



CONSTRUCTION WORKS OF PROTOTYPE COASTAL WAVE BARRIER AND SHORELINE PROTECTION STRUCTURE

FINAL REPORT



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NATIONAL WATER RESEARCH INSTITUTE OF MALAYSIA (NAHRIM)**

PREPARED BY

**IR. TS. MUHAMMAD RIZAL RAZALI
SITI SALIAH HJ. MOHD SENDEK**

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Project Director

Ir. Dr. Lee Hin Lee

Deputy Project Director

Ts. Saiful Bahri Hamzah

Project Manager

Ir. Ts. Muhammad Rizal Razali

Technical Advisor

Prof. Madya Ir. Ts. Dr. Mohamad Hidayat Jamal, UTM
Mohd Kamarul Huda Samion

Project Team

Imran Abdul Halim Zaki

Siti Salihah Hj. Mohd Sendek

Tc. Al Imran Abdul Manan

Mohd Yasin Sulo

Muhammad Zulkhairi Zulkafli

Saiful Izmi Ahmad

Hairy Ijun

Lavine Wong (UTM Research Assistant)

Mohamad Khairi Akmal Bin Mohamad Kamal (UTM Research Assistant)

Nur Kamilia Afiqah Mohammad Sofian (UMT Internship)

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1.0 INTRODUCTION

National Water Research Institute Malaysia (NAHRIM) has conducted research on crown unit designs application to reduce wave runup and overtopping for the study “Construction Works of Prototype Coastal Wave Barrier and Shoreline Protection Structure” in completion of the analysis and documentation.

This document describes detail aspects of the scope of work that has been conducted. The construction works have been undertaken by NHI services will analyse the output from a series of experimental works to determine efficiency of proposed designed crown units by NAHRIM. The report includes the following aspects:

- (i) Project background
- (ii) Literature review
- (iii) Experimental work set-up
- (iv) Results and analysis
- (v) Assessment of crown units’ efficiency
- (vi) Report writing and documentation

1.1 Project Background

Wave overtopping is a complex coastal phenomenon with significant implications for coastal communities and ecosystems. Studying and managing overtopping events are essential components of coastal management and engineering, helping to mitigate risks, protect infrastructure, and adapt to the challenges posed by a changing climate and dynamic coastal environments. The phenomenon of wave overtopping, which poses a significant concern in coastal regions, is experiencing an upward trend attributable to the effects of climate change-induced sea-level rise and the heightened intensity of storm occurrences (Chen et al., 2021). This phenomenon presents significant risks, such as the occurrence of floods, erosion, and infrastructure damage, which have adverse effects on the safety of coastal communities, the stability of ecosystems, and the economic prosperity of these areas.

Overtopping can erode coastal land, undermining the stability of beaches, cliffs, and dunes. This leads to the loss of valuable coastal real estate and infrastructure. Besides that, coastal overtopping can result in the inundation of low-lying coastal areas, causing flooding in coastal communities. Over time, overtopping damages or weakens coastal defenses, such as seawalls, revetments, and dikes, which are designed to protect against erosion and flooding (Siegal, 2020). If located near estuaries or riverbanks, overtopping introduces saltwater into freshwater ecosystems, affecting local flora and fauna (Li and Plater, 2020). It also transports pollutants and debris into coastal waters, harming marine life and ecosystems.

"Runup" in the context of coastal overtopping refers to the phenomenon where water from waves or storm surges rushes up the shoreline and inundates coastal areas beyond the normal waterline (CIRIA, 2007). It is a crucial concept in understanding and predicting the impacts of overtopping events along coastal regions. Wave runup is the vertical rise of water level on the beach or shoreline caused by the incoming waves. As waves approach the shore, they carry energy that pushes water up the beach slope. This elevated water level can extend significantly inland, depending on wave height, frequency, and the beach's slope.

In addition to ordinary wave runup, wave overtopping can involve the combined effects of high waves and storm surges. A storm surge is a rapid, abnormal rise in sea level due to atmospheric pressure changes and strong winds associated with storms, such as hurricanes or cyclones. When waves and storm surges coincide, they can result in more extensive and higher runup, leading to flooding and erosion of coastal areas. Among several factors that influence the magnitude of runup during coastal overtopping events includes wave characteristics which involve the size, period, and direction of incoming waves (CIRIA, 2007). Larger and longer-period waves tend to produce higher runup. Besides that, the slope and shape of the beach or shoreline affect the runup. A steeper beach slope generally results in more significant runup. The strength and duration of the storm, as well as the storm surge's height, are also critical factors in runup. More intense storms with higher surges typically lead to more substantial runup.

Runup has various consequences, including flooding, erosion, and infrastructure damage. In coastal communities, excessive runup results in the inundation of low-lying areas, leading to property damage, displacement of residents, and safety hazards. Runup contributes to erosion of the beach and shoreline, potentially leading to the loss of valuable coastal land and infrastructure. Coastal structures, such as seawalls, dikes, and buildings, can be damaged or weakened by the impact of runup during overtopping events (Sugano et al., 2014). Runup is a critical aspect of coastal overtopping, representing the vertical rise of water on the beach or shoreline due to wave action and storm surges. Understanding runup and its influencing factors is essential for coastal management, hazard mitigation, and the protection of coastal communities and infrastructure. Accurate prediction of runup is essential for coastal management and engineering. Coastal engineers and researchers use mathematical models and empirical data to estimate runup heights for different scenarios. Mitigation strategies may include the construction of protective structures (e.g., seawalls, revetments, and breakwaters) or beach nourishment to reduce the impacts of runup.

One method to mitigate the problem of overtopping and runup is to increase the crest level or the construction of crown walls, which contributes to the resilience of rock revetments. Crown walls in rock revetments are essential components in coastal management and coastal engineering innovation (Contestabile et al., 2020). They enhance protection, contribute to sustainability, provide valuable data for decision-making, and play a pivotal role in adapting to the evolving challenges posed by climate change and shifting coastal conditions. Their continued development and integration into coastal protection strategies are critical for the resilience and sustainability of coastal areas worldwide. Crown walls in rock revetments play a crucial role in coastal management and coastal engineering innovation, particularly in the context of overtopping studies.

Crown walls, while primarily associated with rock revetments and coastal defense structures, also serve as an alternative or complementary measure for beach protection. Crown walls can be constructed at the landward edge of a beach to dissipate the energy of incoming waves (Di Lauro et al., 2020). By partially blocking and deflecting wave energy, they reduce the erosive forces that can erode the beach

and shoreline. This helps maintain the width and stability of the beach. Besides that, crown walls mitigate wave runup by providing a raised barrier. This prevents excessive inundation of the beach, which is particularly important for low-lying coastal areas. Crown walls also act as a protective barrier against beach erosion, helping to retain sand and sediment on the beach, while preserving the natural features of the coastal environment.

In beachfront areas where buildings, roads, or other infrastructure are vulnerable to coastal erosion and over wash, crown walls can be used to shield these structures from the direct impact of waves and storm surges. Crown walls can be integrated into coastal dune systems which play a vital role in protecting the hinterland from coastal hazards to reinforce their stability, reducing the risk of dune erosion and over wash (Almarshed, 2020). It also preserves beaches meant for recreation and critical habitats for various species by stabilizing the beach and preventing habitat loss due to erosion. In the face of climate change and sea-level rise, beach protection measures need to be adaptive and resilient. Crown walls can be designed to accommodate changing conditions, providing a flexible solution that can evolve with the beach environment. It can be designed with flexibility in mind, allowing for adjustments to changing sea levels and wave patterns. Thus, this study proposes two crown wall designs to reduce wave runup and overtopping. A physical model work is conducted to assess two improved crown wall designs to reduce wave overtopping effect on coastal defense structure.

1.2 Objectives

The objectives of the physical experiment are:

- i. To investigate wave runup and overtopping on rock revetment
- ii. To evaluate the performance of crown wall in reducing wave runup and overtopping

1.3 Scope of Work

The scope of works that will be provided for the physical study to evaluate performance of coastal protection includes the following:

- i. Carry out literature review.
- ii. Prepare necessary equipment and materials for testing work in the laboratory including moulds and construction work on two crown wall breakwater models of different designs according to design and specifications.
- iii. Preparation and construction work on coastal profile of materials for rock revetment structure using granite stones with weights under category 100-200gm, 500-700gm and chipping stone in 2D flume.
- iv. Physical revetment rock structure model set-up according to pre-determined design with slope of 30°.
- v. Calibration of wave generation in flume experiment.
- vi. Conduct physical experiments for wave runup and overtopping on rock revetment with and without crown wall models.
- vii. Data collection for wave height and runup.
- viii. Assess the effectiveness of crown wall design from overtopping analysis.
- ix. Report writing, documentation and submission of report.

2.0 LITERATURE REVIEW

2.1 Introduction

Coastal regions around the world face the relentless and often destructive forces of the sea. Coastal erosion is the loss, displacement of land, or long-term removal of sediment and rocks along the coastline due to the action of waves, currents, tides, wind-driven water, and other impacts of storms (Kuenzer et al., 2014). Continual motion from surface waves, tides, storm surges and nearshore currents maintained the circulation of the coastal waters which brings the sediment particles on the move. Wave energy drives the sediment transport process in the coastal zone. The coastal processes that occur in Malaysia area are greatly influenced by the East Asian monsoon that conveys a great intensity of the related physical phenomena associated with waves, currents, winds, and rainfall that leads to the changes within the coastal zones. The shoreline of Malaysia is exposed to threats of coastal erosion. According to the National Coastal Erosion Study (NCES) in 2015, 1348 km from a total of 8840 km or 15% of the shoreline is currently facing erosion problems under three categorizations: critical, significant, and acceptable erosions (DID, 2015).

As climate change continues to drive rising sea levels and more intense storm events, the resilience of coastal structures becomes paramount. However, over time coastal structures are exposed to failure due to various factors including coastal overtopping. Coastal overtopping is a phenomenon that occurs when waves, tides, or storm surges exceed the height of coastal defences or structures, allowing water to flow over their tops. It is a critical aspect of coastal dynamics and has significant implications for coastal communities, infrastructure, and the natural environment (Kuenzer et al., 2014). Understanding coastal overtopping is essential for coastal management, engineering, and hazard mitigation efforts. Among the causes resulting in coastal overtopping includes both wave action and storm surges. Waves generated by wind, tides, or storms are the primary drivers of coastal overtopping. As these waves approach the shoreline, they carry energy and water that can surpass the elevation of coastal defences. Meanwhile, storm surges including tropical storms, hurricanes, or intense low-pressure systems can cause rapid and substantial

increases in sea level. When combined with high waves, these surges can lead to severe coastal overtopping events.

Coastal environments are dynamic and subject to constant change, requiring coastal engineering innovation to design structures that can adapt to evolving conditions. Wave overtopping leading to damage is a common issue encounter by coastal structure at the coast. This present study aims to determine the efficiency of two crown model design in reducing wave overtopping. The goal is to ensure the protection of human lives, assets, and coastal ecosystems through coastal structures considering the evolving dynamics of coastal areas.

2.2 Coastal Protection: Revetment Structures

Coastal protection structures including seawalls, groins, breakwaters, jetties, revetments serve to protect shoreline from erosion from the impact of wave action. There are many types of revetments including rock, gabion, stepped, curve-faced concrete and block revetments. Despite having their differences, they all serve the same main purpose which is for wave energy dissipation and shoreline protection (Appelquist et al., 2015).

Gabion revetments are economical and simpler to install, adapt to beach morphology and controls erosion while allowing water to flow through a permeable structure. But over time, the wire mesh may degrade with the possibility of greenery developing within the gabion construction, causing its appearance to be visually unpleasing (Nordstrom, 2014). Stepped revetment increased stability from fitting steps or blocks, allows accurate construction, robust and long-lasting as it provides long-term dissipation of wave energy. On the downside, it incurs high initial construction costs, less visually appealing and requires labor specialists for both construction and maintenance (Naylor et al., 2017). Curved-face revetment offers precise construction, effective in wave energy dissipation, robust, long-lasting and aesthetically pleasing but requires high cost of construction and difficult to maintain. Concrete revetment with blocks provides great stability, strong and reliable defense against erosion and can

meet site needs. The major disadvantage is the higher costs of regular maintenance work maintenance (Pilarczyk, 2017; Liu and Wang, 2020).

A concrete revetment with a smooth surface fit nicely with tourism but experiences a lot of wave overtopping (Saengsupavanich et al, 2017). Although a concrete revetment is aesthetically pleasing, a rock revetment is more resilient and can greatly reduce wave run-up. Rock revetments can be less flexible in design than counterparts made of concrete and big rocks are expensive to place and move. But it has outstanding control over erosion and dispersion of wave energy, more robust and durable while requires lesser maintenance (Reeve and Fleming, 2018). The implementation of revetments demonstrated a capacity to reduce wave overtopping rates when compared to smooth dikes, showcasing the practical advantages of these structures in coastal defence (Schoonees et al., 2021).

Crowned revetments, a specific category within the realm of coastal defences, further exemplify the versatility and adaptability of rock revetments of breakwaters. They act as bulwarks against wave action and serve as protective measures for coastal towns and low-lying areas, shielding them from the detrimental effects of flooding caused by rivers and waves (van Bergeijk et al., 2022). Rubble mound breakwaters are usually crowned by a concrete wall to gain higher crest freeboard by reducing the number of granular materials to be placed (Molines et al, 2016).

As the mentioned studies have contributed significantly to our understanding of revetments and their effectiveness in coastal protection, the global significance of revetments in coastal protection emphasizes the need for comprehensive and diverse research endeavours in this field. Due to the impacts of climate change, including rising sea levels and the anticipation of more frequent and severe storm occurrences, addressing the reduction of wave overtopping becomes imperative in the context of long-term coastal resilience planning. Thus, research studies have sought to understand and enhance the effectiveness of revetments in wave overtopping mitigation through numerical and physical modelling approaches.

2.3 Numerical Modelling Approach

Coastal models have been utilised to study the efficiency of designs of coastal protection structures. Fortes et al. (2020) provide conducted wave overtopping research conducted through the system, HIDRALERTA which was developed through a Python framework. The systems evaluate wave overtopping and flooding scenarios using sea-wave, tidal input and wind forecast data and implemented at ports to act as forecast and risk assessment at coastal and port region (Fortes et al, 2020). In contemporary times, design guides have incorporated specifications for maximum individual overtopping volumes, necessitating the examination of wave overtopping in a wave-by-wave manner. Koosheh et al. (2021) examined techniques for estimating the likelihood of overtopping and statistical methodologies employed to analyse the distribution of individual overtopping volumes. The precise computation of wave overtopping holds significant importance in the determination of the necessary crest height and geometry of a dike. Chen et al. (2021) utilised a 2D numerical model OpenFOAM to forecast wave overtopping at dikes characterised by intricate configurations comprising berms and roughness features.

Vrijling et al. (2020) evaluated the implications that arise from including uncertainty into the design of flood protection structures using the FORM method. The system utilises offshore sea-wave and wind forecast data, in addition to tide data, as input parameters for the purpose of assessing wave overtopping occurrences at designated areas. The study discovered that the probability of overtopping of revetment with a given height was contributed mostly by uncertainty in wind speed and wave height in accordance with FORM calculation. Besides that, Zhang et al. (2019) utilized computational fluid dynamics (CFD) approach to study greenwater overtopping of a 2D fixed rectangular object.

Numerical studies to study the efficiency of crown structure has been conducted by Ghasemi et al (2023) on caisson breakwater using Flow3D. The crown designs as shown in Figure 2.1 were tested to evaluate each design performance for simulation time of 160s. Design C demonstrated the least overtopping, followed by designs E, A, B, and D. Design D demonstrated the highest overtopping volume. Utilizing design D

as the reference volume, design C reduces overtopping volume by 59.2%, followed by design E (15.58%). The higher efficiency in structure C is due curved portions of the unique shape and curvature of upper portion that manage to deflect the wave that approaches the structure resulting in less overtopping compared to other shapes (Ghsasemi et al. 2023).

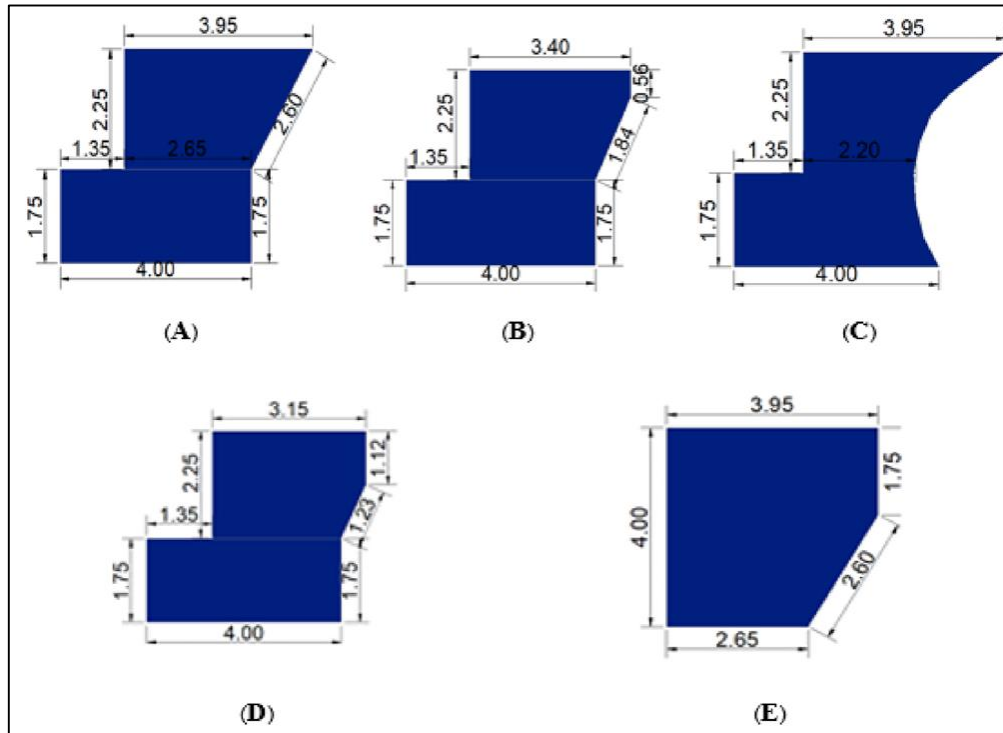


Figure 2.1 Various geometries of crown wall (Ghsasemi et al. 2023)

One main advantage of using numerical methods to study wave effect on structures includes faster and lower cost. Mathematical relationships built in the model system enables representation of processes at large scale area and visualisation of application in real case scenario (Geng et al., 2021). Numerical models also allow for more comprehensive information from simultaneous computation of different problem-relevant quantities such as current, water level, wind and weather forecast and its impact on wave generation upon the structure (Lashley et al., 2022). Despite its advantages, computer models may overlook immediate visual feedback of physical processes. Therefore, physical modelling approach provides an alternative for

examining processes and phenomena at reduces scale which is beyond user's analytical skills.

