EXPERIMENTAL STUDY ON THE WAVE INTERACTION WITH SUBMERGED GEO-TUBE BREAKWATER IN FRONT OF CEMENT CONCRETE BLOCK REVETMENT

M.Sc. Engineering Thesis

By

MD. KHAIRUL HASAN

Student ID: 0416162002 P

DEPARTMENT OF WATER RESOURCES ENGINEERING
BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY
DHAKA-1000, BANGLADESH

JUNE 2021

EXPERIMENTAL STUDY ON THE WAVE INTERACTION WITH SUBMERGED GEO-TUBE BREAKWATER IN FRONT OF CEMENT CONCRETE BLOCK REVETMENT

Submitted

By

Md. Khairul Hasan

(Student ID: 0416162002 P)

In partial fulfillment of the requirement for the degree of

MASTERS OF SCIENCE IN WATER RESOURCES ENGINEERING

DEPARTMENT OF WATER RESOURCES ENGINEERING
BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY
DHAKA-1000, BANGLADESH

JUNE 2021

Certification of Thesis

The thesis titled "EXPERIMENTAL STUDY ON THE WAVE INTERACTION WITH SUBMERGED GEO-TUBE BREAKWATER IN FRONT OF CEMENT CONCRETE BLOCK REVETMENT", submitted by Md. Khairul Hasan, Student ID. 0416162002P, Session April 2016, to the Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, has been accepted as satisfactory in partial fulfillment of the requirements for the degree of Master of Science in Water Resources Engineering and approved as to its style and content. Examination held on June 05, 2021.

Dr. Md. Ataur Rahman — Chairman
Professor
Department of Water Resources Engineering
Bangladesh University of Engineering and Technology
Dhaka

Dr. Anika Yunus — Member (Ex-Officio)
Professor and Head
Department of Water Resources Engineering
Bangladesh University of Engineering and Technology
Dhaka

Dr. K.M. Ahtesham Hossain — Member
Assistant Professor
Department of Water Resources Engineering
Bangladesh University of Engineering and Technology
Dhaka

Dr. Jiban Kumar Sarker, PEng. — Member (External)
Superintending Engineer
Design Circle -04
Bangladesh Water Development Board (BWDB)
Dhaka

Candidate's Declaration

It is hereby declared that this thesis or any part of it has not been submitted elsewhere for the award of any degree or diploma.

Candidate

Khairul

Md. Khairul Hasan
Student ID: 0416162002P

ACKNOWLEDGEMENT

I express my gratitude and homage to the Almighty Allah for providing me the knowledge and capability to successfully complete the research work.

I would like to take this opportunity to express my sincere gratitude to Dr. Md. Ataur Rahman, Professor, Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, for his advice and guidance throughout this study as well as his constant encouragement and productive criticism in my endeavors. I consider myself fortunate for getting the opportunity to work under his supervision.

My sincere appreciation is due to Dr. Anika Yunus, Professor and Head, Department of Water Resources Engineering, BUET, for his timely co-operation and guidance. I also intend to express my gratitude to Dr. K.M. Ahtesham Hossain, Assistant

Professor, Department of Water Resources Engineering, BUET, for his valuable comments, careful review and suggestions. I am also grateful to Dr. Jiban Kumar Sarker, PEng., Superintending Engineer, Design Circle–04, Bangladesh Water Development Board, Dhaka, for his kind consent to be a member of the examination board. His precious comments, constructive criticism and suggestions in this study are duly appreciated.

I am very much grateful to T M Tanver Hossain, undergraduate student, Department of WRE, BUET, for his sincere assistance in the collection of data during the laboratory experiments. I also acknowledge the great help and support provided by all laboratory staff of Hydraulics and River Engineering Laboratory, DWRE, BUET.

I also would like to express my gratitude to Faisal Mohammad Alvee, Lecturer, Department of Water Resources Engineering, BUET, Md. Abul Kausar, Additional Chief Engineer, Design (Retd.) Md. Abdul Awal Islam, Sub-Divisional Engineer, BWDB for their continuous help, support and encouragement throughout the study.

Last but not the least; I would like to express my gratitude to my parents for their incessant inspiration and encouragement. The sacrifices, patience, moral support and help from my wife Mst. Marzia Akter Chowdhury, are really invaluable and I am really thankful to her for always being there for me.

Finally, I would like to express my sincere gratitude to all other teachers and members of the Water Resources Engineering Department, BUET, for their cooperation and help in the successful completion of the work.

Md. Khairul Hasan

ABSTRACT

Engineering advancements on coastline protection have prompted more noteworthy counter-measures to protect the shorelines. Revetment and breakwater are the common counter-measures that are widely used to protect the shorelines from wave attack by damping the wave energy. Though individually, their effects on wave energy damping are well defined, their combined effects are poorly studied. To address this gap, the interaction between waves and submerged geo-tube breakwater in front of cement concrete block revetment has been investigated experimentally. To do this, a set of experiments have been carried out at still water depth, h_w = 50 cm with the submerged geo-tube breakwater of four different breakwater heights, h_b = 15 cm, 20 cm, 25 cm and 30 cm for four different wave periods, T = 1.7 sec, 1.8sec, 1.9 sec and 2.0 sec in a two-dimensional wave flume. A revetment consist of cement concrete blocks was constructed at the end of the flume with a slope of 4(H):1(V). Wave height (H) and wave period (T) were downscaled and Pilarczyk equation was used to determine the size of cement concrete block. The downscaled block size was 2.0 cm x 2.0 cm x 2.0 cm. The stability of the revetment against transmitted wave height was investigated. Though the incident wave heights, H_i = 11 cm, 12 cm, 14 cm, and 15 cm were larger than design wave height, H_s = 9.5 cm, only for three experimental runs the revetment failed partially among the sixteen experimental runs. For other experimental runs, the revetment was stable due to the reduction of incident wave height by the submerged breakwater. From the experimental investigations, it has been seen that for any particular wave period the relative breakwater height (h_b/h_w) and the relative breakwater width (breakwater width/wave length, B/L) are the important parameters for the reduction of incident wave height. As the relative breakwater height (h_b/h_w) increases, the transmitted wave reduces more due to breaking caused by the breakwater for any particular wave period. In this experiment, the highest reduction occurs for 60% submergence of the breakwater for any particular wave period. Also as the relative breakwater width (B/L) increases, the reduction of wave height also increases for any particular relative breakwater height. In this experiment, the highest reduction occurs, when breakwater width is 30% of the wave length for any particular relative breakwater height. A relationship among transmission coefficient (K_t), relative breakwater height (h_b/h_w), wave period (T) and significant wave height (H_s) has been established ($K_t = 1.05 - 0.67(\frac{h_b}{h_w})^{0.83} - 2.4 \times 10^{-4}(\frac{gT^2}{H_s})$) from the experimental results. This relationship will be helpful to design revetment at the shore and breakwater at shallow water for larger wave heights.

Table of Contents

List of Figures

List of Tables

List of Symbols

B	Breakwater width (m)
b	Exponent related to the interaction process between waves and revetment
D	Thickness of the cover layer (m)
g	Acceleration due to gravity
H_i	Incident wave height (m)
H_s	Significant wave height (m)
H_t	Transmitted wave height (m)
h_b	Breakwater height (m)
h_w	Water depth (m)
K_t	Transmission coefficient
T_m	Average wave period (sec)
W	Weight of revetment material (kg)
ρ_s	Density of protection material (kg/m^3)
ρ_w	Density of water (kg/m^3)
Δm	Relative density of submerged material = $(\rho_s - \rho_w)/\rho_w$
α	Bank normal slope, (°)
ψ_u	System determines stability upgrading factor
φ_{sw}	Stability factor for incipient motion
ξ_z	Wave breaker similarity parameter

CHAPTER 1

INTRODUCTION

1.1 General:

The coastal zone is a dynamic area comprising the natural boundary between land and ocean. The coastal zone of our country is situated in the most northern part of the Bay of Bengal. The Coastal Zone is one of the most important parts of any country because of its contribution to the economy and biodiversity (Williams *et al.*, 2018). The coastal zone of Bangladesh covers a land area of 47,201 sq. km. (Kamal and Rob, 2003). This area provides shelter, sustenance, and livelihood for approximately 46 million people, with 2.85 million hectares of cultivable land supporting 20% of the rice production of Bangladesh (Kamal and Rob, 2003). The total length of the coastline of Bangladesh is about 710 km (Islam and Ahmad, 2004).

Coastal Zone Policy 2005 defines that the coastal zone of Bangladesh consists of nineteen districts comprising one hundred and forty-seven Upazilas (Coastal Zone Policy, 2005). Among them a total of forty-eight Upazilas are the exposed coast,s and the remaining ninety-nine Upazilas are interior coasts (Coastal Zone Policy, 2005).

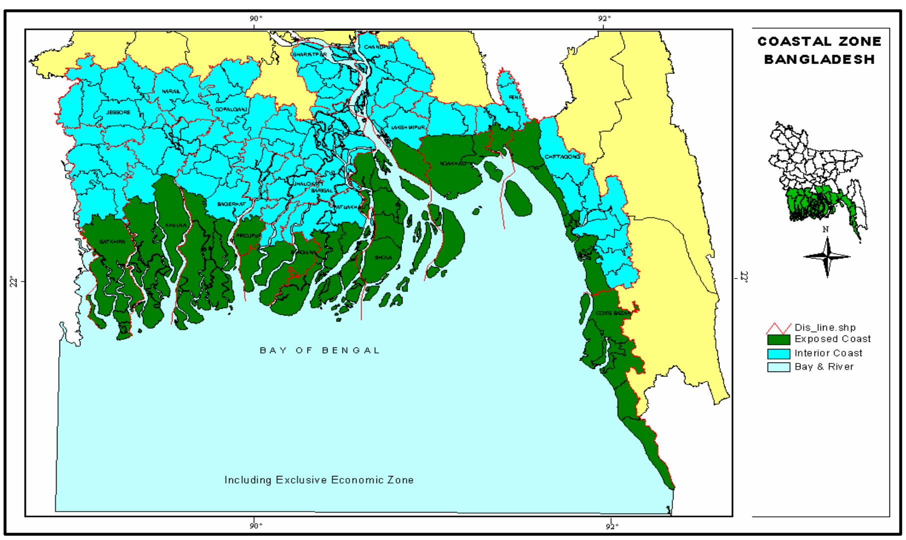


Figure 1.1: Coastal Zone of Bangladesh (Islam and Ahmad, 2004)

The three basic natural system processes that govern opportunities and vulnerabilities of the coastal zone of Bangladesh are: tidal fluctuations; salinities (soil, surface water, ground water); cyclone and storm surge risk. To prevent these vulnerabilities, one hundred and thirty-nine polders have been created through the construction of 5700 kilometers embankment (WARPO, 2006). But the performance of coastal polders mainly depends on the tidal characteristics of the sea and rivers. So the exposed coastal areas of Bangladesh need appropriate coastal protection works badly to prevent the intrusion of seawater into the mainland and to mitigate shoreline erosion.

1.2 Background of the study:

The Cox's Bazar-Teknaf Marine Drive Road (MDR) was built to facilitate tourism opportunities, develop the fishing industry, enhancing regional connectivity and improve the management of natural resources (IWM and BUET, 2014). Initially, the Cox's Bazar Marine-Drive Road Project was approved to construct an 80 km long Cox's Bazar-Teknaf Marine Road in 1993 but due to the unavailability of the fund, the project was phased out. The 1st phase was designated for 24km road from Kolatoli to Inani, another 24 km in the 2nd phase from Inani to Shilkhali and the 3rd phase has earmarked 32 km from Shilkhali to Teknaf. The project is located in a very vulnerable area that is hazard-prone (Rahaman and Hossain, 2015). The project area falls under the Ecologically Critical Area declared in 1997 (*The Environment Conservation Rules*, 1997).

The government of Bangladesh took shore protection work along the right bank of the road as a separate project. The design was prepared by Bangladesh Water Development Board (BWDB) and followed for implementation under 17 ECB, Bangladesh Army (Rahaman and Rahman, 2013). Bangladesh Water development Board considered cement concrete block as an armor unit of the protective work. Figure 1.2 represents a typical cross-section of the designed shore protection work along the marine drive road at Cox's Bazar by BWDB.

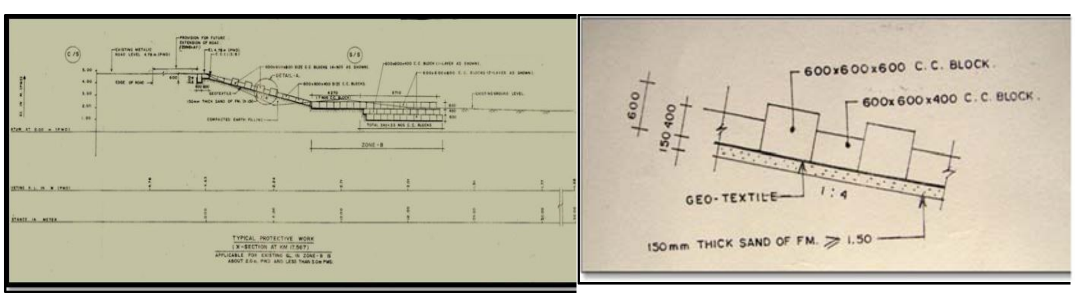


Figure 1.2: Typical cross-section of the designed shore protection work along marinr drive road, Himchari, Cox's Bazar, constructed on 2008 (IWM and BUET, 2014)

Over the years the Road has experienced severe erosion and damage at few locations mainly due to sea wave action (IWM and BUET, 2014). Consequently, the second phase of the project was suspended due to the lack of comprehensive study on wave action (Rahaman and Rahman, 2013). The revetment was built in the year 2008 (Figure 1.3). The revetment did not sustain for a long time. The revetment failed after completion of the work in the year 2009 (Figure 1.4)

Figure 1.3: C.C. block revetment on Marine drive (2008) (© M. A. Rahman)

Figure 1.4: Failure seen on the revetment (2009) (© M. A. Rahman)

A study "Coastal Hydraulic and Morphological Study and Design of Protection Measures for Marine Drive Road" by the Institute of Water Modeling and Bangladesh University of Engineering and Technology conducted after the failure of the C.C. block revetment (IWM and BUET, 2014). In this study, sleeping defense was proposed as a protective measure (Figure 1.5). One of the disadvantages of sleeping defense is that nourishment must be carried out at regular intervals and involves the mobilization of special equipment. But in the context of Bangladesh, regular maintaining is not suitable. For this reason, this option was not implemented. On the other hand, the highest wave height was found 2.53 m on the project area and C.C. block revetment is not recommended for higher waves ($H_s \geq 1.50$ m), which is one of the reasons for the failure of the marine drive protection measure (IWM and BUET, 2014).

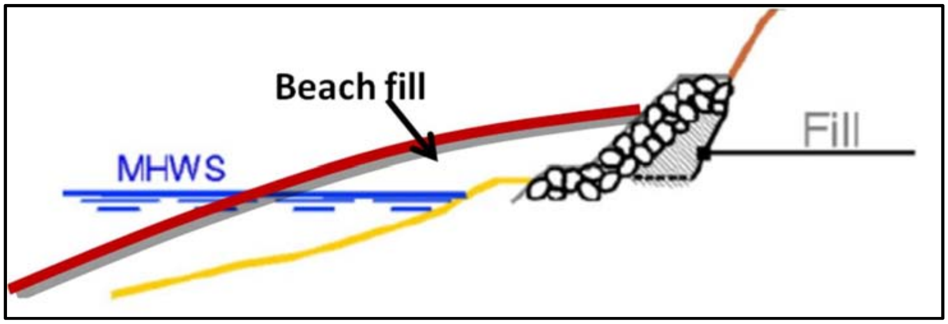


Figure 1.5: Section of sleeping defense (IWM and BUET, 2014)

Considering the above-mentioned discussion and limitations of sleeping defense and C.C. block revetment, this research aims to study alternative two-layer protection measure. To do that the performance of two-layer protection for higher waves ($H_s \geq 1.50$ m) has been investigated experimentally by installing submerged geo-tube breakwater at shallow water depth in front of C.C. block revetment at the shore. The submerged breakwater dissipates wave energy up to a certain limit (Figure 16). Though C.C. block revetment is not recommended for higher waves, it is effective for reducing wave energy of low height waves ($H_s < 1.50$ m) (Figure 1.7).

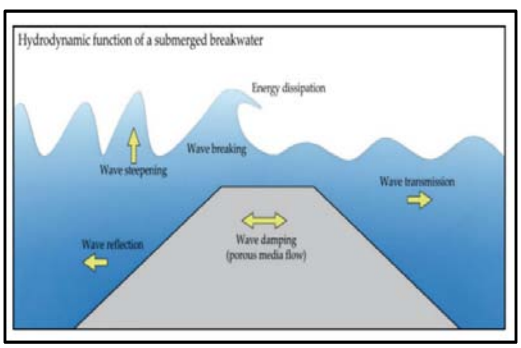


Figure 1.6: Submerged Breakwater reducing wave height (Cardenas-Rojas *et al.*, 2021).

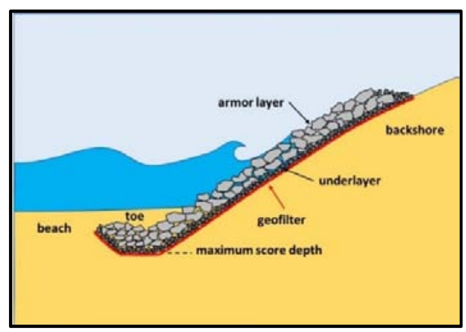


Figure 1.7: C.C. block revetment (© coastalwiki)

Therefore, the rationale of this study of two-layer protection is that in the places for higher waves ($H_s \geq 1.50$ m) C.C. block revetment is effective if the wave height is reduced ($H_s < 1.50$ m) in front of the revetment by installing submerged breakwater. To analyze the performance of two-layer protection, experiments have been carried out in the laboratory flume to investigate and analyze the physical stability of the revetment consists of C.C. block along with submerged geo-tube breakwater under different wave conditions and different submergences of the breakwater.

1.3 Objective of the study:

The specific objectives of this research are:

(1) To conduct experimental investigation and assess the physical stability of C.C. block revetment along with submerged geo-tube breakwater under different wave actions.

(2) To analyze wave transmission coefficients for different wave heights (H) and breakwater heights (h_b).

(3) To develop a relationship among water depth (h), wave height (H) and breakwater height (h_b) considering revetment stability.

1.4 Scope of the study:

Graded riprap in the shore protection revetment is not recommended for higher waves (i.e. $H_s \geq 1.5$ m) (Fundamentals of Design, 2011) . It is to be noted that the shore protection revetment with C.C. block along the Cox's Bazar coast (near Himchari) failed due to wave action during the year 2009, just one year after its construction. Later a study "Coastal Hydraulic and Morphological Study and Design of Protection Measures for Marine Drive Road" by IWM and BUET found the maximum wave height 2.53 m along the Cox's Bazar coastline (IWM and BUET, 2014). When the significant wave height is more than 1.50 m, other types of armor unit such as Tetrapod, X-bloc, Core-loc, etc. are recommended to use in the revetment ('Coastal Engineering Manual, Part IV', 2011). The construction and placement of those types of armor unit are difficult and also not cost-effective. Moreover, any hard protection structure like revetment may protect the shore from wave action, but the beach in front of it will be lost by wave-breaking action at the toe and slope of the revetment. On the other hand, submerged breakwater dissipates wave energy up to a certain limit and remaining wave energy passing over the breakwater may cause shore erosion to some extent. Considering the above-mentioned limitations of C.C. block revetment and submerged breakwater as single-layer protection structure, this research aims to study the performance of two-layer protection in the places of higher waves ($H_s \geq$ 1.50 m) by using C.C. block revetment at the shore and geo-tube breakwater at the shallow water depth. This two-layer protection will not only protect the shore but also will nourish the beach.

