ESTIMATION OF MAXIMUM POSSIBLE REPRESENTATIVE WAVE HEIGHTS IN SURF ZONE

BY

KHIEM QUANG TRAN

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (ENGINEERING AND TECHNOLOGY)

SIRINDHORN INTERNATIONAL INSTITUTE OF TECHNOLOGY

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Abstract

ESTIMATION OF MAXIMUM POSSIBLE REPRESENTATIVE WAVE HEIGHTS IN SURF ZONE

By

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Maximum possible representative wave heights ($H_{rep,max}$) are an essential required factor for design of coastal structures. Breaker height formulas were applied to compute maximum possible representative wave heights in this study. The maximum possible representative wave heights approach is usually used when a long-term measured wave data of some coastal areas are not available or less reliable. In practice, $H_{rep,max}$ are usually determined from the formulas computing for representative wave heights at the incipient breaking point ($H_{rep,b}$) based on an assumption that $H_{rep,b} = H_{rep,max}$. Because the existing formulas are not general fomulas for computing common representative wave heights, main objective of this study is general formulas used for predicting representative wave heights (H_{rep}). The number of experimental data is an important impact factor and affects to predictive ability of empirical formulas. Therefore, a total of 60824 data points from 39 sources [including 18 experiments of regular wave heights (with 547 cases and 8744 data points), 21 experiments of irregular wave heights (with 2022 cases and 52080 data points)] of pulished experimental data were used to examine and modify the existing breaker height formulas. Fourteen existing breaker fomulas were examined to identify the best formula for computing the maximum possible wave height. The best three formulas were modified to be better in estimation of maximum possible wave height ($H_{max,max}$ *or* H_{mp}). Subsequently, top two modified formulas for computing maximum possible wave

height were chosen against considering accuracy and simplicity of the formulas, to extend to compute other maximum possible representative wave heights.

Keywords: Breaking-limited wave height, Maximum possible wave height, Representative wave heights, Surf zone

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Table of Contents

List of Tables

Tables **Page**

List of Figures

Figures **Page**

Chapter 1
Introduction

1.1 General

A coastal structure, which is an important function to solve problems relating to coastal area, is designed based on many significant factors. One of the most significant factors is wave height at an arbitrary position, thus several methods have been proposed by many researchers to determine the design wave height.

There are many types of representative wave heights (H_{rep}), i.e. maximum wave height (H_{max}), one-tenth wave height ($H_{1/10}$), one-third wave height ($H_{1/3}$), spectral significant wave height (H_{mo}), root-mean-square wave height (H_{rms}), and mean wave height (H_m). Comparing among the $H_{rep,}$ the highest one-third wave height and maximum wave height are often used to design a structure. Because structures are constructed to sustain wave conditions during many years, structure's height should be able to withstand the maximum wave height in those of years. It means that the height of structures is determined based on the design wave height at a particular depth. How to determine design wave height with the high accuracy is the big problem, because overestimate or underestimate will make the structures too waste or less safe, respectively. Some researchers proposed formulas to compute maximum possible representative wave heights ($H_{rep,max}$) relied on traditional method, which is breaking limited approach. This method is often used in the area where longterm measured wave data are not available or less reliable. Hence, it is used in this study.

1.2 The statement of problems

During many years, several literatures have researched on the breaker height (H_b) formulas, which can be used to compute the maximum possible representative wave heights. As all existing formulas were developed based on empirical or semi-empirical approach, limit of existing formulas's applicability depends on the number of experimental data. Consequently, the existing formulas may not be appropriate for general conditions. Moreover, although many existing H_b formulas are available, the applicability of the H_b formulas to determine $H_{rep,max}$ is not clear in surf zone.

1.3 The purposes of study

The purposes of the present study are as follows:

- To verify the applicability of existing H_b formulas to compute $H_{rep,max}$ of regular and irregular waves.
- A large number of experimental data is used in the verification.
- To modify some existing H_b formulas for computing $H_{rep,max}$.

1.4 Scope of study

- This study uses experimental data of regular and irregular waves from 39 sources to verify the existing formulas on computing the representative wave heights ($H_{rep,max}$).
- The fourteen existing breaker height formulas are considered in this study. The formulas contain the term of deep-water wave height (H_o) are not considered to examine.
- This study considers six types of representative wave heights, i.e. H_{max}, wave height $H_{1/10}$, $H_{1/3}$, H_{mo}, H_{rms}, and H_m.

Chapter 2
Literature Review

2.1 Representative wave heights

A major required parameter to design coastal structures and study beach deformation is wave height. In this research, factor of wave height will be described by using representative wave heights, which are determined against individual wave height by using the zero-crossing method.

Representative wave heights, i.e. H_{max}, $H_{1/10}$, H_{mo}, $H_{1/3}$, H_{rms}, H_{mean} are determined by rearranging experimental data following decreasing value. Then, these types of representative wave heights are determined as follows:

a) Maximum wave height is determined from the highest wave in a record.

b) The mean wave height value of all waves in a record is mentioned as mean wave height.

c) Average of the highest one-third and one-tenth wave height are computed from the highest one-third and one-tenth of recorded wave heights, respectively.

d) Root mean square wave height value of all wave heights in a record is called as root mean square wave height.

e) The significant wave height computed from the zeroth moment wave spectrum (m_o) is called spectral significant wave height.

Because wave heights in shallow water are not available, it can be found out from a wave model. Nevertheless, many existing wave models predict the root-mean-square wave height. Therefore, conversion formulas are used to compute the other types of representative wave heights from H_{rms}.

2.2 Breaking limited approach

Breaking-limited (or depth-limited) approach is used for computing the maximum possible representative wave heights in surf zone. It is used when a structure is designed against $H_{rep,max}$ at a particular depth. This approach is a standard design procedure in Japan and several countries in the world (Goda, 2012). A reliable estimation of $H_{rep,max}$ is a fundamental requirement in the design of coastal structures. In addition, many researchers consider $H_{rep,max}$ as an upper limit of the wave height distribution in the surf zone for the derivation of energy dissipation rate (e.g. Battjes and Janssen, 1978; Thornton and Guza, 1983; Baldock et al., 1998; and Janssen and Battjes, 2007).

It is well recognized that $H_{rep,max}$ in deep-water is generally limited by wind speed, duration, and fetch-length, whereas $H_{rep,max}$ in the surf zone is limited by wave breaking. The breaking limited approach is proposed based on the concept that when a wave height reaches the breaking limit, the wave will break and the wave height will decrease due to energy dissipation. Therefore, the wave height at any location should not be greater than the breaking limit wave height (or maximum possible wave height) of that location. The maximum possible representative wave heights represent a worst-case scenario in terms of wave conditions. It is expected that the breaking-limited approach would generally give conservative results for the design of structures. The concept of breaking-limited design is very useful in the design of structures located in surf zone. The maximum possible representative wave heights are usually determined from a breaker height formula. The incident (or deep-water) wave height at a selected return period is not necessary in this approach. Because of its simplicity, the approach may be used as a first approximation of the design wave heights or used in the area where a long-term measured wave data is not available or less reliable.

Wave height at a selected return period is usually determined from frequency analysis of a long-term measured wave data. If a short-term measured wave data is available, the long-term wave data are often derived from a long-term measured wind data by using wave hindcasts method. With the uncertainties in wave measurements and hindcasting procedures, the uncertainties in a long-term wave climate are substantial (Kamphuis, 2000). In addition, in recent years, extreme events are likely to occur more frequent than before due to global warming. The global warming may cause the changing in the probability distribution of extreme events and the design wave height. A design wave height at a selected return period is expected to increase due to this human-caused climate change. Worse than that, the measured wave data is not available in many coastal areas, especially in developing countries. Because most engineers seem to prefer a simple approach for practical work but give good accuracy, the breaking-limited approach is a simple approach and it is suitable for computing the design wave height when a long-term wave data is not available or less reliable. As the breaking-limited approach is very useful in practical work (especially for the developing countries), it is considered in the present study.

2.3 Existing breaker wave height formulas

The maximum possible representative wave heights are computed from breaker height formulas, which are often proposed against empirical or semi-empirical method. The existing breaker height formulas describe a relationship between the wave height at breaking wave position (H_b) and wave properties, i.e. bottom slope (m), wave period (T), deep-water wavelength (L_o), wavelength at breaking wave position (L_b), water depth at breaking wave position (h_b), and deep-water wave height (H_o). The existing formulas used in this study are summarized below:

(a) McCowan (1894), hereafter referred to as MC94, proposed a formula based on solitary wave theory. The wave height is proposed as a function of water depth at breaking wave position.

$$\frac{H_b}{h} = 0.78 \tag{2.1}$$

(b) Miche (1944), hereafter referred to as MI44, considered the limiting wave steepness or the breaking condition of deep-water wave. The breaking criterion for periodic waves relied on semi-empirical method in arbitrary water depth.

$$\frac{H_b}{L} = 0.14 \tanh\left(\frac{2\pi h}{L}\right) \tag{2.2}$$

(c) Galvin (1969), hereafter referred to as GA69, used the regular wave data from his experiment to develop breaker height formulas. Moreover, he also added the data of Iversen (1952) and McCowan (1894) into his data to increase the number of experimental data to make the formula more reliable. Two formulas are used in different conditions of slope as the following.

$$\frac{H_b}{h} = \frac{1}{1.4 - 6.85m} \quad \text{For } m \le 0.07 \tag{2.3a}$$

$$\frac{H_b}{h} = \frac{1}{0.92} \quad \text{For } m \le 0.07 \tag{2.3b}$$

(d) Collins and Weir (1969), hereafter referred to as CW69, used three experiments (Suquet (1950), Hamada (1951), and Iversen (1952)) performed in two-dimensional wave tanks to developed a breaker height formula. Their formula describes the relationship between breaking wave height and two parameters [i.e. slope (*m*) and water depth (*h*)] at breaking wave position. That formula is shown as the following.

$$\frac{H_b}{h} = (0.72 + 5.6m) \tag{2.4}$$

(e) Goda (1970), hereafter referred to as GO70, utilized the existing experiments of Iversen (1952), Mitsuyasu (1962), and Goda (1964); to develop a diagram of breaking criterion. The diagram can be replaced by the following formula:

$$\frac{H_b}{h} = 0.17\frac{L_o}{h}\left\{1 - \exp\left[-1.5\frac{\pi h}{L_o}\left(1 + 15m^{4/3}\right)\right]\right\} \tag{2.5}$$

In 2010, Goda modified his formula by changing the slope coefficient from 15 to 11, the modified formula as.

$$\frac{H_b}{h} = 0.17\frac{L_o}{h}\left\{1-\exp\left[-1.5\frac{\pi h}{L_o}\left(1+11m^{4/3}\right)\right]\right\} \tag{2.6}$$

(f) Weggel (1972), hereafter referred to as WE72, re-analyzed the previous laboratory data of Iversen (1952), Reid and Bretschneider (1953), Galvin (1969), Jen and Lin (1970), and Weggel and Maxwell (1970) to develop a formula for estimating maximum breaker height based on empirical approach. His formula is shown as the following.

$$\frac{H_b}{h} = \frac{1.56gT^2/\left[1+\exp(-19.5m)\right]}{gT^2+43.75g\left[1+\exp(-19.5m)\right]} \tag{2.7}$$

(g) Madsen (1976), hereafter referred to as MA76, developed a breaker height from the formulas of Galvin and Collins. The formula was proposed as a function of two wave parameters (i.e. bed slope (m) and water depth (h) at breaking position).

$$\frac{H_b}{h} = 0.72\left(1+6.4m\right) \tag{2.8}$$

(h) Battjes and Janssen (1978), hereafter referred to as (BJ78), modified Miche (1944)'s formula by including the term of $\gamma/0.88$ in their formula. The formula was incorporated to his wave model. The coefficient γ was determined by the model calibration. Laboratory experiments with two types of beach conditions were used to calibrate the model. One is a plane beach in the flume. It was designed with 0.88m in height, 9.8m in length and 1:20 seaward slope. The other types including two plane section with 1:40 and 1:20 slope was the bar beach. In the bar beach, the plane section with 1:40 shoreward slope and 4.4m in length connected two plane sections with 1:20 seaward slope. The plan section with 1:20 slope in bar beach had 10m long. Their formula is proposed as the following.

$$\frac{H_b}{L} = 0.14\tanh\left(\frac{0.8}{0.88}\frac{2\pi h}{L}\right) \tag{2.9}$$

(i) Ostendorf and Madsen (1979), hereafter referred to as OM79, suggested a criterion relied on two previous criteria of Miche (1944) and Madsen (1976). Their formula can be expressed as.

$$\frac{H_b}{L} = 0.14\tanh\left[(0.8+5m)\frac{2\pi h}{L}\right] \qquad \text{For } m \le 0.1 \tag{2.10a}$$

$$\frac{H_b}{L} = 0.14\tanh\left[\left(0.8+5(0.1)\right)\frac{2\pi h}{L}\right] \qquad \text{For } m \le 0.1 \tag{2.10b}$$

(j) Seyama and Kimura (1988), hereafter referred to as SK88, used the experimental data of Pierso-Moskowitz including four bottom slopes (i.e. 1:10, 1:20, 1:30, 1:40) to investigate the characteristics of wave shoaling, wave breaking and wave decay. A formula was proposed to compute for irregular wave breaking.

$$\frac{H_b}{h} = \left\{0.16\frac{L_o}{h}\left\{1-\exp\left[-0.8\pi\frac{h}{L_o}\left(1+15m^{4/3}\right)\right]\right\}-0.96m+0.2\right\} \tag{2.11}$$

For computing the breaker height of regular waves, the wave height at breaking position in Eq. (2.11) need to be multiplied with 1.25 as.

$$\frac{H_b}{h} = 1.25\left\{0.16\frac{L_o}{h}\left\{1-\exp\left[-0.8\pi\frac{h}{L_o}\left(1+15m^{4/3}\right)\right]\right\}-0.96m+0.2\right\} \tag{2.12}$$

(l) Kamphuis (1991), hereafter referred to as KA91, added the term of bottom slope into Miche (1994)'s formula to determine wave height at breaking position as in the following.

$$\frac{H_b}{L} = 0.127\exp(4m)\tanh\left(\frac{2\pi h}{L}\right) \tag{2.13}$$

(m) Rattanapitikon and Shibayama (2000), hereafter referred to as RS20b and RS20c, modified the formula of Goda (1970) by changing the terms of bottom slope effect. A large number of experimental data points was used in the calibration and verification. And he proposed two formulas, which are not affected by the term of H_o.

$$\frac{H_b}{h} = 0.17\frac{L_o}{h}\left\{1 - \exp\left[\frac{\pi h}{L_o}(16.21m^2 - 7.07m - 1.55)\right]\right\} \quad (2.14a)$$

$$\frac{H_b}{L} = 0.14\tanh\left[(-11.21m^2 + 5.01m + 0.91)\frac{2\pi h}{L}\right] \quad (2.14b)$$

Chapter 3
Methodology

3.1 Basic wave theory

In the 1845, George Biddell Airy developed a new theory of wave. His method is known as Airy wave theory or linear wave theory. It is usually used to describe the wave motions. Because of its simplicity, the theory is still utilized in the practice work also depicted ocean wave.

The Airy wave theory is used to describe wave propagation in x-z plane. Basic wave parameters are mentioned in Figure 3.1, and described as follows:

The wave speed or celerity: $$c = \frac{L}{T} \tag{3.1}$$

The angular frequency: $$\sigma = \frac{2\pi}{L} \tag{3.2}$$

Wave number: $$k = \frac{2\pi}{L} \tag{3.3}$$

From the above equations, we can find out the relationship between the angular frequency, the wave speed or celerity (*c*) and wave number (*k*) as:

$$\sigma = ck \tag{3.4}$$

where *H* is wave height, *T* is the wave period, *h* is water depth, and *L* is the wavelength.

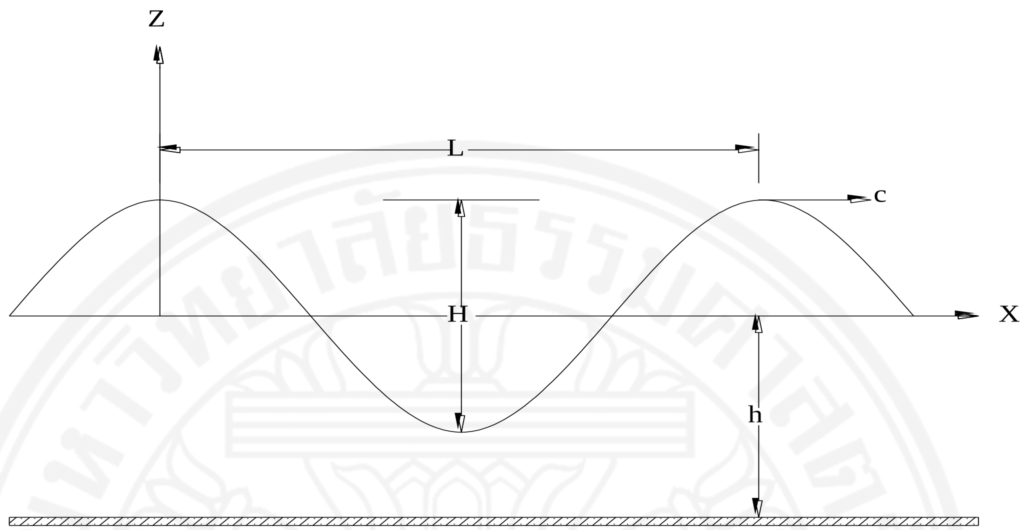


Figure 3.1 Definition sketch of wave parameters

The linear wave theory is derived based on three main assumptions that water surface elevation with small value, considering 2-D (x-z plane) and constant depth. Governing equation and boundary conditions are described as follows (see Figure 3.2 for definition sketch of boundary conditions).

Governing Equation: $$\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial z^2}=0 \quad \text{for } 0<x<L,\ -h<z<\eta \tag{3.5}$$

Bottom Boundary Equation: $$w=-\frac{\partial\phi}{\partial z}=0 \quad \text{on } z=-h \tag{3.6}$$

Kinematic surface boundary condition: $$w=-\frac{\partial\phi}{\partial z}=\frac{\partial\eta}{\partial t} \quad \text{on } z=0 \tag{3.7}$$

Dynamic surface boundary condition: $$-\frac{\partial\phi}{\partial t}+g\eta=C(t) \quad \text{on } z=0 \tag{3.8}$$

Lateral boundary conditions:
$$\phi(x,t)=\phi(x+L,t) \tag{3.9}$$
$$\phi(x,t)=\phi(x,t+T) \tag{3.10}$$

where ϕ is the velocity potential, w is the velocity in z direction, g is the gravity acceleration, η is water surface elevation, and C is constant.

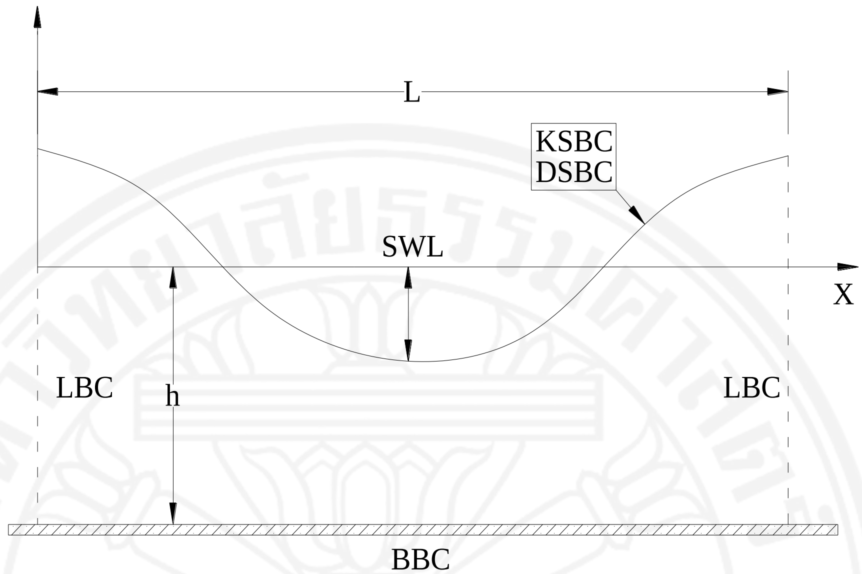


Figure 3.2 Definition sketch of boundary conditions

To find out the velocity potential, the separation of variables is used to resolve this problem. The equation is described as:

$$\phi = -\frac{Hg \cosh\ k(h+z)}{2\sigma \cosh\ kh} \sin(kx - \sigma t) \tag{3.11}$$

The sinusoidal wave profile is found by substituting the Eq. (3.11) into the Eq. (3.8), the result is:

$$\eta = \frac{H}{2} \cos(kx - \sigma t) \tag{3.12}$$

Substituting Equations 3.11 and 3.12 into the kinematic surface boundary Equation 3.7, yields:

$$\sigma^2 = gk \tanh(kh) \tag{3.13}$$

This equation is usually used to compute wave number from water depth (h) and wave period (T) is called dispersion relation or dispersion equation. However, this equation is solved by using the iterative method.