2.4 Physical Modelling Works

Physical modelling offers an alternative approach to determine effectiveness of improved designs of coastal structure. For instance, a study conducted by Lee et al., 2022 aimed to distinguish the effectiveness of preventing overtopping by wave-absorbing revetments and vertical revetments. It was observed that overtopping was exclusively prevalent in the wave-absorbing revetment, while the vertical revetment did not experience any overtopping despite having the lowest wave height. As overtopping is affected by several factors including significant hydraulic parameters such as wave height, the wave period, the wave direction (obliquity), and the water depth at the structure toe, physical model tests are particularly useful to assess wave overtopping, allowing physical processes to be demonstrated. Physical case studies conducted by different researchers include testing of wave overtopping of dike slope 1:6 in a river basin with wind generator, development and use of Wave Overtopping Simulator set up on dikes or levees at Netherlands, Germany, and tests using Wave Impact Generator (Eurotop, 2007).

Zhang et al. (2021) shed light on the remarkable ability of a revetment breakwater, constructed upon a foundation of compacted calcareous sand, to withstand the forceful impact of strengthened ocean waves. However, despite its overall stability, overtopping was still observed, underscoring the complex interplay between wave characteristics and revetment design. Schoonees et al., (2021) tested two full scale step revetments with step height 0.5m and 0.17m. His study discovered that wave overtopping is influenced by relative overtopping rate, characteristic step height, and wavelength. Relative overtopping rate influences the roughness factor with large overtopping rates leaving a layer of water on the revetments and reduce the friction of the step revetment making it prone to overtopping.

Researchers also explored the cost and performance efficiency of revetments through experimental methods. Coelho et al. (2016) found that one of the most effective strategies to produce a wall revetment involves increasing the crest height and constructing an intermediate berm. Such insights are invaluable for coastal engineering projects seeking to optimize resources while enhancing protection. In addition to these field studies, experiments in the Netherlands conducted by van Bergeijk et al., (2022). aimed to quantitatively assess the impact of changes in cover type and height on overtopping load. This research delved into identifying the significant hydraulic processes occurring at these transitions, emphasizing the importance of elevation in determining overtopping.

The design for crown unit plays a factor in the stability and overtopping of waves. Molines et al (2011) conducted an experiment on crown wall stability and overtopping test using an L shape crown on top of two distinct cubipod and cube armour layer. The difference in armour layer is to investigate the stability of rubble mound due to extreme impact to maintain the position of crown and also the impact of surface roughness of armour on wave dissipation. This study showed that the roughness factor is the main influence of reducing wave overtopping where the surface of cubipod is rougher than cube thus, more effective in breaking waves and reducing overtopping.

Han et al (2022) also studied a about L-shaped crown wall but the study only focusses on the wave forces acting on the crown wall instead of overtopping. The data of the experiment were compared to other previous methods such as Martin method (1999), CCD method, Pederson method (1996), and also Nørgaard method (2013). The parameters compared were uplifting wave force and horizontal wave force. The outcome of this study is an improved version of Martin, Pederson, and Nørgaard method where the improved martin method is the most accurate in predicting horizontal force while improved Nørgaard method in predicting uplifting force amongst the seven methods.

2.4.1 Stability of Structure

Understanding structural stability is important to ensure construction of structure on site to be able to withstand continuous impact of wave energy. A study conducted by Mares-Nassarre et al. (2019) examines the impact of the bottom slope on the occurrence of overtopping discharge for overtopping layer thickness (OLT) and overtopping flow velocity (OFV) that exceed 2% of incoming waves, $h_{c,2\%}$ and $u_{c,2\%}$. The study shows that as the gradient of the bottom slope increases, $h_{c,2\%}$ while $u_{c,2\%}$ increases. Zhao et al. (2019) numerically investigated morphological changes that occur in a sandy bed following the overtopping of an impermeable seawall by a tsunami-like wave and calibrated findings with experimental data. The assessment of the damage energy of the tsunami-like wave is based on the analysis of the force exerted on the barrier and the contour of the scour hole.

In a study conducted by Imaizumi et al. (2019), the rates of overtopping for sloping sea dikes were assessed under various conditions, including varied significant wave heights and average periods using the SWASH model. The study examined the impact of varying gradients of the fore slope, as well as the elevation, freeboard, and width of the sea dike crest. The design of the crest elevation of mound breakwaters takes into account a permissible average wave overtopping discharge.

For stability of rock revetments, depending on the kind of wave attack in both black box model and analytical model, the final form of the design method can be shown as a critical relationship between the load and strength. For revetments, the basic form of this relation is (Klein Breteler, 1996, Pilarczyk, 1998):

$$\left(\frac{H_s}{\Delta D}\right)_{er} = \frac{F}{\xi_{op}^{2/3}} \quad (2.1)$$

Where F = revetment (stability) constant (-), H_s = local significant wave height (m), $\Delta = \rho_s/\rho_w - 1$ = relative density (-), D = top layer thickness (m), and ξ_{op} = breaker parameters (-),

2.4.2 Wave Transmission and Reflection

The phenomenon of wave run-up refers to the vertical distance that a wave reaches above the still water level when it approaches and interacts with a shoreline or a coastal structure. The phenomenon of lake oscillations refers to the periodic fluctuations in the water level of a lake. (Vrijling et al., 2020)

Wave transmission and reflection are important aspects of wave energy dissipation. A numerical result showed that as the submerged breakwater's slope gradient decreased, so did the reflection and transmission coefficients. (Hur et al. 2011). This shows that the more the energy is loss, the steeper the slope gradient of submerged breakwaters.

$$K_T^2 + K_R^2 + E_L = 1 \quad (2.2)$$

Where K_T = Coefficient of Transmission, K_R = Coefficient of Reflection, and E_L = Energy Loss

2.4.3 Damage Criteria based on Overtopping Discharge

Low overtopping discharge reduces damage of embankments and seawalls revetment. The critical overtopping discharges and suggested critical values of prototype for various design situations is shown in Table 2.1 (Allsop et al., 2005; CIRIA, 2007). Figure 2.2 shows the categorization of damage based on coastal engineering manual (USACE, 2011).

Table 2.1: Critical overtopping discharge design situations (Allsop et al., 2005)

Design situations	Mean overtopping discharge, q (m ³ /s/m)	
Pedestrian	Unsafe for unaware pedestrians, no clear view of the sea, relatively easily upset or frightened, narrow walkway or proximity to edge	$q > 3 \times 10^{-5}$
	Unsafe for aware pedestrians, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	$q > 1 \times 10^{-4}$
	Unsafe for trained staff, well shod and protected, expected to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	$q > 1 \times 10^{-3} - 0.01$
Vehicles	Unsafe for driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	$q > 1 \times 10^{-5} - 5 \times 10^{-5}$
	Unsafe for driving at low speed, overtopping by pulsating flows at low levels only, no falling jets	$q > 0.01 - 0.05$
Embankment seawalls	No damage	$q < 2 \times 10^{-3}$
	Damage if crest not protected	$2 \times 10^{-3} < q < 0.02$
	Damage if back slope not protected	$0.02 < q < 0.05$
	Damage even if fully protected	$q > 0.05$
Revetment seawalls	No damage	$q < 0.05$
	Damage if promenade not paved	$0.05 < q < 0.2$
	Damage even if promenade paved	$q > 0.2$

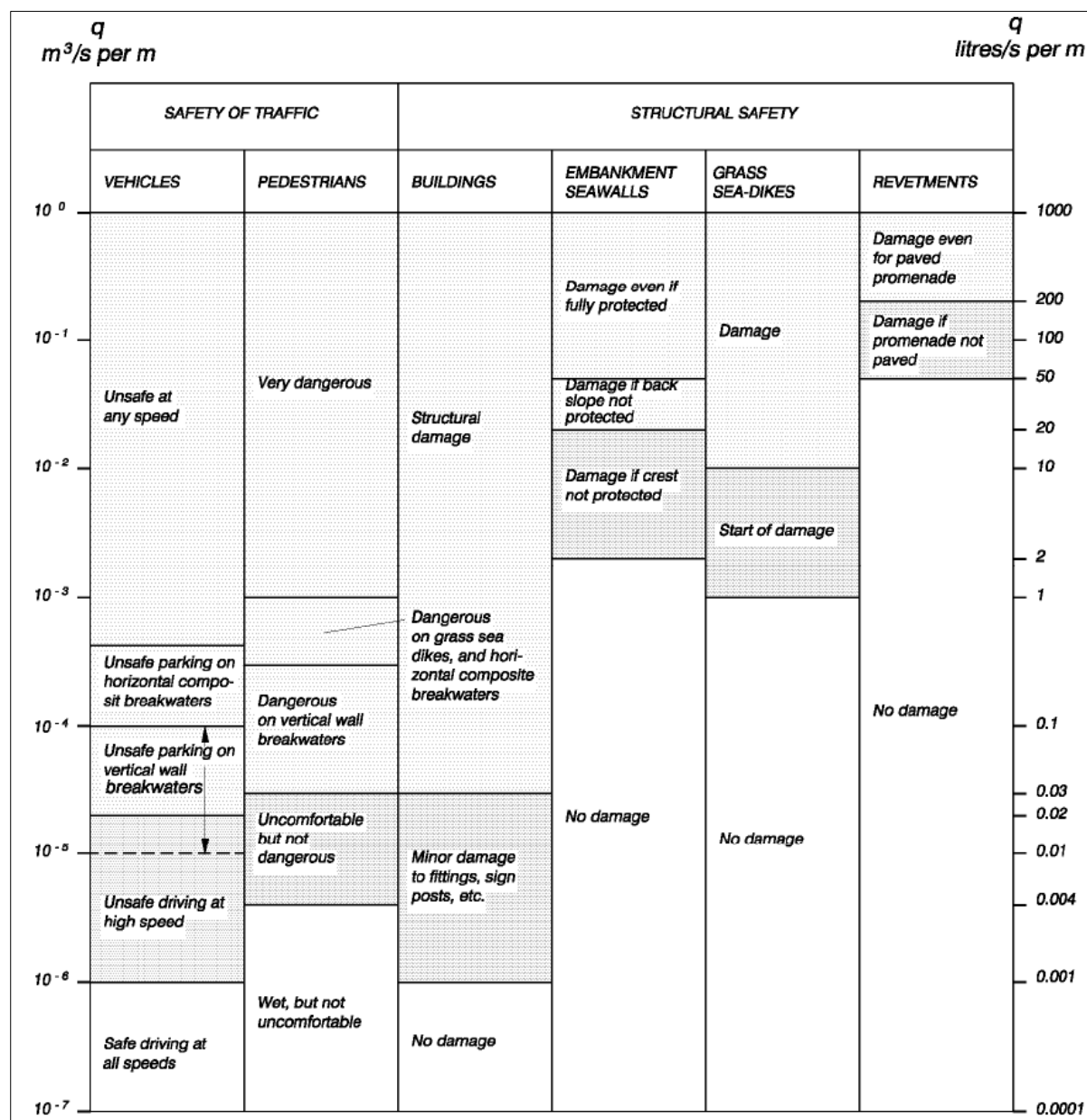


Figure 2.2: Critical overtopping discharge design situations (USACE, 2011)

3.0 METHODOLOGY

3.1 Data Collection

This chapter presents experimental data which have been carried out in hydraulics laboratory, NAHRIM. There are three main stages involving in the work methodology namely:

Stage 1: Test facilities and instruments system set-up

Stage 2: Rock revetments and crown units preparation

Stage 3: Evaluation of performance of crown design.

3.2 Stage 1 Test Facilities and Instruments System Set-Up

The 2D physical tests were carried out in the Hydraulic and Instrumental Laboratory of NAHRIM. The 2D tests are carried out in a wave flume, see Figure 3.1. The flume has a total length of 50m, a width of 1.5m and depth of 2m, with a maximum water depth of 0.88m. This flume is equipped with Multi Element Random Wavemakers with active wave absorption. It will generate either regular, irregular, bi-directional or user defined wave type based on standard wave spectra like Jonswap, Pierson Moskowitz, Derbyshire Coastal/Ocean, ISSC, ITTC, BTTP, Neumann and Bretschneider. The spectrum is based on the significant wave height (H_s) and peak wave period (T_p). The wave signals used in this project are generated from the Jonswap spectra with a peak enhancement factor of $\gamma = 3.3$ is applied. The maximum wave generating capability is 0.4m, with period ranges from 1 to 1.5 s at a maximum flow of 200l/s. The motion is initiated by a hydraulic actuator, controlled by a closed-loop servo-system. The command signal for this loop is obtained from computer software providing a random signal with a predetermined wave energy density spectrum. The system can also be used to examine aspects such as wave run-up and overtopping, the stability of rock and concrete armoured structures and wave impacts loadings, toe scour, etc.

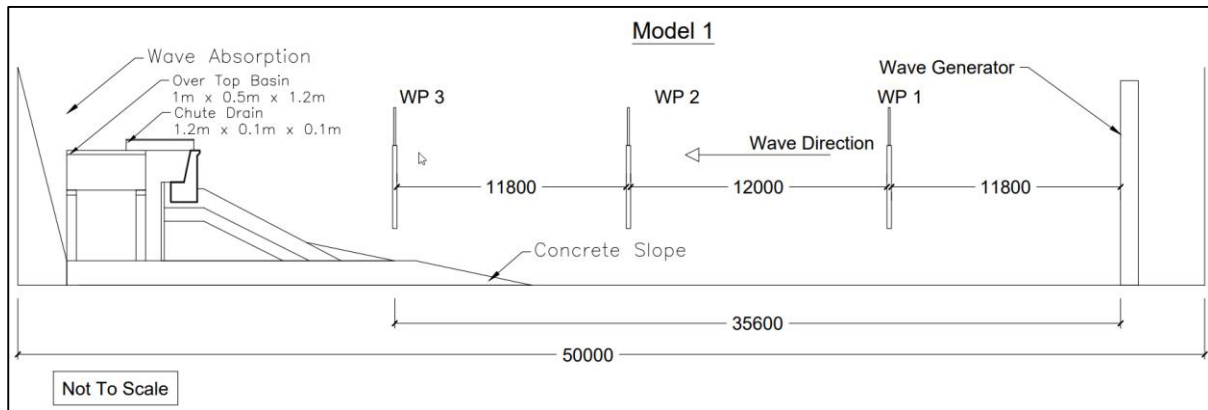


Figure 3.1: 2D AutoCAD drawing wave flume channel (side view)

3.2.1 Wave Probe

There are three types of wave probes which are change in resistance, change in capacitance, and change in conductivity. The wave probe used in this model is based on the change in resistance, see Figure 3.2. The wave probe operates by measuring the resistance of the water between a pair of parallel rods. The resistance between the rods is proportional to the immersion depth. Electrolysis is prevented by using an AC drive from a low impedance current amplifier. Resistance wave probes provide several advantages over other wave probe technologies:

- Excellent following of water surface.
- High data rates.
- Work well in large arrays.
- Probes are simple and low-cost.
- Integrated corrections in wavemaker software.

The Wave Probe controller electronics are completely software-controlled: the hardware requires no physical adjustment during setup and calibration. The software guides the user through the calibration process, and the process itself is completely configurable. Linearity of the probes can be measured and corrected for in the software.



Figure 3.2: Wave probe

3.2.2 Wave Generator System

Wave generator system used HR Merlin (previously known as HR Wavemaker) wave generation package written and developed by HR Wallingford Ltd. Its purpose is to control single and multi-element wavemakers to simulate a range of sea states in the wave basin, wave flumes and towing tanks. The program generates regular (sinusoidal) waves and random waves (irregular waves) for several commonly used wave spectra using digitally filtered white noise. The facility generates user defined spectral shapes is also incorporated. Regular and irregular waves propagations are generated for in this project.

3.2.3 Data Acquisition System

The Data Acquisition and Analysis Software Program, HR DAQ (Figure 3.3, excluded the software) is capable of logging 64 channels at a frequency of 3000 Hz per channel. The term 'wave counting' is not limited to water level records. The output of any analogue instrument such as a displacement sensor, force transducer or accelerometer, can be loosely called a 'wave' record and analyzed as such. The analogue signal is a wavy signal line in which crests and troughs appear above and below signal mean level. A typical analogue signal is shown in Figure 3.4 with principal parameters marked on it. HR DAQ uses a wave counting technique based on up crossings of the mean level of the signal. An upward crossing is detected by the previous value being below the crossing level and the present value above or equal to it. The up-crossing technique defines a wave as occurring between two such successive crossings following international standards. The parameters calculated by HR DAQ are listed below:

Mean	mean value of all the data
Max value	maximum value above the instrument zero
Min values	maximum value below the instrument zero
Std	standard deviation of the values from the mean
N_{waves}	number of zero crossing waves in the record
$T_{average}$	mean wave period of waves
H_{max}	highest wave height of the signal
H_{10}	mean of the highest tenth of the wave heights
$H_{1/3}$	mean of the highest third of the wave heights
H_{m0}	spectral significant wave heights
$H_{average}$	mean wave height
Alpha	(maximum-minimum)/Std
H_s	spectral wave height parameter = H_{m0} the significant wave height = $4\sqrt{(m_0)}$
T_p	peak wave frequency of the spectrum
m	zero, second and fourth moments of the spectra



Figure 3.3: HR DAQ

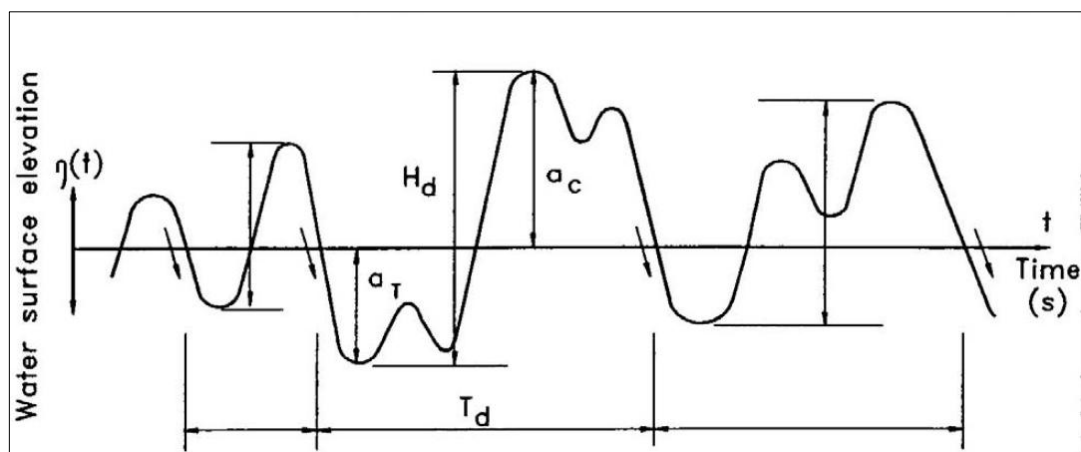


Figure 3.4: Example of analogue signal output

3.2.4 Instruments Calibration (Wave Calibration)

At the start of each test, the wave flume was filled up to the required water level which is 0.88m from the floor of the flume. Then, before taking any reading, wave probes have been fixed according to the designed position and calibrated to avoid error in measurements of wave heights. This calibration is important to create paddle movements that would make it possible to reproduce the required incidents wave parameters. Three probes will be placed at a fixed position throughout the entire experiment. The first probe is placed at 11.8m from the paddle, the second probe was located at 12m from probe one and the third probe was placed at the point of breaking wave nearest to the revetment which is at 11.8m from probe two. The calibration of the wave probes was done by altering either the wave heights or the wave period in the HR Merlin software to achieve a specific wave height at the probe nearest to the revetment which is probe three. The flume was calibrated by ensuring that the value of $H_{1/3}$ at probe three is closest to the desired wave height value.

