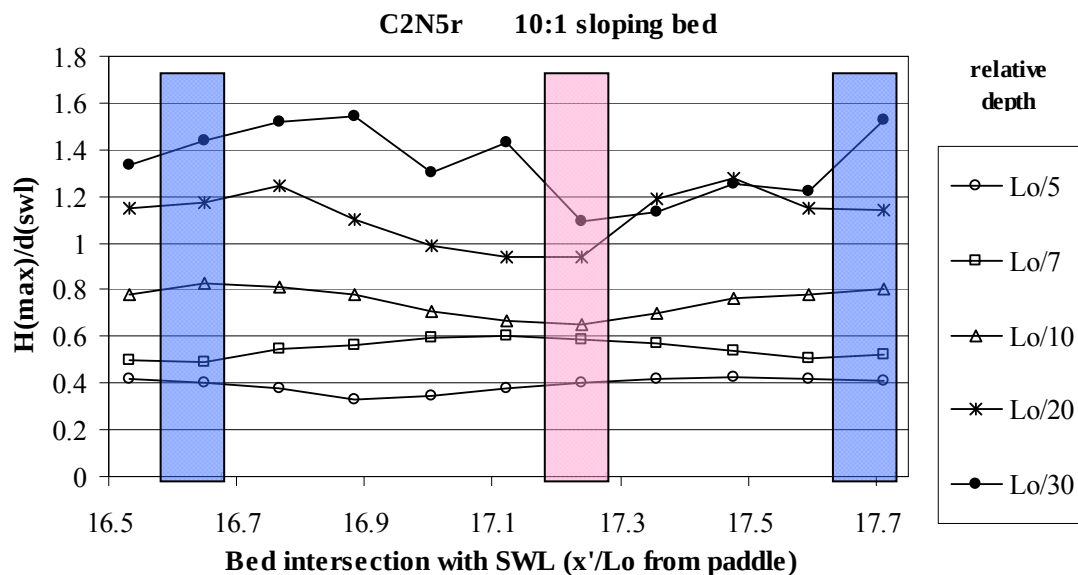
Figure C-4 Frequency-space energy spectra for a Case 2, N = 5 type wave group propagating onto a 10:1 slope bed. White spaces depict gaps in the record due to either non-overlap of adjacent series or sidewall obstructions.

Appendix D. Wave group shoaling and breaking results

- D-1 C2N5r wave group on 10:1 slope**
- D-2 C2N5r wave group on 5:1 slope**
- D-3 C2N5r wave group on 20:1 slope**
- D-4 C2N3r wave group on 10:1 slope**
- D-5 C2N5sb wave group on 10:1 slope**
- D-6 C3N5r wave group on 10:1 slope**

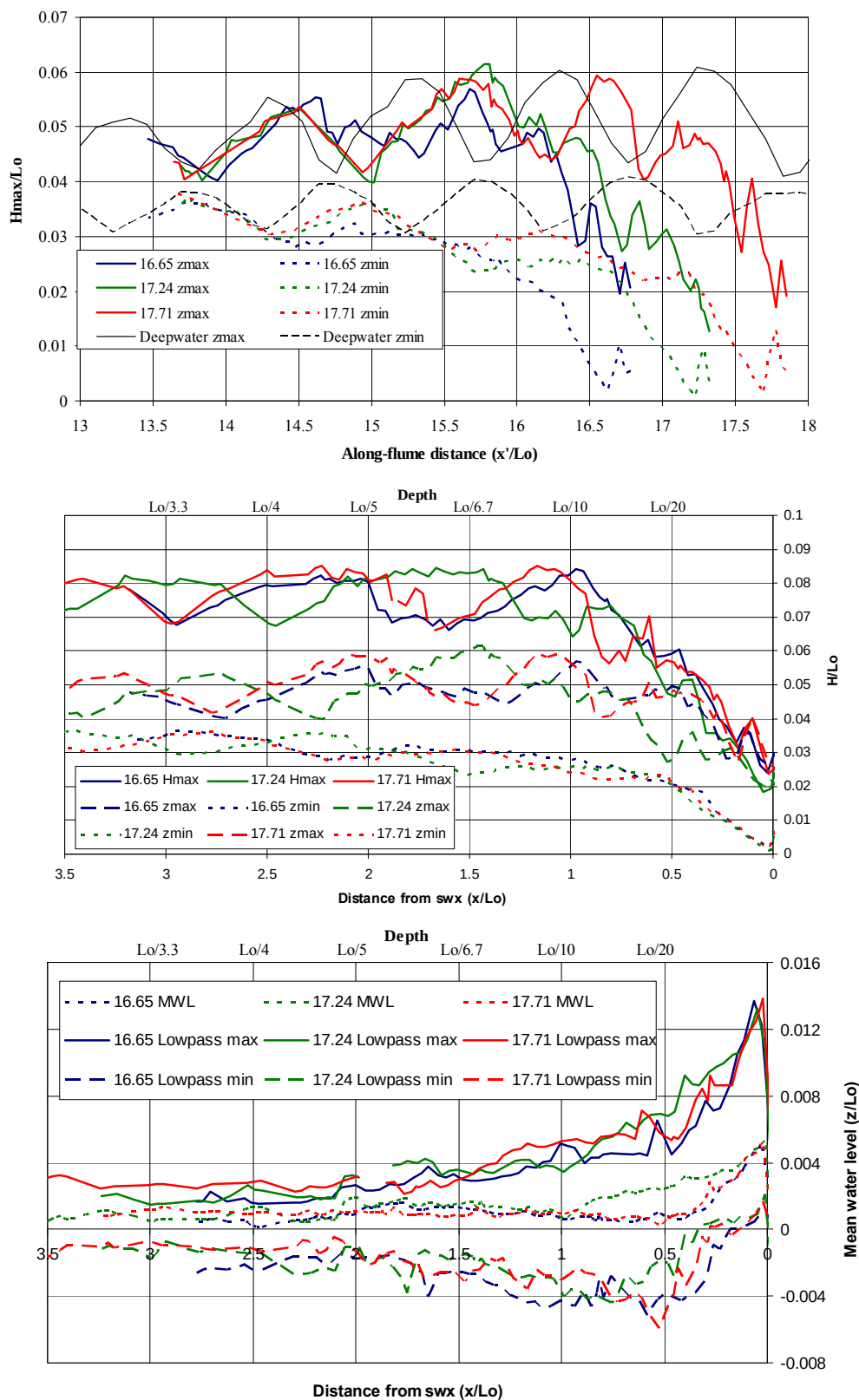
D-1 C2N5r wave group on 10:1 slope

D-1a $H_{\max}/d$ ratios as a function of bed position and relative depth

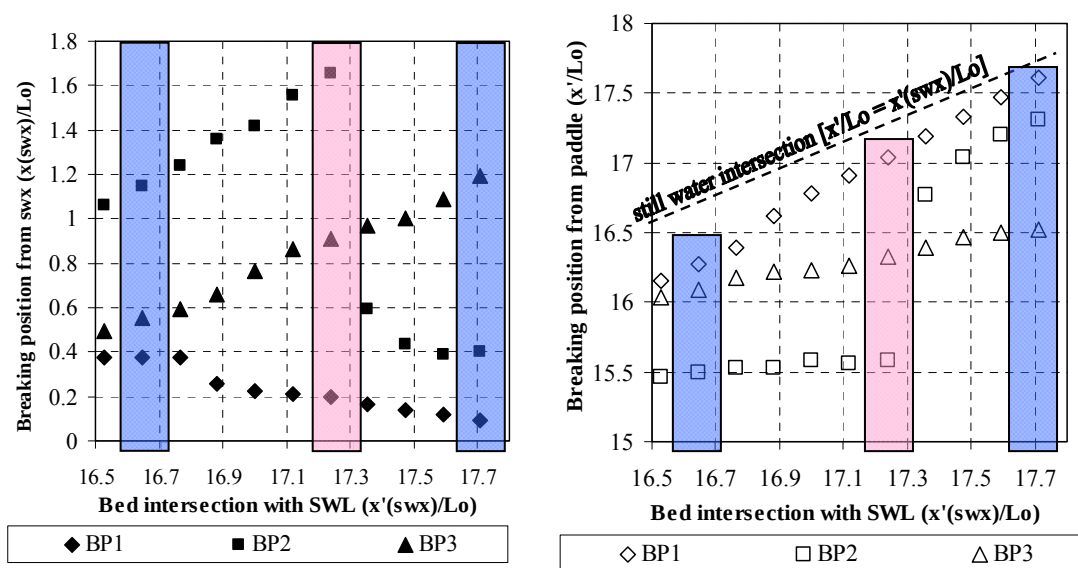


Non-dimensional depth	$L_0/5$	$L_0/7$	$L_0/10$	$L_0/15$	$L_0/20$	$L_0/30$	$L_0/40$
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
16.53	0.42	0.49	0.78	1.00	1.15	1.34	1.51
16.65	0.40	0.49	0.82	0.94	1.17	1.44	1.55
16.77	0.38	0.54	0.81	0.86	1.24	1.52	1.66
16.89	0.33	0.57	0.78	0.94	1.10	1.54	1.64
17.00	0.35	0.60	0.70	0.98	0.99	1.30	1.62
17.12	0.38	0.61	0.67	0.98	0.94	1.43	1.59
17.24	0.40	0.59	0.65	0.96	0.94	1.09	1.37
17.36	0.42	0.57	0.70	0.92	1.19	1.13	1.27
17.48	0.42	0.54	0.76	0.91	1.27	1.25	1.34
17.59	0.42	0.51	0.78	0.93	1.15	1.23	1.93
17.71	0.41	0.52	0.81	0.95	1.14	1.53	1.73
Max difference	0.09	0.11	0.17	0.14	0.34	0.45	0.66

D-1b Evolutional properties of $H_{\max}$, $z_{\max}$, $z_{\min}$, $LP_{\max}$, $LP_{\min}$ and MWL for selected extreme cases

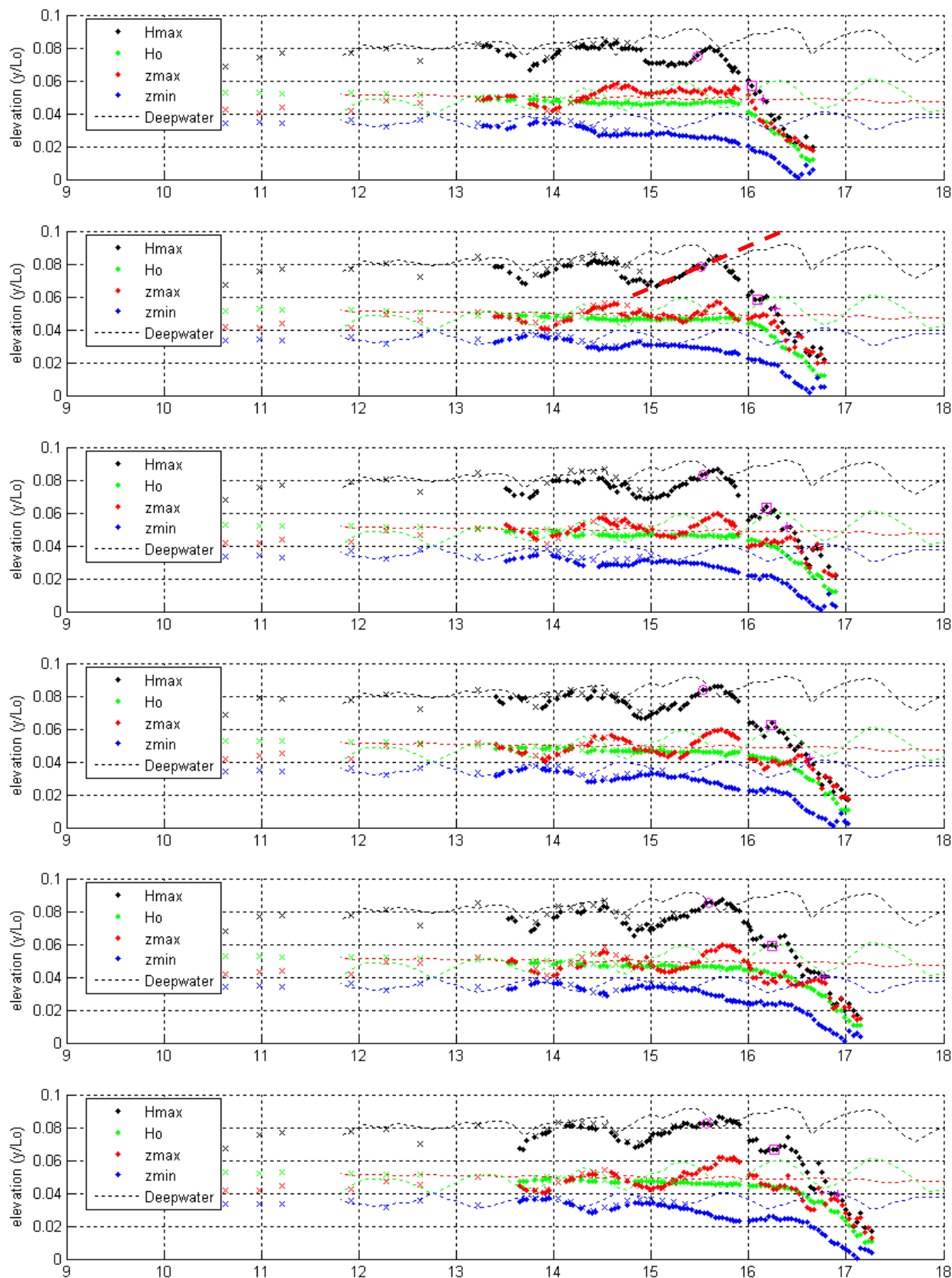


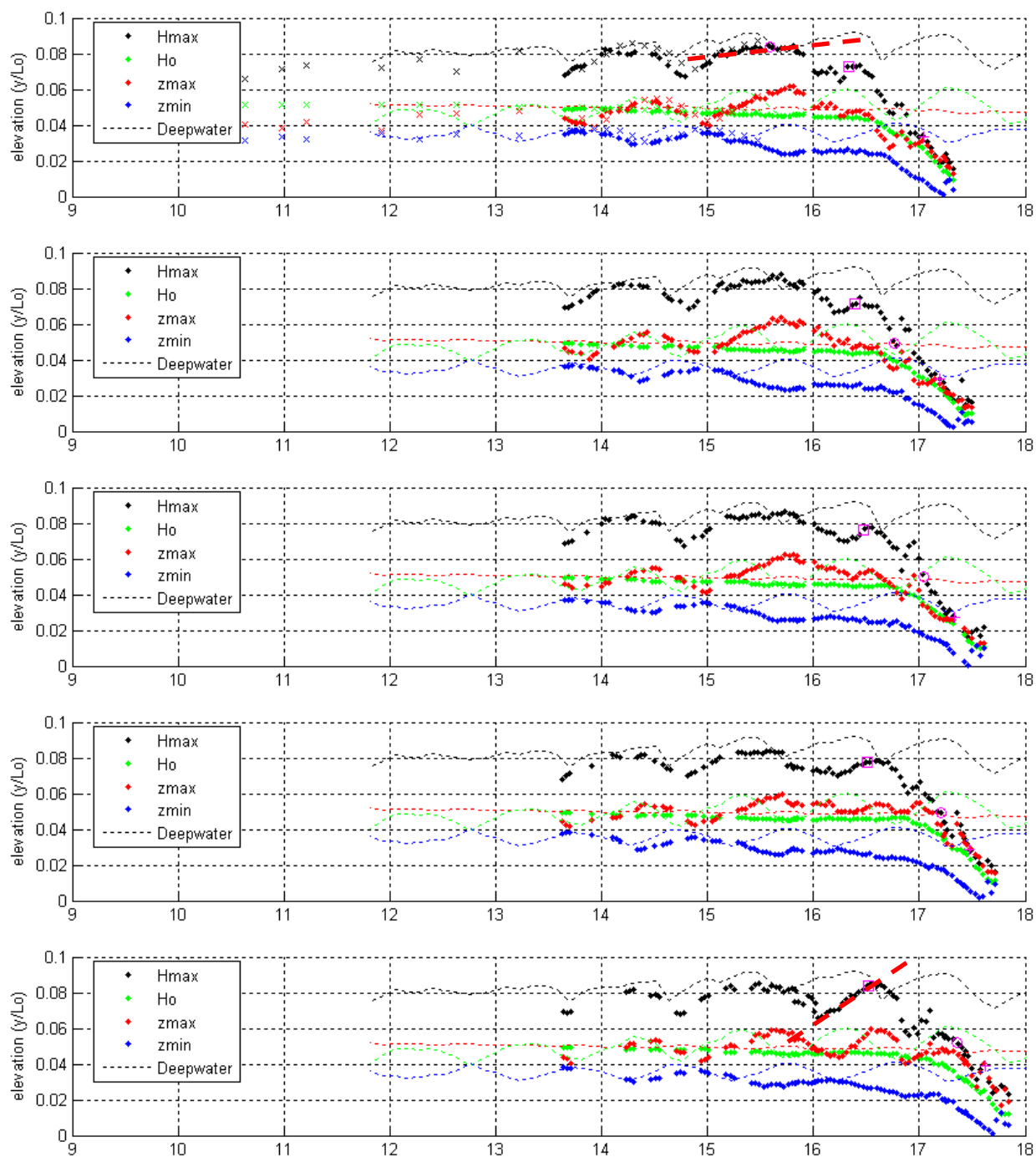
D-1c Breaking location and type as a function of bed position



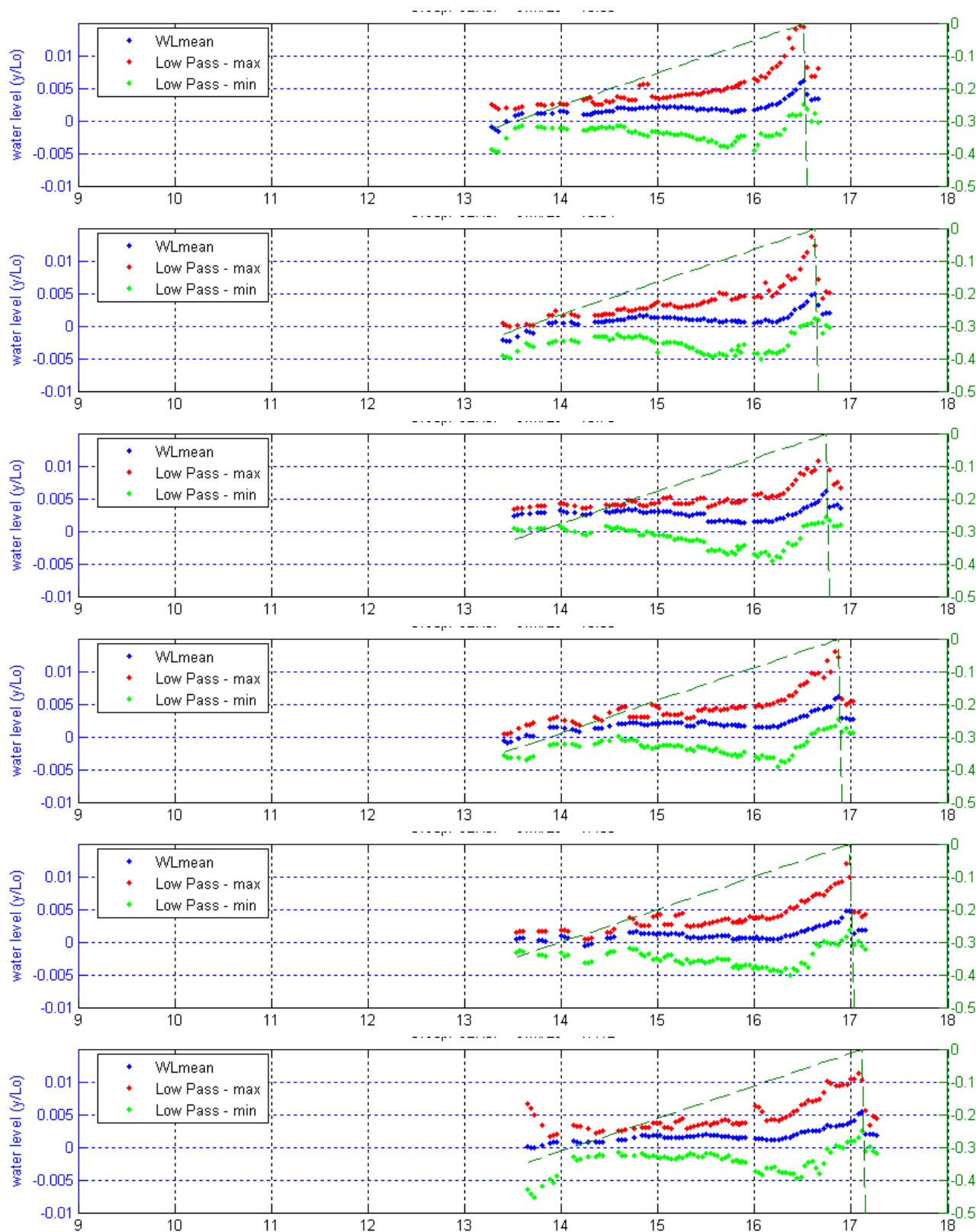
	Break point 1				Break point 2				Break point 3			
Position of swx (x'/L_0)	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type
16.53	0.38	16.15	0.049	6.0	1.06	15.47	0.075	4.0	0.50	16.03	0.057	7.5
16.65	0.38	16.27	0.052	6.5	1.15	15.50	0.078	4.0	0.55	16.09	0.058	7.0
16.77	0.38	16.39	0.051	5.0	1.24	15.53	0.083	2.5	0.59	16.17	0.063	7.0
16.89	0.26	16.62	0.042	6.5	1.36	15.53	0.084	2.5	0.66	16.22	0.063	7.0
17.00	0.22	16.78	0.040	8.0	1.42	15.58	0.085	2.0	0.77	16.23	0.059	7.0
17.12	0.21	16.91	0.039	8.0	1.56	15.56	0.082	2.0	0.86	16.26	0.067	7.0
17.24	0.20	17.04	0.033	8.0	1.65	15.58	0.084	2.0	0.91	16.33	0.073	7.0
17.36	0.17	17.19	0.029	9.0	0.59	16.77	0.050	5.0	0.97	16.39	0.071	7.0
17.48	0.14	17.33	0.027	9.0	0.44	17.04	0.050	5.0	1.00	16.47	0.076	6.0
17.59	0.12	17.47	0.029	9.0	0.39	17.20	0.049	5.0	1.09	16.51	0.078	6.0
17.71	0.09	17.62	0.039	-	0.40	17.31	0.052	-	1.19	16.52	0.084	6.0