Because of the complexity for computing wave number by iterative method, Hunt (1979) proposed a formula for computing wave number directly as:

$$k=\frac{1}{h}\sqrt{y^2+\frac{y}{1+\sum_{n=1}^{6}d_n y^n}} \tag{3.14}$$

where: $y=\frac{\sigma^2 h}{g}$,d_1=0.6666666666, d_2=0.3555555555, d_3=0.1608465608, d_4=0.0632098765, d_5=0.0217540484, and d_6=0.0065407983.

Beach profiles can be separated into three regions based on the relative depth criterion *kh* or *h/L*. The classification is shown in Table 3.1:

Table 3.1 Classification of beach profiles

Classifications	Limits		
	kh	*h/L*	tanh(*kh*)
Shallow water	$0<kh<\frac{\pi}{10}$	$0<\frac{h}{L}<\frac{1}{20}$	$\approx kh$
Intermediate depth (Transitional)	$\frac{\pi}{10}<kh<\pi$	$\frac{1}{20}<\frac{h}{L}<\frac{1}{2}$	tanh(*kh*)
Deep water	$\pi<kh<\infty$	$\frac{1}{2}<\frac{h}{L}<\infty$	≈ 1

Using the conditions in Table 3.1, wave celerity and wavelength can be simplified as:

❖ In Shallow water ($0<kh<\frac{\pi}{10}$), $\tanh(kh)\approx kh$, the dispersion equation becomes:

$$\sigma^2=gk^2h \tag{3.15}$$

The wave celerity and wavelength can be written as:

$$c = \sqrt{gh} \tag{3.16}$$

$$L = \sqrt{gh}\,T \tag{3.17}$$

❖ In Deep water ($\pi < kh < \infty$), $\tanh(kh) \approx 1$, The dispersion equation becomes:

$$\sigma^2 = gk \tag{3.18}$$

The wave celerity and wavelength as below:

$$c_o = \frac{gT}{2\pi} \tag{3.19}$$

$$L_o = \frac{gT^2}{2\pi} \tag{3.20}$$

3.2 Existing breaker height formulas.

The existing formulas for computing maximum possible representative wave heights are shown as Table 3.2.

Table 3.2 Existing breaker height formulas

Researchers	Abbrev-iation	Formulas
McCowan (1894)	MC94	$\frac{H_b}{h} = 0.78$
Miche (1944)	MI44	$\frac{H_b}{L} = 0.14\tanh\left(\frac{2\pi h}{L}\right)$
Galvin (1969)	GA69	$\frac{H_b}{h} = \frac{1}{1.4 - 6.85m}$ For $m \le 0.07$ $\frac{H_b}{h} = \frac{1}{0.92}$ For $m \le 0.07$
Collins and Weir (1969)	CW 69	$\frac{H_b}{h} = (0.72 + 5.6m)$
Goda (1970)	GO70	$\frac{H_b}{h} 0.17\frac{L_o}{h}\left\{1 - \exp\left[-1.5\frac{\pi h}{L_o}\left(1 + 15m^{4/3}\right)\right]\right\}$

Table 3.2(cont.) Existing breaker height formulas

Researchers	Abbrev-iation	Formulas
Weggel (1972)	WE72	$\frac{H_b}{h} = \frac{1.56gT^2 / \left[1+\exp(-19.5m)\right]}{gT^2 + 43.75g\left[1+\exp(-19.5m)\right]}$
Madsen (1976)	MA76	$\frac{H_b}{h} = 0.72\left(1+6.4m\right)$
Battjes and Janssen (1978)	BJ78	$\frac{H_b}{L} = 0.14\tanh\left(\frac{0.8}{0.88}\frac{2\pi h}{L}\right)$
Ostendorf and Madsen (1979)	OM79	$\frac{H_b}{L} = 0.14\tanh\left[(0.8+5m)\frac{2\pi h}{L}\right]$ For $m \le 0.1$ $\frac{H_b}{L} = 0.14\tanh\left[\left(0.8+5(0.1)\right)\frac{2\pi h}{L}\right]$ For $m \le 0.1$
Seyama and Kimura (1988)	SK88	$\frac{H_b}{h} = 1.25\left\{0.16\frac{L_o}{h}\left\{1-\exp\left[-0.8\pi\frac{h}{L_o}\left(1+15m^{4/3}\right)\right]\right\}-0.96m+0.2\right\}$
Kamphuis (1991)	KA91	$\frac{H_b}{L} = 0.127\exp(4m)\tanh\left(\frac{2\pi h}{L}\right)$
Rattanapitikon and Shibayama (2000)	RS00b	$\frac{H_b}{h} = 0.17\frac{L_o}{h}\left\{1-\exp\left[\frac{\pi h}{L_o}(16.21m^2 - 7.07m - 1.55\right]\right\}$
Rattanapitikon and Shibayama (2000)	RS00c	$\frac{H_b}{L} = 0.14\tanh\left[(-11.21m^2 + 5.01m + 0.91)\frac{2\pi h}{L}\right]$
Goda (2010)	GO10	$\frac{H_b}{h} = 0.17\frac{L_o}{h}\left\{1-\exp\left[-1.5\frac{\pi h}{L_o}\left(1+11m^{4/3}\right)\right]\right\}$

3.3 Examination of formula accuracy

The method, which is used to evaluate accuracy of formulas, is average of the mean percent error. The formula is described as:

$$MPE_{avg} = \frac{100}{N}\sum_{i=1}^{N}\left(\frac{H_{mi} - H_{ci}}{H_{mi}}\right) \tag{3.21}$$

where i is the wave height number, H_{mi} is the measured wave height number *i*, H_{ci} is the computed H_{mp} number *i*, H_{ci} is the computed H_{mp} number *i*, and *N* is the total number of data.

The condition for using this formula in the study is measured wave height larger than computed wave height ($H_{mi} \geq H_{ci}$). If $H_{mi} \leq H_{ci}$, the errors of those points will be considered to be zero.

3.4 Collected data

Description of wave mechanism by using available wave theories is difficult because of the complication of wave breaking mechanism. Therefore, to make it easier, this study bases on empirical formula. The problem is how to make the empirical formula trustworthy. To solve this problem, a large amount and wide range of experimental data are used to make sure that the empirical formula will be become reliable. Because the formulas for computing maximum possible representative wave heights are determined against regular and irregular wave heights, the experiments conducted in regular and irregular wave conditions are used to examine formulas.

3.4.1 Regular wave heights

The experiments, which are used in this study to exam and modify the existing formulas, are briefed as follows:

- The experiment of Cox and Cobayashi (1995) was used for research of Cox and Cobayashi in 1997's. It was applied to verify a new kinematic undertow profile model with logarithmic boundary layer. They measured at six positions on a plane beach sloping 0.029 in this experiment. Position of wave generator was located at x = 980 cm away from the first measurement position (x=0 cm).

The other measured positions were located at 240, 360, 480, 600, 720 cm. Wave period of experiment is 2.2 s.

- The experiment of De serio and Mossa (2006) was conducted to investigate the spreading of turbulence in the breaking region and the velocity and Reynolds shear stress distributions in the shoaling zone. A flume of 45 m in length, 1 m in width, and 0.47 m in depth was used for this laboratory experiment, which was just employed regular wave heights. The first test performed spilling and plunging breaker, the second test conducted spilling breaker and the last test performed plunging breaker. Wave periods of those tests are $T_1 = 2$ s, $T_2 = 1$ s, and $T_3 = 4$ s, respectively.

- The experiment of Hansen and Svendsen (1984) was conducted in a wave channel of 32 m in length, 60 cm in width. The total length of this channel was separated into 2 sections, including 12.33 m long (section 1) with a plane beach sloping 1:34.25 and 14.78 m long (section 2) with horizontal slope and 36 cm in depth. The experiment was performed to study the undertow.

- The experiment Horikawa and Kuo (1966) was performed to study wave transformation in surf zone. The wave flume used for this experiment is described in the Figure 3.3 bellow.

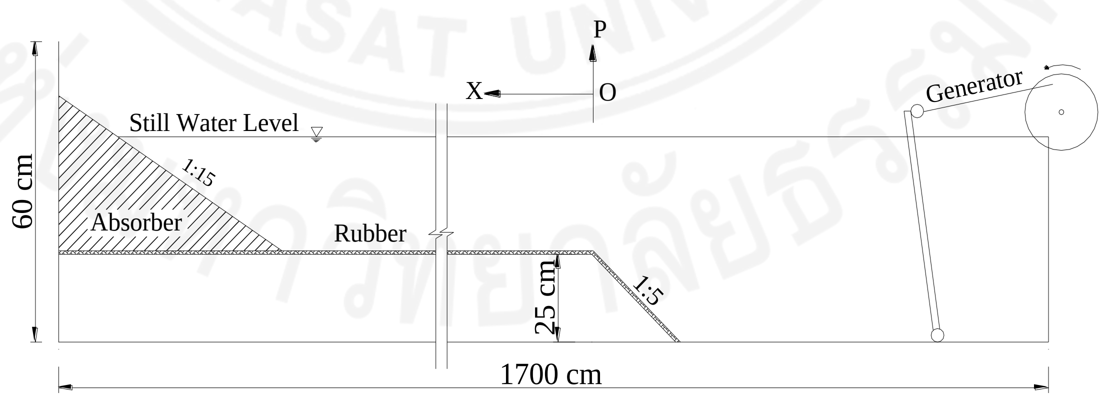


Figure 3.3 Laboratory installation of Horikawa and Kuo (1966)

▪ Hurue (1990) designed a wave channel using for laboratory experiment. This experiment was conducted on a plane beach to understand wave characteristics and undertow velocity. The size of channel was 17 m in length, 0.5 m in width and 1:20 in plane bottom slope. Hurue created wave against wave properties as 0.09m wave height and 1.26 s wave period. Elevations of water surface were recorded by using capacitance-type gages at seven cross-shore positions.

▪ Nadaoka et al (1982) performed an experiment to research the internal velocity field in the surf zone because of its importance in wave deformation, design of coastal structures and others. This experiment is illustrated in the Figure 3.4.

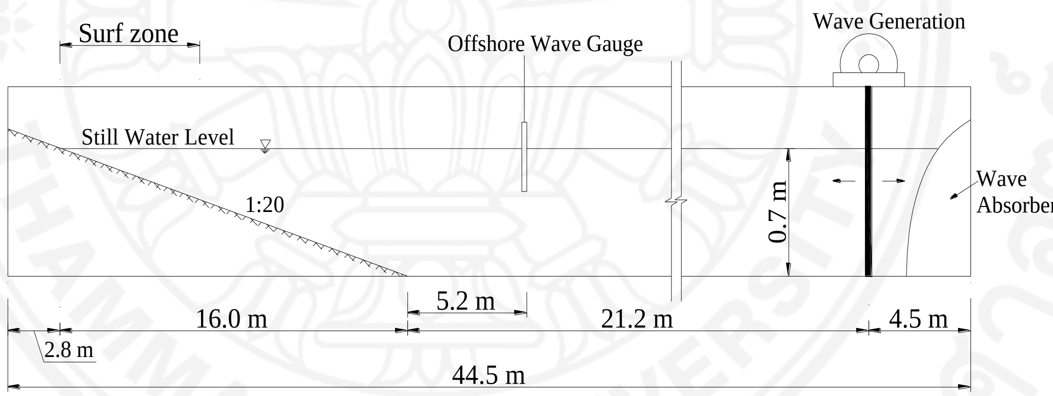


Figure 3.4 Laboratory installation of Nadaoka (1982)

▪ Okayasu (1988) conducted an experiment to predict vertical profile of undertow in the surf zone. Ten experiments were performed on 1:20 and 1:30 constant slopes of smooth beds for various incident waves. The flume was 23 m long and 0.8 m wide.

▪ The experiment of Sato (1989) was conducted in small-scale wave flume performed to consider characteristics of long-wave component in near-bottom velocities under arbitrary waves. The region measured had 6.3 m long and 1:40 slope. The region of uniform depth was set a slope of 1:10 and length of 2 m.

The still water depth over the horizontal bed was 36 cm throughout the experiments

- Smith and Kraus (1990) conducted an experiment to study the macro-features of wave breaking over bar and artificial reefs. A wave tank including 2 sections was designed with 45.7 m in length, 0.46 m in width and 0.9 m in depth. Section 1 was 21 m long with zero sloping and the other was 24.7 m long with 1:30 sloping. Bars, which were located at the calculated crest depth, were designed following the angle of seaward and shoreward bar's face (angle created by bar's face and horizontal) ranging from 5-40 and 0-40 degree, respectively. Both of regular and irregular wave data with 113 cases were recorded in this study. Resistance-type gages were deployed to record elevations of water surface at eight positions.

- Smith and Seabergh (2001) carried out an experiment with an idealized inlet, which was 99 m in length, 46 m in width and 0.6 m in wall's height. The aim of this experiment is to provide the data to develop a dissipation function for wave breaking.

- The experiment of Stive and Wind (1986) were performed in a wave channel with 55 m in length, 1 m in width and 1 m in height and bed slope of 1:40 at breaking position designed by concrete. Periodic waves with minimal free second-harmonic components were generated in a water depth of 0.85 m and conductivity-type wave gauges were used to measure surface elevation in two-dimensional surf zone. This experiment was conducted to study the cross-shore mean slow to suggest a model for the undertow.

- Ting (2011) designed a titling flume with a length of 25 m, width of 0.9 m, depth of 0.75 m and slope of 3%. In constant depth area, still water depth was measured to be 0.36 m. The properties of Cnoidal wave with a wave height and wave period of 0.122 m and 2 s, respectively; were generated by wave generator. A number of wave measured positions mounted in total length of this experiment were 22. A maximum wave height corresponding with still water depth of 0.206 m and horizontal distance of 6.873 m and 5.694 m from maximum wave height position to still water depth shoreline and to wave generator, respectively; was 0.17 m. The main objective of his performance is to investigate the characteristics of eddied affecting on the bed, the influence of the turbulent flow structures generated by the breaking of spilling regular waves on the mean flow and vice versa.

- Ting (2013) designed a titling flume with a length of 25 m, width of 0.9 m, depth of 0.75 m and slope of 3%. In constant depth area, still water depth was measured to be 0.36 m. The properties of Cnoidal wave with a wave height and wave period of 0.17 m and 2 s, respectively; were generated by wave generator. A number of wave measured positions mounted in total length of this experiment were 22. A maximum wave height corresponding with still water depth of 0.219 m and horizontal distance of 7.29 m and 5.27 m from maximum wave height position to still water depth shoreline and to wave generator, respectively; was 0.217 m. The main objective of his performance is to investigate the structure of eddies created by the breaking of spilling regular waves.

- Kraus and Smith (1994) based on an experiments referred SUPERTANK project to study sediment transport and cross-shore hydrodynamic. A wave tank with 104 m in length, 3.7 m in width, and 4.6 m in depth was designed. The sandy beach with 76 m in length was simulated in this tank. The collected data including 992 wave records were separated into 62 cases of regular waves.

Table 3.3 Summary of collected experimental data of regular wave heights for H_{rep}

No.	Sources	No.of case	No.of data	Bed condition	Apparatus
1	Cox and Kobayashi (1997)	1	6	PB	SS
2	De serio and Mossa (2006)	3	25	PB	SS
3	Hansen and Svendsen (1979)	17	491	PB	SS
4	Hansen and Svendsen (1984)	1	5	PB	SS
5	Horikawa and Kuo (1966)	213	2127	SB	SS
6	Hurue (1990)	1	6	PB	SS
7	Nadaoka et al. (1982)	2	14	PB	SS
8	Nagayama (1983)	12	263	PB, BB and SB	SS
9	Okayasu et al. (1988)	10	61	PB	SS
10	Sato et al. (1988)	3	25	PB	SS
11	Sato et al. (1989)	2	11	PB	SS
12	Smith and Kraus (1990)	101	808	PB and BB	SS
13	Smith and Seabergh (2001)	14	168	SB	SS
14	Stive and Wind (1986)	1	6	PB	SS
15	Ting (2011)	1	22	PB	SS
16	Ting (2013)	1	20	PB	SS
17	Kajima et al. (1983)	102	3694	MB	LS
18	Kraus and Smith (1994)	62	992	MB	LS
	Total	547	8744		

Remarks : PB = Plane beach; BB = Barred beach; SB = Stepped beach; MB = Movable beach; SS = Small scale:; LS = Large scale

3.4.2 Irregular wave heights

The experiments, which are used in this study to exam and modify the existing formulas, are briefed as follows:

- Ting (2001 and 2002) carried out an experiment by designing a small-scale wave flume with the length of 37 m long, width of 0.91 m, depth 1.22 and a false bed slope of 1:35 to study characteristics of irregular waves in surf zone. According to TMA spectrum (Bouws 1985), the irregular waves were created following spectral wave properties as wave period of 2 s, wave height of 15.24 cm. Water surface elevations were measured by using resistance-type gages at six positions following Figure 3.5. The detailing parameters were described as schematic diagram Figure 3.5.

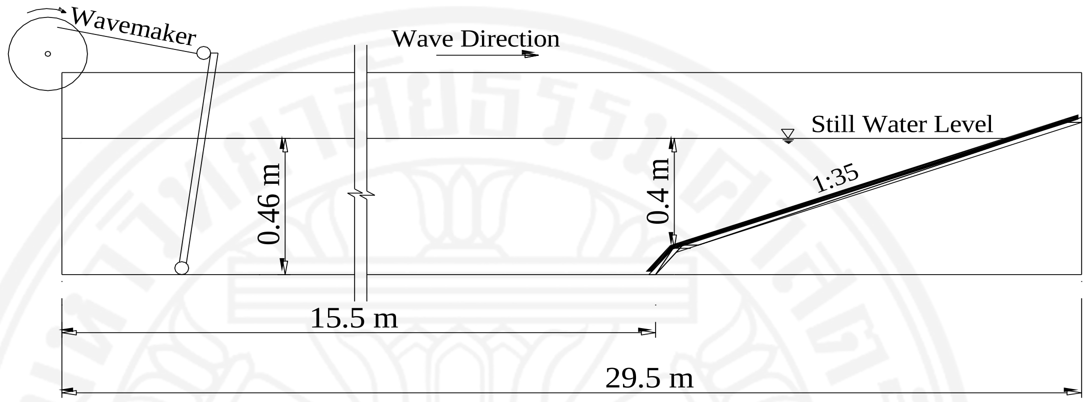


Figure 3.5 Laboratory installation of Ting (2002)

- Kraus and Smith (1994) conducted an experiments referred SUPERTANK project to study sediment transport and cross-shore hydrodynamic. A wave tank with 104 m in length, 3.7 m in width, and 4.6 m in depth was designed. The sandy beach with 76 m in length was simulated in this tank. The collected data including 2083 wave records were separated into 128 cases of irregular waves.

- SAFE Project (Dette et al., 1998) was conducted to study beach nourishment. A large wave tank was constructed with 300 m in length, 5 m in width, and 7 m in depth. The sandy beach with 76 m in length was simulated in this tank. A large number of wave data was recorded in 138 cases of irregular waves

- The experiment of Smith et al. (2012) was conducted in a wave basin with length of 51.8 m, width 29.0 m, and depth 1.5m. The basin contained a 14.8 m long and 22.0 m wide compound-slope steel platform on which reef bathymetry was installed. The platform was configured into a compound slope including reef slopes of 1:4 (0.29 m long) and 1:13 (2.43 m long), a 7.3 m flat section, and a 1:10 foreshore slope (4.8 m long).

- Long (1991) carried out an experiment with 11 test cases in Duck, NC to evaluate the influence of two models as Rayleigh and Beta-Rayleigh probability functions on experiment data of representative wave heights. To conduct this experiment, two instruments were used. A linear wave gage with nine bottom mounted pressure sensors was located in isobaths positions on a straight line near wave rider instrument to investigate the frequency direction spectral of ocean surface wave. The other is wave rider buoy located at water depth of 8 m to obtain the wave height information and a measure of sea surface displacement.

- COAST3D Project was performed to collect field data at Teigmond for evaluating numerical models and to gain a better understanding of morphological processes.