1.5 Organization of the thesis:

This thesis has been organized into five chapters, which are described below:

The first chapter provides a background with the rationale of the study, objectives, scope of the study and organization of this thesis. In the second chapter reviews of formulae and previous studies related to this study have been provided.

The detailed descriptions of the laboratory experiments and data collection techniques and methodology have been included in the third chapter. It also includes the downscaling of prototype parameters to the laboratory parameters.

In chapter four analyses of data with results have been discussed and this chapter is titles as "Results and Discussion". Finally 'Conclusions and Recommendations' have been presented in chapter five.

CHAPTER 2

LITERATURE REVIEW

2.1 General:

Waves and waves generated forces determines the geometry of beaches, the planning and design of marines, waterways, shore protection measures and hydraulic structures. It is essential to know the wave condition and type of shore problem to select a suitable hydraulic structure. In this chapter various types of hydraulic structure widely use to protect coast areas specially breakwater and revetment are discussed. This chapter also deals with previous various experimental studies on wave interaction with breakwaters and revetments, and the hydrodynamic efficiency of these breakwaters and revetments.

2.2 Previous studies on wave interaction with revetment:

Many experimental and theoretical studies were done for determining the efficiency and stability of revetment with different armor units. The design of C.C. block revetment in Bangladesh is generally done with Pilarczyk (1990) formula for wave protection.

Pilarczyk, (1990)

$$D = \frac{H_s \xi^b}{\Delta m \psi_u \varphi_{sw} \cos\alpha} \quad \text{.............................. (2.1)}$$

Where,

H_s = Significant wave height (m)

D = Thickness of the cover layer (m)

ψ_u= System determines stability upgrading factor.

φ_{sw} = Stability factor for incipient motion.

ξ_z^b= Breaker parameter $= \tan\alpha \frac{1.25 T_m}{\sqrt{H_s}}$ ....(2.2)

T_m= Average wave period (sec)

Δm= Relative density of submerged material = $(\rho_s - \rho_w)/\rho_w$

b = Exponent related to the interaction process between waves and revetment

α = bank normal slope, (°)

Breteler ***et al.***, (2014) studied the stability of block revetments under wave attack. It focuses on the stability in the run-up zone (above the still water level, SWL) and compares this with the stability in the wave impact zone (below SWL). Block revetments usually are very well capable to withstand high velocity flow, but problems can arise if the revetment is not in a perfect state. During the years the surface may become uneven because of differences in settlement or other causes (Figure 2.1). However, once there are blocks sticking out among the surrounding blocks, the hydraulic forces can be substantial. Although block revetments have a quite smooth surface, lack of maintenance can lead to exposed block edges.

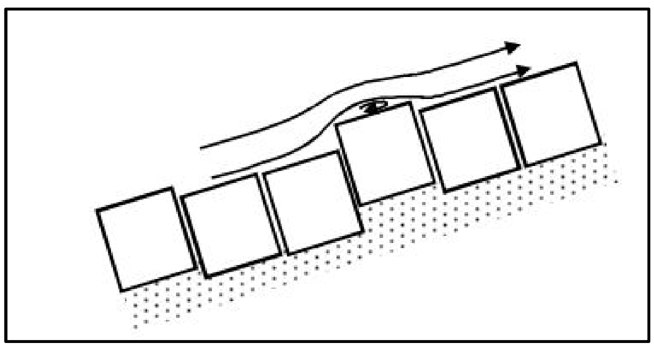


Figure 2.1: Uneven surface in the run-up zone (vertical cross section).

They found that one aspect of the uplift pressure comes from the flow which is blocked by the block (Figure 2.2). The high pressure on the upstream side of the block is transmitted to the filter layer, contributing to the uplift pressure. The other aspect, which is usually dominant, comes from the curved streamlines over the block. This leads to a substantial decrease in the pressure on the block, which can be even lower than the atmospheric pressure.

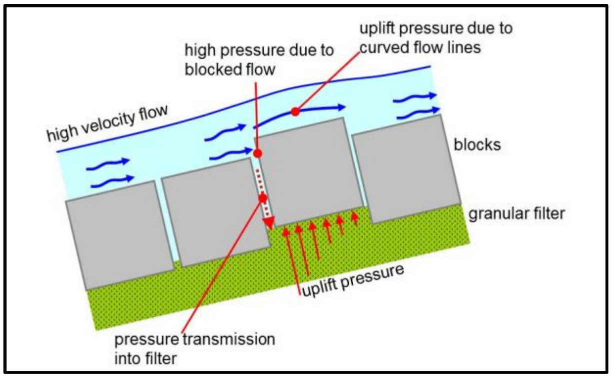


Figure 2.2: High pressure on upstream side of block is transmitted to filter.

The study shows that the revetment thickness in the run-up zone can be constructed much thinner than in the wave impact zone. But it also shows that the revetment should be well maintained. If blocks are sticking out among adjacent blocks, the revetment should be repaired.

Permana ***et al.***, (2017) studied both experimentally and numerically wave induced hydrostatic pressure distribution in order to evaluate the stability of revetment block against wave attack. Figure 2.3 shows pressure distribution due to wave attack.

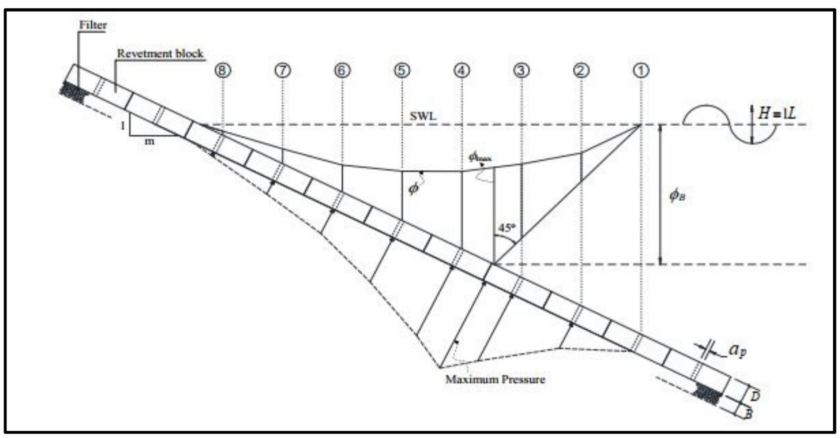


Figure 2.3: Pressure distribution beneath the revetment

Results from both physical and numerical models indicated that the filter-pore parameter greatly influenced the pressure beneath the block. The pressure decreases with the decreasing of the filter-pore parameter value. A small value of the filter-pore parameter can be obtained by using a smaller filter material, and a bigger pore dimension. The magnitude of pressure potential depends primarily upon the filter-pore parameter, oblique wave attack, the revetment slope, and wave height – block length ratio.

Kerpen *et al.*, (2019) studied wave overtopping of stepped revetment experimentally (Figure 2.4). Wave overtopping is the excess of water over the crest of a coastal protection infrastructure due to wave run-up. Wave overtopping of a smooth slope can be reduced by introducing slope roughness. A stepped revetment ideally constitutes a slope with uniform roughness and can reduce overtopping volumes of breaking waves up to 60% compared to a smooth slope. The effectiveness of the overtopping reduction decreases with increases of Iribarren number.

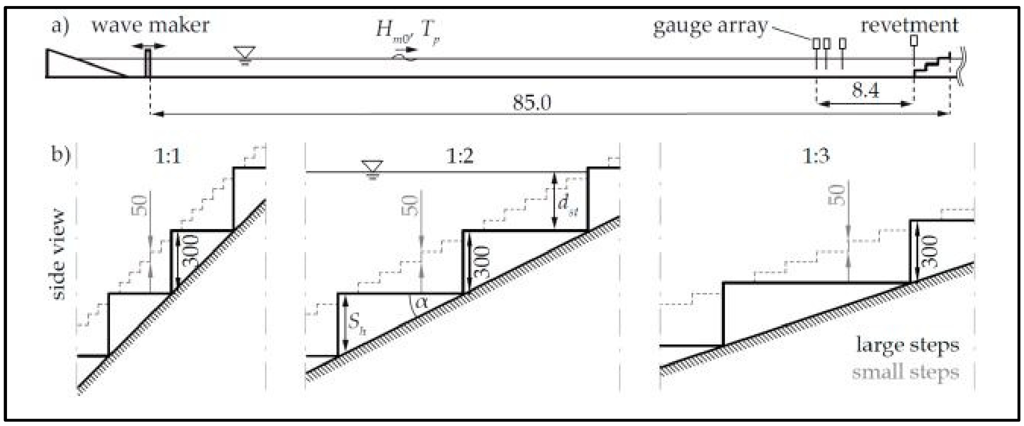


Figure 2.4: (a) Side view of the model set-up with wave gauge positions (units in meter). (b) Side view of the tested step geometries (units in millimeter).

Breteler *et al.*, (2012) carried out an experiment in the Delta Flume of Deltares with an 8.5 m high dike with a clay liner and sand core. Below the still water level, the dike was protected with a concrete block revetment and above the water level, there was grass on the clay (Figure 2.5). The tests were carried out with large waves: H_s = 1.6 m.

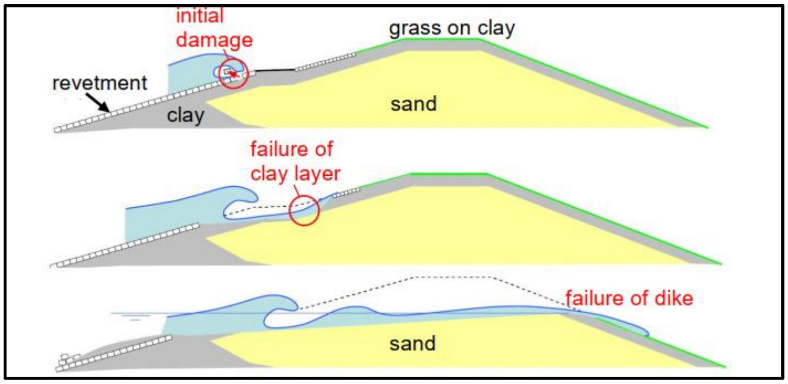


Figure 2.5: Cross-section of typical Dutch dike and failure process due to wave attack

By bringing together the results of the large scale experiments, numerical calculations and theoretical considerations, they found that there is a linear relationship between erosion volume and duration of wave attack for structured clay (usually found up to approximately 1 m below the clay surface) for the erosion of clay layer, and the erosion rate of unstructured clay is much smaller than structured clay. On the other hand, the erosion rate of the sand core of the dike is less than for dunes.

Nandi, (2002) conducted a study on the stability of C.C. blocks revetment against wave attack experimentally. The study compares the laboratory results with the present-day widely used design formula of Pilarczyk (1990) and it had shown that the agreement is reasonably satisfactory at least for surf range less than two (ξ<2.0). Results on wave run-up had been compared with surf parameters and agreed with Pilarczyk (1990) formula with a percentage of error of only 3.06 for experimental with 1:10 scale. The discrepancy was greater for experimental runs done with 1:20 scale. The failure mechanism of c.c. blocks over geotextiles was catastrophic. As soon as the displacement of one block started it propagated downward very quickly.

Hossain, (2013) investigate the stability of different types of armor units used in the shore protection structure. In this study, Armor unit is designed for 15 cm wave height and 1(V):4(H) revetment slopes by using Hudson (1961) equation. In this study C.C. block both uniformly and staggeredly placed, X-block and tetrapod were used. The Performance of C.C. block was found good when it was staggeredly placed than uniformly placed. The performance of tetrapod was better than any other armor unit.

2.3 Previous studies on wave interaction with breakwater:

Various aspects of two and three dimensional problems of wave interaction with submerged, bottom founded, or floating surface-piercing structures have been studied by many investigators.

Liao *et al.*, (2013) investigated experimentally the criteria of wave breaking and energy loss caused by a submerged breakwater on a horizontal bottom in a 2-D wave tank. Wave conditions of T and H as well as the freeboard of the submerged breakwater, with the front slope of 1/2 and 1/5, were varying in the experiments. Reflected and transmitted waves were recorded by wave gauges for the analysis of wave energy loss. They found that almost all waves can be triggered to break when the ratio of the estimated equivalent deep-water wave height to the freeboard of the submerged breakwater is greater than one, in all tested conditions. They also found that a milder front slope of submerged breakwater may not trigger wave breaking more efficiently as that with a steeper front slope does and allow waves with larger wave height to travel without breaking over a submerged breakwater with a milder front slope. This may be because the submerged breakwater with a milder front slope dissipates more wave energy caused by the resistance force as the waves traveling over a wider porous area, and therefore reduce the risk to break. It was found that a submerged breakwater will function much more efficiently if waves can be triggered to break by the structure. Furthermore, it was also found that the submerged breakwater with a front slope of 1/5 consumed more wave energy than that with ½ front slope. This may imply that the milder front slope does dissipate more wave energy through a wider range of porous structures comparing to the steeper front slope.

Vona *et al.*, (2020) studied the impact of submerged breakwaters on sediment distribution along marsh boundaries using Delft3D-SWAN. Breakwaters are structures that break incoming waves to reduce their energy at the shoreline, are able to trap sediments, and thus can promote the strengthening of the coast.

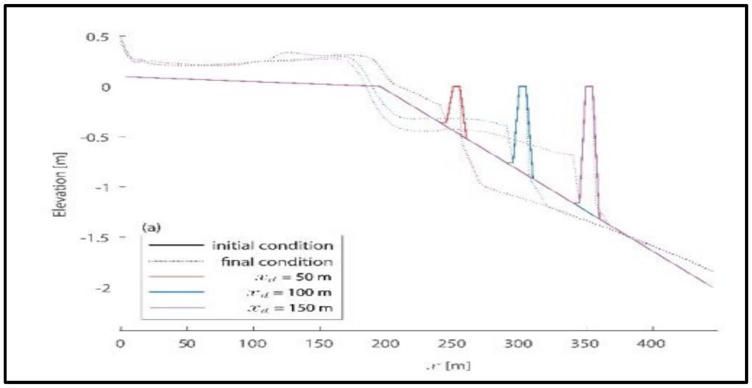


Figure 2.6: Bed level profile after the end of the simulation compared to the initial condition.

The distance of the breakwater from the shoreline plays an important role in sediment transport. Breakwater distance to the shoreline was negatively correlated with the amount of sediment deposited into the marsh and proportional to H_s (Figure 2.6).

Chen *et al.*, (2016) studied experimentally the protection of tsunami-induced scour by offshore breakwaters. A set of laboratory experiments were reported in this study on the protection of tsunami-induced scour by submerged or emerged breakwaters on a sandy beach. FLOW-3D was used to calculate the flow field of tsunami wave propagation over the breakwater in order to understand the sediment transport and tsunami scour process. The experiments show that the submerged breakwater could not effectively reduce the tsunami scouring and only could affect the height and position of the deposition sand bar. The emerged breakwater could significantly effectively reduce the tsunami scouring on the sandy beach; meanwhile, local scouring caused by plunging jet occurs mainly on both sides of the structure.

Rageh, (2009) studied experimentally the efficiency of the vertical thick submerged or emerged porous breakwaters. This study was under normal and regular waves with wide ranges of wave heights and periods under constant water depth. The efficiency of the breakwater was presented as a function of the transmission, the reflection and the wave energy loss coefficients. It was found that, both the transmission and the reflection coefficients decrease as the relative breakwater width increases, while the energy loss coefficient takes the opposite trend. Also, the breakwater is more effective in reducing the transmitted waves and reflecting the incident waves as the breakwater crest elevation higher than the still water level, in case of, the transmission coefficient is less than 0.3 and the reflection coefficient is more than 0.5, especially when the breakwater height reaches 1.25 the water depth.

Sultana and Rahman, (2019) investigated hydrodynamic performance of multiple row pile breakwater for both submerged and emerged condition as a shore protection structure. Experimental results reveal that, for transmitting smaller part of wave energy thorough the breakwater minimum transmission coefficient is obtained for breakwater with lowest porosity and short wave. Minimum reflection coefficient is obtained for breakwater with highest porosity. It was also seen that wave energy loss coefficient (K_L) increases with increasing porosity.

Rahman and Womera, (2013) investigated both experimentally and numerically wave interaction with submerged breakwater. It had been seen that for any particular wave period the relative structure height, hs/h (hs= structure height and h= water depth) and the relative structure width B/L (B=structure width along the wave direction and L= wave length) are the important parameters for the reduction of incident wave height. It was found that as relative structure height hs/h increases, the incident wave reduces more due to breaking caused by the breakwater.

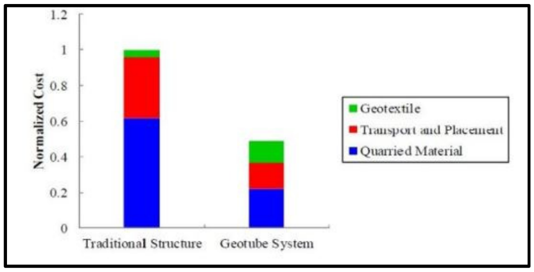
Akter, (2013) investigated experimentally the hydrodynamic performance of rectangular porous breakwater both submerged and emerged way. Different hydrodynamic coefficients like transmission coefficient (K_t), reflection coefficient (Kr), and wave energy loss coefficient (K_L) were calculated from the measuring water surface data. These coefficient values were then analyzed with respect to relative submergence (h_b/h), relative breakwater width (kxB), and porosity of breakwater. Experimental results reveal that for transmitting the smaller part of wave energy through the breakwater minimum transmission coefficient was obtained for breakwater with the lowest porosity and short wave. Minimum reflection coefficient was obtained for breakwater with the highest porosity and with minimum submerged condition. It was also noticed that the effect of increasing relative breakwater width (kxB) on the transmission coefficient (K_t) and reflection coefficient (K_r) is less when the breakwater height is kept fixed. Porosity has effect on the wave energy loss coefficient. Wave energy loss coefficient increases with increasing porosity.

Afroz, (2015) performed both experimental and numerical studies on the horizontal slotted submerged porous breakwater. To find out the effective size and porosity of this protection structure for the reduction of wave height. From this study, it was found that Wave reflection coefficient increases as relative breakwater width (k.B) increases (k.B= 2Bπ/L, where k is the wave number). Also, the reflection coefficient decreases as porosity (n) increases. The transmission coefficient decreases as relative breakwater width k.B increases. This implies that the breakwater reduces the transmitted waves as the breakwater width (B) increases or the wave length (L) increases.

2.3.(a) Previous studies on wave interaction with geo-tube breakwater:

Koerner, (2000) reported that geotextile tubes can provide better protection for beach erosion. Geotextile tubes of diameters of up to 3m, made up of woven or knitted high strength fabric have been effectively used to control both inland and oceanfront erosion. Length of geo-tubes is decided based on ease in handling/placing and sand filling. The main tubes are generally flanked by tubes of smaller diameters on the upstream side which help in resisting lateral pressure. Also, it is required to provide cover to geo-tubes to protect them from degradation/damage.

Shabankareh *et al.*, (2017) showed that for projects which require a relatively high amount of quarried material, using geo-tubes filled with dredged material leads to significant cost savings (Figure 2.7). The size of structures, the distance from the quarry, and the sail distance to the dredged material disposal site are the key parameters that have the most environmental influence and the first one is found to be the parameter of the highest importance. Geo-tube system produces over 85% less CO_2 in comparison with traditional rock structure (Figure 2.8). Thus, the geo-tube system is not only an economical method than the traditional method of breakwater construction, but also it offers a better environmental alternative to the traditional breakwater system.