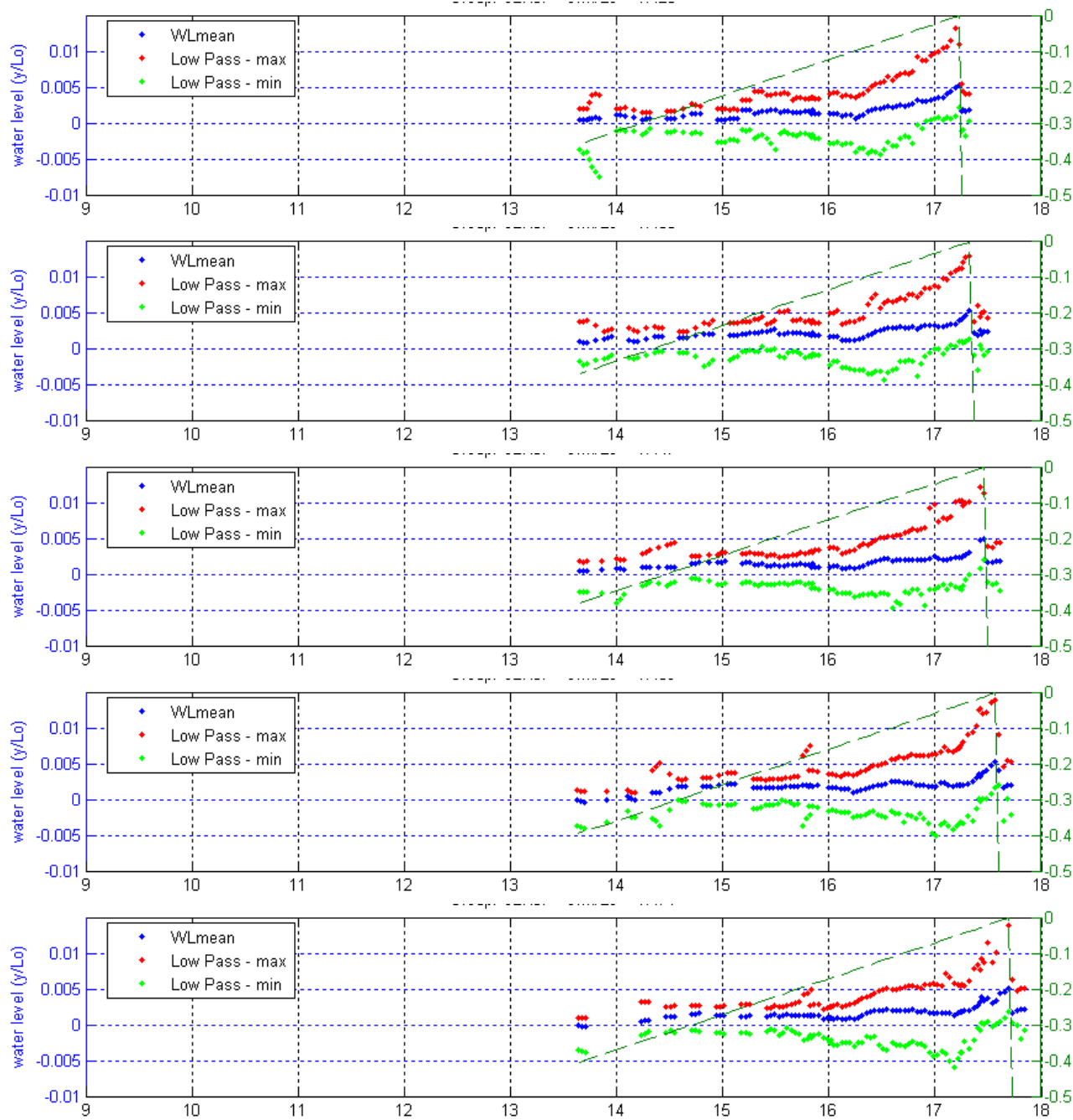
D-1d Evolution of $H_{\max}$, H_0 , $z_{\max}$ and $z_{\min}$ compared to deep water values for SWX located at @ $x'/L_0 = 16.53, 16.65, 16.77, 16.88, 17.00, 17.11, 17.24, 17.36, 17.47, 17.59, 17.70$





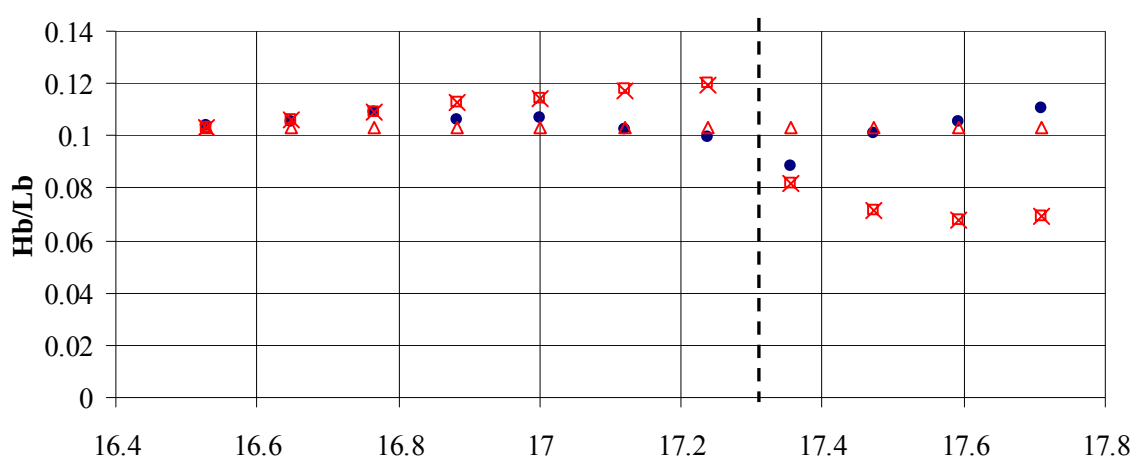
D-1e Evolution of LP_{max} , LP_{min} and mean water level for SWX located at $x'/Lo = 16.53, 16.65, 16.77, 16.88, 17.00, 17.11, 17.24, 17.36, 17.47, 17.59, 17.70$



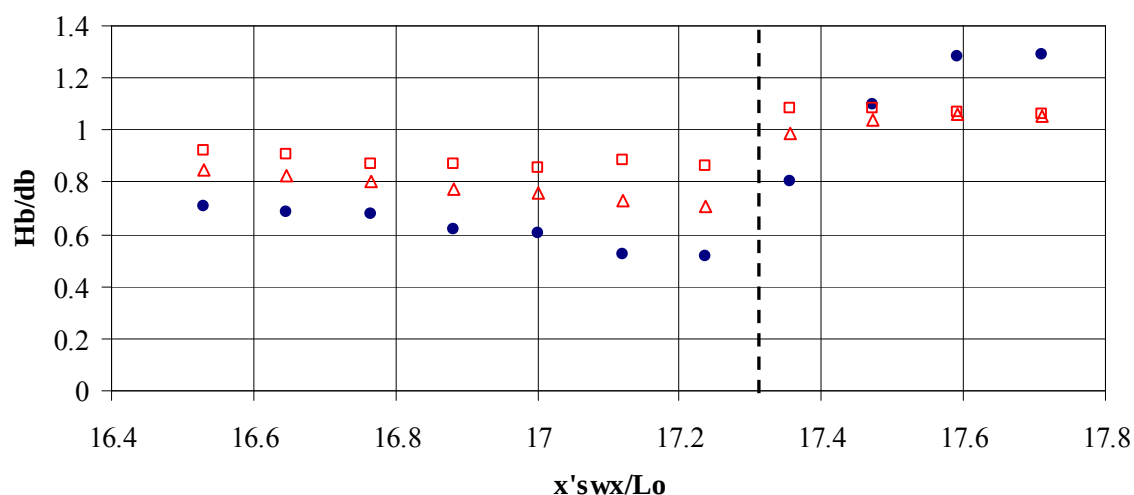


D-1f Observed breaking steepness (H_b/L_b) (●) for varying group spatial phasing positions are depicted within (A) for a C2N5r type group propagating onto a 10:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) (□), Kamphuis (1991) (×) and Rattanapitikon and Vivattanasirisak (2003) (△). Within (B), observed breaker depth index (●) is compared to indices predicted by USACE (2003) (□) and Goda (2000) (△). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

A

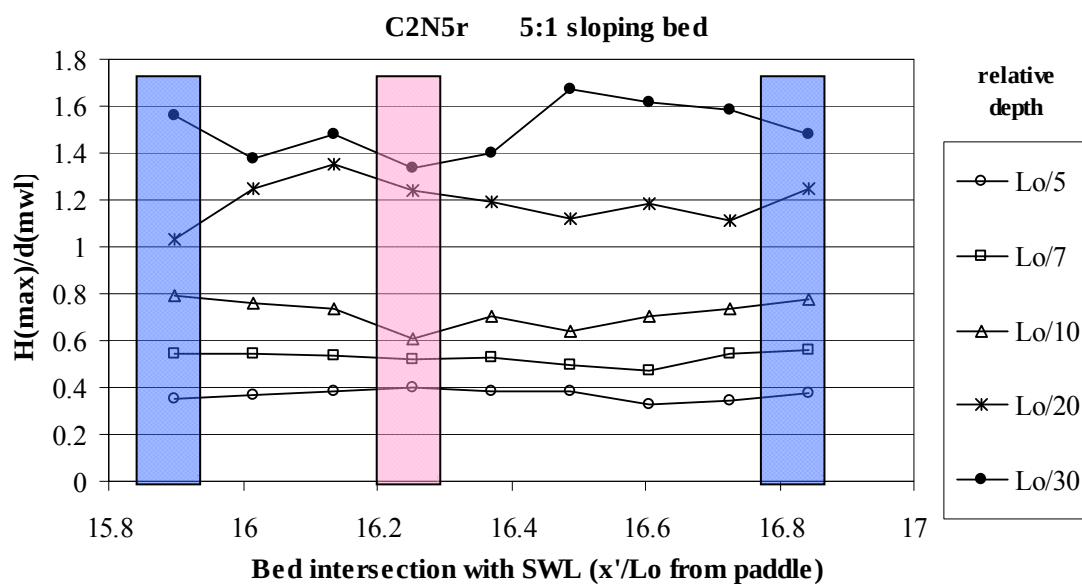


B



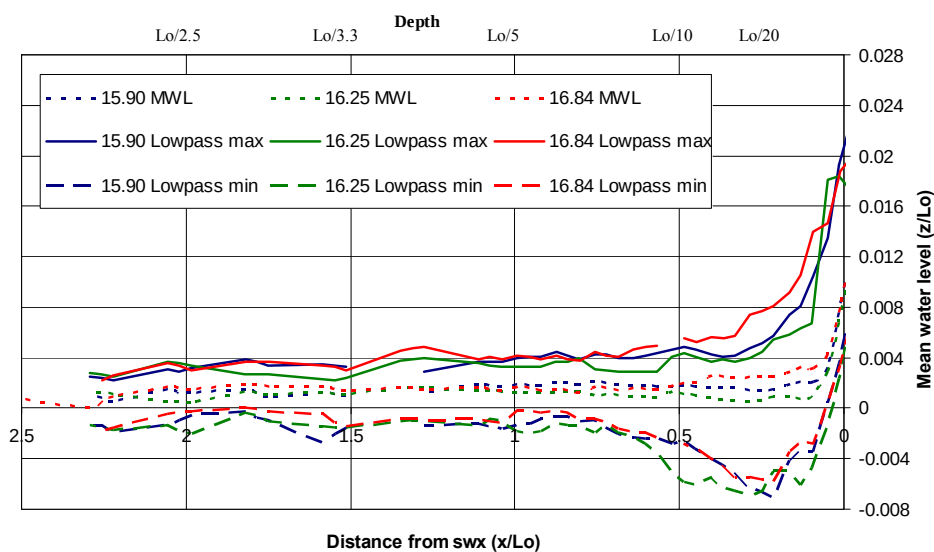
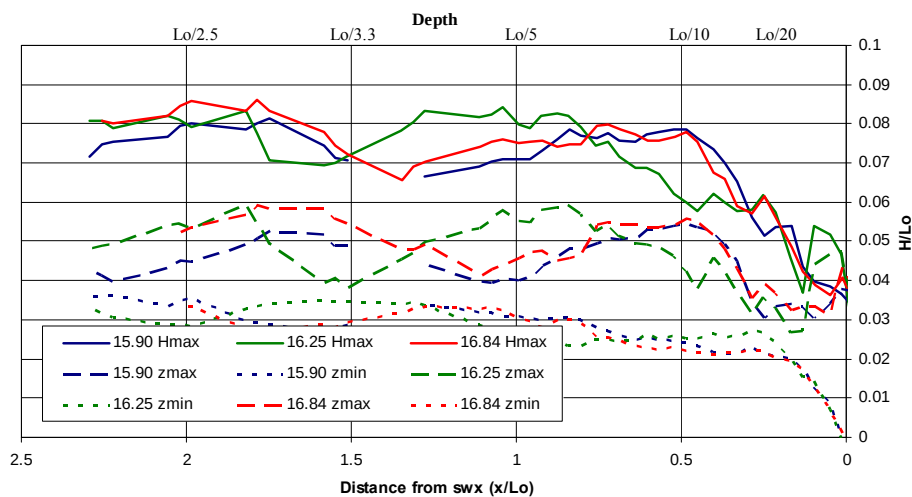
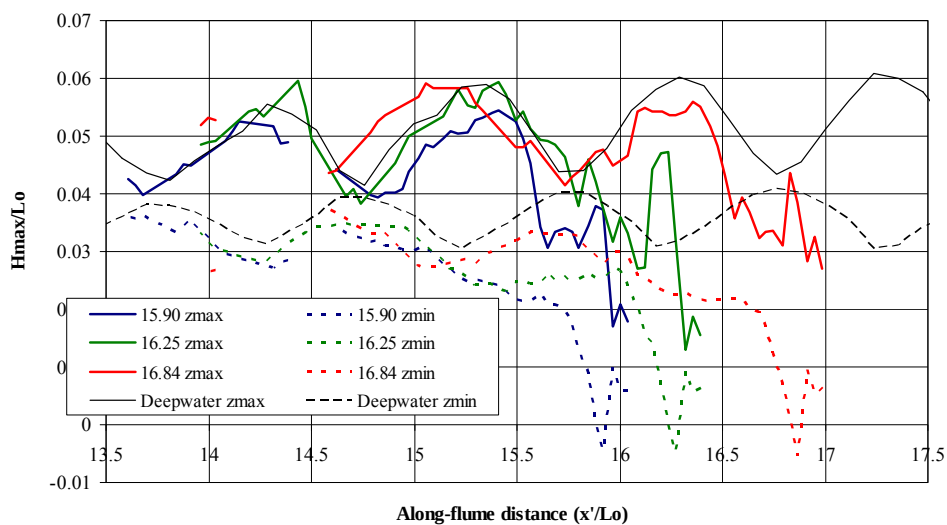
D-2 C2N5r wave group on 5:1 slope

D-2a $H_{\max}/d$ ratios as a function of bed position and relative depth

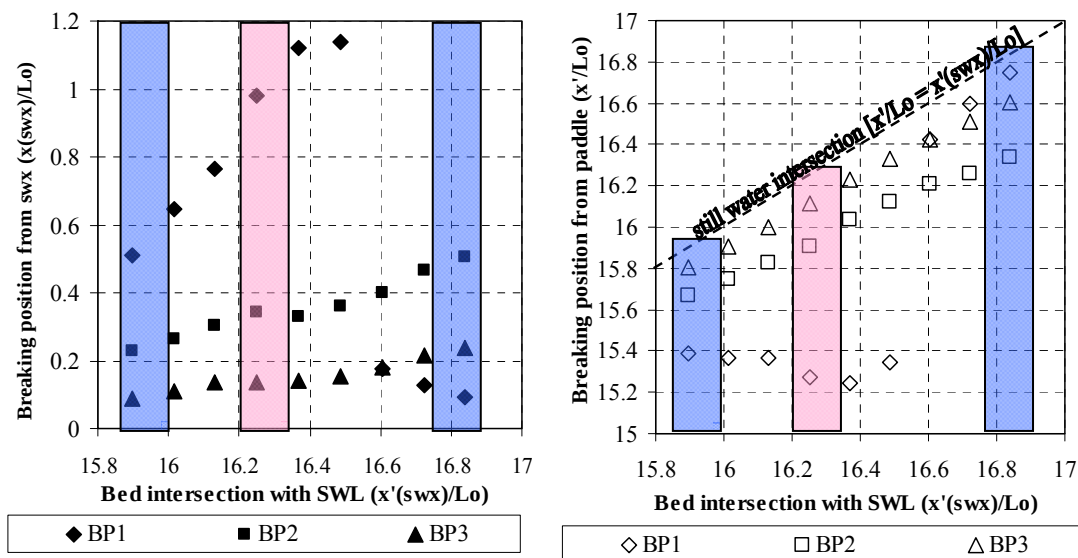


Non-dimensional depth	Lo/5	Lo/7	Lo/10	Lo/15	Lo/20	Lo/30	Lo/40
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
15.90	0.35	0.54	0.79	0.98	1.04	1.56	1.74
16.02	0.37	0.54	0.76	0.96	1.25	1.38	1.58
16.13	0.38	0.54	0.74	0.94	1.35	1.48	1.82
16.25	0.40	0.52	0.61	0.87	1.24	1.34	1.61
16.37	0.39	0.53	0.71	0.88	1.19	1.40	1.44
16.49	0.39	0.50	0.64	0.96	1.12	1.67	1.88
16.61	0.32	0.47	0.70	1.00	1.18	1.61	2.16
16.72	0.34	0.54	0.74	0.96	1.11	1.59	2.08
16.84	0.38	0.56	0.78	0.88	1.25	1.48	1.69
Max difference	0.08	0.09	0.18	0.14	0.32	0.33	0.71

D-2b Evolutional properties of $H_{\max}$, $z_{\max}$, $z_{\min}$, $LP_{\max}$, $LP_{\min}$ and MWL for selected extreme cases

