APPLICATION OF GEOTUBE BREAKWATER FOR MUDDY COASTLINE PROTECTION IN PENINSULAR MALAYSIA

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APPLICATION OF GEOTUBE BREAKWATER FOR MUDDY COASTLINE PROTECTION IN PENINSULAR MALAYSIA

ABSTRACT

Muddy coastlines along the west coast of Peninsular Malaysia are experiencing severe degradation due to construction activities and clear-cutting of mangrove belts for socio-economic development. Direct exposure of the muddy coast to storm surges, tides and waves will accelerate the coastline erosion. As a remedy for eroded coastal areas, revetments and dikes are normally construct along the eroding shorelines. Geotube breakwaters offer an alternative for coastal protection due to quick and ease of installation procedures, less impact to the environment and cost effective. However, published guidelines, technical publications and codes of practice for geotube breakwater are limited. Installation works were mainly based on engineers' experiences and judgement. There were cases where geotube breakwaters experienced sliding, overturning and excessive settlement due to inappropriate design process. This study was carried out to examine the application of geotube as a practical, viable and cost effective muddy coastal protection structure along the west coast of Peninsular Malaysia. Analyses were carried out to evaluate the advantages of utilizing the geotube breakwaters as a more versatile and environmentally friendly coastal protection option, especially on muddy coasts, as compared to other traditional coastal structures. The internal stability and external stability of geotube as coastal breakwater were studied. Optimum height, pumping pressure, maximum tension on geotube, structure deformation, displacement and settlement were analysed and evaluated, based on the wave conditions and geotechnical data from the study site in Sungai Haji Dorani (SHD), Selangor, Malaysia. Analyses of the external stabilities were carried out by using finite element analysis and the results were used to establish the factor of safety of the geotubes based on the geotechnical and

geomorphological conditions in SHD. Based on this study, recommendation for the use of the waste material, quarry dust, as the filling material of geotube breakwaters was proposed. On the other hand, prediction of sediment activities with the presence of geotube breakwaters is important to ensure the optimum protection and nourishment effect. Hydrodynamic models were developed to simulate the changes in wave currents and directions before and after installation of geotube breakwaters. The predictions of sediment activities around geotube breakwaters were developed according to the outcomes of the analytical models and were compared with the field measurements to appraise the sturdiness of the results. Study showed that the geotube breakwaters are good alternatives for coastal protection, especially for muddy coastline, which have a softer and deformable foundation. The leeward regions sheltered by the geotubes breakwaters will provide a calm area for mangrove rehabilitation. In most of the tropical countries, matured mangroves act as natural barriers to minimize the dynamic effect of waves. Therefore, rehabilitation of mangroves along eroded coastlines is an important action to preserve the natural environment. The wave and geotechnical conditions used in this study represent the majority of the eroded mangrove mud coast along the west coast of Peninsular Malaysia. Thus, the methods applied in this study can be replicated to simulate or to predict the behaviours and effectiveness of geotubes breakwaters, with relatively similar coastal environment.

Keywords: geotube breakwater, finite element analysis, stability, muddy coast

KEGUNAAN PEMECAH GELOMBANG GEOTIUB SEBAGAI PERLINDUNGAN PANTAI LUMPUR DI SEMENANJUNG MALAYSIA

ABSTRAK

Pantai lumpur di sepanjang pantai barat Semenanjung Malaysia mengalami degradasi serius akibat aktiviti pembinaan dan penebangan pokok bakau bagi perkembangan social dan ekonomi. Maka, kawasan pantai berlumpur terdedah kepada lonjakan ribut, pasang surut dan gelombang yang akan mempercepatkan degradasi pantai. Bagi mengatasi masalah tersebut, lapisan perlindungan pantai dan tanggul dibina di sepanjang pantai terhakis. Pemecah gelombang geotiub merupakan alternatif perlindungan pantai sebab pemasangan yang cepat dan mudah, kos efektif dan tidak meninggalkan banyak kesan negatif kepada alam sekitar. Walau bagaimanapun, garis panduan, penerbitan teknikal dan kod amalan bagi pemecah gelombang geotiub adalah amat terhad. Kebanyakan projek pemasangan geotiub adalah berdasarkan pengalaman dan pertimbangan jurutera. Terdapat kes di mana pemecah gelombang geotiub menggelongsor, terbalik dan terbenam disebabkan proses reka bentuk yang tidak sesuai. Penyelidikan ini dijalankan bagi mengkaji penggunaan geotiub sebagai struktur perlindungan pantai lumpur di sepanjang pantai barat Semenanjung Malaysia yang praktikal, berdaya maju dan kos efektif. Analisis telah dijalankan demi menilai kelebihan menggunakan pemecah gelombang geotiub sebagai perlindungan pantai yang lebih serba boleh dan mesra alam, terutama di kawasan pantai berlumpur, berbanding dengan struktur tradisional pantai yang lain. Kestabilan dalaman dan kestabilan luar geotiub sebagai pemecah gelombang telah dikaji. Ketinggian optimum, tekanan pam, ketegangan maksimum pada geotiub, perubahan bentuk struktur, anjakan dan pembernaman telah dianalisis dan dinilai, berdasarkan keadaan gelombang dan data geoteknik dari tapak kajian di Sungai Haji Dorani (SHD), Selangor, Malaysia. Analisis kestabilan luar telah dijalankan dengan menggunakan analisis unsur terhingga dan keputusan digunakan untuk mencari faktor keselamatan

geotiub berdasarkan keadaan geoteknikal dan geomorfologi di SHD. Berdasarkan kajian ini, penggunaan bahan sisa iaitu debu kuari, sebagai bahan pengisian pemecah gelombang geotiub telah dicadangkan. Ramalan aktiviti sedimen dengan kehadiran pemecah gelombang geotiub adalah penting bagi memastikan kesan perlindungan yang optimum. Model hidrodinamik telah diguna untuk mensimulasikan perubahan dalam arus dan arah gelombang, sebelum dan selepas pemasangan pemecah gelombang geotiub. Ramalan aktiviti sedimen di sekeliling pemecah gelombang geotiub telah dibuat berdasarkan hasil analisis. Ramalan ini dibandingkan dengan ukuran di SHD demi menilai kekukuhan keputusan. Kajian menunjukkan bahawa pemecah gelombang geotiub adalah alternatif yang baik untuk perlindungan pantai, terutamanya pantai berlumpur yang tapaknya lebih lembut dan mudah ubah bentuk. Kawasan lindungan oleh geotiub adalah lebih tenang dan sesuai untuk pemulihan bakau. Dalam kebanyakan negara-negara tropika, pokok bakau yang matang bertindak sebagai halangan semula jadi bagi mengurangkan kesan dinamik daripada gelombang. Oleh itu, pemulihan hutan bakau di sepanjang pantai terhakis adalah tindakan penting untuk memelihara alam semula jadi. Data gelombang dan geoteknik yang digunakan dalam kajian ini mewakili keadaan persekitaran kebanyakan pantai lumpur di sepanjang pantai barat Semenanjung Malaysia. Oleh itu, kaedah yang digunakan dalam kajian ini boleh diaplikasikan bagi meramal keberkesanan pemecah gelombang geotiub, di pantai yang mempunyai persekitaran yang agak sama.