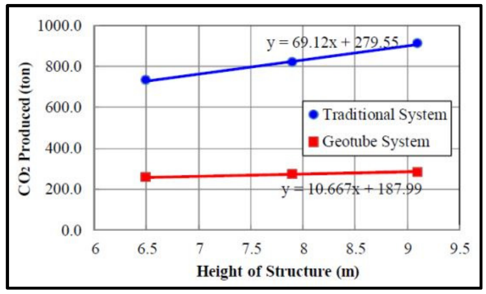


Figure 2.7: A comparison of total cost between traditional structure and geo-tube system

Figure 2.8: A comparison of CO2 emissions between traditional structure and geo-tube system

Hidayat and Andrianto, (2018) demonstrate that geotextile tube is an innovative technology that can be used as the core of a breakwater structure to address shoreline erosion problems. One of the advantages of geotextile Tubes is the use of sand fill material which is available in most project sites. Another significant advantage of geotextile Tube system is simple and fast in installation. When a filled geotextile tube is combined with rocks and concrete armor units, geotextile tube breakwaters become an effective structure in absorbing wave energy thus preventing shoreline abrasion.

2.4 Previous studies on two layer shore protection:

A study "Coastal Hydraulic and Morphological Study and Design of Protection Measures for Marine Drive Road" by the Institute of Water Modeling and Bangladesh University of Engineering and Technology suggested sleeping defense (a combination of revetment and beach nourishment) for Marine drive road, Cox's Bazar (IWM and BUET, 2014). Sleeping defense has been suggested considering easy access to the beach and no obstacle to sea view. But it needs nourishment at a regular interval and needs special equipment. Nourishing the beach at a regular interval is not suitable in the context of Bangladesh. Therefore, this suggestion was not implemented later.

Jeng *et al.*, (2005) studied experimentally the interaction between water waves, a submerged breakwater, a vertical wall and a sandy seabed. Laboratory experiments were conducted to record the water surface elevation and the pore pressures inside the seabed foundations.

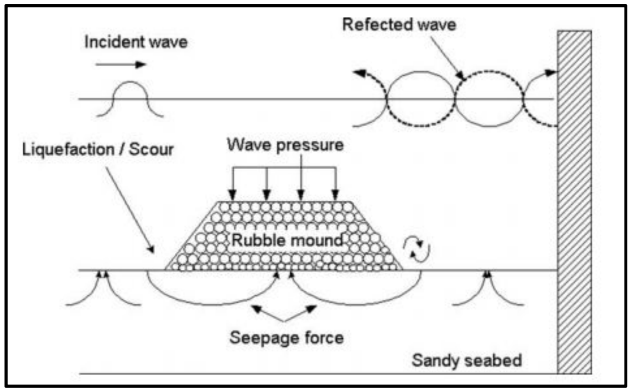


Figure 2.9: Sketch of interaction among water waves, submerges breakwater, vertical wall and a sandy seabed.

The strong interaction of surface waves between a submerged breakwater and a vertical wall causes a significant change in the wave-induced pore pressure within the seabed. The experimental results indicate that the wave-induced pore pressure beneath the submerged breakwater is greater than that at the toe. The interaction between the incident wave and reflected waves from the vertical wall and submerged breakwater will increase the pore pressure amplitude within the seabed.

2.5 Dimensional analysis:

Dimensional analysis is a method for reducing complex physical problems to their simplest (most economical) forms prior to quantitative analysis or experimental investigation (Sonin, 2004). One of the objectives of this research is to establish a relationship among wave height (H), water depth (h_w) and breakwater height (h_b). To establish the relationship a dimensional analysis is required and Buckingham π theorem has been used in this research.

Buckingham π theorem (also known as Pi theorem) is used to determine the number of dimensional groups required to describe a phenomena (Gupta *et al.*, 2014). According to this theorem "the number of dimensionless groups to define a problem equals the total number of variables, n, (like density, viscosity, etc.) minus the fundamental dimensions, p, (like length, time, etc.)." If we call these dimensionless groups π1, π2, π3, etc., then the equation expressing the relationship among the variables has a solution of the form

$F(\pi 1, \pi 2, \pi 3 \ldots\ldots) = 0$………. ……………………………. (2.3)

If in a problem n = 5 and p = 3 then n − p is equal to two and the solution would be either

$F(\pi 1, \pi 2) = 0$ ……………………………………… (2.4)

Or

$\pi 1 = f(\pi 2)$ ……………………………….…….. (2.5)

In this case, the experimental data may be represented by plotting π1 vs. π2. The resulting curve gives a relationship between π1 and π2 which cannot be deduced from dimensional analysis.

2.6 Summary:

From the above point of view, it can be concluded that any hard protection structure like revetment may protect the shore from wave action, but the beach in front of it will be lost by wave breaking action at the toe of the revetment. On the other hand, submerged breakwater dissipates wave energy up to a certain limit and remaining wave energy passing over the breakwater may cause shore erosion to some extent. Considering the above-mentioned limitations of C.C. block revetment and submerged breakwater, this research aims to study the performance of two-layer shore protection structures using submerged geo-tube breakwater at shallow water depth and C.C. block revetment at the shore. One of the additional advantages of this two-layer protection is the automatic nourishment of the land between the revetment and the submerged breakwater. In this research, experiments have been carried out in the laboratory flume to investigate and analyze the physical stability of the revetment consists of C.C. block along with submerged geo-tube breakwater under different wave conditions and different submergences of the breakwater.

CHAPTER 3

METHODOLOGY

3.1 General:

Laboratory measurements and observations are the key techniques used to understand and improve the knowledge of the underlying physics based on the physical processes that take place at and around the test models. The techniques used in this experiment allow to assess physical stability of revetment consist of cement concrete block along with submerged geo-tube breakwater against wave action.

To investigate the performance of the revetment and submerged geo-tube breakwater, experimental studies are carried out in a two dimensional wave flume at the Hydraulics and River engineering Laboratory of Bangladesh University of Engineering and Technology. A set of experiments are carried out with submerged geo-tube breakwater of different height for different wave period in the same wave flume in front of C.C. block revetment. Apart from the discussion on development of the submerged geo-tube breakwater and revetment, this chapter also outlines the detail of the test facilities and instrumentation employed in this experimental study. These apparatus were carefully inspected to ensure the accuracy and quality of the measured data. The complete test program towards achieving the study objectives is also explained in detail.

In this experiment the methodology of Nandi, (2002) and Hossain, (2013) has been followed for C.C. block revetment, and the methodology of Rahman and Womera, (2013) has been followed for submerged breakwater. All the three experiment was conducted in the Hydraulics and River engineering Laboratory of Bangladesh University of Engineering and Technology

3.2 Experimental Set-up:

In 21.3 m long wave flume, the wave generator is placed 200 cm downstream from the starting of the flume. The geo-tube submerged breakwater is installed at a distance of 800 cm from the wave generator. Five different positions are chosen for data collection as shown in the Figure 3.1. Two locations are in front of the submerged geo-tube breakwater to investigate the incident wave properties. Then the three positions are chosen behind the breakwater to observe the effect of breakwater installation in reduction of wave height. The detail of experimental setup is shown in Figure 3.1.

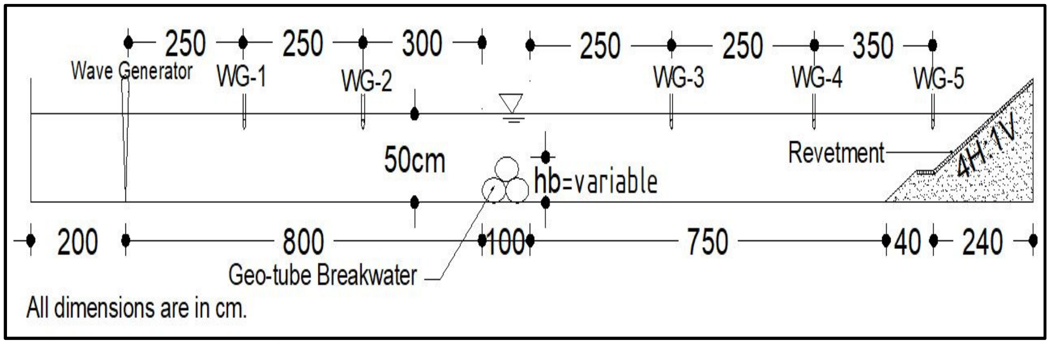


Figure: 3.1 Detail of the experimental setup

3.3 Laboratory Equipment:

3.3.1 Two-dimensional wave flume:

The wave flume is 21.3 meters long, 0.76 meter wide and 0.74 meter deep. The bottom of the wave flume is made of steel, whereas both of its sides are made of glass. In the wave flume artificial regular waves are generated by wave generator. The two-dimensional wave flume is shown in Figure 3.2.

Figure 3.2: Wave flume

3.3.2 Wave generator:

The wave generator generates artificial regular waves of controlled wave period (T). The frequency of the wave paddle is set to the desired wave period (T) from a developed relationship between $4\pi^2h_w/(gT^2)$ and (e+f)/f , where 'h_w' is the still water depth and ‘g’ is the gravitational acceleration. For this particular wave generator, ‘e’ and ‘f’ are the horizontal movement of the wave paddle at still water level and at the bed of the flume respectively (Fig. 3.3(a)). The wave generator is placed at one side of the wave flume which is also 800 cm in front of the submerged geo-tube breakwater. Fig. 3.3(b) shows some photo views of wave generator.

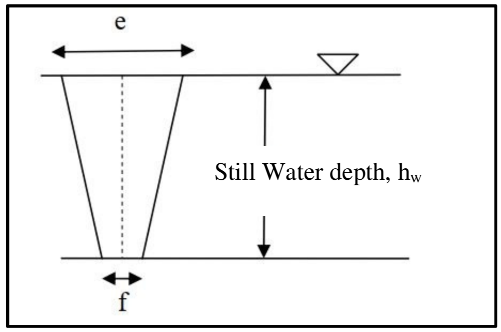


Figure 3.3(a): Wave generator

Figure 3.3(b): Photo views of wave generator

3.3.3 Wire Screens:

Screens made of coarse wire mesh kept at 6 inch from each other were placed in front of the wave generator to reduce wave reflections. In total 15 screens were used. The number and spacing were selected by trial and error method. The wave reflections was considered minimum when the crests of the generated waves in the flume were seen in a straight line from a side view. The picture of the wire screen is shown in Figure 3.4

Figure 3.4: Wire Screens

3.4 Methodology:

3.4.1 Slope Preparation:

At first a wooden support is placed at the end of the laboratory wave flume. It is 76cm height, 74 cm width and 4 cm thick. It acts as a support system, to retain the sand on their desire positions. Two types of sand are used to prepare the slope. One is coarse sand and other is finer sand. Properties of the sand is

Finer sand: Fineness modules (F.M) = 1.42; D_{15} = 0.16; D_{50} =0.25; D_{85} =0.46

Coarse sand: Fineness modules (F.M) = 3.12; D_{15} = 0.45; D_{50} = 0.88; D_{85} = 1.8

From the gradation curve get these values. Gradation curve as shown (Appendix A)

Compact finer sand was laid in first layer for preparation of the slope. Then 4 cm thickness of coarse sand laid over the finer sand. Then 2.5 mm thickness geotextile was placed on the coarse sand layer. After that 3 cm thick stone chips was placed on the geotextile layer. Finally different C.C block revetment was placed over the stone chips. Necessary figures have been attached in Figure 3.5(a) and Figure 3.5(b)

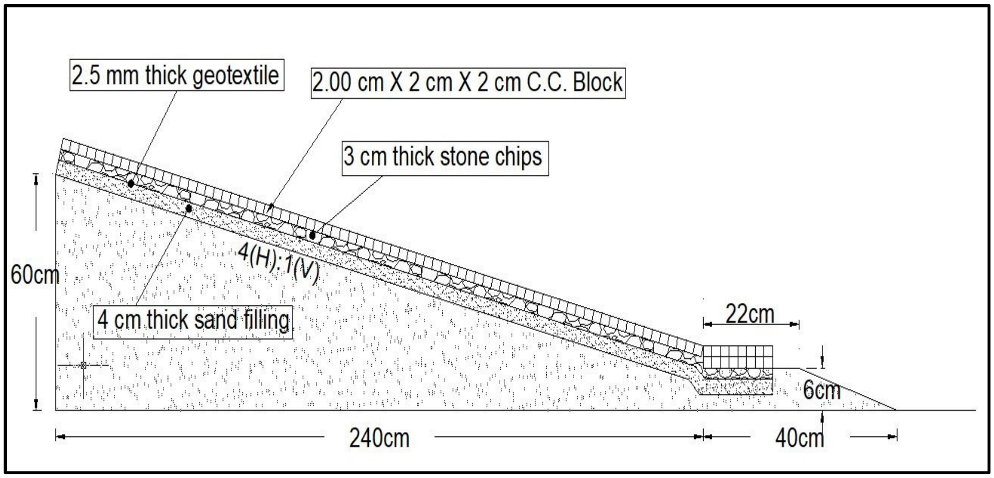


Figure 3.5(a): Details of bank slope

(i) Filling fine sand

(ii) Compaction of Coarse sand

(iii) Compacted coarse sand

(iv) Placing Geo-textile filter

(v) Placing of stone chips

(vi) Placing of C.C. block

(vii) Constructed Revetment (side view)

(viii) Constructed Revetment (top view)

Figure 3.5(b): Figures of revetment preparation

3.4.2 Design of C.C block:

Before analysis the stability of revetment blocks one must have understanding about the system involved in a revetment. The most widely used measure of determining characteristic size armor stability is that developed by Pilarczyk (1990).

$$D = \frac{H_s \xi^b}{\Delta m \psi_u \varphi_{sw} \cos \alpha} \qquad \text{.....................................} \ (3.1)$$

Where,

H_s = Significant wave height (m)

D = Thickness of the cover layer (m)

ψ_u= System determines stability upgrading factor.

φ_{sw} = Stability factor for incipient motion.

$\xi_z{}^b$= Breaker parameter $= \tan\alpha \frac{1.25 T_m}{\sqrt{H_s}}$ ……………………………………..(3.2)

T_m= Average wave period (sec)

Δm= Relative density of submerged material = $(\rho_s\text{-}\rho_w)/\rho_w$

b = Exponent related to the interaction process between waves and revetment

α = bank normal slope, (°)

The formula is restricted to values $\xi z < 3$ and $\cot\alpha \geq 2$, i.e. to plunging breakers, which generate high local pressure heads. Otherwise overestimation of the unit size is likely, because dynamics of the breaking process are diminishing.

In the formula 'b' is the exponent related to the interaction between waves and revetments ($0.5 \leq b \leq 1.0$). For rough and permeable revetments, b = 0.5, for smooth and less permeable placed-block revetments it is close to unity. For other systems b = 0.67 may be applied. Ψu is the system specific stability upgrading factor.

The material and armour layer unit specific coefficients to be applied for design against
wave attack are summarized in Table.3.1

Table 3.1: *Coefficient for design various cover materials against wave action*

Revetment type	Stability factor for incipient motion φ[-]	Stability upgrading factor, Ψu [-]	Interaction coefficient, b [-]
Randomly placed, broken riprap and boulders	2.25-3.00	1.00-1.33	0.50
CC blocks, cubical shape, randomly placed in multi-layer	2.25-3.00	1.33-1.50	0.50
CC blocks, cubical shape, hand placed ,single layer (geotextile filter)	2.25	2.00	0.67 - 1.00
CC blocks, cubical shape, hand placed in single layer, chess pattern (geotextile on sand)	2.25	1.50	0.67 - 1.00
CC blocks cable connected	2.25	1.80	0.67
Wire mesh mattress	2.25	2.50	0.50
Gabions/mattress filling by stone	2.25	2.50	0.50

This experiment was carried out with a fixed bank normal slope 1(V):4(H). All Parameter was scaled down in such a way that Surf Co-efficient (a dimensionless number) remained same for both prototype and laboratory. Necessary data has been tabulated in table-3.2

Table-3.2: *Preliminary Calculation of Parameters for both prototype and laboratory*

Prototype			Laboratory					
H_s (m)	T (sec)	Surf coefficient	Wave height, H_s		Wave period, T		Surf coefficient	Block Size (cm)
			Value (cm)	Scale	Value (sec)	Scale		
1.5	5.7	1.45	9.50	16	1.43	4.0	1.45	2.00

In the above Table-3.2, the prototype data (Significant wave height, H_s and Wave period, T) has been considering wind speed = 37 km/hr, fetch length = 139 km, and wind duration =10 hr (Katsaprakakis, 2020). Here, significant wave height, H_s is 16 times scaled down. T is calculated by 4.0 times scaled down from prototype. Finally the desired block size is determined by Pilarczyk formula (Equation-3.1).

<u>For Prototype:</u>

Estimated Design Wave Height, H_s = 1.50m and associated wave period, T = 5.7 sec.

Since, the revetment slope is 4(H): 1(V) so, $\alpha = \tan^{-1}(1/4)$ = 14.04°

Surf Co-efficient, $\xi_z = \tan\alpha \frac{1.25 T_m}{\sqrt{H_s}} = 1.45$

<u>For Laboratory:</u>

Significant wave height, H_s is assumed in such a way so that surf co-efficient remain constant, 1.45. H_s scaled down 16 times, where 16 is an arbitrary number.

Hence, H_s = 9.50 cm

Wave period, T is determined by 4.0 times scaled down than prototype, where 4.0 is an arbitrary number.

Hence T= 1.43 sec

Co-efficient for design of C.C block revetment against wave attack:

System determines stability upgrading factor, ψ_u= 2.0
Stability factor for incipient motion, φ_{sw} = 2.25
Exponent related to the interaction process between waves and revetment, b = 0.67

Now, From Equation (3.1), $D = \dfrac{H_s \xi^b}{\Delta m \psi_u \varphi_{sw} \cos \alpha}$

$$D = \frac{0.095 \times 1.43^{0.67}}{1.4 \times 2.0 \times 2.25 \times cos 14.04} \approx 2.00 \text{ cm}$$

3.4.3 Making procedure of cement concrete block in laboratory:

In this research, 2.0cm x 2.0cm x 2.0cm cubic steel dices were prepared. The dice of C.C -block was made in such a way that it can be easily opened after casting cement paste. Then mobile and grease was laid on the inner side of the dice so that it could be removed easily after hardening the mortar paste. Cement and sand were mixed properly 1:1 ratios. After mixing, dices were filled with mortar paste. Mortar filled dices were laid on polythene sheet on a plane surface. Later steel dice were removed very safely and then after hardening, the C.C-block got submerged in water for curing. Some pictures have been attached in Figure 3.6 for procedure of making cement concrete blocks:


(i) Formwork of C.C. block


(ii) Cement Concrete mortar

(iii) Pouring the mortar into the formwork (iv) Removing the formwork

(v) C.C. block after removing the formwork (vi) C.C. block stack

Figure 3.6: pictures for procedure of making C. C. blocks

3.4.4 Submerged breakwater:

The geo-tube breakwater is made using geotextile fabric bag. The bag is filled with soil collected from river bed. As the width of the total breakwater was selected 100cm, so two bags having width of 50 cm were used for the base of the breakwater. Depending on the Relative submergence the height of the breakwaters were selected. For Relative submergence of 0.6, two bags were kept side by side and height of both of them were 15 cm and another bag kept at the top to make the height 30 cm. Similarly three bags were used for other submergence.