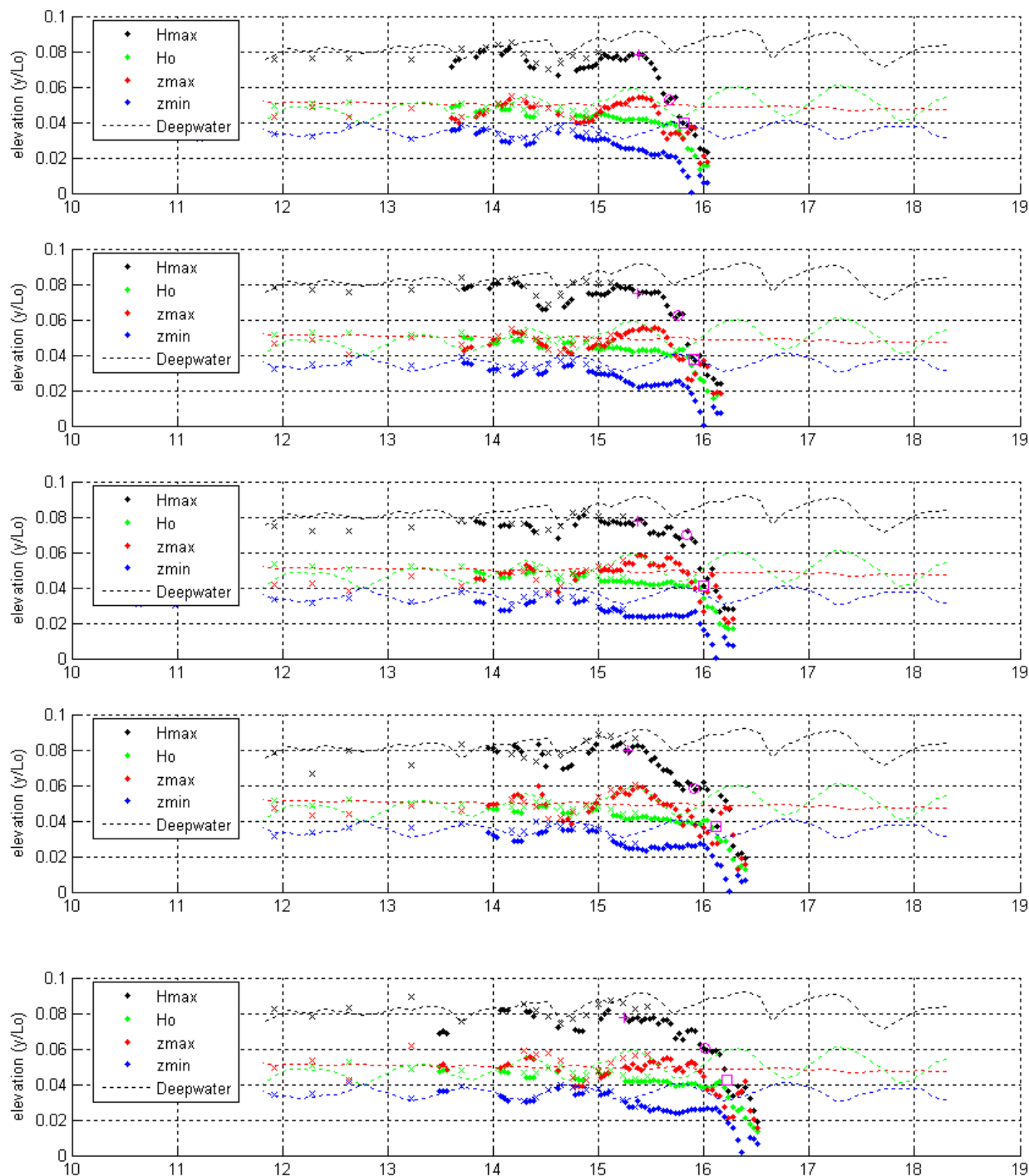


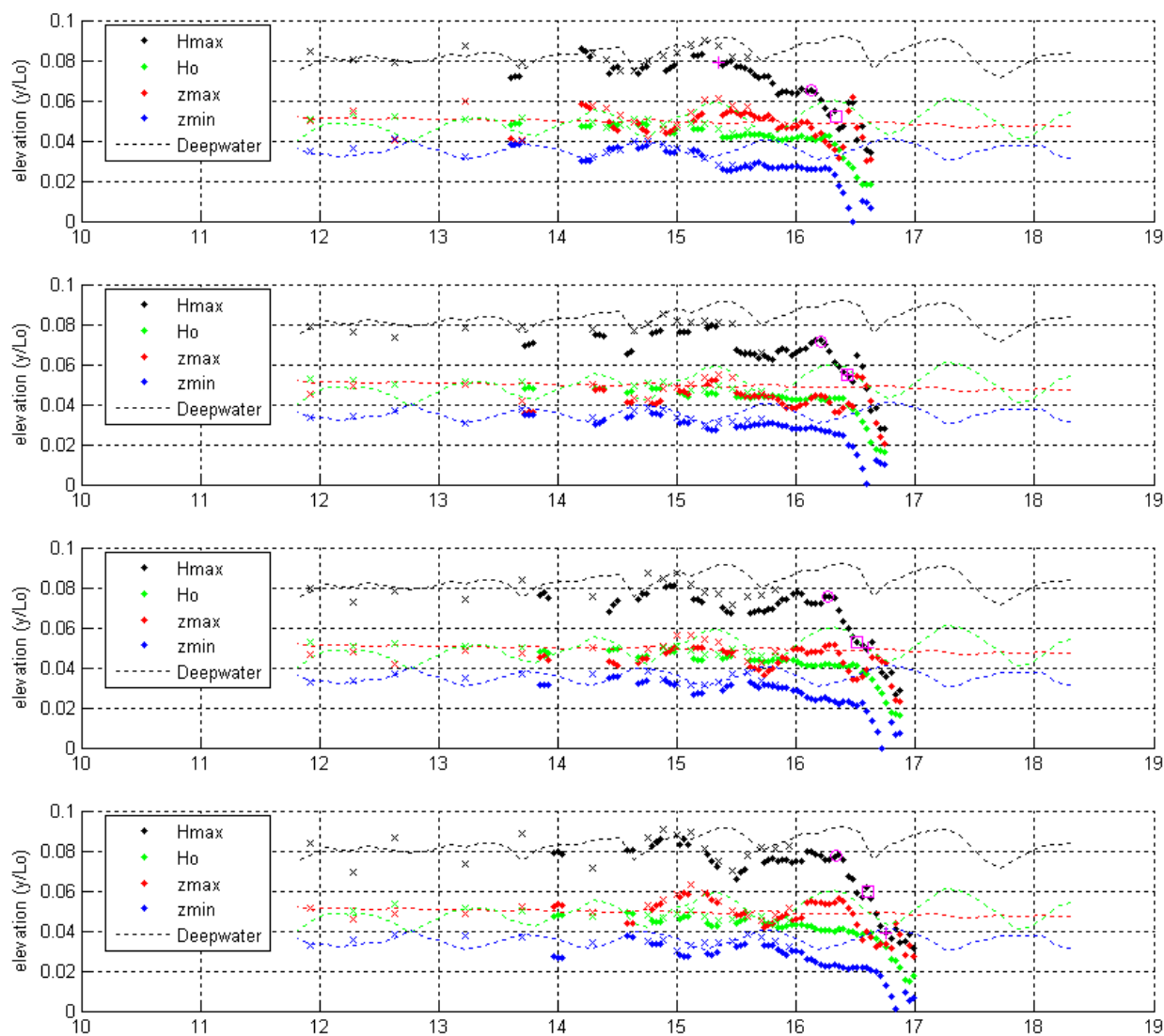
D-2c Breaking location and type as a function of bed position



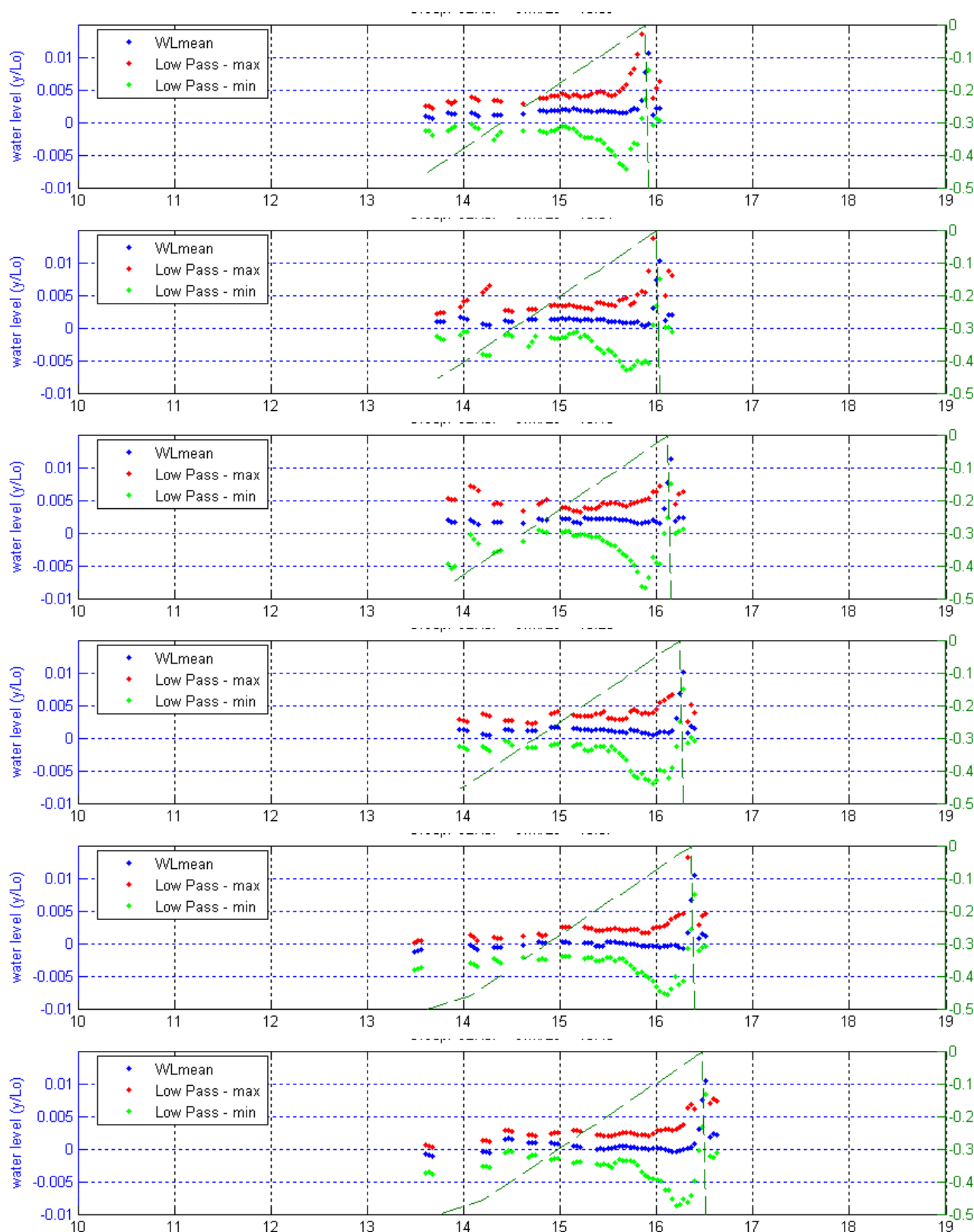
Position of swx (x'/L_0)	Break point 1				Break point 2				Break point 3			
	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type
15.90	0.51	15.39	0.078	4.5	0.23	15.67	0.053	8.5	0.09	15.81	0.040	
16.01	0.65	15.37	0.075	4.0	0.27	15.75	0.062	7.0	0.11	15.90	0.038	9.0
16.13	0.77	15.37	0.078	5.5	0.30	15.83	0.070	8.0	0.14	16.00	0.041	9.0
16.25	0.98	15.27	0.079	2.0	0.34	15.91	0.058	7.0	0.13	16.12	0.037	7.5
16.37	1.12	15.25	0.078	2.0	0.33	16.04	0.060	5.0	0.14	16.23	0.043	7.5
16.49	1.14	15.35	0.079	1.5	0.36	16.12	0.065	5.0	0.16	16.33	0.052	
16.60	0.18	16.43	0.055	7.5	0.40	16.21	0.072	5.5	0.18	16.43	0.055	8.5
16.72	0.13	16.60	0.050	7.5	0.46	16.26	0.075	5.0	0.21	16.51	0.053	8.0
16.84	0.09	16.75	0.039	9.5	0.50	16.34	0.077	3.0	0.24	16.61	0.060	8.0

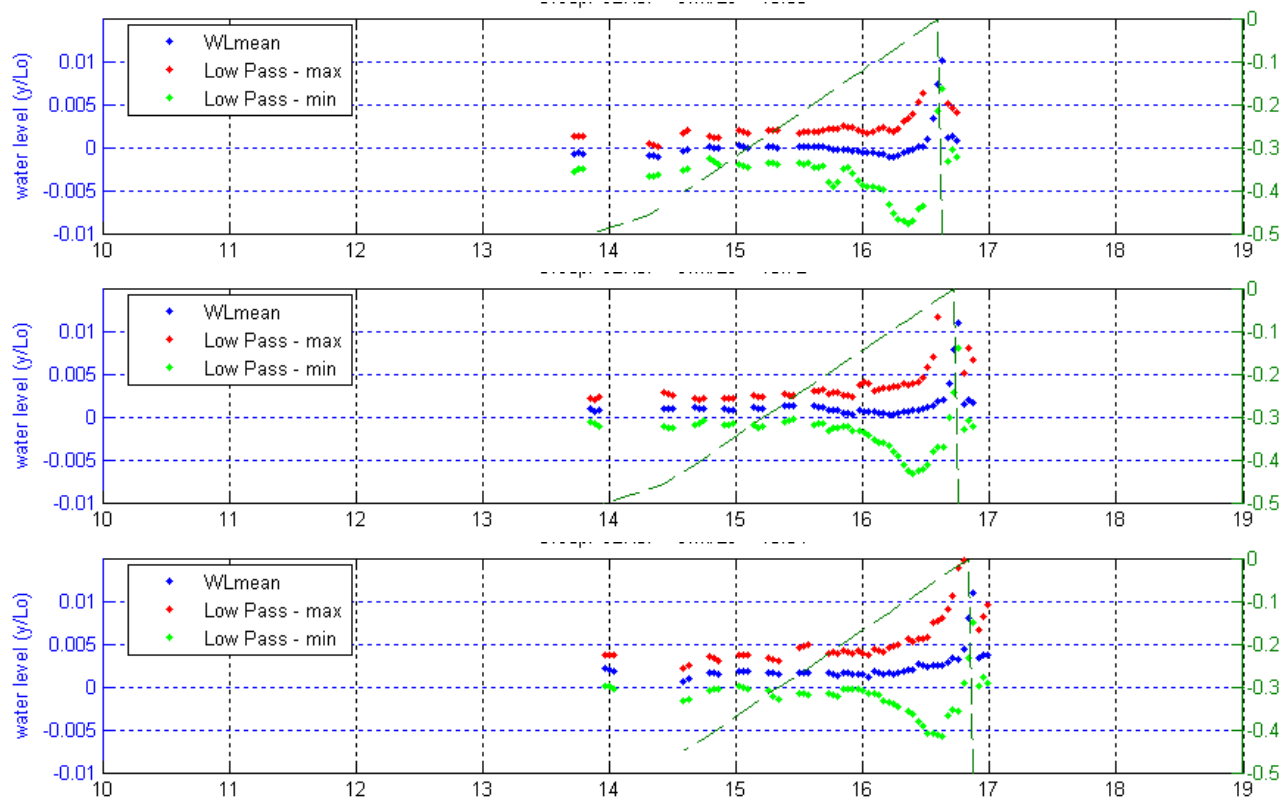
D-2d Evolution of $H_{\max}$, H_0 , $z_{\max}$ and $z_{\min}$ compared to deep water values for SWX located at @ $x'/L_0 = 15.90, 16.02, 16.13, 16.25, 16.37, 16.49, 16.61, 16.72, 16.84$





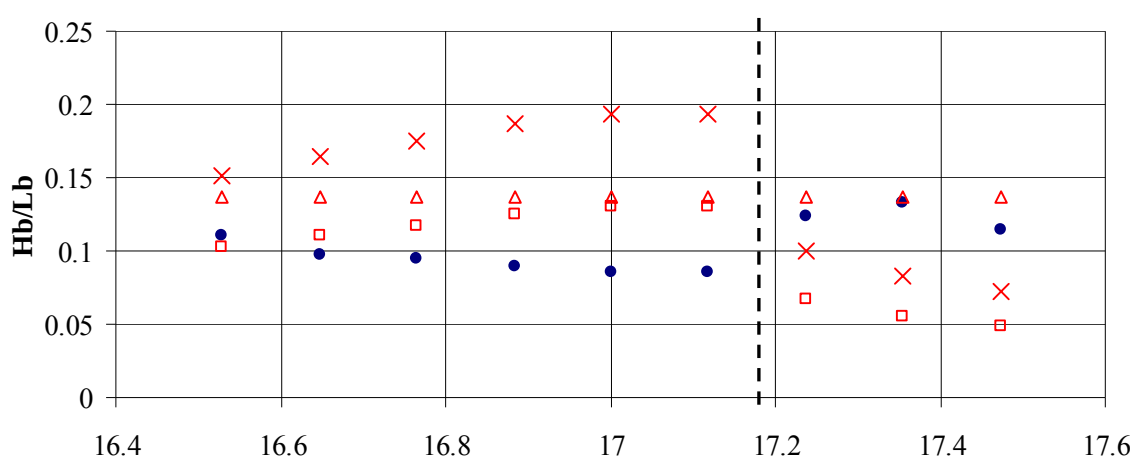
D-2e Evolution of LP_{max} , LP_{min} and mean water level for SWX located at $x'/Lo = 15.90, 16.02, 16.13, 16.25, 16.37, 16.49, 16.61, 16.72, 16.84$



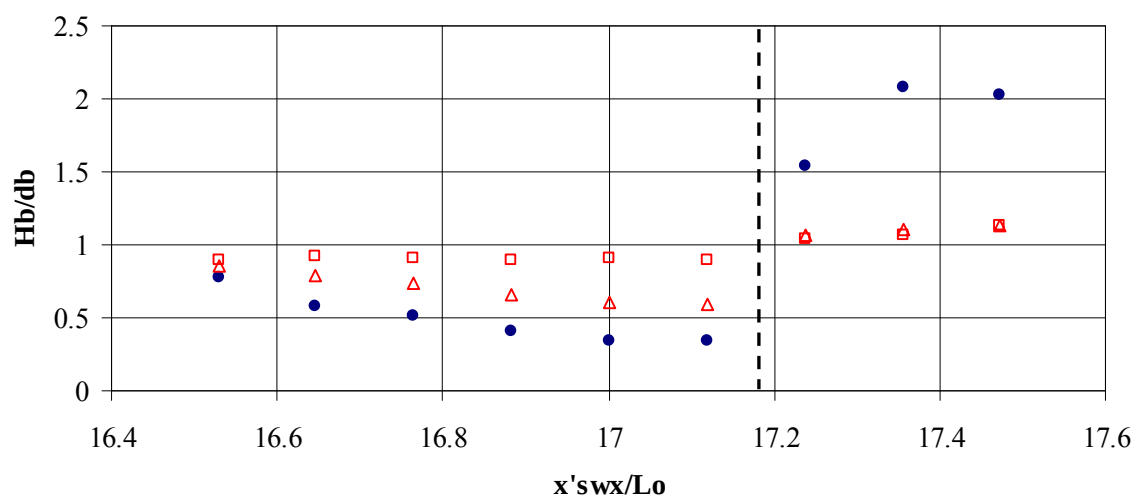


D-2f Observed breaking steepness (H_b/L_b) ($\bullet$) for varying group spatial phasing positions are depicted within (A) for a C2N5r type group propagating onto a 5:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) ($\square$), Kamphuis (1991) ($\times$) and Rattanapitikon and Vivattanasirisak (2003) (Δ). Within (B), observed breaker depth index ($\bullet$) is compared to indices predicted by USACE (2003) ($\square$) and Goda (2000) (Δ). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

A

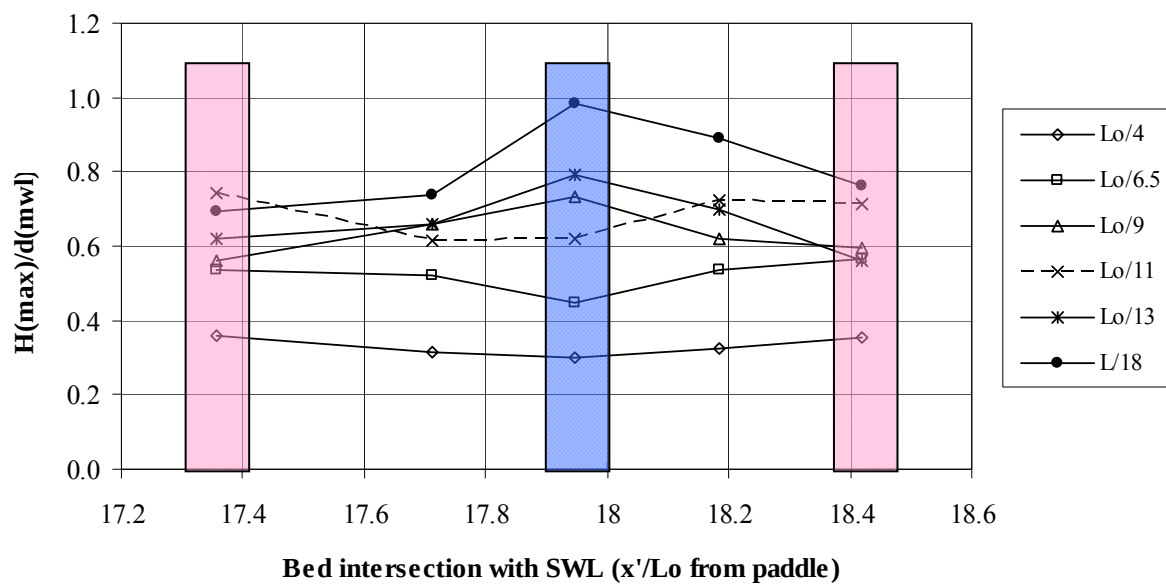


B



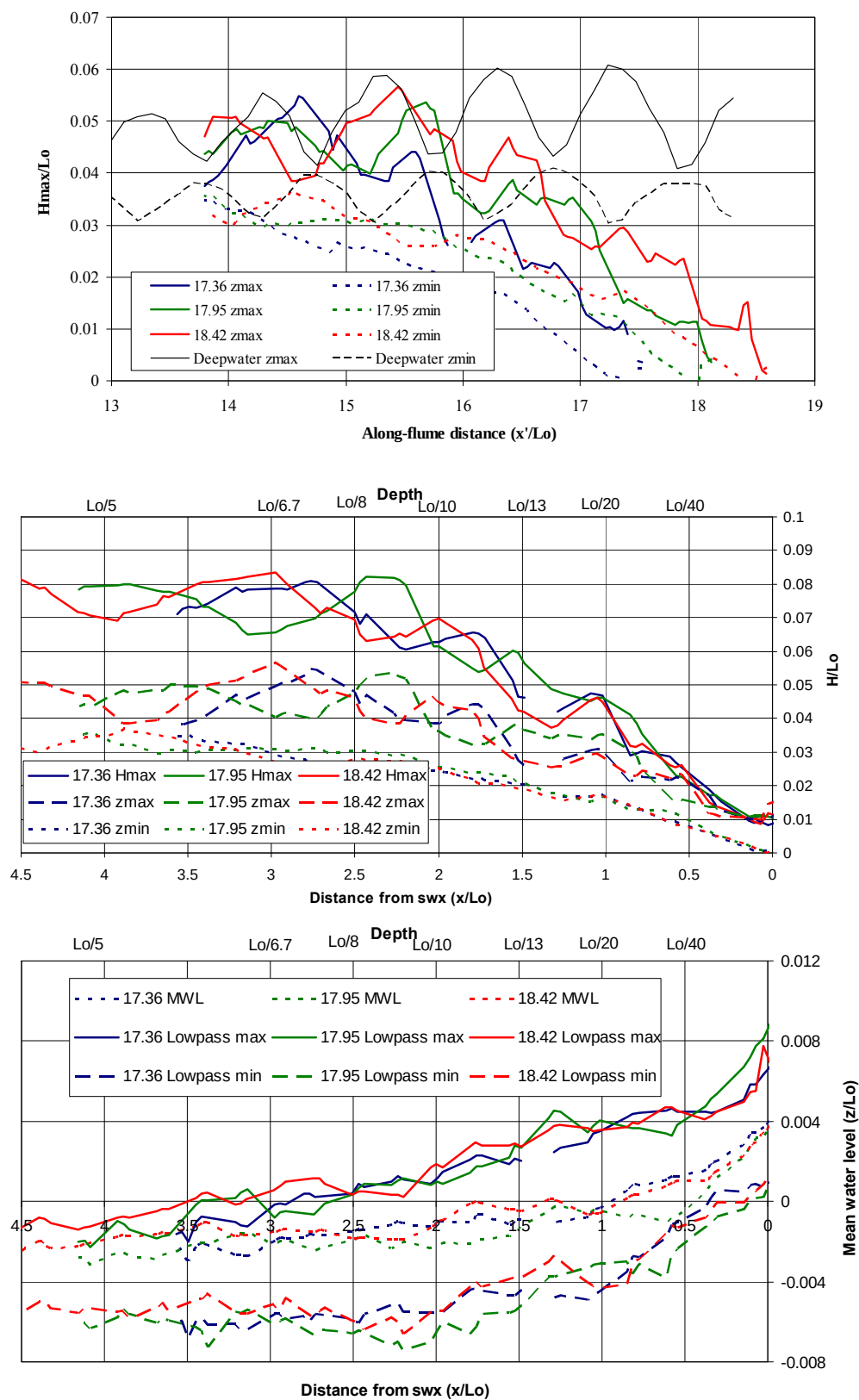
D-3 *C2N5r wave group on 20:1 slope*

D-3a $H_{\max}/d$ ratios as a function of bed position and relative depth

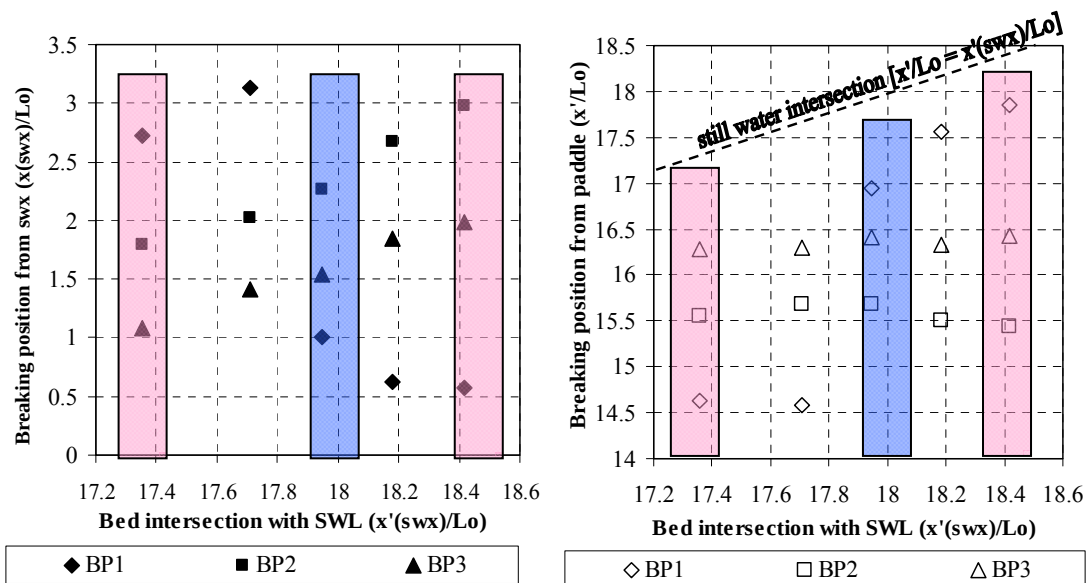


Non-dimensional depth	$L_0/5$	$L_0/7$	$L_0/10$	$L_0/15$	$L_0/20$	$L_0/30$	$L_0/40$
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
17.36	0.40	0.55	0.63	0.67	0.94	0.89	0.96
17.71	0.39	0.51	0.71	0.83	0.73	0.86	0.89
17.95	0.40	0.47	0.61	0.75	0.92	0.87	0.85
18.18	0.39	0.56	0.60	0.58	0.90	0.92	0.97
18.42	0.35	0.56	0.69	0.57	0.89	0.85	0.92
Max difference	0.05	0.09	0.11	0.26	0.21	0.07	0.12

