Investigation on the Dynamic Stability of the Berm Breakwaters

By

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requirements for the
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CERTIFICATION OF APPROVAL

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JANUARY 2014

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

NURUL WAHIDA BT. ABDULLAH

ABSTRACT

The development of erosion control structures as a mitigation to maintain the open coast beaches has grew well throughout the years. Breakwater is one of the methods to address the problem where it functions by reducing wave energy transmitted at the shoreline. The purpose of this study is to investigate the behavior and stability of the reshaping breakwater subject to the variations of wave characteristics. Apart from that, this research study is conducted to get a better understanding on how the reshaped profile will obtain its stability. The gradation and shape of the armor stone used as well as the design cross section of the physical model have been initially set as independent parameters.

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بسم الله الرحمن الرحيم

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CHAPTER 1
INTRODUCTION

1.1. Project Background

Erosion problems in the open coastal areas have been seriously issued by most of the coastal users and managers all around the world. This problem is probably caused by the natural processes occur at the coastal area including the rise of sea level and also the action of waves, current and tides. Another factor that may lead to this problem is from the human activities itself for example excessive sand mining and coastal engineering works.

Prior to that, the management has been considering the inclusion of the erosion control structures in order to protect the shoreline as well as protecting the existing structure constructed near to the beach. If there is no mitigation undertaken to overcome this problem, the upland properties will be demolished and that may cause the declination of economic growth in that area. Hence, several strategies have been outlined as part of the erosion control project.

There are two basic types of coastal erosion control structures which are shoreline hardening structures which function as an upland property protection. They are seawalls, revetments and bulkheads. The second type of the structure is sand retention structures that trap and retain the sand. The structures named as groins and breakwaters. In Malaysia, breakwaters are widely used to provide protection to the port and harbor facility from dynamic forces of the ocean waves.

Breakwaters were generally designed with specific ranges of armor stones to withstand the wave loads. Breakwaters are commonly built as statically stable design breakwaters. There are also breakwaters which are based on dynamically stable design concept. Basically, dynamically stable breakwaters are formed as a result of reshaping by wave action. Priest et al. (1964), Moutzouris (1978), Kogami (1978), Naheer and Buslov (1983) and Bruun (1985) stated that formation of S-shaped profile from dynamically stable berm breakwaters is to be superior in performance to statically stable breakwaters.

1.2. Problem Statement

Breakwaters generally dissipate the incoming wave energy that hit the shoreline. The original design of statically stable breakwaters was built of several stone classes where minimal adjustment is allowed. Still, it is recognized that the reshaping will occur during the life time of the structure but only minor deformation of the breakwater is allowed under design condition.As it is built from only several stone classes and narrow stone size gradation, it occasionally faces quarry yield problem. This is because the preliminary designs are referred to the initial size distribution estimated from the potential quarries. Apart from that, it shows that the demand in constructing this type of structure had caused the increment of the cost and environmental disturbance.

Another concern that has been arising throughout the construction of statically stable breakwaters is the capability of the breakwater profile to change under severe wave condition. This is because the profiles of statically stable breakwaters are not permitted to change meanwhile the profiles of dynamically stable breakwaters may change according to wave climate. This change will lead to more stable slope in future.

As in Malaysia, most of the constructed berm breakwaters are statically stable breakwater. Typically, the material used to replace the armor stones are interlocking unit such as interlocking concrete unit. This happened because of the size of stones for the design berm breakwater is not available in production. The practice of interlocking unit in breakwater construction is applicable since it can replace the armor stone and reduce the volume of stones used in the design. But still, it is not fully reliable because of the higher cost in term of fabrication and royalty.

From the reason stated above, another type of breakwater which is dynamically stable berm breakwater is introduced. In this condition, the structure profile is reshaped into a stable profile resulted from the wave action. These structures are sometimes referred to as Icelandic-type berm breakwaters. These tailor-make structures were designed accordingly to the design wave load, possible quarry yield, and available equipment and transport routes.

1.3. Objectives

Although berm breakwaters have been developed decades ago, they still need to be further studied for better understanding of their great potential applications. This is because berm breakwaters are designed as a supply based and should come out with a functional specification.

There are two objectives of this study which include:

i. To investigate the behavior and stability of the dynamically stable breakwater subject to the variations of wave characteristics.
ii. To assess the influence of breakwater model on the transmitted wave measured behind the model.

1.4. Scope of Study

Dynamically stable berm breakwater is designed with the purpose of handling the quarry yield prediction issue where in some cases; they need to modify the profile of the structure in order to fit the stone classes available in the quarry. This structure is allowed to reshape, where the stones are moving up and down the slope which turn into an S-shape profile. Hence, an attempt is made to investigate the influence of the stone size gradation and wave characteristics on the stability of S-shape breakwater profile.

In order to achieve the result of the investigation, this study will include three steps which are:

i. Designing the berm breakwater model
ii. Testing the model with successively increasing wave height
iii. Analyzing the results indicating the behavior and stability of the model.

1.5. Relevancy of the Project

Negative impacts resulted from the coastal erosion might cause the society as well as the world to be affected in term of the economic point of view. Hence, the developments of erosion control structures have been progressing well throughout the years. These include the conventional and non-conventional protection methods in order to mitigate this problem. Coastal erosion mitigation and protection has been shifting towards enhanced methods rather than the conventional ones. The high performance capability of concrete armor units has made them as one of the best alternatives to protect the coastal area from erosion problem. However, not all coastal areas in Malaysia are applicable to this type of alternative since they only can cater low bearing capacity (muddy soil).

Besides that, the higher cost of its production also contributes to unreliable alternative. Hence, some researchers are interested to shift away to the basic type of coastal erosion control structure which is berm breakwaters. Hence, findings from this study will provide better understanding on the behavior and stability of the reshaping breakwaters.

1.6. Feasibility of the Project

This research is a fundamental study of dynamic stability ofberm breakwaters. It is considered feasible in terms of several aspects such as the material used, equipment and time frame scheduled throughout the two semesters provided. This study focused on the behavior and stability of the berm breakwater subject to variations of wave characteristics. Another main finding from this study is the assessment on the wave transmission behind the breakwater model. In order to complete the project, those activities are planning to conduct during the course of the project:

- ✓ Comprehensive reading of technical paper and journal
- ✓ Meet the expertise such as lecturers.
- ✓ Conduct experiment / Testing the prototype
- ✓ Analysis of results

CHAPTER 2
LITERATURE REVIEW

Coastal erosion that occurs along the world's coastline is resulting from the wave action which leads to the loss of sediment in some areas as well as give an environmental impact to the nearby habitat (Reintroducing Structures for Erosion Control on the Open Coasts of America, 2011). In recent years, most of the coastal managers are reconsidering the development of structures along the shoreline in order to protect the upland properties. These structures are mainly categorizes into two types which are shoreline hardening structure and sand retention structure. The need of constructing these structures is to reduce the beach erosion by dissipating the incoming wave energy.

2.1. Consideration of Erosion Control Structures

Basically, there are two types of erosion control structures that have been widely applied which are:

i. Shoreline hardening structure
ii. Sand retention structure

Structures that fall under shoreline hardening structures are seawall, revetment, and bulkheads, meanwhile sand retention structures are consist of groins and breakwaters where they function to trap and retain sand.

Erosion control structures are considered important mainly because of the erosion problem along the open coast. Internationally described, there are many examples of erosion control structures that have been used to successfully retain sand and control erosion problem such as Holly Beach in Louisiana, The Racine Harbour North breakwater located Lake Michigan, Canada as well as Mortavika berm breakwater in Norway. Those breakwater structures are examples of breakwater field which combined with beach nourishment that performing very well.

In some areas, beach nourishments are needed to increase the aesthetic value of the coastal area. Unfortunately, within nourishment project take place there will be an area known as hot spot erosion areas where the rate of losing sand due to heavy storm that need to be aware. These hot spot erosion areas are good candidates for erosion control structures before the scheduled renourishment take place.

The use of this structure is to slow the erosion of hot spot or to stop it from eroding to worst condition. In this case, appropriate type and number of structures provided in other to mitigate the issue are considered important so that beach renourishment management can effectively manage the volume of sand needed for each nourishment interval.

Campbell and Jenkins (2002) reported that the cost of the sand used for beach renourishment will go slightly higher as the sand sources get progressively further away with each renourishment. This phenomenon can be seen in South Florida where sand can be said as scarce. Hence, in order to reduce the cost of beach renourishment, erosion control structures built in that areas can reduce the erosion problem as well as conserving the usage of sand for a longer period. Thus, the cost for beach re-nourishment can be lowered down with the existence of breakwater.Figure 1 (Alvarez, 2013) illustrates the erosion occurred at Haulover Beach.

Figure 1 Erosion at Haulover Beach Park in Miami-Dade County (Alvarez 2013)

Another reason to consider the use of erosion control structures is the effectiveness of this structure to help in controlling sand migration especially in environmentally sensitive areas(Committee, 2011). These sensitive areas are named it that way because of marine resource habitats are located near shore. The presence of erosion control structures in that area will enable the placement of less sand and to have less frequent nourishment events. Hence, sensitive marine resource areas can be protected.

2.2. Breakwater Structure

Breakwater structures have been introduced long time ago as a solution to reduce coastal erosion. In the early construction of these structures, they are known as rubble mound breakwaters where there are two different typesi.e. conventional rubble mound breakwaters and berm breakwaters. Conventional rubble mound breakwaters are made up of several stone classes with a very narrow stone size gradation. They consist of main armor layer which is designed for a limited damage.

Meanwhile, berm-type breakwaters have been developed in the early of 1980's. These structures were built as one of the method to address the coastal erosion problems by optimizing the structure not only respect to the wave action but also the quarry yield prediction(Bruun, 1985). Basically, the berm breakwaters have been developed in two directions. The first type of structure is referred to dynamically stable berm-type breakwater where this structure is made up of two stone classes where it is allowed to reshape. In this case, the individual stones of the structure are moving up and down till the stable profile is developed.

On the other hand, more stable profile of berm breakwater is established, named as statically stable berm breakwater. This type of structure is more stable compared to the initial breakwater developed because only minor deformation is allowed and the reshaping into S-shape profile is strictly prohibited. This is because the integrity of the structure is highly retainable which respect to the interlocking of the stones used. This berm breakwater is also known as "Icelandic type" since they have been designed and constructed in Iceland during 1983 up until now. Sigurdarson et al. (2006) reported that the design approach of this kind of breakwater is not necessarily to merely meet the prescribed stability number but needs to correlate the size of distribution of stones from the armor stone quarry, the quality of the rock used as well as the wave characteristics at the site.

Figure 2(Verhagen, 2012) below illustrates the berm breakwater cross section with recession. The reshaping breakwater can be easily evaluated by the recession of the structure (Burcharth and Frigaard, n.d). This evaluation made to identify either the berm is failure or not.

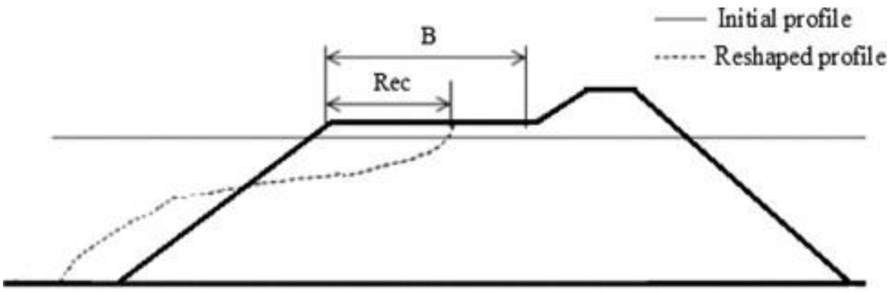


Figure 2 Cross section of berm breakwater design (Verhagen, 2012)

Apart from that, this structure also takes into account the quality of the rock in order to achieve the integrity of the structure. It is also included the design wave height, wave period and direction, water depth and etc. Another important criterion that should be considered is the construction of the breakwater itself. Breakwaters may be constructed fixed or floating, impermeable or permeable in order to allow cross shore sediment transport. But this choice depends on the tidal range and water depth at the constructed area. Figure 3(Halcrow Corporation, 2011) below shows rubble mound breakwater constructed along the shoreline meanwhile Figure 4(Halcrow Corporation, 2011) illustrates constructed rubble mound in few meters shoreward or also called as detached breakwaters.