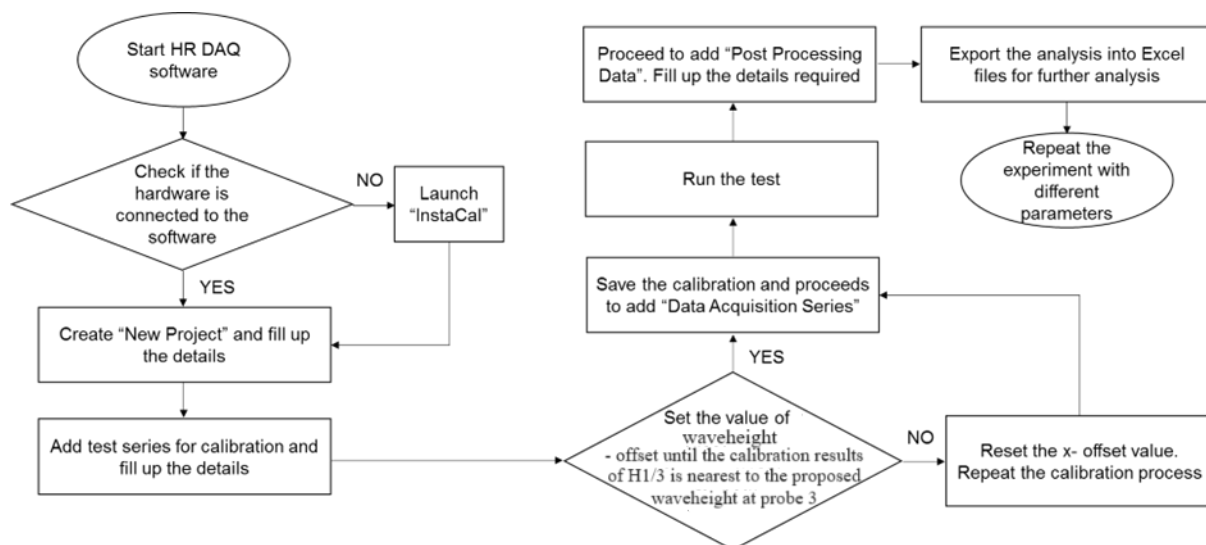


Figure 3.5: Calibration procedure of wave probe 3

The objective of the calibration is to produce the desired wave condition at the location of the breakwater. Waves were calibrated in the flume for each test without crown using wave Probe 3 to resolve the wave height. Table 3.1 gives the position of each probe relative to the breakwater structure and wave paddle. Figure 3.6 shows the schematic diagram of each wave probe location in the flume channel.

Table 3.1: Location of the wave probes in the flume

Wave Probe No	Distance from toe of Breakwater (m)	Distance from Paddle (m)
1	23.8	11.8
2	11.8	23.8
3	0	35.6

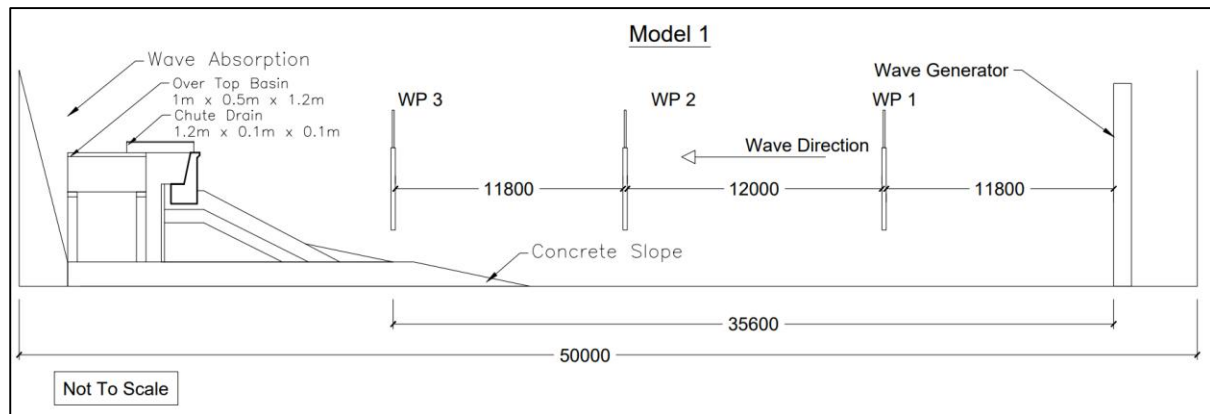


Figure 3.6: Schematic diagram of the 2D model setup

The wave paddle was programmed to generate regular and irregular waves using wave generator system (HR Wavemaker or HR Merlin). The data recorded by wave probes were analysed using the procedures outlined above. The amplitude of the signal driving the wave paddle was adjusted and fixed. The wave height and period were adjusted until the incident wave height at Probe 3 was of the target value.

3.3 Stage 2: Rock Revetments and Crown Units Preparation

The main components to set up the physical model includes the physical model scale rationale where actual parameters in field observations are scaled down to laboratory measurements. This section also entails the design and casting of the proposed crown structures which will be set up above the rock revetment structure in the wave flume.

3.3.1 Geometric Scale

This scale ratio is the most practical based on available area and facilities required for the study while achieving objectives of the study. Although a larger scale model could potentially remove the requirement of a sophisticated control system and reduce the scaling effect, it will also result in a substantial reduction in the coverage area of the physical model. The scale selection should assure that the dominant forces are well represented. The geometric scale, λ represents the linear scale model to which the physical model is constructed. The scaling relationships between measurements in the model and nature can be derived from Froude's law as shown in Table 3.2.

Table 3.2 Scale ratio relationship between model and field data

Quantity	Unit	Scale Ratio
Length	m	$1 : \lambda$
Time	s	$1 : \lambda^{1/2}$
Mass	kg	$1 : \lambda^3$
Wave height	m	$1 : \lambda$

Laboratory wave system was scaled down by 1:10 from the real field data. In fluid motion, the most important forces are gravity and momentum where Froude scaling is needed to be used in physical models. The conversion of mass (kg/m^3) was scaled down using the scale factor of λ^3 . Table 3.3 presents the conversion factor used to adjust the model scale data to the prototype scale. For instance, a 2 m wave height in real case scenario is reduced to 0.2 m in laboratory scale. Meanwhile, Table 3.4 represents the scaled down parameter used in this experiment. Detailed categorisation of twelve test cases is further described in Table 3.5.

Table 3.3 Scaling conversion

Element	Scale Factor	Model	Prototype
Wave Height	1:10	0.1 m, 0.15 m, 0.2 m	1 m, 1.5 m, 2 m
Mass of rock samples	1:10 ³	Approx. 0.0125 kg - 0.7 kg	0.0125 tonnes – 0.7 tonnes
Wave period	1:10 ^{1/2}	1s, 1.5s	3.16s, 4.74s

Table 3.4: Set-up parameter for rock revetment

Parameter	Material
	Rock revetment
Downscale Ratio	1:10
Type of wave	Regular and irregular wave
Stone Size (kg)	0.0125 - 0.7
Water Depth, d (m)	0.88
Slope	1:30
Wave periods (s)	1.5, 1
Wave Heights, H (m)	0.10, 0.15, 0.2

Table 3.5: Conditions for test cases

Type of wave	Duration, (s)	Wave period, T_p (s)	Frequency (Hz)	Wave height, (m)	Test Case
Regular wave	300	1.5	0.67	0.20	1
				0.15	2
				0.10	3
		1	1	0.20	4
				0.15	5
				0.10	6
Irregular wave	900	1.5	0.67	0.20	7
				0.15	8
				0.10	9
		1	1	0.20	10
				0.15	11
				0.10	12

3.3.2 Design and Construction of Crown Units

This propose designs of the two crown units to be tested is shown in Figure 3.7. The designs are based on scale 1:10 for laboratory construction purposes. A steel mould of thickness 3mm with sizes as shown in Figure 3.8 is fabricated and concrete is poured into the mould to obtain the desired dimension of crown units. The assembling and construction work to set up the crown units is seen in Figure 3.8 to Figure 3.11.

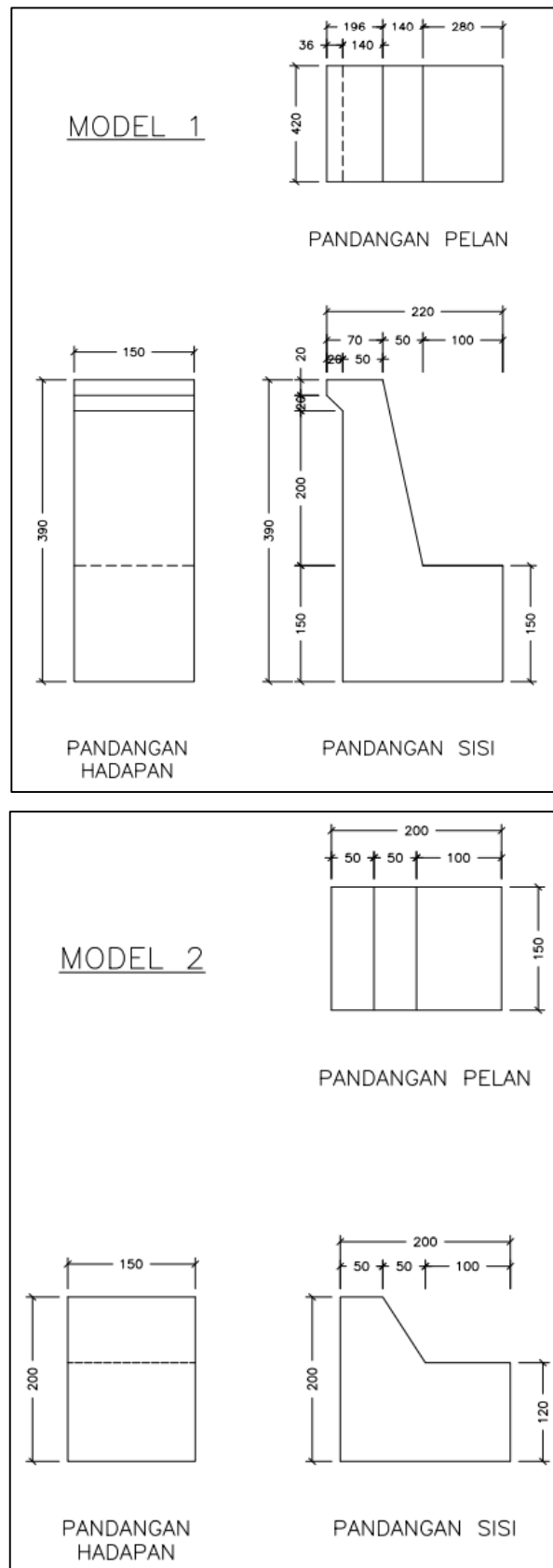


Figure 3.7: AutoCAD drawing of crown structure design dimensions, Model 1 (top) and Model 2 (bottom)



Figure 3.8: Assembly work for mould component (crown structure Model 1)



Figure 3.9: Assembly work for mould component (crown structure Model 2)



Figure 3.10: Casting of crown units structure in pre-set mould



Figure 3.11: Removing of steel plate mould with thickness 3mm

3.3.3 Rock/Stone Selection

The rock revetment sections are constructed to scale 1:10. The stones needed were broken to obtain the shape and weight required by specification from downscaling (Figure 3.12).



Figure 3.12: Breaking stones process according to pre-determined shape and weight for construction of rock revetment structure

The main components of a rock revetment, from the middle of the structure outwards, are the core, the under-armour layer, and the upper-armour layer. The mass of the model (granite stones), as shown in Table 3.2, are scaled accounting for the water and material densities in model and prototype.

Table 3.6: Aggregate classification (stone size)

Prototype	Model
0.5 - 0.7 tonnes	 upper armour (500 – 700 gram)
0.1 - 0.2 tonnes	 under armour (100 – 200 gram)
0.0125 tonne	 chipping stone (< 12.5 gram)

3.3.4 Construction of Rock Revetment

The construction of rock revetment is in 2D wave flume at NAHRIM laboratory is further describe in this section. The base profile is first set up as shown in Figure 3.13. To ensure that the rock revetment structure does not collapse, the toe of the revetment is layered with a plaster cement (Figure 3.14).



Figure 3.13: Rock revetment base layout



Figure 3.14: Cemented layer at toe of revetment base

The core profile layer consists of chipping stone with thickness layer based on design specifications (Figure 3.15). The under layer and filter layer have the function of preventing core material from being displaced through the armour layer. At the same time, they should be open enough for water to penetrate the core because, in this way, energy is dissipated. The under layer comprises of stones weighing between 100g to 200g as shown in Figure 3.16.

