A STUDY ON THE STABILITY OF C.C. BLOCKS AS REVETMENT MATERIAL AGAINST WAVE ATTACK

by

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(Roll No. – 040016020P)

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Water Resources Engineering

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CERTIFICATE

The thesis titled "A Study on the Stability of C.C. Blocks as Revetment Material against Wave Attack", submitted by Biswajit Nandi, Roll No: 040016020P, session: April 2000, has been accepted as satisfactory in partial fulfillment of the requirements for the degree of Master of Science in Water Resources Engineering.

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This is declared that this work entitled "A Study on the Stability of C.C. Blocks as Revetment Material against Wave Attack" has been done by the author under supervision of Dr. Md. Khorshed Alam, Professor, Department of Water Resources Engineering, Bangladesh University of Engineering and Technology (BUET), Dhaka. This is also declared that this or any part of it has not been submitted elsewhere for the award of any degree or diploma.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF PHOTOGRAPHS

LIST OF NOTATIONS

Symbols	Description	unit
α	slope of structure	-
β	angle of wave attack	-
Δ	relative density ($= (\rho_s - \rho_w)/\rho_w$)	-
ρ	density of block	kg/m^3
ξ	surf parameter (= $\tan\alpha/\sqrt{s}$)	-
τ	shear stress	N/m^2
Λ	Leakage length	m
ρ_s , ρ_r	density of stone, rock or blcok	kg/m^3
ρ_w	density of water	kg/m^3
B	width of berm	m
d	depth of water	m
D	thickness of block	m
d_b	water depth related to breaking limit	m
d_h	water level above berm	m
e	rotational displacement of paddle	m
f	transitional displacement of paddle	m
F	fetch	m
g	acceleration of gravity	m/s^2
H	regular wave height	m
h	water depth	m
H_0	wave height at deep water	m
$H_{1\%}$	wave height exceeded by 1% of the waves.	m
H_b	wave height related to breaking limit	m
H_{cr}	critical wave height (regular)	m
H_i	incident wave height	m

H_r	reflected wave height	m
H_s	significant wave height	m
H_{scr}	critical wave height (irregular)	m
$H_{x\%}$	wave height exceeded by x% of the waves.	m
K_D	stability coefficient in Hudson's formula	-
K_r	coefficient of reflection ($K_r = H_r / H_i$)	*m
L	wave length in shallow water	m
L_b	wave length related to breaking limit	m
L_o	wave length in deep water ($= gT^2/2\pi$)	m
L_{slope}	the horizontal length between the two points on the slope 1.5 H_s above	
N	number of waves hitting the structure	
N_{od}	Damage level defined by Van der Meer	
P, p	permeability in Ven der Meer formula	-
R_u	wave run up for regular waves	m
$R_{u2\%}$	wave run up for irregular waves (which is exceeded by two per cent of the incoming waves)	m
s	wave steepness (= H/L)	-
S	damage level in Van der Meer formula	-
t	time	sec
T	regular wave period	sec
T_p	peak wave period	sec
T_s	significant wave period	sec
u	wind velocity	m/s
W	weight of block	N

LIST OF ABBREVIATION

BMD	Bangladesh Meteorological Department
BUET	Bangladesh University of Engineering and Technology
BWDB	Bangladesh Water Development Board
FAP	Flood Action Plan
FPCO	Flood Plan and Co-ordination Organsation
MCSP	Multipurpose Cyclone Shelter Program
SWL	Still Water Level

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ABSTRACT

This study was done to check the validity of the formulae widely used for design of cement concrete (c.c.) block against wave erosion. Experiments have been carried out in a 21.34 m long and 0.76 m wide glass sided laboratory flume of the Hydraulic and River Engineering Laboratory under the Department of Water Resources Engineering, BUET, Dhaka.

One of the main tasks of the present study was to set up the equipment and calibrate the measuring instruments properly for useful collection of representative data. For this purpose a major part of the time had to spend specially for separating reflection component of the incident wave and to set up data acquisition system with high scanning rate.

Experiments have been done with two types of c.c. block sizes for 1:1.5 and 1:3 bank slope. The prototype size of blocks were 400mm x 400mm x 250mm and 350 mm x 350mm x 200mm. Considering available height of the flume the prototype parameters like water depth, wave height, block sizes were scaled down to 1:20(prototype: laboratory standard) scale ratio. Preliminary analysis showed that laboratory representation of the prototype data with this scale ratio represented stronger structure than actual ones. This is because that the clamping and friction between blocks and the friction between blocks and geotextiles cannot be scaled down in the laboratory. This led the subsequent runs to conduct with 1:10 scale ratio.

Comparing the laboratory results with the Pilarczyk(1990) formula the agreement was found satisfactory. Though the clamping and friction between blocks could not be represented even with 1:10 scale the results were found well within the design boundary defined by Schiereck(2001) on theoretical basis and 1:1 prototype experimental results. It was not fully appropriate to compare Hudson (1961) and Van der Meer (1988) formula with the placed blocks system. But comparison have also been made in order to get an idea about the influence of the parameters suggested by them.

Analysis for wave run up has also been done. The experimental runs carried out wiht 1:10 scale ratio were compared with Pilarczyk (1998) formula of wave run up and

the agreement was found very satisfactory for smaller surf parameter (ξ<2.0). For the runs with 1:20 scale ratio the agreement was far way from satisfactory.

Influence of wave frequency (wave period) in selection of c.c. blocks has also been studied by incorporating wave period in the parameters of Hudson formula and through dimensional analysis a power equation was developed.

Chapter 1
Introduction

1.1 Background

Waves are generated and developed in seas, rivers, lakes, haors, beels or any other large mass of water mainly due to wind. It may be produced due to movement of marine vessels, explosion of earthquakes also. Worldwide erosion due to waves is a problem in the conservation of beaches and shoreline, maintenance of dock and harbour, reclamation of land from sea for airport and industry, roads and dams, human settlements etc.

There are two basic types of wave erosion control methods. These are vegetative and structural measures.

Vegetative method involves plantation of trees and woody shrub and the mechanism to check wave erosion is the soil binding properties with large root systems and the damping of wave energy along its propagation. It is widely used in shore, stream bank and around human settlement. Sometimes rocks and boulders are also used to make the system more effective. This method is environment friendly, but needs a certain time to be functional. It is suitable when land is cheap and available for reasonable width and length.

Structural measures are immediate measures and they include sea wall, gravity wall, bulkheads, groins, jetties, revetments and breakwaters. Seawall, groins, jetties are massive structure that dissipate full force of waves and are adopted for shoreline protection. Bulkheads, gravity wall are next in size that retains the fill. Breakwaters are the front line defense structure to protect shorelines against wave actions. Revetments are used as a direct protective structure against moderate waves (SPM, 1984). There are various types of revetments such as loose stones and boulders, gunny bags, cement concrete (c.c.) blocks, articulated mattress etc. Now a days, all over the world the use of c.c. blocks have become very popular due to durability, easy construction and ease in quality control. Designers choose c.c. blocks considering cost and functions of revetments also.

In Bangladesh the areas prone to wave erosion are coastal areas and other wetland areas like haor, baor and beel areas. Coastal area lies in the southern part of Bangladesh and is open to sea. Besides waves produced by wind and tides, the costal area is attacked by storm surges. The storm surges have been noted to be some 3m to 6m in height (ESCAP, 1988). Haor and baor areas are located in northwest part of the country whereas beel areas lie in the southeast part of the country. They are flooded in monsoon. Wind generated waves in haor, baor and beel areas have been reported to vary between 1 m and 1.5 m. Since last decade wave erosion in these causing wetland are causing severe damages of villages and human settlements, roads and embankments.

Protective works against wave erosion in coastal region of Bangladesh normally consists of boulders and cement concrete (c.c.) blocks revetment with 1(V):7(H) slope. Multipurpose cyclone shelters have also been built along the borders of the coast line to save life and properties against storm surges. Along the Chittagong belt plantation and afforstation (vegetative measure) have been done against wave erosion and storm surge erosion and has been proved as very much effective.

In haor region construction of earthen mounds with c.c. blocks has become a cost effective social friendly wave protective structure (CARE, 2000). The use of gunny bags could not be successful against wave erosion in Bangladesh. In some places of haor areas gravity walls have been successful protective structure. In other part of Bangladesh the use of c.c. blocks with geotextiles as wave protective structures is also well established. Because it is labour intensive and can be built of local singles, brick chips etc.

The design of c.c. block revetment in Bangladesh is done with Hudson(1961) formula and Pilarczyk(1990) formula for wave protection in Bangladesh (FCPO, 1993). Hudson formula is popular for its simplicity. But it is not known whether this formula has been modified for placed block system.

For placed block system Pilarczyk (1990) formula has been developed from experimental results. Here no research has been done to check the validity of this formula in context of Bangladesh wave climate.

Van der Meer (1988) has come forward to formulate design formula incorporating other parameters. He included damage level and number of waves. But wave time period has not included in this formula. This formula has also been taken to check the validity in Bangladesh situation.

The selection of crest height of embankment is very much important if overtopping is not permitted. Several researchers have investigated wave run up mostly for run up on smooth, impermeable slopes. Hall and Watts (1953) investigated run up of regular waves on impermeable slopes. Dai and Kamel (1961) investigated wave run up on rubble breakwaters. Miche(1944) treated wave run up height as a function of wave steepness. Le Mėhautė (1963) summarized a formula for wave run up analytically. In that formulae wave run up height was function of depth of water, wave height and slope angle. Pilarczyk (1990) considered wave run up as a function of surf parameter. But it is not known whether the wave run up on placed block revetment has been properly investigated or not.

Filter condition is another important parameter in embankment as a wave protective structure. Use of geotextiles is now a days almost a must. Damage development and propagation are also very important from maintenance point of view. An attempt has been made for better understanding of damage development and its propagation in the present study. One of the aims of the present study is to investigate damage development with geotextiles and a comparison has been made without geotextiles.

A scope is also existing to investigate the influence of wave frequency as a parameter in case of stability of c.c. blocks. This parameter has been incorporated in the parameters of simple Hudson formula.

Therefore the problem of wave protective works needs to be studied in detail to extend the existing knowledge for Bangladesh situation.

1.2 Objectives

With the background stated above the specific objectives of this study are as follows:

- To compare the laboratory result with the present day formulae like Pilarczyk, Van der meer and Husdon formual in context of Bangladesh situation and environment.

- To measure wave run-up for different wave heights.
- To investigate the behaviour of cement concrete (c.c.) blocks for different wave heights with geotextiles and to get insight of damage development and propagation.
- To investigate influence of wave frequency in the selection of c.c. blocks.

The Department of Water Resources Engineering recently has undertaken a research project to develop a design manual for "Wave Protection Works in Haor Areas of Bangladesh" under BUET -DUT Linkage Project Phase III. This study emerges as a part of that research program.

1.3 Organization of The Thesis

The subject matter of this thesis report has been arranged in six chapters. First chapter provides a background with rationale the study, objectives and organization of this report. Second chapter provides related definitions to used in coastal engineering field. In third chapter reviews of formulae and previous work in the same laboratory have been provided. The detailed description of the laboratory experiments and data collection techniques have been included in the fourth chapter. In chapter five analysis of data with results have been discussed and this chapter is titles as "Data Analysis, Results and Discussion". Finally "conclusions and recommendations" have been presented in chapter six.

Chapter 2
Definition Terms

2.1 Introduction

Wave characteristics are important in the field of coastal engineering. In case of wave protective structure several terms come forward. Some important terms related to waves and wave load on structure have been discussed here.

2.2 Wave

A "wave" is the generic term for any periodic fluctuation in water height, velocity or pressure. In this study wave means fluctuation of water surface and more specifically wind generated waves are considered.

The technical treatment of wind waves can be divided roughly into three categories,

Generation: The generation of waves by wind is described with relations of the type: $H, T_{characteristic} = f(u_{wind}, d, fetch)$.

Hydrodynamics: Velocities and forces in waves are, of course, important when dealing with erosion and protection. These parameters are described with relations of the type: $u, \rho, \tau = f(H, T, d)$. Hence u, τ and ρ are wind speed, shear stress and density of water respectively. Again H, T and d wave height, wave period and depth of water respectively. The wave generating forces no longer play a role. The dominating characteristic in wave erosion is wave height and period (Schiereck, 2001).

Statistics: The water surface of wind waves is irregular because the driving force, the wind, is turbulent. It is therefore necessary to characterize a wave field by means of statistical parameters. Relations of the type: $p(H) = f(H_{characteristic}$, distribution function) give the probability of a certain wave height in a wave field.

Wave and its analysis with these three issues should have clear-cut function and sometimes lead to confusion. (Schiereck, 2001). Within the scope of this study only generation of wind forced wave has been investigated.

It is noted that the statistical analysis is the part of choosing design wave height at the site. This part is left for the users of this study-result. On the other hand hydrodynamic force analysis is another vast area that was not considered by any numerical model in this study. But the influence of hydrodynamic force in terms of wave height has been observed in the analysis under this study. Available data in wind speed, depth in haor areas and fetch length have been collected to calculate wave height in haor areas and coastal areas. This helps to select prototype and model scale ratio.

2.3 Characteristics of Wave Field

Though in the laboratory regular waves are generated characteristics of wave field in the real life are irregular. Wave height of irregular waves is usually characterized by the value of significant wave height (H_s) occurring at the peak of storm during about 15 minutes or half an hour. Significant wave height is the average height of the one third part of the highest waves.

In general $H_{x\%}$ is a characteristic waves in irregular wave field and it can be stated as $H_{x\%}$ = wave height exceeded by x% of the waves. Therefore other characteristic waves are defined as in the following way;

$$H_{1\%} = 1.52H_s,$$

$$H_{2\%} = 1.4H_s,$$

$$H_{5\%} = 1.22H_s,$$

$$H_{13.5\%} = H_s,$$

$$H_{50\%} = 0.59H_s, \text{(median wave height)}$$

Other characteristic of waves are peak wave period, T_p . Wave period at peak of a spectrum is called peak wave period which is 1.1 to 1.3 times of average wave period.

Deep-water wavelength is the length at deepwater which is based on T_p. Mathematically deep water wave length is expressed as $L_0 = \frac{gT_p^2}{2\pi}$, where g = acceleration of gravity.

Wave steepness is another important characteristic of wave filed. It is defined as $s = \frac{H_s}{L_0}$. Again Irribarren number or surf parameter is of crucial importance in all kinds of problems in shore protective works. It is defines as $\xi = \frac{\tan\alpha}{\sqrt{H_s / L_0}}$ where α is the angle of the slope and $\frac{H_s}{L_o}$ is already defined as the wave steepness.

For breaking waves water depth is greater than three times of significant wave height $(d \geq 3H_s)$.

For comparison of tests with regular and irregular waves at large scale $H_{cr}/H_{scr} = 1.4$ is used, where H_{cr} is regular waves causing initial damage and H_{scr} is the significant waves causing initial damage.

But T_p (for irregular waves) $\approx$T (for regular waves). Thus regular surf parameter is obtained by multiplying irregular surf parameter by 1.4 times.

2.4 Hydraulic Load on Structure

The hydraulic loads on structures like sea wall, revetments, groynes, dam are due to the following causes.

i) flows

ii) water level and

iii) waves.

Hydraulic load due to flows

It is an important load on coastal structure. But the study does not allow this load on the structure. Therefore the hydraulic load due to flows are not considered here.

Hydraulic load due to water level

Water level in front of the structure may cause seepage pressure if other side has differential waver level. Water becomes as an imposed load on the structure. During design of hydraulic structure this load must be considered.

Hydraulic load due to waves

Wave is an important and dominating force in design of coastal structures. Waves are generated in large mass of water body like in sea, haor, baor , lake , river etc. Among other causes of wave generation wind, earthquake, ship etc are important. Again wind generated waves are frequently observed in all wetlands and seas if enough fetch length, wind speed etc are available.

It is very complex to define a wave load on structure. The behaviour of wave load is not static. It imposes somewhere dynamic load and other where quasi-static load. It is well known that the pressure under a wave increase and decreases with the wave cycle as long as the water keeps in touch with the point where the pressure is considered. This is often called the quasi-static wave load, as shown point 1 in Figure 2.1

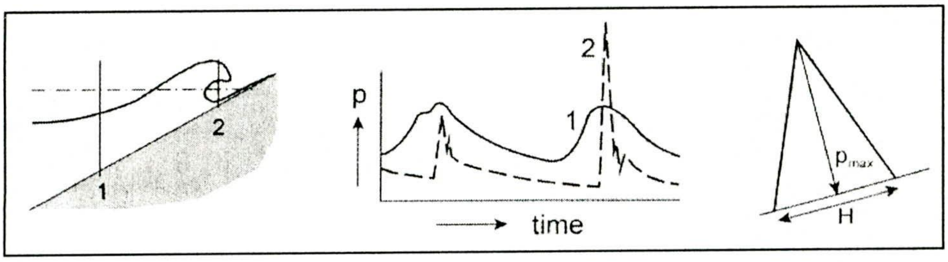


Figure 2.1: Wave Impact on Slope (Schiereck, 2001)

When water from the wave collides with the surface, a very short, very high, impact pressure will occur, This is called the dynamic wave load, wave impact or wave shock, shown in point 2 of Figure 2.1.

$$p_{\max 50\%} \approx 8\rho_w g H_s \tan\alpha \qquad\qquad p_{\max 0.1\%} \approx 16\rho_w g H_s \tan\alpha \qquad (2.\ 1)$$

in which $P_{max\ 0.1\%}$ is the maximum pressure exceeded by 1 in 1000 waves. This expression gives values several times higher than follows from the quasi-static pressures in the wave itself. The shape of the impact pressure distribution is assumed to be a triangle with H as base length. The conclusion is that wave height is the dominating characteristic to determine wave load on the structure.

Hydraulic boundary conditions of water level and waves arise due to wind (storm). All of these causes load on a structure and shown in Figure 2.2.

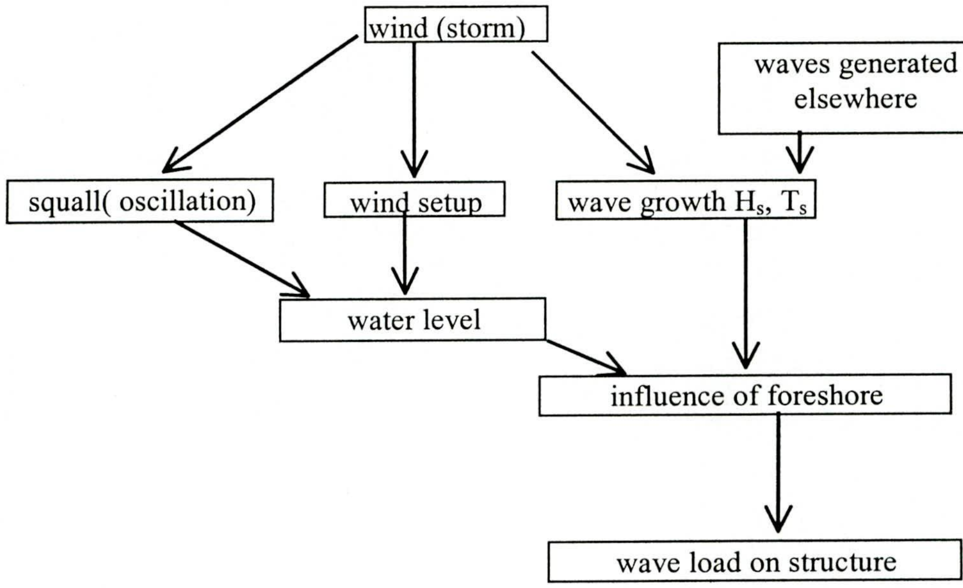


Figure 2.2: Hydraulic load on a structure as a result of a wind (Schiereck, 2001)

Wind generates waves in water body and progressively come towards structure. Thus wave-growth is observed in the seas and other wetlands. Loads due to waves imposed on a structure in a different way also. Besides progressive waves there are squall (oscillation) in water body. This type of oscillation and wind setup changes water level dynamically. Thus waves come to the structure with energy and release it to the structure and cause erosion.

2.5 Process Affecting Hydraulic Loads on a Structure

Wind generated waves, tide and wave set up cause rise in water level. The water level causes force on the structure. Besides forces due to water level waves act as dynamic force on structure. However there are several causes that affect hydraulic loads on the structure. These are shoaling, refraction, diffraction and breaking and shown in Figure 2.3.