- LIP11D Project (1995) was carried out to study about hydrodynamics and sediment transport dynamics on a natural 2DV beach under equilibrium, erosive and accretive conditions. The length and height of the DELTA-flume were 203 m and 7 m, respectively. Depending on maximum wave height, run-up and dune height, the actual water depth was between 4 and 5 m. In view of the wave generation, it was advisable to have at least the first 20 m from the wave paddle without sand. The bottom heights were larger than 3.7 m with slope of 1:30 and less than 1.6 m with slope of 1:20, respectively.

- Smith and Vincent (1992) performed an experiment in a small wave flume of 45.7 m long, 0.46 m wide, and 0.9 m deep to study shoaling and decay of multiple wave trains. The wave flume was separated into two sections. The first section was 21 m long with horizontal flume bottom and the other was 24.7 m long with bed slope of 1:30. The number of wave gages was 9 and the water depth in horizontal bottom was 6 cm. According to TMA spectrum

(Bouws 1985), the irregular waves were created following spectral wave properties following wave period of 2.5 s and 1.25 s for wave height of 15.2 cm and wave period of 2.5 s and 1.75 s for wave height of 9.2 cm.

- Smith and Seabergh (2001) conducted an experiment to improve dissipation function relied on experimental data. An experiment was conducted in an idealized inlet with a 46 m in width, 99 m in length and 0.6 m in height. Wave parameters including wave height, wave period and velocity were collected from electrical capacitance wave gauge and velocity meters. In addition, wave conditions were used in this study following zero moment wave parameters as wave height of 3.7 and 5.5 cm, wave period of 0.7 and 1.4 s, and incident wave direction perpendicular to the jetties. One more parameter was the current velocities of 0, 12, 24, and 32 cm/s, which were determined from instruments mentioned above.

- DELILAH Project (Birkemeier et al., 1997) was performed to understand wave characteristics and changing bathymetric. Four interdependent arrays including two cross shore subarrays and two long shore subarrays were deployed on the area of 550 m long shore by 375 m cross shore. The first cross shore array had nine pressure wave gauges and nine current meters. Trough and crest array, which were long shore arrays, had one pressure wave gauge for each. Five and four current meters were used in trough and crest array, respectively. The second cross shore had three current meters. This study considered 745 cases covering a range of wave period from 3.4 to 13.5 s and significant wave height from 0.4 to 0.7 m.

- DUCK94 Project (Herbers et al., 2006) was performed to enhance the understanding about morphologic evolution and sediment transport. This project was carried out from 8/8/1994 to 24/10/1994 and was employed the instruments as current meter, pressure gauge, acoustic altimeter, thermometer, suspended sediment concentration gauge, scanning sonars, void fraction sensors and directional wave buoys. In addition, remote sensing systems were used to measure dynamic properties and tower-mounted video systems were used to observe in surf zone and swash zone. This study considered 587 cases covering a range of wave period from 4.4 to 11.4 s and significant wave height from 0.2 to 2.6 m.

- Hamilton and Ebersole (1992) performed an experiment in a basin of 30 m cross shore, 50 m long shore, and 1.4 m deep to study sediment transport processes. A beach with concrete bottom was 21 m cross shore, 31 m long shore and 1:30 bottom slope. Cross shore remainder of the basin with water depth of 66.7 cm and bottom slope of 0 was used to locate wave generators. According to TMA spectrum (Bouws 1985), the irregular waves were created following spectral wave properties following wave period of 2.5 s with wave height of 18.9 cm and the regular waves were created following spectral wave properties following wave period of 2.5 s and with wave height of 23.3 cm.

- DUCK85 Project (Herbers et al., 2006) was a field experiment, which was conducted from 1/6/1985 to 30/9/1985 to measure the longshore sand transport rate in surf zone. 14 positions in the surf zone were mounted cameras at target poles to film water surface elevations to measured wave height. In addition, the bottom profile and still water level were measured by mean of a survey station located the main building of FRF and tide gage located 6-min intervals, respectively.

- Hotta (1982) performed a field scale experiment at Ajigaura beach in Japan to study wave characteristics in near shore zone. A camera system was deployed to observe activities of Ajigara beach by utilizing a pier at this beach as a platform. The beach slope and average tidal were about 1:60 to 1:70 and 1.2 m, respectively. The observation time for three observations was conducted from 2/1980 to 9/1981. Cameras were employed in first, second and third observations and the number of cameras was six pairs, seven pairs and five pairs of 16 mm cameras, respectively. In addition, some other equipments (e.g. sled, pole) were also used to serve for those observations. The observation range was about 400 m from offshore to shoreline and the breaking wave heights in this area were from 1.8 to 2.2 m.

Table 3.4 Summary of collected experimental data of irregular wave heights for H_{rep}

No.	Sources	No. of Cases	No. of data						Beach conditions	Apparatus
			H_{max}	H_{mo}	H_{mean}	$H_{1/3}$	$H_{1/10}$	H_{rms}		
1	Smith and Kraus (1990)	12	96	-	96	96	-	96	PB and BB	SS
2	Hurue (1990)	1	-	-	-	7	-	-	PB	SS
3	Katayama (1991)	2	-	-	-	16	-	-	PB	SS
4	Smith and Vincent (1992)	4	-	36	-	-	-	-	PB	SS
5	Hamilton and Ebersole (2001)	1	-	10	-	-	-	-	PB	SS
6	Smith and Seabergh (2001)	15	-	180	180	180	-	-	SB	SS
7	Ting (2001)	1	7	-	7	7	7	7	PB	SS
8	Ting (2002)	1	7	-	7	7	7	7	PB	SS
9	Smith et al. (2012): SWIMS Project	256	2253	-	-	-	-	-	PB	SS
10	Kraus and Smith (1994): SUPERTANK Project	128	2038	2046	2046	2046	2046	2046	MB	LS
11	Roelvink and Reniers (1995): LIP 11D Project	95	170	923	-	170	170	170	MB	LS
12	Dette et al. (1998): SAFE Project	138	3556	3556	-	3556	3556	3556	MB	LS

No.	Sources	No. of Cases	No. of data						Beach conditions	Apparatus
			H_{max}	H_{mo}	H_{mean}	$H_{1/3}$	$H_{1/10}$	H_{rms}		
13	Goodknight and Russell (1963)	4	80	-	80	80	80	80	MB	FS
14	Goda (1974)	4	15	15	15	15	-	-	MB	FS
15	Hotta et al. (1982)	3	18	-	-	18	18	18	MB	FS
16	Kraus et al. (1989): DUCK 85 Project	8	90	90	90	90	90	90	MB	FS
17	Thornton and Guza (1986)	4	-	-	-	-	-	60		
18	Long (1991)	11	11	-	11	11	11	11	MB	FS
19	Birkemeier et. al (1997) :DELILAH Project	745	-	5043	-	-	-	-	MB	FS
20	Whitehouse and Sutherland (2001): COAST3D Project at Teigmond	2	796	796	796	796	796	796	MB	FS
21	Herbers et. al (2006):DUCK94 Project	587	-	6097	-	-	-	-	MB	FS
	Total	2022	9137	18792	3328	7095	6781	6937		

Remarks : PB = Plane beach, BB = Barred beach, SB = Stepped beach, FS = Field scale, MB = Movable beach, SS = Small scale, LS = Large scale

Chapter 4

Estimation of The Maximum Possible Wave Height

4.1 Introduction

Highest wave height is one of the most significant required factors for the design of coastal structures. A coastal structure is often designed against H_{max} of an extreme wave condition that is likely to occur during a lifetime of the structure or against the maximum possible wave height (H_{mp} *or* $H_{max,max}$) at a particular depth. An accurate design wave height is important for a sustainable design of coastal structures. While underestimation of H_{max} makes a structure less safe, overestimation causes wastefulness of structure. The maximum possible wave (or breaking-limited approach) is used in this study because of the simplification and accuracy of the approach.

The breaking-limited approach is a traditional method to determine the upper limit (maximum possible) of wave height at a particular location in surf zone for design coastal structure. It was proposed against the concept that when a wave height reaches the breaking limit, the wave will break and the wave height will decrease due to energy dissipation. Therefore, the wave height at any location should not be greater than the breaking limit (or maximum possible wave height) of that location. The present study concentrates on the empirical formulas for estimating maximum possible wave height.

4.2 Data collection

As the existing breaker height formulas were proposed based on the empirical approach or semi-empirical approach, the applicability of those existing formulas will be affected by the range experimental conditions. Most of the existing formulas were developed based on a limited range of experimental conditions. Therefore, the formulas may not be applied for general conditions. In this study, a large amount and wide range of experimental data were used to verify and modify the existing formulas for computing H_{rep}.

4.2.1 Regular wave heights

Data of regular wave heights from 18 sources (totaling 547cases with 8744 data points) have been collected to examine the applicability of the formulas. A summary of the collected laboratory data of regular waves is given in Table 4.1. The measured wave heights at all available locations (except in the swash zone) are used in the examination. The experiments cover a wide range of wave and bottom topography conditions, including small-scale and large-scale experiments. Most of the experiments were carried out in small-scale wave flumes under fixed bed conditions (plane, stepped, and barred beaches), except the experiments of Kajima et al. (1983) and Kraus and Smith (1994), which were carried out in large-scale wave flumes under sandy bed conditions. The data cover the relative depth (kh) ranging between 0.06 and 4.55, and the bottom slope (m) ranging between 0 and 1.12.

Table 4.1 Summary of collected experimental data of regular wave heights for H_{max}

No.	Sources	No.of case	No.of data	Bed condition	Apparatus
1	Cox and Kobayashi (1997)	1	6	PB	SS
2	De serio and Mossa (2006)	3	25	PB	SS
3	Hansen and Svendsen (1979)	17	491	PB	SS
4	Hansen and Svendsen (1984)	1	5	PB	SS
5	Horikawa and Kuo (1966)	213	2127	SB	SS
6	Hurue (1990)	1	6	PB	SS
7	Nadaoka et al. (1982)	2	14	PB	SS
8	Nagayama (1983)	12	263	PB, BB and SB	SS
9	Okayasu et al. (1988)	10	61	PB	SS
10	Sato et al. (1988)	3	25	PB	SS
11	Sato et al. (1989)	2	11	PB	SS
12	Smith and Kraus (1990)	101	808	PB and BB	SS
13	Smith and Seabergh (2001)	14	168	SB	SS
14	Stive and Wind (1986)	1	6	PB	SS
15	Ting (2011)	1	22	PB	SS
16	Ting (2013)	1	20	PB	SS
17	Kajima et al. (1983)	102	3694	MB	LS
18	Kraus and Smith (1994)	62	992	MB	LS
	Total	547	8744		

Remarks : PB = Plane beach BB = Barred beach SB = Stepped beach
MB = Movable beach SS = Small scale: LS = Large scale

4.2.2 Irregular wave heights

Data of maximum wave height (H_{max}) of irregular waves from 13 sources (totaling 663 cases with 9137 data points) have been collected for examination of the formulas. A summary of the collected experimental data of irregular waves is shown in Table 4.2. The measured data of H_{max} in offshore and surf zones (excluding those in swash zone) are used in the examination. The experiments cover a wide range of wave and bottom topography conditions, including small-scale, large-scale, and field experiments. The small-scale experiments were performed under fixed bed conditions (plane and barred beaches), while the large-scale and field experiments were performed under sandy bed conditions. The data cover the relative depth (kh) ranging between 0.06 and 4.61, and the bottom slope (m) ranging between 0.00 and 0.40.

Collected data are divided up three groups based on the experiment scale, i.e. small-scale, large-scale and field-scale experiments. The experiments of Smith and Kraus (1990), Ting (2001), Ting (2002), and Smith et al. (2012) were performed in small-scale wave flumes under fixed bed conditions, the second group is the experiments of Kraus and Smith (1994), Roelvink and Reniers (1995) and Dette et al. (1998) were undertaken in large-scale wave flumes under movable bed (sandy bed) conditions and the last group that is the experiments of Goodknight and Russell (1963), Goda (1974), Kraus et al. (1989), Long (1991) and Whitehouse and Sutherland (2001) were undertaken in field-scale wave flumes under movable bed (sandy bed) conditions.

Table 4.2 Summary of collected experimental data of irregular wave heights H_{max}

No.	Sources	No.of case	No.of data	Bed condition	Apparatus
1	Smith and Kraus (1990)	12	96	PB and BB	SS
2	Ting (2001)	1	7	PB	SS
3	Ting (2002)	1	7	PB	SS
4	Smith et al. (2012): SWIMS Project	256	2253	PB	SS
5	Kraus and Smith (1994): SUPERTANK Project	128	2038	MB	LS
6	Roelvink and Reniers (1995): LIP 11D Project	95	170	MB	LS
7	Dette et al. (1998): SAFE Project	138	3556	MB	LS
8	Goodknight and Russell (1963)	4	80	MB	FS
9	Goda (1974)	4	15	MB	FS
10	Hotta et al. (1982)	3	18	MB	FS
11	Kraus et al. (1989): DUCK 85 Project	8	90	MB	FS
12	Long (1991)	11	11	MB	FS
13	Whitehouse and Sutherland (2001): COAST3D Project at Teigmond	2	796	MB	FS
	Total	663	9137		

Remarks : PB = Plane beach; BB = Barred beach; SB = Stepped beach; MB = Movable beach; SS = Small scale:; LS = Large scale; FS = Field scale

4.3 Existing formulas

Maximum possible wave height ($H_{max,max}$ or H_{mp}) is computed from a formula, which computes the maximum wave height at the incipient breaking points against assumption that $H_b = H_{max,max} = H_{mp}$. Most of the existing breaker height formulas were developed based on empirical or semi-empirical approache. In this study, fourteen H_b formulas shown in Table 4.3 were applied to compute H_{rep}.

Table 4.3 Existing breaker height formulas for computing H_{mp} *or* $H_{max,max}$

Researchers	Abbrev-iation	Formulas
McCowan (1894)	MC94	$\frac{H_{mp}}{h} = 0.78$ (4.1)

Table 4.3(cont.) Existing breaker height formulas for computing H_{mp} *or* $H_{max,max}$

Researchers	Abbrev -iation	Formulas
Miche (1944)	MI44	$\frac{H_{mp}}{L} = 0.14\tanh\left(\frac{2\pi h}{L}\right)$ (4.2)
Galvin (1969)	GA69	$\frac{H_{mp}}{h} = \frac{1}{1.4-6.85m}$ For $m \le 0.07$ (4.3a) $\frac{H_{mp}}{h} = \frac{1}{0.92}$ For $m \le 0.07$ (4.3b)
Collins and Weir (1969)	CW 69	$\frac{H_{mp}}{h} = (0.72+5.6m)$ (4.4)
Goda (1970)	GO70	$\frac{H_{mp}}{h} = 0.17\frac{L_o}{h}\left\{1-\exp\left[-1.5\frac{\pi h}{L_o}\left(1+15m^{4/3}\right)\right]\right\}$ (4.5)
Weggel (1972)	WE72	$\frac{H_{mp}}{h} = \frac{1.56gT^2/\left[1+\exp(-19.5m)\right]}{gT^2+43.75g\left[1+\exp(-19.5m)\right]}$ (4.6)
Madsen (1976)	MA76	$\frac{H_{mp}}{h} = 0.72\left(1+6.4m\right)$ (4.7)
Battjes and Janssen (1978)	BJ78	$\frac{H_{mp}}{L} = 0.14\tanh\left(\frac{0.8}{0.88}\frac{2\pi h}{L}\right)$ (4.8)
Ostendorf and Madsen (1979)	OM79	$\frac{H_{mp}}{L} 0.14\tanh\left[(0.8+5m)\frac{2\pi h}{L}\right]$ For $m \le 0.1$ (4.9a) $\frac{H_{mp}}{L} 0.14\tanh\left[\left(0.8+5(0.1)\right)\frac{2\pi h}{L}\right]$ For $m \le 0.1$ (4.9b)
Seyama and Kimura (1988)	SK88	$\frac{H_{mp}}{h} = 1.25\left\{0.16\frac{L_o}{h}\left\{1-\exp\left[-0.8\pi\frac{h}{L_o}\left(1+15m^{4/3}\right)\right]\right\}-0.96m+0.2\right\}$ (4.10)
Kamphuis (1991)	KA91	$\frac{H_{mp}}{L} = 0.127\exp(4m)\tanh\left(\frac{2\pi h}{L}\right)$ (4.11)
Rattanapitikon and Shibayama (2000)	RS00b	$\frac{H_{mp}}{h} = 0.17\frac{L_o}{h}\left\{1-\exp\left[\frac{\pi h}{L_o}(16.21m^2-7.07m-1.55\right]\right\}$ (4.12)
Rattanapitikon and Shibayama (2000)	RS00c	$\frac{H_{mp}}{L} = 0.14\tanh\left[(-11.21m^2+5.01m+0.91)\frac{2\pi h}{L}\right]$ (4.13)
Goda (2010)	GO10	$\frac{H_{mp}}{h} 0.17\frac{L_o}{h}\left\{1-\exp\left[-1.5\frac{\pi h}{L_o}\left(1+11m^{4/3}\right)\right]\right\}$ (4.14)

4.4 Formulas examination for H_{mp}

In this section, 14 existing formulas (shown in Table 4.3) were examined by using the term of mean percent average error. The examination is separated into three parts, which include regular wave heights, irregular wave heights and both of regular and irregular wave heights.

4.4.1 Regular wave heights

A total of 547 cases (including 8744 data points) from 18 sources of published experimental results are used to examine the applicability of 14 existing breaker height formulas on predicting the maximum possible wave height of regular waves. The errors are separated into 4 groups, based on the term of kh. The examination results of the fourteen formulas are shown in Table 4.4 below:

Table 4.4 The mean percentage error (*MPE*) of the existing formulas comparing with regular waves

Sources	$kh<0.1\pi$ (1499 data)	$0.1\pi<kh<0.2\pi$ (4040 data)	$0.2\pi<kh<0.3\pi$ (1781 data)	$0.3\pi<kh$ (1424 data)	All data (8744 data)
MC94	9.92	2.19	0.95	0.03	2.91
MI44	6.70	1.71	1.22	0.25	2.23
GA69	5.47	0.90	0.24	0.00	1.40
CW69	4.14	0.50	0.07	0.00	0.95
GO70	3.63	0.62	0.33	0.04	0.99
WE72	2.68	0.42	0.24	0.02	0.71
MA76	4.96	0.63	0.09	0.00	1.16
BJ78	9.53	2.54	1.82	0.44	3.25
OM79	6.20	1.50	1.04	0.15	1.99
SK88	7.81	1.49	0.45	0.01	2.12
KA91	3.80	0.56	0.21	0.02	0.96
RS00b	4.10	0.92	0.60	0.06	1.26
RS00c	4.15	0.89	0.57	0.07	1.25
GO10	4.56	0.84	0.43	0.06	1.27

The examination results with regular wave data (mentioned Table 4.1) can be summarized in the following:

a) The errors are inversely proportional to the term *kh*. It means, the regions of $kh < 0.1\pi$, $0.1\pi < kh < 0.2\pi$, $0.2\pi < kh < 0.3\pi$, $0.3\pi < kh$ have diminution of the errors, respectively.

b) In the shallow water ($kh < 0.1\pi$), the formula of Weggel (1972) gives the best prediction (MPE= 2.68%).

c) In the region of $0.1\pi < kh < 0.2\pi$, the formula of Weggel (1972) gives the best prediction (MPE=0.42%).

d) In the region of $0.2\pi < kh < 0.3\pi$, the formula of Collins and Weir (1969) gives the best prediction (MPE=0.07%).

e) In the region of $0.3\pi < kh$, three formulas (Galvin (1969), Collins and Weir (1969) and Madsen (1976)) give the best prediction (MPE=0%).

f) The formula of Weggel (1972) gives the best estimation for four regions of *kh* with the error of 0.71%.