D-3b Evolutional properties of $H_{\max}$, $z_{\max}$, $z_{\min}$, $LP_{\max}$, $LP_{\min}$ and MWL for selected extreme cases

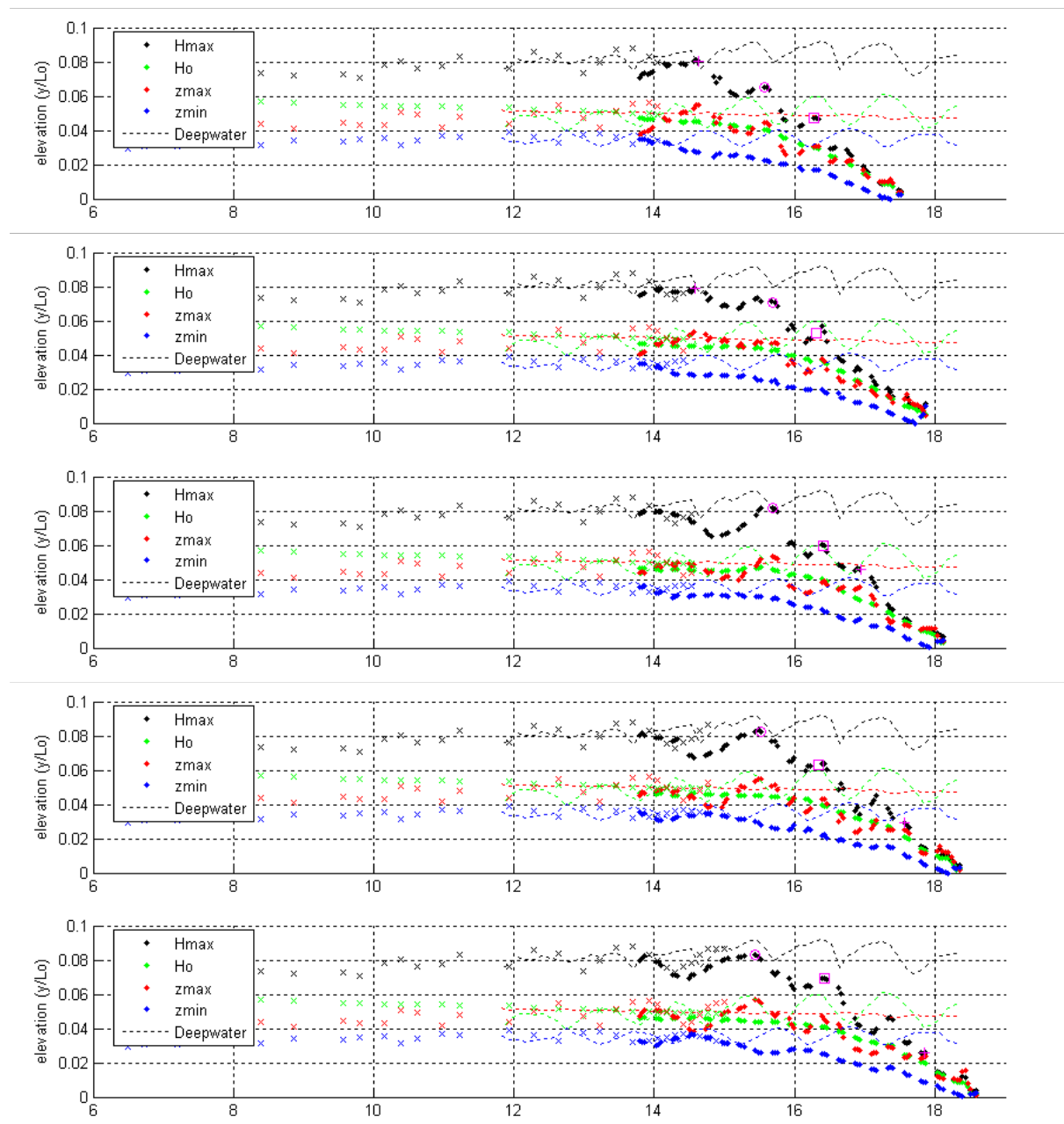


D-3c Breaking location and type as a function of bed position

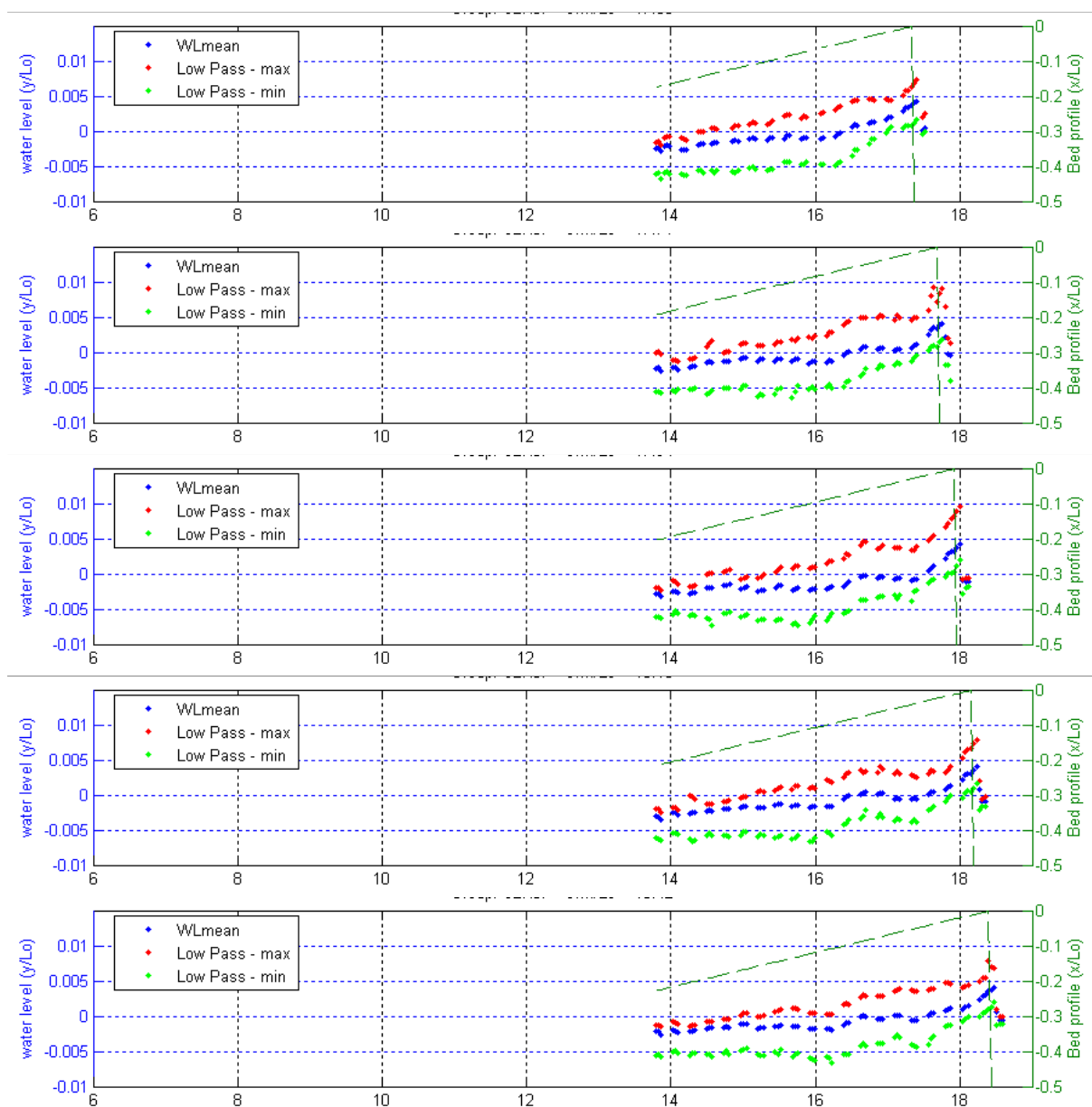


	Break point 1				Break point 2				Break point 3			
Position of swx (x'/L_0)	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type
17.36	2.72	14.64	0.080	1.5	1.79	15.56	0.065	2.5	1.08	16.28	0.047	-
17.71	3.13	14.58	0.079	-	2.03	15.68	0.071	3.5	1.41	16.30	0.053	5.0
17.95	1.00	16.95	0.046	-	2.27	15.68	0.082	2.0	1.54	16.40	0.060	3.5
18.18	0.62	17.56	0.029	-	2.67	15.51	0.082	2.5	1.85	16.33	0.063	4.0
18.42	0.57	17.84	0.026	-	2.98	15.43	0.083	1.5	1.99	16.43	0.069	3.0

D-3d Evolution of $H_{\max}$, H_0 , $z_{\max}$ and $z_{\min}$ compared to deep water values for SWX located at @ $x'/L_0 = 15.90, 16.02, 16.13, 16.25, 16.37, 16.49, 16.61, 16.72, 16.84$

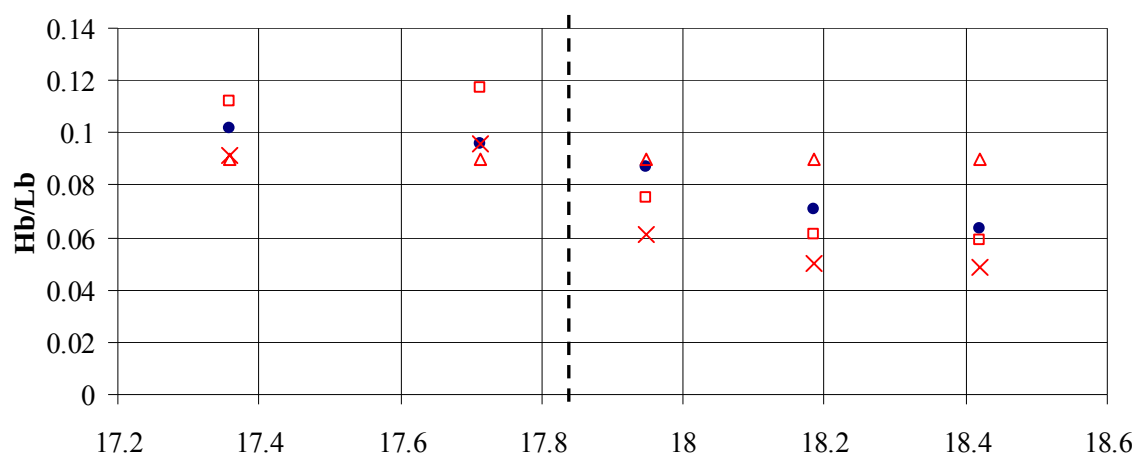


D-3e Evolution of LP_{max} , LP_{min} and mean water level for SWX located at $x'/Lo = 15.90, 16.02, 16.13, 16.25, 16.37, 16.49, 16.61, 16.72, 16.84$

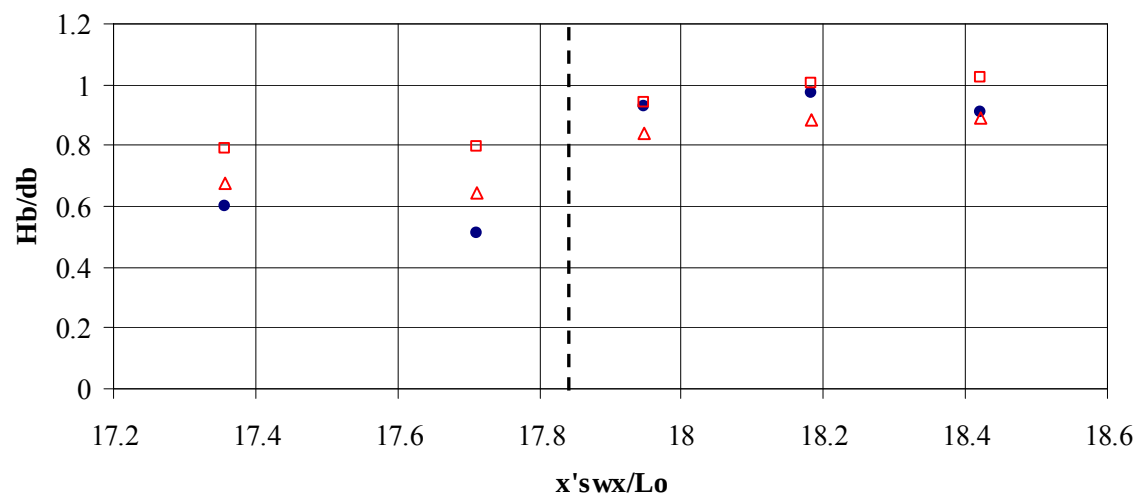


D-3f Observed breaking steepness (H_b/L_b) ($\bullet$) for varying group spatial phasing positions are depicted within (A) for a C2N5r type group propagating onto a 20:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) ($\square$), Kamphuis (1991) ($\times$) and Rattanapitikon and Vivattanasirisak (2003) (Δ). Within (B), observed breaker depth index ($\bullet$) is compared to indices predicted by USACE (2003) ($\square$) and Goda (2000) (Δ). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

A

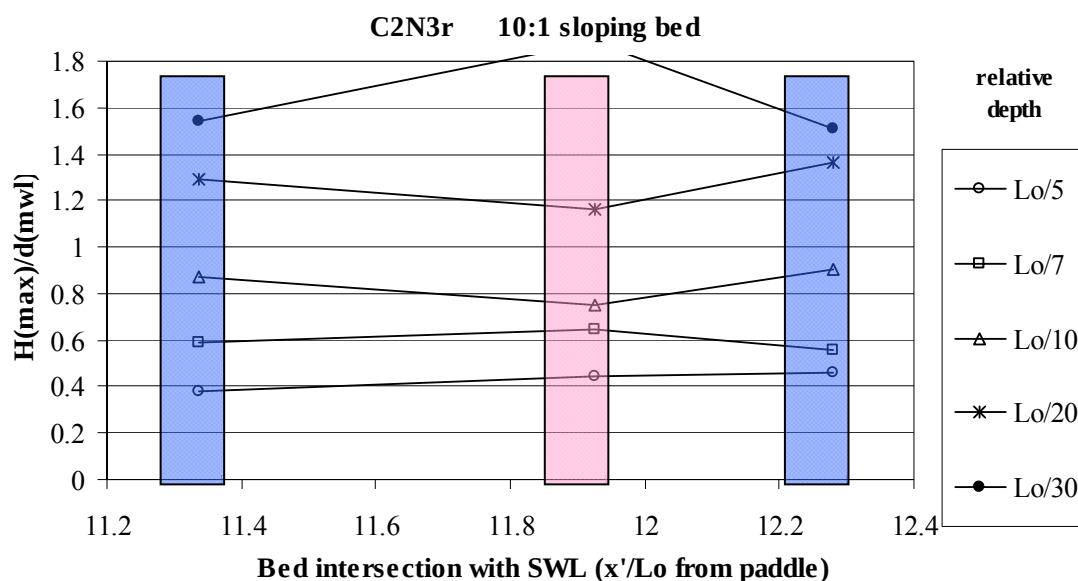


B



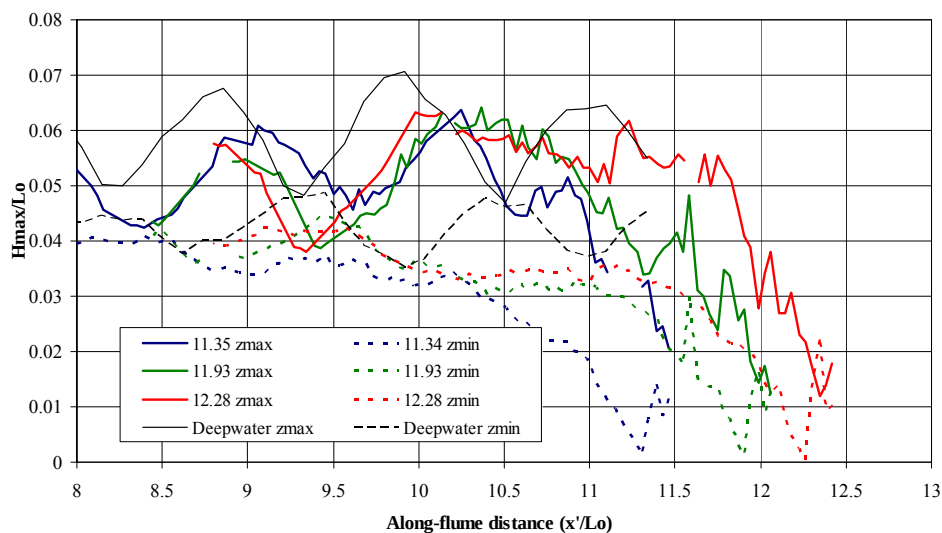
D-4 C2N3r wave group on 10:1 slope

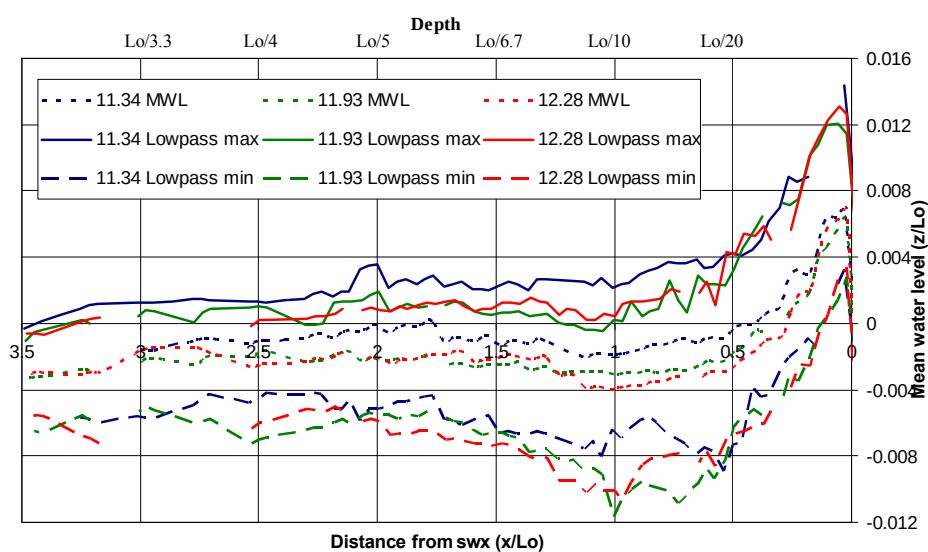
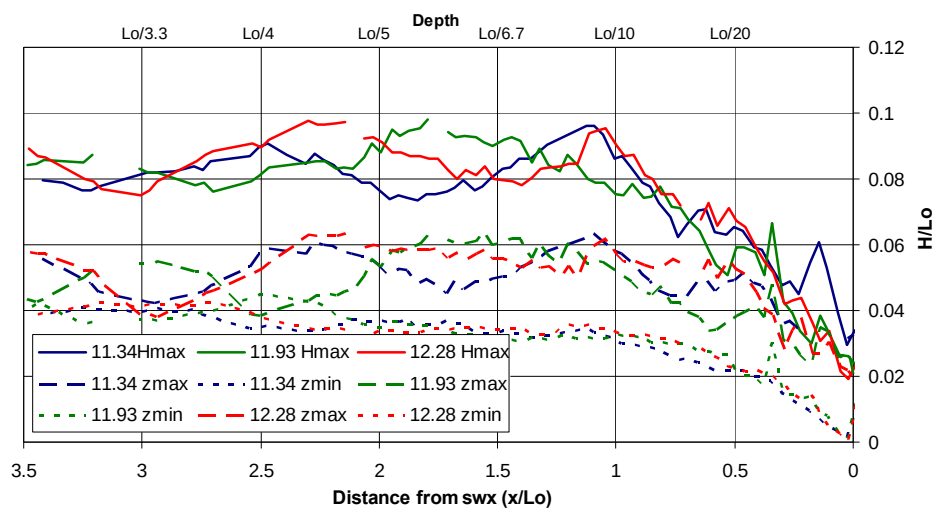
D-4a $H_{\max}/d$ ratios as a function of bed position and relative depth