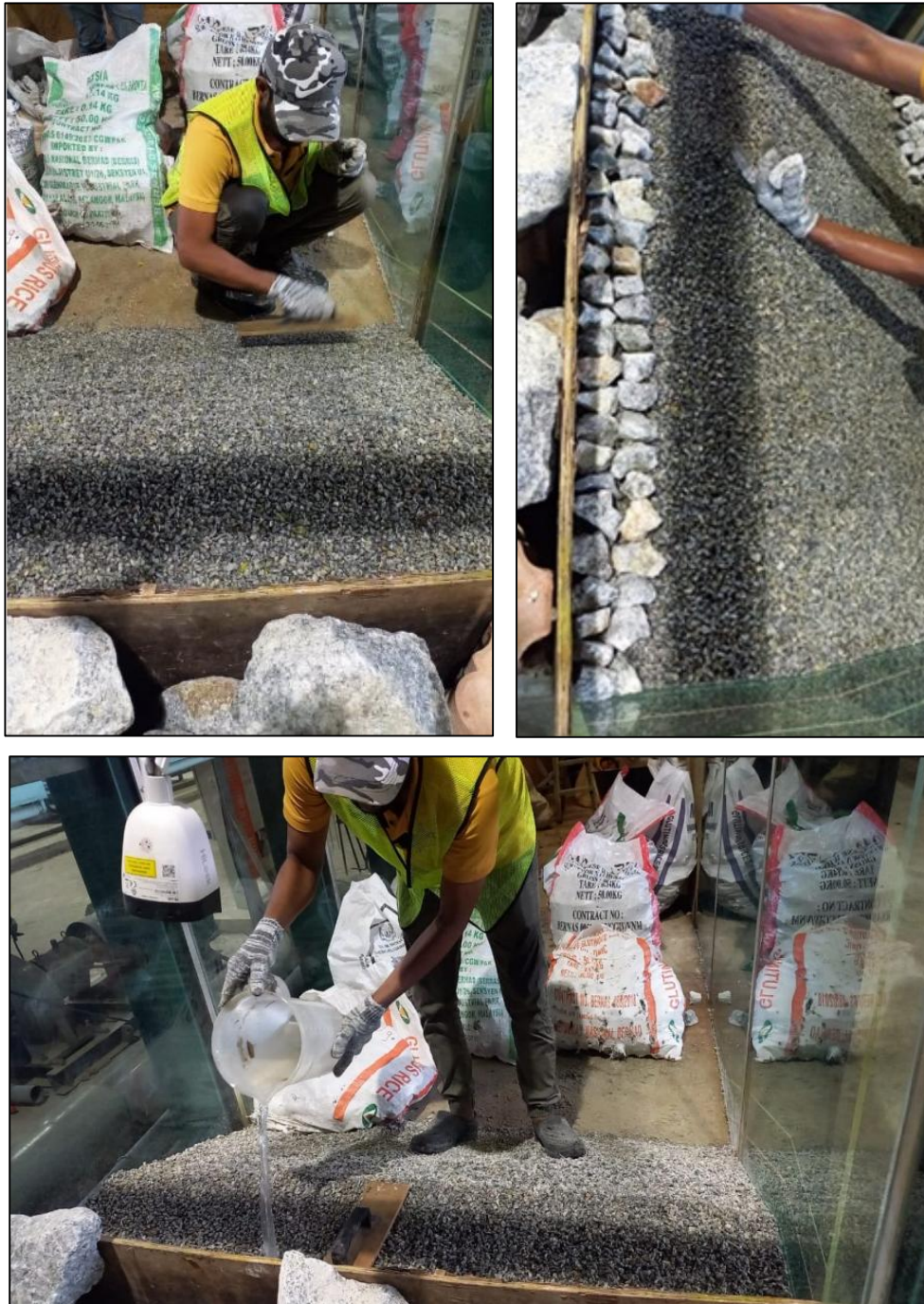


Figure 3.15: Construction of core profile layer

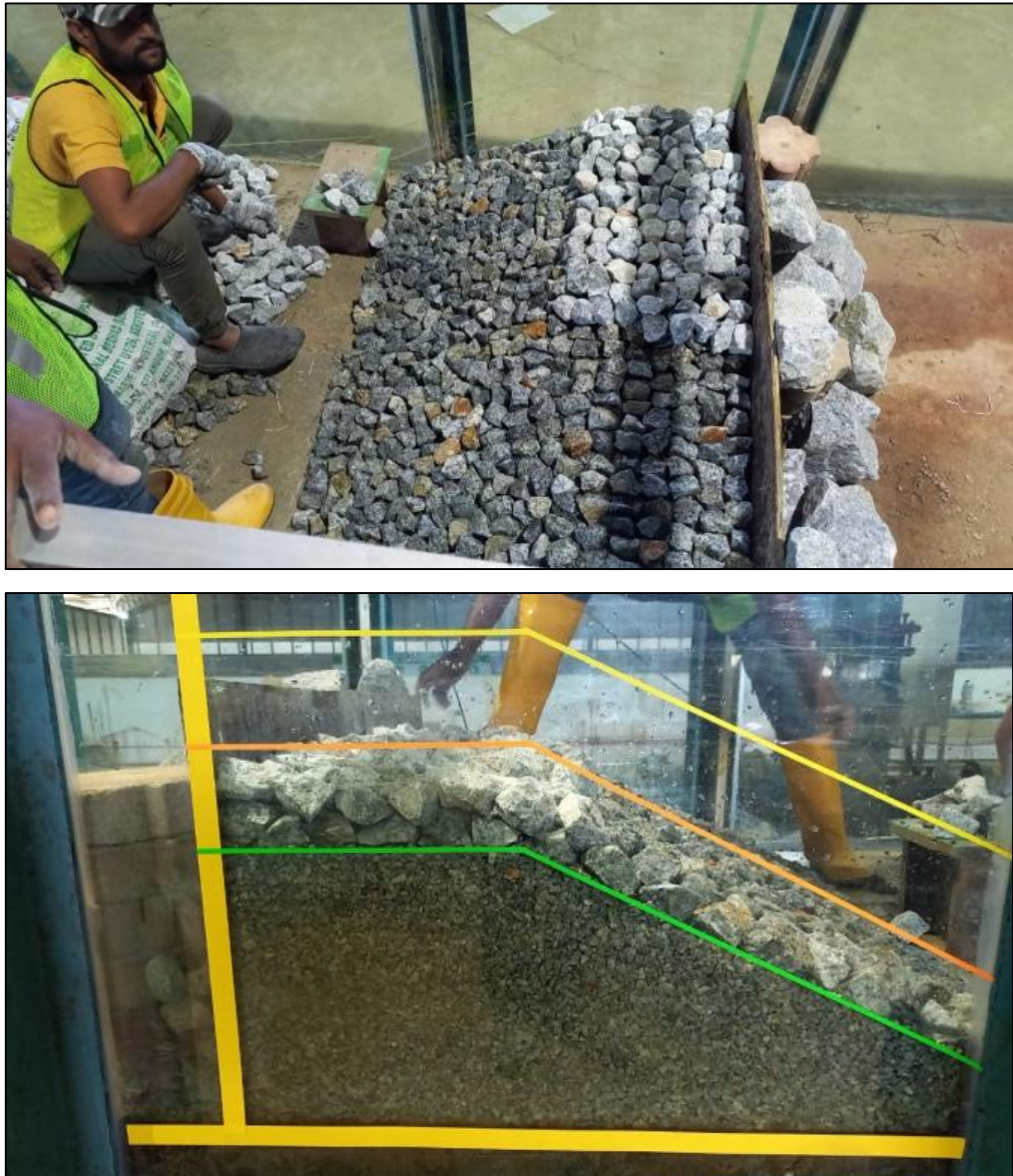


Figure 3.16: Construction of under armour layer

The upper layer, also known as the armor layer of the revetment is constructed using stones weighing between 500g to 700g (Figure 3.17).



Figure 3.17: Laying of upper armour layer

Figure 3.18 illustrates the final arrangement of rock revetment prototype setup located at 35.6m from the paddle in the wave channel flume

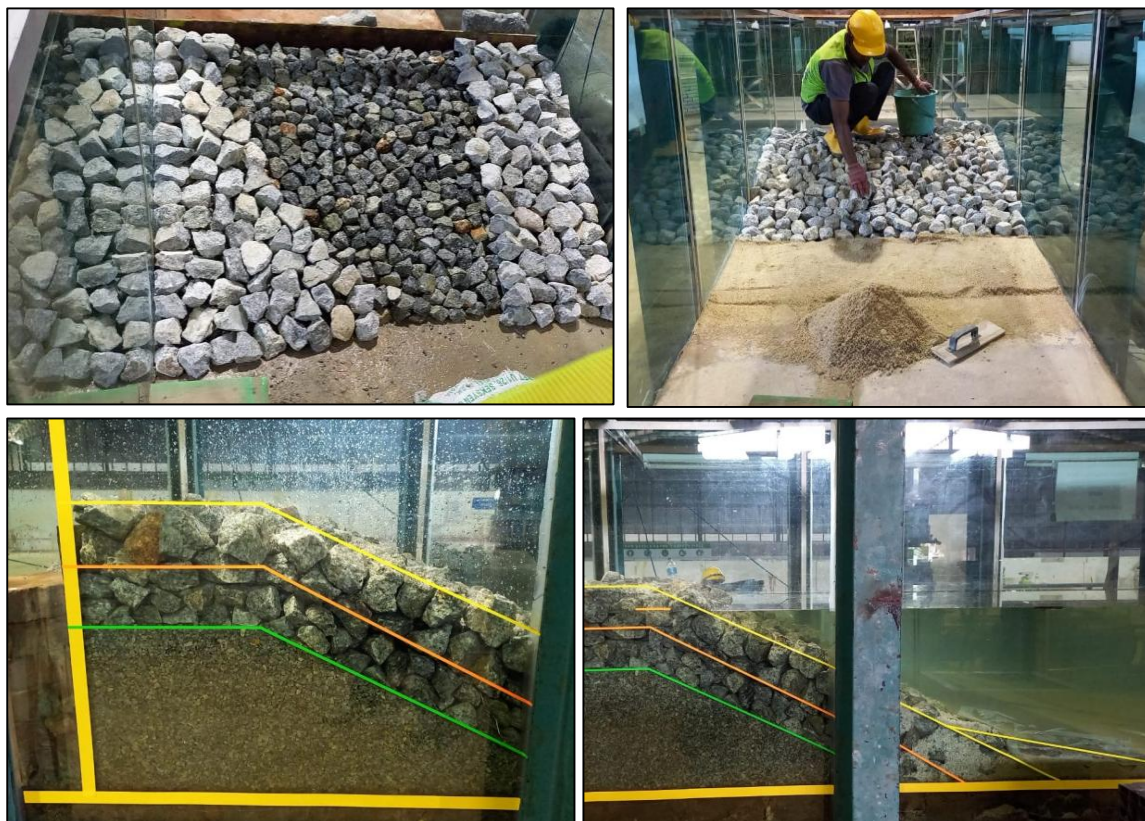


Figure 3.18: Rock revetment set-up

The crown units were placed on top of the armour layer and several test involving regular and irregular waves were propagated to test the breakwater and crown walls.

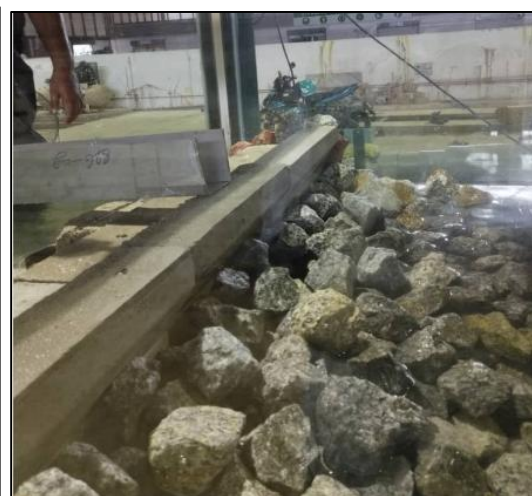
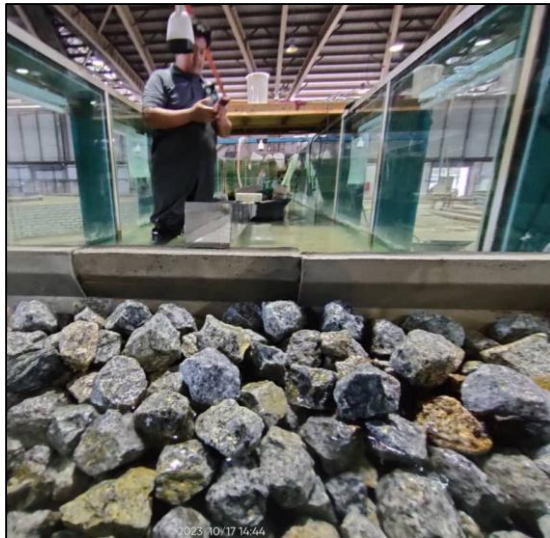




Figure 3.19: Crown wall set-up

3.4 Flow of Experimental Works (Wave Propagation)

A typical procedure was implemented during all the tests, which consisted of:

- Designing the revetment crown
- Setting wave flume and experiment apparatus
- Setting up the water level
- Adjusting and calibrating the wave probes
- Preparation of the steering files for the wavemaker, HR Merlin and for the measuring software, HR DAQ
- Taking the photographs before the test from the fixed camera point
- Setting up recorder for recording video for each wave case (Figure 3.21)
- Start the test program.
- The wavemaker starts after approximately 60 seconds.
- Observation of the breakwater performance, focusing on the movement and rocking of the armor units
- Measurement of the wave overtopping discharges, as mentioned.
- Stop the test program after 300 seconds (regular waves) and 900 seconds (irregular waves)
- Turning the video cameras off
- Taking the photographs after the test from the same fixed camera points
- Record data



Figure 3.20: Setting up video recorder for recording data

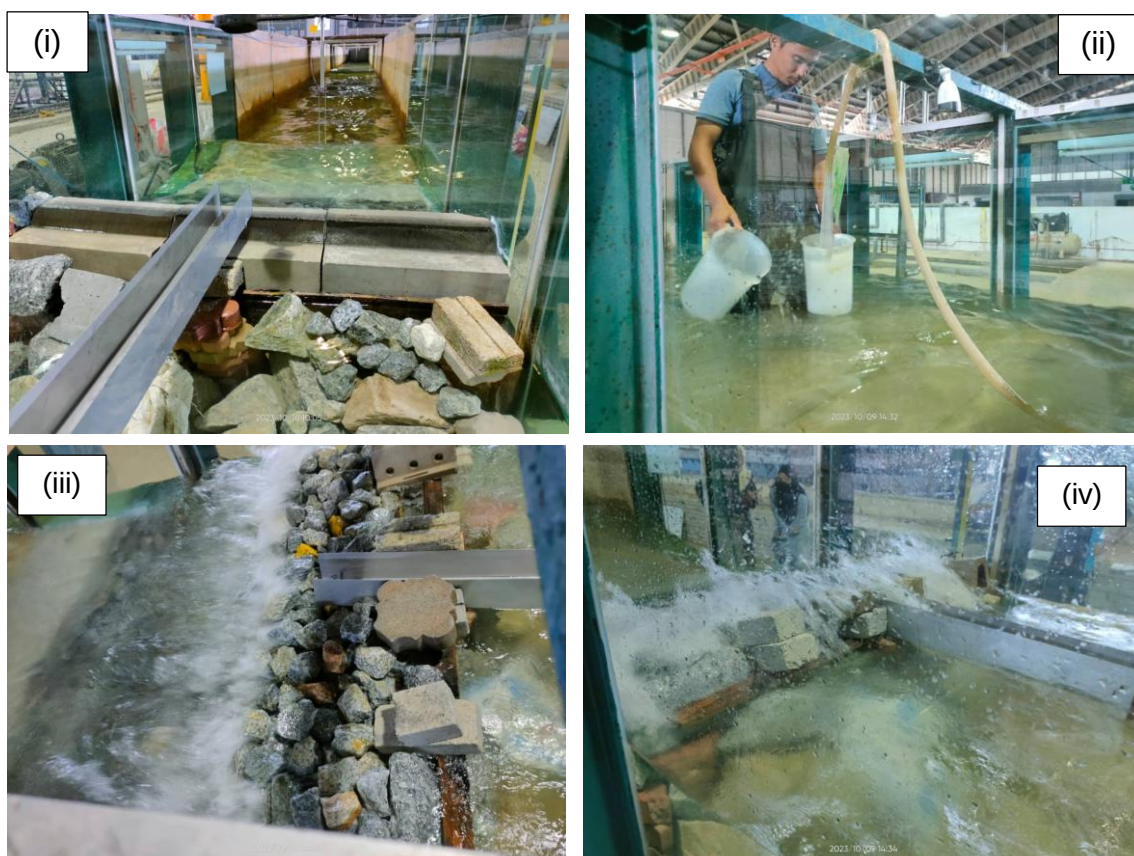


Figure 3.21: Example of experimental works snapshot in the flume. (i) Setting a chute at the crest level of type 1 crown wall, (ii) pumping and collecting wave overtopping, (iii), (iv) wave propagating to revetment

4.0 RESULTS AND ANALYSIS

4.1 Wave Probe Calibration

For wave calibration purposes, wave height produced at probe 3 which is located nearest to the rock revetment is utilised to ensure the wave generated is similar as required. Take for example Case 2, the proposed wave height is 0.2m, thus the wave height at probe 3 should be exactly 0.2m by adjusting the wave height or the wave period at the probe nearest to the paddle (probe 1) with very little tolerance. All test cases are calibrated during without crown conditions by ensuring $H_{1/3}$ reaches the desired wave height values. Table 4.1 to 4.12 summarises the $T_{1/3}$ and $H_{1/3}$ values obtained from the wave acquisition program for condition without crown units, model 1 and model 2 crown units at wave probe 1, 2 and 3.

Table 4.1: Test Case 1

Regular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.20$, $T_p = 1.5$	1	198	0.247573	1.480
		2	198	0.211759	1.498
		3	198	0.200002	1.502
Crown Model 1	$H_i = 0.20$, $T_p = 1.5$	1	199	0.220816	1.492
		2	199	0.195945	1.492
		3	199	0.193328	1.484
Crown Model 2	$H_i = 0.20$, $T_p = 1.5$	1	198	0.291273	1.484
		2	198	0.253079	1.498
		3	198	0.240049	1.486

Table 4.2: Test case 2

Regular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.15$, $T_p = 1.5$	1	200	0.192881	1.470
		2	198	0.156177	1.482
		3	198	0.149948	1.487
Crown Model 1	$H_i = 0.15$, $T_p = 1.5$	1	198	0.202891	1.478
		2	198	0.172421	1.497
		3	198	0.162541	1.477
Crown Model 2	$H_i = 0.15$, $T_p = 1.5$	1	198	0.223924	1.495
		2	198	0.18682	1.500
		3	198	0.171405	1.498

Table 4.3: Test Case 3

Regular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.10$, $T_p = 1.5$	1	198	0.140066	1.482
		2	198	0.112493	1.479
		3	198	0.104414	1.485
Crown Model 1	$H_i = 0.10$, $T_p = 1.5$	1	198	0.147195	1.491
		2	198	0.125084	1.488
		3	198	0.111431	1.493
Crown Model 2	$H_i = 0.10$, $T_p = 1.5$	1	198	0.158634	1.497
		2	198	0.134299	1.493
		3	198	0.11946	1.498

Table 4.4: Test Case 4

Regular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.20$, $T_p = 1$	1	298	0.222251	1.003
		2	298	0.227051	1.025
		3	298	0.182182	1.012
Crown Model 1	$H_i = 0.20$, $T_p = 1$	1	298	0.224424	0.999
		2	298	0.217173	1.012
		3	298	0.191702	1.038
Crown Model 2	$H_i = 0.20$, $T_p = 1$	1	298	0.239109	0.998
		2	298	0.235164	1.016
		3	297	0.198551	1.030

Table 4.5: Test Case 5

Regular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.15$, $T_p = 1$	1	298	0.162257	1.001
		2	298	0.156009	1.011
		3	298	0.136159	1.008
Crown Model 1	$H_i = 0.15$, $T_p = 1$	1	298	0.180832	1.002
		2	297	0.151521	1.005
		3	298	0.138228	0.999
Crown Model 2	$H_i = 0.15$, $T_p = 1$	1	298	0.196925	1.001
		2	297	0.163609	1.006
		3	297	0.150823	1.013

Table 4.6: Test Case 6

Regular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.10$, $T_p = 1$	1	298	0.126245	0.999
		2	298	0.117747	1.001
		3	298	0.100946	0.994
Crown Model 1	$H_i = 0.10$, $T_p = 1$	1	298	0.138241	1.001
		2	298	0.126598	1.001
		3	299	0.107917	0.980
Crown Model 2	$H_i = 0.10$, $T_p = 1$	1	298	0.145775	1.001
		2	298	0.139719	1.002
		3	297	0.118539	0.997

Table 4.7: Test Case 7

Irregular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.20$, $T_p = 1.5$	1	651	0.251176	1.283
		2	632	0.232787	1.333
		3	638	0.203502	1.288
Crown Model 1	$H_i = 0.20$, $T_p = 1.5$	1	656	0.256005	1.253
		2	637	0.246007	1.301
		3	657	0.216707	1.248
Crown Model 2	$H_i = 0.20$, $T_p = 1.5$	1	662	0.279427	1.237
		2	647	0.262313	1.258
		3	665	0.22615	1.205

Table 4.8: Test Case 8

Irregular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.15$, $T_p = 1.5$	1	716	0.165954	1.147
		2	702	0.157555	1.159
		3	717	0.136285	1.132
Crown Model 1	$H_i = 0.15$, $T_p = 1.5$	1	692	0.189495	1.169
		2	695	0.179525	1.174
		3	707	0.156246	1.113
Crown Model 2	$H_i = 0.15$, $T_p = 1.5$	1	704	0.201737	1.158
		2	695	0.190748	1.164
		3	695	0.167869	1.194

Table 4.9: Test Case 9

Irregular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.10$, $T_p = 1.5$	1	730	0.113859	1.107
		2	734	0.110013	1.113
		3	742	0.09308	1.105
Crown Model 1	$H_i = 0.10$, $T_p = 1.5$	1	715	0.129945	1.137
		2	726	0.12574	1.108
		3	733	0.106466	1.120
Crown Model 2	$H_i = 0.10$, $T_p = 1.5$	1	718	0.138034	1.138
		2	714	0.135875	1.144
		3	738	0.114399	1.114

Table 4.10: Test Case 10

Irregular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.20$, $T_p = 1$	1	859	0.194598	1.008
		2	808	0.173115	1.071
		3	808	0.142195	1.040
Crown Model 1	$H_i = 0.20$, $T_p = 1$	1	851	0.195945	1.025
		2	812	0.174096	1.071
		3	812	0.141604	1.030
Crown Model 2	$H_i = 0.20$, $T_p = 1$	1	864	0.196728	1.009
		2	820	0.174113	1.045
		3	810	0.143999	1.012

Table 4.11: Test Case 11

Irregular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.15$, $T_p = 1$	1	869	0.176942	0.995
		2	814	0.159085	1.061
		3	819	0.130075	1.006
Crown Model 1	$H_i = 0.15$, $T_p = 1$	1	859	0.187419	1.012
		2	801	0.169248	1.097
		3	812	0.13684	1.029
Crown Model 2	$H_i = 0.15$, $T_p = 1$	1	854	0.204805	1.032
		2	804	0.180945	1.091
		3	814	0.148829	1.025

Table 4.12: Test Case 12

Irregular wave					
Model	Intended H_i (m) and T_p (s)	Probe	No of waves	$H_{1/3}$ (m)	$T_{1/3}$ (s)
Without crown	$H_i = 0.10$, $T_p = 1$	1	915	0.129479	0.951
		2	897	0.117811	0.953
		3	899	0.098033	0.922
Crown Model 1	$H_i = 0.10$, $T_p = 1$	1	917	0.136288	0.937
		2	899	0.124816	0.942
		3	902	0.104186	0.926
Crown Model 2	$H_i = 0.10$, $T_p = 1$	1	914	0.147081	0.948
		2	911	0.132809	0.926
		3	899	0.110918	0.929

4.2 Comparison of Significant Wave Height ($H_{1/3}$) and Intended Wave Height (H_i)

Wave characteristics for significant wave height, $H_{1/3}$ at Probe 1, 2 and 3 is displayed in Figure 4.1 and 4.2 for regular wave conditions and Figure 4.3 and 4.4 for irregular wave conditions. In general, from the experiment conducted, intended wave propagation of $H_i=0.20$ m generated higher significant wave at Probe 1, 2 and 3 compare to $H_i=0.15$ m and $H_i=0.10$ m wave heights except during irregular wave condition of period $T_p=1.0$ s where the waves generated for $H_i=0.20$ m and $H_i=0.10$ m is almost the same (Figure 4.4). Since the wave probe is calibrated is Probe 3 to produce significant wave height as intended wave, in most cases only without crown conditions meets the requirement of intended wave height. However, there is a pattern where $H_{1/3}$ increases at Probe 3 when Model 1 and Model 2 is placed, which shows that wave propagated with the existence of crown models are higher than intended. The higher wave heights obtained can be due to higher wave heights at wave paddle during initial wave generation stage which results in an increasing wave energy flux towards the revetment structure.