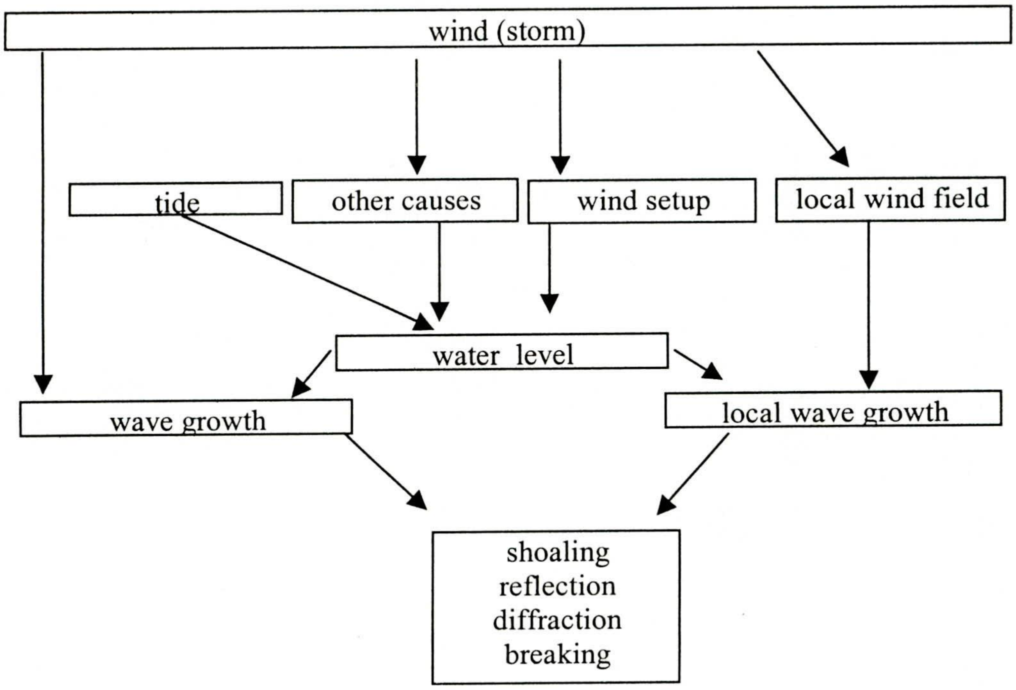


Figure 2.3: Processing affecting hydraulic load at a structure

2.5.1 Shoaling Effect

Changes of water level cause changes in wave height. In deep water wave heights are called as deep water wave height when $\frac{d}{L_o} \geq 0.5$. Here d = depth of water and L_o = wave length. But at $\frac{d}{L_o} \leq 0.2$, the waves are called shallow water waves.

2.5.2 Refraction

If waves propagation not in right angles to the depth contour refraction occurs. Refraction is important for several reasons. It determines wave heights in any particular depth. It contributes a general description of bottom topography.

There are several methods to determine refraction of waves. SPM (1984) is available for calculating refraction using nomograms. Other graphical and analytical methods are also available to calculate refractions.

2.5.3 Breaking Waves

It is well known that waves break when they reach a critical state. This critical state is its steepness and strong and sensitive influence of bed configuration also.

Breaker Types

Breaking waves can be classified in to four types on the way in which they break (Patrick and Wiegel, 1955). and (Wiegel, 1964):

(i) Spilling breaker: Spilling breakers break gradually and are characterized by white water at the crest. The limiting wave shape is not so unsymmetrical as in the case of the plunging breaker. The spilling breaker is characterized by the appearance of " white water" at the crest. The wave generally breaks gradually and turbulent water spills down front face of the wave.

(ii) Plunging breaker: This shows a very unsymmetrical profile with a steeper front face compared to the back surface. The crest curls over a large air-pocket. Air-entrained horizontal roller or vortex and splash usually follow.

(iii) Surging breaker: The wave peaks up as if to break in the manner of the plunging breaker, but when the base of the wave surges up the beach face with the resultant disappearance of the collapsing wave crest.

(iv) Collapsing breaker: This type of breaker is defined by Galvin (1968). The collapsing breaker occurs over the lower half of the wave. Minimal air-pockets and usually no splash-up follow. Bubbles and foam are formed.

For different values of ξ waves break in a completely different ways shown as follows.

spilling breaker : $0.4>\xi$

plunging breaker : $0.4<\xi<2.0$

surging breaker : $2.0<\xi$

2.6 Wave Run-up for Regular waves

Run-up is defined as the maximum water level on a slope height during a wave period. This is defined relative to still water level.

Prediction of wave run-up may be based on simple empirical equations. Those equations are developed by model test results or numerical models of wave structure interaction.

For breaking waves($\xi < 2.5 - 3.0$) on smooth slopes, Pilarczyk(1990)'s formula gives :

$$\frac{Ru}{H} = \xi \qquad (2.2)$$

Here R_u is wave run up, H is regular wave height and ξ is the surf parameter earlier.

Wave run up of regular wave height shows directly proportional to surf parameter and slope is here one. Depending on surface roughness of armour layer this slope may vary.

Riprap slopes give values which are about 50% lower. Run up appears to be maximum around $\xi = 2.5 - 3.0$, which means just at the transition between breaking and non-breaking.

2.7 Wave Run-up for Irregular waves

Wave run-up is often indicated by $R_{u2\%}$. This is the run-up level, vertically measured with respect to the (adjusted) still water level (SWL), which is exceeded by two per cent of the incoming waves. Note that the number of exceeded waves is here related to the number of incoming waves and not to the number of run-up levels.

The relative run-up is given by $R_{u2\%}$/Hs, with Hs the significant wave height, This H_s is the significant wave height at the toe of the structure. The relative run-up is usually given as a function of surf similarity parameter or breaker parameter(ξ).The wave steepness is a fictitious or computation quantity, especially meant to describe the influence of a wave period.

With ξ_0 < 2 -2.5 the waves will tend to break on the dike or seawall slope. This is mostly the case with slopes of 1 :3 or milder. For larger values of ξ_0 the waves do not break on the slope any longer.

The general design formula that can be applied for wave run-up on dikes is given by:

$$\frac{R_{u2\%}}{H_s} = 1.6\gamma_b\gamma_f\gamma_\beta\xi_0 \qquad \text{with a maximum of } 3.2\gamma_f\gamma_\beta \qquad \textbf{(2. 3)}$$

where: γ_b = reduction factor for a berm,

γ_f = reduction factor for slope roughness and

γ_β = reduction factor for oblique wave attack.

The formula is valid for the range $0.5 < \gamma_b\xi_o < 4$ or 5. The relative wave run-up $R_{u2\%}/H_s$ depends on the breaker parameter ξ_o and on three reduction factors, namely: for berms, roughness on the slope and for oblique wave attack.

2.7.1 Reduction Factors on Wave Run-Up

Above equations, described as general formula on wave run-up, include the effects of a berm, friction on the slope, oblique wave attack and a wall on the slope. These effects will be described in the following paragraphs.

Definition of the average slope angle

Research is very often performed with nice straight slopes and the definition of tanα is then obvious. In practice, however, a dike slope may consist of various more or less straight parts and the definition of the slope angle needs to be more precisely defined. The slope angle becomes average slope angle. Figure 2.4 gives the definition of slope (Pilarczyk et.al, 1998).

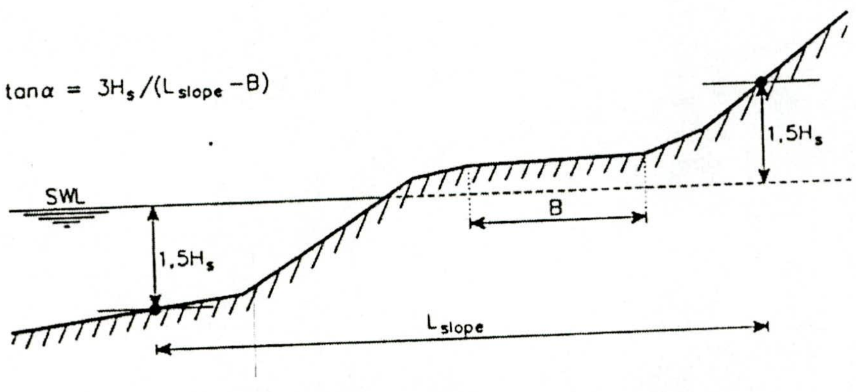


Figure 2. 4: Determination of average slope angle (Pilarczyk, et al, 1988)

The wave action is concentrated on a certain part of the slope around the water level. Examination of many tests showed that the part 1.5 Hs above and below the water line is the governing part (Pilarczyk et.al, 1998). As berms are treated separately the berm width should be omitted from the definition of the average slope. The average slope is then defined as:

$$\tan\alpha = 3H_s/(L_{slope} - B) \tag{2. 4}$$

where L_{slope} = the horizontal length between the two points on the slope 1.5 H_s above and below the water line, and B = the berm width.

Reduction factor γ_b for a berm

A berm is defined as a flat part in a slope profile with a slope not steeper than 1: 15. The berm itself is described by its berm width, B (shown in Figure 2.4), and by its location with respect to the still water level, d_h. This depth parameter is the vertical distance between the still water level and the middle of the berm. A berm at the still water level gives d_h=0

Noe reduction factor form berm can be described by the following equation:

$$\gamma_b = 1 - \frac{B}{L_{berm}}\left(1 - 0.5\left(\frac{d_h}{H_s}\right)^2\right)$$

with $0.6 \leq \gamma_b \leq 1.0$ and $-1.0 \leq d_h/H_s \leq 1.0$

Between d_h = 1 H_s and d_h = 2 H_s the reduction factor γ_b increases linearly to γ_b = 1 (the influence of the berm reduces linearly to zero). With a high berm the influence also decreases linearly from $\gamma_b = 1.0 - 0.5B/L_{berm}$ at $d_h/H_s = -1.0H_s$ to $\gamma_b = 1$ if $R_{u2\%}$ is reached on the down slope.

Reduction factor γ_f for roughness on the slope

The influence of a kind of roughness on the slope is given by the reduction factor γ_f. Reduction factors for various types of revetments have been published earlier. The origin of these factors dates back to Russian investigations performed in the 1950' s with regular waves(Pilarczyl et al,1998). A table on these factors was further developed in TAW (1974) and published in several international manuals. New studies, often large-scale, and conducted with random waves have led to a new table (Table 2.1) of reduction factors for rough slopes.

The reduction factors in Table 2.1 apply for $\gamma_b \xi_o < 3$. Above $\gamma_b \xi_o = 3$, the reduction factor increases linearly to 1 at $\gamma_b \xi_{op} = 5$. The reduction factors in Table 2.1 apply if the part between 0.25 $R_{u2\%, smooth}$ below and 0.5 $R_{u2\%, smooth}$ above the still water level is covered with roughness. The extension "smooth" means the wave run-up on a smooth slope. If the coverage is less, the reduction factor has to be reduced.

Table 2. 1: Reduction factors γ_f for a rough slope (source: Pilarczyk, et al. 1998)

Type of Slope	Reduction Factor γ_f
Smooth, concrete, asphalt	1.0
Closed, smooth, block revetment	1.0
Grass (3 cm)	0.95
Block revetment (basalt, basalton)	0.9
1 rubble layer (H_s/D=1.5-3)	0.6
¼ of placed block revetment (0.5*0.5 m^2) 9 cm above slope	0.75

Run-up formulae for rock slopes with a double layer of rock have been given by Van der Meer and Stam (1992). After some modification from mean period to peak period the equations become:

$R_{u2\%} / H_s = 0.88\xi_o$ for $\xi_o < 1.5$ and (2. 5)

$R_{u2\%} / H_s = 1.1\xi_o^{0.46}$ for $\xi_o > 1.5$ (2. 6)

One can use these formulae to calculate wave run-up for rock slopes.

It is possible that roughness is only present on a small part of the slope. First of all, it showed that roughness solely below the still water level (and a smooth slope above) does have any influence. If also roughness above the still water level is present an average weighing can be done over the area 0.25 $R_{u2\%, smooth}$ below and 0.5 $R_{u2\%, smooth}$ above the water level. The part to be taken into account below SWL may never exceed the part above SWL. Suppose within the given area three different slope sections exist with lengths of respectively l_1, l_2 and l_3 and reduction factors of $\gamma_{f,1}$, $\gamma_{f,2}$ and $\gamma_{f,3}$. The average reduction factor for roughness become then:

$$\gamma_f = \frac{\gamma_{f,1} l_1 + \gamma_{f,2} l_2 + \gamma_{f,3} l_3}{l_1 + l_2 + l_3}$$

Reduction factor γ_β for the angle of wave attack

The angle of the wave attack β is defined as the angle of the propagation direction with respect to the normal of the alignment axis of the dike. Perpendicular wave attack is therefore given by $\beta=0^0$ .The reduction factor for the angle of wave attack is given by γ_β. Until recently few investigations were carried out with obliquely incoming waves but these investigations had been performed with long-crested waves. "Long-crested" means that the length of the wave crest is in principle assumed to be infinite. In investigations with long-crested waves the wave crest is as long as the wave board and the wave crests propagate parallel to one another.

In nature, waves are short-crested. This implies that the wave crests have a certain length and the waves a certain main direction. The individual waves have a direction around this main direction. The extent to which they vary around the main direction (directional spreading) can be described by a spreading value. Only long swell, for example coming from the ocean, has such long crests that it may virtually be called "long-crested". A wave field with strong wind is short crested.

In Van der Meer and Janssen (1995) results of an investigation were described into wave run-up and overtopping where the influence of obliquely incoming waves and directional spreading was studied. Figure 2.5 summarizes these results. The reduction factor γ_β has been set out against the angle of wave attack β.

Long-crested waves with $0° < \beta < 30°$ cause virtually the same wave run-up as with perpendicular attack. Outside of this range, the reduction factor decreases fairly quickly to about 0.6 at $\beta = 60°$. With short-crested waves the angle of wave attack has apparently less influence. This is mainly caused by the fact that within the wave field the individual waves deviate from the main direction β. For run-up with short-crested waves the reduction factor decreases linearly to a certain value at $\beta = 80°$. This is around $\gamma_\beta = 0.8$ for the 2% run-up. So, for wind waves the reduction factor has a minimum of 0.7 -0.8 and not 0.6, as was found for long-crested waves. Since a wave field under storm conditions can be considered to be short-crested, it is recommended that the lines in Figure 2.3 be used for short-crested waves.

For oblique waves, different reduction factors apply to run-up levels. The cause for this is that here the incoming wave energy per unit length of structure is less than that for perpendicular wave attack. The use of the lines given in Figure 2.5 for short-crested waves is recommended and can be described by the following formulae:

For the 2%-wave run-up with short-crested waves:

$$\gamma_\beta = 1 - 0.0022\beta \qquad (\beta \text{ in degree}) \tag{2. 7}$$

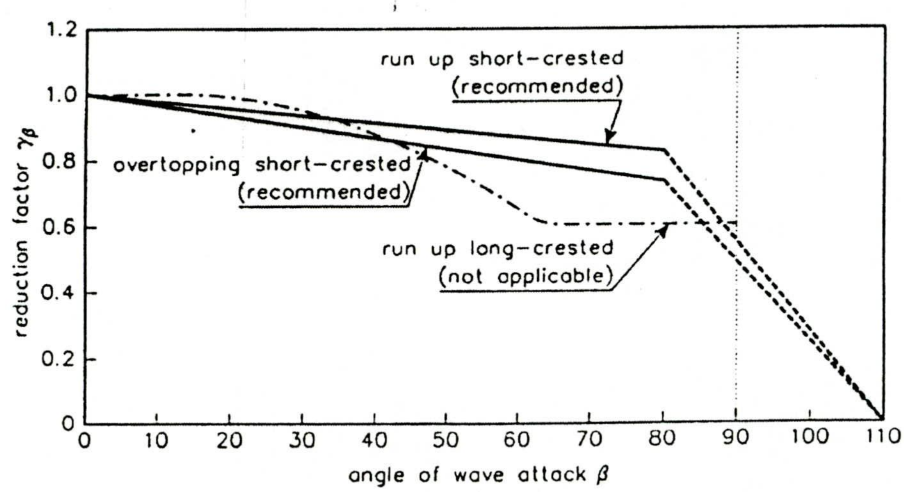


Figure 2.5: The reduction factor for oblique wave attack (Pilarczyk, et al, 1988)

For wave angles with β > 80° the reduction factor will of course rapidly diminish. As at β=90° still some wave run-up can be expected, certainly for short-crested waves, it is fairly arbitrary stated that between β= 80° and 110° the reduction factor linearly decreases to zero.

2.8 Wave Reflection

The wave motion in front of a reflecting structure is mainly determined by the reflection coefficient K_r.

If 100% of the incoming wave energy is reflected, one can safely assume that the reflection coefficient $K_r = H_r / H_i = 1$. This is generally valid for a rigid vertical wall of infinite height. The reflection coefficient for sloping structures, rough or permeable structures, and structures with a limited crest level is smaller. Postma (1989) has investigated the reflection from infinitely high rock slopes. He found a clear influence of the breaker parameter ξ, and of the "permeability" P as defined by Van der Meer.

For a first estimate, Postma(1989) proposes the use of a simple formula:

$$K_r = 0.140\xi_o^{0.73}$$

For a more accurate approach, he gives the formula:

$$K_r = 0.081P^{-0.14}\cot\alpha^{-0.78}\left(\frac{H}{L}\right)^{-0.22}$$

This formula can only be used within the validity range of the various parameters as given below:

$0.1 < P < 0.6$

$1.5 < \cot\alpha < 6$

$0.004 < \sqrt{H/L} < 0.06$

$0.7 < \xi < 8$

$0.1 < K_r < 0.8$

$0.03 < h/L_{op} < 0.3$

$0.09 < H_{si} / h < 0.23$

$2 < H_{si} / D_{n50} < 6$

In the laboratory reflection has been occurred due to slope of the structure only.

2.9 Failure Mechanism

Failure of revetment can occur on the slope by wind-generated waves or on the crest by wave overtopping and over flowing.

Failure can be occurred in several ways.

- Lifting out of one individual block initiate failure of revetments
- Migration of sand silt from base into filter layer leading to subsidence of top layer.
- Sliding down of top layer due to insufficiently stable toe structure
- Geotechnical instability

2.10 Scaling Consideration

Large scale physical model investigations are defined when prototype : model = 1:3 or larger. Then restrictions are observed with respect to building up the structure and wave boundary conditions.

In small-scale physical model investigations many restrictions are observed. Stability can be determined by the way of exception due to the fact that scaling rules for flow are incompatible to those for waves.

Chapter 3
Literature Review

3.1 Introduction

For many years breakwater design was a question of trial and error. It was shortly before World War II that, in an attempt to understand the influence of rock density, Iribarren developed a theoretical model for the stability of stone on a slope under wave attack. Iribarren continued his efforts throughout the years until his final publication on the subject at the PIANC Conference of 1965 in Stockholm.

In the meantime, in the USA, the US Army Corps of Engineers had developed a keen interest in the stability of breakwaters, and long series of experiments were carried out by Hudson at the Waterways Experiment Station in Vicksburg.

In 1988 Van der Meer presented his PhD thesis on "Rock slopes and Gravel Beaches under Wave Attack". In his research a revolution comes in the design of sloping dyke with rock. In this manner many other researchers contribute in this field. Krystian Pilarczyk (1984), Gerrit J. Schiereck (2001) of the Netherlands are now promoting in this field. In the following articles some of the contributions in this field are discussed.

3.2 Iribarren Formula

Iribarren (1938) considered the equilibrium of forces acting on a block placed on a slope. Since the considerations of Iribarren referred to forces, the weight of the block W is introduced as a force, and thus expressed in Newton. It is important to realize that in literature, one finds the block size indicated either by weight or by mass. Although this is confusing, it is the result of a less strict application of the ISO standard (mks system) in the past. When using the formulae it is wise to check whether g is introduced in the formula. In that case, the weight is calculated in N. If g is not present in the formula, the result is the mass of the block in kg.

The forces acting on a unit positioned on a slope at an angle α are (shown in Figure 3.1):

- Weight of the unit (vertical downward)
- Buoyancy of the unit (vertical upward)

- Wave force (parallel to the slope, either upward or downward)
- Frictional resistance (parallel to the slope, either upward or downward, but contrary to the direction of the wave force)

Iribarren resolved these forces into vectors normal and parallel to the slope. Loss of stability occurs if the friction is insufficient to neutralize the other forces parallel to the slope.

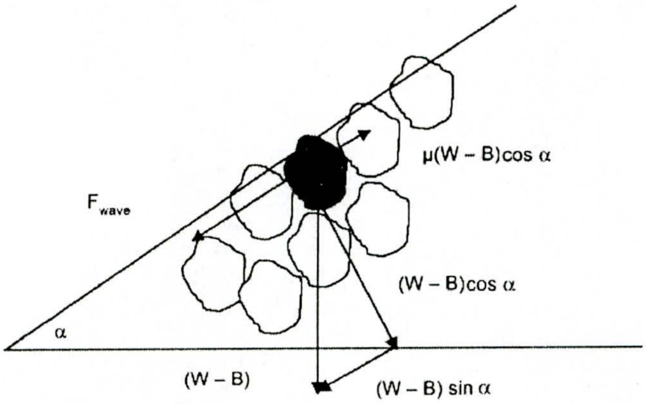


Figure 3. 1: Equilibrium after Iribarren (down rush)

Iribarren assumed a set of simple relations between F_{wave} , D_n , H,ρ and g as follows:

$$F_{wave} = r_w g D_n^2 H \quad \textbf{(3. 1)}$$

$$W - B = (\rho_r - \rho_w) g D_n^3 \quad \textbf{(3. 2)}$$

and

$$W = \rho_r g D_n^3 \quad \textbf{(3. 3)}$$

It must be mentioned here that these relations may be criticized, since they are too simple. It must be expected that the shape of the block and the period of the wave play a role. Furthermore, the relation between the wave force and the wave height and stone size indicate the dominance of drag forces, whereas acceleration forces are neglected.