Kata kunci: pemecah gelombang geotiub, analisis unsur terhingga, kestabilan, pantai lumpur

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LIST OF SYMBOLS AND ABBREVIATIONS

ø'	:	Interface friction angle between geotextile and base sand
Δ_t	:	Relative density of geotube (kg/m^3)
β	:	Empirical coefficient related to the height of geotube, wave height and initial water height
b	:	Width of contact or contact area between muddy foundation and structure
B	:	Width of equivalent rectangular shaped tube
c	:	Cohesion of base soil
Cc		Coefficient of curvature
C_u	:	Coefficient of Uniformity
D_{10}	:	Sieve size where 10% of sand materials passes through
D_{30}	:	Sieve size where 30% of sand materials passes through
D_{60}	:	Sieve size where 60% of sand materials passes through
D_{85}	:	Sieve size where 85% of sand materials passes through
D_k	:	Length of geotube if the geotube is parallel to the wave direction, or Width of geotube if the geotube is perpendicular to the wave direction
d_s	:	Arc length
D_v	:	Vane diameter, 50.8 mm
e′	:	Eccentricity of the hydrodynamic pulsating load
ϵ_p	:	Elongation
ε_m	:	Maximum strain of the geotextile
E	:	Young modulus
F	:	Equivalent wave load
F_{hp}	:	Hydrodynamic pulsating force

$F_{s\text{-}id}$	:	Reduction factors for installation damage
$F_{s\text{-}cd}$	:	Reduction factors for chemical degradation
$F_{s\text{-}bd}$	:	Reduction factors for biological degradation
$F_{s\text{-}cr}$	:	Reduction factors for creep damage
$F_{s\text{-}ss}$	:	Reduction factors for seam strength
FEM	:	Finite Element Method
FoS	:	Factor of Safety
g	:	Gravity acceleration
Gs	:	Specific gravity of sediments
h	:	Water level from foundation
H	:	Height of geotube
H_f	:	Final height of geotube, after consolidation
H_s	:	Significant wave height
h_{GT}	:	Effective height of geotextile tube
J	:	Tensile stiffness
k_w	:	Hydraulic conductivity of GCL to water
L	:	Geotube's circumference
M_R	:	Moment preventing rotation
M_o		Moment causing rotation (Nm)
N_c, N_γ	:	Bearing capacity factors by the internal friction angle of saturated base soil
O_{90}	:	Average geotextile pore size where 90% of the sand (> 60 μm) remain on it
O_{95}	:	Apparent opening size of geotextile
$P_{horizontal}$	:	Horizontal force
P_h	:	Hydrostatic pressure applied on foundation

ρ_m	:	Density of slurry
ρ_s	:	Density of soil
ρ	:	Fluid density
P_o	:	Pumping pressure
P_v	:	Overburden pressure and gravity weight of geotextile tube
P_w	:	Hydrodynamic pulsating load
r	:	Radius of curvature
R	:	Tensile test speed rate
SHD	:	Sungai Haji Dorani
S_u	:	Undrained shear strength from the vane
T	:	Circumferential tensile force
T_{max}	:	Maximum allowable tensile strength of the geotextile
T_{ult}	:	Ultimate tensile strength of geotextile
T_{work}	:	Tensile force under load conditions
t_g	:	Thickness of the geotextile fabric
T_{max}	:	Maximum allowable tensile strength of the geotextile
W_s	:	Specimen's width
γ	:	Unit weight of slurry
γ_{final}	:	Saturated unit weight of consolidated fill
γ_w	:	Unit weight of sea water
γ_s	:	Submerged unit weight of base soil

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CHAPTER 1: INTRODUCTION

1.1 Background of Study

Coastal erosion and accretion are inevitable natural processes as the coastal sediments are constantly in motion due to tides, waves, wind and currents. However, coastline recession requires management and mitigation approaches when human lives and properties are threatened.

The anthropogenic activities such as sand mining, construction of ports and harbours, agriculture and aquaculture industries, have contributed to a sediment deficit along the coastlines. Climate change and sea level rise in recent decades added another layer of complexity to the coastal erosion issue (Wang et al., 2014; Le Van Cong et al., 2014; Cui et al., 2015). Erosion of coastline has become an issue that requires attentions and concerns all over the world. According to the Malaysia's National Coastal Erosion Report, 29% of the 4,809 km coastline in Malaysia are experiencing coastline degradation (Department of Irrigation and Drainage Malaysia, 2012).

Coastal regions are important areas for socioeconomic activities. Therefore, coastal protections are essential to maintain the mean position of a stable coastline over a period of time, or in short, to encourage the dynamic equilibrium of coastline (González & Medina, 2001; Ashton et al., 2011). Primary aims of coastal protections include wave reduction, flood prevention, encourage sediment nourishment and prevent further recession of coastline. Conventional mitigation measures include construction of higher or stronger coastal protection structures. The main challenges for the coastal and geotechnical engineers are to develop cost-effective, ecologically-friendly and viable approaches for coastal management (Chu et al., 2012).

In Malaysia, beaches, coral reefs and mangrove forests are valuable coastal habitats which formed the basis for agriculture, aquaculture, tourism and recreational economies.

These coastal habitats play their roles as the natural coastal defences and shelters for the maritime and coastal species. Kathiresan & Rajendran (2005) reported the common existence of mangroves along muddy coastlines and their ability to significantly reduce the destruction impacts from high intensity waves.

Along muddy coastlines in Peninsular Malaysia, there were thick mangrove belts that protect coastlines. However, change in land use and development in fisheries, agriculture, shipping and tourism industries, resulted in severe mangrove recession and shoreline erosion. In order to facilitate mangrove rehabilitation and reduce active coastline erosion, soft engineering structures such as geotube and artificial reef; or hard engineering structures such as concrete breakwater and revetment were normally constructed (Raja Barizan et al., 2008).