Figure 3.7: Geotextile tube filled with sand

Basis of breakwater size selection:

Dick and Brebner, (1968) proved in their study on solid and permeable submerged breakwaters that for optimum reduction in transmitted wave height, breakwater width (B) should be as large as possible in fact up to wave lengths.

This is unlikely to be an economical proposition, so that some narrower breakwater with greater height was investigated later. Kawasaki and Iwata, (2001) investigated the breaking limit, the breaker type and the breaking point due to various submerged trapezoidal breakwaters. In their study, they found usually the waves break when the relative breakwater width B/L is in the range of 0.2 to 0.4.

In this research, the laboratory experiments are conducted for four different wave periods ranging from T= 1.7 sec to 2.0 sec and corresponding wavelengths of 332 cm to 406 cm to investigate the interaction of waves with submerged body of different relative breakwater heights of 0.3, 0.4, 0.5 and 0.6 by installing breakwater heights of 15 cm, 20 cm, 25 cm and 30 cm in 50 cm depth of water. For optimum reduction in transmitted wave height, the breakwater width along the wave direction was selected as 100 cm so that the relative breakwater width B/L ranges from 0.2 to 0.4. Breakwater lengths are usually selected so that they can cover the protection required length of the coastline. In this two-dimensional study, the breakwater length is selected as 76 cm which covers the full width of the two- dimensional wave flume.

3.5 Data acquisition:

3.5.1 Laboratory experimental run conditions:

The experimental run was carried out after adjusting the wave generator. Setting the frequency of the wave paddle of the wave generator results in a deviation of the wave period (maximum 0.1%) from the required wave period.

In this experiment, four different relative breakwater heights as h_b/h_w= 0.3, 0.4, 0.5 and 0.6 were used in still water depth of 50 cm. Four runs was conducted for four different regular waves with submerged geo-tube breakwater in front of a revetment for a particular height i.e. 15 cm, 20 cm, 25 cm and 30 cm. Four regular waves of wave periods 1.7 sec, 1.8 sec, 1.9 sec, and 2.0 sec were generated by setting the frequency of the wave paddle of the wave generator. The detail of the experimental run conditions are given in Table 3.3.

Table-3.3: *Test Scenario of the experiment*

Run No	Relative breakwater height ($\frac{h_b}{h_w}$)	Wave period (T) sec	Incident Wave Height, H_i (cm)	Incident Wave Length, L_i (cm)
1	0.3	1.7	11	332
2		1.8	12	357
3		1.9	14	381
4		2.0	15	406
5	0.4	1.7	11	332
6		1.8	12	357
7		1.9	14	381
8		2.0	15	406
9	0.5	1.7	11	332
10		1.8	12	357
11		1.9	14	381
12		2.0	15	406
13	0.6	1.7	11	332
14		1.8	12	357
15		1.9	14	381
16		2.0	15	406

3.5.2 Data collection:

To achieve the basic objective of understanding the wave interaction of the submerged geo-tube breakwater in front of a revetment, data of water surface elevation has been collected at five different locations. Data of water surface has been collected manually by introducing vertical scales on the flume side made of glass (Figure 3.8).

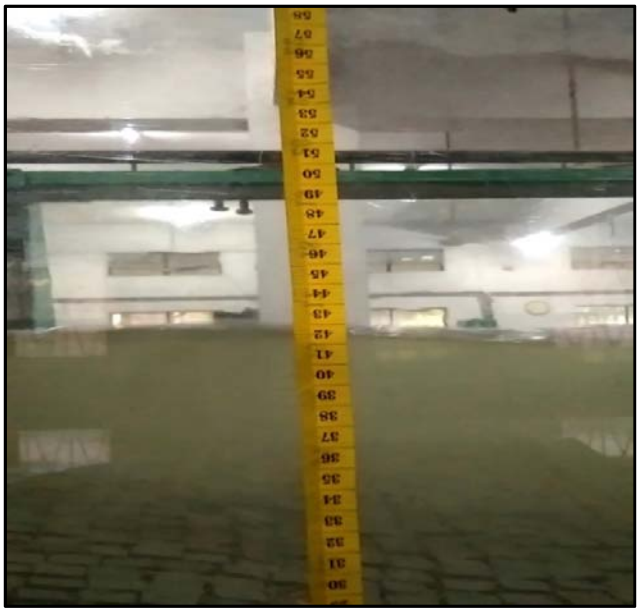


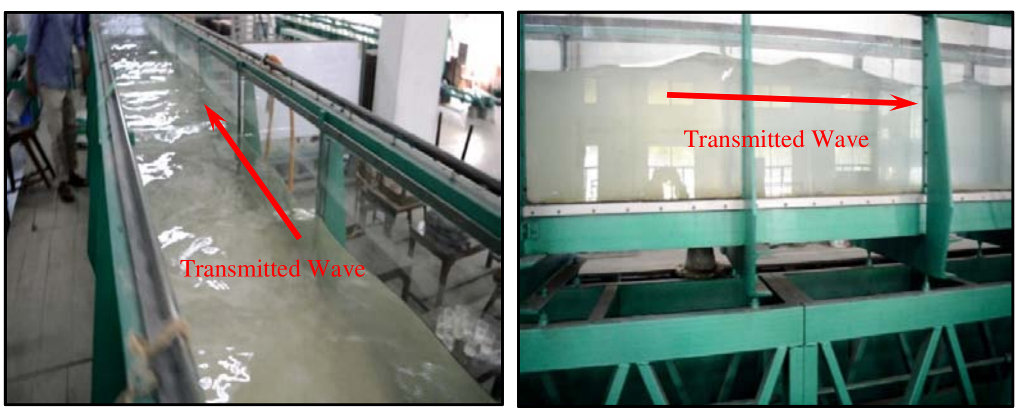
Figure 3. 8: Data collection using measuring tape

Five different locations both in front of and behind the submerged geo-tube breakwater were selected for data collection. Among these five positions two were in front of the breakwater and the three positions were behind the submerged breakwater. These positions were chosen in such a way to understand the effect of installing submerged breakwater in reducing incident wave height. At each position data of water surface have been collected for one minute duration at five seconds interval.

Still photographs and video recordings are taken during each run. Some photographs taken during the experimental runs are given in Fig. 3.9(i) to Fig. 3.9(iv) These are categorized as the incident waves approaching the breakwater; the breaking of waves over the breakwater, the transmitted wave after passing the breakwater and wave breaking on the cement concrete block revetment.

(i): Wave approaching the breakwater

(ii): Wave breaking near breakwater



(iii): Transmitted wave after passing the breakwater

(iv): Wave breaking on revetment

Figure 3.9: Wave breaking

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Effect of relative breakwater height on wave height reduction:

In Figure 4.1 the effect of relative breakwater height (h_b/h_w, h_b is the height of breakwater and h_w is the still water depth) on wave transmission coefficient, k_t is shown for four different wave periods, where the transmission coefficient, $k_t = H_t/H_i$ (where, H_t is the transmitted wave height and H_i is the incident wave height). From this figure, it is clear that as the relative breakwater height, h_b/h_w increases, the transmission coefficient, k_t decreases for any particular wave period. It is also observed that transmission coefficient, k_t decreases with the decreases of wave period, T for any particular relative breakwater height, h_b/h_w.

For relative breakwater height, h_b/h_w= 0.6 and wave period, T=2 sec, wave height reduces 40% after breaking. For relative breakwater height, h_b/h_w= 0.5, 0.4 and 0.3 the wave height reduction occurs up to 37%, 27% and 20% respectively for the same wave period, T= 2.0 sec. For other experimental runs, the variation of wave height reduction follows the similar trend.

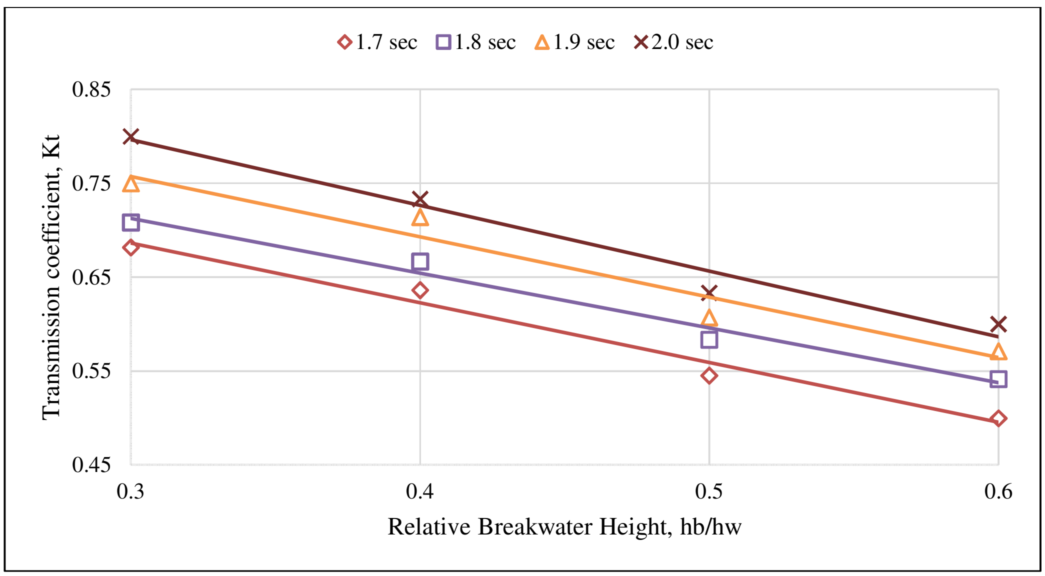


Figure 4.1: Effect of relative breakwater height on wave height reduction

4.2 Effect of relative breakwater width on wave height reduction:

Figure. 4.2 shows the effect of relative breakwater width, B/L (where, B is the width of breakwater along the wave direction, and L is the wave length) on wave transmission coefficient, k_t for four different wave periods and for four different relative breakwater heights (h_b/h_w=0.3, 0.4, 0.5 and 0.6), where the transmission coefficient, $k_t = H_t/H_i$ (where, H_t is the transmitted wave height and H_i is the incident wave height). For a particular relative breakwater height, it is observed that as the relative breakwater width (B/L) increases, the reduction of wave height due to breaking occurs more.

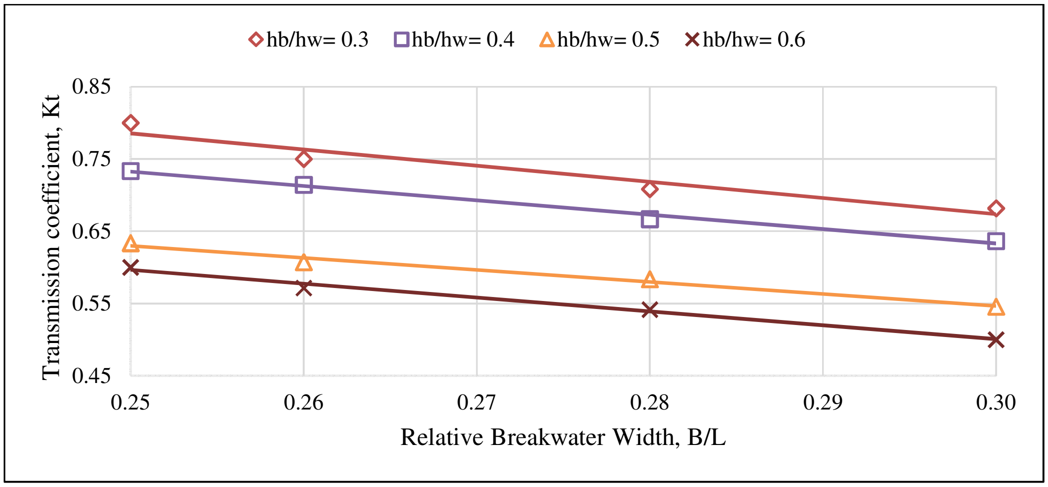


Figure 4.2: Effect of relative breakwater width on wave height reduction.

For example, for relative breakwater height, h_b/h_w= 0.5 and relative breakwater width, B/L= 0.25, the wave height reduces 37% after breaking. For relative breakwater width, B/L= 0.26, 0.28, and 0.30 the wave height reduction occurs up to 39%, 42% and 45% respectively for relative breakwater height h_b/h_w= 0.5. For any ratio of B/L, this trend is almost similar. So, as the relative breakwater width increases, the reduction of wave height occurs more for any particular value of h_b/h_w.

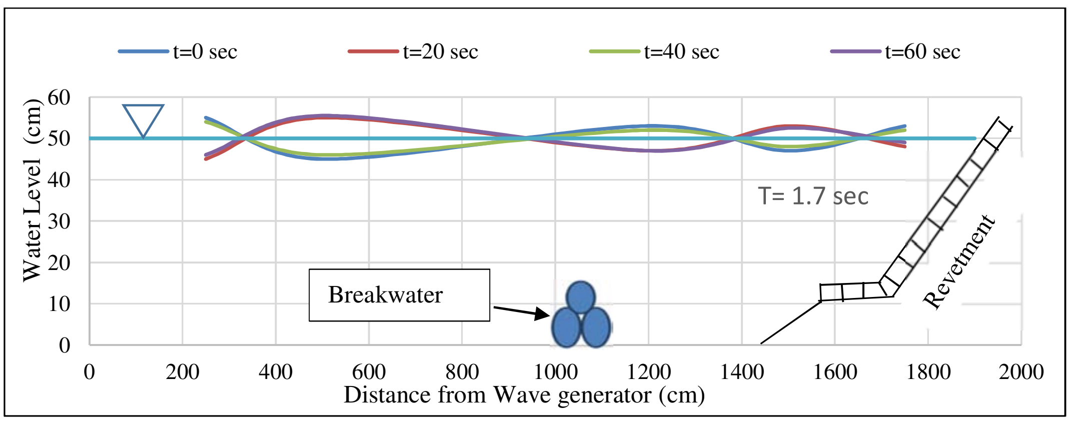
4.3 Water surface profile:

The high energy of incident wave is reduced drastically because of installing breakwater. This is evident in all the Figures from 4.3 to 4.6 that the incident wave height reduces after passing the breakwater.

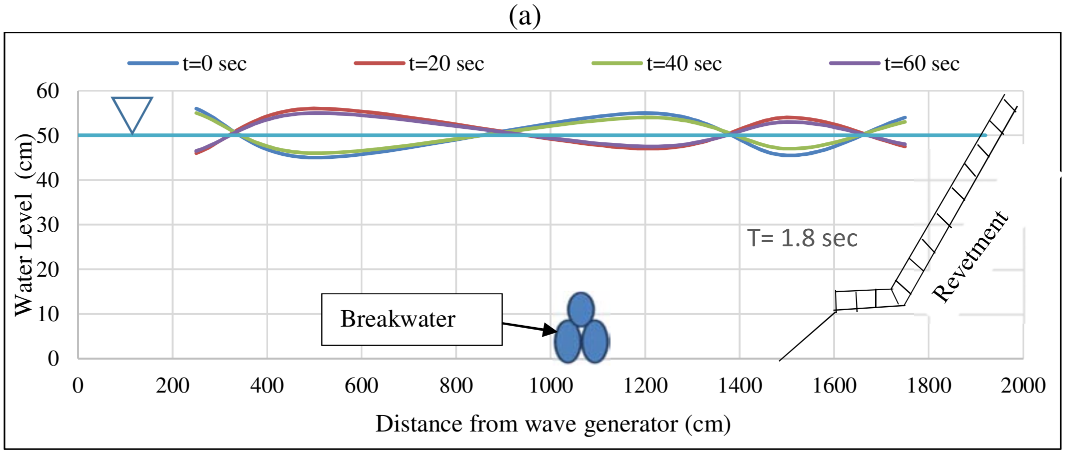
Among the four different relative breakwater height (h_b/h_w) and four different wave period (T) for which the experiments are conducted, maximum reduction of wave height occurs when wave period is minimum for a particular relative breakwater height (h_b/h_w) i.e. the maximum wave height reduction occurs for wave period, T= 1.7 sec among the four different wave period (T=1.7 sec, 1.8 sec, 1.9 sec, 2.0 sec) for any particular relative breakwater height.

4.3.1 Water surface profile for relative breakwater height 0.3:

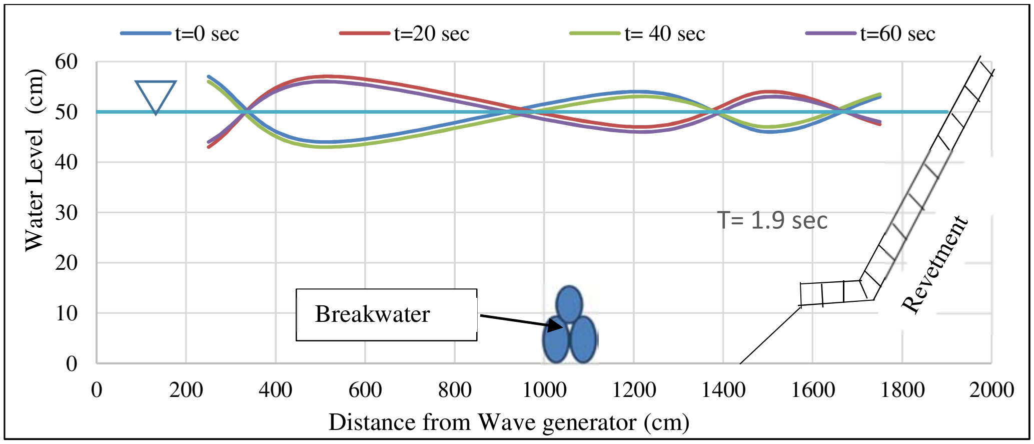
Figure 4.3(a) represents the water surface profile for wave period 1.7 sec and relative breakwater height 0.3. The wave period, T= 1.7 sec produces an incident wave height 11 cm. The incident wave height is reduced 32% for relative breakwater height 0.3. Figure 4.3(b) represents the water surface profile for wave period 1.8 sec and relative breakwater height 0.3. The incident wave height becomes 12 cm for the wave becomes 1.8 sec, and this incident wave height reduces 29% for the same relative breakwater height. Figure 4.3(c) shows the water surface profile for wave period 1.9 sec and relative breakwater height 0.3. 14 cm incident wave height are produced by the wave period 1.9 sec. 14 cm incident wave height are reduced by 25% for the relative breakwater height 0.3. The water surface profile for wave period 2.0 sec and relative breakwater height 0.3 is shown in Figure 4.3(d). The incident wave height is 15 cm for the wave period 2.0 sec. The relative breakwater height 0.3 reduces 20% incident wave height for wave period 2.0 sec. Thus, as the wave period increases the reduction of incident wave height is decreases for relative breakwater height 0.3.