Nevertheless, considering the equilibrium for down rush along the slope, this leads to a requirement for the block weight:

$$W \geq \frac{N \rho_r g H^3}{\Delta^3 (\mu \cos\alpha - \sin\alpha)^3} \quad \textbf{(3. 4)}$$

For uprush, the formula changes into:

$$W \geq \frac{N\rho_r g H^3}{\Delta^3(\mu\cos\alpha + \sin\alpha)^3} \qquad \textbf{(3. 5)}$$

N is a coefficient that depends, amongst other factors, on the shape of the block, and its value must be derived from model experiments. The friction factor μ can be measured by tilting a container filled with blocks and determining the angle of internal friction.

In Iribarren [1965], recommendations are given for values of N and μ. The most important values are given in Table 3.1. The values of N refer to zero damage.

It must be kept in mind that the coefficient N represents many different influences. At first, it is a function of the damage level defined as "loss of stability". It also includes the effect of the shape of the blocks, but not the internal friction, because this is accounted for in the separate friction coefficient. Finally, it covers all other influences not accounted for in the formula. The friction coefficient μ seems to be on the high side, but is clearly related to the test procedure that Iribarren used. He found a large difference in friction, depending on the number of units in the slope.

Table 3.1: Coefficients for Iribarren formula (SPM, 1984)

type of block	downward stability ($\mu\cos\alpha - \sin\alpha$) 3		upward stability ($\mu\cos\alpha + \sin\alpha$) ^{3}		transition slope between upward and downward stability
	μ	N	μ	N	$\cot\alpha$
rough angular quarry stone	2.38	0.430	2.38	0.849	3.64
cubes	2.84	0.430	2.84	0.918	2.80
tetrapods	3.47	0.656	3.47	1.743	1.77

3.3 Hudson

Since 1942, systematic investigations into the stability of rubble slopes have been performed at the Waterways Experiment Station in Vicksburg, USA. On the basis of these experiments, Hudson (1961a, 1961b) proposed the following expression as the best fit for the complete set of experiments:

$$W \geq \frac{\rho_r g H^3}{\Delta^3 K_D \cot\alpha} \quad (3.6)$$

The formula is applicable for slopes not steeper than 1:1 and not gentler than 1:3. The coefficient K_D represents many different influences, just like the coefficient N in the formula of Iribarren. At first, it is a function of the damage level defined as "loss of stability". It also includes the effect of the shape of the blocks and the internal friction. Finally, sit covers all other influences not accounted for in the formula. Recommended values for K_D have frequently been published and updated by the Corps of Engineers in the Shore Protection Manual(1984). In the 1977 edition of SPM the wave height H is defined as the significant wave height H_s , and the values for the most common types of blocks are given in Table 3.2.

Table 3.2: Recommended K_D values given in SPM 1977

type of block	number of layers (N)	structure trunk K_D breaking wave	structure trunk K_D non breaking wave	structure head K_D breaking wave	structure head K_D non breaking wave
rough angular quarry stone	1	**	2.9	**	2.3
rough angular quarry stone	2	3.5	4.0	2.5*	2.8*
rough angular quarry stone	3	3.9	4.5	3.7*	4.2*
tetrapod	2	7.2	8.3	5.5*	6.1*
dolos	2	22.0	25.0	15.0	16.5*
cube	2	6.8	7.8		5.0

* There is a slight variation of recommended K_D value for different slopes

** Use of single layer is not recommended under breaking waves

In the 1984 edition following a number of dramatic failures of rubble mound breakwaters, the use of H_{10}, the average of the highest 10 % of all waves is recommended. This is equal to 1.27 H_s.

This values of K_D are to be too conservative. A comparison between Table 3.2, and Table 3.3, shows a much more conservative design recommendation in 1984. Not only have the values of K_D been changed, but also the replacement of H_s by H_{10} is quite a dramatic change, certainly if one realizes that the wave height appears with a third power in the Hudson formula. In the opinion of many designers this results in too conservative an approach (d'Angremond, 2001).

Hudson defines the K_D value for initial damage: 0-5% of the blocks in the armour layer. He counts the number of blocks from the center of the crest down the outer slope to a level equal to the "no-damage wave height", $H_{D=0}$, below still water level. It is important, however, to know what happens when the wave height is greater than

Table 3.3: Recommended K_D values given in SPM 1984

type of block	number of layers (N)	structure trunk K_D		structure head K_D	
		breaking wave	non breaking wave	breaking wave	non breaking wave
rough angular quarry stone	1	**	2.9	**	2.2
rough angular quarry stone	2	2.0	4.0	1.6*	2.8*
rough angular quarry stone	3	2.2	4.5	2.1*	4.2*
tetrapod	2	7.0	8.0	4.5*	5.5*
dolos	2	15.8	31.8	8.0	16.0*
cube	2	6.5	7.5		5.0
akmon	2	8	9	n.a.	n.a.
Accropod " (1:1.33)		12	15		

* There is a slight variation of recommended K_D value for different slopes

** Use of single layer is not recommended under breaking waves

the zero damage wave height, in other words, when the structure is overloaded. The Shore Protection Manual gives data for various types of armour units and various levels of over-loading. These data are summarized in Table 3.4. From this table, it can easily be seen that traditional rubble mounds have an inherent safety coefficient because of the fact that complete failure occurs only at 50% overloading. This safety margin is considerably smaller when concrete armour units are used instead of quarry stone.

It is noted that to be careful, damage due to breaking of units is not included here and damage percentage 30-40 often means total failure

3.3.1 Comparison of Hudson and Iribarren formulae

When comparing the formulae of Iribarren and Hudson, the difference appears to be large. The influences of wave-height, rock density and relative density are equal. The coefficients are different, but can easily be compared. The main difference occurs in

Table 3.4: Damage due to over-loading (d'Angeremond , 2001)

unit		Damage (D) in percent						
		0-5	5-10	10-15	15-20	20-30	30-40	40-50
quarry stone smooth	H/H_{D-0}	1.00	1.08	1.14	1.20	1.29	1.41	1.54
quarry stone rough	H/H_{D-0}	1.00	1.08	1.19	1.27	1.37	1.47	1.56
tetrapod	H/H_{D-0}	1.00	1.09	1.17	1.24	1.32	1.41	1.50
dolos	H/H_{D-0}	1.00	1.10	1.14	1.17	1.20	1.24	1.27

the influence of the slope. A comparison of the two expressions within the validity area of the Hudson formula (1.5 < cot α < 4) reveals that the correct choice of coefficients leads to a minor difference between the two formulae only. It is evident that for very steep slopes (close to the angle of natural repose) Hudson cannot give a reliable result. It is also likely that for very gentle slopes waves will tend to transport material up the slope, a factor that was not considered by Hudson at all. This becomes clearer when one takes the third root from both formulae. The stability expression then changes to:

Hudson: $$\frac{H}{\Delta D} = \sqrt[3]{K_D \cot\alpha} \qquad (3.7)$$

Iribarren: $$\frac{H}{\Delta D} = (\mu \cos\alpha \mp \sin\alpha) N^{-1/3} \qquad (3.8)$$

The coefficients in the formula are sorts of waste bins for all kind of unknown variables and unaccounted irregularities in the model investigations. (d'Angremond,2001). However the variables brought together in the coefficients K_D and N are:

- Shape of the blocks
- Layer thickness of the outer ("armour") layer
- Manner of placing the blocks
- Roughness and interlocking of the blocks
- Type of wave attack
- Head or trunk section of the breakwater
- Angle of incidence of wave attack

- Size and porosity of the underlying material
- Crest level (overtopping)
- Crest type
- Wave period
- Shape of the foreshore
- Accuracy of wave height measurement (reflection!)
- Scale effects, if any

In view of this, one cannot expect a good consistency in reported values of K_D . In fact, there is a tremendous scatter in the results, and this is no surprise. For the designer it means that he must be extremely careful when applying the formulae. When using the formulae, one must realize what influence uncertainties have on the final result. This applies to the selection of the coefficients, and to the choice of wave height and relative density. Small changes have a big influence on the required block weight. Since there is no basic difference between the two formulae (as long as one applies the Hudson formula within the limits for the slope), one can work with either formula. Many designers prefer the Hudson formula because it is a little simpler to use and because there are far more experimental data on the coefficient K_D than on the Iribarren coefficients.

3.4 Placed-block revetments: Pilarczyk's Approach

In coastal defense work the placed block revetments are widely used in world. This type of revetment has been marked as economic solution of rip-rap. But until Pilarczyk's approach design of this type of revetment was based on trial and error in the Netherlands and other countries (Bezuijen,1990). Research on block revetments performed in the last decades in the Netherlands and Germany has lead to design rules based on understanding and a quantitative description of the failure mechanisms that can occur.

In his contribution Pilarczyk has summarized of this research especially in the model tests. This contribution presents background information also based on the results of that research.

Before deriving the formula of stability of revetment blocks one must have understanding about the system involved in a revetment. A filter layer is placed on the subsoil. The blocks are placed on this filter layer. Normally the blocks placed in a way that the revetment has a flat surface. In this way force the forces parallel to the slope caused by wave attack are minimized. Sometimes a rough surface is created to minimize wave run-up. The water movement in the filter and revetment system of this type is normally decisive for the stability of revetment the pressure difference in the plane of perpendicular to the revetment, caused by wave attack, lead to some general design principles.

In front of wave there is a fluctuating of wave level caused by the wave attack. Inside the structure at some distance from the cover layer there will be a phreatic which is hardly influenced by the fluctuation of attack. Figure 3.2 shows schematic diagram of revetment dealt with.

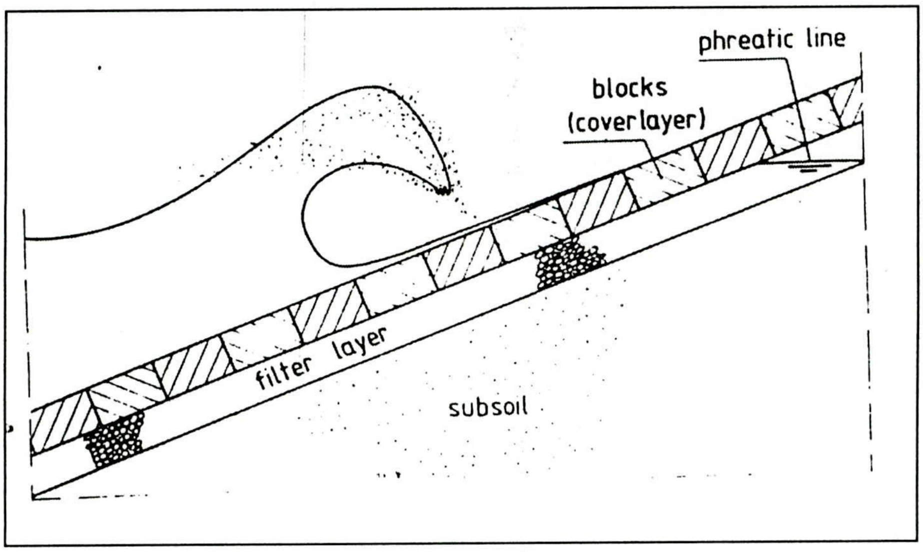


Figure 3.2: Sketch of the revetment dealt with

Failure of blocks means the lifting of blocks from initialization position. Considering highest possible pore water pressure in one hand and the block weight and friction between blocks in another hand as strength the design formula has been derived.

But Pilarczyk came in the formula with the empirical values of $H / \Delta D$. He presented the relation

$$\frac{H_s}{\Delta D} = \phi_0 \frac{\cos\alpha}{\sqrt{\xi}} \qquad (3.9)$$

Hence ϕ_0 is a coefficient, which is 3-3.5 for placed blocks.

But in research field more general form may be used. In this form Pilarczyk (1998) used stability in the following manner.

$$\frac{H_S}{\Delta D} = \Psi\Phi \frac{\cos\alpha}{\xi^b} \qquad (3.10)$$

In this equation the following symbols are used:

H_S = Significant wave height

Δ = Relative density of the concrete

D = Layer thickness

Ψ = System upgrading factor

Φ = Stability factor for incipient motion (=2.25)

α = Slope of the revetment

ξ = Iribarren number [=tanα/$\sqrt{}$(H/L)]

b = Exponent related to the interaction process (0.5<b<1)

However the result of Pilarczyk was based on empirical data. In this data there will always be some influence of clamping between the blocks. Revetment with long leakage factor and loose block will not reach the stability criterion given by Pilarczyk. On the other hand with a careful design it must be possible to come to a higher stability number than the value presented by Pilarczyk.

3.5 Irregular waves, approach of Van der Meer

Between 1965 and 1970, the first wave generators that could generate irregular waves according to a certain predefined spectrum were developed. Model tests in the first years were aimed at ad-hoc . Several researchers attempted to overcome the shortcomings of the Hudson approach by introducing more variables. Initially, their results diverged. In his PhD thesis at Delft University, Van der Meer (1988) succeeded in presenting an approach based on irregular waves that has gradually been accepted throughout the engineering community. In the first place, he used a clear and measurable definition of damage. Initially, this was expressed by the parameter

$$S = \frac{A}{D_{n50}^2} \qquad \textbf{(3. 11)}$$

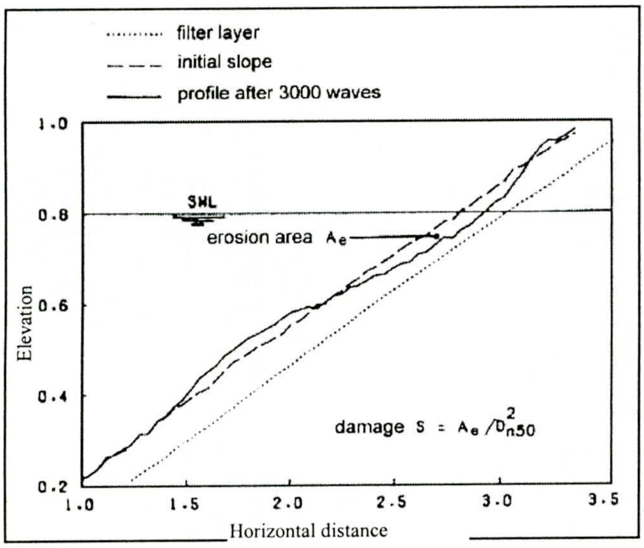


Figure 3. 3: Damage(S) based on erosion area (A)

here $D_{n50}^2 = (W_{50} / g\rho_r)^{1/3} = (M_{50} / \rho_r)^{1/3}$ **(3. 12)**

For a definition sketch, refer to Figure 3.3. The area A is often measured by using a rod with a half sphere of a specific size attached to it.

The erosion in the area A is partly caused by settlement of the rock profile and partly by removal of stones that have lost stability. Since the erosion area is divided by the area of the armour stone, the damage S represents the number of stones removed from the cross-section, at least when permeability/porosity and shape are not taken into account. In practice, the actual number of stones removed from a D_{n50} wide strip is between 0.7 and 1.0 times S. If the armour layer consists of two layers of armour units, one can define limits for acceptable damage and failure. These limits are more liberal for gentler slopes, since in that case, the damage is distributed over a larger area. Critical values for S are given in Table 3.5.

At a later stage, the definition of damage was slightly adapted. A value N is defined, which is the number of units displaced from one strip of the breakwater with a width of D_{n50}. The relation with S is established via the permeability/porosity. When the number of displaced units is counted, the settlement of the mound is omitted from the considerations of damage. The number N is often used when studying the stability of armour layers consisting of concrete units. Van der Meer chose to express the stability in terms of H_s/D_{n50} , and then investigated the influence of several parameters that he considered relevant. These parameters are briefly discussed below.

Table 3.5: Classification of damage levels S for quarry stone (d'Angremond,2001)

Slope	Initial Damage (needs no repair)	Intermediate Damage (needs repair)	Failure (core exposed)
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4	3	8 – 12	17
1:6	3	8 – 12	17

Wave period

Van der Meer assumed the effect of the wave period to be connected with the shape and intensity of breaking waves. He therefore used the Iribarren parameter (as mentioned in Chapter 2)

$$\xi = \frac{\tan\alpha}{\sqrt{H / L_0}} \qquad (3.13)$$

Using the characteristic values for irregular waves in deep water H_s and T_p or T_m , this leads to the use of ξ_{sop} and ξ_{som} respectively.

It must be noted that the value of H_s in the expression $H_s / \Delta D$ is measured at the location of the toe of the structure after elimination of any wave reflection. Contrary to Hudson and Iribarren, Van der Meer found a clear influence of the storm duration. The longer the storm, the more damage. This can easily be explained by the model technique. Hudson and Iribarren used regular waves. A longer duration of the test series did not change the wave attack on the structure. In an irregular wave field, a longer storm duration leads to a higher probability of the occurrence of extremely high waves. Apparently, these extremely high waves are responsible for ongoing damage.

Van der Meer also finds a certain influence resulting from the permeability or porosity of the breakwater structure as a whole. He expresses this influence 'notional permeability' as a factor P, for which he indicates values based on a global impression of the stone size in subsequent layers (Figure 3.4). It is emphasized that,

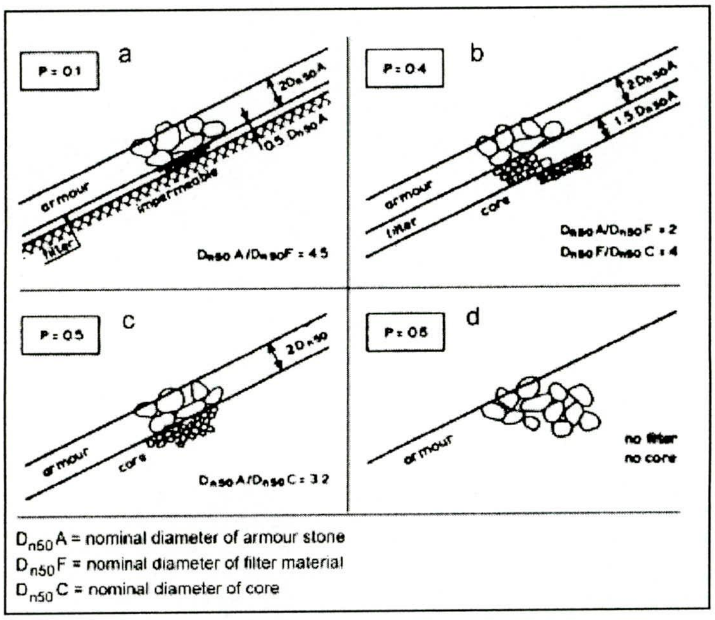


Figure 3. 4: Permeability coefficients for various structures

in fact, P is not a permeability parameter, although it is referred to as being the permeability parameter. It merely indicates the composition of the breakwater in terms of the mutual relations of the grain sizes in subsequent layers.

Quarry stone

After extensive curve fitting, Van der Meer concludes that for quarry stone, the stability is ruled by:

For plunging waves:

$$\frac{H_s}{\Delta D} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\frac{1}{\sqrt{\xi_m}} \qquad \textbf{(3. 14)}$$

For surging waves:

$$\frac{H_s}{\Delta D} = 6.2P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_m^{P} \qquad \textbf{(3. 15)}$$

The transition between plunging and surging waves can be derived by intersecting the two stability curves, which yields:

$$\xi_{mcrit} = \left[6.2P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{p+05}} \quad \textbf{(3. 16)}$$

Depending on slope and permeability, the transition lies between ξ_{som} = 2.5 and 4. The reliability of the formula can be expressed by giving the relative standard deviation σ/μ (in percent) for the coefficients 6.2 and 1.0. These relative standard deviations are respectively 6.5% and 8%, as compared to a reliability of the Hudson formula of 18%.

Concrete blocks

When testing armour layers of artificial material like concrete, it makes no sense to vary the slope of the breakwater. Since the block weight is not so strictly limited as it is for quarry stone (the quarry has a clear maximum block size), it is much more effective to increase the concrete block weight than to reduce the slope. This makes using the Iribarren number ξ in a formula less realistic, since this expresses the influence of both, wavelength or period and slope. All formulae for concrete units, except the Accropod® are based on a slope of 1:1.

Since the mechanical strength of the concrete blocks may play a role, it is useful to distinguish damage due to actually displaced units (their number is indicated by N_{od} , and damage due to blocks that might break because they are rocking against each other (their number is indicated by N_{or}).

The total number of moving units is equal to the number of displaced blocks plus the number of rocking blocks i.e. Nomov = Nod + Nor.

The value of Nod is compatible with the value of S, (compare Equation 3.9). S is about double the value of Nod.

Van der Meer [1988] gives the stability for various frequently used blocks. He makes a distinction between displaced blocks and moving blocks. The difference appears to be a reduction of the stability number by 0.5. The scatter of data for cubes and Tetrapods is normally distributed and has a relative standard deviation σ/μ = 0.1.