Rate of coastal erosion and accretion is highly affected by the nature of a coast, such as sediment types, sea level, wave climates, and geomorphologic setting (van Rijn, 2011). Thus, coastal protection or management approaches are very site-specific, depending on sediment types, protection goals, safety level requirements, as well as social, economic and political factors (Szmytkiewicz, 2008). Hard engineering solutions such as revetments, groins, seawalls and breakwaters, have been proved effective in local scale's coastline restoration (Basco et al., 1997; Schoonees et al., 2006; Fanini et al., 2009; Elsharnouby et al., 2012; Saengsupavanich, 2013). Despite its effectiveness, hard engineering structures are less environmentally friendly because majority of these structures are built from rocks, wood and concrete. Rocks exploitation is involved and carbon dioxide is emitted in great amount during the production of concrete. Besides, in region where natural rock resource is limited, construction cost will be very high. In order to construct these coastal protection structures, removal of natural wave barriers such as mangroves and vegetation are required, and this contradicts the aim to protect the coastline and habitats.

Polomé et al. (2005) described the importance of coastal rehabilitation to every country such as providing preserved areas for fishing, agriculture, recreation and tourism, storm protection, preventing loss of sediments and habitats, landscape preservation, and most importantly passing on natural and heritage assets for the future generations. Recent schemes for coastline stabilization and preservation encourage solutions that are able to reduce incoming wave forces, ecological friendly, fulfilling aesthetic requirements, time and cost effective, while provide socio-economic benefits to coastal communities (Borsje et al., 2011; Edwards et al., 2013). Innovative methods such as geosystems, i.e. geotube, geocontainer, geobag and artificial reefs have recently received increasing demands as compared to alternative coastal protection methods due to their lower uptake of construction cost and time (Frihy et al., 2004; Düzbastılar & Şentürk, 2009; Yang et al., 2010; Chu, et al., 2012). These approaches are also claimed to be eco-friendlier as compared to the conventional concrete or rock structures.

Geosynthetic structures such as geomat, geocells and geotubes have been used for hydraulic and marine engineering projects for the past decades (Alvarez et al., 2006; Chu et al., 2011). Geosynthetic structures are high strength permeable woven or non-woven geosynthetic materials that are commonly filled with sand slurry through pumping procedures. The selection of geosynthetic material is very important to ensure good solids retention and permeability. In normal practice, some important considerations are often neglected to simplify the designs. For instance, these considerations are long term wave motion resistance, biological and chemical resistance, hydrodynamic forces exerted on structure, structure's strength, density and size of filling material. Over simplification of important parameters such as assuming impermeable and non-deformable foundation might result in improper designs which lead to structural instability or failure (Chew et al., 2003).

On the other hand, geological materials i.e., filling materials and foundation are inherently variable, adding complexity in design process. Pilarczyk (2000) reported that there were failed application of geosynthetic structures for coastal protection. Over simplification in designs leads to failure mechanisms such as structure overturning, sliding and excessive settlement. Besides, damage of geosynthetic structures during the installation processes happen very frequently (Perkins & Edens, 2003).

In order to prevent major structural damage during installation and to foresee potential obstacles, simulations is very important. Prediction of potential obstacles helps to avoid major structural damage, while achieving the objectives of coastal protection. Computer modelling or simulations allow us to foresee the potential risks or issues during the application of geosynthetic structures for coastal protection. Prediction of the safety factors and structural stabilities during the design phase helps to save cost and time while ensuring safer for long term.

Researches on the design and materials used for coastal protection structures result in a significant amount of cost and time savings, while providing better mitigation measures for coastal erosion. Design of coastal protection structures should strive for safety, cost-effectiveness, environmentally friendly, yet fulfilling the nation's environment demands (Shiming et al., 2012).

1.2 Problem Statements

According to the Department of Irrigation and Drainage Malaysia (2012), 29% of Malaysia's coastal areas are experiencing critical erosion issue. The east coast of Peninsular Malaysia is mainly sandy beach while the west coast is mainly muddy coast.

Mangrove forests are commonly seen along the muddy coast in Malaysia in previous decades. These mangrove habitats slowly depleted due to the severe coastal erosion issue.

Coastal areas are important as they provide places for residential purposes, tourism, harbours, fisheries and aquaculture industries. Severe coastline degradation leads to flooding and property damage. Besides, ecosystems along muddy coasts, such as mangrove swamps are very fragile. Once destroyed, mangroves need a very long period to recover or rehabilitate. It is necessary to prevent or counter the coastal erosion issues, in order to protect environment, ecosystems and to maintain the development of near coast socio-economic activities.

The major causes of the coastal erosion can be categorized into natural phenomenon and human activities. Beaches naturally experience wave cycles, sediment erosion and deposition. Coastline is considered stable as long as the mean position of the coastline does not change drastically. However, natural phenomenon such as storm, tsunami and cyclone erode the coast severely. Likewise, human activities and developments, such as construction of ports also erode the coast severely.

There are many well-established mitigation measures for coastal erosion and they can be categorised into two main groups, i.e. hard engineering methods and the soft engineering methods. Hard engineering methods are well established with many technical reports, codes and standards as design references. However, these methods are generally the more expensive options for coastal protection. Hard engineering structures are mainly made of the depleting natural resources such as rocks, wood and concrete. Exploitation of the natural resources brings negative impacts to landscape or environment. Examples of hard engineering structures include seawall, revetment, groin, rock armour, concrete block, and concrete breakwater.

On the other hand, soft engineering approaches are alternative methods that have less impact to the environment. Soft engineering approaches include natural beach nourishment, mangrove reforestation, geotube breakwaters, artificial sea grass and artificial corals. Hard engineering structures like concrete breakwaters are relatively more expensive to construct and maintain, especially in the regions where natural rocks are in shortage. In such regions, geosynthetic structures can be an alternative approach for coastal protection. However, there is still space for improvement in design codes or standards for geosynthetic coastal structures. Most of the construction of these structures was done by specialist companies based on the experts' experience and judgment and technical reports were not well published (Pilarzky, 2000). Even though the geosynthetics structures are often claimed to be effective and easy to maintain, more studies should be carried out to verify the practicability and viability of geosynthetic structures for coastal protection.

Moreover, inherent heterogeneity of muddy coastal sediments, wave and wind climates, beach condition and nearby developments are some of the factors that need to be carefully considered during design. The soft and deformable foundation of muddy coast are often assumed to be rigid and non-deformable during computer modelling in order to simplify the simulation. However, there were reports on geosynthetic coastal structure failures due to underestimation in designs, such as excessive settlement, sliding and overturning of structures. Hence, it is difficult but yet important to foresee potential failure mechanisms of coastal structures.

Geotube breakwater is one of the soft engineering methods that received increasing demands for coastal protection. Advantages of using geotube breakwaters include short installation time, cost effectiveness, versatility, easy transportation, simple construction procedures and less impact to the environment (Chu et al., 2012; Lee et al., 2014).