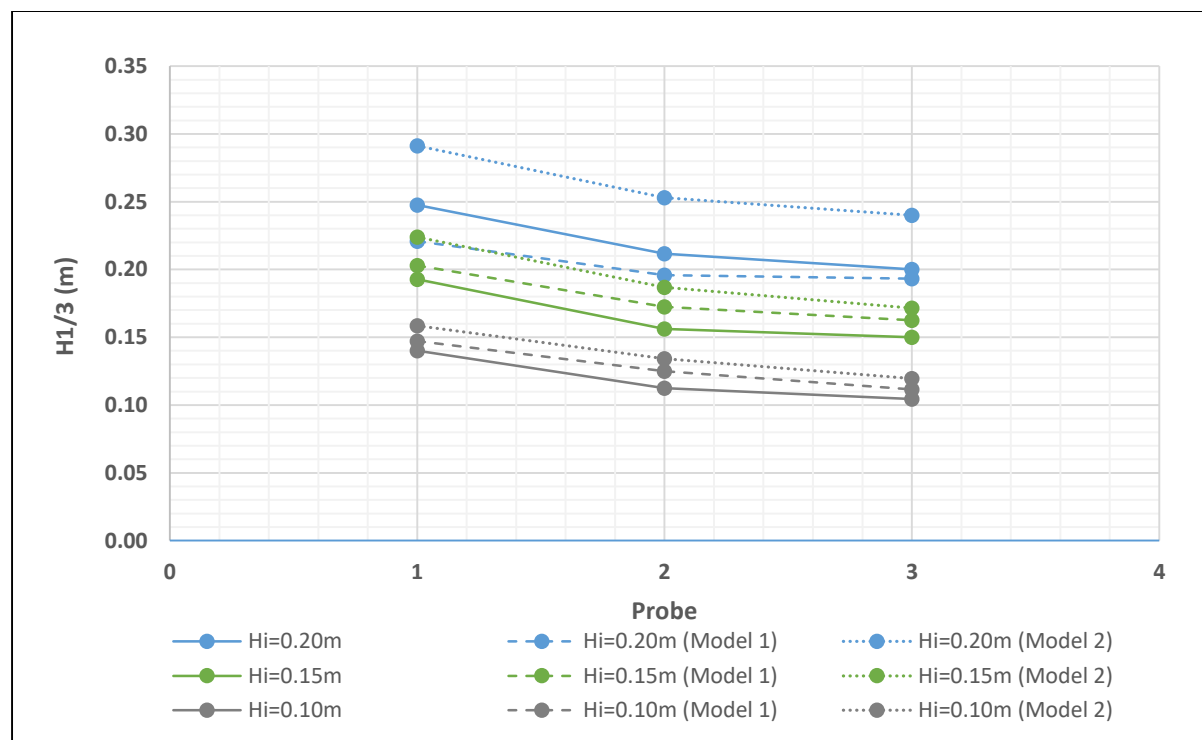


Figure 4.1: Significant wave height, $H_{1/3}$ at Probe 1, 2 and 3 at mean period $T_p=1.5\text{s}$ for regular wave

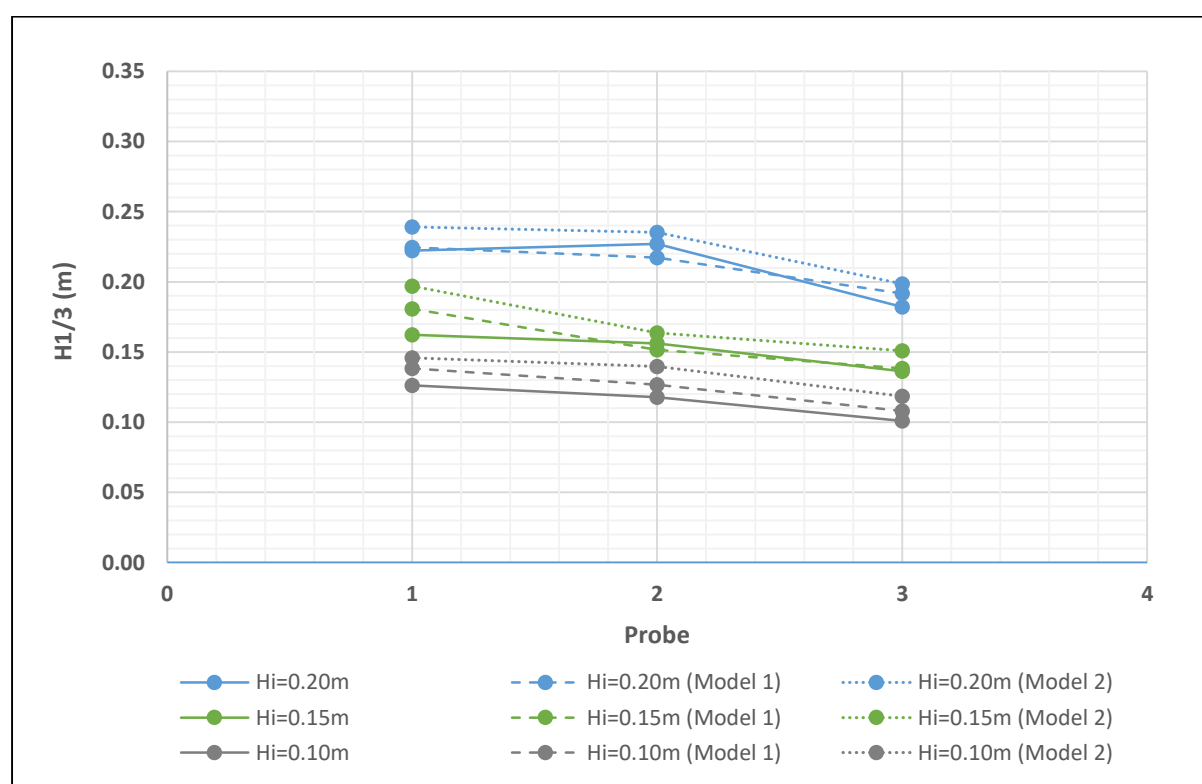


Figure 4.2: Significant wave height, $H_{1/3}$ at Probe 1, 2 and 3 at mean period $T_p=1.0\text{s}$ for regular wave

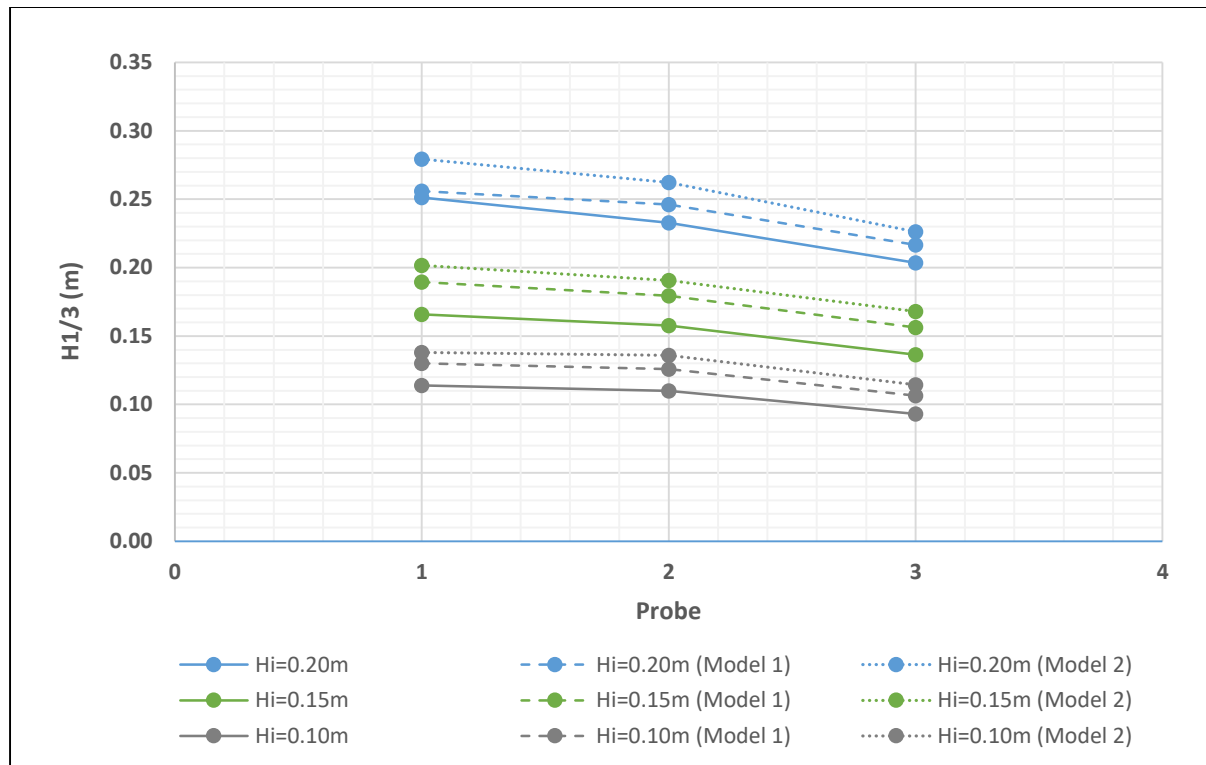


Figure 4.3: Significant wave height, $H_{1/3}$ at Probe 1, 2 and 3 at mean period $T_p=1.5\text{s}$ for irregular wave

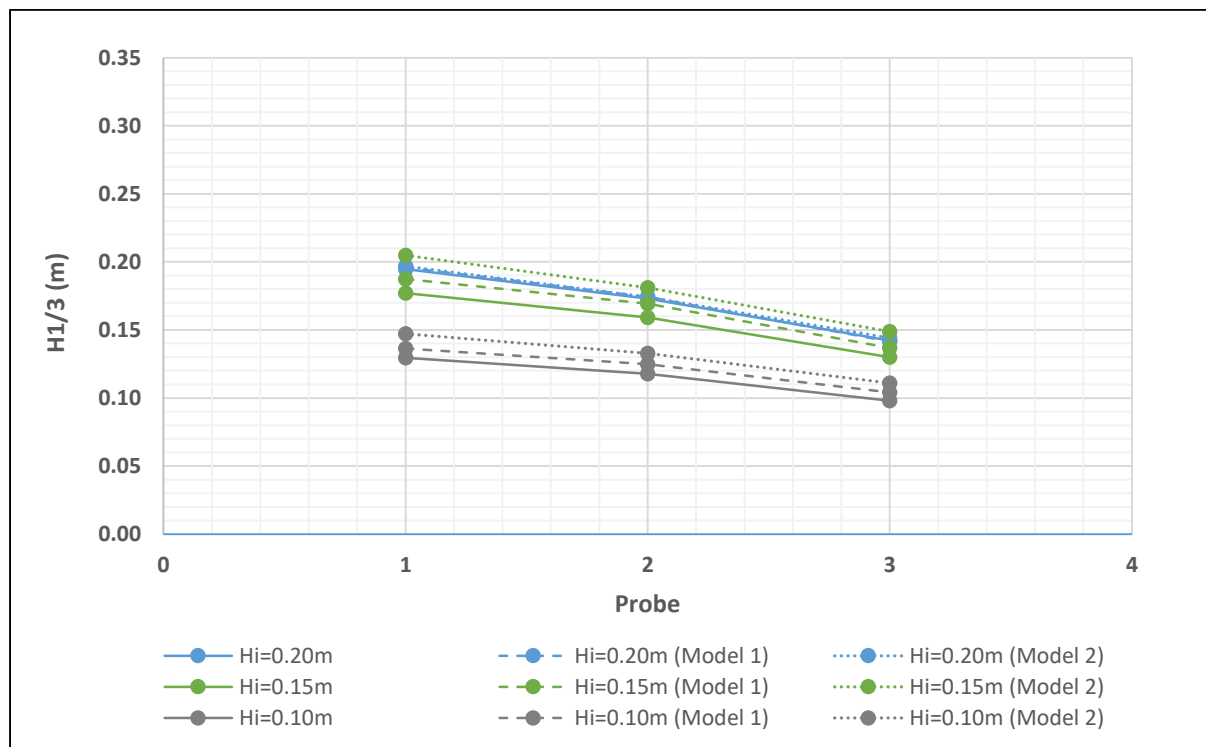


Figure 4.4: Significant wave height, $H_{1/3}$ at Probe 1, 2 and 3 at mean period $T_p=1.0\text{s}$ for irregular wave

4.3 Empirical Analysis on Wave Runup

Wave action on a sloping structure causes the water surface to oscillate over a vertical range that is generally greater than the incident wave height. The extreme levels reached for each wave are known as run-up, R_u defined vertically relative to the still water level, SWL (Figure 4.5). If extreme run-up levels exceed the crest level, the structure will be overtopped. Prediction of run-up may be based on simple empirical equations obtained from model test run results. All calculation requires parameters to be defined precisely. The most common irregular wave run-up parameter is $R_{u2\%}$ where 2% describes the exceedance level related to the number of incoming waves is considered.

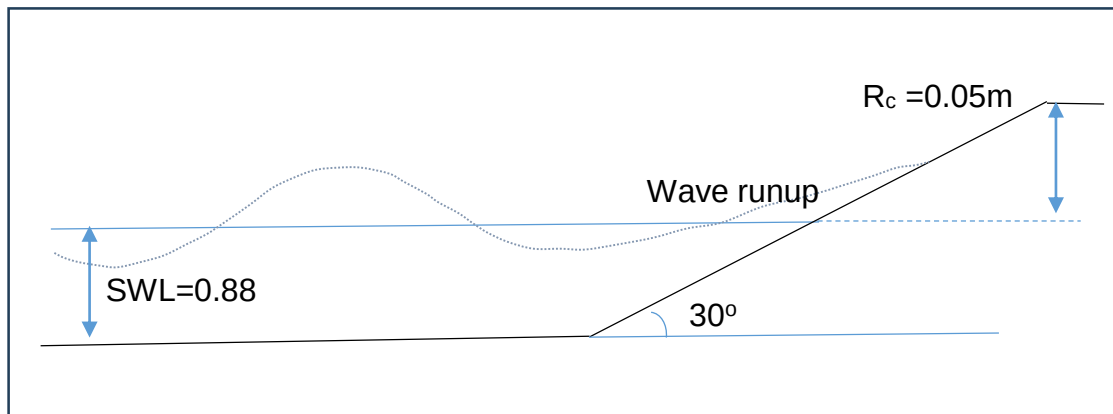


Figure 4.5: Wave runup

Wave runup (R_u) is calculated using the formulation in Equation 4.1 to 4.3 by Van Der Meer (1988).

$$R_u = \gamma_i c \xi_m^d H_s \quad (4.1)$$

$$\xi_m = \frac{\tan \alpha}{\sqrt{H_s/L_m}} \quad (4.2)$$

$$L_m = \frac{gT_m^2}{2\pi} \quad (4.3)$$

Where γ_i is the reduction factor (0.6), ξ_m is the surf similarity parameter, L_m is the deepwater wavelength for the modal period, c and d are constant depending on the exceedance probability of runup. The wave run-up exceedance probability for 2%, 10%, and 50% for wave height (H_i) 0.20m, 0.15m and 0.10m is shown in Table 4.13. The value of runup exceeds the freeboard, R_c of 0.05m even for exceedance probability of 50% which concludes that the rock revetment experience overtopping without crown structure.

Table 4.13: Wave run-up during incident wave, $H_o=0.2\text{m}$, 0.15m and 0.10m

Ru exceedance probability (%)	Period T_p (s)	Runup (m)		
		$H_i = 0.2\text{m}$	$H_i = 0.15\text{m}$	$H_i = 0.1\text{m}$
2	1.5	0.211	0.169	0.124
	1.0	0.186	0.140	0.103
10	1.5	0.163	0.130	0.095
	1.0	0.138	0.110	0.080
50	1.5	0.097	0.077	0.055
	1.0	0.091	0.067	0.048

4.4 Overtopping Experimental Results

If extreme run-up levels exceed the crest level, the structure will be overtopped. This may occur for relatively few waves during the design event. A low overtopping rate is often accepted without severe consequences to the structure. The overtopping occurrence is illustrated in Figure 4.6.