Note that D_n is the nominal diameter of the unit, or the cubic root of the volume. For various blocks this leads to:

Cubes	D_n = equal to the side of the cube
Tetrapods	$D_n = 0.65$ D if D is the height of the unit
Dolos	$D_n = 0.54$ D if D is the height of the unit (waist ratio 0.32)
Accropod®	$D_n = 0.7$ D if D is the height of the unit

Like the damage levels for quarry stone, damage levels can also be classified for concrete units as in Table 3.6.

Table 3.6 Classification of damage levels Nod and Nomov for quarry stone (d'Angremond,2001)

Block Type	Slope	Relevant N-value	Start of Damage	Initial Damage (needs no repair)	Intermediate Damage (needs repair)	Failure (core exposed)
Cube	1:1.5	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Tetrapod < 25 ton	1:1.5	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Tetrapod >25 ton	1:1.5	N_{omov}	0	0 – 0.5	0.5 – 1.5	> 2
Dolos < 20 ton	1:1.5	N_{od}	0	0 – 0.5	0.5 – 1.5	> 2
Dolos > 20 ton	1:1.5	N_{omov}	0	0 – 0.5	0.5 – 1.5	> 2
Accropod®	1:1.33		0			> 0.5

Cubes

$$\frac{H_s}{\Delta D_n} = \left(6.7\frac{N_{od}^{0.5}}{N^{0.25}} + 1.0\right) s_{om}^{-0.1} \qquad \frac{H_s}{\Delta D_n} = \left(6.7\frac{N_{od}^{0.5}}{N^{0.25}} + 1.0\right) s_{om}^{-0.1} - 0.5 \quad \textbf{(3. 17)}$$

Tetrapods

$$\frac{H_s}{\Delta D_n} = \left(3.75\frac{N_{od}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} \qquad \frac{H_s}{\Delta D_n} = \left(3.75\frac{N_{od}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} - 0.5 \quad \textbf{(3. 18)}$$

Dolos

Holtzhausen and Zwamborn [1992] investigated the stability of Dolos with the following result:

$$N_{od} = 6250\left[\frac{H_s}{\Delta^{0.74} D_n}\right]^{5.26} s_{om}^{\ 3} w_r^{\ 20 s_{op}^{\ 0.45}} + E$$

The waist ratio has been made a variable in the Dolos design to enable the choice of a less slender shape with less chance of breaking. Waist ratios are between 0.33 and 0.4. The error term E represents the reliability of the formula. It is normally distributed and has a mean value equal to zero, and a standard deviation σ(E):

$$\sigma(E) = 0.01936\left[\frac{H_s}{\Delta^{0.74} D_n}\right]^{3.32}$$

Accropod®

The Accropod® unit is applied in a single layer at a slope of 1:1.33, according to the recommendations of SOGREAH. The recommended placing method is given in Appendix 3, on the basis of documents provided by SOGREAH.

Van der Meer finds no influence of storm duration and wave period for these units. Instead, he defines:

Start of damage, $N_{od} = 0$ at

$$\frac{H_s}{\Delta D_n} = 3.7$$

Failure, $N_{od} > 0.5$ at

$$\frac{H_s}{\Delta D_n} = 4.1$$

The values 3.7 and 4.1 may be considered as stochastic variables with a standard deviation of 0.2. It is clear that failure occurs at a wave height that is only slightly higher than the wave height which is associated with "start of damage". In this way, a built-in safety coefficient that applies to all rubble mound breakwaters is not valid for the single Accropod® layer. Van der Meer recommends therefore the inclusion of a safety coefficient and the use as a design value of: $\dfrac{H_s}{\Delta D_n} = 2.5$

Design formulae discussed above are the existing theoretical knowledge. There is a gap in theory and practice in Bangladesh. The study aims to abridge the gap between the theory and practice.

3.6 Schiereck's Theoretical Formula and Experimental Stability Lines

In placed block revetment works Schiereck(2001) formula is used as theoretical basis. This formula is called conservative formula and preliminary design of block size is done with this formula.

Then final design of block is checked by experimental stability line which has been developed by a series of laboratory experiments with 1:1(prototype: laboratory standard) scale ratio.

The theoretical formula developed by Schiereck has been stated as follows:

$$\frac{Hs}{\Delta D} = 3\frac{\cos\alpha}{\xi} \qquad \textbf{(3. 19)}$$

where Hs = Significant wave height, D = thickness of blocks, Δ = relative density , α= slope of the revetment structure and ξ = surf parameter defined earlier.

Theoretical background of wave loading has been described as follows:

Wave attack on revetments will lead to a complex flow over through the revetment structure (filter layer and cover layer). During wave run up the resulting forces by the waves will be directed opposite to the gravity forces. Therefore the run up is less hazardous then the wave run down.

Wave run down will lead to two important mechanisms:

- the downward flowing water will exert a drag force on the cover layer and the decreasing seepage level coincide with downward gradient in the filter.
- During maximum wave rundown there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a wall of water giving a high pressure under the point of maximum rundown.

Thus the high pressure front will lead to an upward flow in the filter layer. This flow will meet the downward flow in the rundown region. The result is an outward flow and uplift pressure near the point of maximum wave run down.

To magnitude of the uplift force, the relation between the permeability of the top layer and that of the filter layer, expressed in the leakage length, Λ , is very important. According to Figure 3.5 piezometric head ($\phi = p/\rho g + z$) on the top layer and in filter layer are ϕ_F and ϕ_T. Based on continuity the following equation can be written:

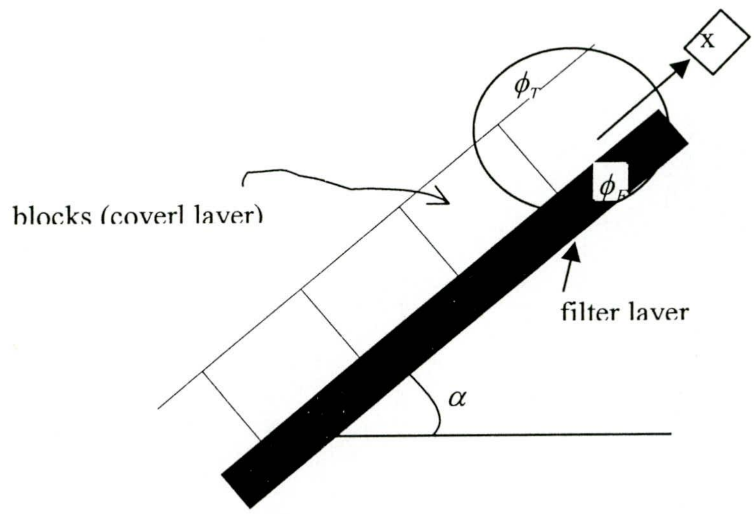


Figure 3.5: Piezometric head at cover and filter layer

$$\phi_F - \phi_T = -\Lambda^2 \frac{d^2\phi_F}{d^2\phi_T}$$

It shows that at higher the leakage length, condition for stability will be more unfavourable.

Then the head difference caused by blocks is solved and the following relation has been developed.

$(\rho_s - \rho_w)gd\cos\alpha - \rho_w g 0.33\xi H_s = 0$

Considering some assumptions, such as factor 3, finally this equation can be written as $\frac{H_s}{\Delta D} = 3\frac{\cos\alpha}{\xi}$

This theoretical approach now established as most conservative stability line.

On the other hand the experimental line of stability has been developed by a series of experimental runs. The experimental runs have been conducted with 1:1 scale ratio. The representative geotechnical and dynamic wave actions are then simulated in the model.

The results of the runs are then plotted and from that plot a graphical solution has been revealed. The plotted graph shows a stability number and surf parameter.

Design of blocks is said to be satisfied when the point of stability number and surf parameter for the block lies below the stability line.

3.7 Riprap Protection Structure, Masoom (2002)

Recently Masoom (2002) has also studied the riprap protective structure with soil reinforcement. The study has observed the influence of riprap placement type such as uniformly placed riprap and randomly placed riprap on the performance of bank protection work subject to wave action.

The study revealed that about 200 percentage of increment of structural strength (compared to design strength as per Hudson) has been observed with uniformly placed blocks on compacted backfill and geotextiles filter. Whereas the same structure with randomly placed riprap shows a 128 percentage increment of strength. However the best structural performance is observed with uniformly placed blocks on geotextile filter without reinforcement.

In the study it is shown that randomly placed riprap on geotextiles shows for progressive failure which provides time for repairing. But uniformly placed riprap shows sudden failure.

Scale models of riprap protection structure are not used for the study. Therefore it has been recommended to use scale model for similar types of study. In the present study similar type of works have been conducted considering a prototype and scaling down to a laboratory standard.

Chapter 4

Laboratory Experiments and Data Collection

4.1 Introduction

The experimental runs have been carried out in the Hydraulics and River Engineering Laboratory of the Department of Water Resources Engineering of Bangladesh University of Engineering and Technology, Dhaka.

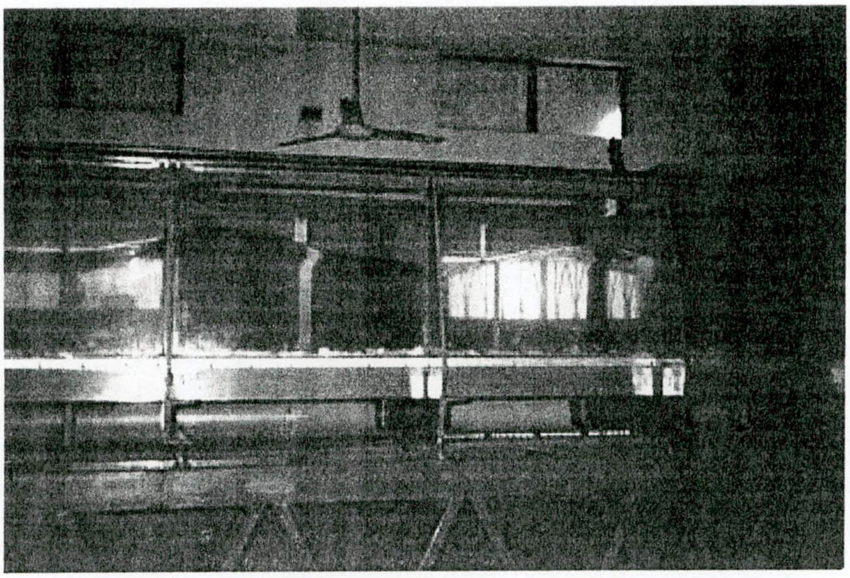
4.2 Laboratory Equipments

For collection of the necessary data the following components were used for experimental runs.

i) Laboratory flume
ii) Wave height meter
iii) Data acquisition system

4.2.1 Laboratory Flume

The flume used for experimental runs is 21.34 m long, 0.76 m wide and 0.76 m deep. The side walls are made of glass and bed is painted by water resistance colour as shown in Photograph 4.1.



Photograph 4.1: Laboratory flume with waves

In the flume wave generator was set at one end and at other end bank slope was prepared. It consists of a reservoir and a stilling chamber. The stilling chamber is located behind the wave generator. This chamber is approximately 3.0 m in length. Bank slope requires another 3.0 m length of the flume. In front of the structure turbulence of water has been observed. Length of that turbulence is approximately

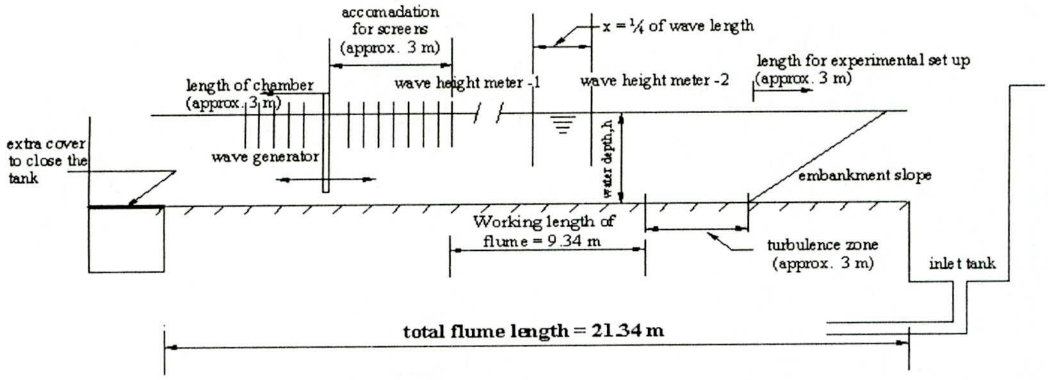


Figure 4.1: Sketch of laboratory flume showing working length

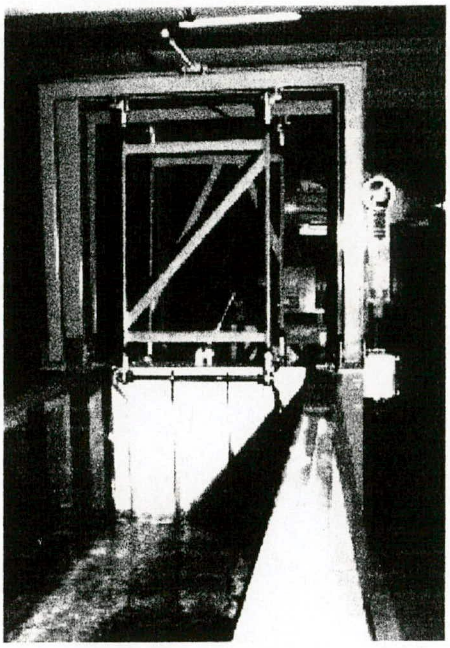
3.0 m. Therefore the available working length in the flume becomes approximately 9.34 m as shown in Figure 4.1.

The bed of the flume was kept horizontal and section of the flume section is rectangular. The flume is supported on an elevated steel truss. Laboratory flume height is limited to carry out runs in a large-scale experiment. To generate a highest possible wave height rubber pads were attached such that water cannot be flush out from the flume. Flume was set and ensured in such a condition that there was no leakage.

Wave generator:

Wave generator has a motor and paddle with two vertical limbs as shown in Photograph 4.2. Waves are generated by rotating paddle. Wave period of generated waves can be altered by rotating its rotational speed and wave height can be altered by changing the arm of paddle. Rotational speed can be altered from 20 rpm to 120 rpm and paddle arm can be altered from 25 mm to 320 mm.

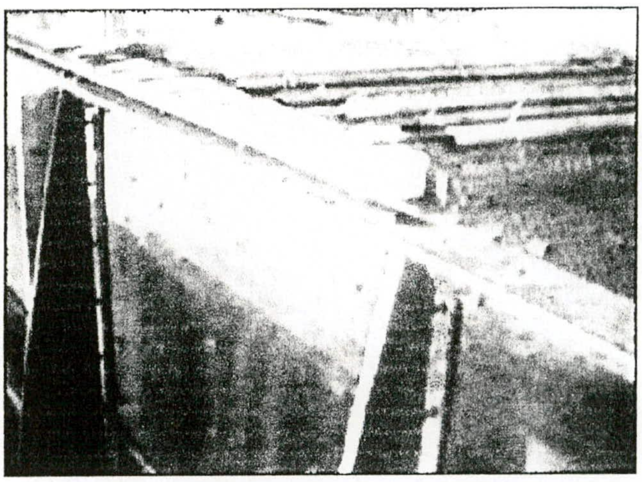
During movement of paddle two displacements were observed. One is rotational displacement and another is vertical displacement. By adjusting vertical limbs these two types of displacements were adjusted.



Photograph 4. 2: Wave paddle with generator

Wire Screens to reduce wave reflections

Several screens were set to reduce wave reflections. Screens were made of coarse wire mesh. They were placed in front of wave generator as shown in Photograph 4.3.



Photograph 4.3: Screens for wave re-reflection control

Numbers of screens and spacing have been determined by trial and error method. Screens were kept at approximately 5 cm apart from each other. In this study finally 20 screens have been used to reduce reflections. When the crests of waves generated were seen in a straight line from side view reflection of the wave was considered to be reduced at the minimum.

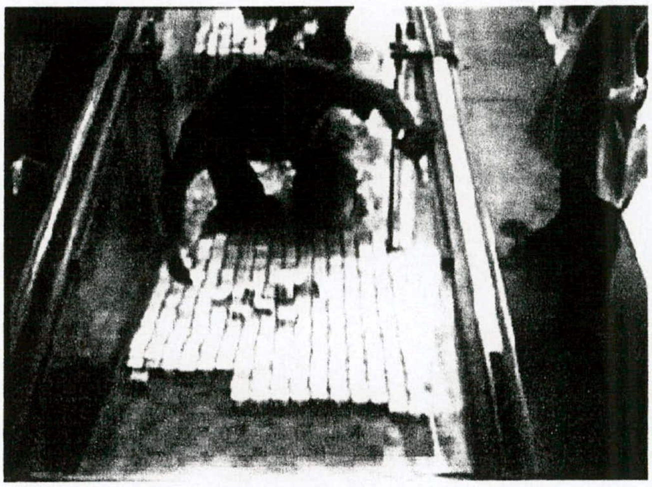
Bank slope preparation

Wooden frame used as wave damper has been taken as a base on which the experimental bank slope was set. First the damper was turned over and flat surface was used as a base of slope. Then an acrylic sheet of 1.5 cm thickness was set over it

Photograph 4. 4: Wooden frame with acrylic sheet over it.

as shown in Photograph 4.4. Then cotton net was used as representative of geotextiles and was glued over it to make sufficient friction between blocks and sheet. Sheets were screwed so that it is easy to alter slope. The wooden frame was kept fixed so that it could not be moved.

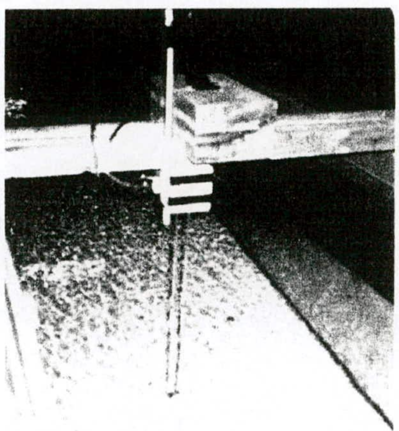
Blocks have been placed on the bank slope with close block system for wave run up observations as shown in Photograph 4.5. To check the stability of blocks for particular set up free block system has been used. In this system blocks were made frictionless at the sides by gluing a separator (a piece of wire) on side surface. The system was considered to represent free blocks system of the prototype situation. In the free block system only line contacts between blocks were achieved.



Photograph 4.5: Placement of blocks on slope

4.2.2 Wave Height Meter

The wave height meter has been designed for water level measurements. The instrument consists of a gauge, two parallel stainless rods mounted underneath a small box containing AC-DC converter and pre-amplifier. It has a separate amplifier also. Photograph 4.6 shows a wave height meter placed in a flume.



Photograph 4. 6: Wave height meter placed in a flume

This study has followed Goda and Suzuki(1976) method for wave height meter setup. In this method only wave height meters are necessary in the flume at a quarter of wavelength distance apart as shown in Figure 4.1.

Then wave height meters were connected to a data acquisition system to store data in a computer. Before actual measurements of waves, the waves height meter was calibrated. This was done by moving up and down the probes and measuring the depth and corresponding voltages. A linear relationship between water depth and voltage was obtained for each probe and was set in the data acquisition software as shown in Figure 4.2.

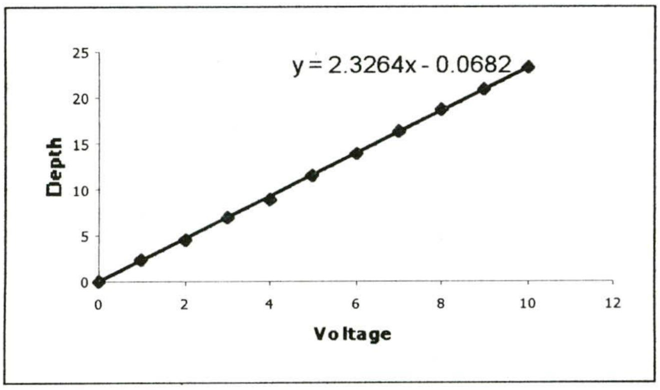
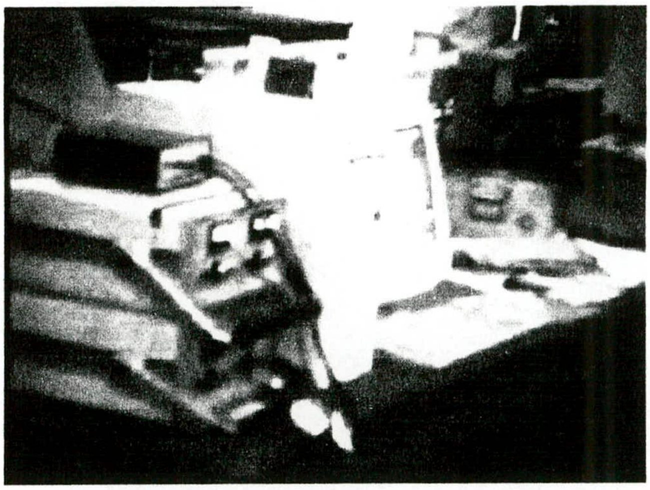


Figure 4.2: Calibrating equation, y= voltage and x=depth of water

4.2.3 Data Acquisition System

In the laboratory data acquisition system consists of a software LabVIEW, amplifier and data acquisition card with computers and shown in Photograph 4.7.