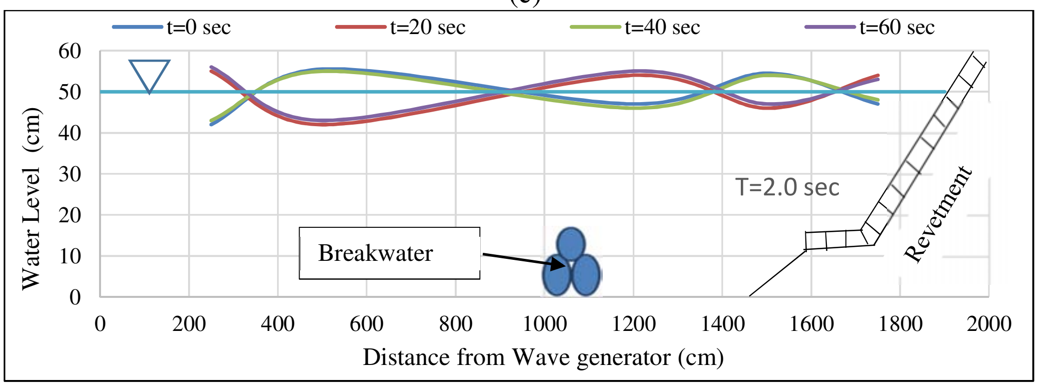
(a)



(b)



(c)

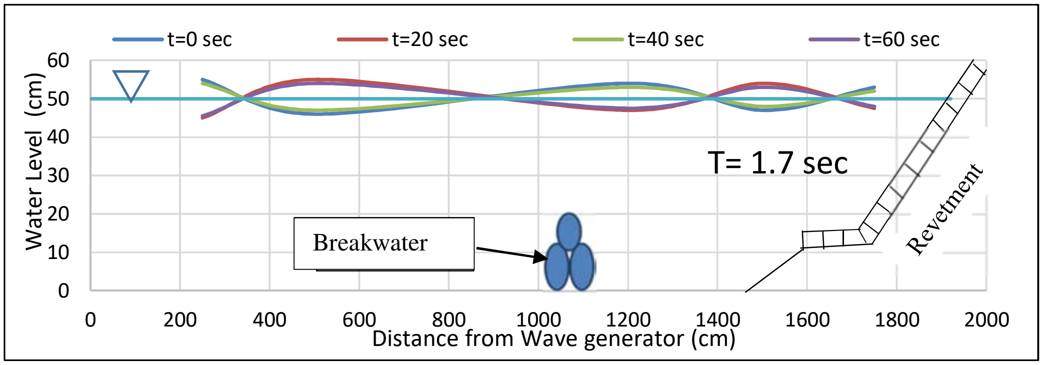


(d)

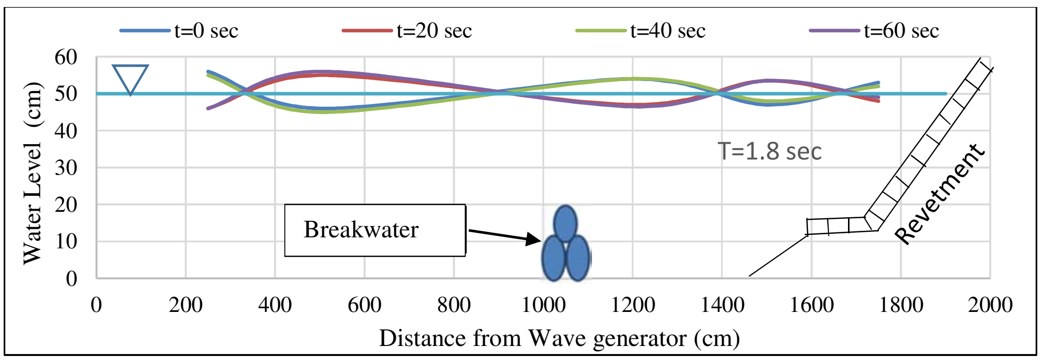
Figure 4.3: Water surface profile for relative breakwater height, $h_b/h_w = 0.3$.

4.3.2 Water surface profile for relative breakwater height 0.4:

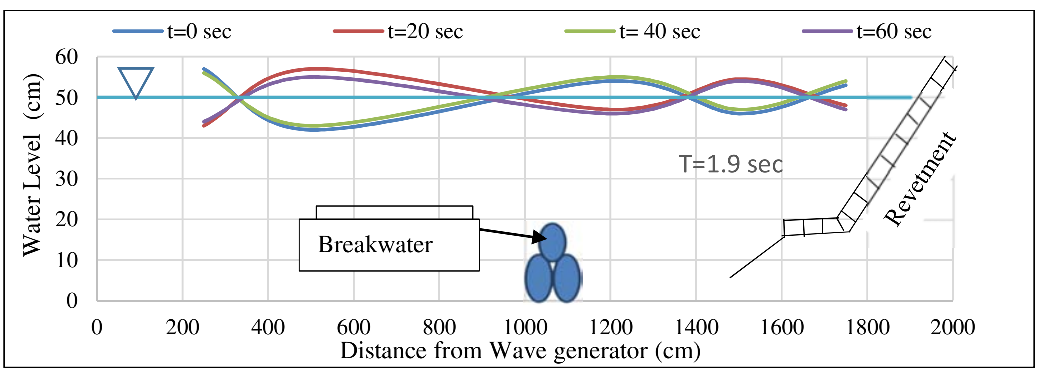
Figure 4.4(a) represents the water surface profile for relative breakwater height 0.4 and wave period 1.7 sec. The incident wave height becomes 11 cm for wave period 1.7 sec. The incident wave height is reduced 36% for relative breakwater height 0.4. Figure 4.4(b) shows the water surface profile for wave period 1.8 sec and relative breakwater height 0.4. The 12 cm incident wave height which is produced by wave period 1.8 sec is reduced 33% for the same relative breakwater height. The water surface profile for wave period 1.9 sec and relative breakwater height 0.4 is shown in Figure 4.4(c). The 1.9 sec wave period produces 14 cm incident wave height. 29% of incident wave height is reduced by the relative breakwater height 0.4 and for wave period 1.9 sec. Figure 4.4(d) illustrates the water surface profile for wave period 2.0 sec and relative breakwater height 0.4. 15 cm incident wave height is produced by wave period 2.0 sec. The relative breakwater height 0.4 reduces incident wave height to 27%. Therefore, the reduction of wave height decreases with increases with wave period for relative breakwater height 0.4.



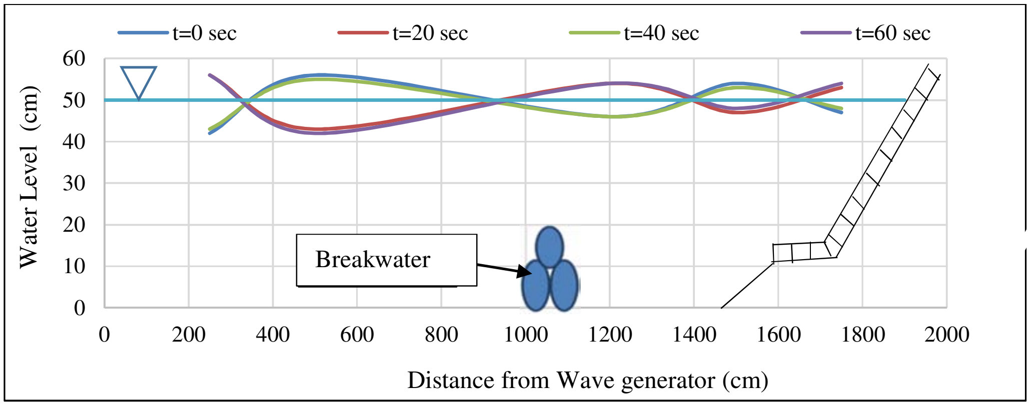
(a)



(b)



(c)

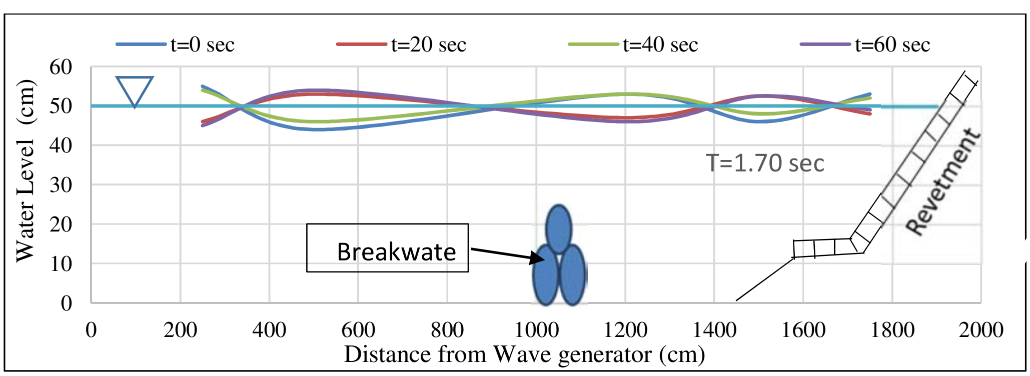


(d)

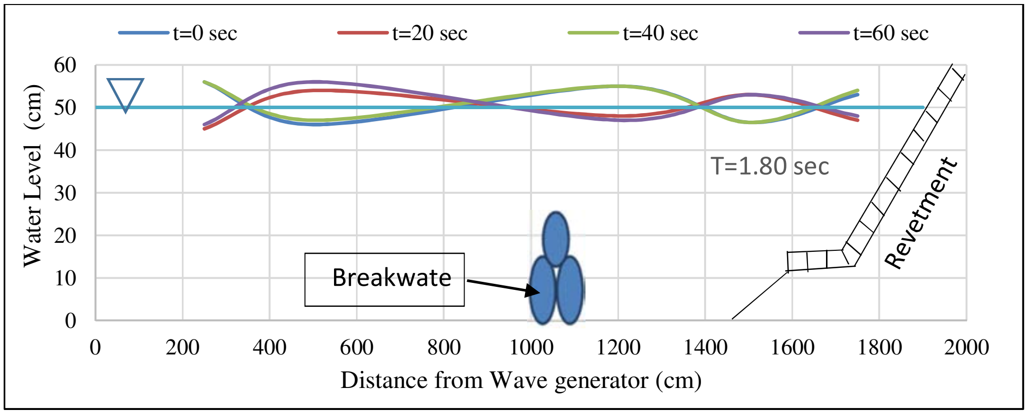
Figure 4.4: Water surface profile for relative breakwater height, $h_b/h_w = 0.4$.

4.3.3 Water surface profile for relative breakwater height 0.5:

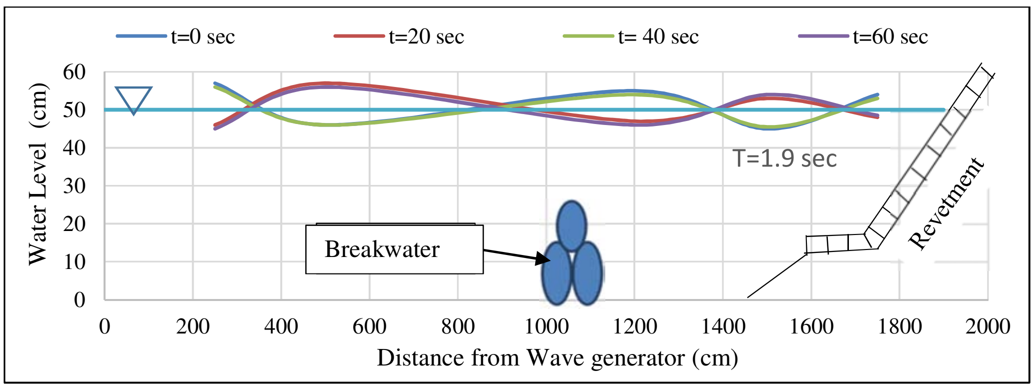
Figure 4.5(a) represents the water surface profile for wave period 1.7 sec and relative breakwater height 0.5. The wave period, T= 1.7 sec produces an incident wave height 11 cm. The incident wave height is reduced 45% for relative breakwater height 0.5. Figure 4.5(b) represents the water surface profile for wave period 1.8 sec and relative breakwater height 0.5. The incident wave height becomes 12 cm for the wave becomes 1.8 sec, and this incident wave height reduces 42% for the same relative breakwater height. Figure 4.5(c) shows the water surface profile for wave period 1.9 sec and relative breakwater height 0.5. 14 cm incident wave height are produced by the wave period 1.9 sec. 14 cm incident wave height are reduced by 39% for the relative breakwater height 0.5. The water surface profile for wave period 2.0 sec and relative breakwater height 0.5 is shown in Figure 4.5(d). The incident wave height is 15 cm for the wave period 2.0 sec. The relative breakwater height 0.5 reduces 37% incident wave height for wave period 2.0 sec. Thus, as the wave period increases the reduction of incident wave height is decreases for relative breakwater height 0.5.



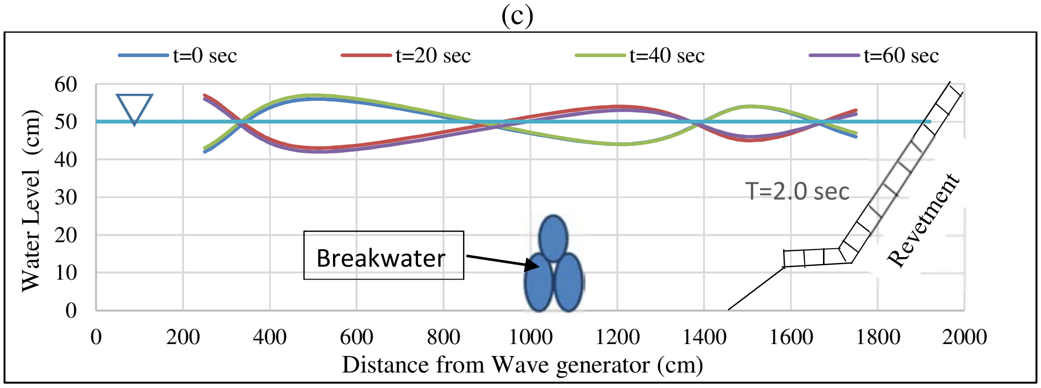
(a)



(b)



(c)

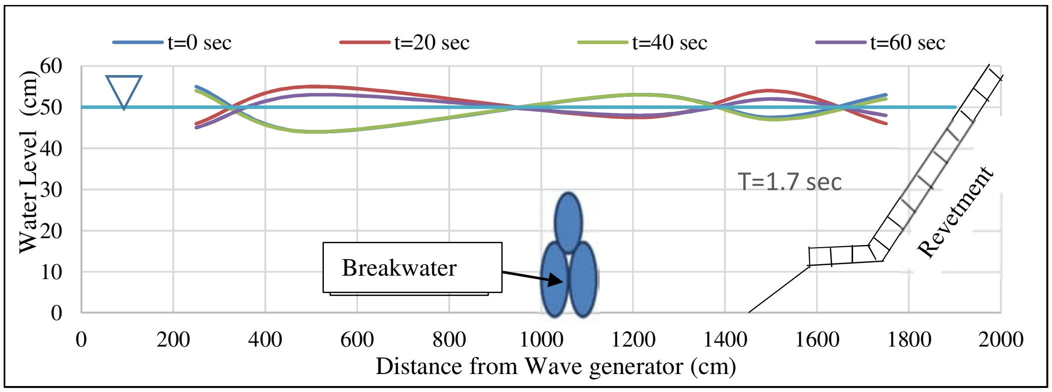


(d)

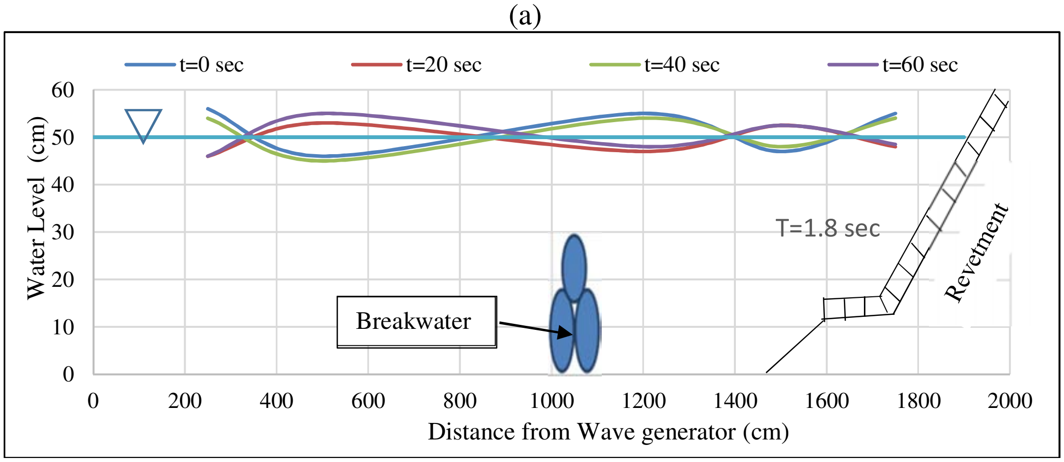
Figure 4.5: Water surface profile for relative breakwater height, $h_b/h_w = 0.5$.

4.3.4 Water surface profile for relative breakwater height 0.6:

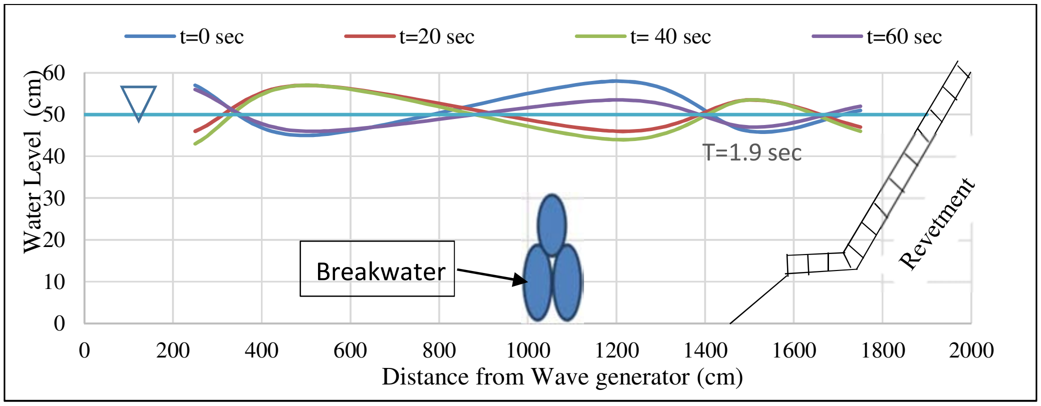
Figure 4.6(a) represents the water surface profile for relative breakwater height 0.6 and wave period 1.7 sec. The incident wave height becomes 11 cm for wave period 1.7 sec. The incident wave height is reduced 50% for relative breakwater height 0.6. Figure 4.6(b) shows the water surface profile for wave period 1.8 sec and relative breakwater height 0.6. The 12 cm incident wave height which is produced by wave period 1.8 sec is reduced 46% for the same relative breakwater height. The water surface profile for wave period 1.9 sec and relative breakwater height 0.6 is shown in Figure 4.6(c). The 1.9 sec wave period produces 14 cm incident wave height. 43% of incident wave height is reduced by the relative breakwater height 0.6 and for wave period 1.9 sec. Figure 4.6(d) illustrates the water surface profile for wave period 2.0 sec and relative breakwater height 0.6. 15 cm incident wave height is produced by wave period 2.0 sec. The relative breakwater height 0.6 reduces incident wave height to 40%. Therefore, the reduction of wave height decreases with increases with wave period for relative breakwater height 0.6.



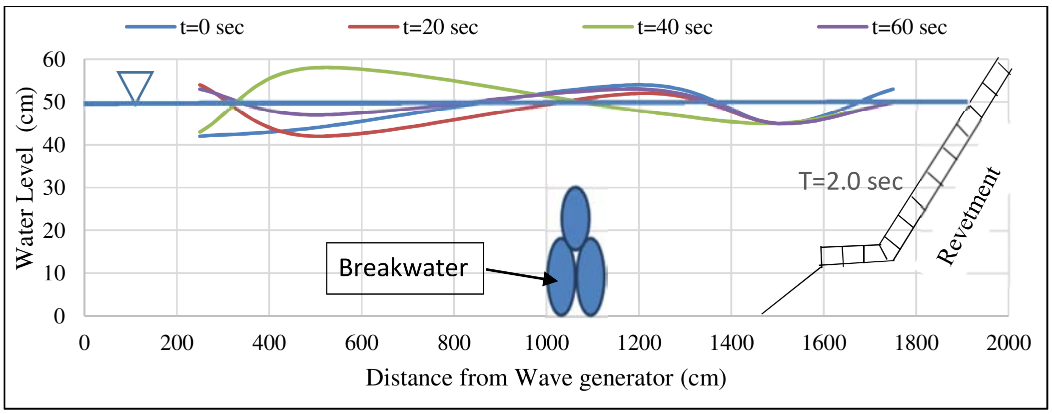
(a)



(b)



(c)



(d)

Figure 4.6: Water surface profile for relative breakwater height, $h_b/h_w = 0.6$.