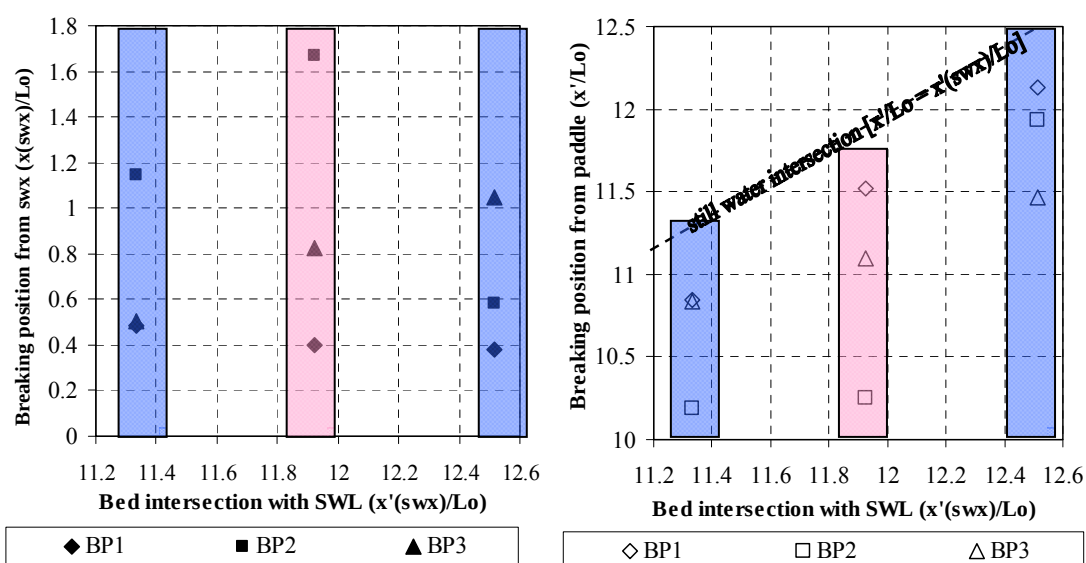
Non-dimensional depth	$L_0/5$	$L_0/7$	$L_0/10$	$L_0/15$	$L_0/20$	$L_0/30$	$L_0/40$
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
11.34	0.38	0.59	0.87	1.05	1.29	1.55	1.92
11.93	0.44	0.65	0.75	0.99	1.16	1.88	1.49
12.28	0.46	0.56	0.90	1.07	1.37	1.51	1.74
Max difference	0.08	0.09	0.15	0.09	0.20	0.37	0.43

D-4b Evolutional properties of $H_{\max}$, $z_{\max}$, $z_{\min}$, $LP_{\max}$, $LP_{\min}$ and MWL for selected extreme cases



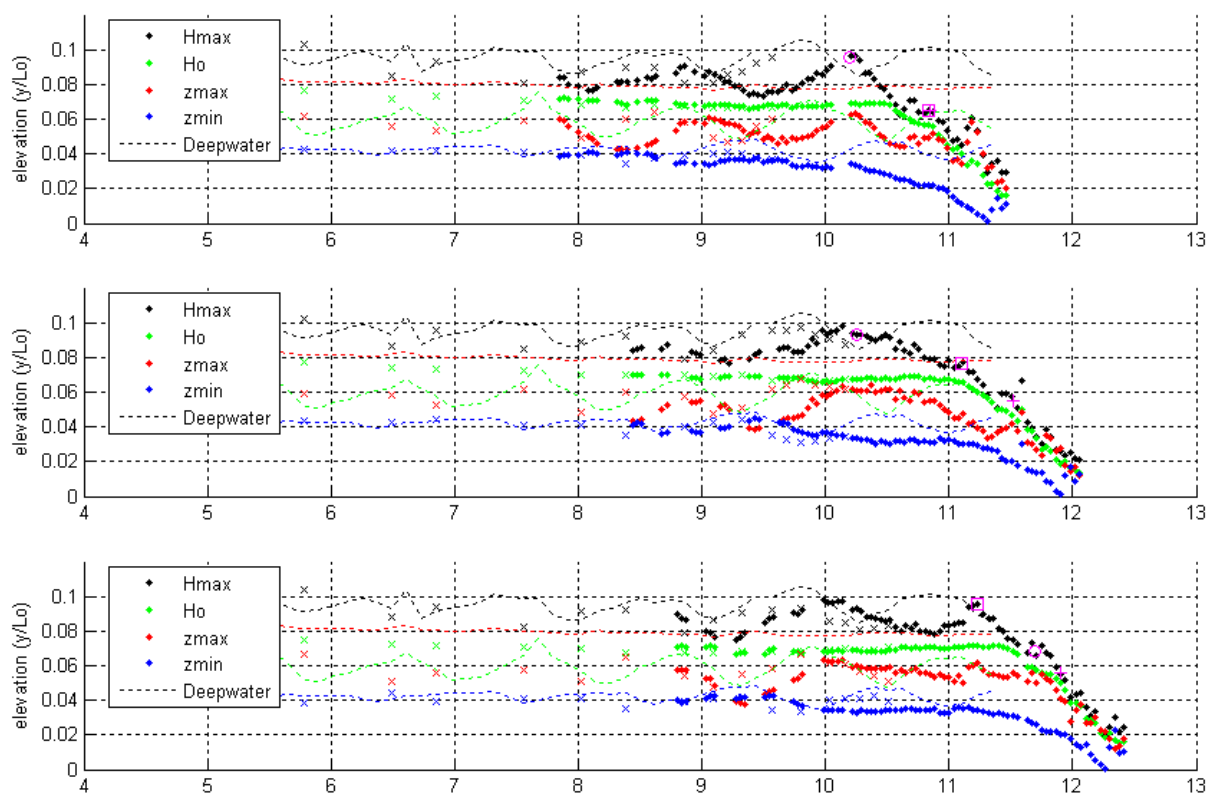


D-4c Breaking location and type as a function of bed position

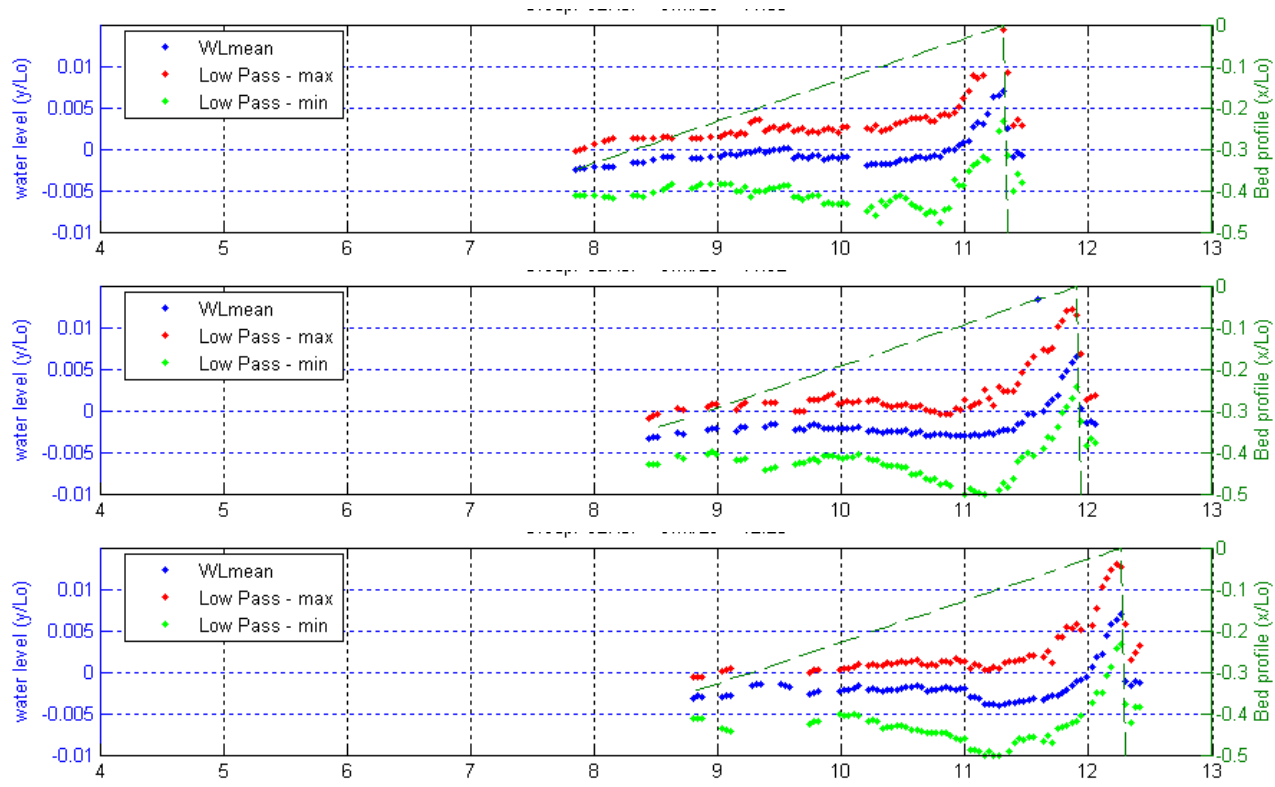


	Break point 1			Break point 2			Break point 3		
Position of swx (x'/L_0)	x/L_0	x'/L_0	H_b/L_0	x/L_0	x'/L_0	H_b/L_0	x/L_0	x'/L_0	H_b/L_0
11.33	0.49	10.85	0.065	1.15	10.19	0.095	0.50	10.83	0.065
11.92	0.40	11.52	0.055	1.67	10.26	0.093	0.83	11.10	0.077
12.51	0.38	12.14	0.056	0.58	11.93	0.068	1.05	11.47	0.095

D-4d Evolution of $H_{\max}$, H_0 , $z_{\max}$ and $z_{\min}$ compared to deep water values for SWX located at @ $x'/L_0 = 11.33, 11.92, 12.51$

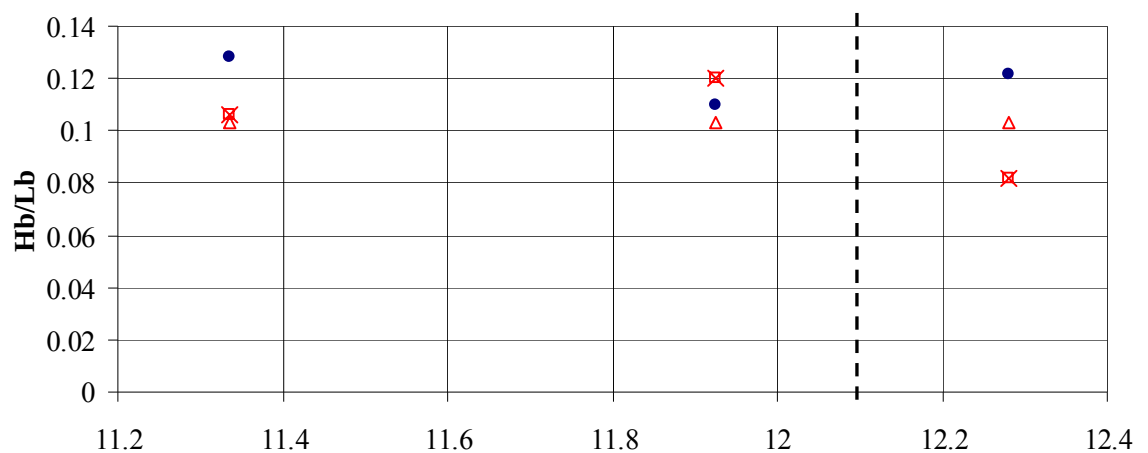


D-4e Evolution of $LP_{\max}$, $LP_{\min}$ and mean water level for SWX located at $x'/Lo = 11.33, 11.92, 12.51$

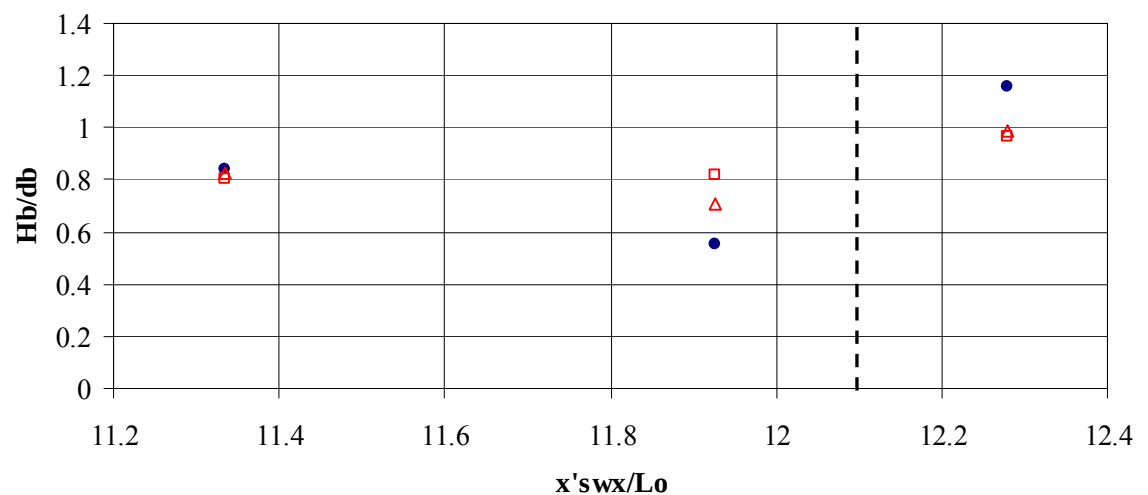


D-4f Observed breaking steepness (H_b/L_b) ($\bullet$) for varying group spatial phasing positions are depicted within (A) for a C2N3r type group propagating onto a 10:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) ($\square$), Kamphuis (1991) ($\times$) and Rattanapitikon and Vivattanasirisak (2003) (Δ). Within (B), observed breaker depth index ($\bullet$) is compared to indices predicted by USACE (2003) ($\square$) and Goda (2000) (Δ). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

A

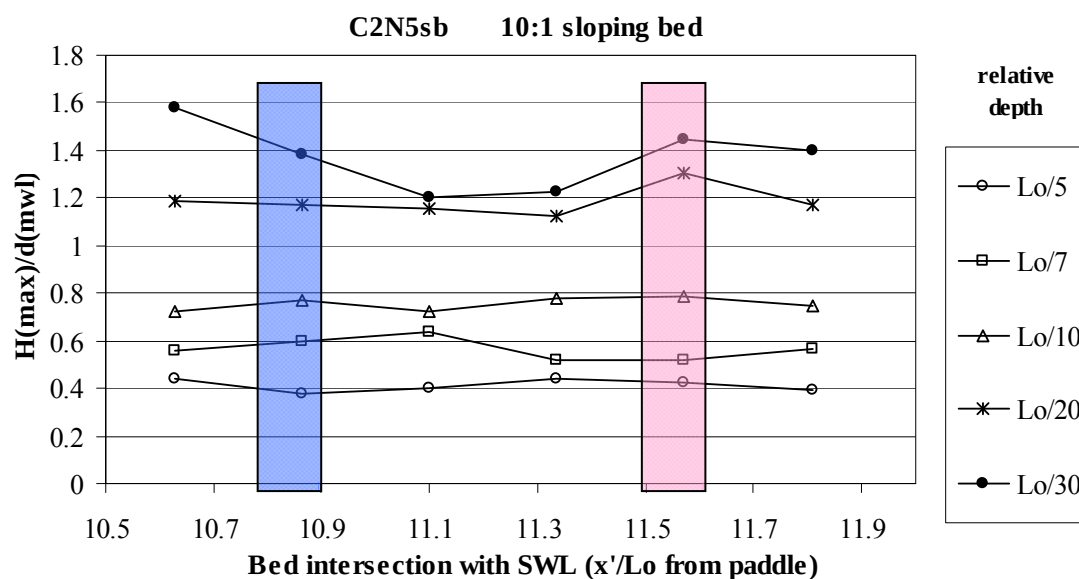


B



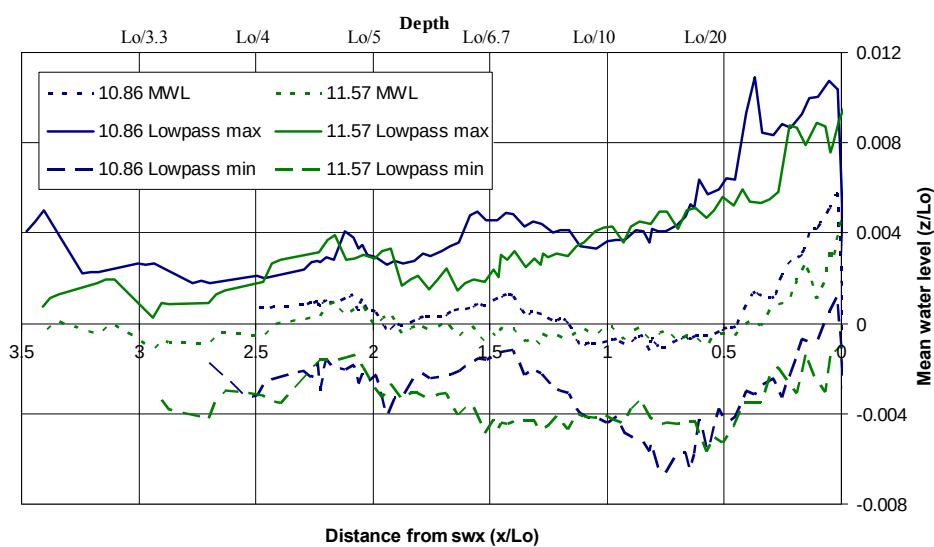
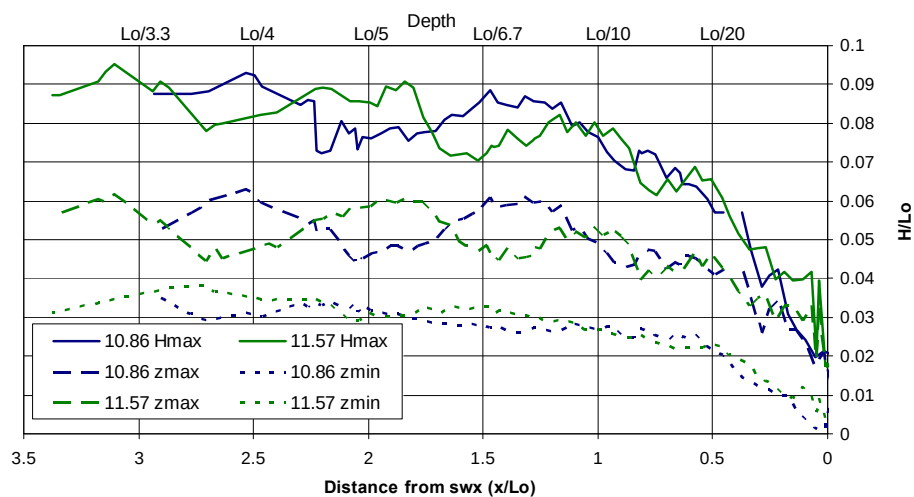
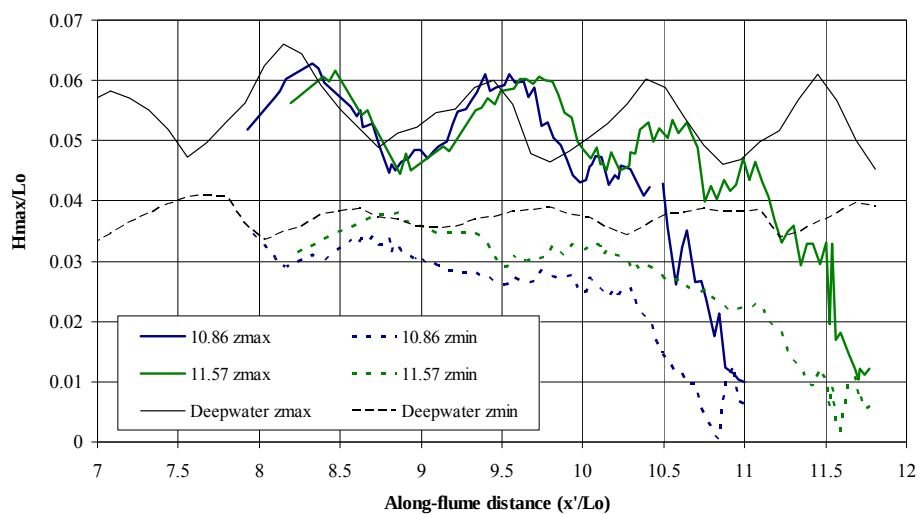
D-5 C2N5sb wave group on 10:1 slope