Installation of geotube breakwaters is normally based on the experience and judgment of experts, as references and code of practices are not well established yet. There were studies that analysed the stability of geotube breakwater but ignored the deformable characteristics of foundation and wave impacts in order to simplify the simulation. These simplifications and assumptions can lead to underestimation of failure potentials, i.e., sliding, overturning and bearing capacity failure. Research on internal and external stabilities of geotube breakwaters placed on deformable muddy coast are needed to improve the application of geotubes in coastal protection. Besides, considerations of the wave impacts, structure's alignment and placement are also important in design.

Design of coastal protection structures is highly dependent on the site's conditions. Geotechnical information, hydrodynamic conditions and wind climates are all crucial concerns during design phase. There is a need to study the performance or stabilities of geotube breakwaters, and to investigate the factors of failure mechanisms. Beach responses such as change in wave currents, wave speeds, and sediment elevations are also important. Placement, arrangement and dimension of geotube breakwaters directly affect the effectiveness of the structure in erosion protection and sediment nourishment. Advancement in computer technologies allows the development of models to predict the structure's behaviours and to simulate real field situation. Good prediction of structure's performance assists the design and planning, which ensure longer structure service period, cost saving, safety and less material waste during construction.

Coastal protection design seeks approaches that are safe, viable, cost and time efficient and bring least impacts to landscape and environment. Applications of geosynthetic structures as muddy coastal protection are not maturely developed like hard engineering methods. Thus, behaviours of geosynthetic coastal structures, beach responses, cost and environmental aspects require more studies, as references to assist engineers in developing good designs and plans.

The primarily purpose of this study is to provide a full insight of major aspects related to application of geotube breakwaters on muddy coast. It is to be hoped that the utilization of geotube breakwaters can be a common alternative to substitute the conventional rock and concrete coastal defence structures, with improved design methods and criteria.

1.3 Objectives

The main objectives of the study are as follows:

1. To identify factors that influence the internal stabilities of geotube breakwaters.
2. To identify external stabilities of geotube breakwaters placed on deformable foundation conditions.
3. To evaluate the suitability of quarry dust as an alternative filling material for geotube breakwaters.
4. To obtain the beach responses after the installation of geotube breakwaters as coastal protection structure.

1.4 Scopes of Research

This study focuses on the behaviours of geotube breakwaters as the muddy coastal protection structure. The detached L-block concrete breakwaters from previous project are also briefly discussed but they are not the main focus of this project. In brief, this study focuses in particular on the following issues:

1. Factors that influence the internal stability of geotube breakwater.
2. External stabilities of geotube structures on the soft and deformable foundation.
3. Suitability of quarry dust to fill the geotube breakwaters.

4. Beach responses such as change in sediment elevation, after the installation of geotube breakwaters.

Internal stabilities are affected by many factors such as the internal pressure from slurry, external pressure from wave, reduction of structural height due to sediment leakage, etc. Therefore, the internal stabilities of geotubes were studied by examining the physical properties of the geotextile and filling materials through laboratory experiments. Parametric analysis was carried out to study the relationship between influencing factors and internal stabilities of geotube breakwaters.

External stabilities such as stability against overturning, sliding and bearing capacity are important considerations during breakwater designs. Geotechnical and hydrodynamic conditions of study site can significantly affect the external stabilities of geotube breakwaters.in this study, the influencing factors of external stabilities were studied through numerical simulation by adopting parameters obtained from study site, i.e. geotechnical and hydrodynamic information.

Besides, quarry dust was recommended to be used as an alternative filling material to replace the sand. The suitability of the sediment will be examined in term of size, and influences on the internal and external stabilities of geotube breakwaters especially in term of the height reduction and internal pressure caused on geotubes.

Finally, the beach responses with the presence of geotube breakwaters such as sediment accretion and erosion were studied. Monitoring works on the sediment elevation were carried out every two months. On the other hand, simulation was carried out to study the change in current speed and direction in order to predict the location of sediment accretion. Monitoring and simulation results were compared and discussed.

1.5 Outline of Thesis

There are five chapters in this thesis. Chapter 1 generally introduces the objectives of the study and scopes of work. The fundamental studies and previous researches done were reviewed and summarized in Chapter 2. Topics reviewed were of wide range which include the coastal processes, coastal management methods, geosynthetic systems and design considerations for geotextile tubes, just to name a few. In Chapter 3, the methodology of this research was thoroughly discussed. The methodology was basically categorized into three groups, i.e., the computer modelling, lab experiments and on-site monitoring. The details of the three scopes of work, equipment and materials used, study site and monitoring structures were included in Chapter 3. Chapter 4 presents the results and discussions which mainly focuses on the internal and external stability of geotube breakwaters, beach responses with the presence of geotube breakwaters, viability and practicality of the structure. The final chapter draws the conclusions and recommendations for future study.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Coastal erosion is a natural phenomenon due to tide, wind, and wave actions (Kilibarda & Shillinglaw,2014; van Wesenbeeck et al., 2015). Sediment transports from one location to the other location, causing the erosion and accretion of sediment. Sediment erosion and accretion along the coastal area shapes the coastline.

However, global warming and sea level rise fasten the coastline erosion problem around the globe (Gopinath & Seralathan, 2005). Besides the natural factors, coastline erosion caused by anthropogenic factors (Forbes et al., 2004). Coastal regions were important places for trading activities, population development, tourism, recreation, fishing and agriculture industries. Increment in population, structures construction and deforestation contributed to socio-economy development, yet inevitably accelerated coastline erosion (Anfuso and Pozo, 2009).

Coastal protection and rehabilitation are required when the erosion is too critical that habitats, human properties and activities are in threat. Main factors of coastline erosion have to be identified to assist coastal management planning and design (Umar et al., 2015). In this chapter, coastal processes, causes of erosion, and mitigation measures for coastline recession are discussed. In engineering point of view, mitigation measures for coastline erosion are categorized into hard engineering approaches, i.e. revetment, breakwater, seawalls and groynes; and soft engineering approaches, i.e., geosynthetic structures, artificial reef and artificial sea grasses. Both approaches are introduced in this chapter.

2.2 Coastal Erosion and Management

2.2.1 Coastal Processes

Approximate 70% of Earth's surface is covered by water. Water exerts force along shoreline and causes coastal landform change, through transportation and deposition of sediments. Therefore, coastal lands and sediments are constantly in motion and change over the years (Becker et al., 2012). Coastlines naturally experience advanced, retreat or alternations of both. Advancing coast phenomenon occurs when rate of accretion is quicker than rate of erosion, or when there is a fall in sea level and uplift of land. Coastline erosion occurs when rate of erosion of sediments is higher than rate of accretion, or when the sea level rise and land subsidence (Bird, 2011).