4.4 Variation of η/H_i with t/T:

Variation of water surface (η) with time (t) (measured at location WG-1 and WG-4 in front and behind the breakwater respectively) is shown in the non-dimensional form as variation of η/Hi with t/T in from Figure 4.8 to Figure 4.11. The location of the WG-1 and WG-4 in the laboratory flume is shown in the following Figure 4.7

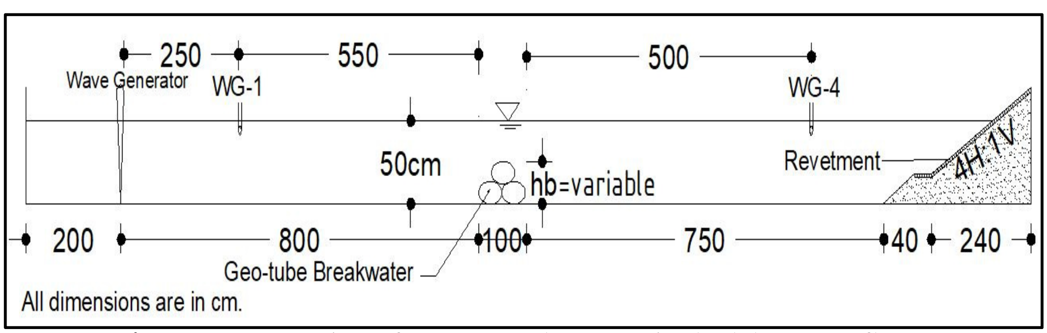


Figure 4.7: Location of WG-01 and WG-04 in the laboratory flume

4.4.1 Variation of η/H_i with t/T for wave period 1.7 sec:

Figure 4.8 shows that incident wave height for wave period 1.7 sec (measured at WG-01) reduces at significant amount after passing the submerged geo-tube breakwater. The transmitted wave height was measured at WG-04. The figures also show that as the relative breakwater height increases the transmitted wave height decreases for a particular wave period.

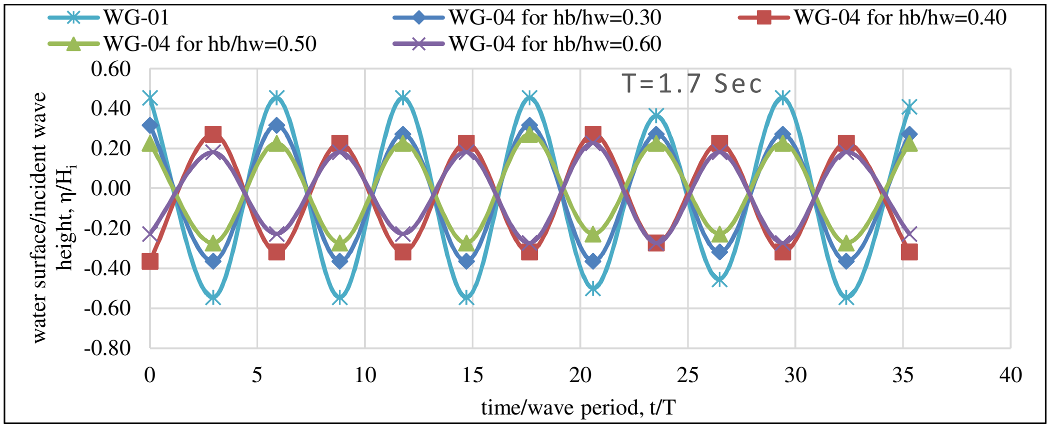


Figure 4.8: Variation of η/Hi with t/T for T= 1.7 sec, H_i= 11 cm

When h_b/h_w becomes 0.6 by installing submerged geo-tube breakwater of 30 cm height, the incident wave height of 11 cm for T=1.7 sec reduces to 5.5 cm after breaking over the breakwater. With submerged geo-tube breakwater of 25 cm height in 50 cm still water depth as the ratio of h_b/h_w becomes 0.5 for the same wave period, the incident wave height 11 cm is reduced to 6 cm. Installation of 20 cm breakwater in the same depth of still water (h_b/h_w =0.4) reduces 11 cm incident wave height to 7 cm. Installation of 15 cm breakwater in the same depth of still water (h_b/h_w =0.3) reduces 11 cm incident wave height to 7.5 cm. Thus for T=1.7 sec, breakwater having h_b/h_w=0.6 reduces 50% of incident wave height, whereas breakwater having h_b/h_w=0.5, h_b/h_w=0.4 and h_b/h_w =0.3 decreases incident wave height up to 45%, 36% and 32% respectively.

4.4.2 Variation of η/H_i with t/T for wave period 1.8 sec:

Figure 4.9 represent the variation of η/H_i with t/T for wave period 1.8 sec. In the figure the incident wave height was measured at the location of WG-1 and the transmitted wave height was measured at the location of WG-4.

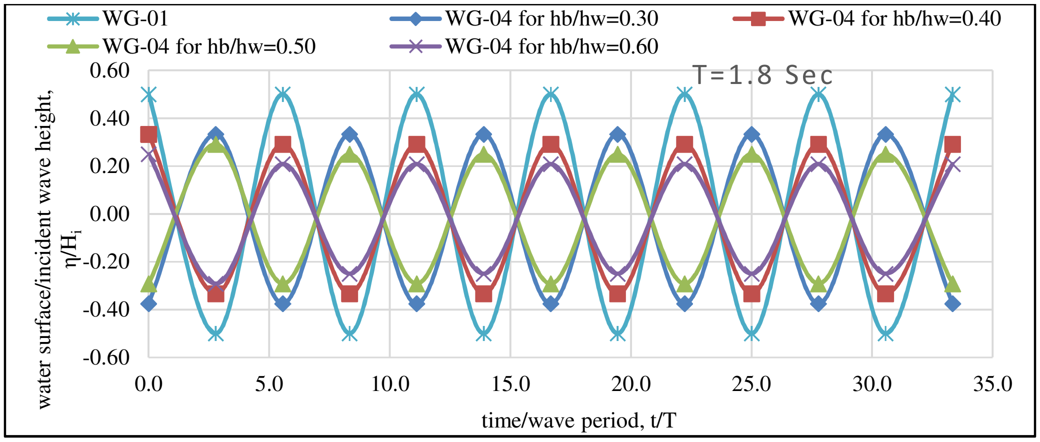


Figure 4.9: Variation of η/Hi with t/T for T= 1.8 sec, H_i= 12 cm

For T= 1.8 sec, 30 cm breakwater covering 60% of still water depth as h_b/h_w =0.6 reduces 46% of incident wave height, whereas 25 cm breakwater with h_b/h_w =0.5 decreases incident wave height up to 42%. 20 cm breakwater with h_b/h_w =0.4 decreases incident wave height up to 33% and minimum reduction of wave height (29%) is caused by 15 cm breakwater having h_b/h_w =0.3 for the same wave period.

4.4.3 Variation of η/H_i with t/T for wave period 1.9 sec:

Figure 4.10 represent the variation of η/H_i with t/T for wave period 1.8 sec. In the figure the incident wave height was measured at the location of WG-1 and the transmitted wave height was measured at the location of WG-4.

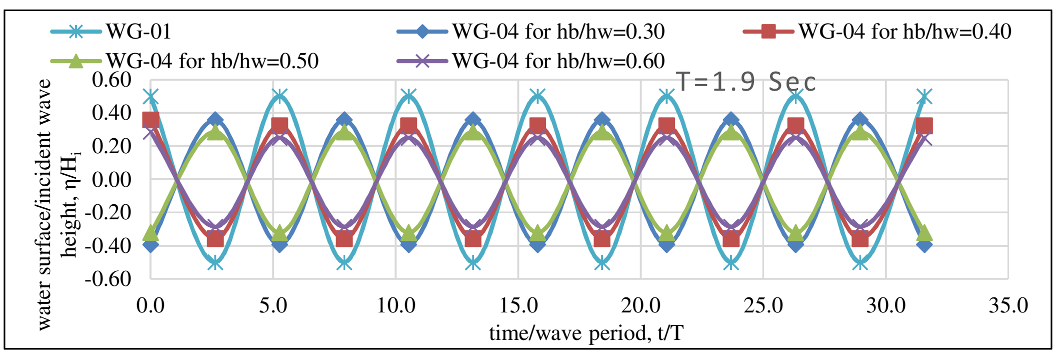


Figure 4.10: Variation of η/Hi with t/T for T= 1.9 sec, H_i= 14 cm

For T= 1.9 sec, maximum reduction of wave height is up to 43% which is caused by breakwater of 30 cm height (h_b/h_w =0.6). For the same wave period, 25 cm high breakwater having h_b/h_w =0.5 decreases incident wave height up to 39%, whereas 20 cm high breakwater reduces 29% of incident wave height, and minimum reduction of wave height (25%) is caused by 15 cm breakwater having relative breakwater height, h_b/h_w =0.3 for the same wave period. So for T= 1.9 sec, 30 cm high breakwater is most effective in reduction of wave height among the breakwaters of four different heights in 50 cm deep water.

4.4.4 Variation of η/H_i with t/T for wave period 2.0 sec:

Figure 4.11 shows that incident wave height for wave period 2.0 sec (measured at WG-01) reduces at significant amount after passing the submerged geo-tube breakwater. The transmitted wave height was measured at WG-04. The figures also show that as the relative breakwater height increases the transmitted wave height decreases for a particular wave period.

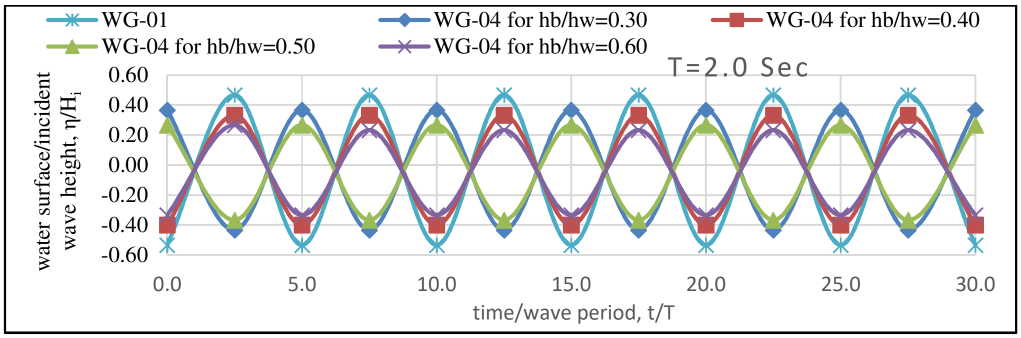


Figure 4.11: Variation of η/Hi with t/T for T= 2.0 sec, H_i= 15 cm

For T= 2.0 sec, breakwater covering 60% of still water depth as h_b/h_w =0.6 reduce 40% of incident wave height and breakwater having h_b/h_w =0.5 decrease incident wave height up to 37%, whereas breakwater having h_b/h_w =0.4 reduces 27% of incident wave height. But for this wave period the minimum reduction (20%) of wave height is caused by breaking of incident wave over 15 cm breakwater.

The above figures and discussion state that among the four different ratios of h_b/h_w for which the experiments are conducted, maximum reduction of wave height occurs for maximum value of h_b/h_w for the same wave period i.e. for ratio of h_b/h_w=0.6. Therefore, the reduction of incident wave height increases with the increase of relative breakwater height for a particular wave period.

4.5 Wave breaking:

A breaking wave is one whose base can no longer support its top, causing it to collapse. A wave breaks when it runs into shallow water, or two wave systems oppose or combine forces. When the slope, or steepness ratio of a wave is too great, breaking is inevitable. Usually an oncoming wave breaks due to any of the following reasons.

- Individual waves in deep water break when the wave steepness H/L (H= wave height, L = wave length) exceeds about 0.17, so for wave height, H> 0.17L.

- In shallow water, with the water depth small compared to the wavelength, the individual waves break when their wave height, H is larger than 0.8 times the water depth, h_w. That is for $H>0.8h_w$.

- Waves can also break if the wind grows strong enough to blow the crest off the base of the wave.

A wave can dissipate its energy in a very short time or gradually. Battjes (1974) shown that the breaker type is closely related to the offshore similarity parameter: $\xi_0 = \tan\alpha / (H_0/L_0)^{0.5}$

Where,

H_0 = deep water wave height, L_0 = deep water wave length, $\tan\alpha$ = beach slope

The breaker type can be distinguished based on the value of ξ_0 as follows:

$\xi_0 < 0.5$: spilling

$0.5 < \xi_0 < 3$: plunging

$\xi_0 > 3$: surging or collapsing

The main function of any breakwater is decreasing the wave energy by breaking the wave. As the breakwater height increases the breaking of incident wave also increases. In this experiment, the incident wave height breaks at submerged geo-tube breakwater and transmitted wave breaks and reflects at the revetment. The Figure 4.12 shows the breaking of the wave at submerged geo-tube breakwater.

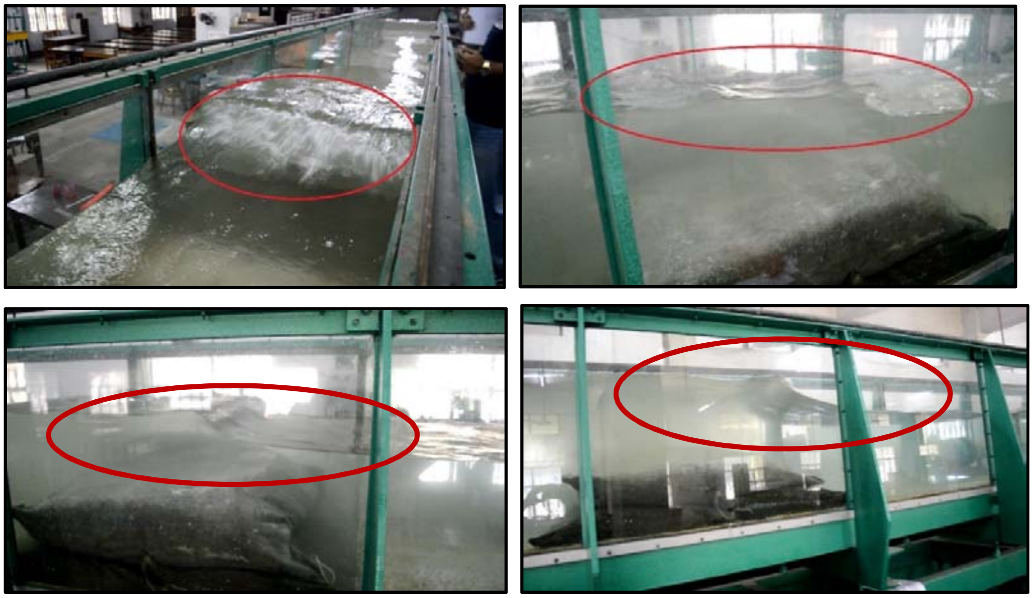


Figure 4.12: Wave breaking

To make the revetment stable, it is necessity that the relative breakwater height (h_b/h_w) will be well enough to make the transmitted wave height equal or less than 9.5cm in the laboratory.

4.6 Revetment stability:

Total 16 experimental runs were performed to investigate the performance of cement concrete block revetment and submerged geo-tube breakwater. As the maximum permissible wave height for single layer revetment is 1.5m, the size of concrete block was calculated taking prototype wave height 1.5m. The laboratory wave height was 9.5cm and the block size was 2cm x 2cm x 2cm which had been determined from Pilarczyk equation.

To understand the performance of geo-tube breakwater and revetment the experimental runs are discussed in below:

4.6.1 Revetment condition when relative breakwater height (h_b/h_w) = 0.3:

Four runs were conducted for four different wave period for relative breakwater height 0.3. The four different wave period was 1.7 sec, 1.8 sec, 1.9 sec and 2.0 sec. The duration of run was 10 minutes for all runs.

Figure 4.13(a) represent the revetment condition during experimental run for wave period 1.7 sec and relative breakwater height 0.3. The incident wave height was 11 cm. The submerged geo-tube breakwater reduces the incident wave height and the transmitted wave height becomes 7.5 cm. As the transmitted wave height (7.5 cm) was lower than the design wave height (9.0 cm), the revetment was stable after the run.

Figure 4.13(b) shows the revetment condition during the experimental run for wave period 1.8 sec and relative breakwater height 0.3. The incident wave height was 12 cm, and after breaking the transmitted wave height became 8.5 cm for wave period was 1.8 sec. So, the transmission coefficient was 0.71. The transmitted wave height (8.5 cm) was lower than the design wave height (9.50 cm). The revetment was stable after the run which confirm the theory.

The revetment condition during experimental run for wave period 1.9 sec and relative breakwater height 0.3 has been shown in the Figure 4.13(c). The incident wave height (H_i) was 14 cm, and the transmitted wave height (H_t) was 10.5 cm. As the transmitted wave height (10.5 cm) was greater than the design wave height (9.50 cm), the revetment was partially failed after the experimental run. Some cement concrete blocks were swift away from the left side of the revetment at the wave breaking zone.

Figure 4.13(d) represents the revetment condition during the experimental run of wave period 2.0 sec and relative breakwater height 0.3. The incident wave height (H_i) was 15 cm, and the transmitted wave height (H_t) was 12 cm after reduction the wave height by the submerged geo-tube breakwater. As the transmitted wave height (12 cm) was higher than the design wave height (9.50 cm), the revetment was unstable after the experimental run.

(a) (b) (c) (d)

Figure 4.13: Revetment condition for relative breakwater height, $h_b/h_w = 0.3$

4.6.2 Revetment condition when relative breakwater height (h_b/h_w) = 0.4:

Figure 4.14(a) represents the revetment condition during the experimental run for wave period 1.7 sec and relative breakwater height 0.4. The incident wave height (H_i) of this experimental run was 11 cm. The submerged geo-tube breakwater reduced the incident wave height and the transmitted wave height (H_t) became 7 cm. As the transmitted wave height (7 cm) was lower than the design wave height (9.50 cm), the revetment was stable after the experimental run.

Figure 4.14(b) shows the revetment condition during the experimental run for wave period 1.8 sec and relative breakwater height 0.4. The 1.8 sec wave period made 12 cm incident wave height (H_i). As the incident wave propagate, the submerged geo-tube breakwater reduces the incident wave height and the transmitted wave height (H_t) become 8 cm. As the transmitted wave height (8 cm) is lower than the design wave height (9.50 cm) the revetment was stable after the experimental run.

The revetment condition during the experimental run for the wave period 1.9 sec and relative breakwater height 0.4 is shown in the Figure 4.14(c). In this experimental run, the incident wave height (H_i) was 14 cm, and the transmitted wave height (H_t) was 10 cm. Though the transmitted wave height (10 cm) was greater than the design wave height (9.50 cm), the revetment was stable after the experimental run.

Figure 4.14(d) represents the revetment condition of the experimental run for wave period 2.0 sec and relative breakwater height 0.4. The incident wave height (H_i) was 15 cm, and the transmitted wave height (H_t) was 11 cm in this experimental run. As the transmitted wave height (11 cm) was greater than the design wave height (9.50 cm), the revetment was unstable after the run. Some cement concrete blocks were swift away from two points of the revetment at the wave breaking zone.