D-5a $H_{\max}/d$ ratios as a function of bed position and relative depth

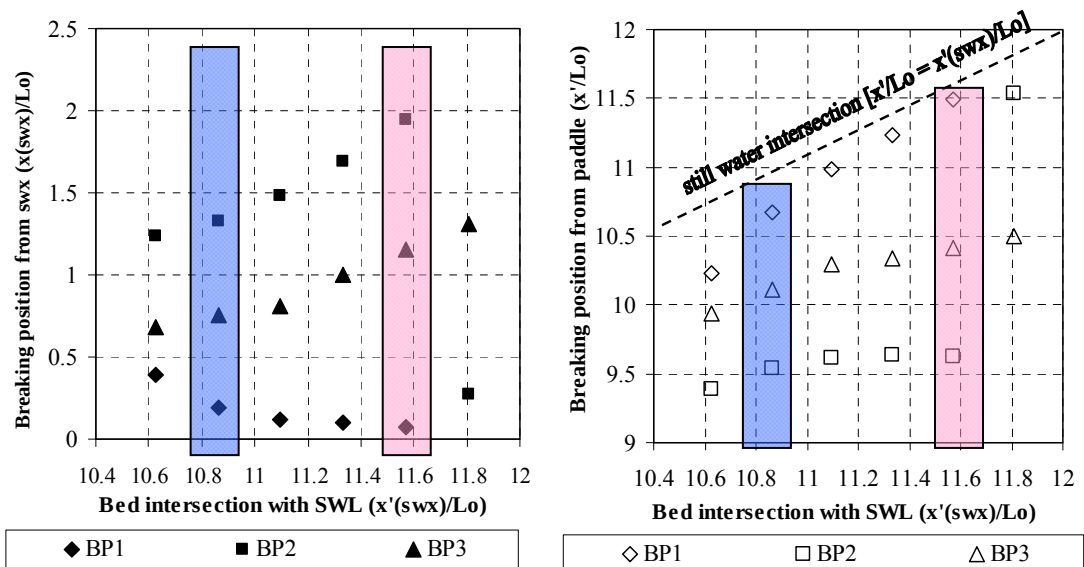


Non-dimensional depth	$L_0/5$	$L_0/7$	$L_0/10$	$L_0/15$	$L_0/20$	$L_0/30$	$L_0/40$
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
10.63	0.44	0.56	0.72	1.07	1.18	1.58	1.70
10.86	0.38	0.59	0.77	1.04	1.17	1.38	1.55
11.10	0.40	0.64	0.73	0.96	1.15	1.20	1.56
11.34	0.44	0.52	0.78	0.96	1.12	1.23	1.65
11.57	0.43	0.52	0.79	0.96	1.31	1.44	1.70
11.81	0.39	0.56	0.75	1.02	1.17	1.40	1.55
Max difference	0.06	0.12	0.07	0.12	0.19	0.38	0.16

D-5b Evolutional properties of $H_{\max}$, $z_{\max}$, $z_{\min}$, $LP_{\max}$, $LP_{\min}$ and MWL for selected extreme cases



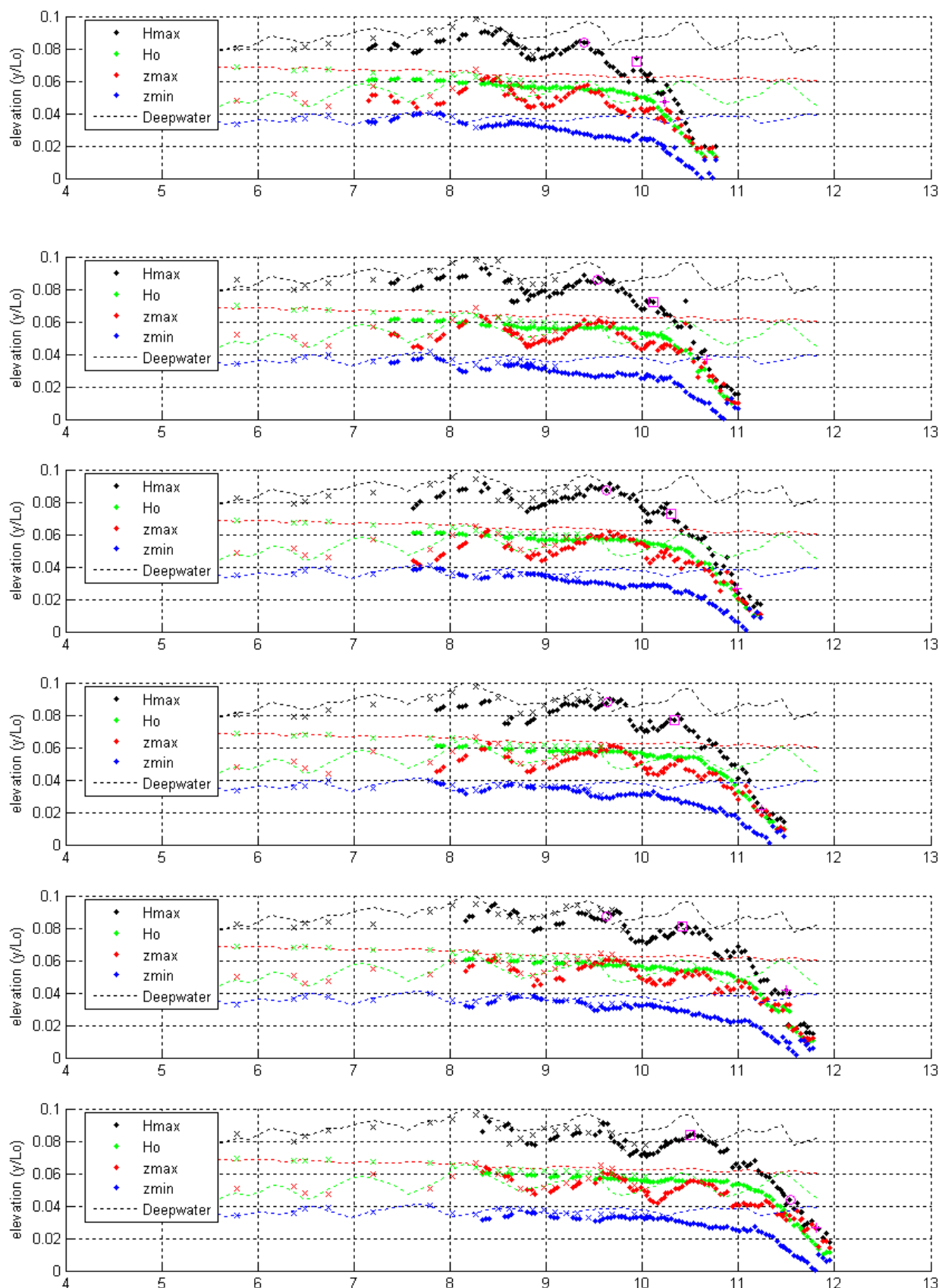
D-5c Breaking location and type as a function of bed position



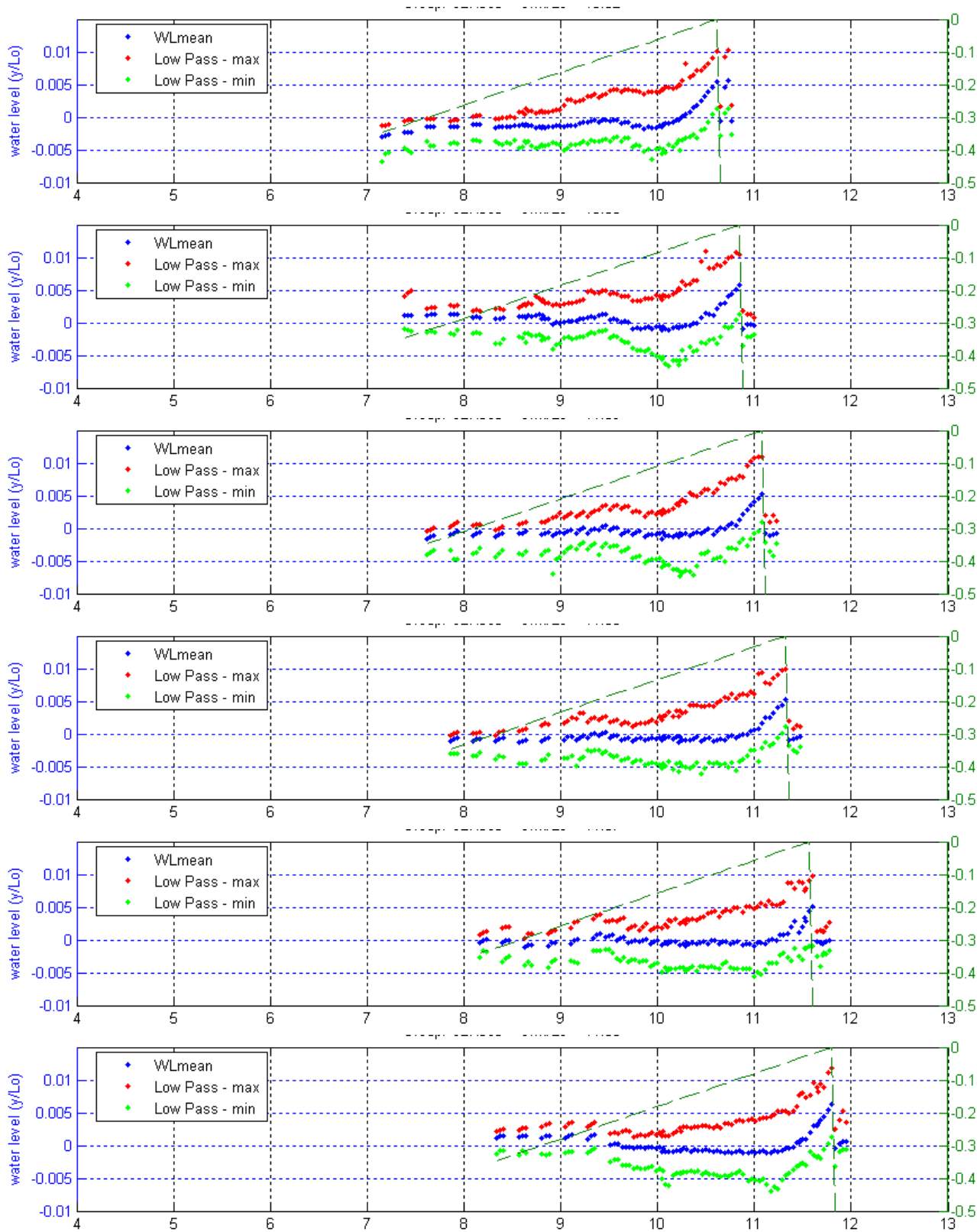
Position of swx (x'/L_0)	Break point 1				Break point 2				Break point 3			
	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type
10.63	0.39	10.23	0.047	5.5	1.24	9.39	0.084	4.0	0.68	9.94	0.072	7.0
10.86	0.19	10.67	0.037	9.0	1.33	9.54	0.086	3.5	0.75	10.11	0.072	7.5
11.10	0.12	10.98	0.026	9.0	1.48	9.62	0.087	3.0	0.80	10.29	0.073	5.0
11.33	0.10	11.24	0.022	9.0	1.70	9.64	0.088	3.0	1.00	10.34	0.077	5.5
11.57	0.07	11.50	0.041	9.5	1.94	9.63	0.087	2.0	1.16	10.41	0.081	4.0
11.81			0.026		0.27	11.54	0.044	8.0	1.31	10.50	0.084	2.5

D-5d Evolution of $H_{\max}$, H_0 , $z_{\max}$ and $z_{\min}$ compared to deep water values for SWX located at @ $x'/L_0 = 10.63, 10.86, 11.10, 11.34, 11.57, 11.81$

7

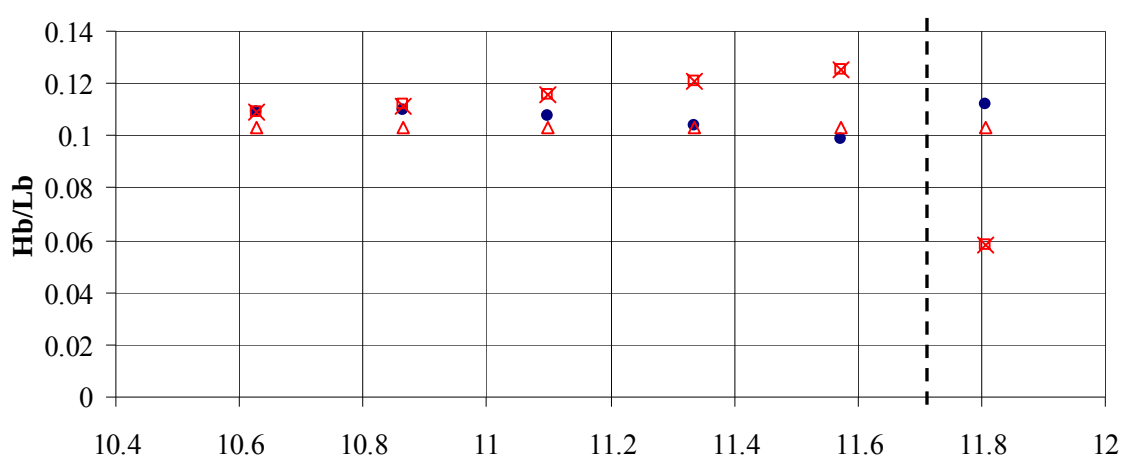


D-5e Evolution of $LP_{\max}$, $LP_{\min}$ and mean water level for SWX located at $x'/Lo = 10.63, 10.86, 11.10, 11.34, 11.57, 11.81$

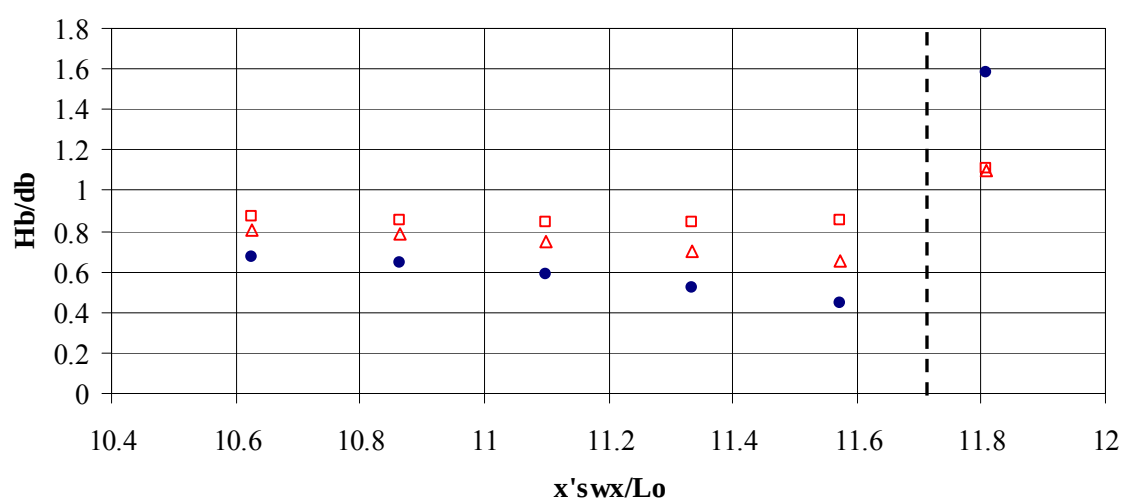


D-5f Observed breaking steepness (H_b/L_b) (●) for varying group spatial phasing positions are depicted within (A) for a C2N5sb type group propagating onto a 10:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) (□), Kamphuis (1991) (×) and Rattanapitikon and Vivattanasirisak (2003) (Δ). Within (B), observed breaker depth index (●) is compared to indices predicted by USACE (2003) (□) and Goda (2000) (Δ). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

A

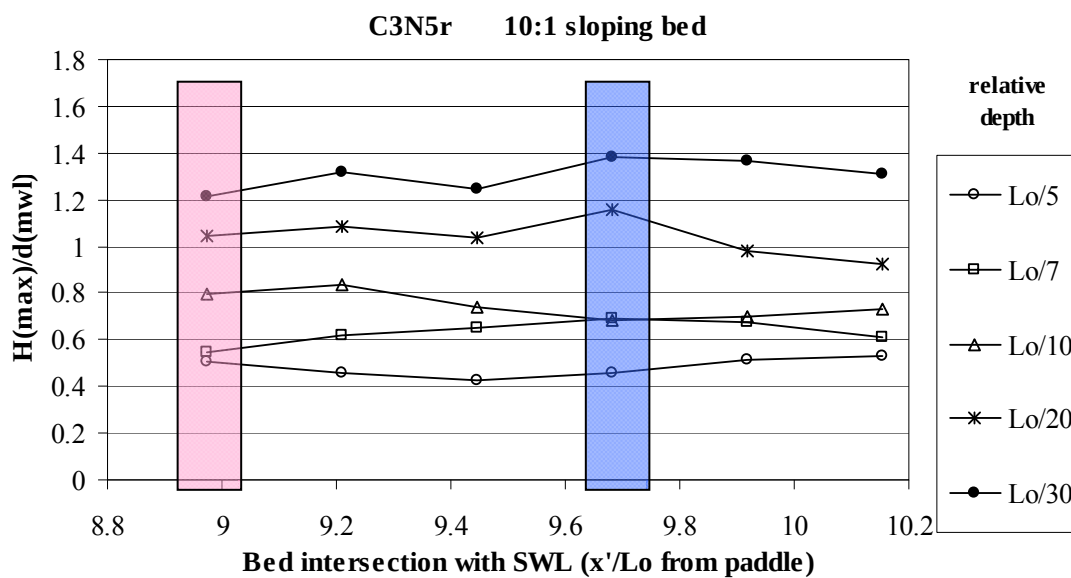


B



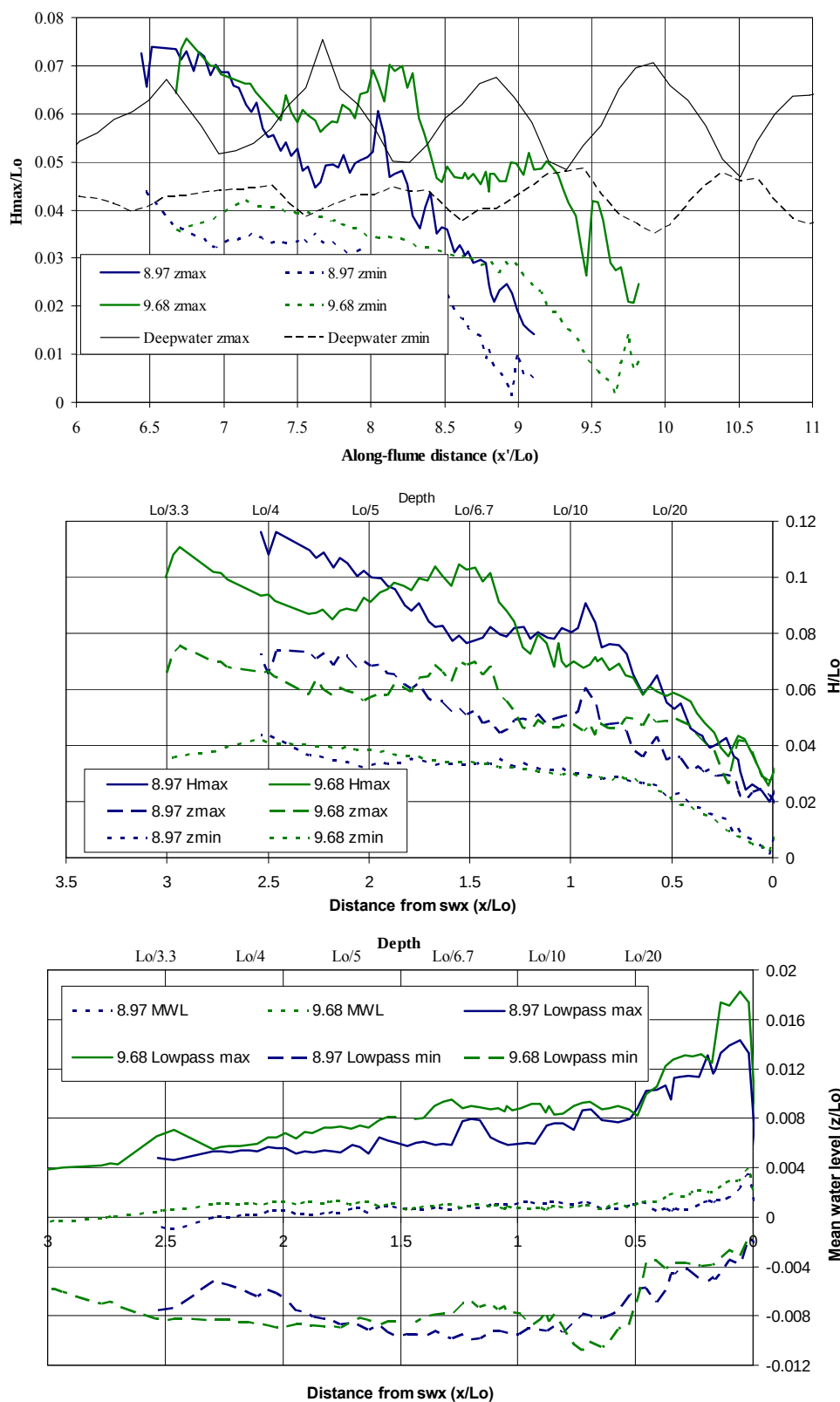
D-6 C3N5r wave group on 10:1 slope

D-6a $H_{\max}/d$ ratios as a function of bed position and relative depth

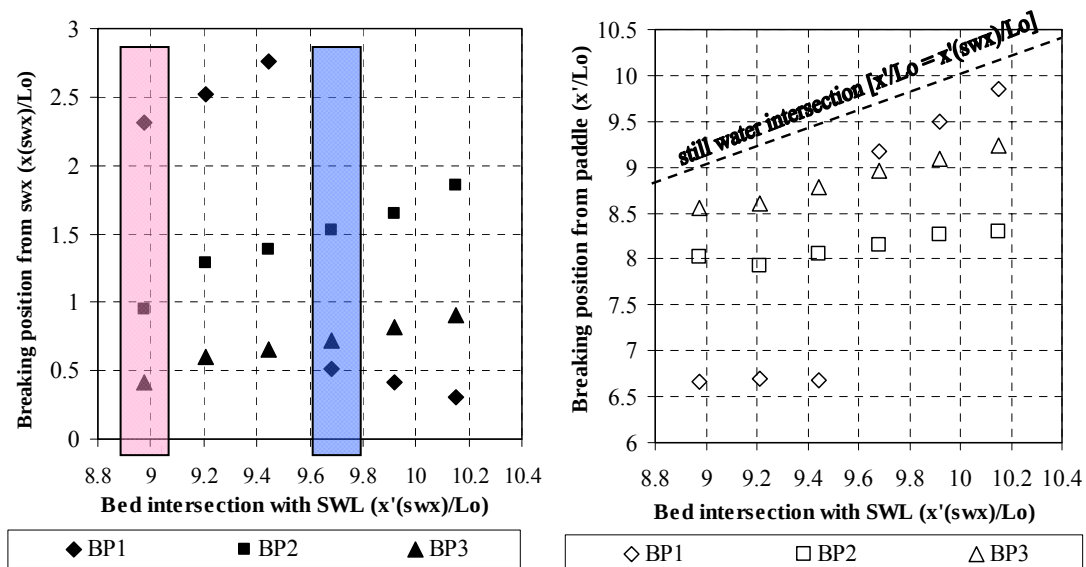


Non-dimensional depth	$L_0/5$	$L_0/7$	$L_0/10$	$L_0/15$	$L_0/20$	$L_0/30$	$L_0/40$
Depth (d/L_0)	0.2	0.142	0.1	0.067	0.05	0.033	0.025
Position of swx (x'/L_0)							
8.97	0.50	0.55	0.80	0.93	1.05	1.21	1.63
9.21	0.46	0.62	0.83	0.97	1.09	1.32	1.50
9.45	0.43	0.65	0.74	1.07	1.03	1.25	1.25
9.68	0.46	0.69	0.69	0.90	1.16	1.38	1.42
9.92	0.51	0.67	0.70	0.92	0.98	1.37	1.48
10.15	0.53	0.61	0.73	0.92	0.92	1.31	1.65
Max difference	0.10	0.14	0.15	0.16	0.24	0.17	0.40