Figure 3 Rubble mound breakwaters constructed along the shoreline (Halcrow Corporation, 2011)

Figure 4 Rubble mound breakwaters constructed few meter shoreward (Halcrow Corporation, 2011)

These two different locations for breakwaters are primarily depending on the design wave height provided by the consultant. Breakwaters that placed away from the shoreline tend to have bigger wave height that break on the structure. This condition requires larger amour sizes that need to protect the structures from failure due to high wave condition. The conventional design of breakwater is consists of two main layers of armor stones. These two layers are differentiated in term of the stone size used where the core layer require narrowly graded stone size meanwhile armor layer need a larger stone sizes. Figure 5 below illustrates the typical cross section of breakwater.

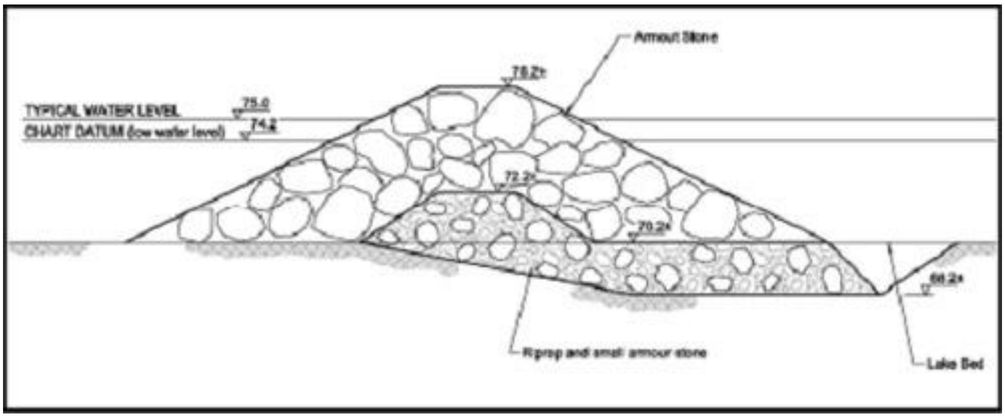


Figure 5 TypicalDesign cross section used for conventional breakwater

2.3. Development of Berm Breakwater

The initial design of breakwater is merely aimed to protect the coastline from erosion without paying attention on the possible yield from amour stone quarry. This structure is more stable because of the stone sizes used in the design are consists of several graded stones that can retain its integrity by its own. The integrity retained will prohibit the structure from reshaping. The good interlocking of the stones is obtained by carefully placed stones at site.

In the late of 19th century,an equilibrium slope as well as the permeability of the structure has been featured by many researchers and engineers(Sigurdarson et al., 2006). The idea to obtain an equilibrium slope of the structure has been applied by just dumping all the quarried material at the breakwater site provided is a rather wide berm. This is shown by the breakwater built in Plymouth, England and Cherbourg.

The breakwater structure in Mangalore, India was built using smaller size of rock in a rather wide berm. S-shaped profile will be developed when it comes to the maturing part. Furthermore, alternative design for Nome terminal in Alaska is developed where the S-shape profile is constructed in order to reduce stone size and crest height of the structure (Bruun, 1985).

Meanwhile, in Australia they learned from the lesson where a conventional rubber mound breakwater was damaged. From that experience, the usage of commonly available size of stone with the highest permeability has been introduced.

Then, the rubble mound breakwater is replaced with a berm breakwater where the concept of a mass armored breakwater is defined. At the beginning, the breakwater is built in unstable form, but with sufficient material, there is a designed structure which allowed a movement of the structure until it goes to S-shape stable profile (Bremner et al., 1987).

A wide berm of one stone class was introduced for a wave protection of a runway extension in Unalaska, Alaska where an armor system has been used in order to fully utilize the quarry yield (Hall et al., 1983). In the early stage of the wave attack, the stability of the armor layer can be studied. From some model tests conducted, they showed that the greater the thickness of the armor layer, the smaller the stones needed to be whereby the thickness of the armor layer can be determined by the gradation of the armor stones as well as the incident wave climate (Bruun, 1985).

In 1983, the breakthrough of the berm breakwater was proved when the design of Helguvik breakwater for a tanker terminal close to the Keflavik NATO air base in Iceland was accepted (Baird and Woodrow, 1987). In the design, they used a concept of two stone classes with a wide layer of 1.7 to 7 tonnes stones and quarry run. Baird and Hall (1984) described that the optimization of the use of locally available quarried material as a design basis procedure for the structure. It is noted that high H_{o} corresponds to low stability; meanwhile low H_{o} will resulted to high stability.

As the time passed by, the design of berm breakwaters were gradually being studied which leads to development of dynamics and reshaping breakwaters (Bruun, 1985). These types of structures were considered when the quarry yield prediction is not available in providing the suitable size of stones that need for the designed berm breakwater. Van der Meer and Pilarczyk (1986) state that berm breakwaters or S-shaped breakwater profiles are having stability number, N_s between 3 and 6. The S-shaped berm breakwater was built up from a relatively small stones with a wide size of gradation.

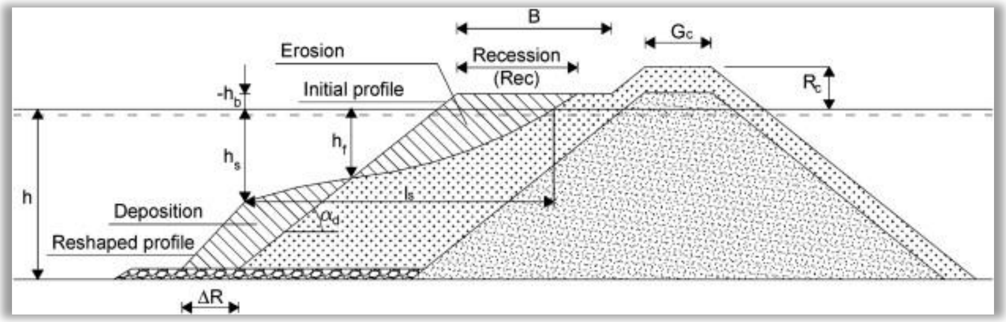


Figure 6 Typical reshaped profile for a berm breakwater (Anderson et.al, 2008)

Figure 6 (Anderson et. al, 2008) above illustrates the typical reshaped profile for a berm breakwater.

The example of dynamic stable reshaped structure is the St. George berm breakwaters constructed in Pribilof Island Chain of Alaska’s Bearing Sea (Gilman, 2002). During the early of the development of the structure, the individual stones did move on along the slope, but by the time passed, the stones were interlocked to each other. This reshaped profile has been due to the gradual settling of the entire mass as the toe has slowly eroded throughout the years (Bruun, 1985).

Berm breakwaters developed more and more in the foreign country where most of the coastal experts shift away from the conventional design breakwater to berm-type breakwater. Uneconomical method to build breakwaters is much preferred among the researchers and engineers since the cost of constructing breakwaters are much higher together with the maintenance needed throughout their operation life. In 1978, the idea of berm-type breakwater has been introduced by Danish Hydraulic Institute. Skopun berm breakwater was built for the protection of Faroe Island(Jensen & Sorensen, 1987). Figure 7(Wikipedia, 2013) below shows the constructed Skopun berm breakwater in Faroe Island.

Figure 7 Skopun Berm Breakwater constructed in Faroe Island (Wikipedia, 2013)

Table 1 (Sigurdarson et al., 2006)presented the numbers of berm breakwater constructed around the world from 80's to 2003.

Table 1 List of berm breakwaters constructed (Sigurdarson et al., 2006)

Country	No. of constructed Berm breakwaters	Year of Construction
Iceland	29	1984
Canada	5	1984
USA	4	1984
Australia	4	1986
Brazil	2	1990
Norway	6	1991
Faroe Island	1	1992
Iran	8	1996
Madeira	1	1996
China	1	1999
India	1	2003
Denmark	1	2003

2.4. Importance of quarry yield prediction

Quarry yield prediction has played an important role in the design phase of harbor breakwater projects in Iceland since the early 1980's (Sigurdarson et al., 2000). The need of quarry selection in the design phase of breakwaters has been proven as a valuable part in order to construct successful breakwater projects. The initial estimation of stone sizes distribution will be used for the preliminary design of breakwater structure and the final design of the structure is tailored to fit the selected quarry. Generally, quarry selection process has been defined as a process aimed to provide rock best suited to the wave condition of the breakwater construction site as well as to minimize the transportation cost and environmental disturbance (Sigurdarson et al., 2006).

Jensen (1984) stated that many projects that Danish Hydraulic Institute (DHI) has been involved need to be modified. This is resulting from the lack of knowledge on the available stone sizes in the quarry that make the modification of the initial design needs to be done after the construction work started. Poor selection of quarry stone sizes available during the preliminary design will cause a modification on the profile in order to fit the actual stone classes available. Hence, this problem will cause the consultant or designer for the breakwater project to alter the design and re-submit the report to the government and this will be a tedious work. Figure 8(West, 2012) below illustrates the quarry selection activities where the experienced contractors are measuring the diameter of the stones provided by the quarry. If the stones have diameter out of the ranges, they will be rejected.

Figure 8 Quarry selection activities (West 2012)

Quarry yields prediction need experienced contractors since the guidelines for drilling and blasting of the armor stones are insufficient. Since dedicated armor stones production is not very common, experienced contractors are very in need in order to handle the requested specifications given by the consultants. A small number of dedicated and experienced contractors in this field havemade the industry to train the contractors to work the quarries for requested specifications. This is aimed to get the contractors familiar with the quarry yield prediction and can rely on their own bids. Smarason (2000) stated that most of the contractors might encounter problem at the beginning of the work that it would be hard to obtain the predicted quarry yield. However, in the end they still manage to obtain the requested specifications through small changes in the blasting design i.e. tilt, burden and spacing of holes as well as the amount of explosives used.

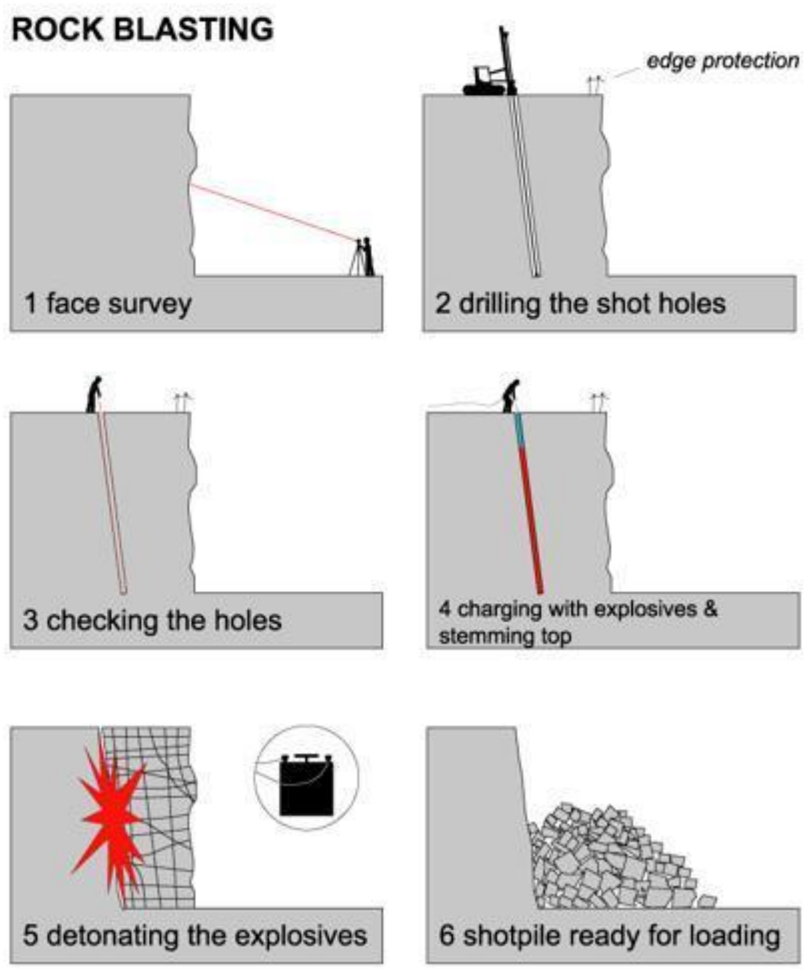


Figure 9 Rock blasting process in quarry (Northstone, n.d.)