(a)

(b)

(c) (d)

Figure 4.14: Revetment condition for relative breakwater height, $h_b/h_w = 0.4$

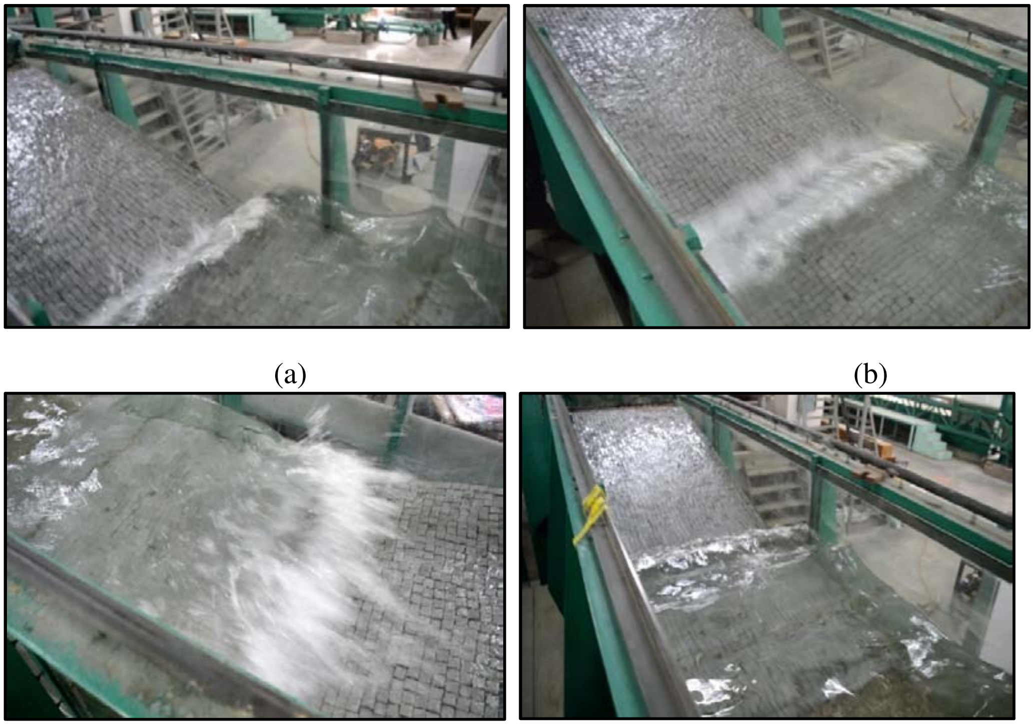
4.6.3 Revetment condition when relative breakwater height (h_b/h_w) = 0.5:

Figure 4.15(a) shows the revetment condition during experimental run for relative breakwater height 0.5 and wave period 1.7 sec. In this experimental run, the incident wave height (H_i) was 11 cm, and the transmitted wave height (H_t) was 6 cm. The transmitted wave height (6 cm) was lower than the design wave height (9.50 cm). The revetment was stable as expected.

The revetment condition during the experimental run has been shown in the Figure 4.15(b) for wave period 1.8 sec and relative breakwater height 0.5. The 1.8 sec wave period creates 12 cm incident wave height (H_i) in the water depth of 50 cm. The submerged geo-tube breakwater reduces incident wave height and the transmitted wave height (H_t) becomes 7 cm. As the transmitted wave height (7 cm) was lower than the design wave height (9.50 cm), the revetment was stable after the experimental run.

Figure 4.15(c) represents the revetment condition during the experimental run for wave period 1.9 sec and relative breakwater height 0.5. In this experimental run, the incident wave height (H_i) was 14 cm, and the transmitted wave height (H_t) was 8.5 cm. As the transmitted wave height (8.5 cm) was lower than the design wave height (9.50 cm, the revetment was stable after the experimental run.

The revetment condition of the experimental run for wave period 2.0 sec and relative breakwater height 0.5 has been shown in the Figure 4.15(d). The incident wave height (H_i) was 15 cm, and the transmitted wave height (H_t) was 9.5 cm in this experimental run. As the transmitted wave height (9.5 cm) was equal to the design wave height (9.50 cm), the revetment was stable after the experimental run.



(a) (b)

(c) (d)

Figure 4.15: Revetment condition for relative breakwater height, $h_b/h_w = 0.5$

4.6.4 Revetment condition when relative breakwater height (h_b/h_w) = 0.6:

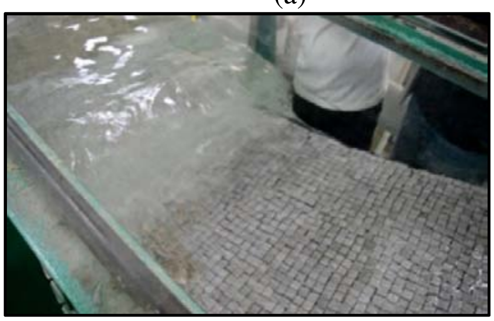
Figure 4.16(a) represents the revetment condition during the experimental run for wave period 1.7 sec and relative breakwater height 0.6. In this run, the incident wave height (H_i) was 11 cm, and the transmitted wave height (H_t) was 5.50 cm. The transmitted wave height (5.50 cm) was lower than the design wave height (9.50 cm). As expected, the revetment was stable after the experimental run.

Figure 4.16(b) shows the revetment condition during experimental run for wave period 1.8 sec and relative breakwater height 0.6. The 1.8 sec wave period creates 12 cm incident wave height (H_i) in the laboratory flume. During propagating over the submerged geo-tube breakwater, the incident wave height reduces and the transmitted wave height (H_t) becomes 6.5 cm. As Tte transmitted wave height (6.5 cm) was lower than the design wave height (9.50 cm), the revetment was stable after the experimental run.

The revetment condition during the experimental run for wave period 1.9 sec and relative breakwater height 0.6 has been shown in the Figure 4.16(c). The incident wave height (H_i) was 14 cm, and the transmitted wave height (H_t) was 8 cm in this experimental run. The transmitted wave height (8 cm) was lower than the design wave height (9.50 cm). The revetment was stable after the experimental run.

Figure 4.16(d) represents the revetment condition during the experimental run for wave period 2.0 sec and relative breakwater height 0.6. The incident wave height (H_i) was 15 cm in this experimental run. The submerged geo-tube breakwater reduces the wave height and the transmitted wave height (H_t) become 9 cm. As the transmitted wave height (9 cm) was lower than the design wave height (9.50 cm), the revetment was stable after the experimental run.


(a) (b)




(c) (d)

Figure 4.16: Revetment condition for relative breakwater height, $h_b/h_w = 0.6$

4.7 Summary of results of experimental runs:

In this study, total sixteen experimental runs were performed to investigate the wave interaction of submerged geo-tube breakwater in front of cement concrete revetment. The details of the experimental runs are discussed in the precious sections. Table 4.1 shows the summary of results of the experimental runs.

Table 4.1: *summary of results of experimental runs*

Run No	Relative breakwater height ($\frac{h_s}{h_w}$)	Wave period (T) sec	Incident Wave Height, H_i (cm)	% of reduction of wave height ($\frac{Hi-Ht}{Hi}$)	Revetment Condition after 10 minutes experimental run
1	0.3	1.7	11	32	Stable
2		1.8	12	29	Stable
3		1.9	14	25	Partially Failed
4		2.0	15	20	Partially Failed
5	0.4	1.7	11	36	Stable
6		1.8	12	33	Stable
7		1.9	14	29	Stable
8		2.0	15	27	Partially Failed
9	0.5	1.7	11	45	Stable
10		1.8	12	42	Stable
11		1.9	14	39	Stable
12		2.0	15	37	Stable
13	0.6	1.7	11	50	Stable
14		1.8	12	46	Stable
15		1.9	14	43	Stable
16		2.0	15	40	Stable

Revetment was stable for twelve experimental runs (experimental run no: 1, 2, 5, 6, 9, 10, 11, 12, 13, 14, 15, and 16) among the sixteen experimental runs. The reason for the stability of those experimental runs is that the transmitted wave height of those experimental runs was equal to or less than the design wave height (9.5 cm).

Revetment was partially failed for three experimental runs (experimental run no: 3, 4, and 8). The transmitted wave height of those experimental runs was higher than the design wave height (9.5 cm).

For experimental run-7, though the transmitted wave height (10 cm) was higher than the design wave height (9.50 cm), the revetment was stable after the experimental run. One of the reasons is that the wave height use to determine the size of C.C. block by Pilarczyk equation is significant wave height. The significant wave height (H_s) is defined as the mean wave height of the highest third of the waves. So, there will be higher waves than significant wave height in a wave spectrum. Another reason is the experimental run time. In this experiment, the experimental run time was 10 minutes. The revetment may fail if the experimental run time is increased.

4.8 Relationship among transmission coefficient (K_t), relative breakwater height (h_b/h_w), wave period (T) and significant wave height (H_s):

One of the objectives of this study was to establish a relation among significant wave height (H_s), wave period (T), relative breakwater height (h_b/h_w) and transmission co-efficient (K_t). Following parameters are the governing parameters associated with the performance of breakwaters considering the transmission of waves (Rathnayaka, Rathnayaka and Pathirana, 2016).

$$K_t = f\,(H_s, T, h_w, h_b, B, D_{50}, \tan\alpha, g) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.1)$$

Where, H_s is significant wave height, T is wave period, h_w is the depth of water, h_b is the breakwater height, B is breakwater width, D_{50} is the medium size of the material, $\tan\alpha$ is the seaward slope, g is the acceleration due to gravity.

To develop a relationship it is necessary to make dimensionless parameters. 'Buckingham Pi Theorem' is an established method for dimensionless analysis. In this research, the independent dimensions H_s and T were used as repeating variables. The following dimensionless groups were formed after the dimensionless analysis.

$$K_t = f\,(\frac{H_s}{h_w}, \frac{H_s}{h_b}, \frac{H_s}{B}, \frac{H_s}{D_{50}}, \frac{gT^2}{H_s}, \tan\alpha) \quad \ldots\ldots\ldots\ldots\ldots\ldots.. (4.2)$$

Since breakwater width B, medium size D_{50}, seaward angle α has not been changed in this experiments, $\frac{H_s}{B}, \frac{H_s}{D_{50}}, \tan\alpha$ these three dimensionless parameters have not been analyzed. $\frac{H_s}{h_w}, \frac{H_s}{h_b}$ these two dimensionless parameters have been simplified, and a new dimensionless parameter (relative breakwater height, $\frac{h_b}{h_w}$) has been analyzed. So, for this research, the transmission coefficient is a function of the following parameters.

$$K_t = f\,(\frac{h_b}{h_w}, \frac{gT^2}{H_s}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.3)$$

The experimental result of these parameters has been presented in the Table 4.2.

***Table 4.2:** Dimensionless parameters of the experimental runs*

Run No	T (sec)	H_i (cm)	h_b/h_w	gT^2/H_i	K_t
01	1.7	11	0.3	257.74	0.68
02	1.8	12	0.3	264.87	0.71
03	1.9	14	0.3	252.95	0.75
04	2	15	0.3	261.60	0.80

05	1.7	11	0.4	257.74	0.64
06	1.8	12	0.4	264.87	0.67
07	1.9	14	0.4	252.95	0.71
08	2	15	0.4	261.60	0.73
09	1.7	11	0.5	257.74	0.55
10	1.8	12	0.5	264.87	0.58
11	1.9	14	0.5	252.95	0.61
12	2	15	0.5	261.60	0.63
13	1.7	11	0.6	257.74	0.50
14	1.8	12	0.6	264.87	0.54
15	1.9	14	0.6	252.95	0.57
16	2	15	0.6	261.60	0.60

These parameters has been analyzed using Microsoft excel solver tool and the following equation has been found.

$$K_t = 1.05 - 0.67(\frac{h_b}{h_w})^{0.83} - 2.4 \times 10^{-4}(\frac{gT^2}{H_s}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.4)$$

Where,

K_t = transmission coefficient

H_s = significant wave height

T = wave period

h_w = the depth of water

h_b = the breakwater height

g = the acceleration due to gravity

$\frac{h_b}{h_w}$ = relative breakwater height

The intercept of the equation 4.4 is positive (1.05) and the slope of relative breakwater height is negative (-0.67). Thus means the transmission coefficient will decrease with increase of relative breakwater height for any particular wave.

The sum of squired residuals (SSR) of the equation (4.4) is 0.026. The sum of squired residuals is the sum of squired residuals, and residuals is the deviation of the calculated transmission coefficient by the Equation 4.4 from the laboratory transmission coefficient. The mathematical expression of SSR is states below.

$$SSR = \sum_{i=1}^{n}(y_i - f(x_i))^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.5)$$

Where, y_i is the i^{th} value from laboratory experiment and $f(x_i)$ is the i^{th} value calculated from the new developed relation.

Using the results of the experiment a relationship (Equation 4.4) has been established. The graphical presentation of the relationship is in the Figure 4.17. In the Figure, Y-axis represents the transmission coefficient, K_t. X-axis and Z-axis represent gT^2/H_s and h_b/h_w (relative breakwater height) respectively. The dot(.) in the Figure represents experimental data, and the shaded area represents the developed relation.

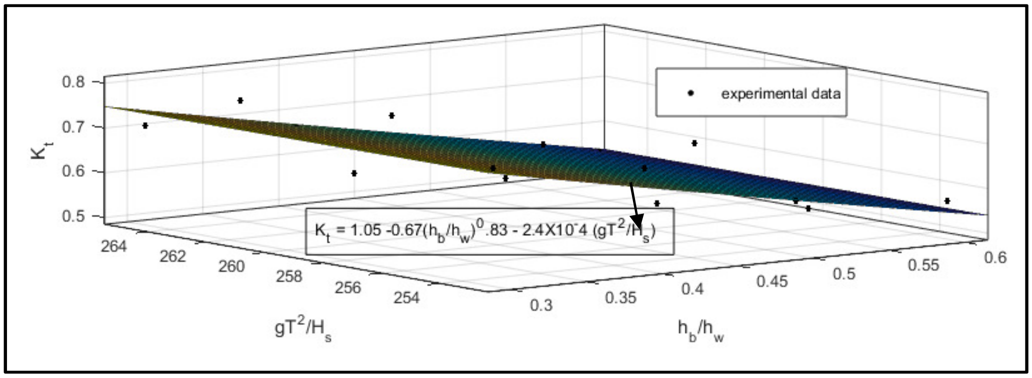


Figure 4.17: Relationship among transmission coefficient (K_t), relative breakwater height (h_b/h_w), wave period (T) and significant wave height (H_s)

4.9 Shore protection design along the Cox's Bazar shoreline using developed relation:

Institute of Water Modelling conducted a study named "Coastal Hydraulic and Morphological Study and Design of Protection Measures for Marine Drive Road". In this study, they suggest sleeping defence for the protection of the Marine Drive Road. The disadvantages of sleeping defence are nourishment must be carried out at regular intervals and it involves mobilisation of special equipment. In the socio-economical context of Bangladesh, it is difficult to maintain a structure regularly and it also involve a huge maintaining cost. Therefore, finally the proposed method of that study was not implemented. In this study two layer protection was experimentally investigated to find a new solution of this problem. Extreme wave analysis has been performed at 11 locations of Cox's Bazar sea beach in the aforementioned study. The relative positions of the 11 points chosen along the shoreline are depicted in Figure 4.18.

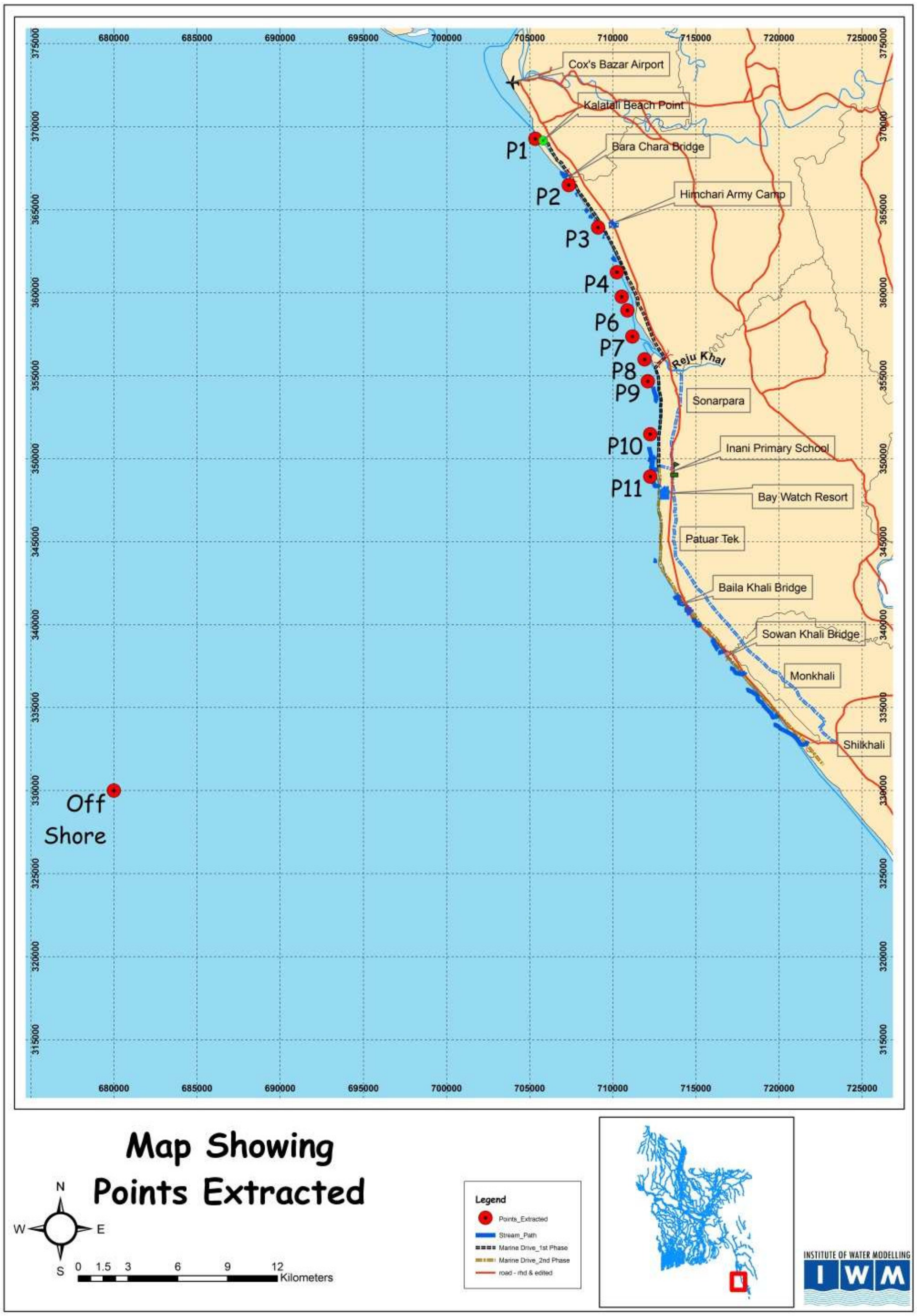


Figure: 4.18: Nearshore location of Extreme Wave Analysis (IWM-BUET,2014)

The significant wave height and wave period of the 11 locations presented in the Table 4.3

Table 4.3 *Design Wave Characteristics along Cox's Bazar Shoreline (from Extreme Wave Analysis)*

Point No	Estimated Design Wave Height (H_s) (m) with Return Period= 30 years	Associated Deep water Mean Wave Period (T_{mo}) (sec)
1	2.32	8.65
2	2.37	8.62
3	2.38	8.7
4	2.28	8.74
5	2.31	8.7
6	2.25	8.79
7	2.43	8.7
8	2.35	8.57
9	2.5	9.09
10	2.42	8.76
11	2.53	8.51

One of purpose of this research is reducing the incident wave height by the breakwater and the transmitted wave height will be break and reflected by revetment. The maximum permissible wave height recommended by the U.S. Army of Crops Engineers for one layer revetment is 1.50 m. So, the breakwater height has to be selected in such a way that the transmitted wave height becomes <= 1.50 m. The relative breakwater height of the 11 location has been designed by the equation no 4.4 and presented in the following Table 4.4.