Cai et al. (2009) summarized types of coastline erosion, according to time scale and spatial scale. Long term erosion is a slower erosion process involves sediment transport and sea level rise, where the coastline experiences a permanent change of position. While, short term erosion refers to quick and sudden change of coastline position, generally due to enormous destruction from hurricanes and storm surges. There are three groups categorized for spatial forms of shoreline erosion, including the (1) coastline retreat which normally experienced by soft coasts without engineering protections, (2) landward movement of the zero depth contours where beach surface incision occurs at coasts with engineering protections, and (3) downward erosion of the lower beach in sub tidal zone by tidal current with the upper flat maintaining its original shape.

Coastline erosion or accretion are mainly affected by wind and wave climates, geological setting, sediment supply and sediment types. These factors are different for different regions. Therefore, coastal engineers must be aware that there is no standard policies or strategies for all regions. Different setting has different erosion and accretion pattern, and not all analytical tools and procedures are suitable for every setting. Hence the engineering strategies must be designed for each region and always flexible to changes according to the local conditions (USACE, 2002).

For a localized coastal management plan, environmental processes, hydrodynamics processes, seasonal meteorological trends, sediment processes, geological processes, long term environmental trends and the social and political conditions are crucial aspects that required attention (Runhaar et al., 2015; Khalizad et al., 2015).

2.2.2 Influencing Factors of Coastal Geology

Coast is a diverse and dynamic environment. There are many influencing factors for the formation of coast (Forbes et al., 2004). These factors can be grouped into two main classes which included the active forces that occur constantly, and the long-term forces or global changes that take place for a long period.

Main factors that influence the coastal geology include natural processes, biological and chemical processes, and human activities (USACE, 2011). The biological components can be both constructive or destructive to coastal areas. Coral reefs, mangroves and sea grasses are very useful in trapping sediments and encourage beach nourishment naturally. Nevertheless, large species of kelp can be the eroding and transporting agent for gravel and cobblestones.

High frequency dynamic processes are the primary cause for sediment erosion and accumulation. The dynamic processes are crucial consideration during the coastal protection designs, especially sources of energy, sediment transport, and modification of existing topography (Carter & Woodroffe, 1994). The dynamic processes are generally influenced by the waves, tides and seabed elevation.

Wave is one of the major factors that affects the formation of beach. Wave is the main energy source that carries sediments along the coastlines. Surface waves derive their energy from winds and dissipate the energy near shore region on the beach. Hence wind and wave climates are crucial for the planning, design and construction of harbours, coastal defence structures and other coastal projects.

While, tides are rise and fall of water levels due to gravitational interactions among the sun, moon and Earth. The periodic changes in water level allows exposure of waves energy in different parts of the shoreline throughout the day. Tides is very important in ensuring the various part of intertidal zones are exposed to both erosion and deposition. Besides, tides themselves can effectively erode and accrete sediments along the shorelines due to the rotational nature of tidal waves (Jensen et al., 2001).

The study of atmospheric phenomena is called meteorology and it is greatly influence by the climate. Wind can directly or indirectly affect the coastal geology. Wind is an agent of erosion and transportation. Dune is the geomorphic features which form and size are results of wind. Winds also indirectly affect the coastal geomorphology by causing the waves and oceanic circulation. Tropical storms can cause severe erosion to the beach while destroying the shorefront properties (Houser et al., 2008).

Changes in sea level especially sea level rise, can accelerate the erosion of shorelines and destruction of human habitats, depending on the sediment types, sediments supply, coastal platform and regional tectonics. In many regions, mismanagement at coastal areas causes greatest influence on beach erosion, while sea level changes become the secondary effect.

Beside the natural causes, human has changes many world's coastlines by construction or sand mining. Local sediment dynamics will be affected by any new structures constructed. In some cases, negative impacts will extend for a few kilometres. Dunes and vegetation removed during the construction of man-made structures will diminish the sediments greatly. Besides, human activities such as sand mining remove the sediments form the beach and reduce the amount of sediment of the littoral system.

2.3 Coastal Protection

Mitigation measures adopted local authorities to counter the identified coastal problems, directly affect the socio-economic development of the area. Coastal management is a very challenging task for local authorities who need to provide infrastructures for local residents and different industries, at the same time need to ensure a balanced ecosystem of coastal areas (Le Van Cong & Shibayama, 2014). According to Department of Environment, Food and Rural Affairs, DEFRA coastal protection measures can be categorized into five main strategies as shown in Table 2.1. Figure 2.1 illustrates five main coastal management strategies.

Table 2.1: Coastal management strategies

Coastal Management Strategies	Descriptions
Do nothing	• No coastal management. • No man made coastal defence structures. • Abandon nearby structures if coastline eroded.
Managed retreat or realignment	• Involves coastal areas of low land value. • Identify new defence lines. • Constructs new defence structures. • Might involve reallocation of residents. • Costs include construction and monitoring.
Hold the line	• Coastal protection structures are built. • Relocate the erosion problems down drift or at the other parts of coast. • Soft or hard method or combination of both.
Move seawards	• New defences constructed seawards. • Can be adopted when land reclamation is needed for new economic and ecological development. • Create land of higher value.
Limited interventions	• Dissipate wave energy and protect land with lower risk. • Low cost. • Slow down the erosion process instead of stop erosion. • vegetation or beach nourishment.

(Adapted from DEFRA, 2001)