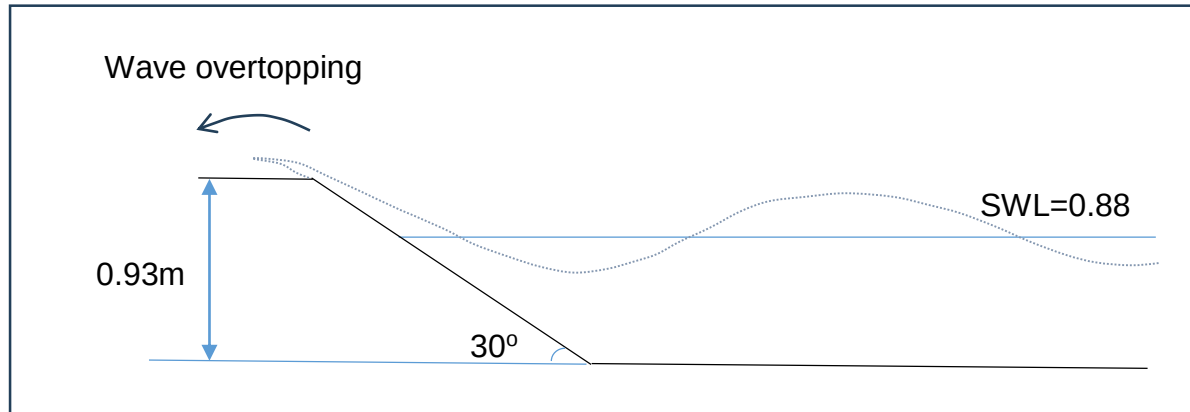


Figure 4.6: Wave overtopping

4.4.1 Experimental Results

Table 4.13 displays the overtopping volume data collection summary for both regular and irregular wave for total period of 300s and 900s for three conditions, without crown, type 1 and type 2 crown condition. All the water passing over the structure were collected and measured manually. Overtopping discharge was calculated using Equation 4.4.

$$q = \frac{V_m}{BT_R} \quad (4.4)$$

Where:

q = Overtopping discharge rate (l/s/m)

V_m = measured volume (l)

T_R = model run time (s)

B = overtopping channel width (m) = 0.1m

The water level below crest recorded is 0.04m and the actual water level (h) in the flume is 0.88m. The categorization of test cases based on specific conditions accordingly can be referred in Table 3.5. The introduction of crown units significantly reduces overtopping volume.

Table 4.14: Summary of experimental overtopping volume

Type of wave	Test case	Wave height, H_i (m)	Overtopping discharge, q (l/s/m)		
			Without crown	Crown Model 1	Crown Model 2
Regular	1	0.20	6.700	0.028	0.117
	2	0.15	1.567	0.010	0.043
	3	0.10	0.083	0.003	0.003
	4	0.2	0.733	0.007	0.020
	5	0.15	0.333	0.003	0.007
	6	0.1	0.100	0.007	0.000
Irregular	7	0.2	0.478	0.044	0.062
	8	0.15	0.122	0.009	0.011
	9	0.1	0.034	0.002	0.001
	10	0.2	0.083	0.002	0.003
	11	0.15	0.156	0.002	0.005
	12	0.1	0.017	0.001	0.001

Figure 4.7 to 4.9 shows the overtopping discharge with and without crown unit at intended wave height (H_o) of 0.1m, 0.15m and 0.2m for both regular and irregular waves. In general, the overtopping discharge during mean period, $T_p=1.5s$ is higher than $T_p=1.0s$ for all H_o values. Overtopping discharge decreases with H_o values except for crown model 1 scenario of regular wave at period 1.0s in Figure 4.7 where $H_o=0.10m$ recorded higher value of overtopping at 0.007 l/s/m compared to $H_o=0.15m$ (0.003 l/s/m). This could be due to an experimental error while collecting overtopping discharge manually behind the crown unit model. The regular waves recorded higher values of overtopping volume compared to irregular waves propagation except crown model 1 scenario at $H_o=0.2m$ where overtopping during irregular wave (0.044 l/s/m) is higher than regular wave (0.028 l/s/m). This could be contributed by higher $H_{1/3}$ value during irregular wave at 0.217m (refer to Test Case 7 at Table 4.7) compared to regular wave height at 0.193m (refer to Test Case 1 at Table 4.1)

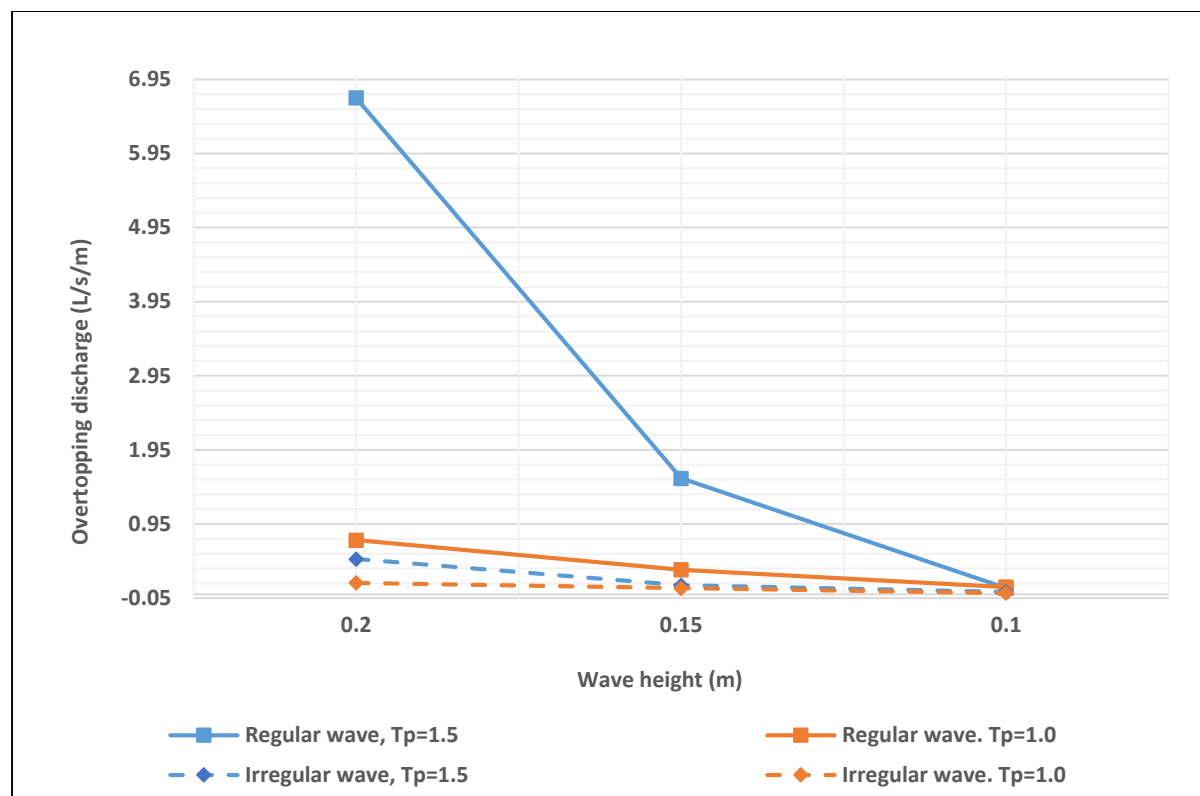


Figure 4.7: Wave overtopping discharge without crown unit at intended wave height 0.1m, 0.15m and 0.2m

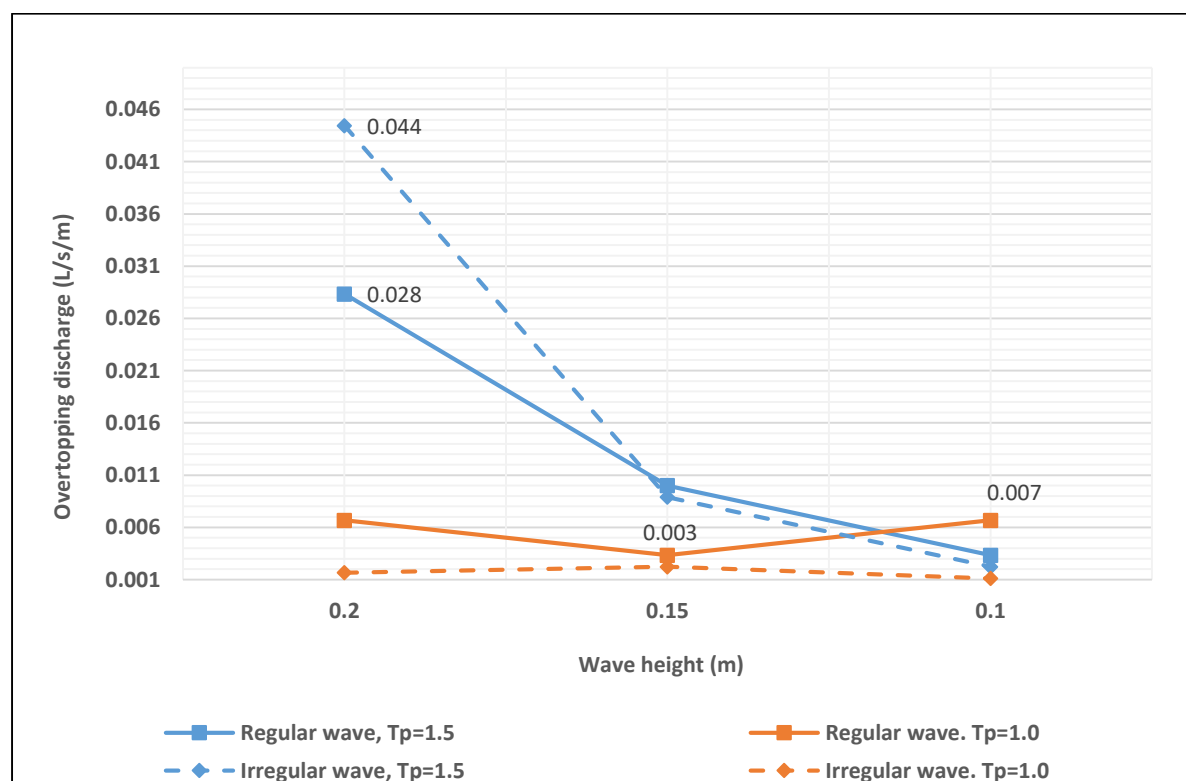


Figure 4.8: Comparison of wave overtopping discharge with crown unit (Model 1) at intended wave height 0.1m, 0.15m and 0.2m

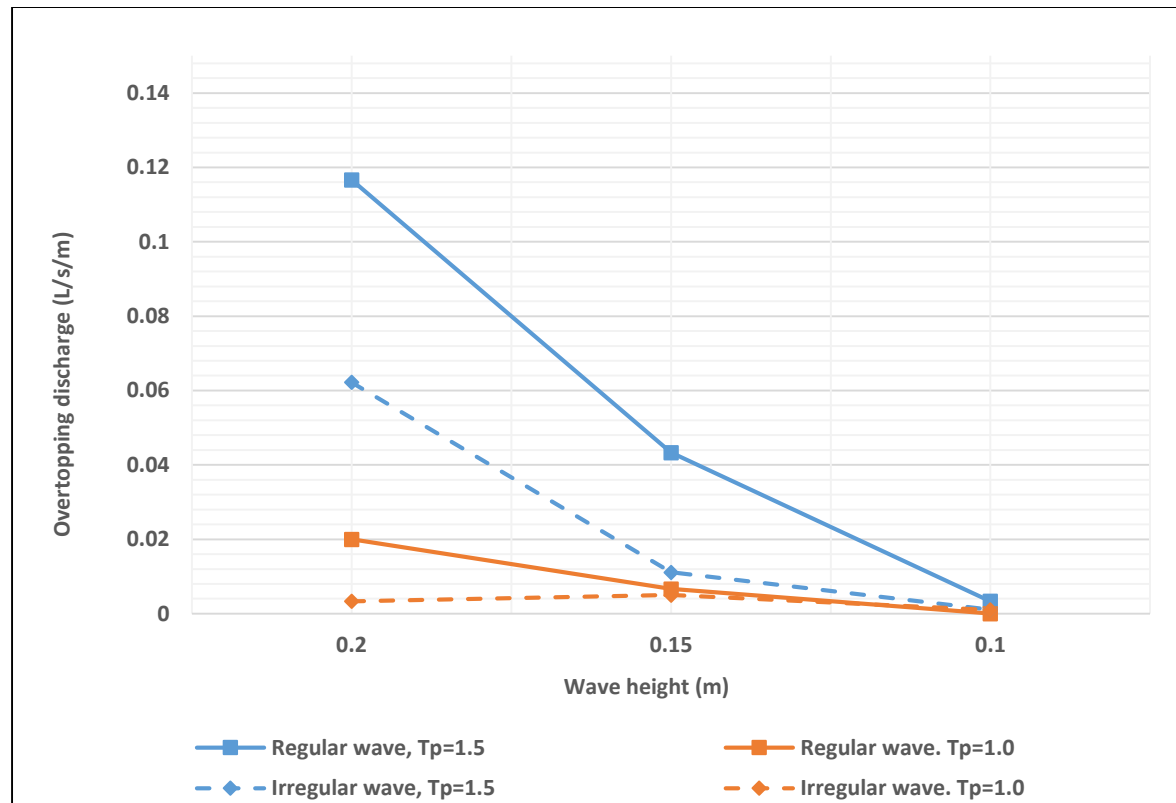


Figure 4.9: Comparison of wave overtopping discharge with crown unit (Model 2) at intended wave height 0.1m, 0.15m and 0.2m

Figure 4.10 displays the comparison of overtopping reduction during regular and irregular wave scenarios with and without crown units. The introduction of crown unit models shows a consistency of significant reduction in all cases.

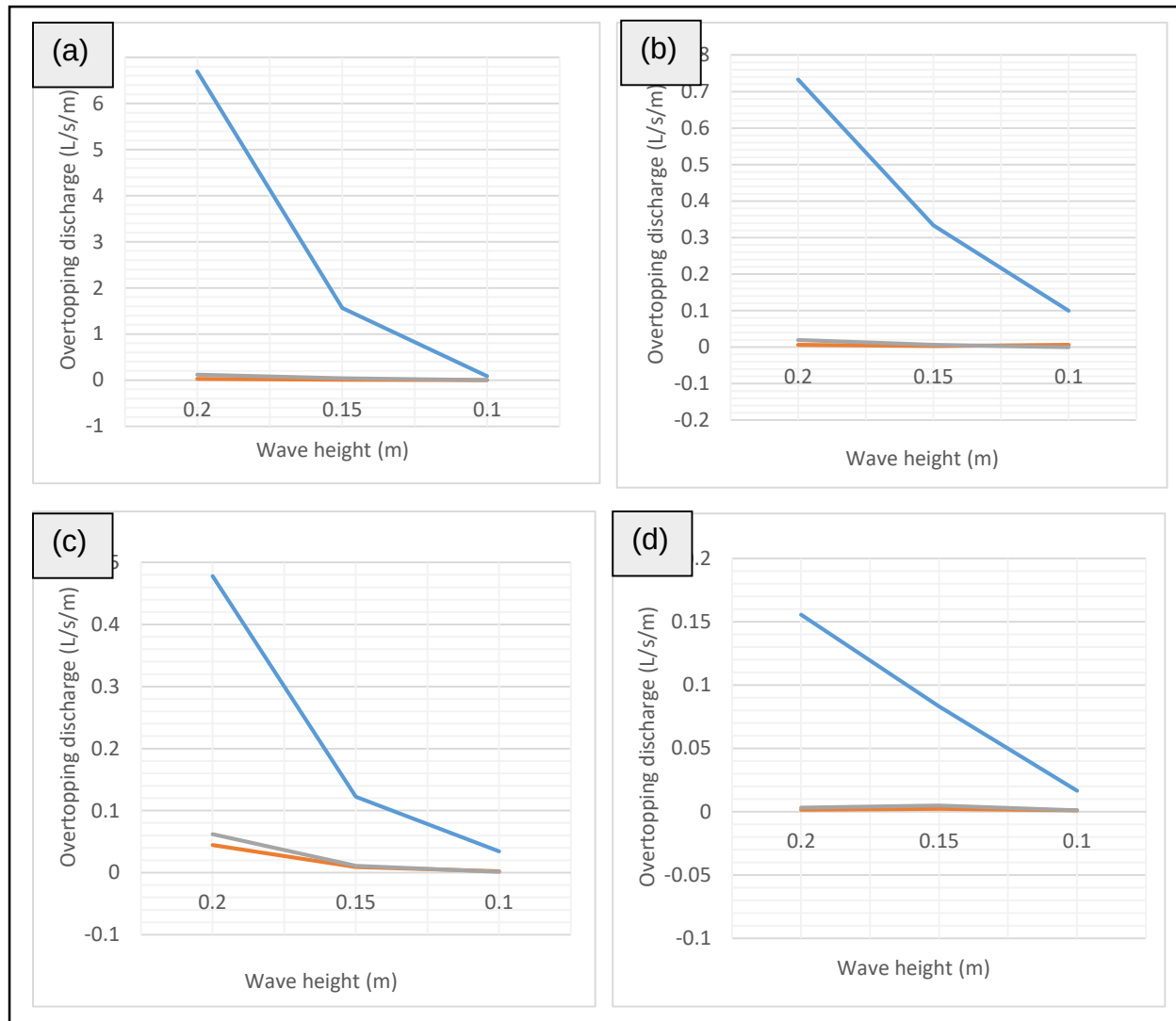
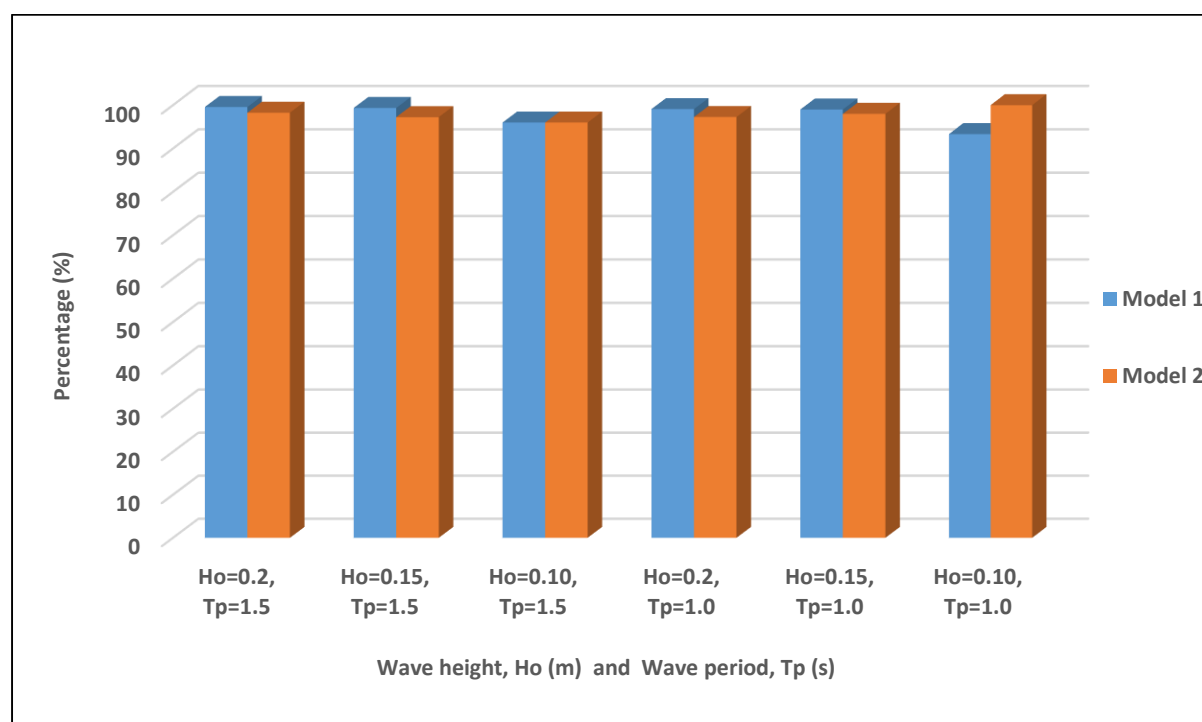


Figure 4.10: Comparison of wave overtopping discharge with and without crown units (a) regular wave, $T_p = 1.5$, (b) regular wave $T_p = 1.0$, (c) irregular wave, $T_p = 1.5$, (d) irregular wave $T_p = 1.0$ where the blue solid line ----- represents without crown, orange solid line ----- represents crown Model 1 and grey solid line ----- represents crown Model 2

Table 4.14 shows the performance of crown model unit between model 1 and 2. In general, all models show efficiency of more than 85 percent in reducing overtopping discharge. Crown unit model 2 express better reduction for wave height of 0.1m in test case 4 to 12 while crown unit model 1 shows better reduction for wave overtopping for wave height of 0.15m and 0.2m in all test case scenarios as shown in Figure 4.11. Overall, there is not much difference as both manage to reduce overtopping volume.