Figure 9 (Northstone, n.d.)shows the steps taken during rock blasting where in this process; stones are extracted from the ground.

Blast design is the most important factor in achieving a successful breakwater project. The stones sizes produced from rock blasting are very crucial since it is the deciding factor in securing the desired fragmentation of the rock. In some vital cases, most of the engineers need to adjust the blasting pattern to suit the requested specifications and it is possible for them to adjust the pattern several times within the same quarry to maximize the results obtained.

The process will continue with drilling procedure, crushing and screening. The screening process is also known as sieving where the size designation of the stones or aggregates will be aligned to the grading categories based on the standard given. Figure 10 (Northstone, n.d.)below is the sieve equipment used in most of the quarry in order to have the requested stone sizes.

Figure 10 Screen deck levels (Northstone, n.d.)

Quarry Products:

A stone quarry typically produces the following products:

- Large size of blocks blasted from the quarry face which is approximately $0.5m^3$ (0.36 tonne weight) up to $1.25m^3$ (5-6 tonne weight). This size of stones are called rip rap or rock armor and will be used in coastal and river flood defense in order to shore up sea fronts and river banks.
- The rubble of armor stones drawn from the shot pile is called as a face fill and used as large scales fill in construction sites.
- The fill used in breakwater construction is also called as scalping where the materials will be screened immediately prior to primary crushing.

Advance knowledge on quarry yield prediction as well as the production of armor stones from various quarries has made the designers to design a breakwater with a provision of extra large stones used at the front slope of the structure, typically intend to improve the stability of the slope to defend against severe wave. Smarason (2000) reported that the increment of the stones sizes used at the edge of berm breakwater by a factor of 2 may increase the design wave height to 25%. 25% increment of design wave condition at the construction site requires large stones sizes where the percentage needs is about 2-5% of the total quarried volume.

Unfortunately, the requirement of large machines usage such as large hydraulic excavators and front loaders are crucial in term of the high cost. Even though all the machines are readily available in market, they may raise the cost of the projects by 3-4%. Since most of the company has been urged to construct a very stable breakwater structure, they have utilized extra large stones in order to gain the stability and high strength of breakwater structure. This is not only applicable to high wave load condition but also to lower wave load condition where quarries with low yield size distribution can be used. However, the size distribution provided by the quarry still needs good yield prediction by the experienced contractors. An unprofessional approach by the contractors to this part of the work may cause considerable overproduction in the quarry, which should by no means be rewarded.

Thorough quarries investigation and quality assurance programme have freed the owners from high compensation to the contractors in that area (Sigurdarson et al., 2000). However, if the quarry is being handled without following the recommendations, unforeseen defects may appeared and lead to the overproduction of stones. This will cause some of the substandard armor stones will be rejected and unforeseen fractures zones have been encountered in some quarries.

Quality assurance programme suggested by Smarason (2000) is aimed to identify some properties of the quarried rocks at an early stage. It is important to know the materials including its properties i.e. rock type, density and absorption, strength and resistance to abrasion in abrasive condition. But, however this identification cannot be done by any test or experiment but the personal visual inspection of the experienced contractors or geologists.

2.5. Structural Stability of Breakwaters

Breakwaters are constructed in order to provide protection for the shoreline from dynamic forces of the ocean waves that may lead to coastal erosion or collapse of the upland properties near the beach areas. Therefore, a very stable breakwater is needed to perform the job very well. The conventional design of the breakwater section is normally in trapezoidal shape which is described in more details in the Shore Protection Manual. Generally, the conventional breakwaters are designed in such a way that no damage or only a little damage is allowed on the structure after some periods of wave attacks. The structural stability of breakwaters can be assessing by performing physical model studies to verify the design for critical structures exposed to unfavorable environmental conditions.

The stability of rubble mound breakwaters relies on the armor units as well as the design geometry of the structure.

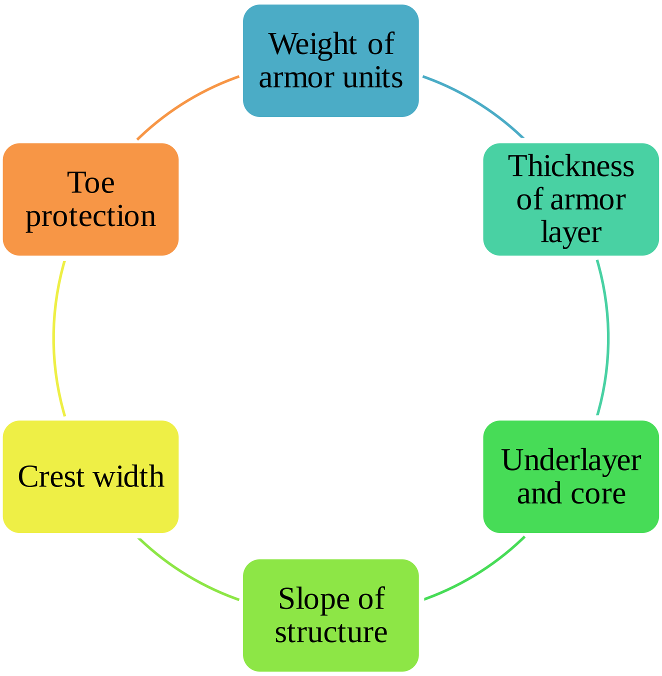


Figure 11 Stability factors of rubble mound breakwaters

2.5.1. Weight of Armor Units

In rubble mound breakwater, the armor units used are armor stones where the weight of the armor stone can be determined by using Hudson formula or Van der Meer formula. Most of the engineers are using Hudson formula since it is much simpler compared to Van der Meer formula.

- **Hudson Formula**

The Hudson formula developed for armor rock was derived from the results of regular wave tests for the armor stability in conditions when the crest of the structure is high enough to prevent overtopping wave. This formula has been widely used because of its simplicity and long period of application. However, this formula does not take account of many factors such as wave period and spectrum, angle of incident wave, shape, type and interlocking of armor units, method of placing the armor units as well as the porosity of the under layer's materials. As per recommendation, this formula should not be used for low crest structure.

The Hudson's equation is commonly used in preliminary design to obtain rough initial estimate of the rock sizes.

$$W = \frac{\gamma_r H^3}{K_D \Delta^3 \cot \theta} \qquad \text{(Eq. 1)}$$

where: W is the design weight of the riprap armor (Newton)

γ_r is the specific weight of the armor blocks (N/m^3)

H is the design wave height at the toe of the structure (m)

K_D is the dimensionless stability coefficient :

K_D = around 3 for natural quarry rock

K_D = around 10 for artificial interlocking concrete blocks

Δ is the dimensionless relative buoyant density of rock

θ is the angle of the slope with the horizontal

However, there are some concerns regarding the effectiveness of using Hudson's equation in determining the stability of rubble mound breakwaters.

- Importance of wave period for the assessment of stability of rubble mound breakwaters

  According to Ahrens (1975),wave period plays important role in the assessment of stability of the riprap. The study of Ahrens (1975) in a large wave tank showed the importance of the wave period on the stability of the riprap. But, however the tests were performed under regular waves. Throughout the experiments, the test data on the stability of dumped quarry stone riprap to wave attack has been analyzed and discussed where the test data showed riprap stability changes with wave period with the lowest stability occurring at a period that creates a collapsing breaker. In addition, Losada and Gimenez-Curto (1980) discussed the importance of the wave period to be considered while evaluating the response of rubble mound breakwater to the action of regular waves. Satisfactory results of the test done were obtained being presented by using interaction curve which defined the stability in a wave height-wave period diagram for a given rubble mound breakwater.
- Importance of permeability of the materials for the assessment of stability of rubble mound breakwaters

  The importance of the permeability of the materials used for the structure has been assessed by Hedar (1960) which is also performed under regular wave attacks. Results obtained from the experiments done showed that fine core material reduces the stability of the structure considerably meanwhile the failure developed gradually than in case of the core material. He concluded that a large sensitivity make it very important to scale the core permeability in the model with respect to the porous flow.

- **Van der Meer Formula**

The Van der Meer formulae were established from the results of a series of model tests using irregular waves which better reflect the real conditions of the sea state. The formula is based on the wide set of model data and can be considered as most widely applicable of the prediction method available. The Van der Meer formulaeare more complex compared to Hudson's equation since they take into account the following variables:

- Wave period
- Breaker parameter
- Duration of storm
- Permeability of the core of the structure
- Damage level
- Breaking wave conditions

These formulae are describes as practical design formulae for armor rock where the formulae are given as follows:

For plunging waves:

$$\frac{H}{\Delta D_{n50}}\sqrt{\xi_m} = 6.2P^{0.18}\ (\frac{S}{\sqrt{N}})^{0.2} \qquad \text{(Eq. 2)}$$

For surging waves:

$$\frac{H}{\Delta D_{n50}} = 1.0P^{-0.13}(\frac{S}{\sqrt{N}})^{0.2}(\sqrt{\cot\alpha}\ \xi_m^P \qquad \text{(Eq. 3)}$$

where: H is the design wave height, taken as significant wave height (m)
D_{n50} is the nominal rock diameter equivalent to that of a cube (m)
$\Delta$is the dimensionless relative mass density of the armor rock
$P$is the notional permeability factor
$\alpha$is slope angle of the structure
$N$is the number of waves

$S$is the damage level defines by $S = \frac{A}{D_{n50}^2}$

$\xi_m^P$is the surf similarity parameter for mean wave period defines by:

$\xi_m^P = \frac{(\tan\alpha)}{\sqrt{s_m}}$

These formulae are said to be more complex because all the factors considered in the equations need to follow the given conditions of the tests run.

2.5.2. Thickness of Armor Layer

The thickness of the armor layert_a can be obtained from the following formula:

$$t_a = nk_\Delta (\frac{W_a}{\gamma_a})^{\frac{1}{3}} \quad \text{(Eq. 4)}$$

where: $W_a$is the weight of an individual armor unit (N)

$n$is the number of armor layers

$k_\Delta$is the layer thickness coefficient

$\gamma_a$is the unit weight of armor unit (N/m^3)

Normally, the thickness of randomly placed of armor rocks are designed to contain a double layer of rocks ($= 2$) with a provision of layer thickness coefficient equal to 1.15 and the volumetric porosity of 0.37. Meanwhile, for concrete armor units, the two layers of units are normally provided but most of the cases, the method of placing should based on the careful testing or as recommended by the originator of concrete units. To obtain best design of breakwater structure, the armor layer should extend below the lowest design water level to a depth equal to 2 multiplywith $H_{\frac{1}{3}}$.

According to (Palmer & Christion, 1998)

> The stability of the armor structure is gradually increased with an increase in armor layer thickness. This is because the thickness of the armor layer will be used to allow for any settlement in future so that sufficient of the armor layer thickness will tolerate settlement without breakage. Allowance for the initial settlement to occur is made as the units nest into a more stable position under wave action.

2.5.3. Underlayer and core

Consideration on the weight of under layer rock is normally taken as not less than one tenth of the weight of the armor. The size of the individual under layer rock should be in the ranges of $\pm$30% of the nominal weight selected for the design which is obtained by using Hudson's equation or Van der Meer formulae. This is only applicable when the armor layer is made up of rock.

Some considerations that need to be taking into account are:

- No filter layer should contain more than 5% of material by weight passing 63µmsieve and that fraction should be cohesionless.
- Filter material should be well graded within the specified limits and its grading curve should have approximately the same shape as the grading curve of the protected material.
- Where the retained fill material contains large proportions of gravel or coarser material, the filter should be designed on the basis of the grading of that proportion of the protected material finer than 20mm sieve.
- The thickness of filter layers should be ample to ensure integrity of the filter when placed underwater. In practice, the thickness of filter layer at 1m below and 0.5m above water level should be the minimum thickness.
- The filters should cover the full depth of the structure.