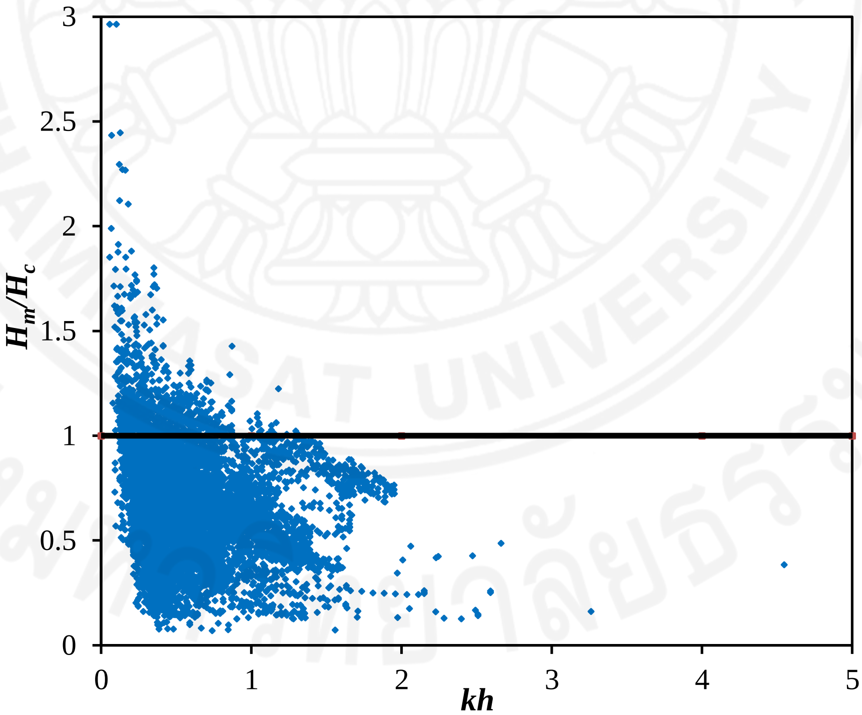


Figure 4.1 Ratio between measured H_m and computed H_c from the formula of Weggel (1972)

4.4.2 Irregular wave heights

A total of 663 cases (including 9137 data points) from 13 sources of published experimental results are used to examine the applicability of 14 existing breaker height formulas on predicting the maximum possible wave height of irregular waves. The errors are separated into 4 groups, based on the term of *kh*. The examination results of 14 formulas are shown in Table 4.5.

Table 4.5 The mean percentage error (*MPE*) of the existing formulas comparing with irregular waves

Sources	$kh<0.1\pi$ (1225 data)	$0.1\pi<kh<0.2\pi$ (4363 data)	$0.2\pi<kh<0.3\pi$ (2218 data)	$0.3\pi<kh$ (1331 data)	All data (9137data)
MC94	37.57	8.90	1.63	0.05	9.69
MI44	32.57	7.46	2.03	0.36	8.47
GA69	30.47	5.16	0.38	0.00	6.64
CW69	22.92	2.51	0.10	0.00	4.30
GO70	21.89	2.83	0.33	0.11	4.38
WE72	22.35	3.27	0.69	0.04	4.73
MA76	25.18	3.05	0.11	0.00	4.86
BJ78	37.19	9.56	2.63	0.63	10.28
OM79	31.74	6.60	1.39	0.28	7.79
SK88	30.59	5.61	0.66	0.03	6.95
KA91	22.53	2.74	0.24	0.08	4.40
RS00b	28.17	5.28	1.24	0.16	6.62
RS00c	27.93	5.09	1.16	0.13	6.47
GO10	24.51	3.57	0.48	0.15	5.13

The examination results with irregular wave data (mentioned in Table 4.2) can be summarized in the following:

a) The error is inversely proportional to the term *kh*. It means, the regions of $kh < 0.1\pi$, $0.1\pi < kh < 0.2\pi$, $0.2\pi < kh < 0.3\pi$, $0.3\pi < kh$ have diminution of the errors, respectively.

b) All of the existing formulas give poor predictions (MPE $\geq$ 10%) in the shallow water ($kh < 0.1\pi$). The formula of Goda (1970) gives the best prediction (MPE= 21.89%) in this region.

c) In the region of $0.1\pi < kh < 0.2\pi$, the formula of Collins and Weir (1969) gives the best prediction with the error of 2.51%.

d) In the region of $0.2\pi < kh < 0.3\pi$, the formula of Collins and Weir (1969) gives the best prediction with the error of 0.1%.

e) In the region of $0.3\pi < kh$, 3 formulas giving the best prediction are Galvin (1969), Collins and Weir (1969) and Madsen (1976) with the error of 0%.

f) The formula of Collins and Weir (1969) gives the best estimation for 4 regions of with the error of 4.3%.

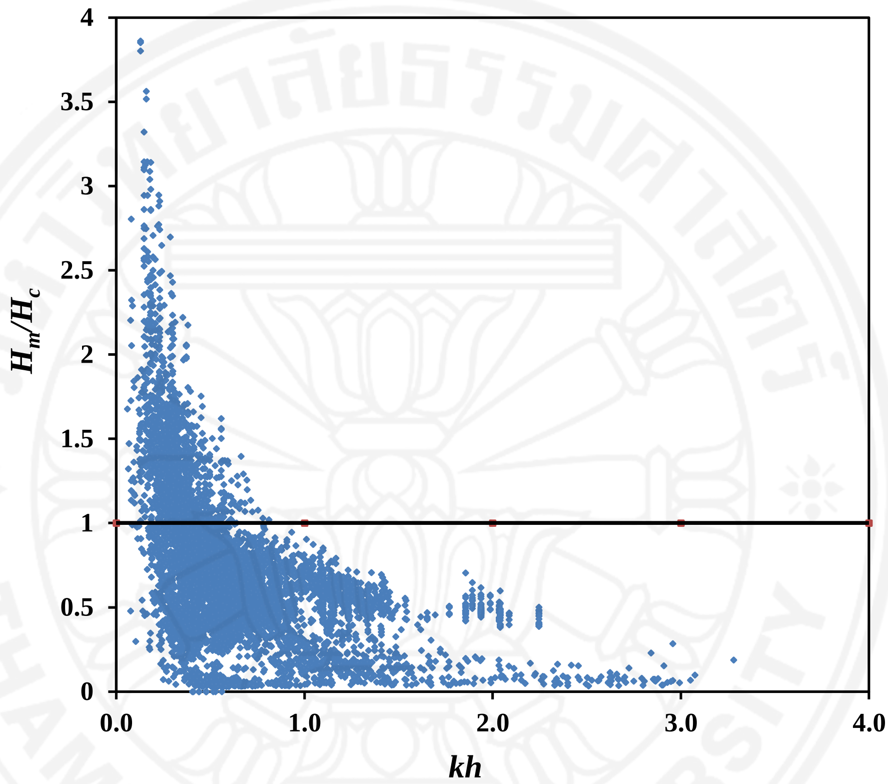


Figure 4.2 Ratio between measured H_m and computed H_c following kh from the formula of Collins and Weir (1969)

4.4.3 Regular and irregular wave heights

A total of 1210 cases (including 17881 data points) from 31 sources of published experimental results are used to examine the applicability of 14 existing breaker height formulas on predicting the maximum possible wave height of regular and irregular waves.

Table 4.6 The mean percentage error (*MPE*) of the existing formula comparing with regular and irregular waves

Sources	All regular wave data (547 case, 8744 data)	All irregular wave data (655 case, 9137 data)	All wave data (1202 case, 17881 data)
MC94	2.91	9.69	6.38
MI44	2.23	8.47	5.42
GA69	1.40	6.64	4.08
CW69	0.95	4.30	2.66
GO70	0.99	4.38	2.72
WE72	0.71	4.73	2.76
MA76	1.16	4.86	3.05
BJ78	3.25	10.28	6.84
OM79	1.99	7.79	4.95
SK88	2.12	6.95	4.59
KA91	0.96	4.40	2.72
RS00b	1.26	6.62	4.00
RS00c	1.25	6.47	3.92
GO10	1.27	5.13	3.24

The examination results of regular and irregular wave are shown in Table 4.6. The results from Table 4.6 can be summarized in the following:

a) The errors of all formulas for regular waves are less than that of irregular waves.

b) The formula of Collins and Weir (1969) gives the best estimation for all wave data (regular and irregular wave heights) with the error of 2.66%.

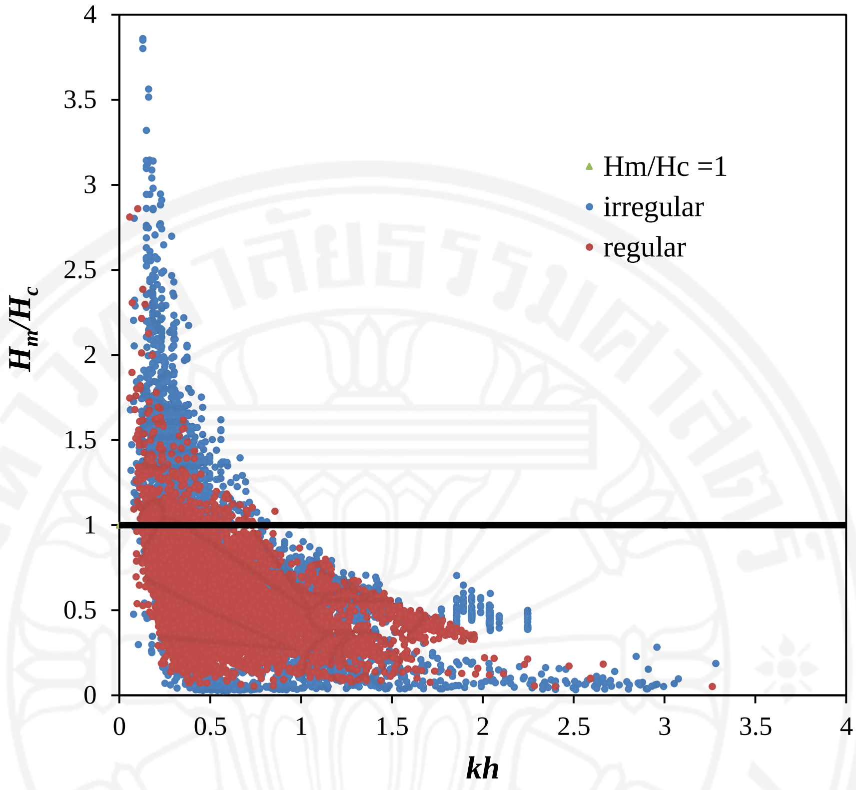


Figure 4.3 Ratio between measured H_m and computed H_c of all data from the formula of Collins and Weir (1969)

4.5 Modification of formulas of H_{mp}

Three existing formulas giving the best prediction are selected to modify to make better accuracy. Those formulas are Collins and Weir (1969), Goda (1970), and Kaimphuis (1991).

The relative depth effect coefficient may be included in the formula of CW69, GO70 and KA91 by adding or replacing the original relative depth term h/L_o, kh or $tanh(kh)$ with the relative depth effect coefficient (K_{CW}, K_{GO} and K_{KA}) as:

$$\frac{H_{mp}}{h} = \left(0.72 + 5.6m\right) K_{CW} \tag{4.15}$$

$$\frac{H_{mp}}{L_o} = 0.17\left\{1 - \exp\left[-1.5\pi\left(1 + 15m^{4/3}\right) K_{GO}\right]\right\} \tag{4.16}$$

$$\frac{H_{mp}}{L} = \exp(4m)K_{KAa} \tag{4.17}$$

where K_{CW}, K_{GO} and K_{KA}, which are function of the relative depth (*h/L*$_o$, *kh or tanh(kh)*), are the relative depth effect coefficient in formulas of Collins and Weir (1969), Goda (1970) and Kaimphuis (1991).

From Eqs. (4.15), (4.16) and (4.17), the equations of relative depth effect coefficient can be written as:

$$K_{CW} = \frac{H_{mp}}{h\left(0.72+5.6m\right)} \tag{4.18}$$

$$K_{GO} = \frac{\ln\left(1-\dfrac{H_{mp}}{0.12L_o}\right)}{-1.5\pi\left(1+15m^{4/3}\right)} \tag{4.19}$$

$$K_{KA} = \frac{H_{mp}}{\exp(4m)L} \tag{4.20}$$

As the experimental data of Kraus and Smith (1994) covers a wide range of experimental conditions, it is used to determine the relative depth effect coefficient. The measured value of K_{CW}, K_{GO} and K_{KA} can be determined from Eq. (4.18) to Eq. (4.20) by using the measured wave height H_{max} (replacing H_{mp} with H_{max}), L_o and m. The relationships of measured relative depth effect coefficients (K_{CW}, K_{GO} and K_{KA}) following *kh* are shown in Figs. (4.4a), (4.5a) and (4.6a), respectively.

It is difficult to determine the curve, which is the upper bound of the scatter diagram shown in Figs. (4.4a), (4.5a) and (4.6a). In this study, the upper boundary is determined by using boundary line method, which fits the representative points in intervals of h/L_o or kh.

The curve is determined by fitting the points represented for intervals. To determine the points in this case, the X axis (*kh*) will be divided into many intervals. The selected point to represent for each interval is maximum point in that interval. The number of representative points is selected based on 2 criteria, i.e. to use at least 10 representative points in order to limit the selection of points to the superior boundary of the scatter points, to maximize the likelihood of developing statistically significant models by increasing the number of representative points. The number of representative points is 20 intervals and the value of each interval for CW69, GO70 and KA91 is 0.09, 0.013 and 0.09, respectively.

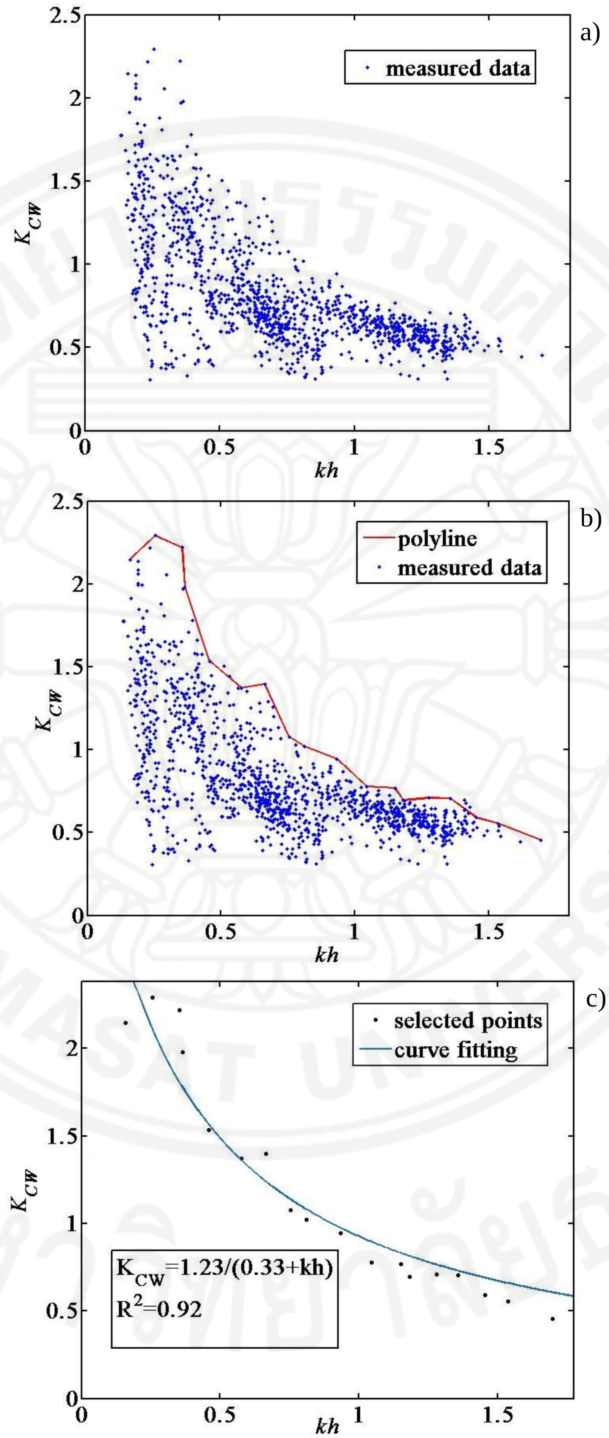


Figure 4.4 Relationship between K_{CW} from formula of CW69 and *kh* for H_{max} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of CW69.

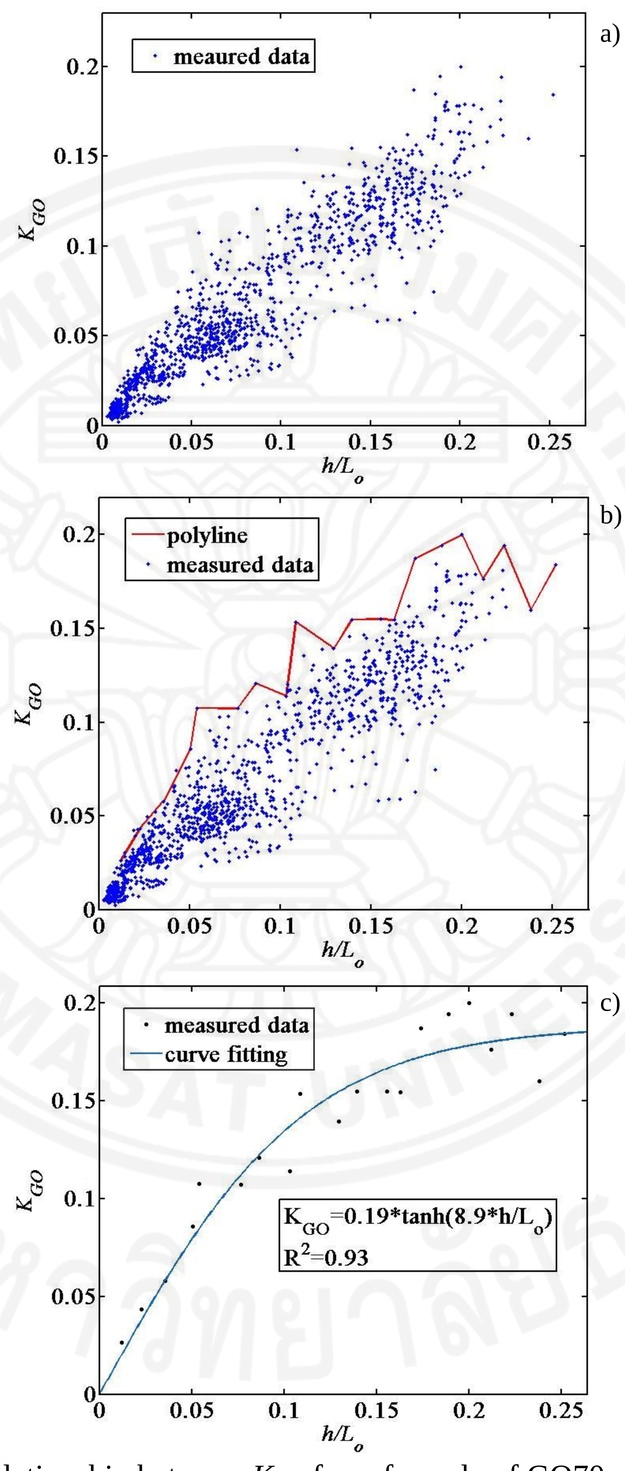


Figure 4.5 Relationship between K_{GO} from formula of GO70 and kh for H_{max} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of GO70.

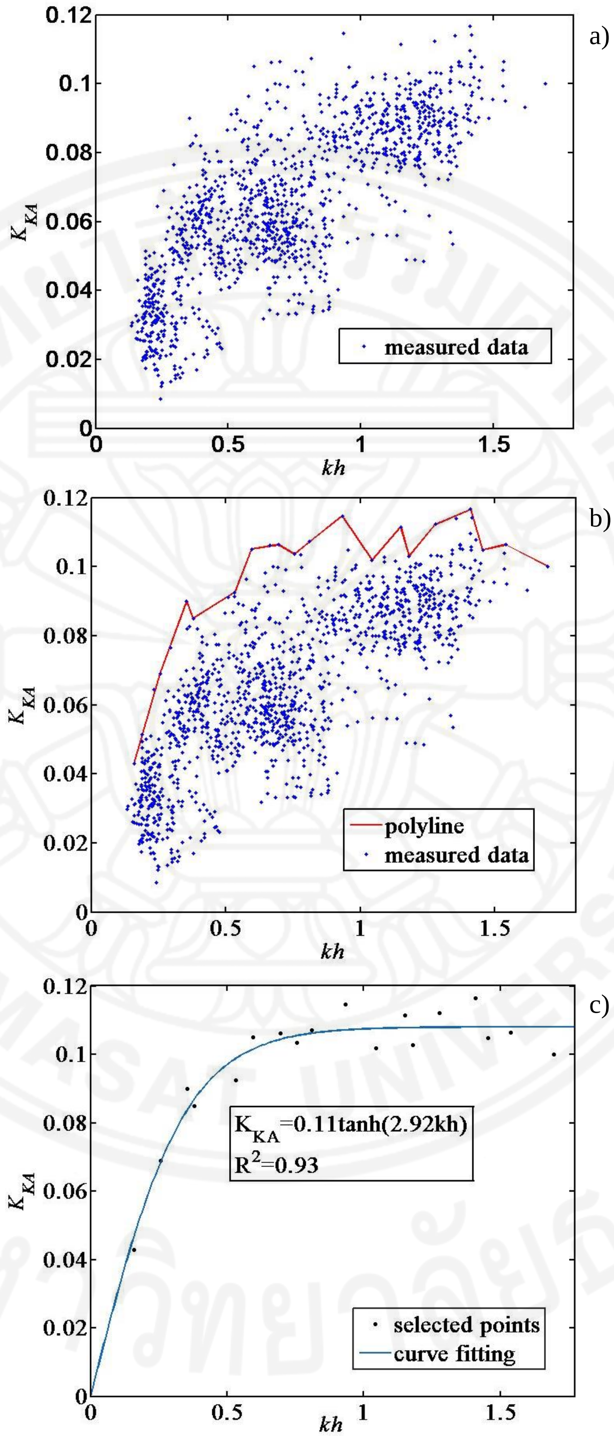


Figure 4.6 Relationship between K_{KA} from formula of KA91 and *kh* for H_{max} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of KA91.