Photograph 4.7: LabVIEW with computer, data acquisition card, amplifier etc

Laboratory Virtual Instrument Engineering Workbench (LabVIEW) is a virtual laboratory setup tools used in laboratory for various purposes. It is a graphical

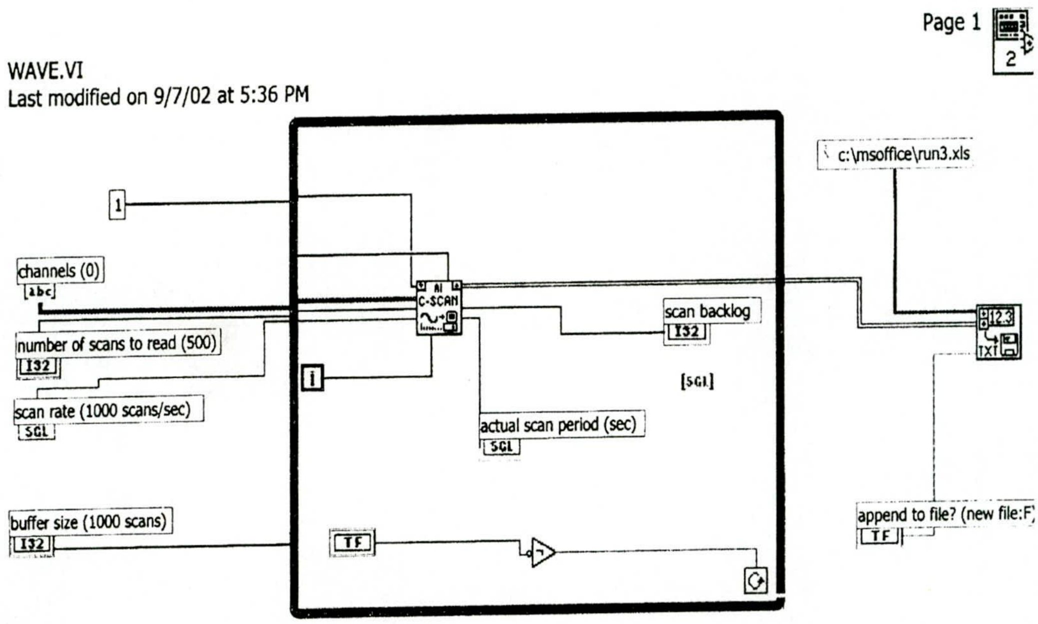


Figure 4. 3: Diagram of a LabVIEW program titled as "wave.vi"

programming for virtual instrumentation. In this study LabVIEW has been used for instant data acquisition and computerization. Amplifier is used to amplifying voltage readings of wave height meters. It is connected to data acquisition card and wave height meters. Data acquisition card is connected to computer and voltage amplifier. It is also configured in LabVIEW software.

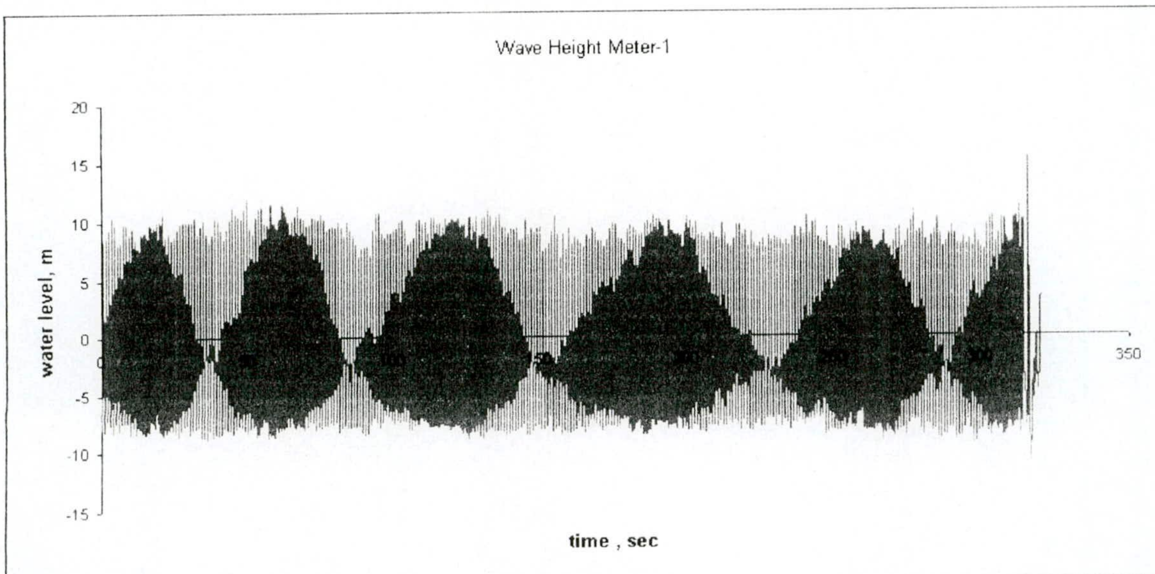
In LabVIEW software a program has been prepared named as "wave.vi" to acquire data from wave height meter. Figure 4.3 explains the diagram of the program written in LabVIEW.

In this program there are several control points. Scan rate, number of scans need to be set per buffering and the size of buffer are the control the system of acquiring data. There is a system to store data in a separate file. In this program it has shown that data has been written in c:\msoffice\run3.xls.

Sample rate and the frequency of sampling were set as input before operation of LabVIEW. Care is needed that buffer size is larger enough than number of scan. Otherwise there is a possibility of losing some of the stored data.

Recorded wave data through a LabVIEW program wave.vi were presented in a CD ROM attached as an Annex-D.

Using LabVIEW program wave.vi experimental data were recorded from wave height meter-1 and 2 and shown in Figure 4.5.



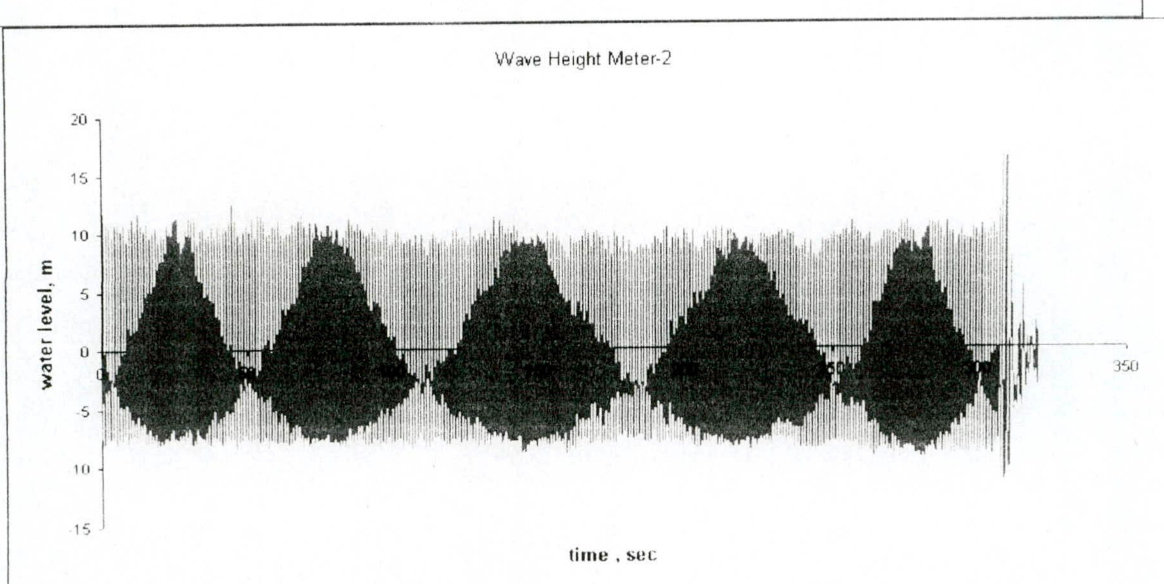


Figure 4.5: Time series wave profile recorded by wave height meter 1 and 2 for Run No 11

As an example (for Run No-11 as stated in section 4.6) time step was taken as 0.01 sec and total number of scans was about 40000. The recorded maximum and minimum water levels from each wave height meters were

	Wave height meter-1	Wave height meter-2
Maximum Water Level, cm	15.343	16.736
Minimum Water Level, cm	-10.993	-11.363
Wave height =	26.336	28.099
Average wave height =	27.22	

Average of two wave heights measured from two wave height meters was recorded as the wave height for that particular run. In this case wave height was recorded as 27.22 cm. In this example datum was 2.43 cm above the still waver level.

4.3 Procedure for Selection of Prototype Boundary Conditions

This study did not simulate any specific case and hence the boundary conditions were set considering Bangladesh haor and coastal situation.

As this study emerges from a research program on haor areas of Bangladesh, it is meaningful to give more importance in defining boundary conditions with respect to Bangladesh haor conditions.

4.3.1 Selection of Toe structure

In order to support the blocks against sliding and to protect the structure against undermining a well protective toe is necessary. In the field the most unfavourable condition has been obtained when water at the country side will not be present and only wetland side water will cause forces on the structure. At this condition function of toe is only to protect blocks against sliding. In the laboratory toe has been protected by placing double layer of blocks as the sketch shown in Figure 4.4. There were also two steel bars placed at two sides of the wooden frame to keep the frame at its position.

4.3.2 Selection of Embankment Slope

In practical case the selection of slope is usually governed by soil conditions. Due to scarcity of land designers use a slope between 1(V) : 2(H) to 1(V):3(H) for embankment and village mount construction in haor areas.

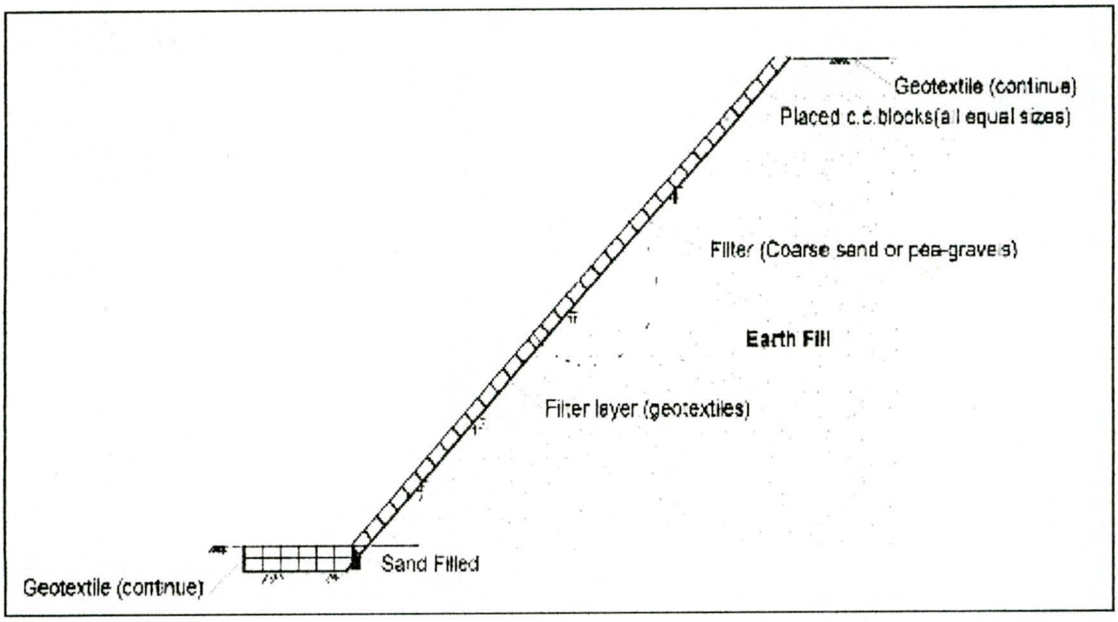


Figure 4.4: Double layers of blocks at toe of the structure to protect against sliding of blocks

This study has adopted two slopes for experimental runs. Of them steeper slope was 1(V) on 1.5 (H) which was adopted by Hudson (SPM, 1984) and milder slope was 1(V) on 3 (H) which was adopted by Van der Meer (Pilarczyk, K.W. et al. 1998).

4.3.3 Selection of Block Size

Considering cost of a total project cubical c.c. blocks are not preferred in design at present days. With the same weight or thickness of block more surface area is covered by flat type blocks. Therefore flat type blocks are used in practical cases. The maximum size that can be manually handled is 400 mm x 400 mm x 250 mm flat type blocks (around 100 kg mass). Considering the minimum range of significant wave height, the block mass calculated theoretically is around 11 kg (200 mm × 200 mm× 100 mm). Blocks sizes should not be so small that they can be misplaced or carried away by local people. So, within ranges between 100 kg and 33 kg three block sizes were selected in this study as shown in Table C.1.

The material proportions (cement, sand and stone chips or shingles) have been selected as 1:2:4 and the specific gravity of the blocks varied between 2.12 and 2.18. as shown in Tables C.2 to C.5. Designers in Bangladesh use specific gravity of c.c. blocks as 2.24 (140 lb/cft).

4.3.4 Selection of Filter Layer

Both granular filter and fiber filter are in use in the practical case but fiber filter (geo-textile) is in common use because of difficulties in maintaining the strict specifications of the granular layer by the contractors. Normally 1.5 mm to 3.0 mm thick non woven needle punched type geotextile with effective opening size 0.12 mm to 0.08 mm is used as fiber filter for revetments. If larger opening sizes of geotextiles are used then silt layer is placed below geotextiles.

In the laboratory experiments geotextiles have been represented by thin cloth and also by cotton net. The cloth or cotton nets are arbitrarily selected just to represent the friction between blocks and geotextiles in the field.

For observation of damage of c.c. blocks over geotextiles has been compared with another situation. For this reason blocks are placed without geotextiles and just over soil.

4.3.5 Selection of Water Depth at Toe of the Structure

Water level in the haor area starts rising during March-April. Depth of flooding becomes maximum in August-September. According to BWDB officials water depth in haor areas usually varies from 2 to 5 m in front of structure. During monsoon 4-5 m water depth is frequently obtained (CARE, 2000). In coastal region water the ranges is little bit higher and vary between 2 m and 7.3 m. (MES, Draft Master Plan, 1998).

For lab experiment prototype water depth has been taken as 4.5 m. With: 1:10 scale (prototype: laboratory standard) this is equivalent to 45 cm depth of water in the flume. Some of the experimental runs have also been conducted considering 6.0 m prototype depth and due to limitation of height of the flume this is represented with 1:20 scale.

4.3.6 Selection of Wave Height and Wave Period

There is no wave recording station in Bangladesh. For this reason wind speed and fetch data have been used to determine significant wave height and significant wave period by shallow water wave forecasting techniques considering water depth discussed above.

Wind speed data of 1969 to 1998 recorded by Bangladesh Meteorological Department (BMD) at Mymensingh were analyzed by CARE (2000). They reported that the maximum observed wind speed reached 167 km/h or 46 m/s. These values have been taken as standard values for haor condition of Bangladesh.

For coastal region Bangladesh Meteorological Department recorded a maximum wind speed between 185.5 km/h or 51.5 m/s (MPSC,1998).Considering all these the wind speeds for wave height calculation have been taken to vary between 10 m/s and 60 m/s. The fetch lengths have been reported to vary between 5km to 40 km (Alam, 2002, Definition Report). At some places like Sunamganj fetch length may be 100 km. In the present study fetch lengths have been considered to vary between 5 km and 100km. The shallow water forecasting wave equations (Equation 4.1 and 4.2) have been used to determine significant wave height and peak wave period. Multiplying peak wave period by 0.95 significant wave period has been obtained. Equation 4.3 has been used to calculate minimum duration needed to develop wave.

$$\frac{gH_s}{u_A^2} = 0.283 \tanh\left[0.530\left(\frac{gd}{u_A^2}\right)^{0.75}\right] \tanh \frac{0.0125\left(\frac{gF}{u_A^2}\right)^{0.42}}{\tanh\left[0.530\left(\frac{gd}{u_A^2}\right)^{0.75}\right]} \qquad \textbf{(4. 1)}$$

$$\frac{gT}{u_A} = 2\pi.1.2 \tanh\left[0.833\left(\frac{gd}{u_A^2}\right)^{0.375}\right] \tanh \frac{0.077\left(\frac{gF}{u_A^2}\right)^{0.25}}{\tanh\left[0.833\left(\frac{gd}{u_A^2}\right)^{0.375}\right]} \qquad \textbf{(4. 2)}$$

$$\frac{gt}{u_A} = 5.37\times 10^2 \left(\frac{gT}{u_A}\right)^{7/3} \qquad \textbf{(4. 3)}$$

Where Hs = significant wave height (m),T = peak wave period(sec), F = fetch(m), d = water depth (m), t = minimum duration to develop wave(sec), u_A = adjusted wind velocity (m/s), g = gravitational acceleration (m/s^2)

Significant wave heights, time periods and duration of time for wave development different water depths, fetch length and wind speed have been presented in Tables C.6 to C.23. From these extensive analysis it is seen that regular wave height varies from 0.56 m to 3.07 m and the wave periods range from 2.24 sec to 6.90 sec. Ranges of prototype and laboratory standard wave height and time period are tabulated in Table 4.1.

Table 4. 1: Ranges for experimental set up

Prototype wave height (regular wave height)	Laboratory standard wave height *scale = 1:10*	Prototype time period (regular wave height)	Laboratory standard wave height *scale = 1: 3.16*
H (p)	H (m)	T (p)	T (m)
m	cm	Sec	sec
0.56	5.6	2.24	0.7
3.07	30.7	6.23	1.97

4.4 Measurement Techniques

In the present study regular wave heights have been measured for each experimental runs. Damage pattern has been measured in term of percentage of damage and failure type of blocks. Wave run up has been measured for corresponding wave height.

Wave height measurement:

Wave height measurement was significant part of the study. Wave height measurement was conducted by wave height meter. This was checked by point gauge reading.

With the available point gauge in the laboratory, only 48 cm height could be measured. To measure the height of wave two point gauges were required. At first two point gauges were pointed at still water level. Taking still water level as reference level, crest of the wave was measured by one point gauge and the trough of wave by another point gauge.

As mentioned earlier and shown in Figure 4.1 measurement of wave height by wave height meter was done by setting two wave height meter and they have been set at two positions keeping apart each other by one fourth of wave length. Before operating wave generator, it needs some adjustment between rotational and transitional movement that depends on wave period and water depth. Figure A.2 (Annex-A) gives rotational, e and transitional, f parameter to develop non-breaking harmonic waves. The stepwise procedure to generate regular waves without breaking at the paddle of wave generator has been presented in Annex-A. For wave periods

0.7 sec to 2.0 sec at water depth 45 cm the required rotational (e) and transitional (f) displacement has been tabulated in the Table 4.2. The table also shows values of e and f for water depth of 30 cm and time period of 2.5 sec and 3.0 sec.

Table 4. 2: Wave generator set up for experimental runs

Time period	Water depth	$\omega^2 h / g$	e	Figure	(e+f)/f
sec	cm		Values are taken from Figure A.2		
0.7	45	3.69	0.67	0.02	133.5 (*)
1	45	1.81	0.77	0.14	6.50
1.2	45	1.26	0.64	0.38	2.68
1.5	45	0.80	0.51	0.72	1.71
2.5	30	0.19	0.24	2.07	1.12
3	30	0.13	0.19	2.08	1.09

(*) fully rotational

Then scan rate, number of scan and buffer sizes were fixed in the LabVIEW program. Then data have been collected with the data acquisition system mentioned earlier in section 4.2.3. Recorded data should be filtered to curtail several initial and ending data. Because these may be misleading. From filtered data wave height can be measured from minimum and maximum values of the recorded data.

But the obtain wave height contains two components, one is incident wave and another is reflected waves with same frequency but in opposite directions. To separate the reflection from a complex wave, a Matlab program "Refreg.m" was available from the Laboratory of Fluid Mechanics of Delft University, the Netherlands. The method has been described by Goda and Suzuki (1976). Description of this program has been presented in Annex-B. A sample of inputs and outputs of Refreg.m program has also been shown in Annex-B.

Damage measurements:

Damage phenomenon were observed with and without geotextiles and for 350 mm x 350 mm x 200 mm prototype c.c. block size only. This was reduced to 1:20 scale for laboratory runs. For experiments without geotextiles 1.5 cm thick median sand layer was provided as granular filter below the blocks as to develop friction between blocks and granular filter. According to Van der Meer(1988) equilibrium damage should be achieved after 5000 waves. Number of blocks removed from initial position from the zone between SWL+H and SWL-H was counted and percentage of damage was measured by counting initial and removed number of blocks.

Damage phenomenon with geotextiles was observed by placing thin cloth below blocks as to represent friction between blocks and geotextiles.