2.5.4. Slope of the structure

The slope angle of the structure depends on the hydraulics and geotechnical stability and should generally be not steeper than 1 (vertical): 1.5 (horizontal). The slope of the structure may influence the amount of interaction between armor units. The contribution of the angle of the slope as well as the interlocking of the material used to the stability of the structure is cause by the friction and the increment of the slope parallel force that applied by the adjacent units.

2.5.5. Crest width

The crest width of the structure should be sufficient enough to accommodate any construction planned in future, operation as well as the maintenance activities on the structure. For the rubble mound breakwaters, the minimum crest width of, B should be sufficient to accommodate at least three crest armor units. Generally, the crest width of the structure is provided in order to provide access for the machines to do some

maintenance in future or it can be used for the backhoe or excavator during the construction of the breakwater.
According to (Seabrook & Hall, n.d.)

> From the physical model studies performed at the Coastal Engineering Research Laboratory in Queen's University, it was shown that the incident wave height and the structure crest width are the most important design variable for breakwater projects. The observation made from the results obtained indicates that the transmission coefficient K_t is most sensitive to the depth of submergence, d_s, the incident wave height, H_i, and the crest width, B.

2.5.6. Toe protection

The wave action occurs in front of the structure can cause severe turbulence at the seabed. In particular, the toe of the structure can be exposed to the action of breaking waves in shallow water that may leads to erosion of the seabed material and scouring at the toe structure.

2.6. Parameters Affecting Stability of Berm Breakwaters

Berm breakwater is said to be more stable than the conventional design since they allowed the movement of the stones up and down the slope till it achieves the stable profile. In designing the dynamically stable berm breakwater, they used smaller size of stones rather than large and heavy rock. This is because a structure with smaller armor unit is much more economic compared to the conventional design. Once they reached their equilibrium state, severe series of waves can be resisted and this dynamically stable reshaped breakwater is characterized by the wave height parameter ($\frac{H}{\Delta D} \geq 6$). The height parameter for S-shaped breakwater profile is in the ranges of 3 to 6 (Van der Meer, 1982)

Generally, the reshaping of the dynamically stable berm breakwaters is dependent on the size and gradation of the armor stone and also the wave characteristics. Since the parameters chosen for this experiment are size gradation of stones and also the wave characteristics, initial configuration of the dynamically stable berm breakwater is needed to be taken care.The reshaping of dynamically stable berm breakwater has been studied where it was subjected to various parameters such as the gradation of the armor stones, wave characteristics and also the wave period that applied to the structure (Kao & Hall, n.d.). Many tests were conducted in order to investigate the damage process of the berm-type breakwater.

Kao &Hall (n.d) stated that the results obtained from series of tests conducted in 2m wide of wave flume shown that the uniformity of stones which is stone size gradation and also the duration of storm gave significant impact on the damage process of the dynamically stable berm breakwater.

Another series of test conducted shown that the behavior of the berm-type breakwater was influenced by the stone size gradation and also the steepness of the initial slope. These factors have impacts on the reshaping mechanism (Merli, Bos, Roelvink, & Uijttewaal, 2013). The design of the berm-breakwater mainly involved the provision of wide berm at or around the water level and the design used the smaller size of armor stones, which are allowed to reshape till the equilibrium slope is achieved (Rao, Pramod, & Rao, 2003).

Hence, the main parameter considered is discussed below:

- Size gradation of armor stone

  All the results obtained from the previous series tests showed that this parameter has the greatest influence on the stability of the berm-type breakwater. Different armor sizes will give a different stability number of the structure. Hudson (1959) described that stability number as the mobility of the single elements on any given rock slope where:

$$N_s = H_s \,/\triangle\, D_{n50} \qquad \text{(Eq. 5)}$$

H_s = Significant wave height

$\triangle$ = Relatives density of stones

D_{n50} = Mediannominaldiameter

Another dimensionless wave load parameter is the wave period which defined as:

$$H_\circ T_\circ = N_s \times \sqrt{\frac{g}{D_{n50}}}T_m \qquad \text{(Eq. 6)}$$

where: g = acceleration of gravity

T_m = mean wave period

The stability of the slope will decrease as Ns increases. Meanwhile the stone size gradation is defined by the following ratio:

$$\mathbf{G} = \frac{\mathbf{D_{85}}}{\mathbf{D_{15}}} \qquad \text{(Eq. 7)}$$

D_{85} is defined as 85% of the sample does not exceed the size meanwhile D_{15} is defined as 15% of the samples is not exceeding the size (Merli, Bos, Roelvink, & Uijttewaal, 2013).

- Wave Characteristics

  It is observed from the result of berm recession experiment that the wave height has a very significance influence on the stability of berm breakwater, as the wave height with the same wave period increases, the berm recession will increase (Shekari & Shafieefar, 2013).

  This is because higher wave height will make the stones move up and down abrasively which reduce the interlocking between each stone.

Shekari and Shafieefar (2013) concluded that wave height has a great influence on the stability of berm breakwater. It can be seen from the graph illustrated below:

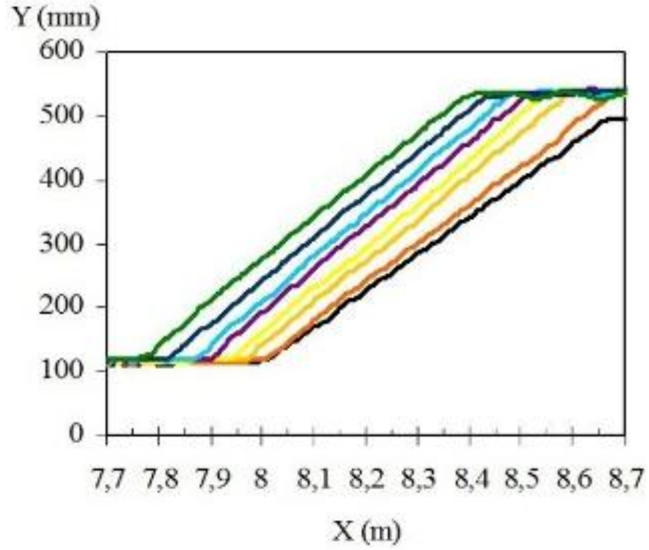


Figure 12 Influence of wave height on reshaped profile

CHAPTER 3

METHODOLOGY

Basically, there are few procedures that should be conducted in order to accomplish the objective of the study which are:

- Review of past research studies
- Determination of dependent and independent parameters studied
- Physical model tests are carried out
- Analysis and presentation of results
- Conclusion

3.1 Procedures

Procedures conducted throughout the research studies are as follows:

Review of past research studies on the existing breakwater concept

↓

Determination of dependent and independent parameters studied

↓

Design breakwater cross section and median nominal weight of armor stones by using Hudson's equation

↓

Determine the test ranges/condition for the selected dependent parameters

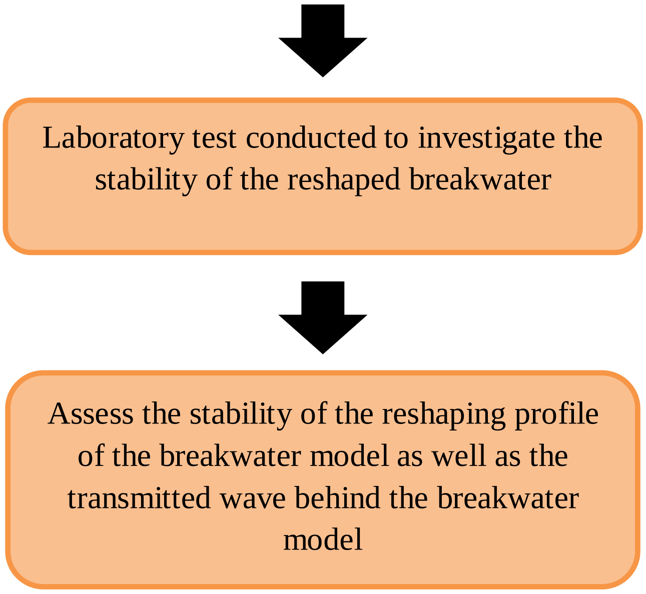


Figure 13 Procedures conducted

3.2 Review of Past Research Studies

Research methodology is the first stage that should be done at the early phase of studies. In this stage, a full attention is needed in order to get a better understanding on the project theoretically and technically. It involves of a review of journal, reports, and research papers and books so that a clear view of the study can be obtained. In this study, the main resources used are the Coastal Engineering Manual, the research papers conducted in a few years back, and also the journals and articles related to berm-type breakwaters. The appropriate research and background study will lead to better understanding and good analytical regarding the project.

Subsequently Gantt chart is constructed with the respective key milestone to ensure the timely completion of the study.

3.3 Project Activities

3.3.1 Site Visit

The author has conducted site visits in two places; Jetty Pantai Siring and PantaiPengkalanBalak, Malacca. The site visit was aimed to give better understanding to the authors on the performance of breakwaters as erosion control structure. In addition, site visit conducted was intended to expose the author about the potential causes of erosion problem.

- **Jetty Pantai Siring, Malacca**

Jetty Pantai Siring is located in Merlimau area where it famous with the fisherman activities. The soil texture of this area is muddy sand with the presence of mangrove area. However, the mangrove areas are being rapidly depleted due to the pressures from growing populations in the coastal areas. Depletion of mangrove trees will promote erosion problem since the main function of mangrove is to attenuate waves (reduce wave energy) by obstructing the wave with its roots and trunks*(Othman, 1991)*. The closer the trees are together, the greater will be the attenuation of wave energy. Erosion of the muddy area commonly starts with the lowering of the mud-flats in front of the mangrove. This caused the roots of the mangrove fringing the sea to be exposed. Eventually these trees will collapse and the erosion will continue further into mangrove belt. Figure below shows the erosion of the muddy coast near the jetty area.

Jetty and Breakwater

Breakwaters have traditionally been used for harbour protection and navigational purposes where they are known as wave energy barriers designed in order to protect the land or near shore area behind them from direct attack of waves. Fundamentally, calm condition is required for the jetty area to keep operate especially for the jetty whose exposed to the open sea. The operations at the jetty are highly dependent on the sea state and have to be suspended whenever the wave or wind conditions are unfavourable. Hence, the construction of segmented breakwaters is encouraged to provide protection over longer sections of the shoreline as well to withstand any wave condition.

Figure 14 Jetty Pantai Siring

A pair of segmented breakwaters is constructed at Pantai Siring so that the wave energy transmitted to the shoreline is reduced. Few factors that need to be considered while designing breakwaters are:

- Environmental conditions
- Navigation requirements
- Availability of the construction material
- Layout of breakwaters

Figure 15 Segmented breakwater constructed at Jetty Pantai Siring

The design cross section of the breakwater constructed is shown below. The breakwater was build up from several size graded layers where it consists of primary and secondary layers. The secondary layer of the breakwater was build up mainly from smaller stone size gradation meanwhile the primary layer is made up from bigger stone sizes. This was aimed to provide stability of the breakwater.

Figure 16 (Diyana, 2013) below illustrates the design cross section of breakwater constructed at Jetty Pantai Siring, Malacca.

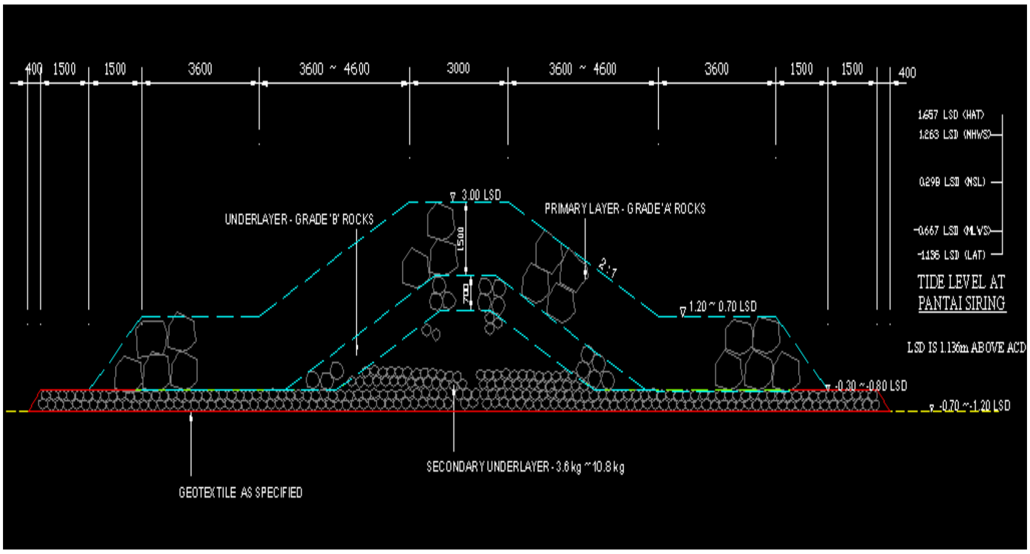


Figure 16 Design Cross Section of the breakwater (Diyana, 2013)

- **KompleksPerahuLayar, PengkalanBalak, Malacca**

KompleksPerahuLayar located at PengkalanBalak is reported under threat of severe erosion due to high wave attack at night. Malacca Coastal Management together with the Drainage and Irrigation Department has taken an action to prevent the erosion problem especially at the most critical area, KompleksPerahuLayar Block.