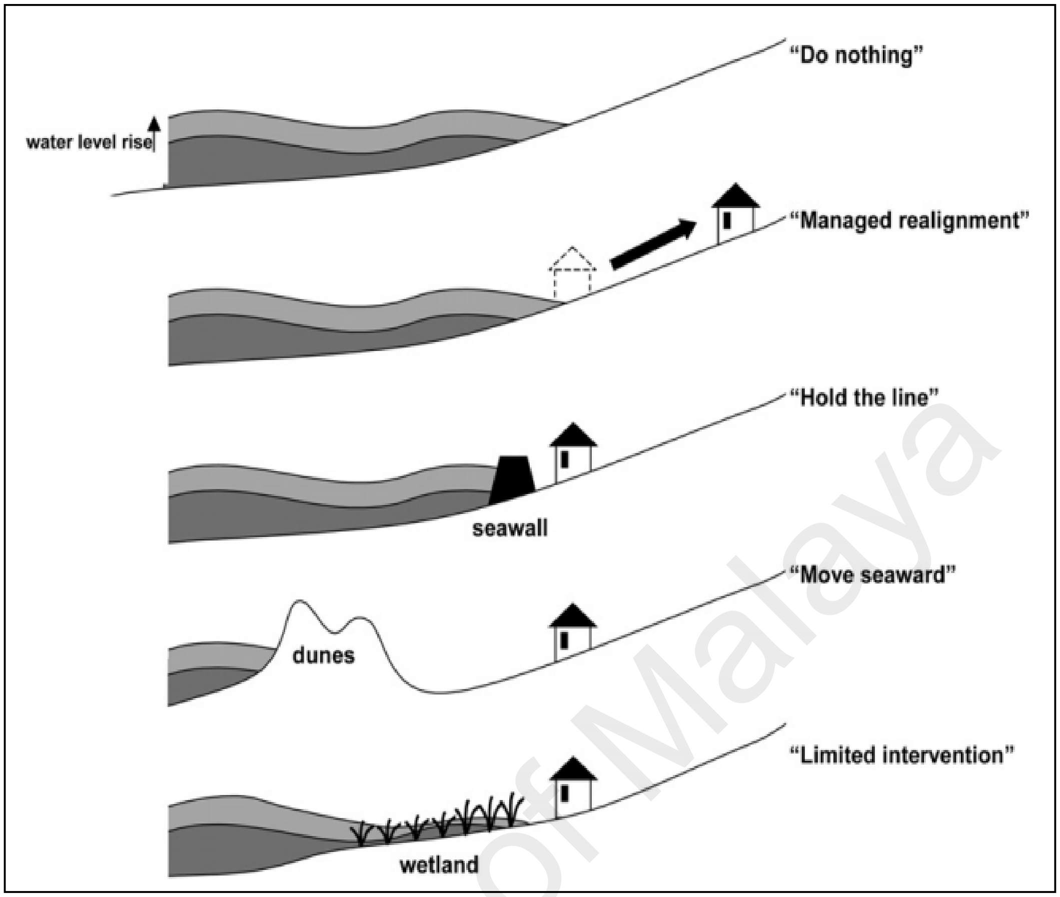


Figure 2.1: Coastal management strategies (DEFRA, 2001)

From the view of engineering, stronger and higher coastal defences structures are constructed to reduce the wave actions from eroding the shorelines. These methods can be categorized into hard engineering methods and soft engineering methods.

2.3.1 Hard Engineering Methods

In coastal engineering, reducing shoreline erosion from natural threats (i.e. sea water level rise, waves and tides) is a major task. The conventional coastal structures were built along the coasts to prevent shoreline erosion. The primarily purposes of the conventional nearshore coastal defences are to reduce the wave actions from hitting the beach. However, coastal protection structures are normally aim to protect a selected area instead of protecting the whole beach protection.

Table 2.2: Types and functions of conventional coastal defence structures

Type of Structure	Objective	Main Function
Sea Dike	Prevent or alleviate flooding by the sea of low-lying land areas	Separation of shoreline from hinterland by a high impermeable structure
Sea Wall	Protect land and structures from flooding and overtopping	Reinforcement of some part of the beach profile
Revetment	Protect the shoreline against erosion	Reinforcement of some part of the beach profile
Bulkhead	Retain soil and prevent sliding of the land behind	Reinforcement of the soil bank
Groyne	Prevent beach erosion	Reduction of longshore transport of sediment
Detached Breakwater	Prevent beach erosion	Reduction of wave heights in the lee of the structure and reduction of longshore transport of sediment
Reef Breakwater	Prevent beach erosion	Reduction of wave heights at the shore
Beach Drain	Prevent beach erosion	Accumulation of beach material on the drained portion of beach
Beach Nourishment and Dune	Prevent beach erosion and protect against flooding	Artificial infill of beach and dune material to be eroded by waves and currents in lieu of natural supply
Breakwater	Shelter harbour basins, harbour entrances, and water intakes against waves and currents	Dissipation of wave energy and/or reflection of wave energy back into the sea
Floating Breakwater	Shelter harbour basins and mooring areas against short period waves	Reduction of wave heights by reflection and attenuation
Training Wall	Prevent unwanted sedimentation or erosion and protect moorings against currents	Direct natural or man-made current flow by forcing water movement along the structure
Storm Surge Barrier	Protect estuaries against storm surges	Separation of estuary from the sea by movable locks or gates
Pipeline Outfall	Transport of fluids	Gravity-based stability
Pipe Structure	Provide deck space for traffic, pipelines, etc., and provide mooring facilities	Transfer of deck load forces to the seabed
Scour Protection	Protect coastal structures against instability caused by seabed scour	Provide resistance to erosion caused by waves and current

(Adapted from USACE, 2011)

The types of these coastal structures are manifold. The utilizations of these structures are very depending on the objectives and settings of the coastal protection projects. Some examples of conventional hard engineering structures are seawall (Jiang et al., 2014; Jin et al., 2015), dykes (van Loon-Steensma, 2014), breakwater (Saengsupavanich, 2013; Schmitt & Albers, 2014; Mikami et al., 2015), revetment (Yasuhara & Recio,2007), and groyne (Schoonees et al., 2006).

Majority of these hard engineering structures are made of concrete and rocks. This involve the exploitation of natural rocks and production of concrete. High emission of carbon dioxide during the setting of concrete will leads to greenhouse effect (Mehta, 2004). Besides, the cost and time consumed for a conventional coastal defence structure is high as compared to some new technologies and methods which will be discussed in next section. Table 2.2 summarizes the types and functions of the conventional coastal defences.

2.3.2 Soft Engineering Methods

There is a growing demand on the coastal defence structures. However, majority of the hard engineering structures required exploitation of natural rocks and production of concrete. These is not compatible to the recent trends for construction which emphasis time and cost saving, durability, versatility, sustainability, aesthetically and environmentally friendly, and of course, highly effective (Lee et al., 2014). These reasons led to the exploration in new materials and resources for coastal defences structures.

In recent decades, the soft engineering methods become a popular alternative for the hard engineering method. For instance, geobag, geotubes, geocontainer, prefabricated reef balls, man-made sea grass and so on (Pilarczyk, 2008). They are called the soft engineering method as these methods implemented environmental-friendly concepts and can be constructed speedily at low cost. These structures can also be removed any time when they are no longer in need. In this project, geotextile tube is the main focus and will be discussed further in the following session.

2.3.3 Considerations in Coastal Defence Structure's Designs

The design of the coastal structures typically carried out after the identification site environment and conditions. Considerations of coastal defence structure's design includes several aspects, for instance, function requirements, boundary conditions, other potential choices, structure's geometry, just to name a few (Pilarczyk, 2000). Except the functional criteria, the sustainable, environmental and economic considerations will also be included. The selection criteria for coastal structures are listed in Table 2.3.

Recent advancement in technologies allow us to have a closer prediction or estimation for coastal structure design and ensure sufficient design with optimal cost. There are several basic inputs that are very crucial in the planning and designing phase for a reliable design of coastal defence structures. These inputs are bathymetry of site, water level, waves, winds, and geotechnical measurements.