Wave run up measurements:

Wave run up was been visually observed and measured by marking points on the slope made of c.c. blocks. The uppermost point was marked finally and vertical reading of that point with point gauge gave wave run-up for that particular setup. Photograph 4.8 shows wave run up over slope 1(V): 3(H).

Photograph 4.9: Wave run up over a slope

4.5 Test Scenarios

Thirty seven runs were conducted for the present study and shown in Table 4.3. Out of them first five runs (Run No. 1 to 5) were made for damage observation. Blocks were placed on geotextiles. One type of blocks was used in these runs. Wave height was gradually increased and damage development was observed.

In the study of stability of c.c. blocks several trial runs were required to obtain a critical wave height (the wave height that produces initial damage). For a particular slope, water level, wave period and blocks size, waves were generated. Approximately 300 waves were applied to the blocks. If the blocks were not failed after 300 waves, higher waves were generated by changing amplitude of wave paddle. Thus wave height was increased gradually and failure of blocks was observed. The wave height that could initiate damage of blocks was recorded as critical wave height. Thus six runs were recorded for stability of blocks.

The rest twenty-six runs (Run No 12 to 37) were done for wave run up measurements. Wave heights were gradually increased and different wave run-ups were measured for the runs. The runs for wave run up were conducted with two different scales such as 1:10 and 1:20. Runs with 1:20 scale were done at wave period 2.5 sec and 3.0sec. But the runs with 1:10 scale were conducted at wave period 1.0, 1.2, 1.5 and 2.0 sec. First wave period and water depths were fixed. Then wave generator was set according to measurement techniques stated earlier. Water was poured and waves were applied in water. Generated waves were measured by wave height meter and checked by point gauge readings.

Table 4. 3: Test Scenarios

Run No	Prototype Block sizes	Laboratory Block sizes	Slope	Water depth	Wave period	Scale (prototype: model)	Observations
	mm x mm x mm	mm x mm x mm	V: H	cm	sec	-	-
1-5	350 x 350x 200	17.5 x 17.5 x 10	1:1.5	30	2.5	1:20	(*1)
6	400 x 400x 250	20x 20 x 12.5	1:1.5	30	2.5	1:20	(*2)
7	400 x 400x 250	20x 20 x 12.5	1:1.5	30	2.4	1:20	
8	400 x 400x 250	40 x 40x 25	1:3	45	1.0	1:10	
9	400 x 400x 250	40 x 40x 25	1:3	45	1.2	1:10	
10	400 x 400x 250	40 x 40x 25	1:3	45	1.5	1:10	
11	400 x 400x 250	40 x 40x 25	1:3	45	2.0	1:10	
12-16	350 x 350x 200	17.5 x 17.5 x 10	1:1.5	30	3.0	1:20	(*3)
17-21	400 x 400x 250	20x 20 x 12.5	1:1.5	30	2.5	1:20	
22-25	400 x 400x 250	40x 40 x 25	1:3	45	1.0	1:10	
26-29	400 x 400x 250	40x 40 x 25	1:3	45	1.2	1:10	
30-33	400 x 400x 250	40x 40 x 25	1:3	45	1.5	1:10	
34-37	400 x 400x 250	40x 40 x 25	1:3	45	2.0	1:10	

(*1) Run No 1-5 for observation of damage with geotextiles (for different wave heights) and stability of blocks

(*2) Run No 6-11 for observation of stability of blocks

(*3) Run No 12-37 for observation of wave run up

Chapter 5

Analysis of Experimental Results

5.1 Introduction

In the present study comparisons between laboratory data and the design formulae, damage development observation, wave run up measurements and influence of wave frequency have been done.

5.2 Comparisons between Laboratory Data and Design Formulae

Laboratory data were compared with the Stability formulae of Pilarczyk(1990), Schiereck(2001), Van der Meer(1988) and Hudson(1961). In the laboratory the determination of stability number of c.c. blocks by using those formulae requires determination of critical wave height for a particular setup. So a number of test runs were done to determine critical wave height. Then comparisons of the different formulae were done.

As mentioned earlier that one critical wave height was obtained from the data for damage development. So it is important to analyze damage data here. Experimental set up was constructed with a slope of 1(V): 1.5(H), block size was 17.5 x 17.5 x 10 mm^3 and blocks were placed over represent geotextiles that was cotton cloth. In this setup water depth was 30 cm and number of waves applied was 5000 to achieve equilibrium damage as defined earlier in Section 4.4. Wave period was kept as 3.0 sec. Damage for different wave height was observed from this setup. Table 5.1 shows the results obtained from the analysis of damage development. From that table critical wave height was achieved from Run No 5.

Table 5.2 shows the critical wave heights for different setups. Three types of block sizes were used here. Relative densities of blocks were 1.12 and 1.18. The experiments were conducted at two scales (1:20 and 1:10). Reflection corrections were varied between 15% and 28%. Wave heights of Run No 6 to 8 were recorded without reflection corrections. Two types of slopes are used in the experimental runs.

Table 5. 1: Experimental data for damage observation with geotextiles

water depth =30cm , block size = 17.5 mm x 17.5 mm x 10 mm, slope = 1(V): 1.5(H)

Run No	Wave period	Wave height	No of waves (*)	Damage
	T	H	N	-
	sec	cm	-	%
1	2.5	6	5000	not failed
2	2.5	7.1	5000	not failed
3	2.5	7.5	5000	not failed
4	2.5	9.2	2500	catastrophic fail
5	***2.5***	***8.6***	***4500***	***catastrophic fail***

(*) approximate no of waves are recorded

Table 5. 2: Experimental results to be used for comparison of stability formulae

Run No	Water depth	Laboratory block sizes	Weight of block	Slope	scale	Wave period	Relative density	Recorded Incident Wave height	Reflection coefficient	Reflected Wave height	Corrected Wave height
	d		W	α		T	Δ	H_i	K_r	H_r	H
	cm	mm x mm x mm	gm	-	-	sec	-	cm	-	cm	cm
5	30	17.5 x 17.5 x 10	7.1	1.5	1:20	2.5	1.12	8.6	-	-	8.6
6	30	20x20x1.25	13.1	1.5	1:20	2.5	1.12	7	-	-	7
7	30	20x20x1.25	13.1	1.5	1:20	2.4	1.12	7.9	-	-	7.9
8	45	40x40x2.5	87.4	3	1:10	1	1.18	14.5	0.18	2.6	11.9
9	45	40x40x2.5	87.4	3	1:10	1.2	1.18	21.3	0.21	4.5	16.8
10	45	40x40x2.5	87.4	3	1:10	1.5	1.18	25.7	0.28	7.2	18.5
11	45	40x40x2.5	87.4	3	1:10	2	1.18	27.2	0.22	6.0	21.2

5.2.1 Comparison with Pilarczyk Formula

Laboratory data were observed by changing water depth, slope, block size, and wave height and wave period. Data obtained with 1:10 scale were conducted with 1:3 slope, 40 x 40 x 25 mm^3 block size and with 30cm water depth. Then wave period were fixed for a particular set up and by changing wave heights stability of blocks were observed. Same procedure was applied in the runs with 1:20 scales. In this way obtained laboratory data have been shown in Table 5.3.

Based on empirical data Pilarczyk(1990) have developed the formula as stated in Chapter 3. Laboratory data to check the validity of stability formula were plotted on Figure 5.1.

Table 5. 3: Comparison of laboratory data with Pilarczyk formula

Run No	Water depth	Scale	Slope	Lab. block thickness	Density	Wave period	Wave height	Surf parameter	Laboratory stability	Pilarczyk Stability	Block thickness as per Pilarczyk	% of deviation
	d		cot α	D	ρ	T	H_s	ξ			$D_{(Pilarczyk)}$	
	cm		-	cm	gm/cc	sec	cm	-	-	-	cm	
5	30	1:20	1.5	1	2.12	2.5	6.14	8.402	5.48	0.45	12.18	1118.25(*)
6	30	1:20	1.5	1.25	2.12	2.5	5.00	9.313	3.57	0.40	11.16	-792.86(*)
7	30	1:20	1.5	1.25	2.12	2.4	5.64	8.767	4.03	0.43	11.71	-836.88(*)
8	45	1:10	3	2.5	2.18	1	8.50	1.429	2.88	4.06	1.87	25.23
9	45	1:10	3	2.5	2.18	1.2	12.00	1.443	4.07	4.02	2.67	-6.61
10	45	1:10	3	2.5	2.18	1.5	13.21	1.719	4.48	3.27	3.61	-44.28
11	45	1:10	3	2.5	2.18	2	15.14	2.141	5.13	2.53	5.34	-113.72

(*) Runs were carried out with smaller scale (1:20) and reflections were not corrected.

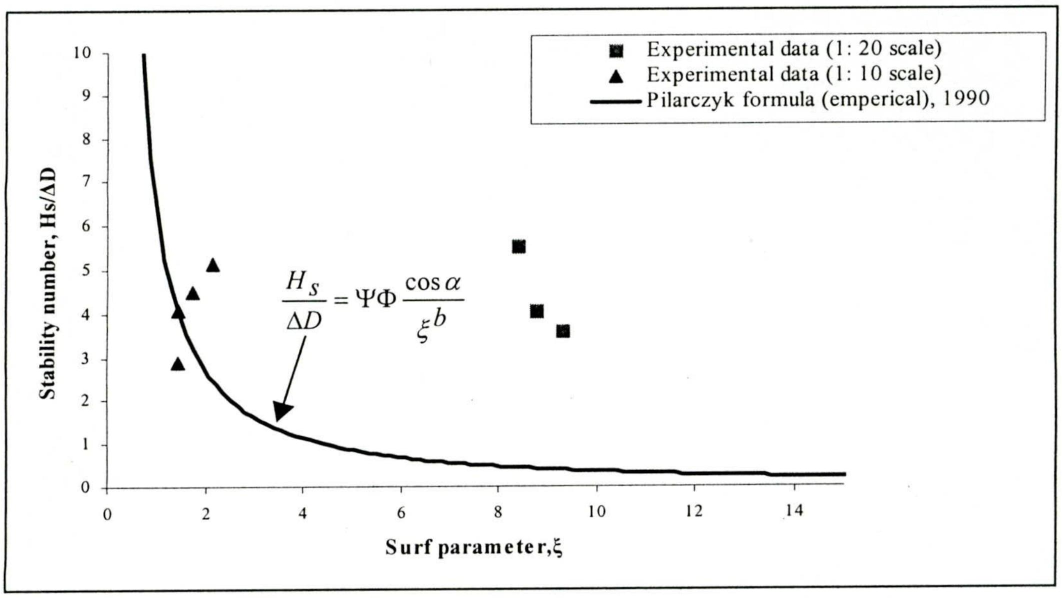


Figure 5.1 Comparison of laboratory data with Pilarczyk formula

In the same figure Pilarczyk formula has been plotted and compared with the laboratory results. In the Pilarczyk formula a coefficient Φ is used. This coefficient depends on the armour unit. For placed blocks Φ varies from 3 to 3.5. For the present case the value of Φ is taken as 3.25 as an average. System upgrading factor Ψ for concrete block is 2.0 and exponent b is given for concrete as 0.67. These values are used in this formula to plot in Figure 5.1

Block thickness as per Pilarczyk shows 35% deviation from the laboratory data. But negative deviation implies that laboratory data are plotted below the stability line of Pilarczyk. So it is evident from the figure that the laboratory results obtained at larger scale ratio (1:10) have agreed satisfactorily with the stability line of Pilarczyk formula.

Laboratory runs carried out at 1:20 scale ratio show that block thickness of laboratory data were smaller that block thickness as per Pilarczyk. Because the laboratory data shows more stability number than Pilarczyk's stability. In laboratory friction between blocks and geotextiles might not be represented as prototype. For this reason more stability of laboratory blocks was observed.

However the study reveals that Pilarczyk formula should be suitable at lower value of surf parameter ($\xi<2.0$) which covers the Bangladesh haor condition.

5.2.2 Comparison with Schiereck's Theoretical and Experimental line of stability

Laboratory data for stability of blocks were used in this section to check the validity of Schiereck's theoretical and experimental line of stability.

In 2001 Scheireck, in the Netherlands, developed a theoretical line of stability. It is $H/\Delta D=3\cos\alpha/\xi$. In the Netherlands the experimental line of stability has been developed from series of laboratory experiments with laboratory setup constructed at 1: 1 (prototype : laboratory set up) ratio. These two lines are the design boundary for the designers of the Netherlands.

Table 5.4 shows the laboratory data along with the Schiereck stability formula. Figure 5.2 gives a comparison between Schiereck's theoretical value and the laboratory data.

The Figure 5.2 also shows the theoretical and experimental lines developed in the Netherlands.

It is seen from the figure that laboratory data has shown satisfactory results with the prototype stability line. Because the laboratory data are shown in the band defined by Schiereck theoretical line and 1:1 prototype experimental lines.

Table 5. 4: Comparison of laboratory data with Schiereck formula

Run No	Water depth	Scale	Slope	Lab. block thickness	Density	Wave period	Wave height	Surf parameter	Laboratory stability	Schiereck stability	Block thickness as per Schiereck	% of deviation
	d		cot α	D	ρ	T	H_s	ξ			$D_{(Schiereck)}$	
	cm		-	cm	gm/cc	sec	cm	-	-		cm	
5	30	1:20	1.5	1	2.12	2.5	6.14	8.402	5.48	0.30	18.27	1727.38(*)
6	30	1:20	1.5	1.25	2.12	2.5	5.00	9.313	3.57	0.27	16.53	1222.75(*)
7	30	1:20	1.5	1.25	2.12	2.4	5.64	8.767	4.03	0.28	17.98	1338.78(*)
8	45	1:10	3	2.5	2.18	1	8.50	1.429	2.88	1.99	3.81	-52.55
9	45	1:10	3	2.5	2.18	1.2	12.00	1.443	4.07	1.97	5.44	-117.55
10	45	1:10	3	2.5	2.18	1.5	13.21	1.719	4.48	1.66	7.11	-184.21
11	45	1:10	3	2.5	2.18	2	15.14	2.141	5.13	1.33	10.16	-306.55

(*) Runs were carried out with smaller scale (1:20) and reflections were not corrected.

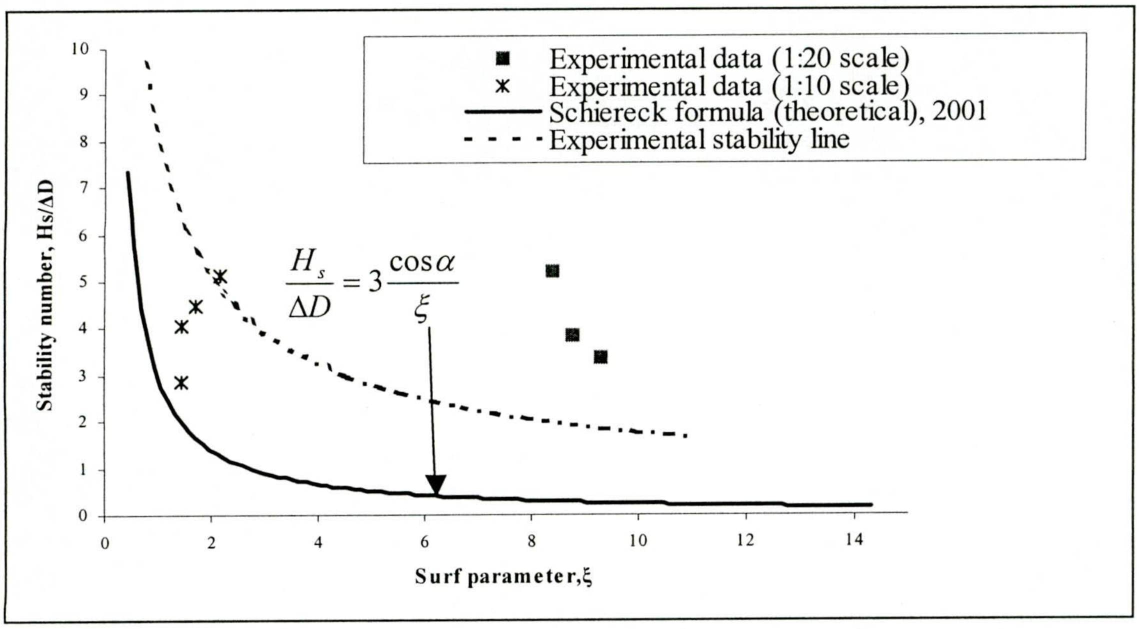


Figure 5. 2: Comparison of laboratory data with theoretical and experimental lines

Table 5.4 shows that block thickness as per Schiereck formula was deviated from laboratory value by 165% (runs with 1:10 scale). This value indicates that the laboratory block thickness should be 1.65 times higher to satisfy stability criterion of Schiereck. Results for 1:20 scale ratios show higher stability than Schiereck and 1:1 prototype experimental line of stability. The cause behind this fact was stated in the previous section. .

5.2.3 Comparison with Van der Meer Formula

Vna der Meer (1988) formula is not comparable for placed blocks system. However to check the validity this formula with the laboratory data has been made for curiosity. In this comparison Van der Meer formula for concrete cube is used. Stability parameter of Van der Meer formula as mentioned in Chapter 3 is a function of damage level and number of waves. If the damage condition is considered as no repair case, then the value of damage level N_{od} becomes 0-0.5. Taking N_{od} as 0.5 Table 5.5 has presented the Van der Meer stability number and corresponding laboratory stability number. Figure 5.3 shows the laboratory data and Van der Meer formula.

In addition to the above analysis Table 5.6 has shown the ratio with Van der Meer formula and experimental data. The table shows that Van der Meer formula is in good agreement and block thickness as per Van der Meer was deviated from laboratory value by 44%. Positive deviation implies that laboratory thickness is good and conservative according to Van der Meer formula. It is important to note that at wave steepness lower than 10.0 x 10^{-3} laboratory data has been plotted below the line of Van der Meer formula. That is the points are in stable zone considering this formula. Here the designer should take care to use Van der Meer formula considering allowable damage levels.

Table 5. 5: Comparison of laboratory data with Van der Meer formula

Run No	Water depth	Wave height	Wave period	Slope	Weight of block	Density	Block thickness	Number of waves	Wave steepness	Laboratory stability	Van der Meer Stability
	d	H_s	T	$\cot\alpha$	W	ρ	D	N	s_{om}		N_{od} = 0.5
	cm	cm	sec	-	gm	gm/cc	cm	-	-		
5	30	6.14	2.5	1.5	7.1	2.12	1.50	80	0.006	3.666	2.626
6	30	5.00	2.5	1.5	13.1	2.12	1.84	80	0.005	2.433	2.681
7	30	5.64	2.4	1.5	13.1	2.12	1.84	80	0.006	2.746	2.649
8	45	8.50	1	3	87.4	2.18	3.42	284	0.054	2.105	1.544
9	45	12.00	1.2	3	87.4	2.18	3.42	237	0.053	2.971	1.619
10	45	13.21	1.5	3	87.4	2.18	3.42	189	0.038	3.272	1.773
11	45	15.14	2	3	87.4	2.18	3.42	142	0.024	3.750	1.991

Table 5. 6: Comparison of block thickness between Laboratory and Van der Meer

Laboratory stability	Van der Meer Stability	Experimental block thickness	Block thickness as per Schiereck	% of deviation
	$N_{od} = 0.5$	D	$D_{(Van\ der\ Meer)}$	
-	-	cm	cm	-
3.666	2.626	1.50	1.50	-49.54
2.433	2.681	1.84	1.83	-46.79
2.746	2.649	1.84	1.83	-46.71
2.105	1.544	3.42	3.61	-44.21
2.971	1.619	3.42	3.61	-44.25
3.272	1.773	3.42	3.60	-44.19
3.750	1.991	3.42	3.60	-44.19

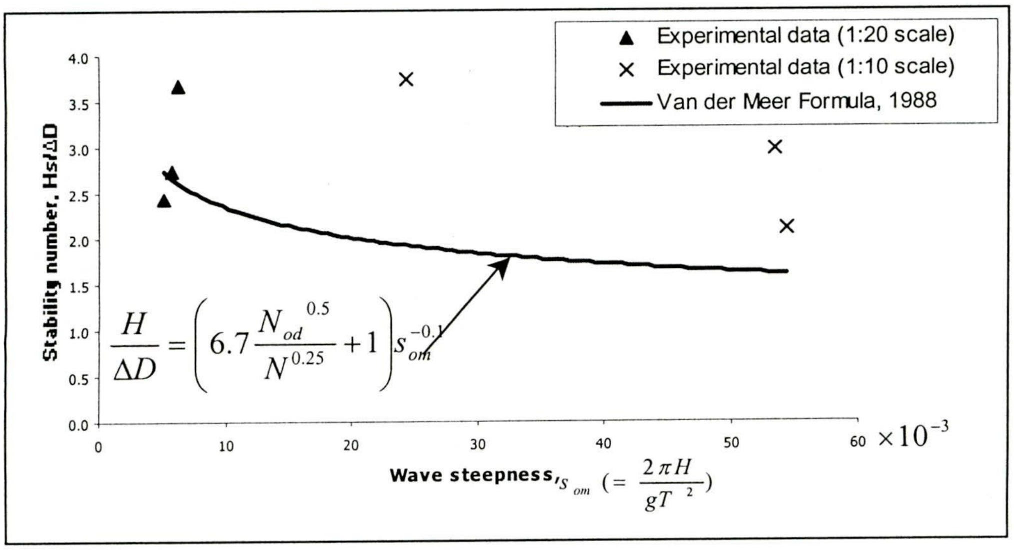


Figure 5. 3: Comparison of Laboratory data with Van der Meer formula

5.2.4 Comparison with Hudson Formula

Hudson formula (1961) is used for stability of dumped stones or cement concrete blocks. There is no research for placed c.c. blocks in one layer in Hudson's work. Hudson's stability coefficient K_D is constant for specific armour layer and is taken as 7.5 considering the modified cubes placing in one layer. For comparison of K_D values from experiments, data have been calculated and are shown in Table 5.7 and in Figure 5.4. It is seen from the table that laboratory runs with 1:20 scale are 5 to 12 times greater than the Hudson's K_D. But K_D values for experimental runs with 1:10 scale are larger than Hudson's value by 1.14 to 6.43 times.