Table 4.15: Percentage reduction of overtopping volume for crown model 1 and 2

Type of wave	Test case	Wave height, H_i (m)	Percentage reduction (%)	
			Crown Model 1	Crown Model 2
Regular	1	0.20	99.58	98.26
	2	0.15	99.36	97.23
	3	0.10	96.00	96.00
	4	0.20	99.09	97.27
	5	0.15	99.00	98.00
	6	0.10	93.33	100.00
Irregular	7	0.20	90.70	86.98
	8	0.15	92.73	90.91
	9	0.10	93.55	96.77
	10	0.20	98.93	97.86
	11	0.15	97.33	94.00
	12	0.10	93.33	94.00



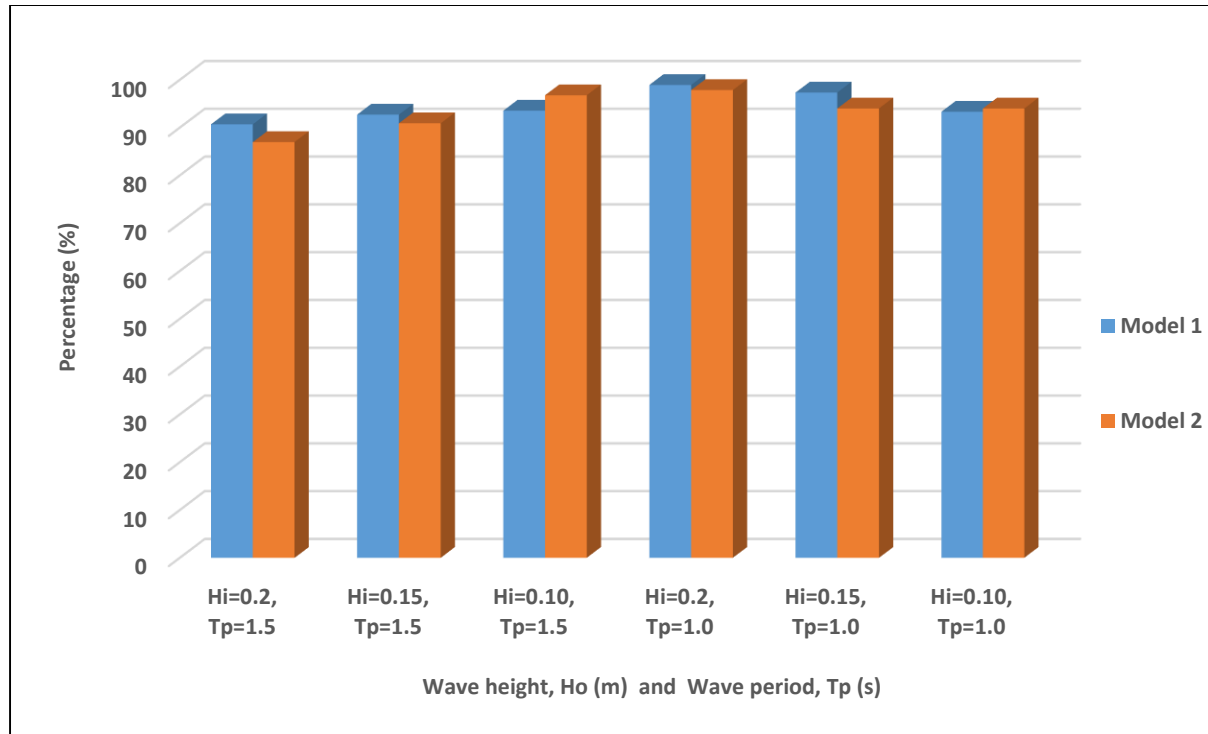


Figure 4.11: Comparison of efficiency between crown units model 1 and model 2 for regular wave (top) and irregular wave (bottom)

4.4.2 Empirical Formulation

Wave overtopping discharge (q) is calculated using the formulation in Equation 4.5 by Taw's method based on Rock Manual. A value of $q < 0.05$ is categorized as no damage to revetment seawalls.

$$\frac{q}{\sqrt{gH_{mo}^3}} = \frac{A}{\sqrt{\tan\alpha}} \gamma_b \xi_{m-1.0} \exp\left(-B \frac{R_c}{H_{mo}} \frac{1}{\xi_{m-1.0} \gamma_b \gamma_f \gamma_\beta}\right) \quad (4.5)$$

Where H_{mo} represents the significant wave height. α is the slope of revetment, $\xi_{m-1.0}$ is the spectral wave parameter. A and B are coefficient of average trend used in safety margin. γ_b , γ_f and γ_β is the berm factor, roughness reduction factor and correction factor for oblique waves respectively. The differences between empirical formula and experimental results are shown in Table 4.15. The experimental values obtained from overtopping are larger than empirical values.

Table 4.16: Comparison between empirical formulation and experimental output for overtopping discharge

T _p (s)	Overtopping (L/s/m)								
	H _o = 0.20m			H _o = 0.15m			H _o = 0.10m		
	Taw	Experimental		Taw	Experimental		Taw	Experimental	
		Reg	Irreg		Reg	Irreg		Reg	Irreg
1.5	4.994	6.700	0.478	2.226	1.567	0.122	0.588	0.083	0.034
1.0	3.463	0.733	0.156	1.171	0.333	0.083	0.261	0.100	0.017

4.4.3 Damage Criteria

The crest level of hydraulic structures is designed based on the wave overtopping discharge. The overtopping discharge varies greatly from wave to wave under random waves. Based on experimental values of overtopping discharge, the value of overtopping volume of prototype is determined by upscaling experimental q with scaling factor $1 : \lambda^{1.5}$. Table 4.17 to Table 4.22 shows the prototype overtopping discharge and its suitability for pedestrians, vehicles and damage for embankment or revetment seawalls based on Allsop et al., (2005) which is available in Section 2.4.3 in the literature.

Based on Table 4.17 and 4.18, more serious damage of the revetment is expected during regular waves compared to irregular waves without crown conditions. Under regular wave condition, wave height of 0.2m with 1.5s period will result in damage even if fully protected if embankment seawall is to be built. Meanwhile, construction of revetment seawall under similar conditions will be damaged even if promenade is paved. For embankment seawalls, a certain level of damage is still expected even with reduction in wave heights and wave periods while revetment seawalls show no damage for waves height 0.2m and 1.0s period and below. Under most wave conditions, pedestrians will expect the overtopping flows at low levels and is unsafe for trained staff, well shod and protected, expected to get wet, no falling jet, low danger of fall from walkway based on Allsop's criteria while USACE (2011) categorized most

design conditions as very dangerous for pedestrians. With the shortest wave height design of 0.10m and period 1.0s where q is larger than 1×10^{-4} , then the design is considered unsafe for aware pedestrians, clear view of the sea, and pedestrians will not be easily upset or frightened and able to tolerate getting wet with a wider walkway (Allsop et al., 2005). As for USACE (2011) criteria, the lowest wave conditions meet the criteria of q in between 5×10^{-4} and 1×10^{-3} , categorizing it as dangerous on grass sea dikes and horizontal composite breakwaters.

Table 4.18 displays similar characteristics of criteria damage as regular wave conditions (Table 4.17) for pedestrians. Meanwhile, less damage is expected under irregular waves conditions. If construction of revetment seawalls is considered, no damage is expected at all for all wave configurations. However, embankment wall construction of wave heights 0.15m and 0.20m will likely cause damage in structure if crest is not protected.

Table 4.17: Prototype overtopping discharge and critical damage without crown (Regular wave)

H _o (m)	Period (s)	Mean overtopping discharge, q (m³/s/m)		Embankment seawall		Revetment seawalls		Pedestrian	
		Exp	Prototype	Allsop et al., (2005)	USAC E (2011)	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)
0.20	1.5	0.00670	0.2119	q > 0.05 (Damage even if fully protected)		q > 0.2 (Damage even if promenade paved)		q > 1 x 10 ⁻³ – 0.01 (Unsafe for trained staff)	q > 1 x 10 ⁻³ (Very dangerous)
	1.0	0.00073	0.0231	0.02 < q < 0.05 (Damage if back slope not protected)		q < 0.05 (No damage)			
0.15	1.5	0.00157	0.0496	2 x 10 ⁻³ < q < 0.02 (Damage if crest not protected)					
	1.0	0.00033	0.0104						
0.10	1.5	0.00008	2.5 x 10 ⁻³					q > 1 x 10 ⁻⁴ (Unsafe for aware pedestrians)	5 x 10 ⁻⁴ < q < 1 x 10 ⁻³ (Dangerous on grass sea dikes and
	1.0	0.00010	3.2 x 10 ⁻⁴	q < 2 x 10 ⁻³ (No damage)					

							horizontal composite breakwaters)
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Table 4.18: Prototype overtopping discharge and critical damage without crown (Irregular wave)

H _o (m)	Period (s)	Mean overtopping discharge, q (m ³ /s/m)		Embankment seawall		Revetment seawalls		Pedestrian	
		Exp	Prototype	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)
0.20	1.5	0.000478	0.0151	2 x 10 ⁻³ < q < 0.02 (Damage if crest not protected)		q < 0.05 (No damage)		q > 1 x 10 ⁻³ – 0.01 (Unsafe for trained staff)	q > 1 x 10 ⁻³ (Very dangerous)
	1.0	0.000156	4.9 x 10 ⁻³						
0.15	1.5	0.000122	3.9 x 10 ⁻³						
	1.0	0.000083	2.6 x 10 ⁻³						
0.10	1.5	0.000034	1.1 x 10 ⁻³	q < 2 x 10 ⁻³ (No damage)				q > 1 x 10 ⁻⁴ (Unsafe for aware pedestrians)	5 x 10 ⁻⁴ < q < 1 x 10 ⁻³ (Dangerous on grass sea dikes and horizontal composite breakwaters)
	1.0	0.000017	5.4 x 10 ⁻⁴						

Table 4.19 to 4.22 shows the improvement of damage criteria with the introduction of crown units. Table 4.19 and 4.20 displays the criteria damage for overtopping with Crown 1 model under regular and irregular wave condition. For construction of embankment and revetment seawalls under regular wave, all wave configurations shows overtopping discharge of q less than 2×10^{-3} and 0.05, showing no damage to structure if crown unit 1 model is introduced to prototype. Overtopping discharge values of q more than 1×10^{-4} still shows that the seawall on site is unsafe for aware pedestrians. They will obtain clear view of sea, will not be easily upset or frightened and will be able to tolerate getting wet with a wider walkway based on Allsop et al., (2005). Since q values are between 5×10^{-4} and 1×10^{-3} , revetment is still considered dangerous on grass sea dikes and horizontal composite breakwaters according to USACE (2011).

For construction of embankment and revetment seawalls under irregular wave, all wave configurations show overtopping discharge of q less than 2×10^{-3} and 0.05, showing no damage to structure if crown unit 1 model is introduced to prototype. Overtopping discharge values of q more than 1×10^{-4} still shows that the seawall on site is unsafe for aware pedestrians. They will obtain a clear view of the sea, will not be easily upset or frightened and will be able to tolerate getting wet with a wider walkway. Meanwhile, q larger than 3×10^{-5} is unsafe for unaware pedestrians with no clear view of the sea and relatively easily upset or frightened with narrow walkway or proximity to edge based on Allsop et al., (2005). Wave configuration of 0.20m height and period 1.5s display q larger than 1×10^{-3} which is very dangerous for pedestrians based on USACE (2011). Other configurations display q values between 3×10^{-5} and 5×10^{-4} whereby it is dangerous for pedestrians on vertical wall breakwaters. Overtopping discharge of q between 5×10^{-4} and 1×10^{-3} shows revetment is considered dangerous for pedestrians on grass sea dikes and horizontal composite breakwaters according to USACE (2011).

Table 4.19: Prototype overtopping discharge and critical damage with model Crown 1 (Regular wave)

H _o (m)	Period (s)	Mean overtopping discharge, q (m ³ /s/m)		Embankment seawall		Revetment seawalls		Pedestrian	
		Exp	Prototype	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)
0.20	1.5	0.0000283	8.9×10^{-4}	$q < 2 \times 10^{-3}$ (No damage)		$q < 0.05$ (No damage)		$q > 1 \times 10^{-4}$ (Unsafe for aware pedestrians, clear view of sea, not easily upset or frightened, able to tolerate getting wet, wider walkway)	$5 \times 10^{-4} < q < 1 \times 10^{-3}$ (Dangerous on grass sea dikes and horizontal composite breakwaters)
	1.0	0.0000067	2.1×10^{-4}						
0.15	1.5	0.0000100	3.2×10^{-4}						
	1.0	0.0000033	1.0×10^{-4}						
0.10	1.5	0.0000033	1.0×10^{-4}						
	1.0	0.0000067	2.1×10^{-4}						

Table 4.20: Prototype overtopping discharge and critical damage with model Crown 1 (Irregular wave)

H _o (m)	Period (s)	Mean overtopping discharge, q (m ³ /s/m)		Embankment seawall		Revetment seawalls		Pedestrian	
		Exp	Prototype	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)
0.20	1.5	0.0000444	1.4×10^{-3}	$q < 2 \times 10^{-3}$ (No damage)		$q < 0.05$ (No damage)		$q > 1 \times 10^{-4}$ (Unsafe for aware pedestrians, clear view of sea)	$q > 1 \times 10^{-3}$ (Very dangerous)
	1.0	0.0000017	5.4×10^{-5}					$q > 3 \times 10^{-5}$ (Unsafe for unaware pedestrians, no clear view of sea)	$3 \times 10^{-5} < q < 5 \times 10^{-4}$ (Dangerous on vertical wall breakwaters)
0.15	1.5	0.0000089	2.8×10^{-4}					$q > 1 \times 10^{-4}$ (Unsafe for aware pedestrians, clear view of sea)	$5 \times 10^{-4} < q < 1 \times 10^{-3}$ (Dangerous on grass sea dikes and horizontal

							composite breakwaters)
	1.0	0.0000022	7.0×10^{-5}				
0.10	1.5	0.0000022	7.0×10^{-5}			$q > 3 \times 10^{-5}$ (Unsafe for unaware pedestrians, no clear view of sea)	$3 \times 10^{-5} < q < 5 \times 10^{-4}$ (Dangerous on vertical wall breakwaters)
	1.0	0.0000011	3.5×10^{-5}				

Table 4.21 and 4.22 display the criteria damage for overtopping with Crown 2 model under regular and irregular wave condition. For construction of embankment and revetment seawalls under regular wave, all wave configurations show overtopping discharge of q less than 2×10^{-3} and 0.05, showing no damage to structure if crown unit 1 model is introduced to prototype except for wave height of 0.20m and period 1.5s where q values are between 2×10^{-3} and 0.02, displaying damage if crest is not protected for embankment seawall. Overtopping discharge values of q more than 1×10^{-4} shows that the seawall on site is unsafe for aware pedestrians ($H_0=0.2\text{m}$, $T_p=1.0\text{s}$ and $H_0=0.15\text{m}$, $T_p=1.0\text{s}$). Overtopping discharge values of q more than 1×10^{-3} shows that the seawall is unsafe even for trained staff, pedestrians are well shod and protected, while expected to get wet, overtopping flows at lower levels only, no falling jet and low danger of fall from walkway ($H_0=0.2\text{m}$, $T_p=1.0\text{s}$ and $H_0=0.15\text{m}$, $T_p=1.0\text{s}$). At low wave height and period ($H_0=0.1\text{m}$, $T_p=1.0\text{s}$), q is more than 3×10^{-5} where the revetment is unsafe for unaware pedestrians as no clear view of the sea and will relatively be easily upset or frightened, narrow walkway or proximity to edge based on Allsop et al., (2005). Revetment is considered very dangerous for pedestrians at q larger than 1×10^{-3} under wave configurations of $H_0=0.2\text{m}$ and 0.15m with $T_p=1.0\text{s}$. Overtopping q values between 5×10^{-4} and 1×10^{-3} shows revetment is still considered dangerous on grass sea dikes and horizontal composite breakwaters according to USACE (2011). Only wave condition of $H_0=0.1\text{m}$ and $T_p=1.0\text{s}$ shows pedestrians are likely to be wet, but not uncomfortable.