The formulas of K_{CW}, K_{GO} and K_{KA} mentioned in Fig. (4.4c), Fig. (4.5c) and Fig. (4.6c) are:

$$K_{CW} = \frac{a_1}{b_1 + kh} \tag{4.21}$$

$$K_{GO} = a_2 \times \tanh\left(b_2 \left(\frac{h}{L_o} \right) \right) \tag{4.22}$$

$$K_{KA} = a_3 \times \tanh(b_3 \left(kh\right)\ ) \tag{4.23}$$

where a_i and b_i are constants with i =1, 2 or 3. From using the curve fitting for that collected points, the values of a_i and b_i are shown in Table 4.7. Eq. (4.21), Eq. (4.22) and Eq. (4.23) with the fitted constants (a_i and b_i) is plotted as a solid line in Figures 4.7, 4.8 and 4.9.

Table 4.7 The fitted values

i	Relative depth effect coefficient	a_i	b_i
1	K_{CW} (Eq. (4.21))	1.23	0.33
2	K_{GO} (Eq. (4.22))	0.19	8.9
3	K_{KA} (Eq. (4.23))	0.11	2.92

Substituting Eq. (4.21), Eq. (4.22) and Eq. (4.23) with fitted constants shown in Table 4.7 into Eq. (4.15), Eq. (4.16) and Eq. (4.17), respectively. The modified formulas are expressed as:

$$\frac{H_{mp}}{h} = \left(0.72 + 5.6m\right) \times \frac{1.23}{0.33 + kh} \tag{4.24}$$

$$\frac{H_{mp}}{L_o} = 0.17\left\{1 - \exp\left[-1.5\pi\left(1 + 15m^{4/3}\right) * 0.19 * \tanh\left(8.9\frac{h}{L_o}\right)\right]\right\} \tag{4.25}$$

$$\frac{H_{mp}}{L} = 0.11e^{4m}\tanh\left(2.92 \times kh\right) \tag{4.26}$$

Hereafter, Eq. (4.24), Eq. (4.25) and Eq. (4.26) are referred to as MCW69, MGO70 and MKA91, respectively.

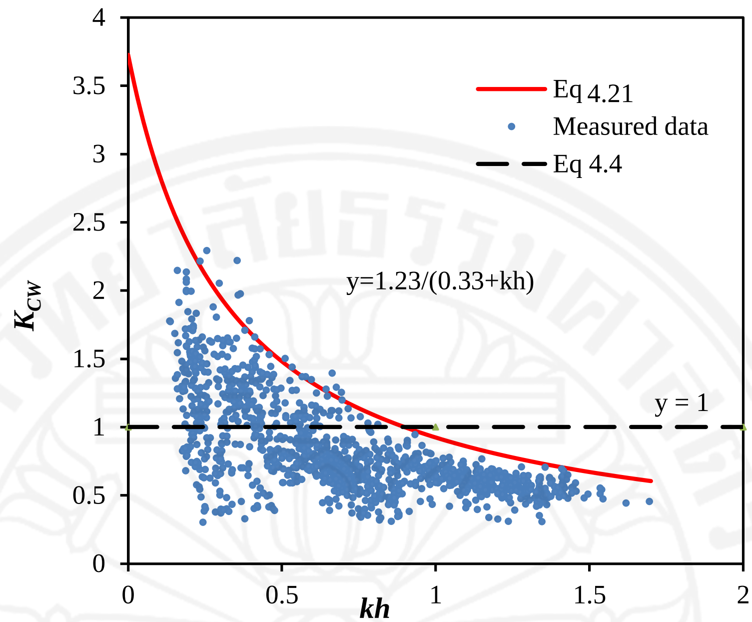


Figure 4.7 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (4.21) with a_i = 1. 23 and b_i = 0.33. Long Dash Dot line is the Eq. (4.4) (existing formula).

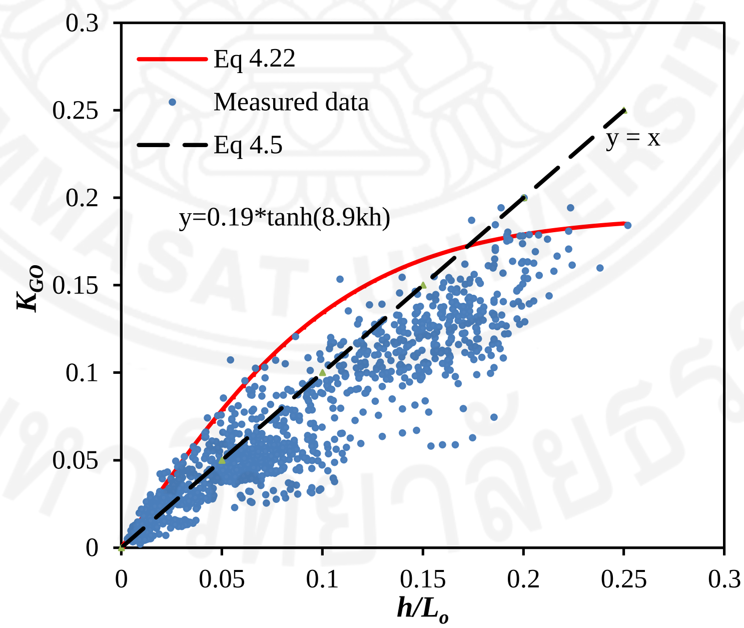


Figure 4.8 Relationship between K_{GO} and h/L_o (measured data from Kraus and Smith 1994). Solid line is the Eq. (4.22) with a_i = 0. 19 and b_i = 8.9. Long Dash Dot line is the Eq. (4.5) (existing formula).

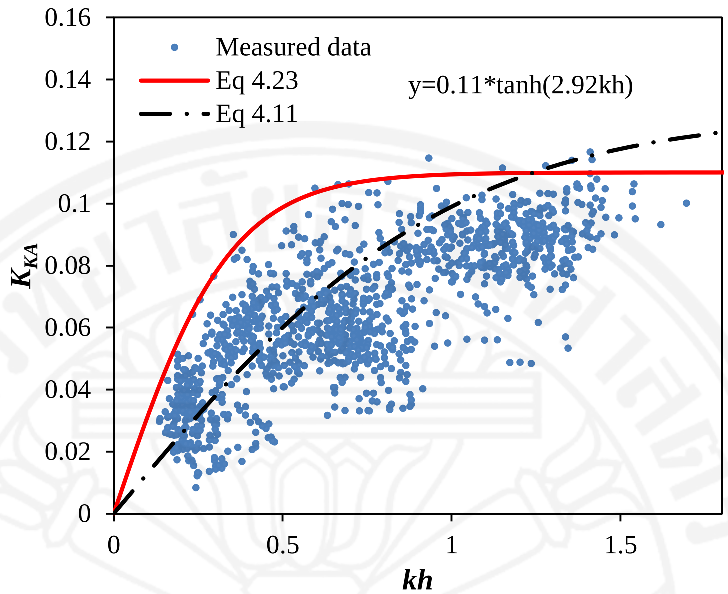


Figure 4.9 Relationship between K_{KA} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (4.23) with a_i = 0.11 and b_i = 2.92. Long Dash Dot line is the Eq. (4.11) (existing formula).

4.6 Verification of modified formula of H_{mp}

In order to confirm the performance of the modified formulas, errors of the modified formulas are compared with those of existing formulas. The computations of the 3 modified formulas are carried out with all collected data shown in Tables 4.1 and 4.2. The Tables 4.8, 4.9 and 4.10 show the errors of the modified formulas for 4 groups of relative depth (*kh*) and all cases. The examination results from Tables 4.4 to 4.9 can be summarized as follows:

a) Similar to those of existing formulas, the errors of modified formulas of regular and irregular waves nearly have the same tendency and the errors of irregular waves tend to be larger than that of regular waves.

b) In the region close to and within shallow water ($kh < 0.3\pi$), the modified formulas give considerable better predictions than those of existing formulas.

c) In intermediate depth ($kh \geq 0.3\pi$), the modified formulas give good prediction as those of most existing formulas.

d) The modified formulas are applicable for either intermediate depth or shallow water regions.

e) The overall errors of the modified formulas ($0.08 \leq MPE \leq 0.41\%$) are considerable less than those of existing formulas.

Table 4.8 The mean percentage error (*MPE*) of the modified formulas comparing with regular waves

Formulas	$kh< 0.1\pi$ (1499 data)	$0.1\pi<kh<0.2\pi$ (4040 data)	$0.2\pi<kh<0.3\pi$ (1781 data)	$0.3\pi<kh$ (1424 data)	MPEavg (8744 data)
MCW69	0.06	0.00	0.01	0.01	0.02
MGO70	0.29	0.00	0.01	0.02	0.06
MKA91	0.12	0.00	0.01	0.01	0.02

Table 4.9 The mean percentage error (*MPE*) of the modified formulas comparing with irregular waves

Formulas	$kh< 0.1\pi$ (1225 data)	$0.1\pi<kh<0.2\pi$ (4363data)	$0.2\pi<kh<0.3\pi$ (2218 data)	$0.3\pi<kh$ (1331 data)	MPE_{avg} (9137 data)
MCW69	0.78	0.05	0.01	0.08	0.14
MGO70	4.11	0.13	0.00	0.79	0.73
MKA91	1.21	0.04	0.00	0.33	0.23

Table 4.10 The mean percentage error (*MPE*) of the modified formulas comparing with regular and irregular waves

Formulas	MPE of irregular (9137 data)	MPE of regular (8744 data)	MPE_{avg} (17881 data)
MCW69	0.14	0.02	0.08
MGO70	0.73	0.06	0.41
MKA91	0.23	0.02	0.13

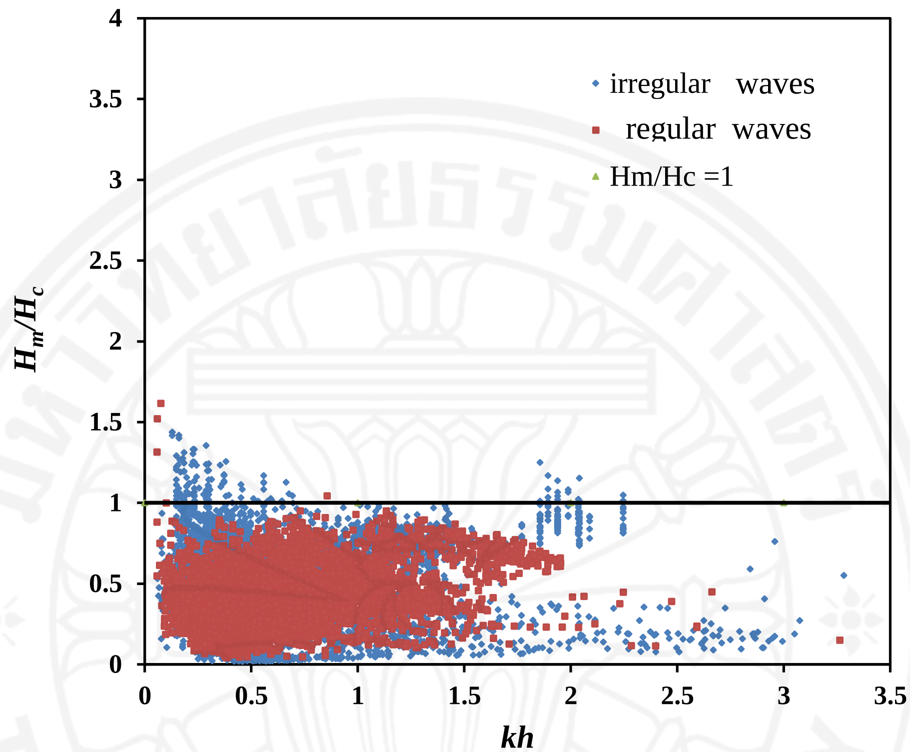


Figure 4.10 Relationships between the ratio of measured H_m and computed H_c versus kh from the Collin and Weir's formula.

Chapter 5
Proposal for Common Formulas to Estimate Maximum Possible Representative Wave Heights

5.1 Introduction

Representative wave heights are an important factor that this study mentioned in previous chapters. Furthermore, many existing formulas, which used to develop based on empirical approach, are proposed to predict representative wave heights. Because of these reasons, it is necessary to find the best formulas for prediction of representative wave heights to make it easier in application. However, so far the existing formulas have never been proposed by any researcher to estimate all types of representative wave heights except maximum wave height and significant wave height. Hence, the problem is how to choose the existing formulas predicting for all types of representative wave heights.

Based on the applicable prediction of 3 modified formulas in Chapter 4, 2 formulas mentioned in Table 5.2 are used to develop for predicting all types of representative wave heights.

5.2 Data collection

The description of wave breaking mechanism by using available wave theories is difficult because of the complication of wave breaking mechanism. Therefore, to make it easier, this study is against empirical formula.

Furthermore, as the existing breaker height formulas were proposed based on the empirical approach or semi-empirical approach, the applicability of those existing formulas will be affected by the number of experiments. The existing formulas were suggested many years ago and at that time the number of experiments is limited by many factors, e.g. budget, equipment and knowledge, etc., hence, the existing formulas may not be suitable to apply for today. It should be replaced by new empirical formulas or new modified empirical formulas.

The problem is how to make the empirical formula trustworthy. To solve this problem, a large amount and wide range of experimental data mentioned in Table 3.5 of Chapter 3 were used to make sure that the empirical formula will be become reliable.

To develop and modify the formulas mentioned in Table 5.2 for computing the representative wave heights, this study has been assembled the measured data of representative wave heights of irregular waves from 21 sources including 2022 cases with 52080 data points. The depth ratio (H_{rep}/h), the relative depth (kh), and the bottom slope (m) of data in 21 experiments are shown as Table 5.1.

Table 5.1 The characteristics of representative wave heights

Representative wave height	No. of experiments	No. of cases	No. of data	H/h	kh	m
H_{max}	13	655	9137	0.02-3.94	0.06-4.61	0.00-0.40
H_{mo}	11	1727	18792	0.00-4.00	0.02-4.61	0.00-0.31
$H_{1/3}$	15	417	7095	0.01-1.73	0.06-4.61	0.00-0.40
$H_{1/10}$	10	383	6781	0.02-2.17	0.06-2.99	0.00-0.31
H_{mean}	10	186	3328	0.01-0.86	0.08-4.61	0.00-0.40
H_{rms}	12	399	6937	0.01-1.25	0.06-2.99	0.00-0.40

The experiments cover a large number of data including small-scale, large-scale, and field experiments. The small-scale experiments were conducted under plane and barred bed conditions, while the large-scale and filed experiments were performed under movable bed conditions. In used sources, there are 9 experiments of small-scale conducted by Smith and Kraus (1990), Hurue (1990), Katayama (1991), Smith and Vincent (1992), Hamilton and Ebersole (2001), Smith and Seabergh (2001), Ting (2001), Ting (2002), and SWIMS Project (2012); 3 experiments of large-scale are performed by SUPERTANK (1994), LIP 11D (1995), and SAFE (1998) project. The others conducted by Goodknight and Russell (1963), Goda (1974), Hotta et al. (1982),Kraus (1989) (DUCK 85 project), Thornton and Guza (1986), Long (1991), Birkemeier et. Al (1997):DELILAH Project, Whitehouse and Sutherland (2001), and Herbers et. al (2006):DUCK94 Project are field-scale experiments.

5.3 Selected existing formulas

The existing formulas, which were proposed by many previous researchers before, do not predict for all types of representative wave heights. It was only proposed for maximum wave height and significant wave height. Therefore, it is not easy to use for the people, who want to estimate representative wave heights.

This study would like to develop some general formulas used for predicting common representative wave heights. 2 formulas are considered as shown in Table 5.2.

Table 5.2 Existing breaker height formulas for computing $H_{rep,max}$

Researchers	Abbrev-iation	Formulas
Collin and Weir (1969)	CW69	$\frac{H_{rep,\max}}{h} = (0.72 + 5.6m)$ (5.1)
Kamphuis (1991)	KA91	$\frac{H_{rep,\max}}{L} = 0.13\exp(4m)\tanh\left(\frac{2\pi h}{L}\right)$ (5.2)

5.4 Genaral formulas for estimating $H_{rep,max}$

The relative depth effect coefficient may be included in the formula of CW69 and KA91 by adding or replacing the original relative depth term *kh* or *tanh(kh)* with the relative depth effect coefficient (K_{CW} and K_{KA}) as:

$$\frac{H_{rep,\max}}{h} = (0.72 + 5.6m)K_{CW} \quad (5.3)$$

$$\frac{H_{rep,\max}}{L} = \exp(4m)K_{KAa} \quad (5.4)$$

where K_{CW} and K_{KA}, which are function of the relative depth (*kh* or *tanh(kh)*), are the relative depth effect coefficient in formulas of Collins and Weir (1969), Goda (1970) and Kaimphuis (1991).

From Eq. (5.3) and Eq. (5.4), the equation of relative depth effect coefficient can be written as:

$$K_{CW} = \frac{H_{rep,\max}}{h\left(0.72 + 5.6m\right)} \tag{5.5}$$

$$K_{KA} = \frac{H_{rep,\max}}{\exp(4m)L} \tag{5.6}$$

As the experimental data of Kraus and Smith (1994) covers a wide range of experimental conditions, it is used to determine the relative depth effect coefficient. The measured value of K_{CW} and K_{KA} can be determined from Eq. (5.5) and Eq. (5.6) by using the measured wave height H_{rep} (replacing $H_{rep,max}$ with H_{rep}), L, and m. The relationship of measured relative depth effect coefficients (K_{CW} and K_{KA}) following *kh* is shown in Figures 5.1a to 5.12a for representative wave heights.

It is difficult to determine the curve, which is the upper bound of the scatter diagram shown in Figures 5.1a to 5.12a. In this study, the upper boundary is determined by using boundary line method, which fits the points represented for intervals of kh.

The curve is determined by fitting the points represented for intervals. To determine the points in this case, the horizontal axis (*kh*) will be divided into many intervals. The selected point to represent for each interval is maximum point in that interval. The number of representative points was selected based on two criteria, i.e. to use at least 10 representative points in order to limit the selection of points to the superior boundary of the scatter points, to maximize the likelihood of developing statistically significant models by increasing the number of representative points. The number of representative points is 20 intervals and the value of each interval for CW69 and KA91 is 0.09 for maximum wave height and 0.045 for the other representative wave heights.

Especially, for Collin and Weir formula (CW69), the representative points staying between 0 and 0.18 of X axis (*kh*) are excluded for all types of representative wave heights except maximum wave height because of the trend difference in scatter diagram between 2 segments which have value, respectively, from 0 to 0.18 and from 0.18 to 0.9.

Some figures of all types of representative wave heights are shown below to describe that procedure :

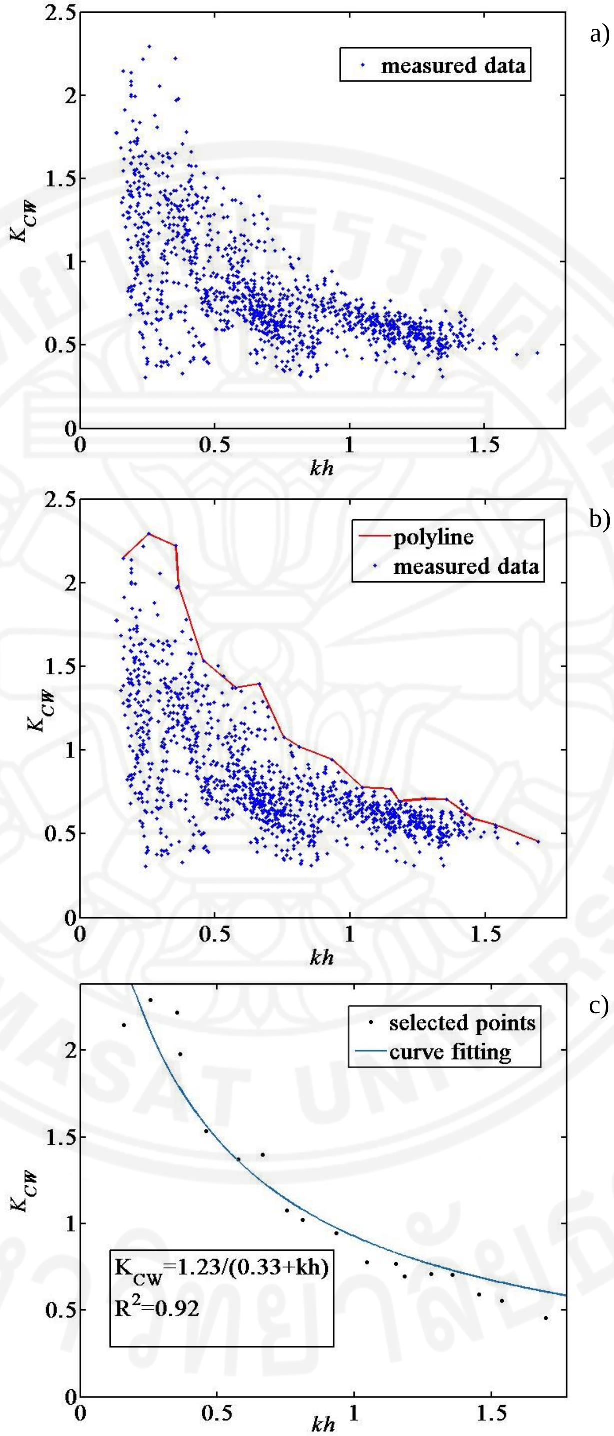


Figure 5.1 Relationship between K_{CW} from formula of CW69 and *kh* for H_{max} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of CW69.