Breakwater

Breakwater design for KompleksPerahuLayar is based on the hydrodynamic studies on the specific area. The breakwater is shore-connected with two long arms of breakwater namely minor and major shore-connected breakwater. The design water depth at the head of the breakwater is 3m with the crest height of 3m LSD. The length of the longer arm is 280m meanwhile the shorter arm is about 90m length. The stone size gradation used for the breakwater design is shown in the Table 2 below. Both head and trunk section of the breakwater have the same stone size used.

Table 2 Stones sizes used for breakwater design

TYPE OF ROCKS	MAX.	NOM.	MIN.	UNIT
GRADE 'A' ROCKS	2077	1662	1246	(kg)
	1100	1010	950	(mm)
GRADE 'B' ROCKS	208	166	125	(kg)
	500	470	420	(mm)

The stone size gradation used for groins constructed near the breakwaters is given in the Table 3 below.

Table 3 Stones sizes used for groins

TYPE OF ROCKS	MAX.	NOM.	MIN.	UNIT
GRADE 'A' ROCKS	653	670	392	(kg)
	720	670	600	(mm)
GRADE 'B' ROCKS	65	52	39	(kg)
	350	310	280	(mm)

3.4 Tool Used

In this project, the tools we need to carry out the experimental works are:

i. Model breakwater
ii. Wave Flume

The model breakwater has been constructed and placed in the wave flume tank located at in Coastal Laboratory of Civil Engineering Department in UniversitiTeknologi PETRONAS. The facility provided in the Coastal Engineering laboratory is equipped with programmable wave generator for wave flume of 23m x 1.5m x 1.5m dimensions. The wave is produced by using single wave paddle.

3.5 Experimental Set up

The experiments were carried outwith varying wave conditions where the gradation and shape of the armor stones used as well as the design cross section of the breakwater model have been initially set as independent parameters.The physical breakwater model was designed for a wave height, H = 0.2m. The water depth that will be used in this experiment was fixed to 0.5m, meanwhile regular wave spectrum were used in all test conditions. Wave probes were placed before and after the structure in order to measure the incident wave height as well as the transmitted wave height behind the breakwater model. The positions of the probes used are depending on the wave period of wave attacks (Mansard & Funke, 1980). Table below shows the probes position in the wave flume.

Table 4 Probes position in the wave flume

						Condition			
Wave period (s)	**Frequency (Hz)**	**L**	**Wave Gen. to WP1**	**WP1 - WP2**	**WP1 - WP3**	**L/5**	**L/3**	**L/6**	**3L/10**
1	1	1.52	2.5	0.15	0.3	0.3	0.51	0.25	0.45
1.2	0.83	2.07	2.5	0.21	0.55	0.41	0.69	0.34	0.62
1.4	0.71	2.6	2.5	0.26	0.6	0.52	0.87	0.43	0.78
1.5	0.67	2.86	2.5	0.29	0.7	0.57	0.95	0.48	0.86
1.6	0.63	2.99	2.5	0.3	0.75	0.6	1	0.5	0.9

3.6 Physical Model Set up

The physical breakwater model was designed with the following value of parameters:

- Mass density of armor stones = 2650 kg/m^3
- Dimensionless stability coefficient, K_D = 2
- Angle of the slopes for the breakwater = 1(H) :1.5 (V)

The primary armor weight, W was determined by using Hudson's equation which is 1758 g with a provision of primary armor layer thickness of 0.2 m and the under layer thickness of 0.4 m. The crest width provided was 0.5m. The size distribution of the stones used in the physical breakwater model was determined in terms of the diameter where three sizes of stones have been used:

- D_{20} = 0.055 m
- D_{50} = 0.088m
- D_{80} = 0.10 m

The underlayer material was placed first to the required level and next the primary layer was constructed based on the breakwater design cross section. Figure 17 below illustrates the physical breakwater model constructed in the wave flume.

Figure 17 Physical model of breakwater

3.7 Determination of test ranges used

The capacitance type wave probes along with the amplification units were used for acquiring the data. The probes were used during the experimental works, for acquiring incident wave height as well as the reflection envelope.

The experiments were conducted for a configuration of model shown below:

Figure 18 Design cross section of physical breakwater model

Test conditions for the experiments are given in the table below:

Table 5 Wave design parameters

						Steepness, h/L		
Wave period (s)	**Frequency (Hz)**	**$L_{\circ}$**	**$d/L_{\circ}$**	**d/L**	**L**	**0.04**	**0.06**	**0.08**
1	1	1.56	0.32	0.33	1.52	0.06	0.09	0.12
1.2	0.83	2.25	0.22	0.242	2.07	0.08	0.12	0.17
1.4	0.71	3.06	0.16	0.192	2.6	0.1	0.16	0.21
1.5	0.67	3.51	0.14	0.175	2.86	0.11	0.17	0.23
1.6	0.63	4	0.13	0.167	2.99	0.12	0.18	0.24

3.8 Gantt chart

Table 6 Gantt chart

Phase	FYP 1														FYP 2														
Week	1	2	3	4	5	6	7	8	9	10	11	12	13	14	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Research study on dynamically berm breakwater			●																			●							
Analysing and comparison between the existing breakwater						●																							
Determination of dependent and independent parameters									●																				
Design cross section of breakwater model														●															
Experiments conducted (Experimental work)																													
Analysing of performance for the design concept of breakwater																									●				
Report preparation																											●	●	

● Key Milestone

3.9 Key Milestones

Table provided below is the key milestones that need to be done throughout two semesters in order to accomplish the objective of the study.

3.9.1 Final Year Project I (Semester 1)

Table 7 Key milestone for FYP I

Milestone	Week
Project Proposal	Week 3
Extended proposal (10%)	Week 6
Proposal Defense (40%)	Week 9
Interim Report (50%)	Week 14

3.9.2 Final Year Project II (Semester 2)

Table 8 Key milestone for FYP II

Milestone	Week
Progress Report (10%)	Week 8
Pre-SEDEX (10%)	Week 11
Technical Report (10%)	Week 13
VIVA (30%)	Week 14
Dissertation (40%)	Week 15

CHAPTER 4

RESULT &DISCUSSION

4.1 Result

The finding from this studycomprised of two sections which are deformation of reshaping structure as well as the measurement of transmitted wave behind the breakwater model. This has been achieved by constructing the dynamically stable breakwater by using several stone classes where the initial configuration of the model test is referred to the previous test conducted for dynamically berm breakwater as well as taking into consideration the practical aspects which make the construction simpler. The wave height was varied throughout the test where the maximum design wave height can be produced by the wave generator is 0.28m.

The expected result for this test can be summarized as the reshaped profile of the berm breakwater for different wave heights and the wave period. Figure 19 below illustrates the deformation of the reshaped profile after some period of wave attacks.

a) Deformation of reshaping structure

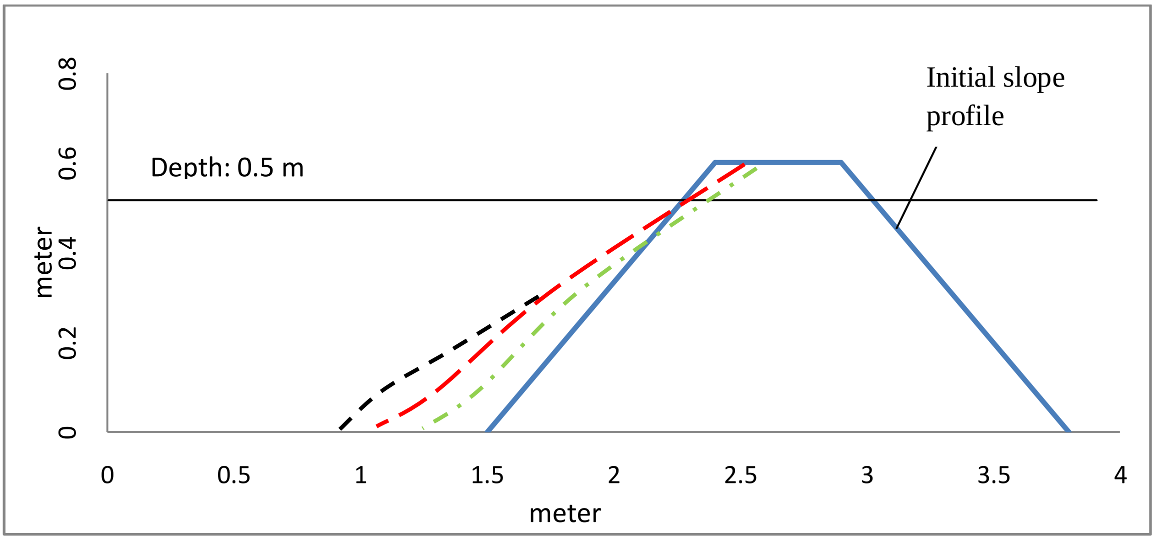
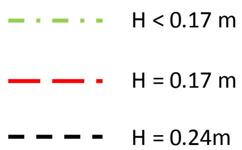


Figure 19 Deformation of the reshaping slope



The graph above illustrates the reshaping profile after wave attack on the model. From the observation made during the experiments run, there is no significant stone movement when small wave is applied. The stones start to move significantly during the wave height is 0.17m. The final reshaping of the slope is when the wave height is equal to 0.24m.

The reshaping profile obtained its stability when the front slope reached the berm-shaped profile. This is because the concept of mass armored breakwater is defined as rubble mound structure designed and built in an initially unstable form, but with the provision of sufficient sizes of material used at the front and top of the structure, they allow natural forces to modify its shape to a stable profile. The stability of the reshaping profile is much depends on the interlocking of the stones used on top of and at the front of the slope. Icelandic berm breakwater concept has developed over the years to design berm breakwater build up from several stone classes where the stability of the structure is gained from the placement of large stones at the top and front slope of the structure (Andersen et al., 2008).

Generally, the stability and reshaping of the profile can be determined by using the parameters as follows:

$$N_s = \frac{H_s}{\Delta D_{n,50}} \quad \text{(Eq.8)}$$

$$H_0 T_0 = N_s \times \sqrt{\frac{g}{D_{n,50}}} T_m \quad \text{(Eq. 9)}$$

$H_0 T_0$of the reshaping slope obtained from the experimentwas calculated to be 20.23. Hence, the final profile acquired can be described as a reshaping profile since a wide range of rubble mound structure lies in complete description of the reshaped profile within the ranges of 3 $< H_0 T_0 <$500.

b) Transmission coefficient

Summary of the transmission coefficients for three types of steepness is given in the Table 9 and presented graphically in Figure 20.