For construction of embankment and revetment seawalls under irregular wave, all wave configurations show overtopping discharge of q less than 2×10^{-3} and 0.05, showing no damage to structure if crown unit 1 model is introduced to prototype. Overtopping discharge values of q more than 1×10^{-4} still shows that the seawall on site is unsafe for aware pedestrians. They will obtain a clear view of the sea, will not be easily upset or frightened and will be able to tolerate getting wet with a wider walkway. Meanwhile, q larger than 3×10^{-5} is unsafe for unaware pedestrians with no clear view of the sea and relatively easily upset or frightened with narrow walkway or proximity to edge based on Allsop et al., (2005). Wave configuration of 0.20m height and period 1.5s display q larger than 1×10^{-3} which is very dangerous for pedestrians based on USACE (2011). Other configurations display q values between 3×10^{-5} and

5×10^{-4} whereby it is dangerous for pedestrians on vertical wall breakwaters according to USACE (2011).

Table 4.21: Prototype overtopping discharge and critical damage with model Crown 2 (Regular wave)

H _o (m)	Period (s)	Mean overtopping discharge, q (m³/s/m)		Embankment seawall		Revetment seawalls		Pedestrian	
		Exp	Prototype	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)
0.20	1.5	0.000117	3.7 x 10 ⁻³	2 x 10 ⁻³ < q < 0.02 (Damage if crest not protected)		q < 0.05 (No damage)		q > 1 x 10 ⁻³ – 0.01 (Unsafe for trained staff)	q > 1 x 10 ⁻³ (Very dangerous)
	1.0	0.000020	6.3 x 10 ⁻⁴		q > 1 x 10 ⁻⁴ (Unsafe for aware pedestrians, clear view of sea)			5 x 10 ⁻⁴ < q < 1 x 10 ⁻³ (Dangerous on grass sea dikes and horizontal composite breakwaters)	
0.15	1.5	0.000043	1.4 x 10 ⁻³		q > 1 x 10 ⁻³ – 0.01 (Unsafe for trained staff)			q > 1 x 10 ⁻³ (Very dangerous)	

	1.0	0.000007	2.2×10^{-4}			$q > 1 \times 10^{-4}$ (Unsafe for aware pedestrians, clear view of sea)	$3 \times 10^{-5} < q < 5 \times 10^{-4}$ (Dangerous on vertical wall breakwaters)
	1.5	0.000003	9.5×10^{-5}			$q > 3 \times 10^{-5}$ (Unsafe for unaware pedestrians, no clear view of the sea)	$q < 4 \times 10^{-6}$ (Wet, but not uncomfortable)
0.10	1.0	0.000000	0.000000				

Table 4.22: Prototype overtopping discharge and critical damage with model Crown 2 (Irregular wave)

H _o (m)	Period (s)	Mean overtopping discharge, q (m ³ /s/m)		Embankment seawall		Revetment seawalls		Pedestrian	
		Exp	Prototype	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)	Allsop et al., (2005)	USACE (2011)
0.20	1.5	0.000062	2.0×10^{-3}	$q < 2 \times 10^{-3}$ (No damage)		$q < 0.05$ (No damage)		$q > 1 \times 10^{-3} - 0.01$ (Unsafe for trained staff)	$q > 1 \times 10^{-3}$ (Very dangerous)
	1.0	0.000003	9.5×10^{-5}					$q > 3 \times 10^{-5}$ (Unsafe for aware pedestrians, no clear view of sea)	$3 \times 10^{-5} < q < 5 \times 10^{-4}$ (Dangerous on vertical wall breakwaters)
0.15	1.5	0.000011	3.5×10^{-4}					$q > 1 \times 10^{-4}$ (Unsafe for aware pedestrians, clear view of sea)	
	1.0	0.000005	1.6×10^{-4}						
0.10	1.5	0.000001	3.2×10^{-5}					$q > 3 \times 10^{-5}$ (Unsafe for unaware)	
	1.0	0.000001	3.2×10^{-5}						

						pedestrians, no clear view of the sea, relatively easily upset or frightened, narrow walkway or proximity to edge)	
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5.0 CONCLUSION

A study was conducted National Water Research Institute Malaysia (NAHRIM) to analyze the efficiency of two crown wall unit designs in reducing wave overtopping and runup on revetment structure. The crown unit models of height 0.39m and 0.2m were designed and constructed at Hydraulic and Instrumental Laboratory, NAHRIM.

A rock revetment scaled down to ratio 1:10 is set up in the wave flume with dimensions 50m (length), 1.5m (width) and 2m (depth), with a maximum water depth of 0.88m. The wave probe was calibrated by ensuring the wave generated at probe located nearest to the revetment structure (Probe 3) is similar as intended wave height before the installation of crown units.

A total of twelve 2D physical test cases were carried out for regular wave and irregular wave propagation for a duration of 300s and 900s respectively at wave heights of 0.20m, 0.15m and 0.10m. Empirical calculations shows that rock revetment experience overtopping without crown structure as value of runup exceeds the freeboard 0.05m for exceedance probability of 2%, 10% and 50% which at all wave height condition.

The volume that exceeds the crest of the structure is collected manually and overtopping volume is measured as overtopping discharge. In general, regular waves recorded higher values of overtopping volume compared to irregular waves propagation. Both empirical and measured values agree that overtopping volume decreases with wave height values. When compared to without crown condition, both crown unit models managed to reduce the overtopping discharge by at least 87% as compared to without crown structure. Thus, the model designs of crown unit structure are effective in reducing wave overtopping effect.

REFERENCES

1. Almarshed, B., Figlus, J., Miller, J., & Verhagen, H. J. (2020). Innovative coastal risk reduction through hybrid design: combining sand cover and structural defenses. *Journal of Coastal Research*, 36(1), 174-188.
<https://doi.org/10.2112/jcoastres-d-18-00078.1>
2. Allsop, W., Bruce, T., Pearson, J., & Besley, P. (2005). Wave overtopping at vertical and steep seawalls. *Proceedings of the Institution of Civil Engineers - Maritime Engineering*, 158(3), 103–114.
<https://doi.org/10.1680/maen.2005.158.3.103>
3. Bayle, P. M., Blenkinsopp, C. E., Conley, D., Masselink, G., Beuzen, T., & Almar, R. (2020). Performance of a dynamic cobble berm revetment for coastal protection, under increasing water level. *Coastal Engineering*, 159, 103712.
<https://doi.org/10.1016/j.coastaleng.2020.103712>
4. Cao, D., Tan, W., & Yuan, J. (2022). Assessment of wave overtopping risk for pedestrian visiting the crest area of coastal structure. *Applied Ocean Research*, 120, 102985. <https://doi.org/10.1016/j.apor.2021.102985>
5. Capel, A. (2015). Wave run-up and overtopping reduction by block revetments with enhanced roughness. *Coastal Engineering*, 104, 76–92.
<https://doi.org/10.1016/j.coastaleng.2015.06.007>
6. Chen, W., van Gent, M. R. A., Warmink, J. J., & Hulscher, S. J. M. H. (2020). The influence of a berm and roughness on the wave overtopping at dikes. *Coastal Engineering*, 156, 103613.
<https://doi.org/10.1016/j.coastaleng.2019.103613>
7. Chen, C., Lin, Z., Beardsley, R. C., Shyka, T., Zhang, Y., Xu, Q., ... & Xu, D. (2021). Impacts of sea level rise on future storm-induced coastal inundations

- over Massachusetts coast. *Natural Hazards*, 106, 375-399.
<https://doi.org/10.1007/s11069-020-04467-x>
8. Chen, W., Warmink, J. J., van Gent, M. R. A., & Hulscher, S. J. M. H. (2021). Numerical modelling of wave overtopping at dikes using openfoam®. *Coastal Engineering*, 166, 103890. <https://doi.org/10.1016/j.coastaleng.2021.103890>
 9. CIRIA/CUR/CETMEF (2007). The Rock Manual. The use of rock in hydraulic engineering (2nd ed.). C683. London: CIRIA
 10. Coelho, C., Cruz, T., & Roebeling, P. (2016). Longitudinal revetments to mitigate overtopping and flooding: Effectiveness, costs and benefits. *Ocean & Coastal Management*, 134, 93–102.
<https://doi.org/10.1016/j.ocecoaman.2016.09.022>
 11. Contestabile, P., Crispino, G., Russo, S., Gisonni, C., Cascetta, F., & Vicinanza, D. (2020). Crown wall modifications as response to wave overtopping under a future sea level scenario: An experimental parametric study for an innovative composite seawall. *Applied Sciences*, 10(7), 2227.
<https://doi.org/10.3390/app10072227>
 12. DID (2015). National Coastal Erosion Study for Malaysia 2015. *Executive Summary*, I, 5-27.
 13. Di Lauro, E., Maza, M., Lara, J. L., Losada, I. J., Contestabile, P., & Vicinanza, D. (2020). Advantages of an innovative vertical breakwater with an overtopping wave energy converter. *Coastal Engineering*, 159, 103713.
<https://doi.org/10.1016/j.coastaleng.2020.103713>
 14. Eurotop Manual. (2007). Wave Overtopping of Sea Defences and Related Structures—Assessment Manual. UK: NWH Allsop, T. Pullen, T. Bruce. NL: JW van der Meer. DE: H. Schüttrumpf, A. Kortenhaus. www.overtopping-manual.com.

15. Fortes, C. J. E. M., Reis, M. T., Pinheiro, L., Poseiro, P., Serrazina, V., Mendonça, A., Smithers, N., Santos, M. I., Barateiro, J., Azevedo, E. B., Salvador, M., & Reis, F. V. (2020). The HIDRALERTA system: Application to the ports of madalena do pico and S. Roque do Pico, Azores. *Aquatic Ecosystem Health & Management*, 23(4), 398–406. <https://doi.org/10.1080/14634988.2020.1807295>
16. Geng, T., Liu, H., & Dias, F. (2021). Solitary-wave loads on a three-dimensional submerged horizontal plate: Numerical computations and comparison with experiments. *Physics of Fluids*, 33(3).<https://doi.org/10.1063/5.0043912>
17. Gómez-Gesteira, M., Cerqueiro, D., Crespo, C., & Dalrymple, R. A. (2005). Green water overtopping analyzed with a SPH model. *Ocean Engineering*, 32(2), 223–238. <https://doi.org/10.1016/j.oceaneng.2004.08.003>
18. Hughes, S. A., & Nadal, N. C. (2009). Laboratory study of combined wave overtopping and storm surge overflow of a levee. *Coastal Engineering*, 56(3), 244–259. <https://doi.org/10.1016/j.coastaleng.2008.09.005>
19. Hunt-Raby, A. C., Borthwick, A. G. L., Stansby, P. K., & Taylor, P. H. (2011). Experimental measurement of focused wave group and solitary wave overtopping. *Journal of Hydraulic Research*, 49(4), 450–464. <https://doi.org/10.1080/00221686.2010.542616>
20. Imaizumi, N., Takahasi, T., Tanaka, H., & Minami, M. (2019). Numerical Analysis of the water level change on the overtopping wave power generator. *APAC 2019*, 1127–1133. https://doi.org/10.1007/978-981-15-0291-0_154
21. Koosheh, A., Etemad-Shahidi, A., Cartwright, N., Tomlinson, R., & van Gent, M. R. A. (2021). Individual wave overtopping at Coastal Structures: A critical review and the existing challenges. *Applied Ocean Research*, 106, 102476. <https://doi.org/10.1016/j.apor.2020.102476>

22. Lashley, C. H., Zanuttigh, B., Bricker, J. D., Van der Meer, J., Altomare, C., Suzuki, T., & Oosterlo, P. (2020). Benchmarking of numerical models for wave overtopping at dikes with shallow mildly sloping foreshores: Accuracy versus speed. *Environmental Modelling & Software*, 130, 104740. <https://doi.org/10.1016/j.envsoft.2020.104740>
23. Lee, W.-D., Choi, S., Kim, T., & Yeom, G.-S. (2022). Comparison of solitary wave overtopping characteristics between vertical and wave absorbing revetments. *Ocean Engineering*, 256, 111542. <https://doi.org/10.1016/j.oceaneng.2022.111542>
24. Li, X., Leonardi, N., & Plater, A. J. (2020). Impact of barrier breaching on wetland ecosystems under the influence of storm surge, sea-level rise and freshwater discharge. *Wetlands*, 40(4), 771-785. <https://doi.org/10.1007/s13157-019-01219-x>
25. Liu, L. L., & Wang, Y. C. (2020, December). Classification and applicability analysis of ecological revetment in different types of river. In IOP Conference Series: Earth and Environmental Science (Vol. 612, No. 1, p. 012008). IOP Publishing. <https://doi.org/10.1088/1755-1315/612/1/012008>
26. Losada, I. J., Lara, J. L., Guanche, R., & Gonzalez-Ondina, J. M. (2008). Numerical Analysis of wave overtopping of rubble mound breakwaters. *Coastal Engineering*, 55(1), 47–62. <https://doi.org/10.1016/j.coastaleng.2007.06.003>
27. Mares-Nasarre, P., Gómez-Martín, M. E., & Medina, J. R. (2019). Influence of mild bottom slopes on the overtopping flow over mound breakwaters under depth-limited breaking wave conditions. *Journal of Marine Science and Engineering*, 8(1), 3. <https://doi.org/10.3390/jmse8010003>

28. Naylor, L. A., Kippen, H., Coombes, M. A., Horton, B., MacArthur, M., & Jackson, N. (2017). Greening the Grey: a framework for integrated green grey infrastructure (IGGI).
29. Nordstrom, K. F. (2014). Living with shore protection structures: a review. *Estuarine, coastal and shelf science*, 150, 11-23. <https://doi.org/10.1016/j.ecss.2013.11.003>
30. Pilarczyk, K. (Ed.). (2017). Dikes and revetments: design, maintenance and safety assessment. Routledge.
31. Puspita, A. I., Thaha, M. A., Pallu, M. S., & Maricar, F. (2020). Effect of wave steepness to relative wave run-up on OWEC Breakwater. *IOP Conference Series: Earth and Environmental Science*, 419(1), 012117. <https://doi.org/10.1088/1755-1315/419/1/012117>
32. Qu, K., Zhang, L. B., Yao, Y., & Jiang, C. B. (2022). Numerical evaluation of influences of onshore wind on overtopping characteristics of coastal seawall under solitary wave. *Ocean Engineering*, 266, 112860. <https://doi.org/10.1016/j.oceaneng.2022.112860>
33. Reeve, D., Chadwick, A., & Fleming, C. (2018). Coastal engineering: processes, theory and design practice. Crc Press.
34. Rosendahl Appelquist, L., & Halsnæs, K. (2015). The Coastal Hazard Wheel System for coastal multi-hazard assessment & Management in a changing climate. *Journal of Coastal Conservation*, 19(2), 157–179. <https://doi.org/10.1007/s11852-015-0379-7>
35. Schoonees, T., Kerpen, N. B., & Schlurmann, T. (2021). Full-scale experimental study on wave overtopping at stepped Revetments. *Coastal Engineering*, 167, 103887. <https://doi.org/10.1016/j.coastaleng.2021.103887>

36. Schüttrumpf, H., & Oumeraci, H. (2005). Layer thicknesses and velocities of wave overtopping flow at seadikes. *Coastal Engineering*, 52(6), 473–495. <https://doi.org/10.1016/j.coastaleng.2005.02.002>
37. Shao, S., Ji, C., Graham, D. I., Reeve, D. E., James, P. W., & Chadwick, A. J. (2006). Simulation of wave overtopping by an incompressible SPH model. *Coastal Engineering*, 53(9), 723–735. <https://doi.org/10.1016/j.coastaleng.2006.02.005>
38. Siegel, F. R., & Siegel, F. R. (2020). Structures that protect coastal populations, assets, and GDPs: sea dikes, breakwaters, seawalls. Adaptations of Coastal Cities to Global Warming, Sea Level Rise, Climate Change and Endemic Hazards, 11-25. https://doi.org/10.1007/978-3-030-22669-5_3
39. Sugano, T., Nozu, A., Kohama, E., Shimosako, K. I., & Kikuchi, Y. (2014). Damage to coastal structures. *Soils and Foundations*, 54(4), 883-901. <https://doi.org/10.1016/j.sandf.2014.06.018>
40. USACE, (2011). Coastal Engineering Manual. Chapter 5: Fundamentals of Design. US Army Corps of Engineers, Manual No. 1110-2-1902
41. van Bergeijk, V. M., Warmink, J. J., & Hulscher, S. J. M. H. (2022). The effects of transitions in cover type and height on the wave overtopping load on grass-covered flood defences. *Applied Ocean Research*, 125, 103220. <https://doi.org/10.1016/j.apor.2022.103220>
42. Van der Meer, J. W. (1988). 6. Stability of cubes, tetrapods and accropode. *Design of breakwaters*, 71-80. <https://10.1680/dob.13513.0007>
43. Van Doorslaer, K., De Rouck, J., Audenaert, S., & Duquet, V. (2015). Crest modifications to reduce wave overtopping of non-breaking waves over a smooth dike slope. *Coastal Engineering*, 101, 69–88. <https://doi.org/10.1016/j.coastaleng.2015.02.004>

44. van Gent, M. R. A., Wolters, G., & Capel, A. (2022). Wave overtopping discharges at rubble mound breakwaters including effects of a crest wall and a berm. *Coastal Engineering*, 176, 104151. <https://doi.org/10.1016/j.coastaleng.2022.104151>
45. Vidura, A., Nurjaya, I. W., Iqbal, M., & Jaya, I. (2020). Ocean wave measurement and wave energy calculation using overtopping power plant scheme. *IOP Conference Series: Earth and Environmental Science*, 429(1), 012047. <https://doi.org/10.1088/1755-1315/429/1/012047>
46. Vrijling, J. K., & Van Gelder, P. H. A. J. M. (2020). Policy implications of uncertainty integration in design. *Stochastic Hydraulics 2000*, 633–646. <https://doi.org/10.1201/9781003078630-83>
47. Wang, H., Teng, L., Zhou, Z. P., & Zuo, Q. H. (2019). Numerical Study and verification of wave overtopping rates over a seadike. *APAC 2019*, 27–33. https://doi.org/10.1007/978-981-15-0291-0_5
48. Zhang, X., Draper, S., Wolgamot, H., Zhao, W., & Cheng, L. (2019). Eliciting features of 2d Greenwater overtopping of a fixed box using modified dam break models. *Applied Ocean Research*, 84, 74–91. <https://doi.org/10.1016/j.apor.2019.01.006>
49. Zhang, Y., & Ye, J. (2021). Physical modelling of the stability of a revetment breakwater built on reclaimed coral calcareous sand foundation in the South China Sea—random waves and dense foundation. *Ocean Engineering*, 219, 108384. <https://doi.org/10.1016/j.oceaneng.2020.108384>
50. Zhao, E., Qu, K., & Mu, L. (2019). Numerical study of morphological response of the sandy bed after tsunami-like wave overtopping an impermeable seawall. *Ocean Engineering*, 186, 106076. <https://doi.org/10.1016/j.oceaneng.2019.05.058>