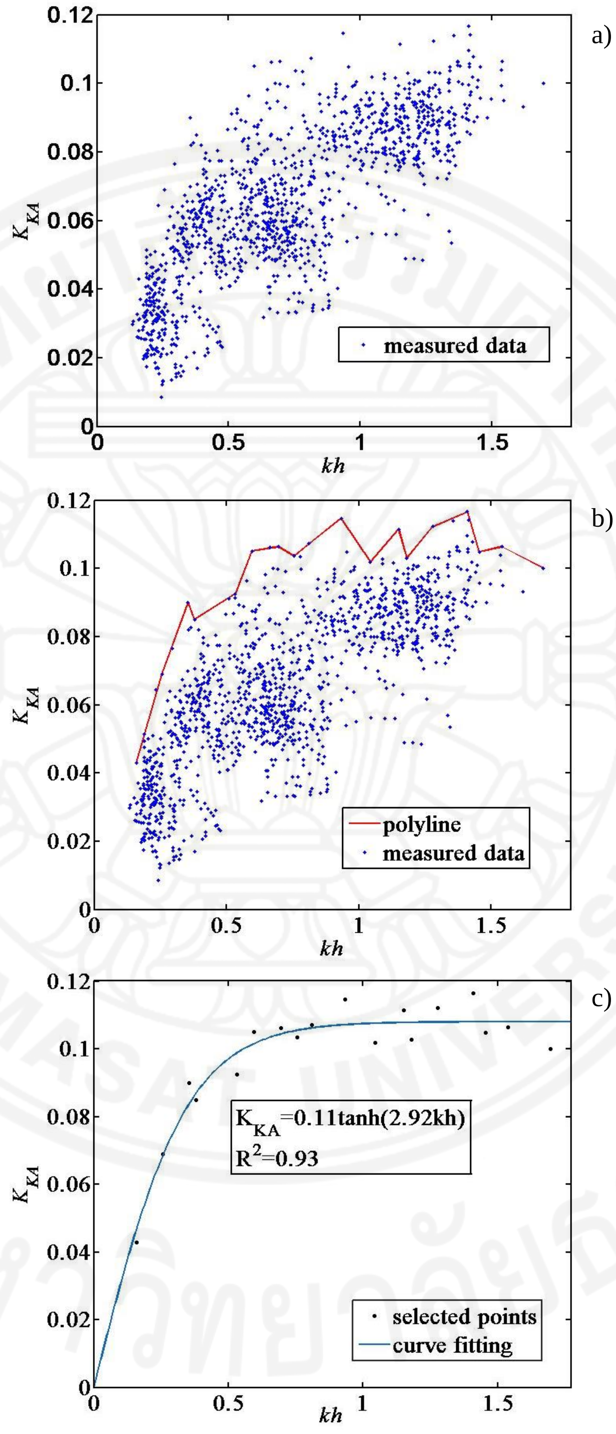


Figure 5.2 Relationship between K_{KA} from formula of KA91 and kh for H_{max} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of KA91.

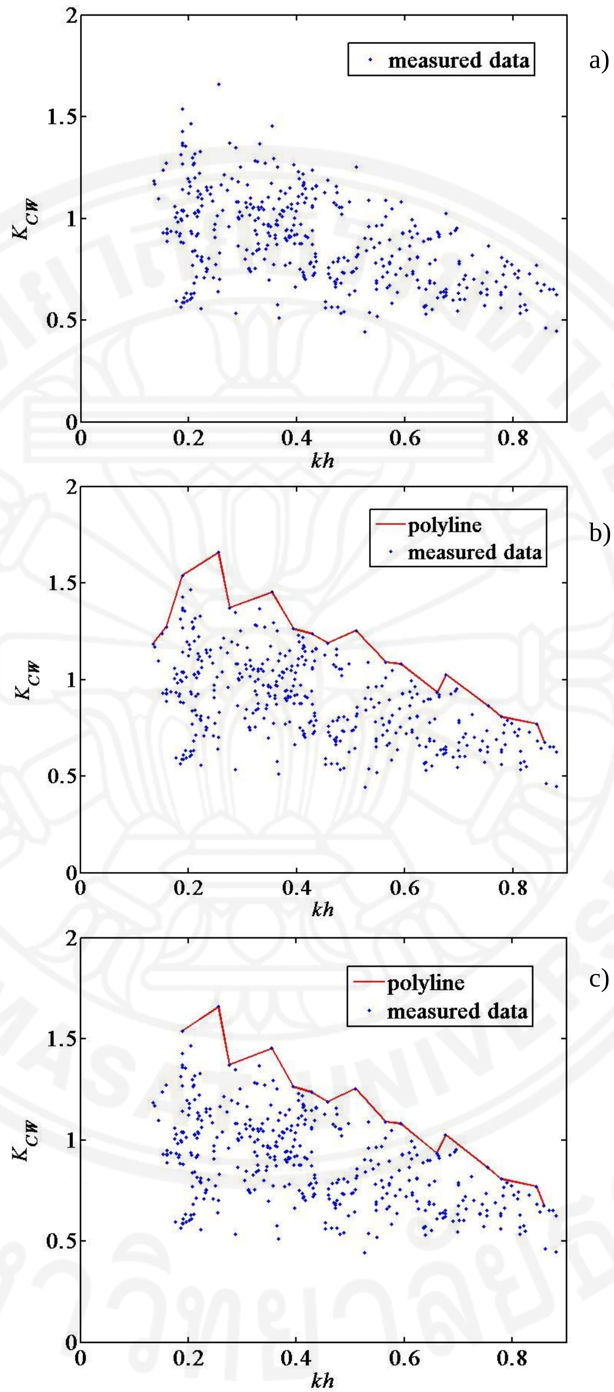


Figure 5.3 Relationship between K_{CW} from formula of CW69 and *kh* for $H_{1/10}$:
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each intervals after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

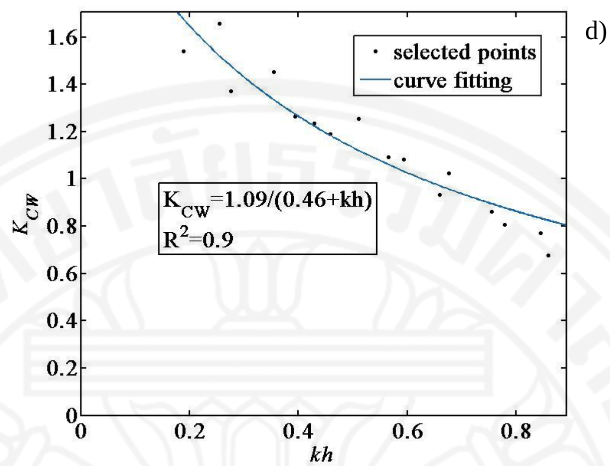


Figure 5.3(cont.) Relationship between K_{CW} from formula of CW69 and *kh* for $H_{1/10}$: a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each intervals after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

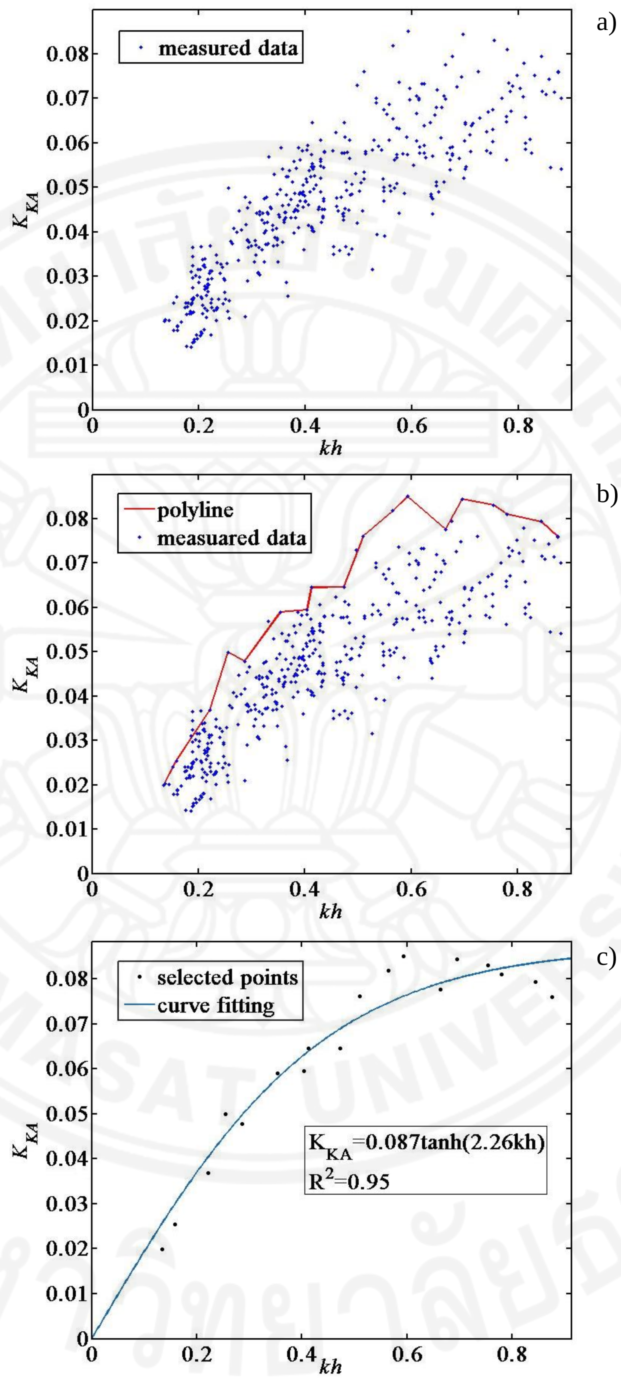


Figure 5.4 Relationship between K_{KA} from formula of KA91 and *kh* for $H_{1/10}$:
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of KA91.

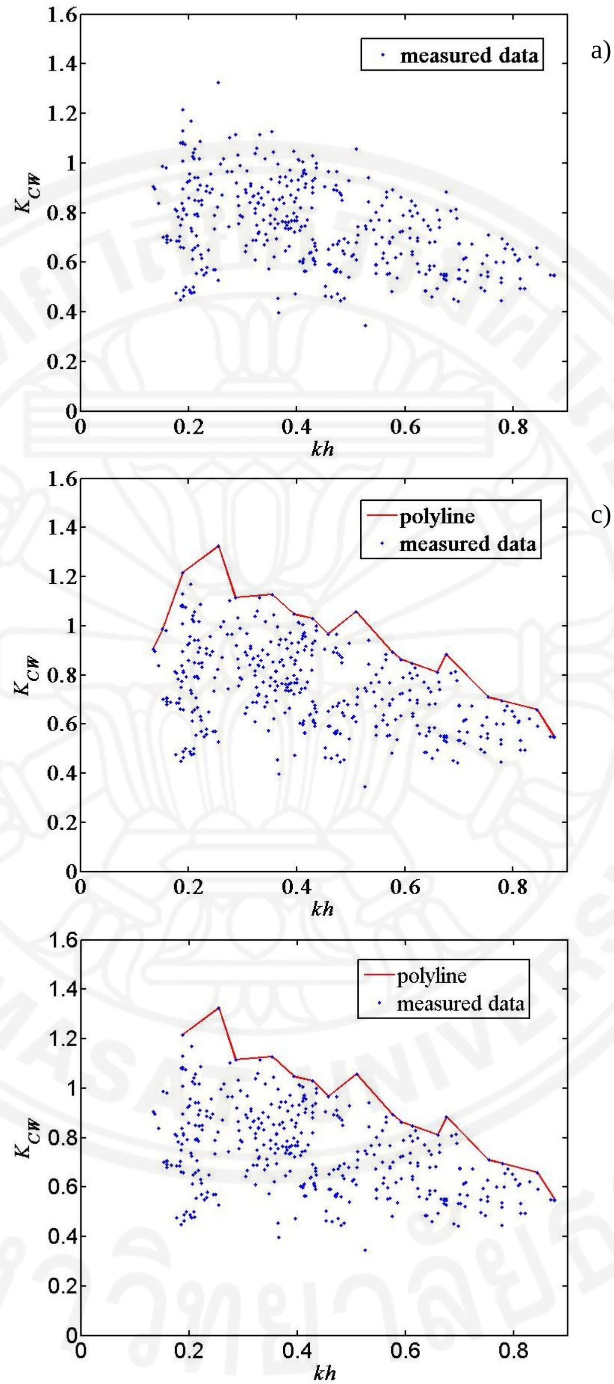


Figure 5.5 Relationship between K_{CW} from formula of CW69 and *kh* for $H_{1/3}$:
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

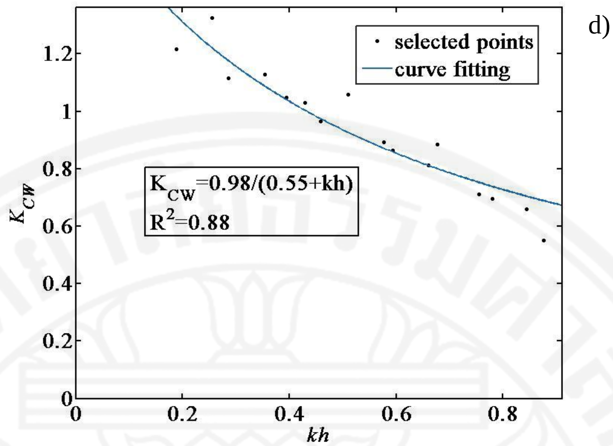


Figure 5.5(cont.) Relationship between K_{CW} from formula of CW69 and *kh* for $H_{1/3}$: a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

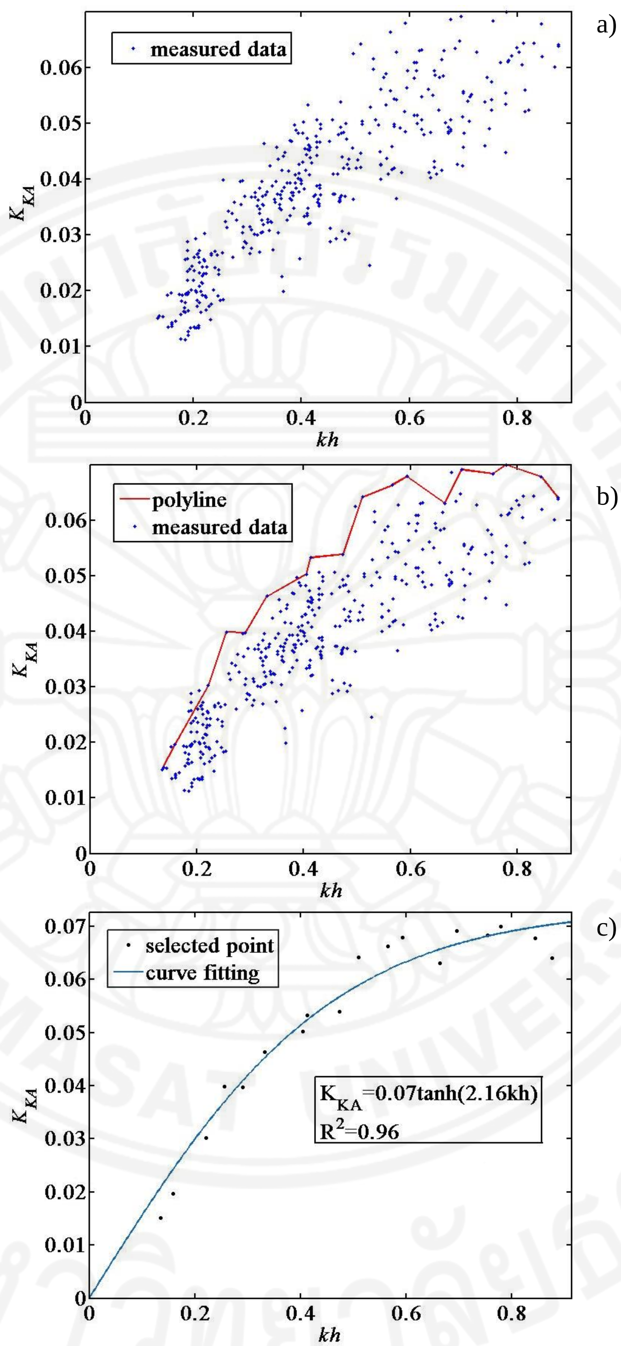


Figure 5.6 Relationship between K_{KA} from formula of KA91 and *kh* for $H_{1/3}$:
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The curve fitting of formula of KA91.