Table 9 Transmission coefficients for Regular Wave

Frequency	Steepness, H/L = 0.04	Steepness, H/L = 0.06	Steepness, H/L = 0.08
	Transmission Coefficient	Transmission Coefficient	Transmission Coefficient
1.0	0.0211	0.0400	0.2090
0.83	0.0952	0.1090	0.2520
0.71	0.0508	0.0512	0.0520
0.67	0.7391	1.0209	1.2171
0.63	0.8142	1.0567	1.3065

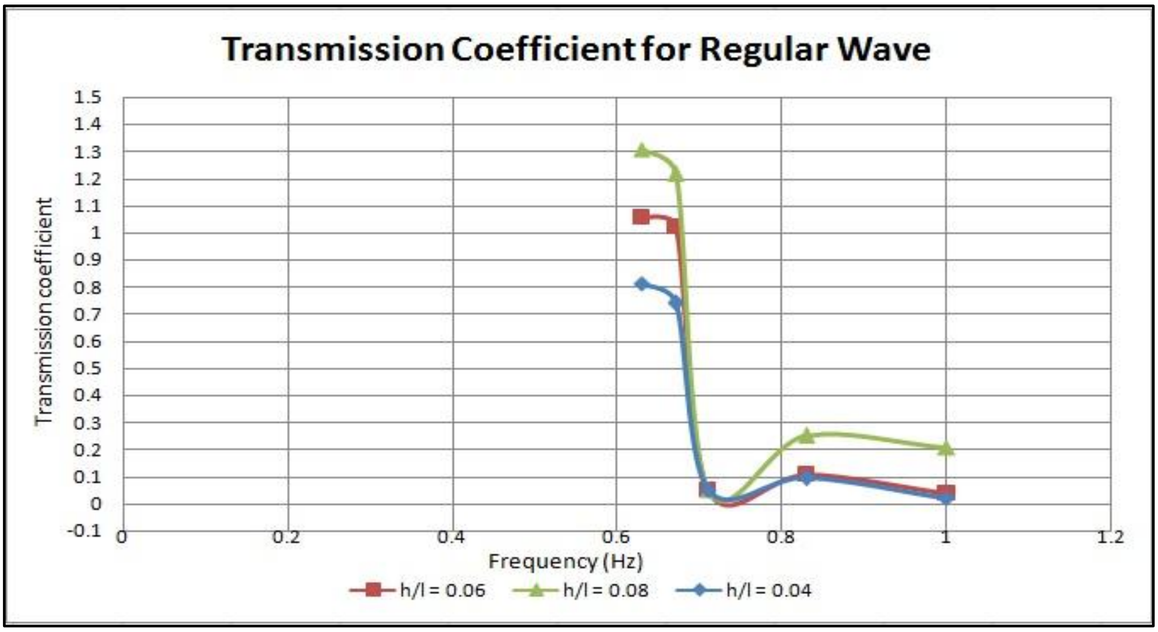


Figure 20 Transmission coefficients under Regular Wave

Transmission coefficient is used in order to show how well the wave energy transmitted through the breakwater model. From the graph shown above, it shows that the higher the wave height applied on the breakwater model, the higher the transmission coefficient. The lowest frequency used in the experiments which is f=0.63Hz resulted highest transmission coefficient with a provision of highest steepness, H/L = 0.08.

CHAPTER 5
CONCLUSION

The reshaping of berm breakwaters due to the incident wave height can be considered as a major concern since it contributes to a very significance impact to the stability of the berm breakwater. This is to identify at which height of the wave, the reshaped profile will attain its stability. The concept of voluminous mass armored structure or conventionally called as dynamically stable rubble mound breakwater with a provision of the placement of large stones at the top as well as at the front slope of the structure pointed out how the permeability of the material (caused by the grading) may enhance the dissipation of the wave energy.

According to the graph of deformation of reshaped profile presented, the dynamically stable breakwater allowed movement of the stones in order to obtain the stability of the slope. The stability of the slope achieved once the berm-shaped profile is obtained after some period of wave attacks. In this experiment, the front slope of the physical breakwater model shows significant stones movement at 0.17m wave height and finally reached the final reshaping profile when the wave height is 0.24m.

The transmission coefficient vs. frequency graph shows that the highest steepness, H/L = 0.08 and highest frequency produced highest value of transmission coefficient. The higher transmission coefficient at submerged breakwater can be relatively defined as the effect of the overtopping wave. As per discussed by Seabrook and Hall (n.d.), the overtopping rate of the structure can be described as the function of wave steepness and structure geometry. The overtopping wave effect for this experiment was represented by the structure crest width and the wave height where insufficient crest width for the structure has led to the wave overtopping to occur.

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APPENDIXES

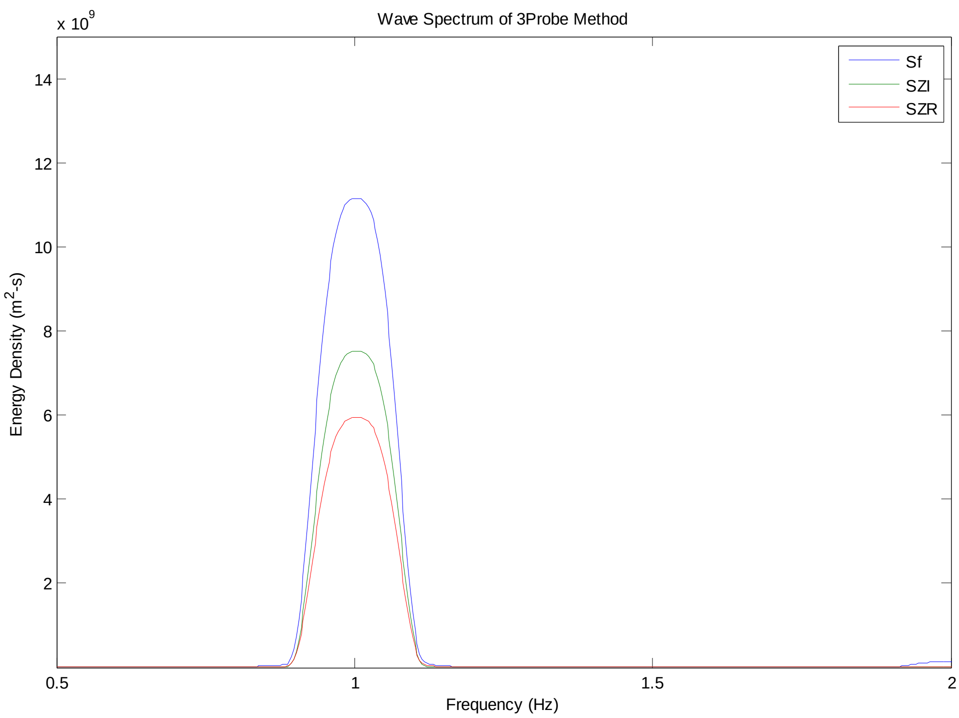
Appendix 1: Wave Spectrum Density

- **Wave spectrum density for incident and reflected wave under regular wave condition**

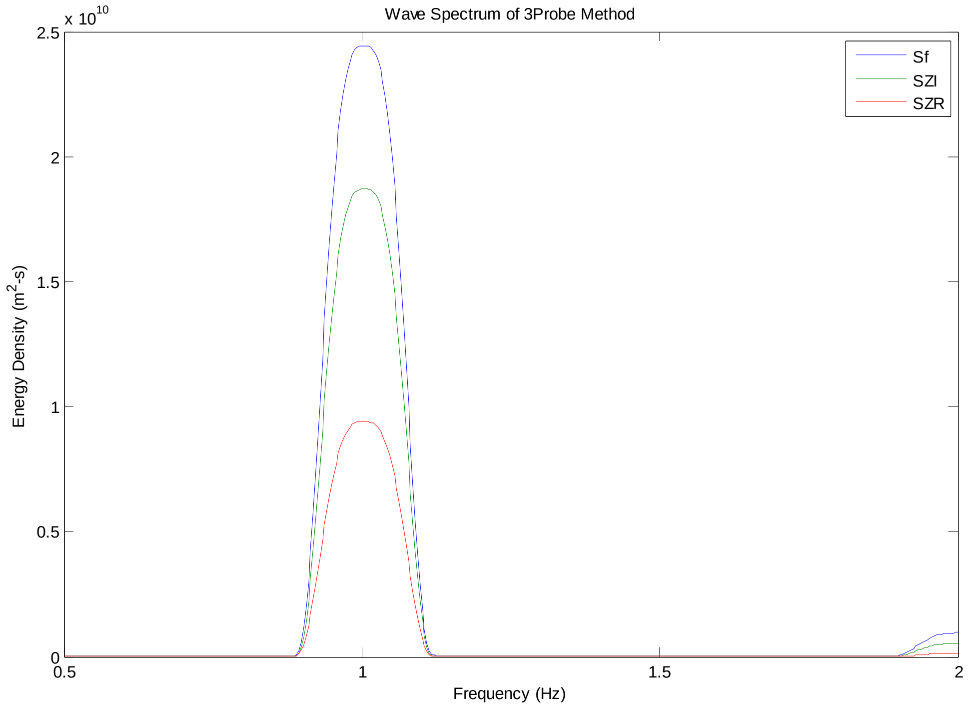
Figures below illustrate the wave spectrum density obtained for the incident and reflected wave under regular wave condition. The legends shown in the graph are consisting of:

- SZI : Incident wave
- SZR : Reflected wave

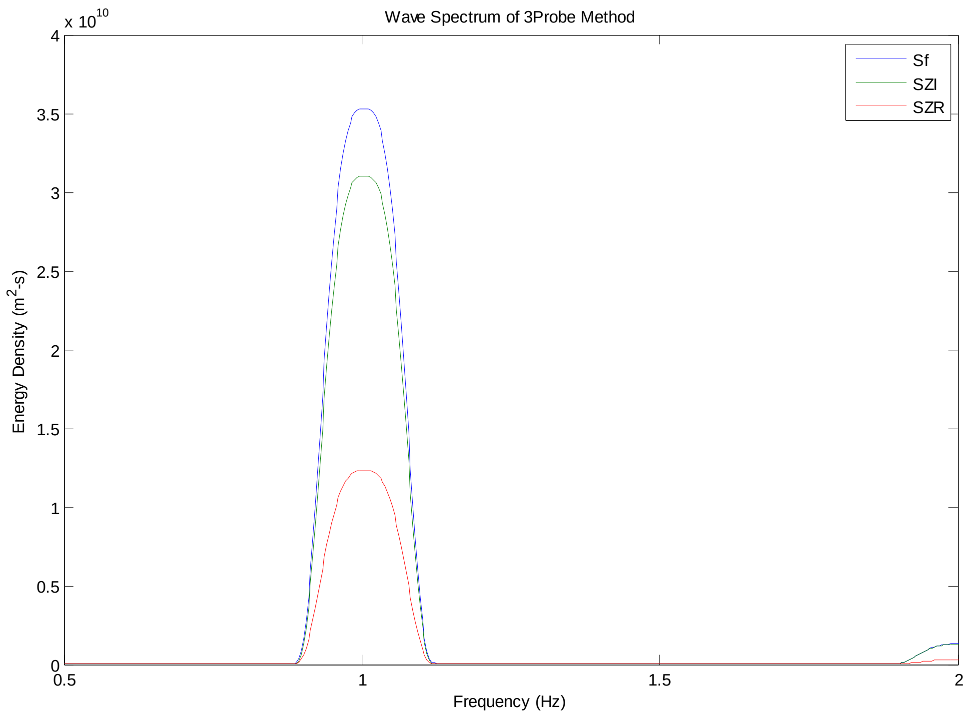
i. Regular wave of wave period = 1.0sec and steepness, H/L = 0.04



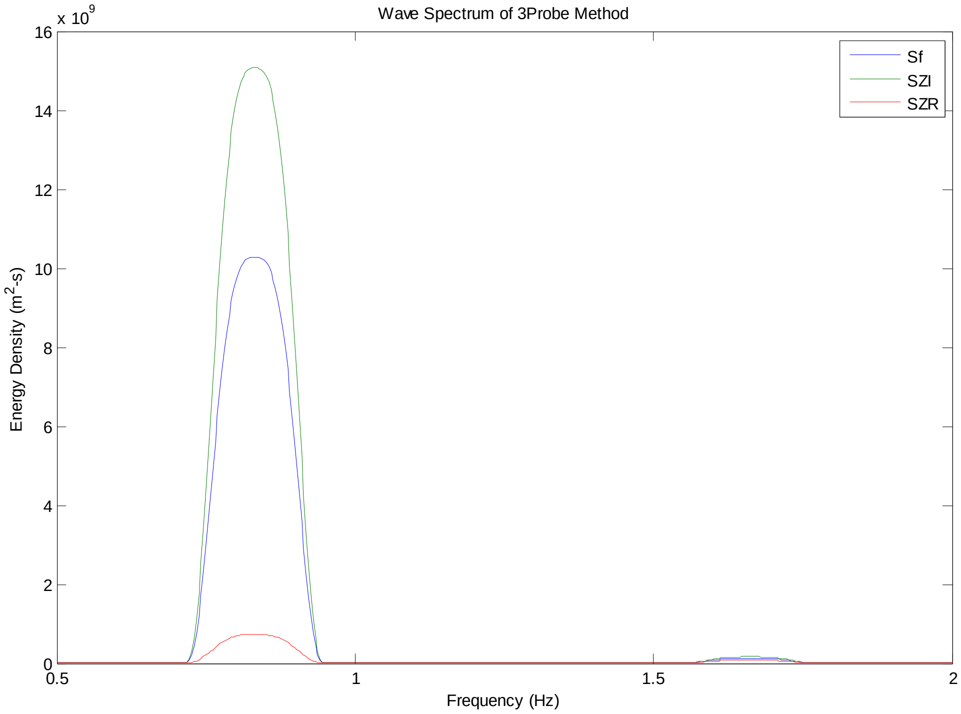
ii. Regular wave of wave period = 1.0sec and steepness, H/L = 0.06