D-6b Evolutional properties of $H_{\max}$, $z_{\max}$, $z_{\min}$, $LP_{\max}$, $LP_{\min}$ and MWL for selected extreme cases

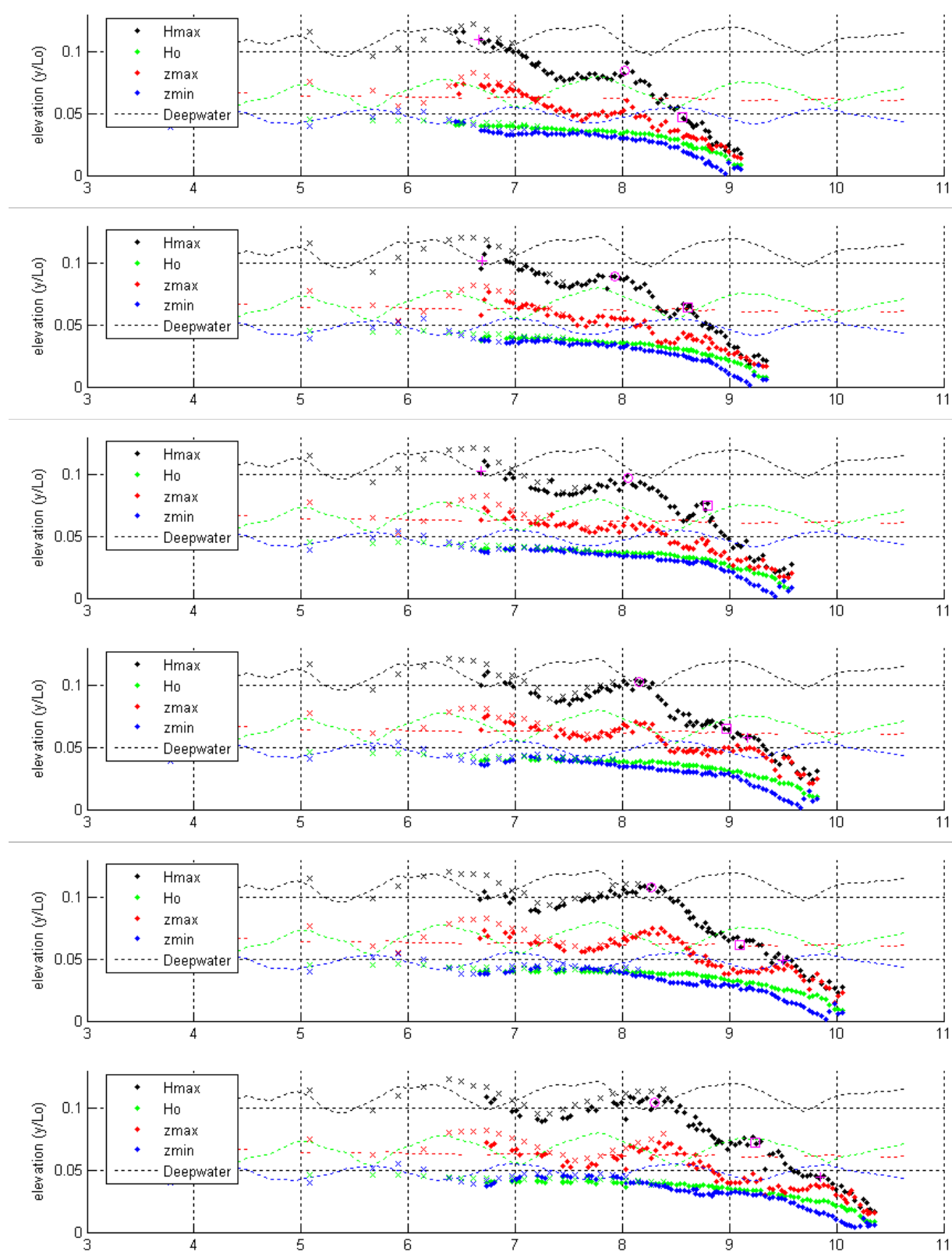


D-6c Breaking location and type as a function of bed position

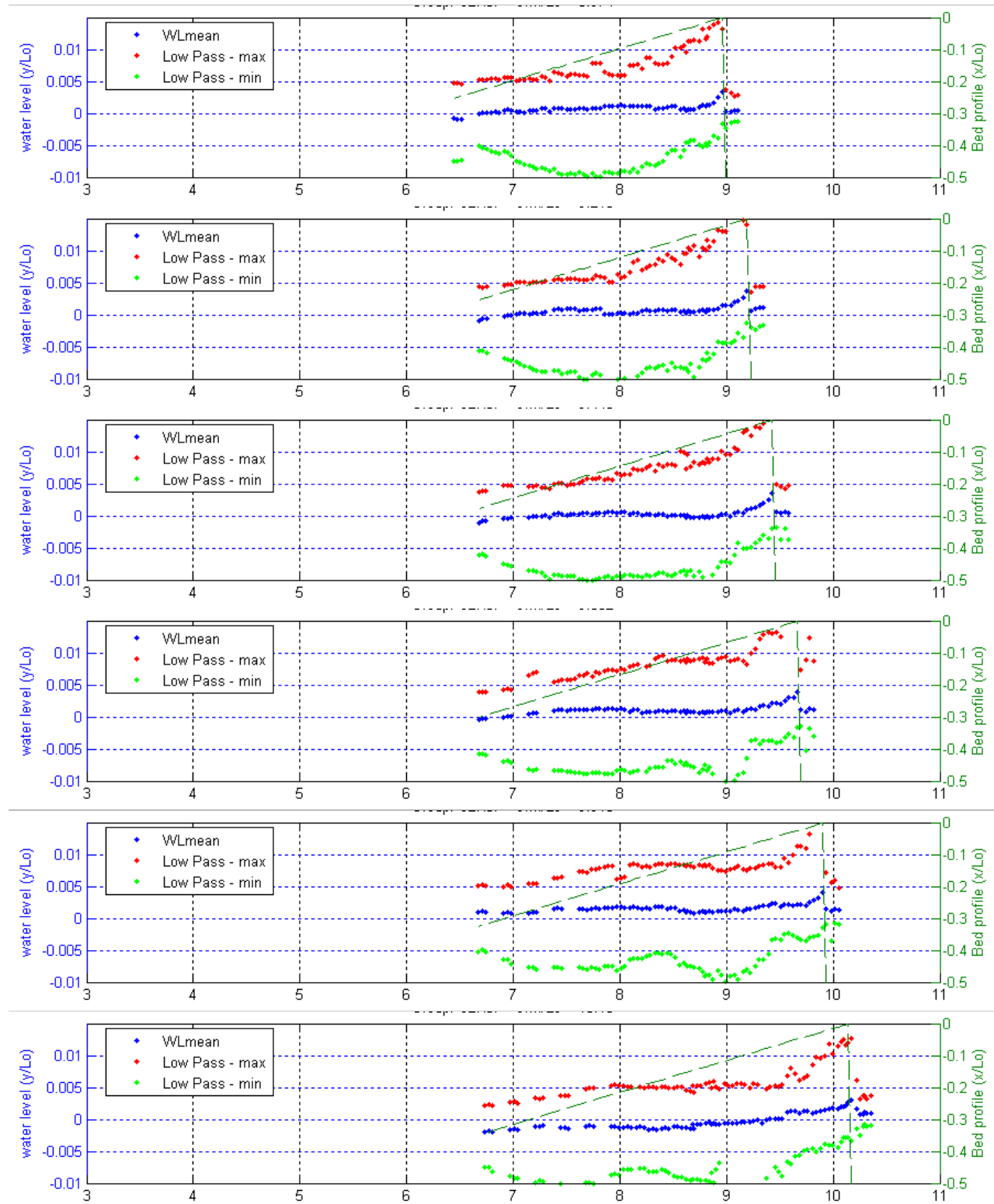


	Break point 1				Break point 2				Break point 3			
Position of swx (x'/L_0)	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type	x/L_0	x'/L_0	H_b/L_0	Break type
8.97	2.31	6.66	0.110		0.95	8.02	0.085		0.41	8.56	0.047	
9.21	2.52	6.69	0.102	2.5	1.28	7.93	0.089	5.5	0.60	8.61	0.065	8.5
9.45	2.76	6.68	0.103	2.0	1.39	8.05	0.097	5.0	0.65	8.79	0.075	7.5
9.68	0.52	9.17	0.058	7.0	1.53	8.16	0.103	3.0	0.72	8.97	0.065	7.5
9.92	0.42	9.50	0.049	7.0	1.65	8.27	0.108	3.0	0.82	9.10	0.061	6.0
10.15	0.30	9.85	0.044	7.5	1.85	8.30	0.104	2.5	0.91	9.24	0.073	7.0

D-6d Evolution of $H_{\max}$, H_0 , $z_{\max}$ and $z_{\min}$ compared to deep water values for SWX located at @ $x'/L_0 = 8.97, 9.21, 9.45, 9.68, 9.92, 10.15$

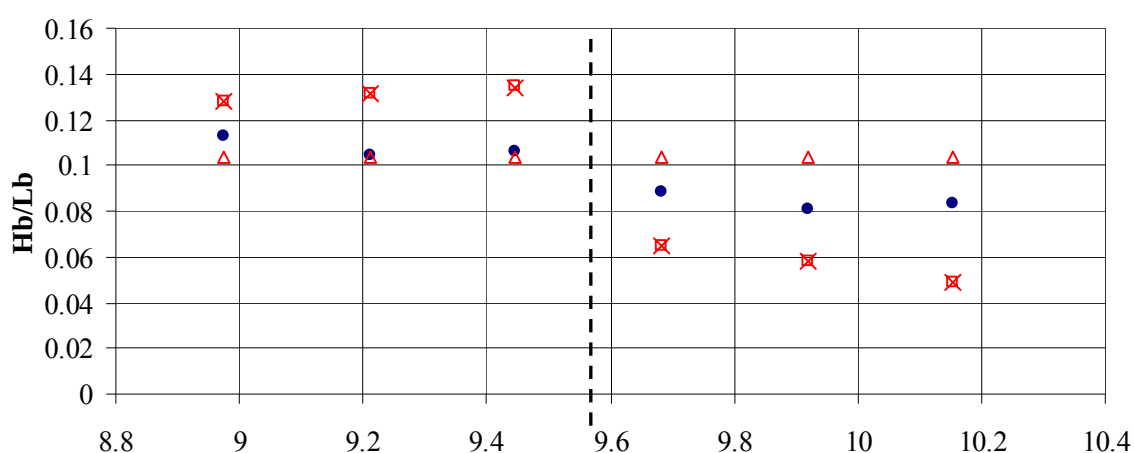


D-6e Evolution of LP_{max} , LP_{min} and mean water level for SWX located at $x'/Lo = 8.97, 9.21, 9.45, 9.68, 9.92, 10.15$

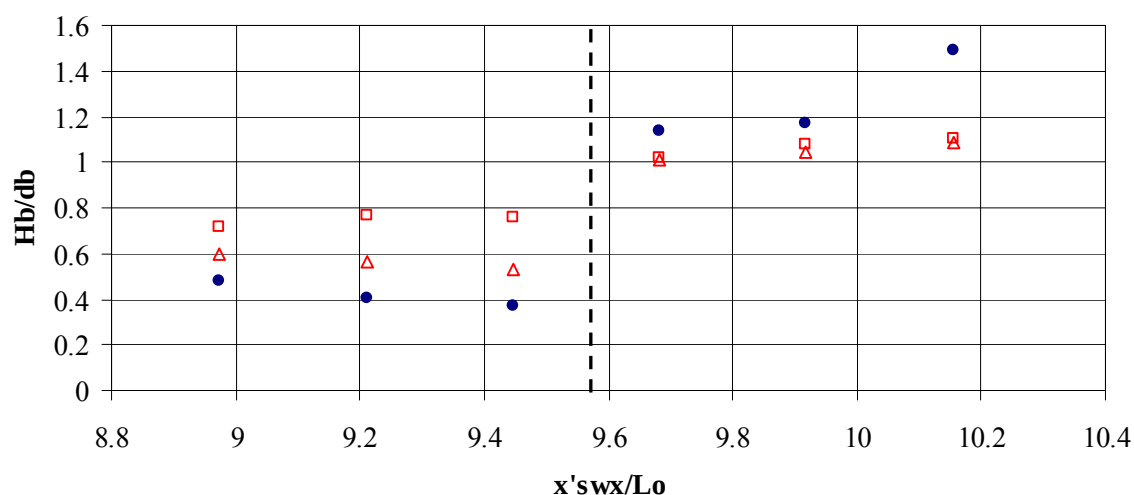


D-6f Observed breaking steepness (H_b/L_b) ($\bullet$) for varying group spatial phasing positions are depicted within (A) for a C3N5r type group propagating onto a 10:1 sloped bed. These are compared to the limiting steepness predicted by Miche (1944) ($\square$), Kamphuis (1991) ($\times$) and Rattanapitikon and Vivattanasirisak (2003) (Δ). Within (B), observed breaker depth index ($\bullet$) is compared to indices predicted by USACE (2003) ($\square$) and Goda (2000) (Δ). The cessation of offshore breaking, after-which breaking position shifts onshore is depicted by a vertical dashed line (:).

A



B



Appendix E. Group effects on overtopping of emerged platforms

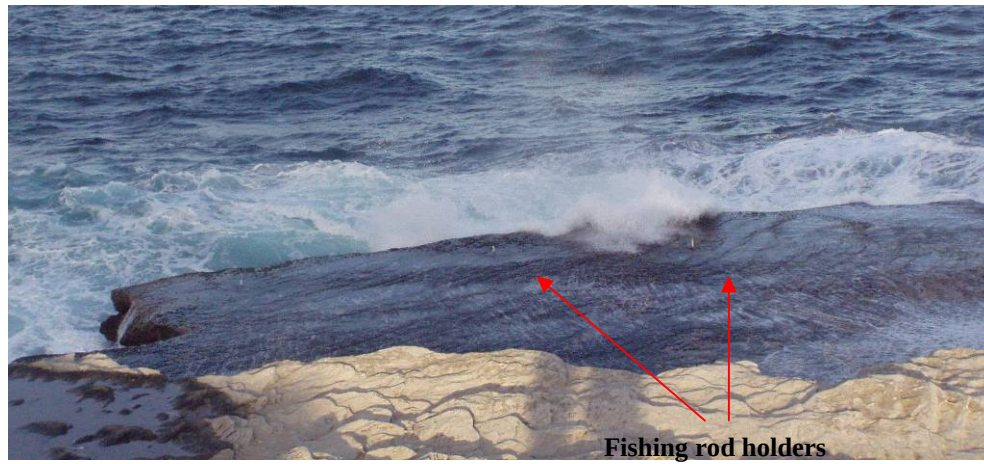
E-1 Site photographs

E-2 Group overtopping imagery

E-3 Group overtopping characteristics

E-1 Site photographs

Photographs 1, 2, 3: A fisherman at Flat Rock, Harbord firstly turning his back on the sea as he walks across the platform, then moving back to his rod and bracing while a wave overtops the rock crest. To the fishermen's credit, they were wearing metal cleats on their footwear and fishing from a higher than average platform location.



Photographs 4, 5, 6, 7: Overtopping at a popular fishing location at Jolong, north of Cape Banks



Photographs 8, 9, 10: Overtopping at a popular fishing location at Malabar in Sydney's Eastern Suburbs



Photograph 11: An Angel-ring installed at Malabar



Photograph 12: Rock fishermen at Malabar providing a reminder that allowances should be made for the stability of smaller persons as well as fully-grown males.



Photographs 13, 14, 15: An example of very rapid green-water overtopping north of Avalon



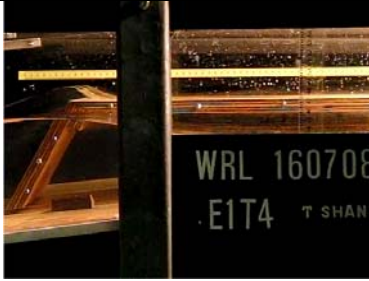
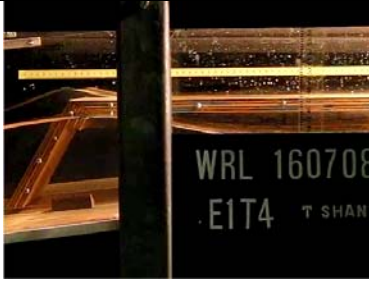
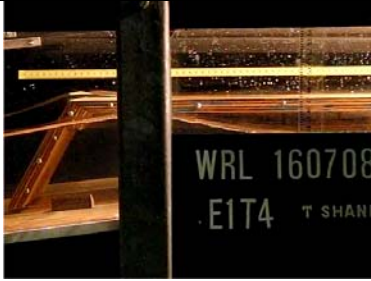
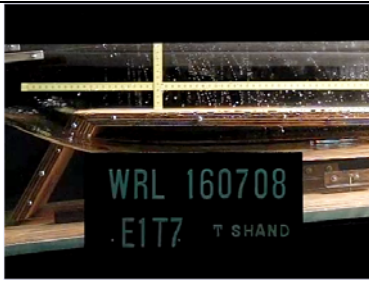
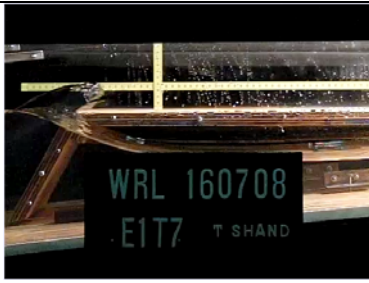
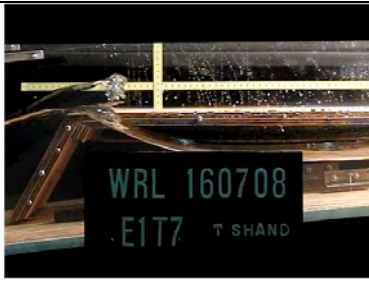
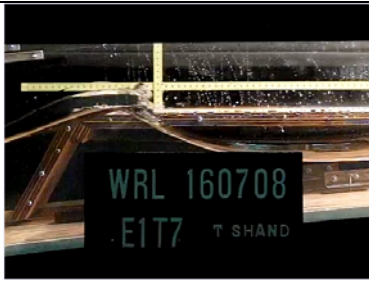
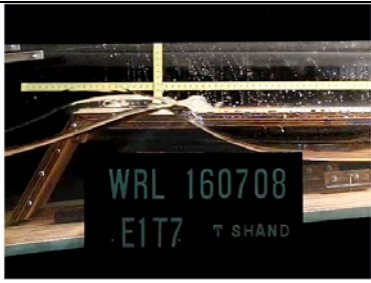
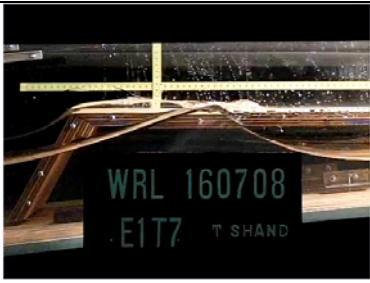
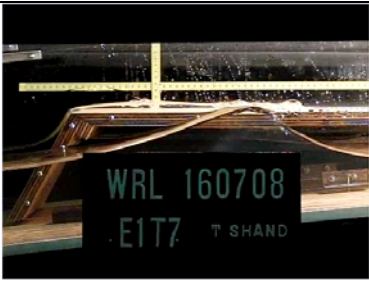
Photographs 16, 17, 18, 19: The Angel-ring at North Curl Curl and significant overtopping occurring during relatively benign offshore wave conditions