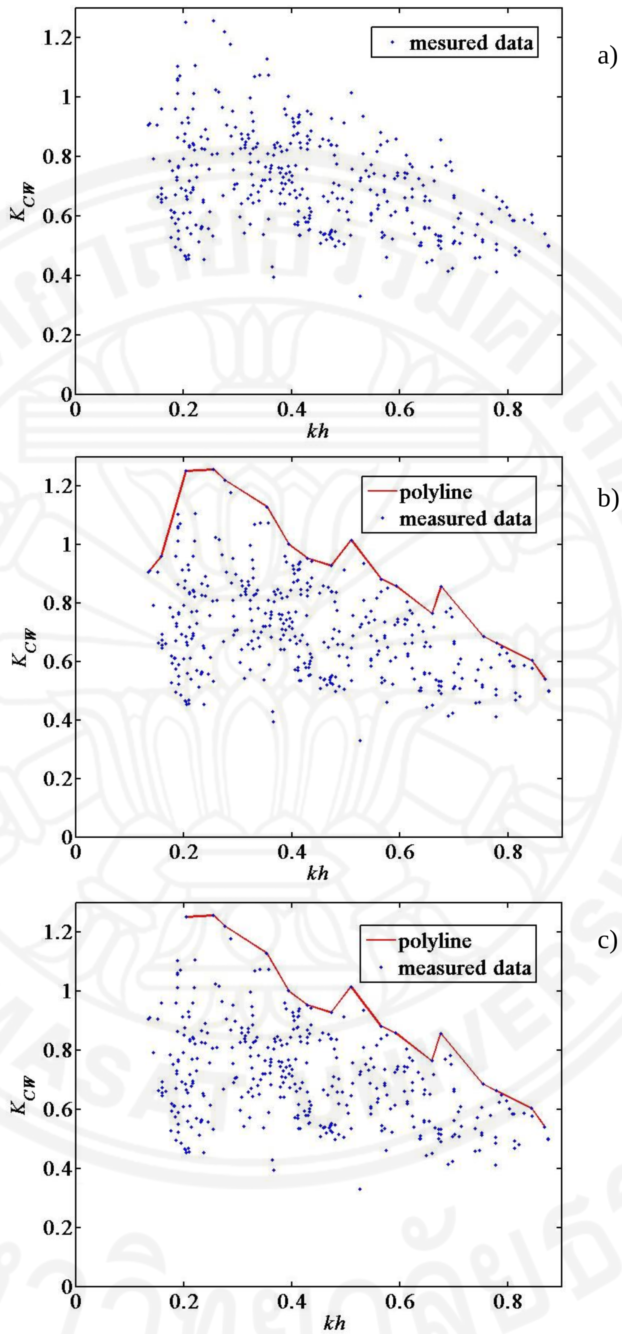


Figure 5.7 Relationship between K_{CW} from formula of CW69 and *kh* for H_{mo} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

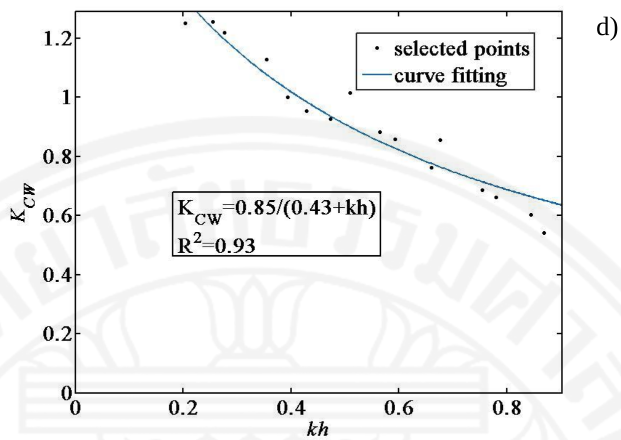


Figure 5.7(cont.) Relationship between K_{CW} from formula of CW69 and *kh* for H_{mo} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

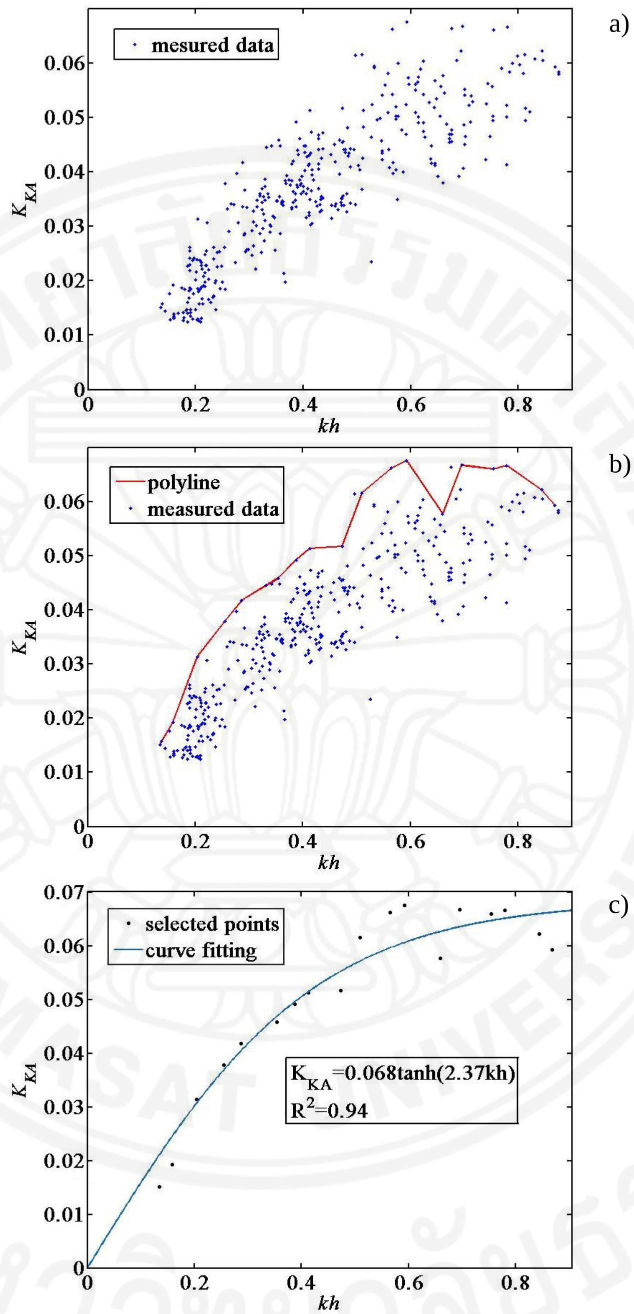


Figure 5.8 Relationship between K_{KA} from formula of KA91 and *kh* for H_{mo} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The curve fitting of formula of KA91

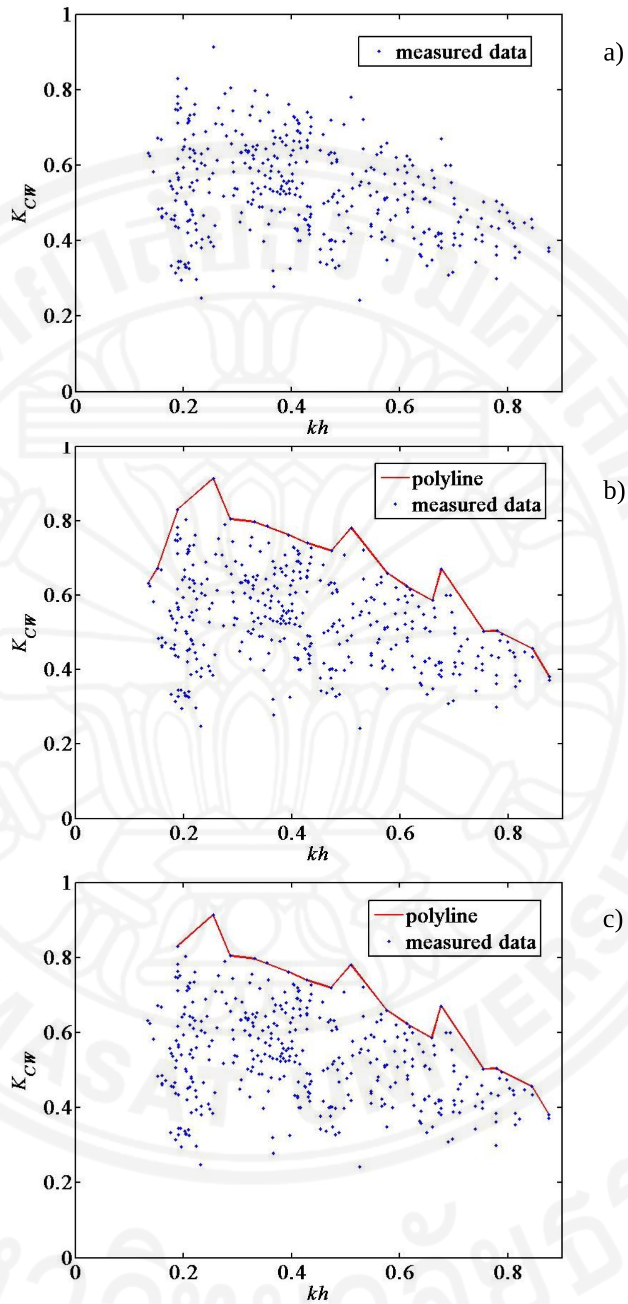
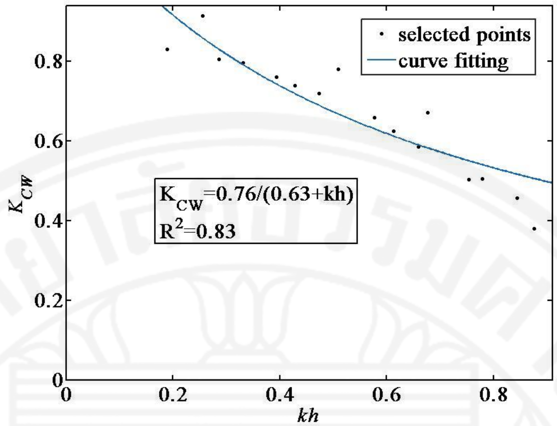


Figure 5.9 Relationship between K_{CW} from formula of CW69 and *kh* for H_{rms} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.



d)

Figure 5.9(cont.) Relationship between K_{CW} from formula of CW69 and *kh* for H_{rms} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

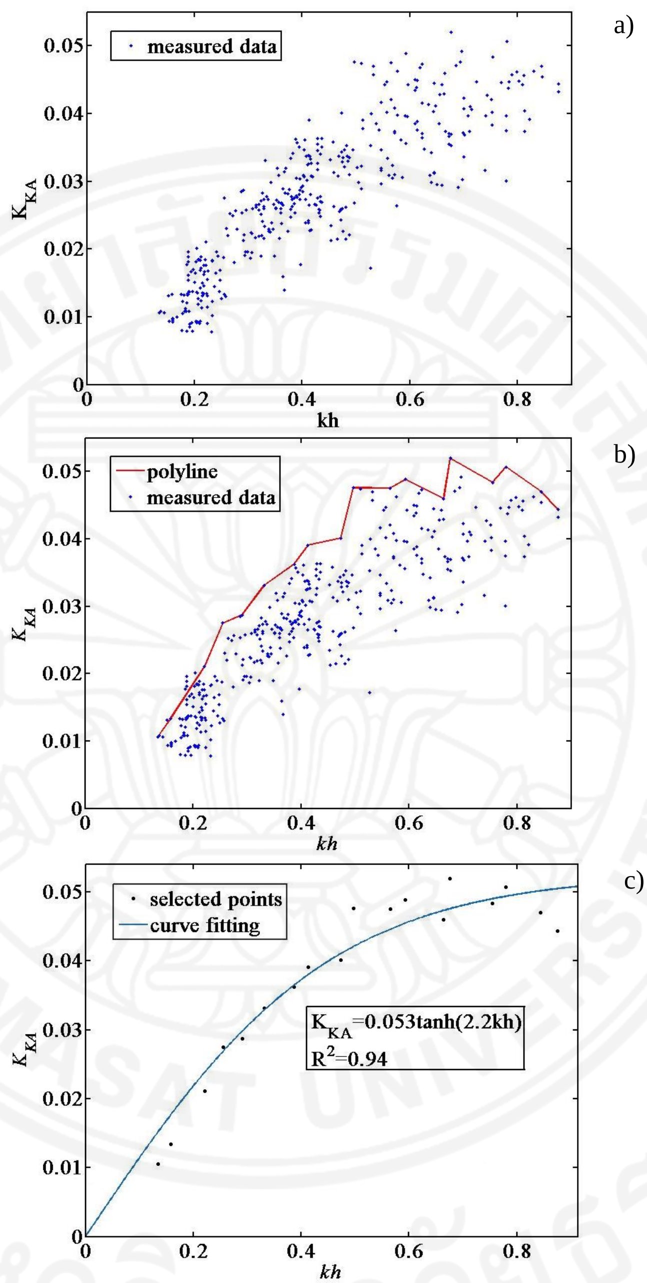


Figure 5.10 Relationship between K_{KA} from formula of KA91 and *kh* for H_{rms} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The curve fitting of formula of KA91.

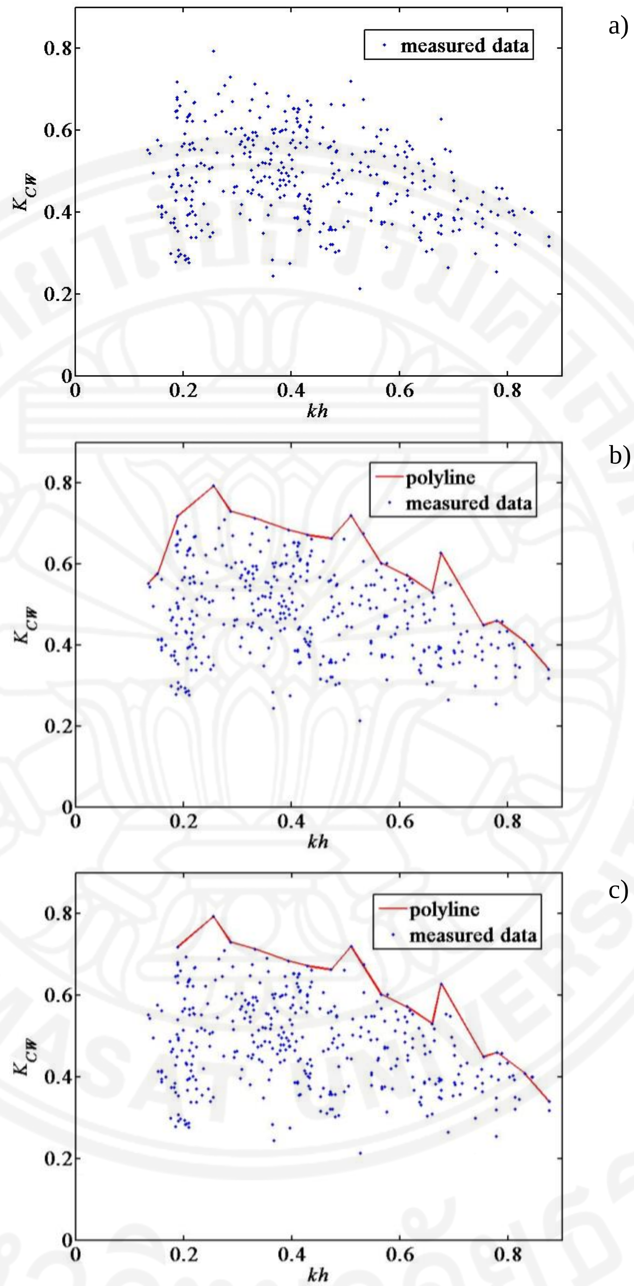
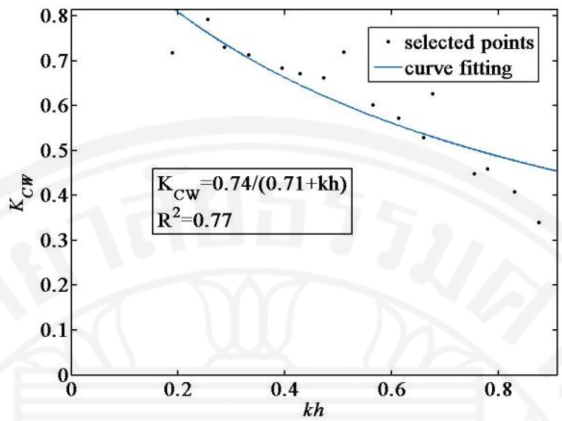


Figure 5.11 Relationship between K_{CW} from formula of CW69 and *kh* for H_{mean} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.



d)

Figure 5.11(cont.) Relationship between K_{CW} from formula of CW69 and *kh* for H_{mean} : a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, c) The polyline fitting the maximum point in each interval after excluding points in 0-0.18 range of *kh*, and d) The curve fitting of formula of CW69.

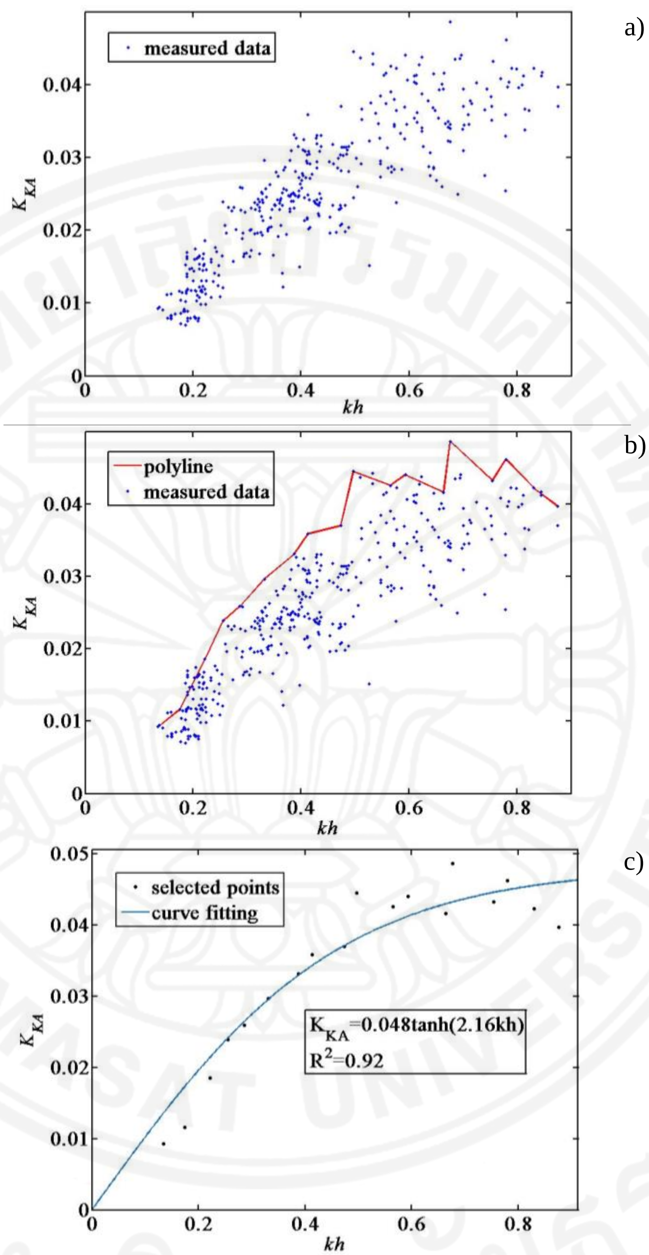


Figure 5.12 Relationship between K_{KA} from formula of KA91 and *kh* for H_{ean} :
a) Measured data from Kraus and Smith 1994, b) The polyline fitting the maximum point in each interval, and c) The curve fitting of formula of KA91.

The formulas of K_{CW} and K_{KA} mentioned in Fig. (5.1c), Fig. (5.2c), Fig. (5.3d) Fig. (5.4c), Fig. (5.5d), Fig. (5.6c), Fig. (5.7d), Fig. (5.8c), Fig. (5.9d) Fig. (5.10c), Fig. (5.11d) and Fig. (5.12c) are:

$$K_{CW} = \frac{a_1}{b_1 + kh} \tag{5.7}$$

$$K_{KA} = a_3 \times \tanh(b_3 \left(kh\right)\ ) \tag{5.8}$$

where a_i and b_i are constants with i =1 or 3. From using the curve fitting for that collected points, the values of a_i and b_i are shown in Table 5.3. Eq. (5.7) and Eq. (5.8) with the fitted constants (a_i and b_i) is plotted as a solid line in Figures 5.13 to 5.24.

Table 5.3 The fitted values

K	a_i, b_i	$H_{\max}$	$H_{1/10}$	$H_{1/3}$	H_{mo}	H_{rms}	H_{mean}
K_{CW}	a_1	1.23	1.09	0.98	0.85	0.76	0.74
	b_1	0.33	0.46	0.55	0.43	0.63	0.71
K_{KA}	a_3	0.11	0.087	0.074	0.068	0.053	0.048
	b_3	2.92	2.26	2.16	2.37	2.2	2.16

Substituting Eq. (5.7) and Eq. (5.8) with fitted constants shown in Table 5.3 into Eq. (5.3) and Eq. (5.4), respectively. The modified formulas are expressed as:

$$\frac{H_{rep,\max}}{L} = (0.72 + 5.6m) \times \frac{a_1}{b_1 + kh} \tag{5.9}$$

$$\frac{H_{rep,\max}}{L} = a_3 e^{4m} \tanh\left(b_3 \times kh\right) \tag{5.10}$$

Hereafter, Eq. (5.9) and Eq. (5.10) with the fitted values in Table 5.3 are referred to as MCW69 and MKA91, respectively.

❖ H_{max}

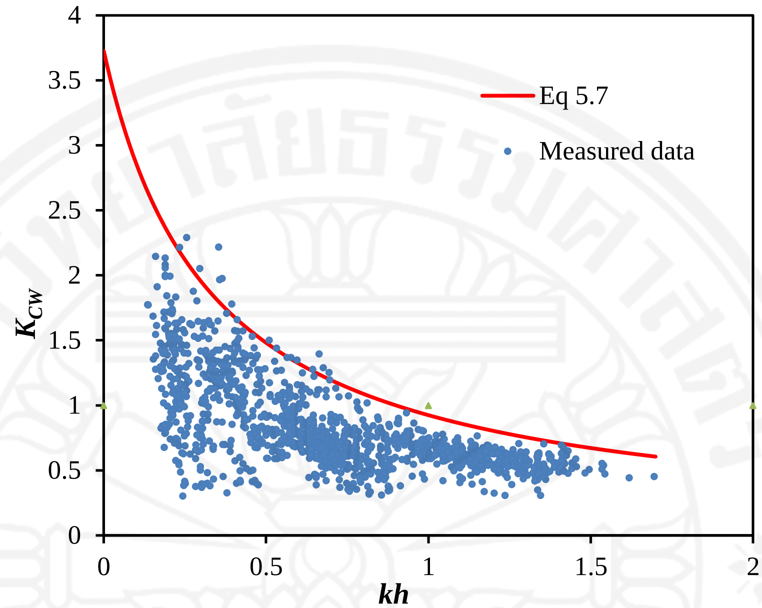


Figure 5.13 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (5.7) with a_1 = 1.23 and b_1 = 0.33.