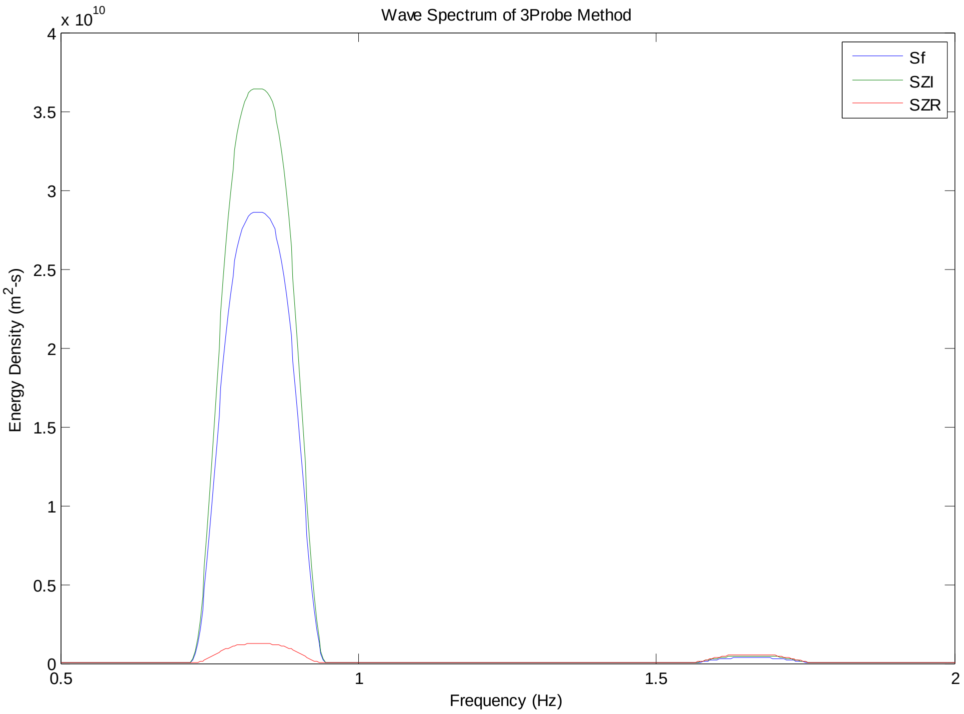
iii. Regular wave of wave period = 1.0sec and steepness, H/L = 0.08



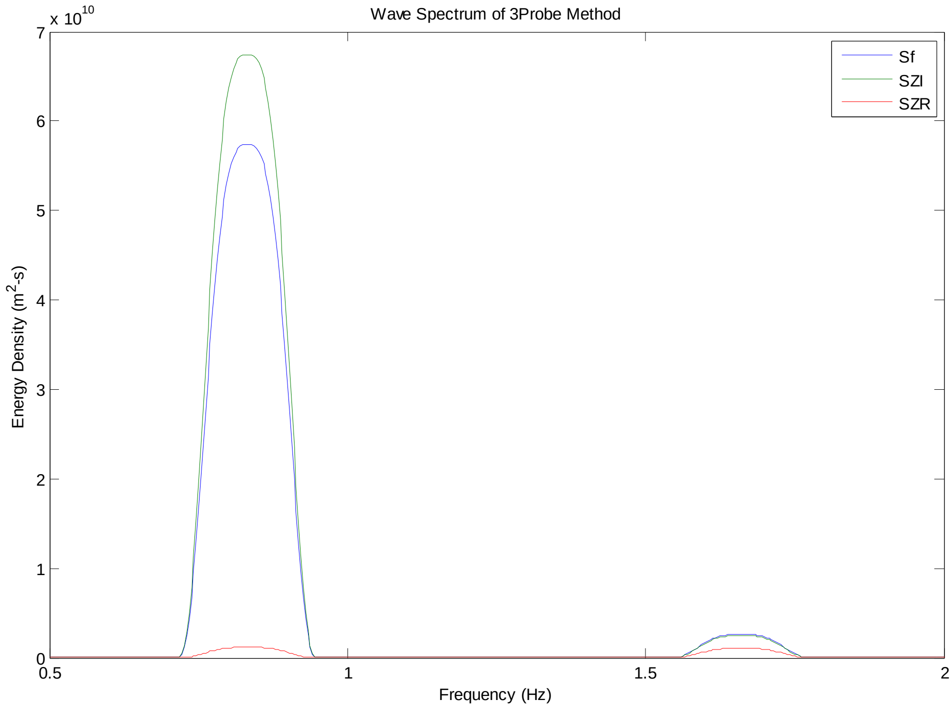
iv. Regular wave of wave period = 1.2sec and steepness, H/L = 0.04



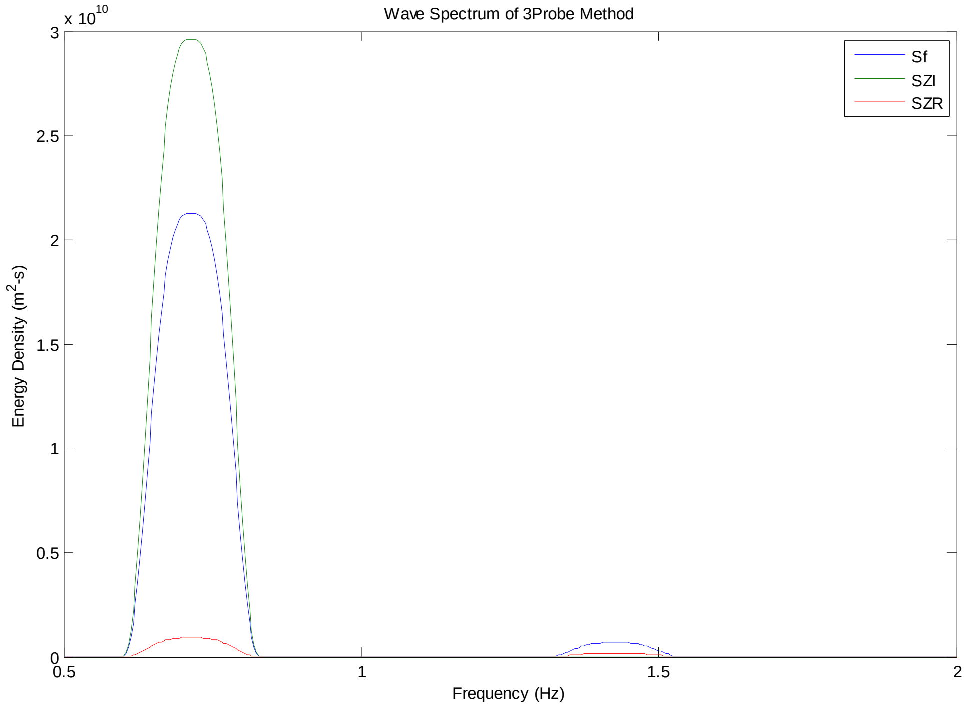
v. Regular wave of wave period = 1.2sec and steepness, H/L = 0.06



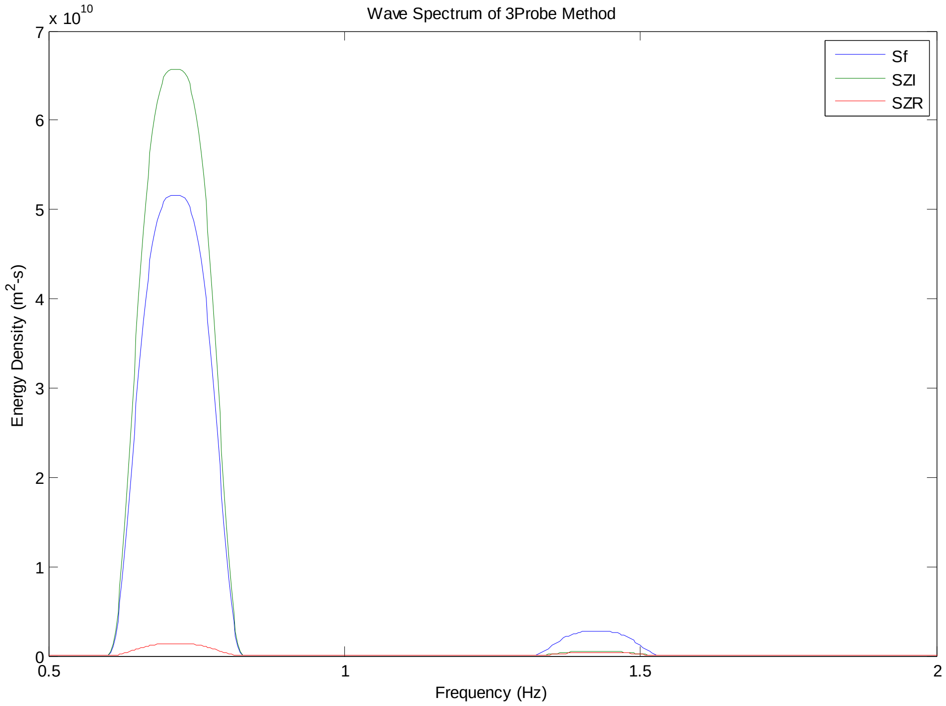
vi. Regular wave of wave period = 1.2sec and steepness, H/L = 0.08



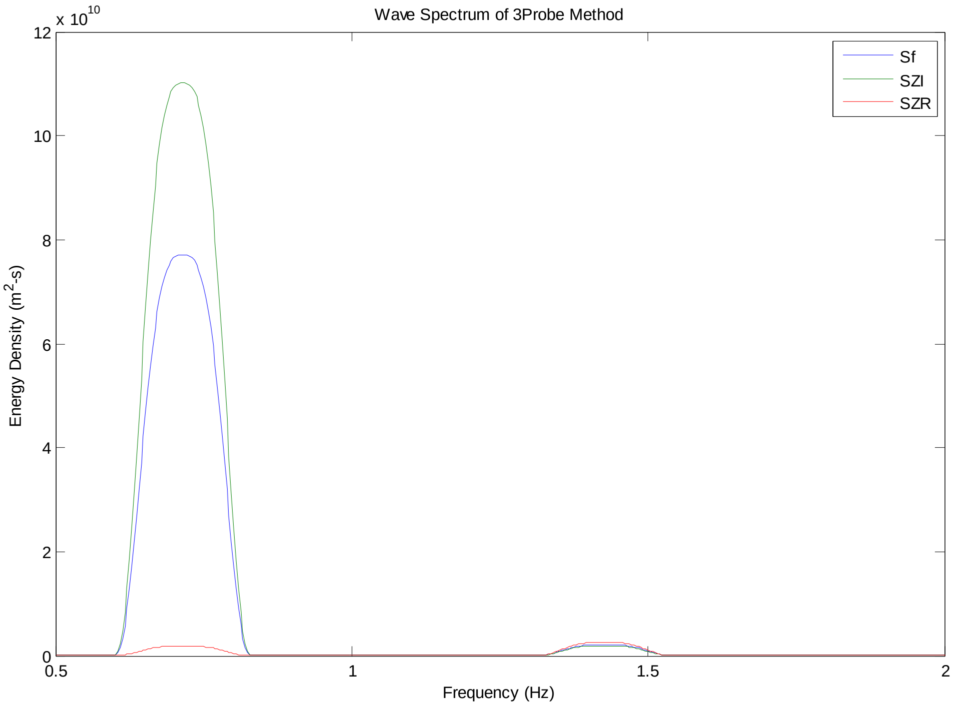
vii. Regular wave of wave period = 1.4sec and steepness, H/L = 0.04