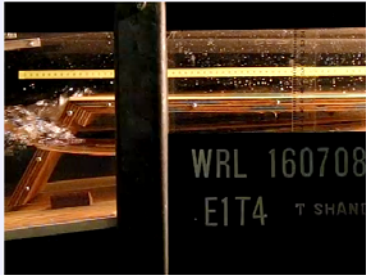
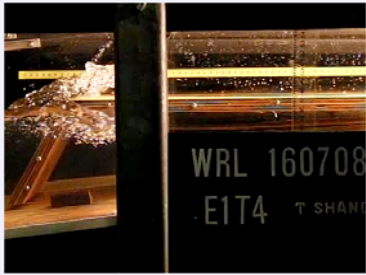
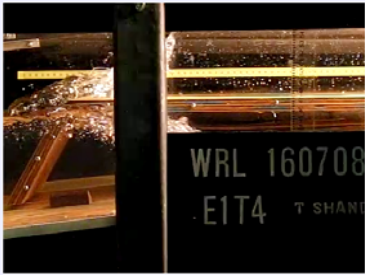
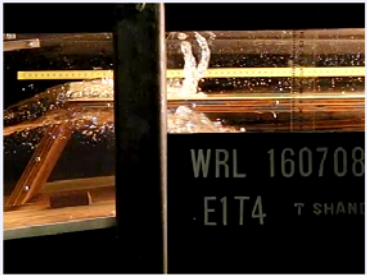
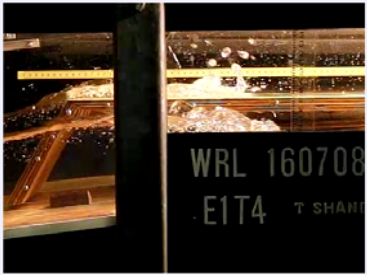
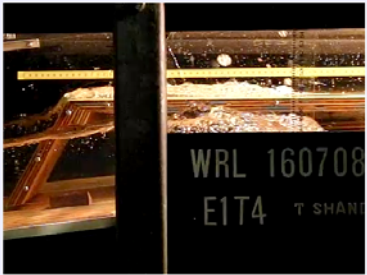
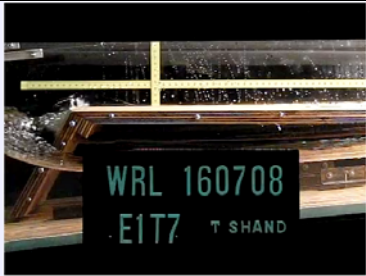
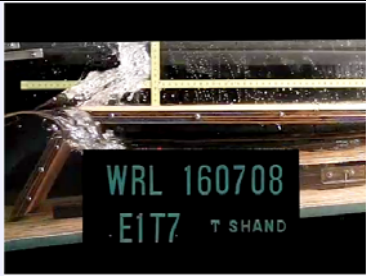
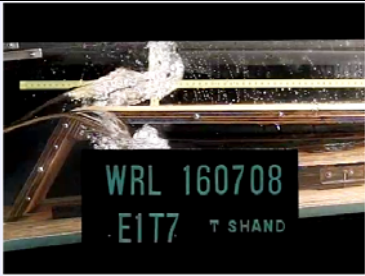
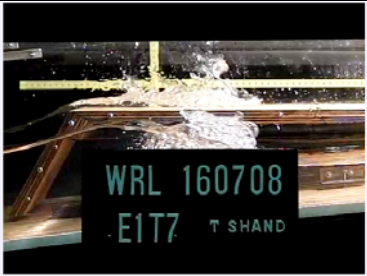
Photographs 20, 21: Significant overtopping occurring during relatively benign offshore wave conditions at North Curl Curl

E-2 Overtopping imagery of the three largest waves for three example group spatial phasing positions

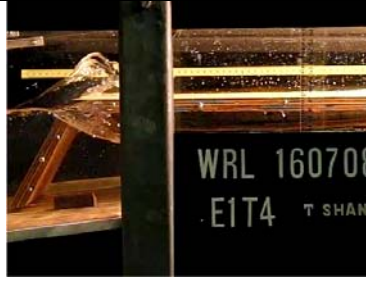
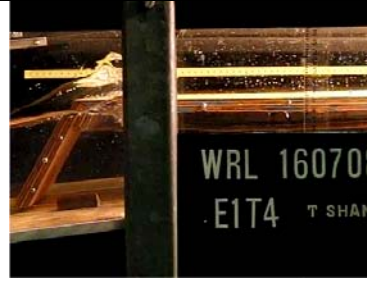
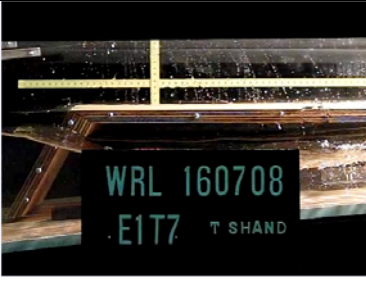
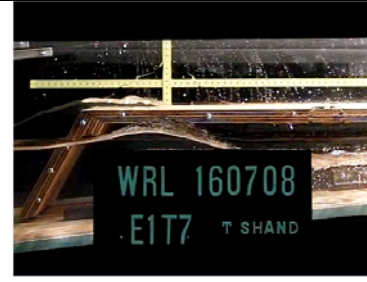
Wave 1

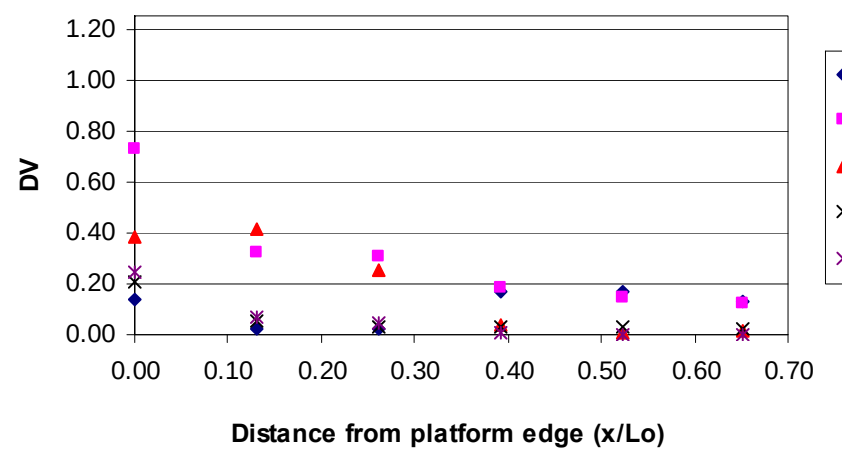
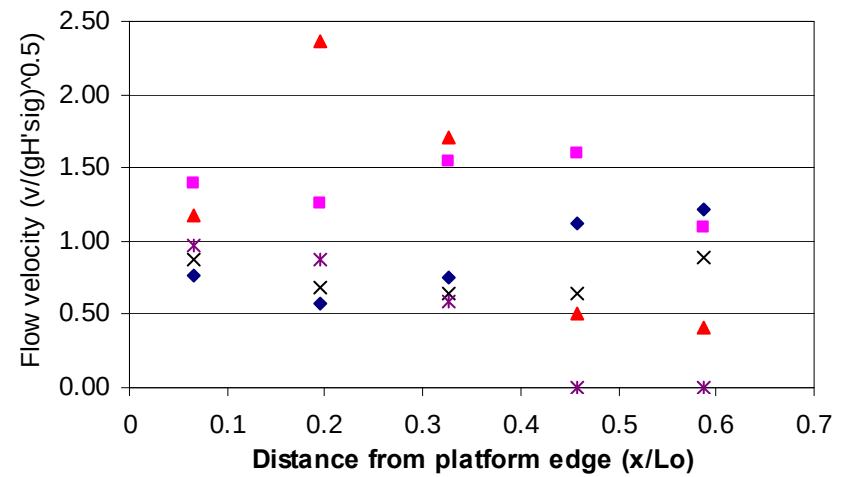
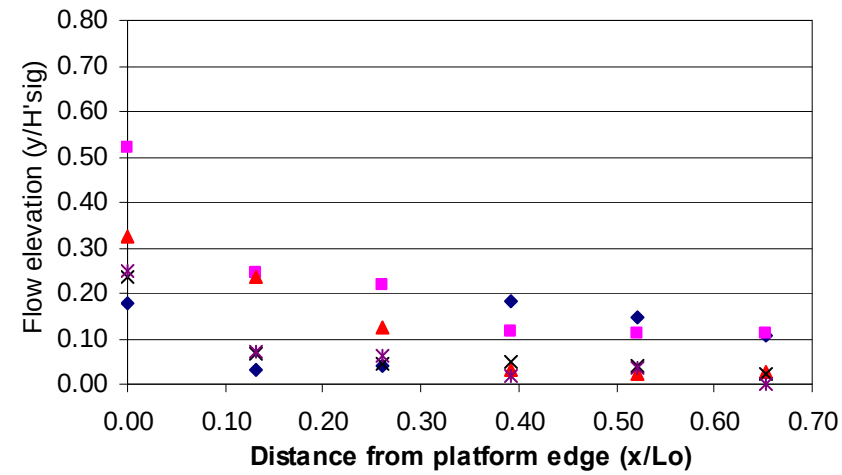
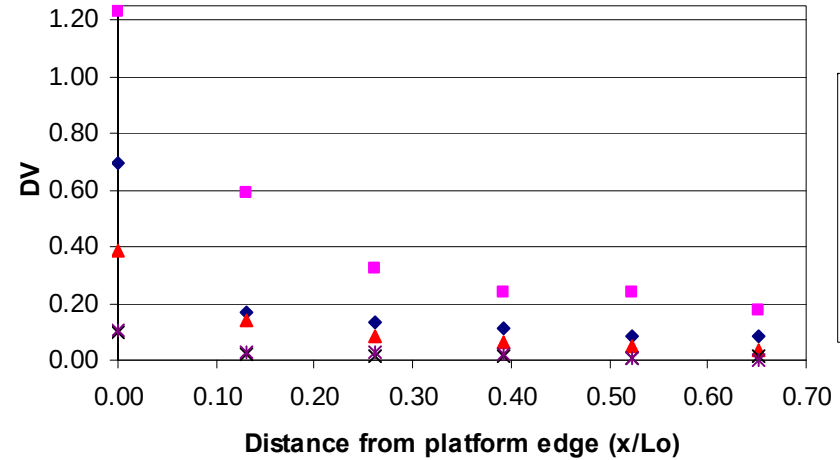
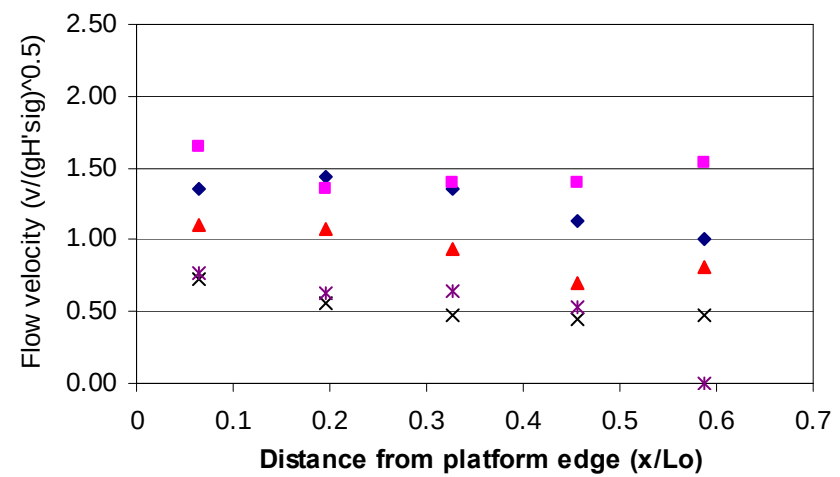
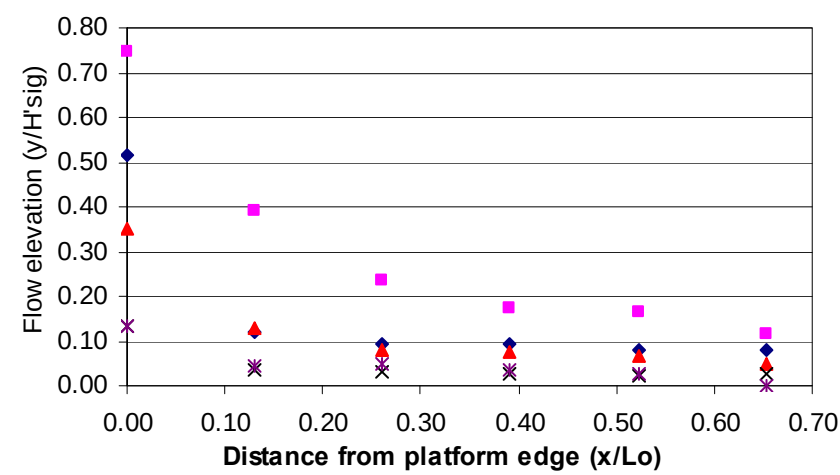
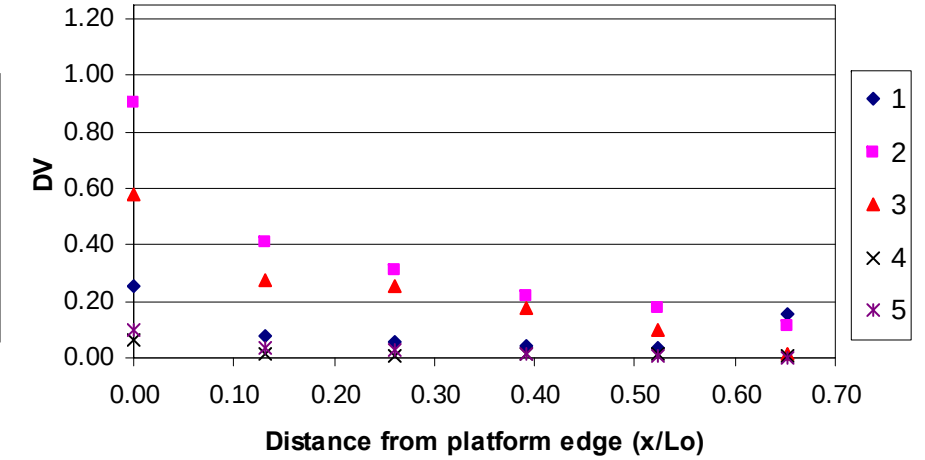
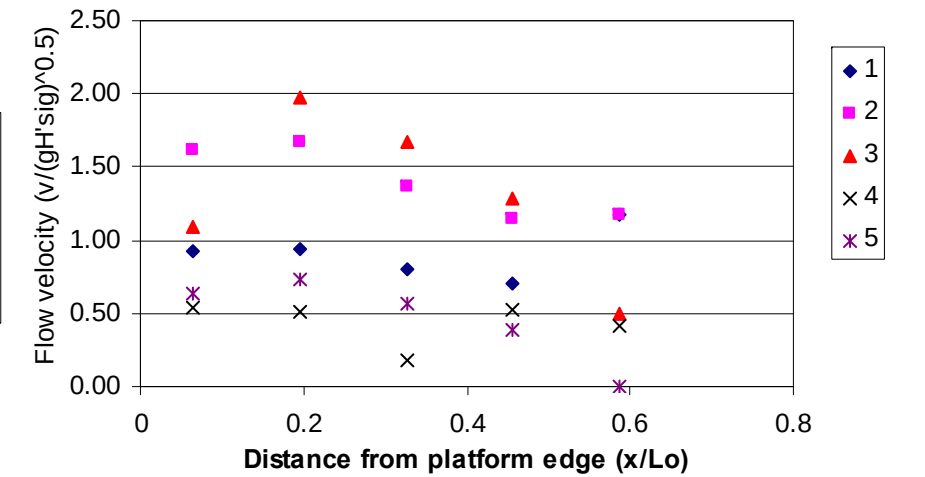
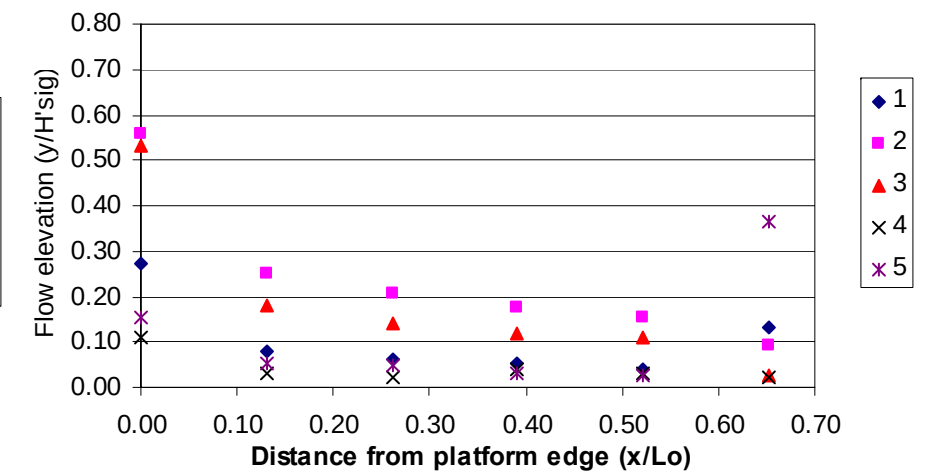
Platform edge (x'/Lo)	T = 0.00	T = 0.05	T = 0.10	T = 0.15	T = 0.20	T = 0.25	T = 0.30
9.24							
9.89							
10.54							

Wave 2

Platform edge (x'/Lo)	T = 0.00	T = 0.05	T = 0.10	T = 0.15	T = 0.20	T = 0.25	T = 0.30
9.24							
9.89							
10.54							

Wave 3

Platform edge (x'/Lo)	T = 0.00	T = 0.05	T = 0.10	T = 0.15	T = 0.20	T = 0.25	T = 0.30
9.24							
9.89							
10.54							

E-3 Overtopping characteristics for three example group spatial phasing positions**Platform edge at $x'/Lo=9.24$** **Platform edge at $x'/Lo=9.289$** **Platform edge at $x'/Lo=10.54$** 

Overtopping flow elevation (D) is non-dimensionalised by deep water significant wave height (H_{sig}) and flow (V) by $(H_{sig} \cdot g)^{0.5}$. D.V is the non-dimensional product of elevation and flow. Waves are numbered in order of overtopping, with the wave number given according to the plot key.

**Appendix F. Design of rock armour for non-
overtopped slopes (USACE, 2003)**

F-1 Method of Hudson (1974)

F-2 Method of van der Meer (1988)

EM 1110-2-1100 (Part VI)
1 Jun 06

Table VI-5-22
Rock, Two-Layer Armored Non-Overtopped Slopes (Hudson 1974)

Irregular, head-on waves

$$\frac{H}{\Delta D_{0.50}} = (K_D \cot \alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha} \quad (\text{VI-5-67})$$

where	H	Characteristic wave height (H_s or $H_{1/10}$)
	$D_{0.50}$	Equivalent cube length of median rock
	M_{50}	Median mass of rocks, $M_{50} = \rho_s D_{0.50}^3$
	ρ_s	Mass density of rocks
	ρ_w	Mass density of water
	Δ	$(\rho_s / \rho_w) - 1$
	α	Slope angle
	K_D	Stability coefficient

K_D -values by SPM 1977, $H = H_s$, for slope angles $1.5 \leq \cot \alpha \leq 3.0$. (Based entirely on regular wave tests.)

Stone shape	Placement	Damage, D^4			
		0-5%		5-10%	10-15%
		Breaking waves ¹	Nonbreaking waves ²	Nonbreaking waves	Nonbreaking waves
Smooth, rounded	Random	2.1	2.4	3.0	3.6
Rough angular	Random	3.5	4.0	4.9	6.6
Rough angular	Special ³	4.8	5.5		

K_D -values by SPM 1984, $H = H_{1/10}$.

Stone shape	Placement	Damage, $D^4 = 0.5\%$	
		Breaking waves ¹	Nonbreaking waves ²
Smooth rounded	Random	1.2	2.4
Rough angular	Random	2.0	4.0
Rough angular	Special ³	5.8	7.0

¹ Breaking waves means depth-limited waves, i.e., wave breaking takes place in front of the armor slope. (Critical case for shallow-water structures.)

² No depth-limited wave breaking takes place in front of the armor slope.

³ Special placement with long axis of stone placed perpendicular to the slope face.

⁴ D is defined according to SPM 1984 as follows: The percent damage is based on the volume of armor units displaced from the breakwater zone of active armor unit removal for a specific wave height. This zone extends from the middle of the breakwater crest down the seaward face to a depth equivalent to the wave height causing zero damage below still-water level.

Shore Protection Manual (1977) versus When considering that $H_{1/10} = 1.27 H_s$ for Rayleigh distributed wave heights (non-depth-limited waves) it is seen that the recommendations of

Shore Protection Manual (1984): *Shore Protection Manual* (1984) introduce a considerable safety factor compared to the practice based on *Shore Protection Manual* (1977).

Uncertainty of the formula: The coefficient of variation of Eq VI-5-67 is estimated to be 18% by van der Meer (1988). Melby and Mlakar (1997) reported a coefficient of variation for K_D of 25% for stone and 20% for Dolosse.

Table VI-5-23
Rock, Two-Layer Armored Non-Overtopped Slopes (van der Meer 1988)

Irregular, head-on waves

$$\frac{H_s}{\Delta D_{n50}} = 6.2 \cdot S^{0.2} P^{0.18} N_z^{-0.1} \xi_m^{-0.5} \quad \text{Plunging waves : } \xi_m < \xi_{mc} \quad (\text{VI-5-68})$$

$$\frac{H_s}{\Delta D_{n50}} = 1.0 \cdot S^{0.2} P^{-0.13} N_z^{-0.1} (\cot \alpha)^{0.5} \xi_m^P \quad \text{Surging waves : } \xi_m > \xi_{mc} \quad (\text{VI-5-69})$$

$$\xi_m = s_m^{-0.5} \tan \alpha \quad \xi_{mc} = \left(6.2 P^{0.31} (\tan \alpha)^{0.5} \right)^{1/(P+0.5)}$$

where	H_s	Significant wave height in front of breakwater
	D_{n50}	Equivalent cube length of median rock
	ρ_s	Mass density of rocks
	ρ_w	Mass density of water
	Δ	$(\rho_s/\rho_w) - 1$
	S	Relative eroded area (see Table VI-5-21 for nominal values)
	P	Notional permeability (see Figure VI-5-11)
	N_z	Number of waves
	α	Slope angle
	s_m	Wave steepness, $s_m = H_s/L_{om}$
	L_{om}	Deepwater wavelength corresponding to mean wave period

Validity:

- 1) Equations VI-5-68 and VI-5-69 are valid for non-depth-limited waves. For depth-limited waves H_s is replaced by $H_{2\%}/1.4$.
- 2) For $\cot \alpha \geq 4.0$ only Eq VI-5-68 should be used.
- 3) $N_z \leq 7,500$ after which number equilibrium damage is more or less reached.
- 4) $0.1 \leq P \leq 0.6$, $0.005 \leq s_m \leq 0.06$, $2.0 \text{ tonne/m}^3 \leq \rho \leq 3.1 \text{ tonne/m}^3$
- 5) For the 8 tests run with depth-limited waves, breaking conditions were limited to spilling breakers which are not as damaging as plunging breakers. Therefore, Eqs VI-5-68 and VI-5-69 may not be conservative in some breaking wave conditions.

Uncertainty of the formula: The coefficient of variation on the factor 6.2 in Eq VI-5-68 and on the factor 1.0 in Eq VI-5-69 are estimated to be 6.5% and 8%, respectively.

Test program: See Table VI-5-4.