Table 2.3: Selection criteria of coastal structures

Aspects	Requirements
Functions	Protection against the wave attack at acceptable risk.
Construction's constraints	Conditions of site suitable for structure construction (e.g., able to transport materials)
Project cost and time	Cost and time of construction should be minimized or at an acceptable level. Cost for maintenance works is preferable to be minimal. Cost is one of the major factors in selections of structures.
Materials and equipment	Availability of land-based equipment and labours. Availability of waterborne equipment. Access to site and operability.
Environmental impacts	Structures should bring minimal negative impacts to the ecology, environment and landscape.
Legal restrictions	Permitted by laws and local authorities.

(Adapted from Pilarczyk, 2000)

There were several phases in the design of coastal defence structures. First the design phase, follow by the simulation phase and evaluation phase. In the design phase, there are several stages which includes the conceptual and preliminary design, detailed engineering design, construction and maintenance operation (Pilarczyk, 1990; Pilarczyk & Zeidler, 1996).

Simulation phase aims to investigate the behaviours or responses of coastal structures with different approaches. The commonly used simulation methods include the empirical, physical and numerical modelling (Samaras & Koutitas, 2014). Empirical solution is where adaption of formulas from existing geotechnical or hydraulic field, for the coastal applications. The selection of coastal structures is highly dependent on the project locations, size, budget, and sensitivity of risk. Different requirements of the projects will require different type of simulation methods. For a simple estimation, empirical solutions are often adequate.

Numerical modelling allows insight and understanding on processes and behaviour of structures, and this will help in design judgment (Tang et al., 2015). On the other hand, physical modelling is of high reliability in representing the coastal structures under loads and can be used for final verification of a design. However, it also results in higher cost of the project.

2.3.4 Typical Failure Mechanisms of Coastal Defence Structures

Failures might happen during the installation stage or during the operation period. Failure of the structure is where the structure experienced excessive displacement or deformation due to certain loadings. The common mistakes made that lead to the failure of designs include the insufficiency of data, inaccuracy of data or measurements, mistakes made during modelling and inappropriate model used (Muñoz-Vallés & Cambrollé, 2014;

Jayaratne et al., 2015). Failure can also be caused by the improper handling procedures during the construction stage.

The failure mechanisms can be grouped into two groups which are the rapid failure and gradual failure mechanisms. The rapid failure mechanism such as the sliding failure, do not have space for mitigation, repair or maintenance works. While gradual failure happens at a slower rate, for instance, the wave induced movement. Damage of coastal structures often related to the gradual loss of the structure's functions. When the damage is too severe that the structure fully loss its functions, failure of structures occurs. Table 2.4 summarizes the common failure mechanisms of coastal structures (Pilarczyk, 2000).

Table 2.4: Failure mechanisms of coastal structures

Failure mechanisms	Loading	Responses
Settlement	Weight of structures	Lowering of structure's height Horizontal displacement
Sliding	Structure's weight Other structure element's weight	Sliding of structure Structure collapse
Scour	Wave Current	Seabed degradation around structure
Cover movement	Wave Current Ice	Rocking Sliding Rolling Lifting
Piping	Hydraulic gradient	Internal material transport rate
Liquefaction	Wave Earthquake	Severe deformation of structure Structure collapse

2.4 Geosynthetic Structures for Coastal Applications

The conventional coastal defence systems, such as rubble or concrete coastal structures, have become very costly to construct and maintain in recent decades. The shortage of the natural rock supply in certain countries also has prompted the growing interest in finding new materials or methods as alternative coastal defence methods. Therefore, there is an increasing demand for lower cost and more environmentally acceptable coastal protection methods (John et al., 2015; Mahalingaiaha et al., 2015; Srisuwan & Rattanamanee, 2015).

Geosynthetics plays an important role in geotechnical engineering works. For coastal application, geosynthetics systems such as geotextile bags, mattresses, containers and tubes, were treated as a cheaper alternative to conventional solutions. Geotextile bags, containers, mattresses and tubes which were filled with dredged materials have been used as dikes and breakwaters around the world (Hornsey et al., 2011; Corbella & Stretch, 2012; Kiran et al., 2015).

Despite the growing interest in these low cost and innovative solutions, the applications are lack of proper design criteria (as compared to the conventional rock and concrete solutions) due to the limited published documents or references of the design of geosynthetics structures for coastal application. Most of these geosynthetics systems were designed based on experience and assumptions rather than valid calculation or analysis. Moreover, the long term and in-depth monitoring of massive projects of the geosynthetics systems is normally kept confidential in the specialist companies and have not been evaluated and discussed to a sufficient level for design guidelines (Pilarczyk, 2000).

2.4.1 Functions and Applications of Geosynthetics

Geosynthetics are high strength polymer materials, often used in contact with soil, rock, and mortar and have a very wide range of applications. Koerner and Koerner (2006) categorized the geosynthetic materials into six main groups which includes the geotextile (woven and non-woven), geogrid, geonet, geomembrane, geosynthetic clay liner and geocomposite.

The main functions of these geosynthetics are filtration, separation, reinforcement, containment and screen. Geosynthetics are widely used in coastal protection (Cantré &

Saathoff, 2013), erosion control (Koerner, 2012), roads and transportation, and waste management (Koerner, 2000). Table 2.5 summarizes the major functions of geosynthetics.

Table 2.5: Major functions of geosynthetics

Geosynthetic Type	Separation	Reinforcement	Filtration	Drainage	Containment
Geotextile	yes	yes	yes	yes	-
Geogrid	-	yes	-	-	-
Geonet	-	-	-	yes	-
Geomembrane	-	-	-	-	yes
Geosynthetic clay liner	-	-	-	-	yes
Geocomposite	varies	varies	varies	varies	varies

Different geosynthetics used for different purposes to fulfil different functional requirements. The primarily considerations before a design is to determine the functions of the geosynthetics in civil engineering projects. For example, geosynthetics used for filtration, separation and drainage must pose good elasticity, permeability, and soil retaining functions. While strong and stiff geosynthetics can be utilized for reinforcement works. For containment or protection applications, geosynthetics need to be elastic, able to retain soil, and in some cases, impermeable.

Geosynthetic commonly used in the sewage treatment plants and handling hazardous waste is geonet, i.e. the net-like sets of integrally joined overlapping ribs at various angles (Ortego et al., 1995; Adams, 1997). In terms of material and configuration, geonet looks similar to geogrid. However, the primary function of geonet is to perform in-plan drainage of liquids and gases instead of soil reinforcement. Thickness, chemical and biological clogging, intrusion and chemical stress cracking are the important factors that affect the in-plane flow rate and service life of geonet (Mok et al., 2012).