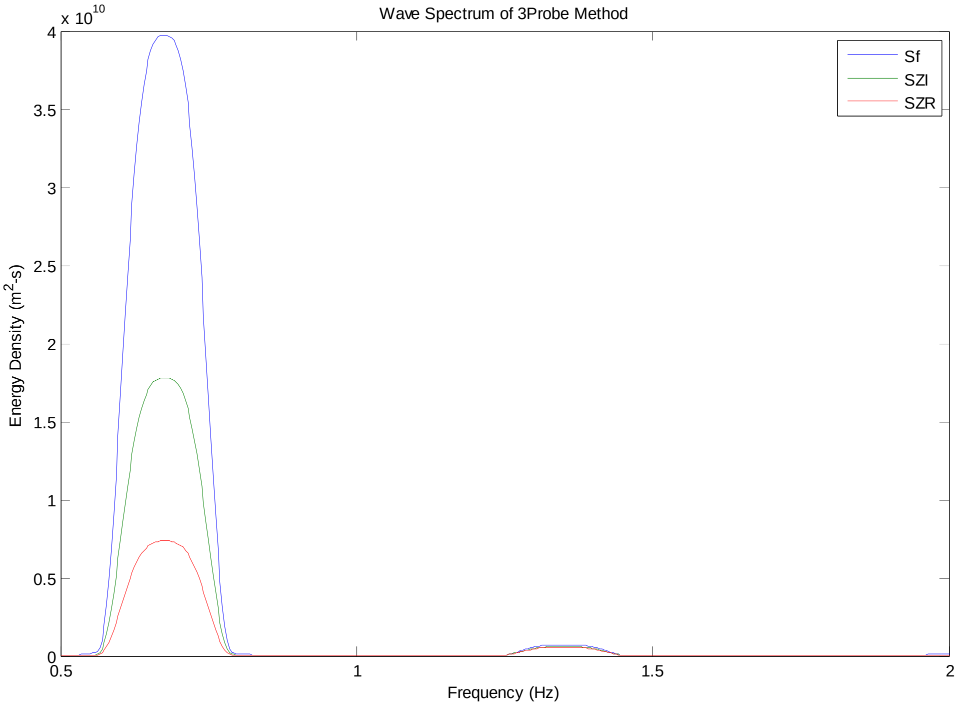
viii. Regular wave of wave period = 1.4sec and steepness, H/L = 0.06



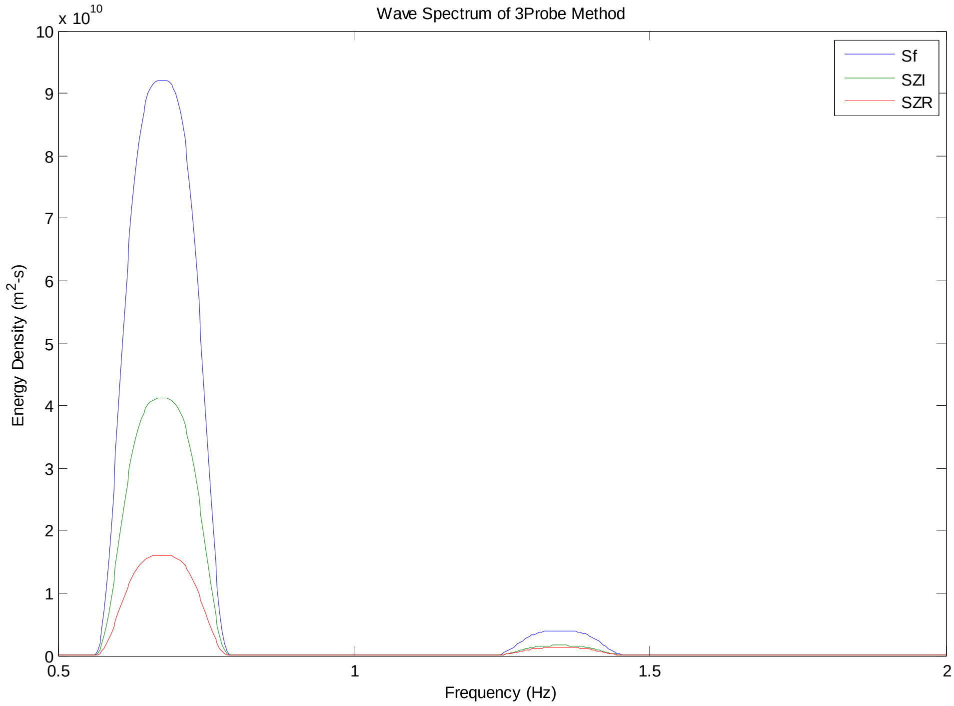
ix. Regular wave of wave period = 1.4sec and steepness, H/L = 0.08



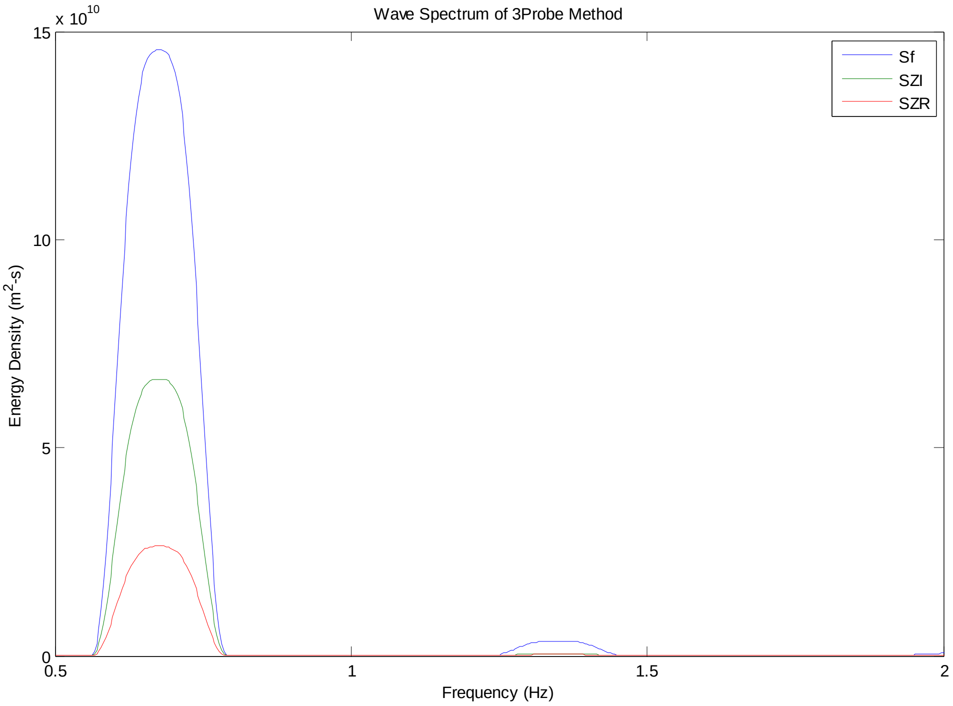
x. Regular wave of wave period = 1.5sec and steepness, H/L = 0.04



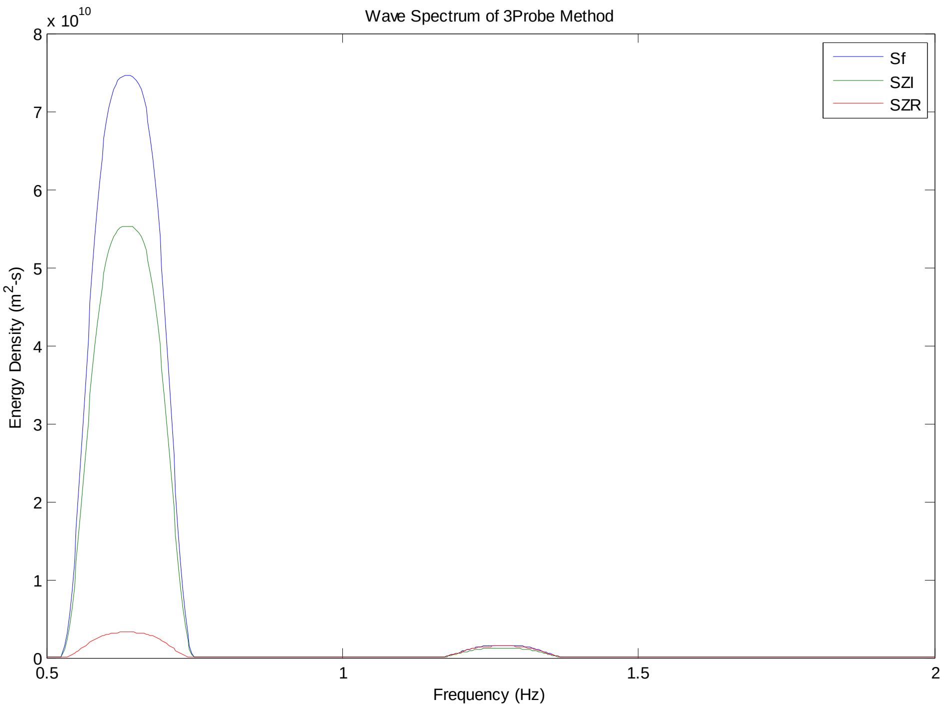
xi. Regular wave of wave period = 1.5sec and steepness, H/L = 0.06



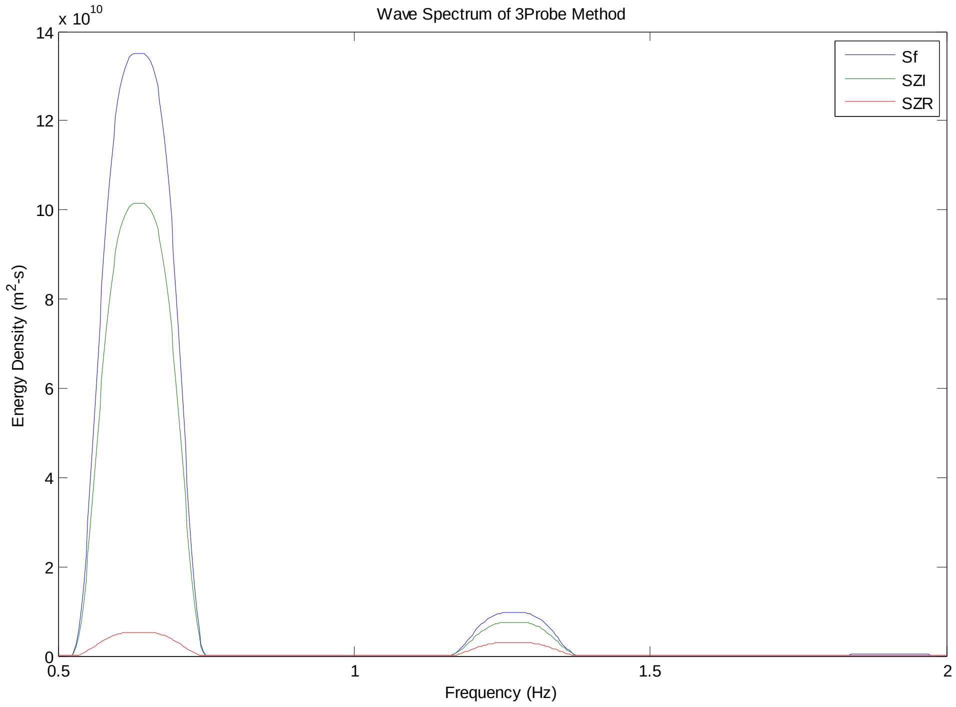
xii. Regular wave of wave period = 1.5sec and steepness, H/L = 0.08



xiii. Regular wave of wave period = 1.6sec and steepness, H/L = 0.04



xiv. Regular wave of wave period = 1.6sec and steepness, H/L = 0.06



xv. Regular wave of wave period = 1.6sec and steepness, H/L = 0.08