Therefore the laboratory results have shown that Hudson formula is too much conservative as shown in right most column of Table 5.8. Hudson formula gives much higher thickness of blocks considering laboratory K_D value. Therefore this formula should not be justified to design revetment with slab type cement concrete blocks.

Table 5. 7: Hudson's formula comparison with laboratory data

Run No	Water depth	Wave period	Slope	Weight of block	Density	Wave height	$\frac{H}{\Delta D}$	$\frac{W}{\rho g H^3}$ x 10^{-3}	Lab.K_D	Hudson K_D	$\frac{Lab.K_D}{HudsonK_D}$
	d	T	$\cot\alpha$	W	ρ	H					
	cm	sec	-	gm	gm/cc	cm					
10	30	2.5	1.5	7.1	2.12	8.6	1.08	5.27	90.12	7.5	12.02
11	30	2.5	1.5	13.1	2.12	7	0.48	18.02	26.34	7.5	3.51
12	30	2.4	1.5	13.1	2.12	7.9	0.54	12.53	37.86	7.5	5.05
13	45	1	3	87.4	2.18	11.9	0.12	23.79	8.53	7.5	1.14
14	45	1.2	3	87.4	2.18	16.8	0.16	8.46	23.99	7.5	3.20
15	45	1.5	3	87.4	2.18	18.5	0.18	6.33	32.04	7.5	4.27
16	45	2	3	87.4	2.18	21.2	0.21	4.21	48.22	7.5	6.43

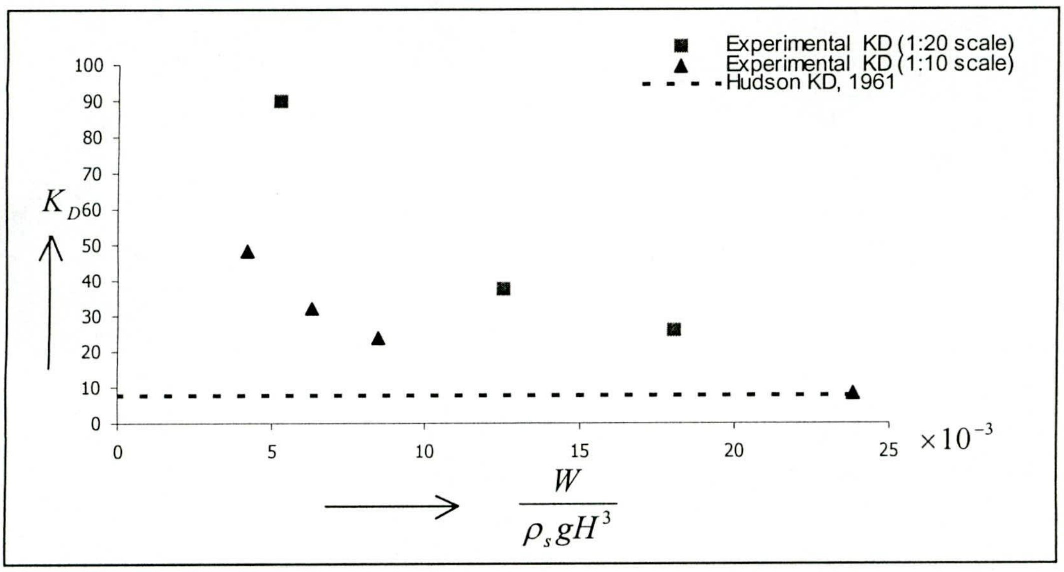


Figure 5. 4: Comparison of Laboratory data with Hudson formula

5.3 Wave run-up

The experimental runs were carried out at two scale ratios as 1:10 and 1:20. Laboratory runs carried out at 1:20 scales has been shown in Table 5.8 and 5.9 and the Tables 5.10 to 5.13 show the results obtained at 1:10 scales. With scale ratio 1:20 the wave heights have been increased gradually and corresponding wave run ups were observed. With this scale, wave period has been fixed at 3.0 sec and 2.5 sec. Wave periods have been fixed at 1.0, 1.2, 1.5 and 2.0 sec for the experimental runs carried out at 1:10 scale.

For placed blocks revetments system Pilarczyk (1990) developed a formula for estimation of wave run up for regular waves expressed as a function of surf parameter as shown in Figure 5.5. In the same figure laboratory data are plotted for comparison.

Table 5. 8: Wave run-up on slope 1(V): 1.5(H), T=3 sec, scale= 1:20, block size = 17.5 x 17.5 x 10 mm^3

Run No	Wave Height	Wave run-up	Laboratory Ru/H	Surf parameter	Pilarczyk Ru/H	Pilarczyk's wave run up	% error
	H	Ru		ξ		Ru	
	cm	cm	-	-			
17	1.9	3.2	1.68	19.34	19.34	25.73	-93.46
18	2.2	3.8	1.73	18.54	18.54	28.55	-90.68
19	2.4	3.6	1.50	17.21	17.21	28.91	-91.28
20	2.8	3.5	1.25	16.43	16.43	32.21	-92.39
21	3.2	4.1	1.28	14.44	14.44	32.34	-91.13

Table 5. 9: Wave run-up on slope 1(V): 1.5(H), T=2.5 sec, scale 1:20, block size = 20 x 20 x 12.5 mm^3

Run No	Wave Height	Wave run-up	Laboratory Ru/H	Surf parameter	Pilarczyk Ru/H	Pilarczyk's wave run up	% error
	H	Ru		ξ		Ru	
	cm	cm	-	-			
22	6	18.5	3.08	8.50	8.50	35.72	-63.74
23	7.1	22.8	3.21	7.82	7.82	38.85	-58.92
24	7.5	24.6	3.28	7.61	7.61	39.93	-56.88
25	9.2	28.6	3.11	6.87	6.87	44.23	-54.73
26	8.6	29.2	3.40	7.10	7.10	61.09	-52.20

Table 5. 10: Wave run-up on slope 1(V): 3(H), T=1.0 sec, scale 1:10, block size = 40 x 40 x 25 mm^3

Run No	Wave Height	Wave run-up	Laboratory Ru/H	Surf parameter	Pilarczyk Ru/H	Pilarczyk's wave run up	% error
	H	Ru		ξ		Ru	
	cm	cm	-	-			
27	7.9	8.6	1.09	1.48	1.48	11.71	-26.56
28	10.58	12.5	1.18	1.28	1.28	13.55	-7.76
29	12.65	13.8	1.09	1.17	1.17	14.82	-6.87
30	13.44	15.8	1.18	1.14	1.14	15.27	3.45

Table 5. 11: Wave run-up on slope 1(V): 3(H), T=1.2 sec, scale 1:10, block size = 40 x 40 x 25 mm^3

Run No	Wave Height	Wave run-up	Laboratory Ru/H	Surf parameter	Pilarczyk Ru/H	Pilarczyk's wave run up	% error
	H	Ru		ξ		Ru	
	cm	cm	-	-			
31	7.9	8.8	1.11	1.19	1.19	9.37	-6.06
32	10.58	10.6	1.00	1.02	1.02	10.84	-2.22
33	12.65	11.8	0.93	0.94	0.94	11.85	-0.46
34	13.44	12	0.89	0.91	0.91	12.22	-1.79

Table 5. 12: Wave run-up on slope 1(V): 3(H), T=1.5 sec, scale 1:10, block size = 40 x 40 x 25 mm^3

Run No	Wave Height	Wave run-up	Laboratory Ru/H	Surf parameter	Pilarczyk Ru/H	Pilarczyk's wave run up	% error
	H	Ru		ξ		Ru	
	cm	cm	-	-			
35	7.9	13.5	1.71	1.78	1.78	14.05	-3.93
36	10.58	15.6	1.47	1.54	1.54	16.26	-4.07
37	12.65	17.5	1.38	1.41	1.41	17.78	-1.58
38	13.44	18.5	1.38	1.36	1.36	18.33	0.94

Table 5. 13: Wave run-up on slope 1(V): 3(H), T=2.0 sec, scale 1:10, block size = 40 x 40 x 25 mm^3

Run No	Wave Height	Wave run-up	Laboratory Ru/H	Surf parameter	Pilarczyk Ru/H	Pilarczyk's wave run up	% error
	H	Ru		ξ		Ru	
	cm	cm	-	-			
39	7.9	18.2	2.30	2.22	2.22	17.56	3.62
40	10.58	21.5	2.03	1.92	1.92	20.33	5.77
41	12.65	22	1.74	1.76	1.76	22.23	-1.02
42	13.44	22.8	1.70	1.70	1.70	22.91	-0.48

It is seen from the tables and figure that for surf parameters less than 2 ($\xi<2.0$) the agreement is good and the percentage of error is 3.06. As the surf parameter increases the percentage of errors is greater. It appears that scale effects have great influence in the results. It is also noted that experimental runs with 1:10 scale display good results. But the experiments with 1:20 scale have shown greater variation with the Pilarczyk line which is around 74.75% error. Since frictional effect in smaller scale ratio may be greater, measured wave run up has become unlikely lower.

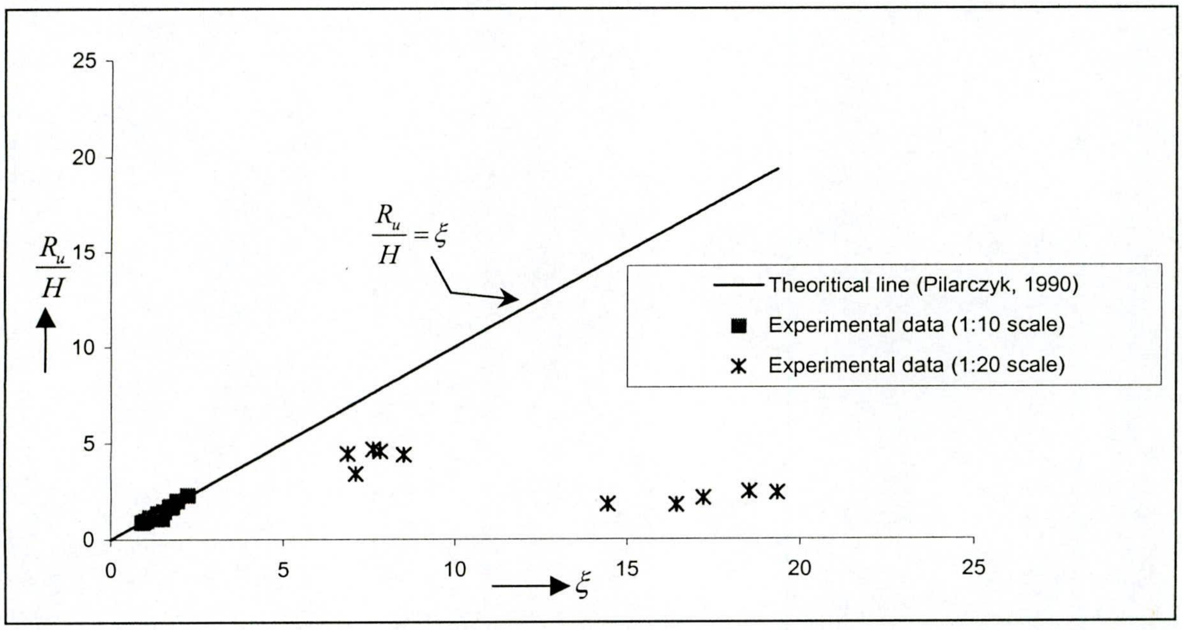


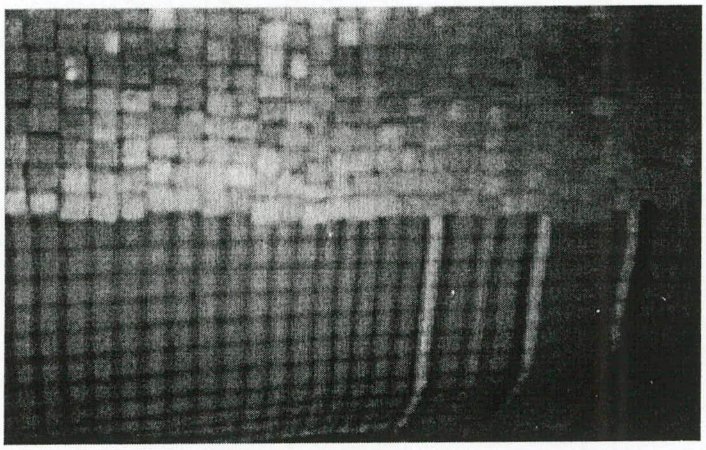
Figure 5.5: Laboratory data of Wave run up

The method of SPM has been described to estimate wave run up at different slope and expressed as waves steepness parameter by using design curves developed experimentally. In this method beach slope is a factor that is not possible to compare with the laboratory results since there is no beach slope. The laboratory runs of the present study have been conducted at two slopes and at different scales. Since wave run up data collection by changing slope is too extensive work to continue within the scope of the study. Moreover wave run up compared at different slopes is outdated. Now a days surf parameter is used to express wave run up and other wave characteristics.

5.4 Behaviour of C.C. Blocks with Geotextiles

This study has observed the behaviour of c.c. blocks over geotextiles. Laboratory data for this purpose have been discussed in Section 5.2. Table 5.2 has shown the experimental runs with sub soil to understand damage development with geotextiles.

In the laboratory flume the failure of blocks used as wave protection works was catastrophic when geotextiles were laid below the blocks. Blocks were placed in a layer. Resisting forces of blocks are friction between block and the surface on which it rests. When a block was displaced from its position, the above blocks did not get any resistance from toe side. Then damage has initiated and propagated after subsequent wave actions. Thus total protection works with blocks experiences catastrophic failure. The Photograph 5.1 has shown a catastrophic failure of blocks in which thin clothe has been observed clearly.



Photograph 5.1: Catastrophic damage of blocks with geotextiles

Shaheli(2002) has studied behaviour of randomly placed block and uniformly placed block over geotextiles. Her study revealed that for uniformly placed blocks catastrophic failure was observed. That is, the present study agreed that result.

5.5 Influence of Wave Frequency in Selection of C.C. Blocks

A functional relationship has been developed where all parameters of Hudson formula have been incorporated together with wave frequency (inverse of wave period). The functional relationship was expressed as $f(H,W,T,g,\rho_s,\rho_w,\alpha)=0$. By applying Buckingham Pi theorem a functional relationship is established as

$\frac{W}{\rho_s g H^3} = f\left(\frac{\rho_s}{\rho}, \frac{H}{gT^2}, \cot\alpha\right)$ .Table 5.14 shows the experimental data for this purpose.

Table 5. 14: Experimental data for influence of wave frequency in selection of c.c blocks

Run No	Wave height	Block size	Wave period	Slope	Weight of a block	Density of block	$\frac{W}{\rho_s gH^3}$	$\frac{H}{gT^2}$
	H		T	cotα	W	ρ_s		
	cm	mm x mm x mm	sec	-	gm	gm/cc	-	-
trial	2.8	17 .5 x 17.5 x 10	3.3	1.5	7.1	2.12	15.26	0.26
5	8.6	17 .5 x 17.5 x 10	2.5	1.5	7.1	2.12	0.53	1.40
6	7	20 x 20 x 12.5	2.5	1.5	13.1	2.12	1.84	1.14
7	7.9	20 x 20 x 12.5	2.4	1.5	13.1	2.12	1.25	1.29

In the table the experimental data were obtained for a constant slope of cotα = 1.5. Blocks of sizes of 17.5mmx17.5mm x10 mm and 20 mm x20 mm x12.5 mm were used It is noted that one trial run has been conducted for this study. The critical wave height obtained in this case was 2.8 cm. This height has also been incorporated in this table.

Figure 5.6 shows power function, which has a constant value 1.581 and exponent of H/gT^2 is -1.734. These coefficient and exponent are an indication of the influence of the wave period that is comparable to stability formulae.

The pattern of the trend line agrees with other research results like Pilarczyk, Van der Meer etc. This result may be a strong base to modify Hudson formula incorporating wave frequency (wave period).

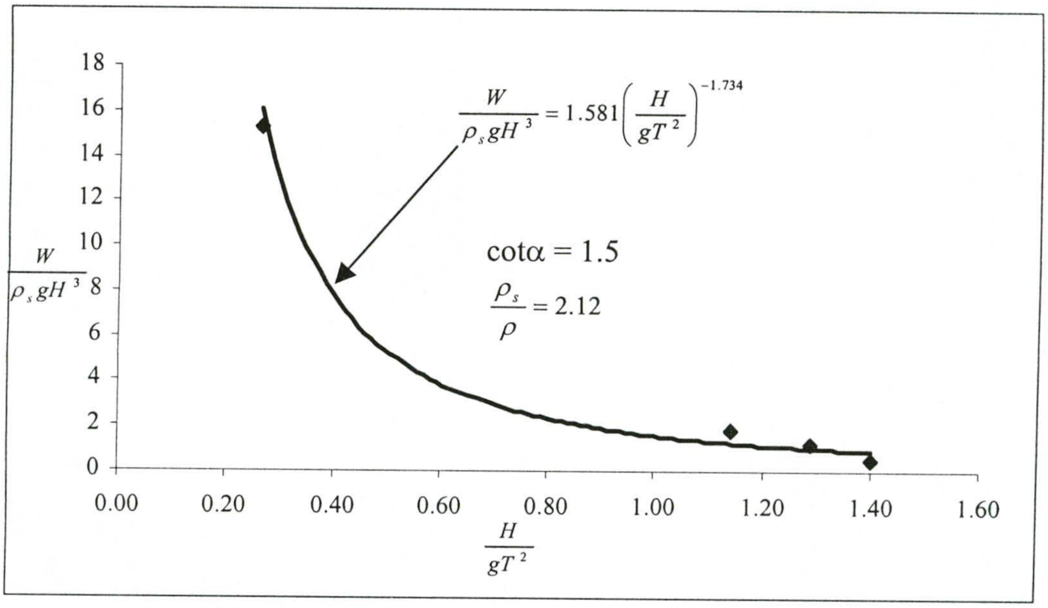


Figure 5.6: Influence of wave frequency in selection of c.c blocks

Chapter 6
Conclusion and Recommendation

6.1 Introduction

Summary and conclusion of the present study can be stated as follows.

6.2 Conclusion

1) The specific gravity of c.c. blocks with 1:2:4 (cement: sand: aggregate) proportions studied in the experiments was between 2.12 and 2.18 whereas the design practice by BWDB and others is to use the value as 2.24 which means under estimation of block dimensions.

2) The existing flume (21.34 m long, 0.76m wide and 0.76 m deep) of Hydraulic and River Engineering Laboratory of WRE, BUET produces significant reflection of waves. This is a constraint in the study of wave erosion with the equipment. However a "Refreg" program written in Matlab5.3 can separate this reflection component successfully. The range of reflection in the present case was found to vary between 15% to 28%.

3) Since the prototype clamping effect and friction between blocks cannot be scaled down in the laboratory flume, experiments with smaller scale than 1:10 (prototype: laboratory standard) may not produce satisfactory results.

4) Comparison of laboratory results with the present day widely used design formula of Pilarczyk (1990) has shown that the agreement is reasonably satisfactory at least for surf range less than two ($\xi<2.0$). This surf parameter is the normal range of the wave climate in haor areas of Bangladesh.

5) Laboratory results for 1:10 scales agreed well with recommended band suggested by Schiereck(2001) and 1:1 prototype experimental values.

6) Comparison of the results with Hudson formula does not seem to be logical because the formula is meant for dumped blocks system. However from a preliminary analysis it shows that the value of K_D which is taken equal to 7.5 as constant may be much smaller in the case of placed block system.

7) Van der Meer proposed formula for dumped blocks design for various ranges of maintenance requirement. Considering no repair condition ($N_{od} = 0.5$), that is, allowing no damage in the design for c.c. blocks, Laboratory results shows that the block sizes are over designed by around two times.