Table 4.4: *Relative breakwater height calculation*

Point No	Location	Measured data at Cox's Bazar		Max. permissible wave height for C.C. block revetment	$K_t = H_t/H_s$	gT^2/H_s	Relative breakwater height, h_b/h_w
		H_s (m)	T (sec)	H_t (m)			
1	Kalatali beach	2.32	8.65	1.5	0.65	316.38	0.42
2	Bara chara bridge	2.37	8.62	1.5	0.63	307.56	0.45
3	Himchari Army camp	2.38	8.7	1.5	0.63	311.98	0.45
4	-	2.28	8.74	1.5	0.66	328.67	0.40
5	-	2.31	8.70	1.5	0.65	321.44	0.42
6	-	2.25	8.79	1.5	0.67	336.87	0.38
7	-	2.43	8.7	1.5	0.62	305.56	0.47
8	-	2.35	8.57	1.5	0.64	306.59	0.44
9	-	2.5	9.09	1.5	0.60	324.23	0.49
10	-	2.42	8.76	1.5	0.62	311.07	0.47
11	Bay watch resort	2.53	8.51	1.5	0.59	280.81	0.52

The last column of Table 4.4 represents the relative breakwater height, h_b/h_w, which has been calculated by the developed relation (Equation 4.4). This parameter will help the coastal engineer to design two-layer protection at marine drive road, Cox's Bazar considering the maximum wave height limitation of C.C. block revetment.

4.10 Practical considerations to use the developed formula:

The water level fluctuates at every moment due to the tidal effect in the coastal area. There are also differences in tide water level due to spring tide and neap tide. Therefore, the water level is an importation parameter, because relative breakwater height depends on it. Statistical analysis has to perform using the high water level of spring tide. Therefore, the breakwater may emerge during neap tide. If the neap tide water level is considered for statistical analysis, the relative breakwater height will be insufficient during spring tide. Therefore, wave height will reduce less, which eventually will lead to the failure of the revetment.

The bearing capacity of the foundation of the breakwater shall be well enough to carry the pressure of the breakwater. If the bearing capacity of the foundation is not sufficient, the breakwater will settle (Figure 4.19).

Figure 4.19: Emerged geo-tube settled in sandy beach (© M.A. Rahman)

Settled geo-tube will eventually decrease the relative breakwater height. The decreased relative breakwater will reduce incident wave relatively less than the initial relative breakwater height. Thus, the transmitted wave height will be large than initial transmitted wave height. Therefore, the revetment at the shore may fail due to excess wave energy. Therefore, the foundation of the breakwater shall be considered with attention.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 General:

Segmented, detached, reef, submerged or emerged breakwaters, revetment with different armor units are variations of coastal structures built to protect the shoreline. The primary function of such structures is to intercept the incident waves and cause them to break or reflect. In this research, the interaction between wave and submerged geo-tube breakwater in front of a revetment has been investigated experimentally to find out the effective height of submerged breakwater against the stability of the revetment. In a two dimensional wave flume, sixteen experimental runs have been conducted with submerged geo-tube breakwater having breakwater height of (h_b=15cm, 20cm, 25cm, and 30cm) in constant water depth, h_w=50cm for four different wave period as T=1.7sec, 1.8sec, 1.9 sec, 2.0 sec respectively in front of a C.C. block revetment having concrete block size of 2 cm x 2 cm x 2 cm.

5.2 Conclusions:

The performance of the submerged geo-tube breakwater and revetment were experimentally investigated. By conducting detail and rigorous experimental investigation and data analysis, presented in the previous chapters, the key findings of the study have been presented below:

1) This is evident from this study that a submerged geo-tube breakwater is effective in reducing wave energy.

2) From the experimental investigations, it is found that for any particular wave period the relative breakwater height, h_b/h_w (h_b= breakwater height and h_w= water depth) and the relative breakwater width B/L (B=breakwater width along the wave direction and L= wave length) are the important parameters for the reduction of incident wave height as mentioned below.

    a) As the relative breakwater height (h_b/h_w) increases, the incident wave reduces more due to breaking caused by the submerged breakwater. For wave period, T=1.7 sec wave height reduction is 32% when breakwater height is 15cm.

When breakwater height is increased to 20cm, 25cm and 30cm respectively for same wave period, the incident wave is reduced up to 36%, 45%, and 50%.

b) As the relative breakwater width (B/L) increases, the reduction of wave height also increases. For relative breakwater height h_b/h_w =0.6, when B/L= 0.25, reduction of wave height due to breaking is 40%, whereas submerged body with B/L= 0.3 can reduce wave height up to 50%.

3) Revetment was stable when transmitted wave height was equal or less than design wave height. When transmitted wave height was higher than design wave height revetment was unstable.

4) Relationship among transmission coefficient (K_t), relative breakwater height (h_b/h_w), wave period (T) and significant wave height (H_s) is $K_t = 1.05 - 0.67(\frac{h_b}{h_w})^{0.83} - 2.4 \times 10^{-4}(\frac{gT^2}{H_s})$.

5) Submerged geo-tube breakwater in front of cement concrete block revetment is an effective shore protection measure for higher wave heights.

5.3 Recommendations:

Naturally any coastal protection work is exposed to continuous hitting by multidirectional random waves, where wind is one the most important driving force to control the wave height as well as to create turbulence in the free surface of water. In this research, the effectiveness of submerged geo-tube breakwater and C.C. block revetment exposed to unidirectional regular waves in the reduction of wave height has been investigated experimentally, where the contribution of wind force induced turbulence has not been considered. To overcome the limitations of the present study, the followings are recommended:

1) Multi-directional random waves can be created from different wave generators simultaneously in a large wave basin.

2) Numerical investigation using numerical methods can be done. Breakwater using different shape and height, and revetment with different slope can be investigated numerically. These investigations also can be done experimentally.

Reference:

Akter, A. (2013) *Experimental study on hydrodynamic performance of vertical thick porous breakwater experimental study on hydrodynamic performance of vertical thick porous,* Msc. thesis, Department of Water Resourses Engineering, Bangladesh University of Engineering and Technology.

Breteler, M. K. *et al.* (2012) 'Resilience of dikes after initial damage by wave attack', in *Proceedings of the Coastal Engineering Conference*, pp. 1–14. doi: 10.9753/icce.v33.structures.36.

Breteler, M. K., Gijsbert, M. and Yvo, P. (2014) 'Stability of Placed Block Revetments in the Wave Run-Up Zone', *Coastal Engineering Proceedings*, 1(34), pp. 1–12. doi: 10.9753/icce.v34.structures.24.

Cardenas-Rojas, D. *et al.* (2021) 'Assessment of the performance of an artificial reef made of modular elements through small scale experiments', *Journal of Marine Science and Engineering*, 9(2), pp. 1–18. doi: 10.3390/jmse9020130.

Chen, J. *et al.* (2016) 'Laboratory study on protection of tsunami-induced scour by offshore breakwaters', *Natural Hazards*, 81(2), pp. 1229–1247. doi: 10.1007/s11069-015-2131-x.

Coastal Zone Policy, (2005) 'Coastal Zone Policy, Ministry of Water Resources, Government of the People's Republic of Bangladesh', *Policy*, pp. 1–14.

Dick, T. M. and Brebner, A. (1968) 'Solid and Permeable Submerged Breakwaters', in *11th International Conference on Coastal Engineering*, pp. 1141–1158. doi: https://doi.org/10.1061/9780872620131.072.

'Fundamentals of Design' (2011) in *Coastal Engineering Manual, Part IV*. US Army Corps of Engineers, pp. VI-5-1-VI-5–356

Gupta, G. S. *et al.* (2014) 'Chapter 3.1 - Process Concept for Scaling-Up and Plant Studies', in Seetharaman, S. B. T. (ed.). Boston: Elsevier, pp. 1100–1144. doi: https://doi.org/10.1016/B978-0-08-096988-6.00040-7.

Hidayat, H. and Andrianto, S. (2018) 'Effectiveness of geotextile tubes as a breakwater core', in *Coastal Engineering Proceedings*. doi: 10.9753/icce.v36.papers.80.

Hossain, I. (2013) *Experimental Study on Stability of Different Types of Armor Units Used in Shore Protection Structure Experimental Study on Stability of Different Types of Armor Units Used in Shore Protection Structure,* Msc. thesis, Department of Water Resourses Engineering, Bangladesh University of Engineering and Technology.

Islam, M. R. and Ahmad, M. (2004) *Living in the Coast: Problems, Opportunities and Challenges*, *Development*.

IWM and BUET (2014) 'Coastal Hydraulic and Morphological Study and Design of Protection Measures for Marine Drive Road'.

Jeng, D. S., Schacht, C. and Lemckert, C. (2005) 'Experimental study on ocean waves propagating over a submerged breakwater in front of a vertical seawall', *Ocean Engineering*, 32(17–18), pp. 2231–2240. doi: 10.1016/j.oceaneng.2004.12.015.

Kamal, A. M. U. and Rob, K. (2003) *Delineation of the Coastal Zone*. Working Paper WP005.

Katsaprakakis, D. Al (2020) *Wave and Wind Energy*. Salina, Italy, European Union..

Kawasaki, K. and Iwata, K. (2001) 'Wave breaking-induced dynamic pressure due to submerged breakwater', in *Eleventh International Offshore and Polar Engineering Conference*. Stavanger, Norway, pp. 488–494.

Kerpen, N. B., Schoonees, T. and Schlurmann, T. (2019) 'Wave overtopping of stepped revetments', *Water (Switzerland)*, 11(5), pp. 1–17. doi: 10.3390/w11051035.

Koerner, R. M. (2000) 'Emerging and future developments of selected geosynthetic applications', *Journal of Geotechnical and Geoenvironmental Engineering*, 126(April), pp. 293–306.

Liao, Y. C. *et al.* (2013) 'Experimental study of wave breaking criteria and energy loss caused by a submerged porous breakwater on horizontal bottom', *Journal of Marine Science and Technology (Taiwan)*, 21(1), pp. 35–41. doi: 10.6119/JMST-011-0729-1.

Nandi, B. (2002) *A study on the stability of c.c. blocks as revetment material against wave attack,* Msc. thesis, Department of Water Resourses Engineering, Bangladesh University of Engineering and Technology.

Permana, M. S., Triatmodjo, B. and Yuwono, N. (2017) ‘Wave-Induced Pressure Distribution on Placed Perforated Revetment Block’, *Procedia Engineering*, 170, pp. 443–450. doi: 10.1016/j.proeng.2017.03.071.

Pilarczyk, K. W. (1990) ‘Design of Seawalls and Dikes — Including Overview of Revetments’, in *Coastal Protection*. Balkema, The Netherlands.

Rageh, O. S. (2009) ‘Hydrodynamic Efficiency of Vertical Thick Porous Breakwaters’, in *Thirteenth International Water Technology Conference*, pp. 1659–1671.

Rahaman, A. Z. and Rahman, A. (2013) ‘Estimation of Design of Wave Height along Marine Drive Road from Kalatali to Inani at Cox’s Bazar’, in *4th International Conference on Water and Flood Management*.

Rahaman, T. and Hossain, S. (2015) ‘Marine Drive Protection By Formation of Sea Forest Creation Utilizing By-Product Slag of Steelmaking Process on Bay of Bengal’.

Rahman, A. and Womera, S. (2013) ‘Experimental and Numerical Investigation on Wave Interaction with Submerged Breakwater’, *Journal of Water Resources and Ocean Science*, 2(6), pp. 155–164. doi: 10.11648/j.wros.20130206.11.

Rathnayaka, R. M. D. B., Rathnayaka, R. M. J. R. and Pathirana, K. P. P. (2016) ‘Experimental Investigation of Transmission Coefficient of Reef Breakwaters’, *Engineer: Journal of the Institution of Engineers, Sri Lanka*, 49(1), pp. 31–37. doi: 10.4038/engineer.v49i1.6916.

Shabankareh, O., Ketabdari, M. J. and Shabankareh, M. A. (2017) ‘Environmental Impact of Geotubes and Geotextiles used in Breakwaters and Small Breakwaters Construction (Case Study : Rigoo Public Breakwater in South of Qeshm island - Iran)’, *International Journal of Coastal & Offshore Engineering*, 5, pp. 9–14.

Sonin, A. A. (2004) ‘A generalization of the Pi-theorem and dimensional analysis.’, in *Proceedings of the National Academy of Sciences of the United States of America*. National Academy of Sciences, pp. 8525–8526. doi: 10.1073/pnas.0402931101.

Sultana, S. and Rahman, A. (2019) ‘Experimental Study on Wave Interaction with Multiple Row Pile Breakwater’, in *IOP Conference Series: Earth and Environmental Science*, pp. 1–9. doi: 10.1088/1755-1315/326/1/012014.

Taylor, E. S. (1974) *Dimensional Analysis for Engineers*. Clarendon, Oxford.

The Environment Conservation Rules (1997) *Government of the People's Republic of Bangladesh Ministry of Environment and Forest*.

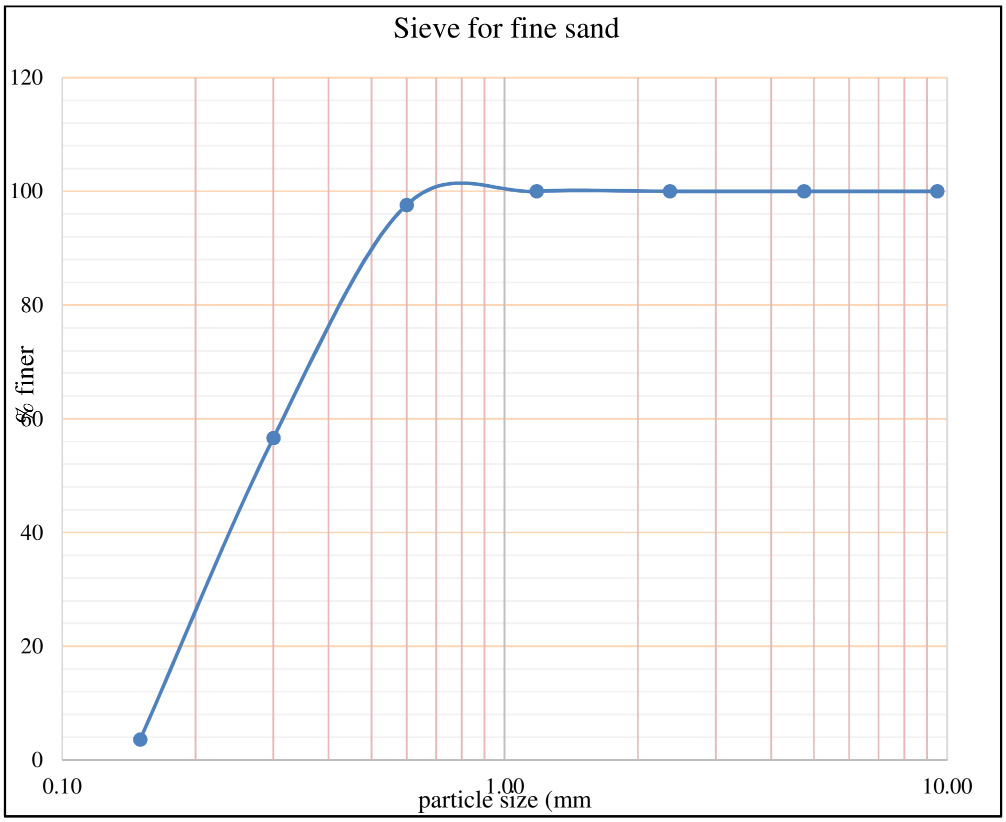
Vona, I., Gray, M. W. and Nardin, W. (2020) 'The impact of submerged breakwaters on sediment distribution along marsh boundaries', ***Water (Switzerland)***, 12(4). doi: 10.3390/W12041016.

Water Resources Planning Organization, Ministry of Water Resources,Government of Bangladesh. (2006) 'Coastal development strategy', (February), pp. 55–64.

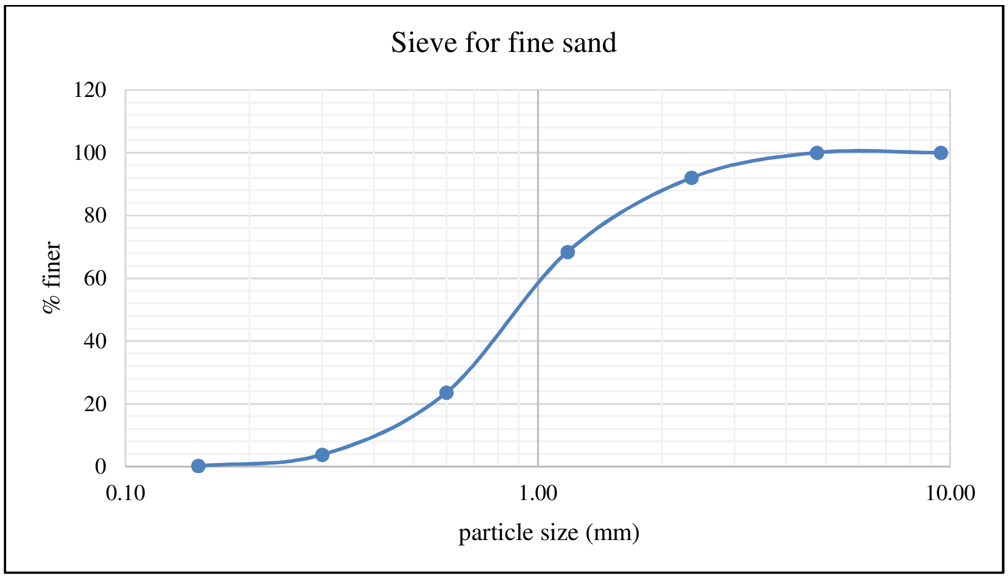
Williams, A. ***et al.*** (2018) 'The management of coastal erosion', *Ocean & Coastal Management*, 156, pp. 4–20. doi: 10.1016/j.ocecoaman.2017.03.022.

Appendix-A

Sample : Fine Sand (500g)					
Sieve size (mm)	Retain (g)	% Retain	Cumulative % retain	% passing	FM
9.50	0	0	0	100	1.42
4.75	0	0	0	100	
2.36	0	0	0	100	
1.18	0	0	0	100	
0.60	12	2.4	2.4	97.6	
0.30	205	41	43.4	56.6	
0.15	265	53	96.4	3.6	
pan	18	3.6	100	0	



Sample : Coarse Sand (500g)					
Sieve size (mm)	Retain (g)	% Retain	Cumulative % retain	% passing	FM
9.50	0	0	0	100	3.12
4.75	0	0	0	100	
2.36	40	8	8	92	
1.18	118	23.6	31.6	68.4	
0.60	226	45.2	76.4	23.6	
0.30	97	19.4	96.2	3.8	
0.15	18	3.6	99.8	0.2	
pan	1	0.2	100	0	



Nomo gram to obtain the value of e and f for the wave generator