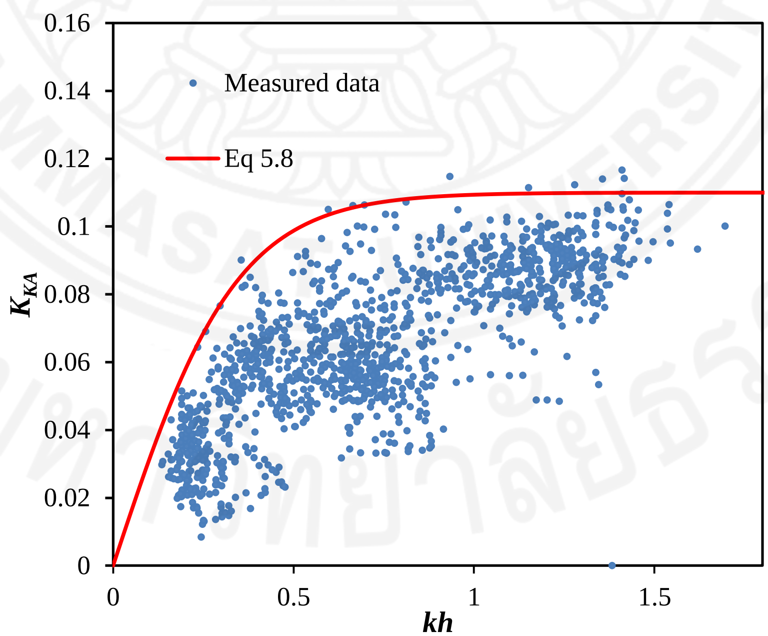


Figure 5.14 Relationship between K_{KA} and kh (measured data from Kraus and Smith, 1994). Solid line is the Eq. (5.8) with a_3 = 0.11 and b_3 = 2.92.

❖ ***$H_{1/10}$***

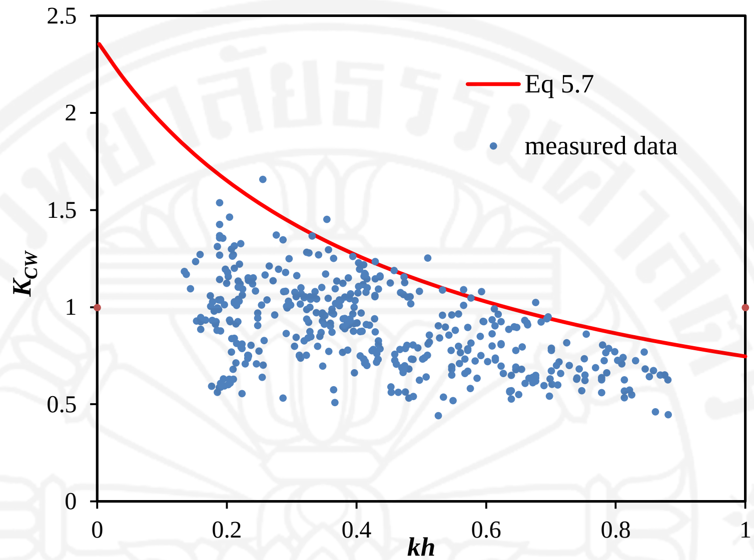


Figure 5.15 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (5.7) with a_1 = 1. 09 and b_1 = 0.46.

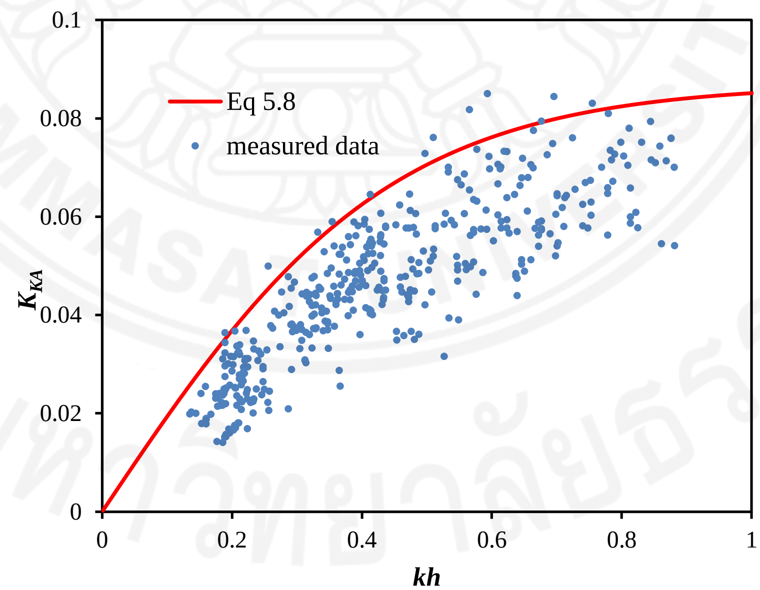


Figure 5.16 Relationship between K_{KA} and kh (measured data from Kraus and Smith, 1994). Solid line is the Eq. (5.8) with a_3 = 0.087 and b_3 = 2.26.

❖ ***$H_{1/3}$***

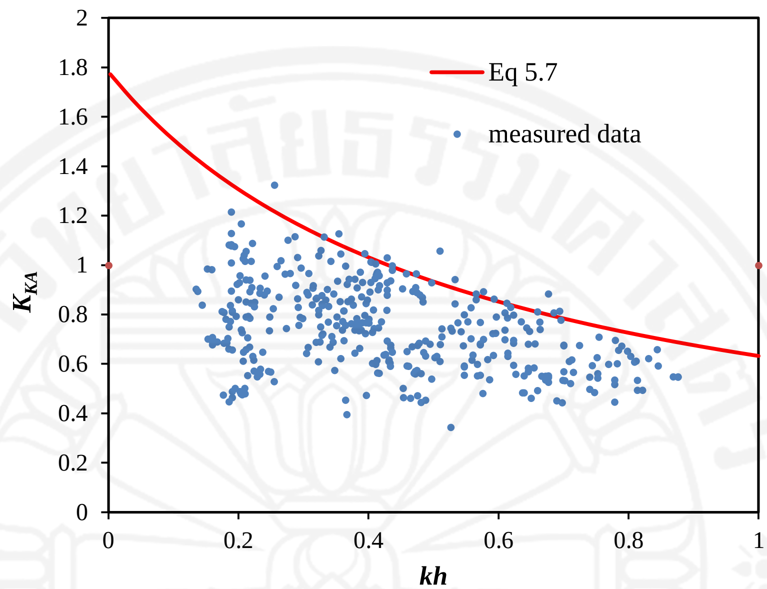


Figure 5.17 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (5.7) with a_1 = 0.98 and b_1 = 0.55.

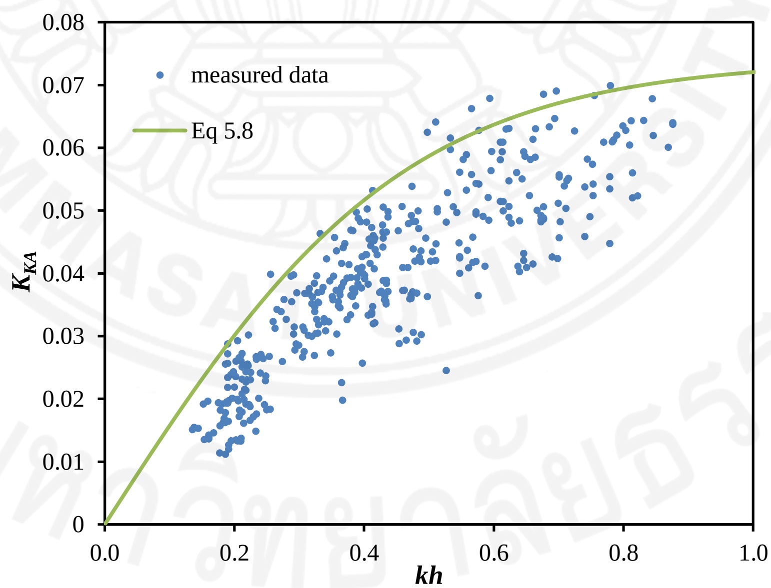


Figure 5.18 Relationship between K_{KA} and kh (measured data from Kraus and Smith, 1994). Solid line is the Eq. (5.8) with a_3 = 0.074 and b_3 = 2.16.

❖ H_{mo}

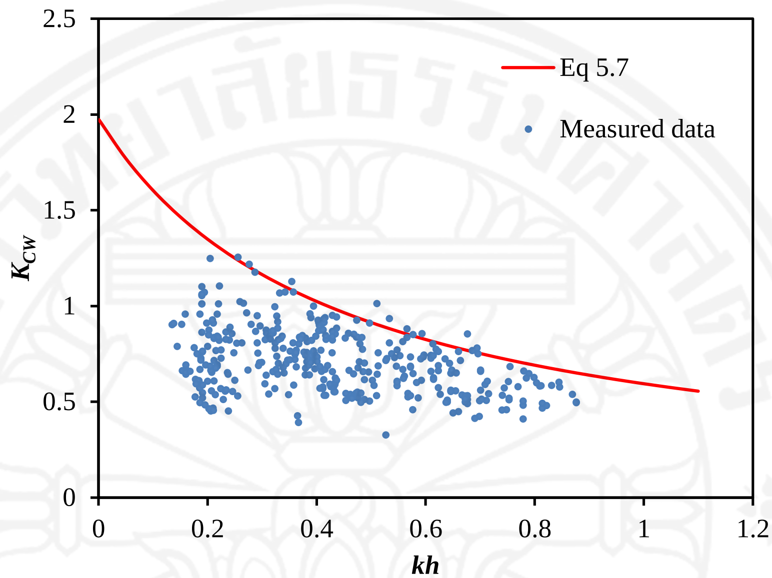


Figure 5.19 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (5.7) with a_1 = 0.85 and b_1 = 0.43.

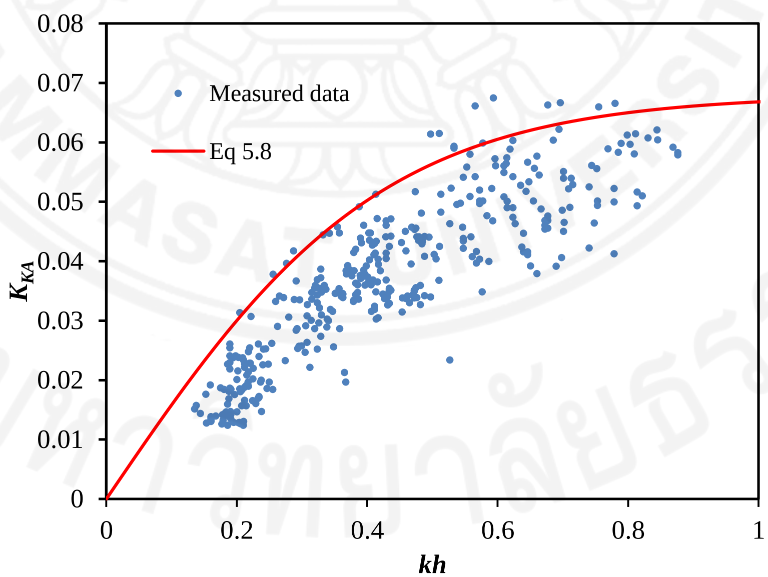


Figure 5.20 Relationship between K_{KA} and kh (measured data from Kraus and Smith, 1994). Solid line is the Eq. (5.8) with a_3 = 0.068 and b_3 = 2.37.

❖ H_{rms}

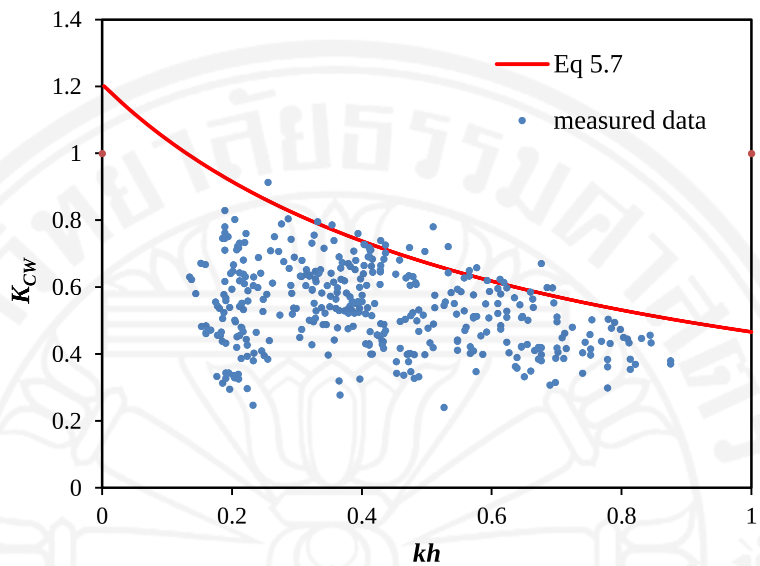


Figure 5.21 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (5.7) with a_1 = 0.76 and b_1 = 0.63.

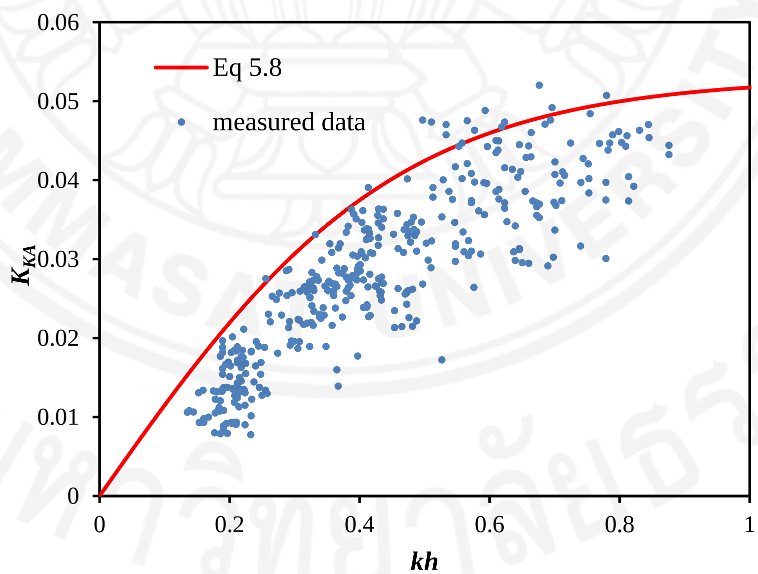


Figure 5.22 Relationship between K_{KA} and kh (measured data from Kraus and Smith, 1994). Solid line is the Eq. (5.8) with a_3 = 0.053 and b_3 = 2.2.

❖ ***H**$_{mean}$*

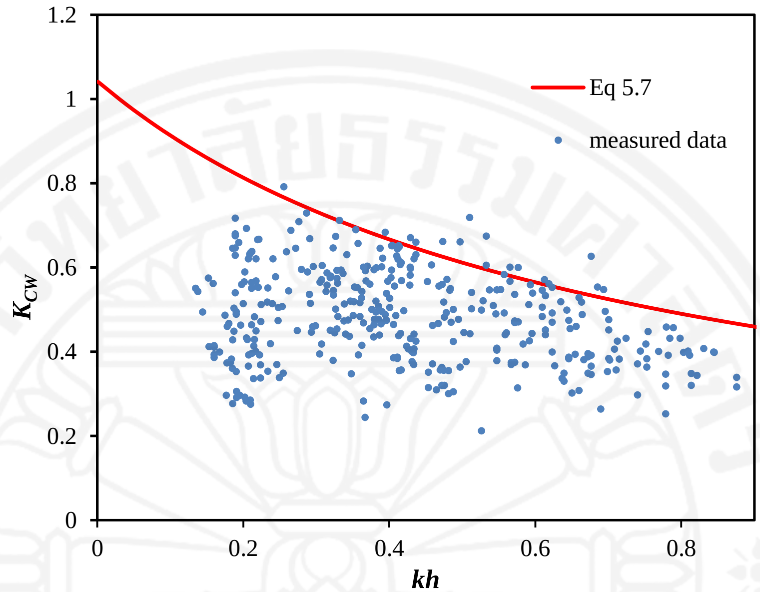


Figure 5.23 Relationship between K_{CW} and kh (measured data from Kraus and Smith 1994). Solid line is the Eq. (5.7) with a_1 = 0.74 and b_1 = 0.71.

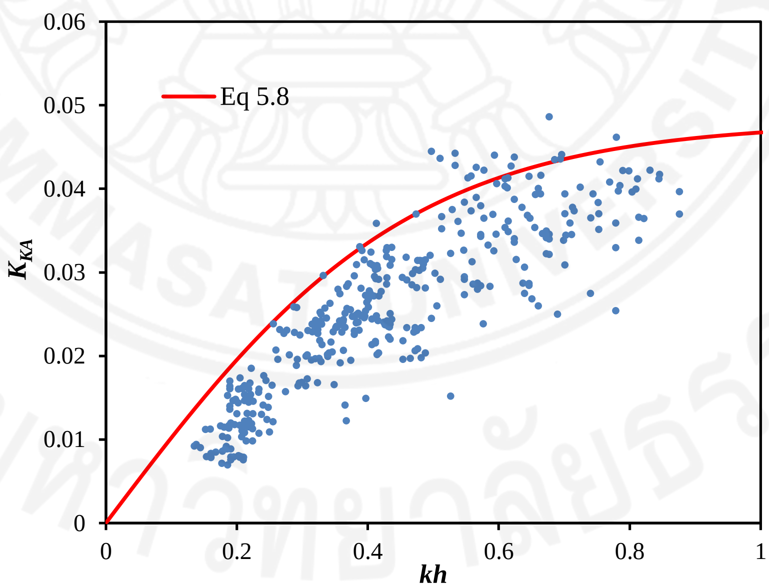


Figure 5.24 Relationship between K_{KA} and kh (measured data from Kraus and Smith, 1994). Solid line is the Eq. (5.8) with a_3 = 0.048 and b_3 = 2.16.

5.5 Verification of general formula of $H_{rep,max}$

In order to confirm the performance of two common formulas. The computations of two common formulas are carried out with all collected data shown in Table 3.4. The Table 5.4 show the errors of the modified formulas for 4 groups of relative depth (*kh*) and all cases.

Table 5.4 The mean percentage error (*MPE*) of two common formulas comparing with irregular waves data shown in Table 3.2

H_{rep}	Formulas	$kh < 0.1\pi$	$0.1\pi < kh < 0.2\pi$	$0.2\pi < kh < 0.3\pi$	$0.3\pi < kh$	MPE_{avg}
H_{max}	MCW69	0.78	0.05	0.01	0.08	0.14
	MKA91	1.21	0.04	0.00	0.33	0.23
$H_{1/10}$	MCW69	0.12	0.01	0.00	0.00	0.02
	MKA91	0.34	0.01	0.00	0.00	0.05
$H_{1/3}$	MCW69	0.25	0.03	0.01	0.01	0.05
	MKA91	0.42	0.02	0.00	0.01	0.06
H_{mo}	MCW69	0.25	0.02	0.02	0.00	0.07
	MKA91	0.59	0.01	0.03	0.00	0.15
H_{rms}	MCW69	0.51	0.08	0.01	0.01	0.11
	MKA91	0.57	0.05	0.01	0.02	0.10
H_{mean}	MCW69	0.07	0.13	0.04	0.01	0.06
	MKA91	0.14	0.16	0.05	0.03	0.09

Chapter 6

Conclusions

For maximum wave height H_{max}, a total of 1210 cases (including 17881 data points) from 31 sources of published experimental results were used to examine the applicability of 14 existing breaker height formulas on predicting the maximum possible wave height of regular and irregular waves. The measured wave heights in offshore and surf zones (excluding those in swash zone) were used in the examination. The experimental data cover a wide range of bottom slope conditions. The examination results are presented in terms of mean percent error (*MPE*). It was found that the errors of the existing formulas on regular and irregular waves have the same tendency, but the errors on irregular waves tend to be larger than that on regular waves. All existing formulas give good predictions in the region of $kh \geq 0.3\pi$, but give considerable underestimation of the maximum possible wave height in the shallow water region ($kh < 0.1\pi$). The errors (*MPE*) of the existing formulas tend to increase with the decreasing of relative depth (kh). Comparing among the existing formulas, the formula of CW69 gives the best overall predictions ($MPE = 2.66\%$). Although the formula of CW69 gives the best overall prediction, the formula is not suitable for using in general cases. Hence, the top three formulas (CW69, KA91, and GO70) are modified by including the new form of relative depth into the formulas for improving the accuracy. The modified formulas give better estimation than those of existing formulas ($0.08 \leq MPE \leq 0.4\%$). Considering accuracy and simplicity of the modified formulas, the formula MCW69 and MKA91 are recommended.

For other representative wave heights, formulas of Collin and Weir (1969) and Kaimphuis (1991) were selected to apply for predicting the maximum possible representative wave heights. The general formulas for computing representative wave heights have never been proposed by any previous researchers. These two formulas can be used as general formulas to estimate for all types of the maximum possible representative wave heights.

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