8) Results on wave run up have been compared with surf parameters and agreed Pilarczyk (1990) formula with percentage of error only 3.06 for experimental with 1:10 scale . The discrepancy is however greater for experimental runs done with 1:20 scale.

9) The failure mechanism of c.c. blocks over geotextiles was catastrophic. As soon as the displacement of one block started it propagated down ward very quickly.

10) Analysing the wave parameters by Buckingham Pi theorem a relationship can be established as follows,

$$\frac{W}{\rho_s g H^3} = k\left(\frac{H}{gT^2}\right)^n$$

This is similar to Hudson formula with an inclusion of the wave period as an influencing parameter. For 1:1.5 slope the values of the exponent 'n' came as -1.734 and constant 'k' as 1.581. However this needs further investigation with other slopes.

6.3 Recommendations

1) Since wave run up studies is not restricted by friction between the blocks, the present studies can be extended with wider range of surf parameters. Only three types of blocks sizes and two bank slopes have been included in the present study. Studies with other block sizes, bank slopes with wider range of surf parameters, that is, wave height, wave steepness etc will therefore be useful.

2) In the same way the influence of wave period (modified Hudson formula) can also be extended for other slopes and blocks sizes.

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Annex -A

Step wise procedure of wave generator operation:

- ***Desk Works:***

1. From wind speed and fetch length wave height can be obtained by following wave forecasting formula or using nomograms (SPM, 1984). Then model T and h has been fixed.
2. Find ω by following the formula $w = \frac{2\pi}{T}$ from T and determine a dimensionless wave parameter $\frac{\omega^2 h}{g}$.
3. Set e and f from Figure A.2 and find $\frac{f+e}{f}$

- ***Setting Wave Generator:***

4. Mark h on the side glass of flume
5. Empty the flume if there is water
6. Turn on the switch of wave generator
7. Fix frequency of wave generator as slow as possible by rotating dial (don't change frequency while it is at rest).
8. Make the vertical arms perfectly vertical (see Figure A.1) for pure translation.
9. Keep the vertical arms apart from each other as possible for pure rotation.
10. Measure f at bottom and f+e on marked line (desired water level).
11. Find $\frac{f+e}{f}$ and compare with the value obtained in step 3. If it does not satisfy adjust vertical arms to alter translation and rotation of paddle.
12. Turn the switch off.

- ***Start runs:***

13. Pour water in the flume up to desired water level.

14. Turn the switch on and quickly increase frequency of wave generator by rotating dial.
15. Measure frequency of wave generator. If it is not satisfied then adjust frequency by rotating dial.

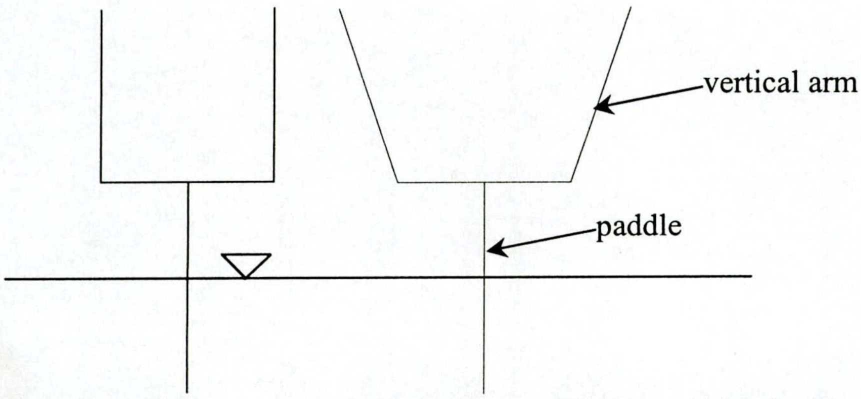

Figure A.1: Line sketch of wave generator (pure translation is shown in left sketch)

Parameters:

T = wave period

h = water depth

ω = angular frequency

f = translation of paddle of wave generator

e = rotation of paddle of wave generator

Formulae:

1. $\omega = \dfrac{2\pi}{T}$

Example:

Model wave period, T is 1.0 sec and depth of water, h is 45 cm. Then dimension less parameter becomes 1.81. From Figure A.2, e and f have been obtained as 0.77 and 0.14 respectively.

The ratio $\frac{e+f}{f}$ is then obtained as 6.5 from this example. In the laboratory wave generator has been adjusted by trial and error such that e+f has been obtained as 13.20 and f as 2.02. Then (e+f)/f has been obtained as 6.53 .

Thus the setup has been completed.

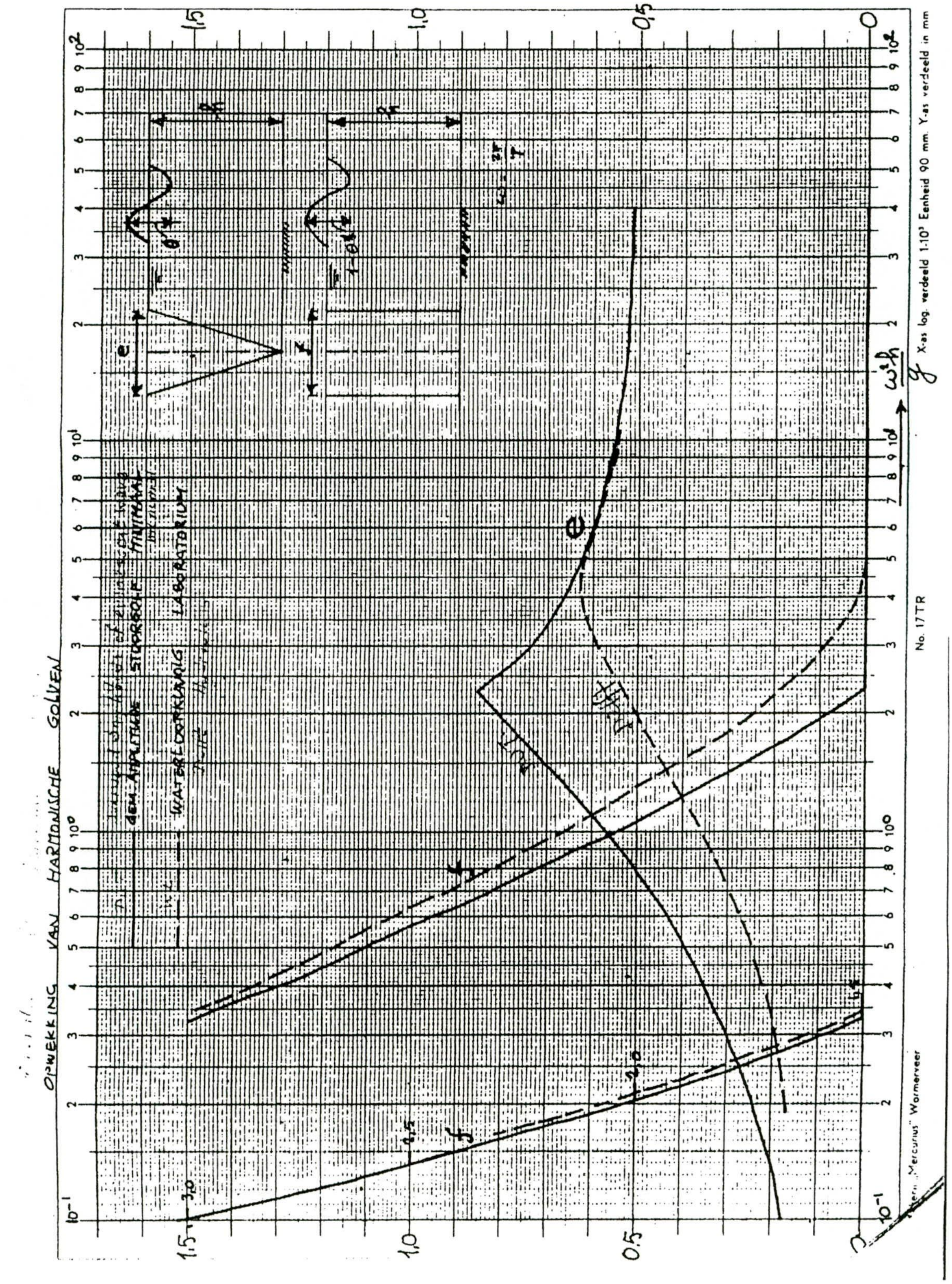

Figure A.2: Graph to get value of e and f

Annex -B

Short description of the program Refreg:

To calculate the reflection of a regular wave, a Matlab program Refreg has been written in the Laboratory of Fluid Mechanics. The method used has been described by Goda and Suzuki (1976), see Goda (1985). In this method two wave gauges are used at a distance of about one fourth of the wave length.

1. Input parameters

Parameters to be typed via dialog boxes :

input file (mouse click),
file properties Labview file in ASCII-format or another ASCII-file,
number of header lines max. number of samples to be read, or all,
time step,
number of harmonic components to be printed,
water depth in m,
first and last sample to be analysed,

for the two wave gauges:
column number of the wave gauge series in the file,
position of the wave gauge in m.
scale factor of measurements from volts to meters.
To get a reliable value of the wave period, the time step must be small enough (at maximum 0.05 times the wave period).

2. Subtraction of mean value

3. Determination of the length of the signal

In the input part of the program, the user specified the part of the data that can be used in the calculation. From the begin and endpoint of this part, two zero crossings with equal sign are searched. This is done for the first wave gauge. The length of the series between the two zero crossings enables the program to find the correct base period from the FFT.

4. Checking the time step

Generally, a period does not contain an integer number of samples. To reduce the error caused by this fact, use at least 20 points per period. If less than 20 points are present, the program will send a warning to the user.

5. Calculating the first harmonic using the FFT of Matlab

The frequency having the maximum C-coefficient will be taken, where $C = \pi(A^2 + B^2)$.

6. Calculating the amplitude and the phase of the incoming and the reflected wave

Equation (6) has the form $L\underline{a} = \underline{b}$, where $\underline{a}$ and $\underline{b}$ are vectors and L is a matrix. This equation system can be solved in Matlab directly.
The wave number k is determined by the dispersion relation in case of free gravitation surface waves is used:

$$\omega = \sqrt{gk \tanh(kh)},$$

where T is the angular frequency and g the gravitational acceleration constant. This is done by the Matlab-function Disper, written by Gert Klopman, is used.

Program used to test Refreg

To test Refreg, a program 'testsig' has been written. In this program, an incoming and a reflected wave with one frequency are generated. The user must specify the following parameters:

name of the output file,

number of wave periods to be generated,

time step,
wave period,
water depth,
distance between water gauges,
reflection coefficient,
phase of the incoming wave,
phase of the reflected wave,
amplitude of normally distributed random noise.

The program has been tested with an amplitude of 1 for the incoming wave (To get an answer in meters, in the program Refreg a scaling factor may be used).

The random noise has been used only to investigate the influence of noise on the frequency of the first harmonic found. If the time step is small, this kind of noise can influence the frequency found, since the zero crossings with equal sign can be shifted by half a period if the frequency of the noise has the order of half the sampling frequency. This kind of noise will normally not be found in waves. It is better to use another kind of disturbance if testing the program. It may be useful to add higher harmonic components to the test signals in the future.

Things to be investigated

1. The influence of the time step, the duration of the measurements, and the wave period on the results.
2. A method to detect the wave period from the measurements, e.g. a regression method.
3. The influence of perturbations, e.g. due to the equipment or caused by undesired objects in the measurement system.

A sensitivity analysis of the results. If some noise of a period of about two times the time step is present, an error may be found in the first harmonic component found from the zero crossings. This kind of noise is not expected in surface waves. On the other hand a small shift of the wave gauges can change the results.

A sensitivity analysis can be done by cross correlation methods.

Additional program points in the future

1. The water velocity as a parameter (now: velocity 0).

2. Adding more (higher) harmonics to the calculation, and splitting into bounded and free components. To be able to find the reflection of more harmonics, more than two wave gauges are needed.

Annex –C

Tables

Table C. 1: Block sizes of the experimental runs

Prototype block size	Laboratory block size	Laboratory block size	Material
--	**at 1:10 scale**	**at 1:20 scale**	**cement: aggregate: sand**
mm x mmx mm	mm x mmx mm	mm x mmx mm	1:2:4
350 x 350 x 250	-	17.5 x 17.5 x 10	1:2:4
400 x 400 x 250	40 x 40 x 25	20 x 20 x 12.5	1:2:4

Table C. 2:Specific gravity of samples of 40x40x25 mm^3 block

Room temperature ,T = 29^{0}C
Block sizes = 40mmx40mmx25mm

Sample no	mass	initial volume	final volume	volume of water	specific gravity	mean specific gravity
	gm	Cc	cc	cc		
(1)	(2)	(3)	(4)	(5)	(7)	(8)
1	88.7	530	570	40	2.23	
2	87.2	570	612	42	2.08	**2.18**
3	86.2	612	651	39	2.22	

Table C. 3: Specific gravity of samples of 17.5x17.5x10 mm^3 block

Room temperature ,T = 25^{0}C
Block sizes = 17.5mmx17.5mmx10mm

Sample no	Mass	Initial volume	final volume	Volume of water	specific gravity	mean specific gravity
	gm	cc	cc	cc		
(1)	(2)	(3)	(4)	(5)	(7)	(8)
1	7.6	200	203.6	3.6	2.10	
2	7.1	203.6	207	3.4	2.08	**2.12**
3	6.5	207	210	3	2.16	

Table C. 4: Specific gravity of samples of 20x20x12.5 mm^3 block

Room temperature ,T = 25^{0}C
Block sizes = 20mmx20mmx12.5mm

Sample no	Mass	Initial volume	final volume	Volume of water	specific gravity	mean specific gravity
	gm	cc	cc	cc		
(1)	(2)	(3)	(4)	(5)	(7)	(8)
1	12.7	500	506	6	2.12	
2	13.4	506	512	6	2.23	**2.12**
3	13.2	512	518.5	6.5	2.02	

Table C. 5: Wave growth Calculation (depth of water = 3m and fetch = 5km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.40	0.56	2.24	57.28
20	0.73	1.03	3.05	46.66
30	0.98	1.37	3.61	40.39
40	1.18	1.65	4.06	36.09
50	1.34	1.87	4.43	32.89
60	1.48	2.07	4.75	30.37

Table C. 6: Wave growth Calculation (depth of water = 3m and fetch = 10 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.48	0.67	2.51	74.84
20	0.81	1.13	3.38	59.42
30	1.03	1.44	3.97	50.41
40	1.21	1.69	4.43	44.33
50	1.36	1.90	4.81	39.87
60	1.49	2.09	5.14	36.41

Table C. 7: Wave growth Calculation (depth of water = 3m and fetch = 20 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.54	0.75	2.77	94.12
20	0.84	1.18	3.68	72.61
30	1.05	1.47	4.29	60.26
40	1.22	1.71	4.75	52.11
50	1.37	1.91	5.13	46.24
60	1.50	2.09	5.45	41.77

Table C. 8: Wave growth Calculation (depth of water = 3m and fetch = 30km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.56	0.79	2.91	105.54
20	0.85	1.20	3.84	80.00
30	1.06	1.48	4.45	65.53
40	1.22	1.71	4.90	56.12
50	1.37	1.91	5.28	49.43
60	1.50	2.09	5.59	44.39

Table C. 9: Wave growth Calculation (depth of water = 3m and fetch = 40km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.58	0.81	3.00	113.43
20	0.86	1.20	3.94	84.91
30	1.06	1.48	4.54	68.93
40	1.22	1.71	5.00	58.65
50	1.37	1.91	5.36	51.40
60	1.50	2.09	5.68	45.97

Table C. 10: Wave growth Calculation (depth of water = 3m and fetch = 50km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.59	0.82	3.07	119.32
20	0.86	1.20	4.01	88.46
30	1.06	1.48	4.61	71.34
40	1.22	1.71	5.06	60.41
50	1.37	1.91	5.42	52.75
60	1.50	2.09	5.73	47.04

Table C. 11: Wave growth Calculation (depth of water = 3m and fetch = 100 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.60	0.84	3.24	135.70
20	0.86	1.21	4.19	97.81
30	1.06	1.48	4.77	77.38
40	1.22	1.71	5.21	64.67
50	1.37	1.91	5.56	55.93
60	1.50	2.09	5.86	49.51

Table C. 12: Wave growth Calculation (depth of water = 4m and fetch = 5km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.43	0.60	2.30	60.64
20	0.82	1.15	3.15	50.39
30	1.14	1.59	3.76	44.21
40	1.40	1.96	4.24	39.90
50	1.61	2.26	4.64	36.66
60	1.80	2.52	4.99	34.09

Table C. 13: Wave growth Calculation (depth of water = 4m and fetch = 10 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.52	0.73	2.59	80.62
20	0.94	1.31	3.53	65.60
30	1.24	1.73	4.17	56.52
40	1.47	2.06	4.68	50.27
50	1.67	2.34	5.10	45.62
60	1.84	2.58	5.46	41.97

Table C. 14: Wave growth Calculation (depth of water = 4m and fetch = 20 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.61	0.86	2.89	103.45
20	1.01	1.42	3.88	82.15
30	1.29	1.81	4.55	69.35
40	1.51	2.11	5.07	60.69
50	1.69	2.37	5.49	54.36
60	1.85	2.60	5.86	49.46

Table C. 15: Wave growth Calculation (depth of water = 4m and fetch = 30km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.66	0.92	3.05	117.49
20	1.04	1.46	4.08	91.87
30	1.30	1.82	4.75	76.59
40	1.51	2.12	5.27	66.39
50	1.69	2.37	5.69	59.01
60	1.86	2.60	6.05	53.36

Table C. 16: Wave growth Calculation (depth of water = 4m and fetch = 40km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.68	0.95	3.16	127.46
20	1.05	1.47	4.20	98.55
30	1.31	1.83	4.88	81.44
40	1.51	2.12	5.39	70.12
50	1.69	2.37	5.81	61.99
60	1.86	2.60	6.17	55.81

Table C. 17: Wave growth Calculation (depth of water = 4m and fetch = 50km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.70	0.97	3.24	135.07
20	1.06	1.48	4.29	103.52
30	1.31	1.83	4.97	84.96
40	1.51	2.12	5.48	72.78
50	1.69	2.37	5.90	64.09
60	1.86	2.60	6.25	57.53

Table C. 18: Wave growth Calculation (depth of water = 4m and fetch = 100 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.73	1.02	3.45	157.05
20	1.07	1.49	4.52	117.21
30	1.31	1.84	5.20	94.30
40	1.52	2.12	5.70	79.62
50	1.69	2.37	6.10	69.35
60	1.86	2.60	6.44	61.71

Table C. 19: Wave growth Calculation (depth of water = 5m and fetch = 5km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.44	0.62	2.33	62.96
20	0.88	1.23	3.22	53.05
30	1.25	1.75	3.85	46.97
40	1.57	2.20	4.36	42.70
50	1.84	2.57	4.79	39.46
60	2.07	2.90	5.16	36.88

Table C. 20: Wave growth Calculation (depth of water = 5m and fetch = 10 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.55	0.77	2.65	84.70
20	1.03	1.45	3.63	70.13
30	1.40	1.97	4.31	61.09
40	1.70	2.38	4.85	54.80
50	1.94	2.72	5.30	50.06
60	2.16	3.02	5.69	46.32

Table C. 21: Wave growth Calculation (depth of water = 5m and fetch = 20 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.66	0.93	2.97	110.23
20	1.15	1.61	4.03	89.39
30	1.50	2.09	4.75	76.43
40	1.76	2.47	5.31	67.51
50	1.99	2.78	5.77	60.90
60	2.19	3.06	6.16	55.74

Table C. 22: Wave growth Calculation (depth of water = 5m and fetch = 30km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.72	1.01	3.14	126.35
20	1.20	1.68	4.25	101.10
30	1.52	2.13	4.98	85.43
40	1.78	2.49	5.54	74.76
50	2.00	2.80	6.01	66.92
60	2.19	3.07	6.40	60.86

Table C. 23: Wave growth Calculation (depth of water = 5m and fetch = 40km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.76	1.06	3.27	138.01
20	1.22	1.71	4.39	109.33
30	1.54	2.15	5.13	91.61
40	1.79	2.50	5.70	79.63
50	2.00	2.80	6.16	70.90
60	2.19	3.07	6.55	64.19

Table C. 24: Wave growth Calculation (depth of water = 5m and fetch = 50km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.78	1.09	3.36	147.03
20	1.24	1.73	4.50	115.58
30	1.54	2.16	5.24	96.19
40	1.79	2.50	5.80	83.19
50	2.00	2.80	6.26	73.76
60	2.19	3.07	6.65	66.57

Table C. 25: Wave growth Calculation (depth of water = 5m and fetch = 100 km)

Wind speed	Significant wave height	Regular wave height	Regular time period	Time to develop
u	Hs	H	T	t
m/s	m	m	sec	min
10	0.84	1.17	3.61	173.86
20	1.26	1.76	4.78	133.38
30	1.55	2.17	5.53	108.81
40	1.79	2.51	6.08	92.69
50	2.00	2.80	6.53	81.23
60	2.19	3.07	6.90	72.62