Geogrid which is a planar geosynthetic product consisting of mesh-like interconnecting ribs with large apertures, has a high tensile resistance and is primarily applied in soil reinforcement. Aperture shape affects the tensile behaviour of geogrid as the stress and strain distributions of geogrid with triangular apertures are more uniform as compared to rectangular apertures (Dong et al., 2011). Ferrotti et al. (2012) reported the contribution of fibre glass geogrid for road pavement, i.e. increasing the performance of asphalt concrete in terms of repeated loading cycles and enabling longer service life of the reinforced systems. Researches on the effectiveness of geogrid in reducing deformation of soil showed positive improvements in terms of load carrying capacity and soil settlements (Gniel & Bouazza, 2010; Deb et al., 2011; Rajesh & Viswanadham, 2011; Demir et al., 2013)

Impermeable geomembrane is often used as a barrier or liner, primarily for liquid and solid storage. Geomembranes are widely used in building underlayment, fuel storage tanks, solid and hazardous waste landfills, so the durability became the main concern (Barrett Jr & Stessel, 1999; Rowe et al., 2009; Lupo, 2010). Lupo and Morrison (2007) performed the design approaches for geomembranes application in the mining industry which were exposed to harsh climates and high loads. Hebeler et al. (2005) described the hook and loop interaction of the geomembrane surface which affected the peak interface strength. Hence the manufacturing and texturing techniques shall be considered during the selection of geomembranes for engineering design.

The geosynthetic clay liners (GCLs) which were often used as containment barrier are made from a thin layer of bentonite, contained between two geotextile layers. The hydraulic conductivity of GCLs to water, k_w is very low (less than 10^{-10} m/s). Apart from that, GCLs are economically favourable which made it a very good alternative material for the bottom lining of solid waste containment facilities, protection barriers of transportation facilities, and liners of subsurface strata and groundwater protection (Bouazza, 2002; Hornsey et al., 2011).

A combination of two or more materials of which one is at least a geosynthetic and possess a specific function, is called the geocomposite. Geocomposite were used exclusively in engineering projects with different purposes, such as separation, filtration, drainage, reinforcement, and containment (Hamir et al., 2001; McKean & Inouye, 2001; Jaisi et al., 2005; McCartney & Berends, 2010). Austin and Theisen (1996) mentioned that asphalt layers reinforced with geocomposite showed significant resistance in surface deterioration. Correia and Bueno (2011) reported that impregnation of bitumen on geotextile increased its strength values at strain levels less than 0.05%, decreased in stiffness as strain increased, and showed a drastic decrease in permeability properties.

Geotextile was commonly used in coastal protection and rehabilitation during the recent decades (Alvarez et al., 2006; Lee et al., 2014). Geotextile is a high strength polymer fabric which can form a larger unit by filling with air, water, sand and clay slurry or mortars. The coastal defence structures made of geotextile were known as geosystems which included geobags, geotextile tubes and geocontainers. Geosystems were proved to have impressive strength, durability and performance (Harris & Sample, 2005).

2.4.2 Geosystems

Geosystems are the containment units in different shapes and sizes, typically filled with sand, water or mortar. These geosystems are economical, environmentally friendly and innovative systems which are utilized as the alternative options for the conventional coastal structures (Saengsupavanich, 2013). The most commonly used geosystems are the geobags, geomattresses, geocontainers, geotubes and geomattresses, i.e., open containment units that contain gravel or stone (das Neves et al., 2015). Currently, these products can be found in different properties and sizes, supplied by various of specialist companies all around the globe.

Table 2.6: Applications of geosystems.

Applications	Geobag	Geotube	Geocontainer	Geomattress
Beach groins	√	√	-	-
Breakwater	√	√	√	-
Dune toe protection	√	√	-	-
Channel repair	√	√	√	√
Land reclamation	-	√	√	-
Underwater reef	√	-	√	-
Bed protection	√	-	-	√
Bank protection	√	√	-	√
Temporary dam	√	√	√	-
Sediment management	√	√	√	√

(Adapted from Pilarczyk, 2000)

Geosystems are often called 'soft solutions' for coastal protection due to the minimal impacts it will brings to the environment and landscape. However, Corbella and Stretch (2012) described the geosystems as 'pseudo-soft solutions' as geosystems can impede the natural morphodynamic of shoreline and create static shoreline, yet able to be removed easily when they are no longer in need. Despite coastal protection and mangrove rehabilitation, geosystems can also be utilized for reclamation, isolation and filtration projects. The applications of these geosystems are summarized in Table 2.6.

The advantages of these innovative geosynthetics units as compare to the traditional rock structures are the simple installation work and equipment, speedy installation time, lower in cost, and do not required skilled workers. Local contractors can handle the installation work with very minimal advice or supervision form specialist.

However, geosynthetics have limitations such as degradation by ageing, chemical, mechanical and biological attack, mechanical damage, creep and hydrolysis (Bezuijen & Vastenburg, 2008).

2.4.2.1 Geobags

Geobag as shown in Figure 2.2, is geosystem structures with the smallest size and volume. Geobags are widely applied for variety of coastal structures. For instance, groins, breakwaters (Bergado, 2007) and dikes (Chu et al., 2012). Application of geobags are of relatively higher cost as filling the geobag units is labour intensive. Besides, volume and size of the geobags are limited and required many units to form a coastal structure (Yan and Chu, 2010). Despite the higher cost of geobags installation, they are widely utilized along river and coastal areas due to its unique advantages.

Geobags are good substitution or alternative for rock armours especially at regions where rock or stones are shortage or expensive. Geobags can be removed very easily when their objective to protect the shorelines from erosion is accomplished or when they lose their function.

Figure 2.2: Geobags for river scour protection

Geobag is efficient and is able to be executed speedily at lower cost. The sand-filled high strength woven geobags are commonly used in the riverbank protection and coastal protection. Heibaum (1999) described the utilization of 48,000 geobags in protecting sandy beach from further erosion and scouring. Stockton Beach revetment, constructed with 480 staple fibre non-woven geotextile bags of volume 0.75m^3 successfully protected the beach from further erosion, for ten years, which is longer than the temporary design period of six months (Saathoff et al., 2007).

2.4.2.2 Geocontainers

Geocontainer is geosynthetics encapsulating system with very large containing volume, range from 100 m^3 to 800 m^3. As shown in Figure 2.3, the large container is carry by the split barge to the dropping point and then is dropped through the water from the split barge (Pilarczyk, 2000). The huge size and volume of geosynthetic container make it a system with plenty applications. For example, constructions of containment dam, offshore submerged dikes, artificial reef, breakwater and so on.

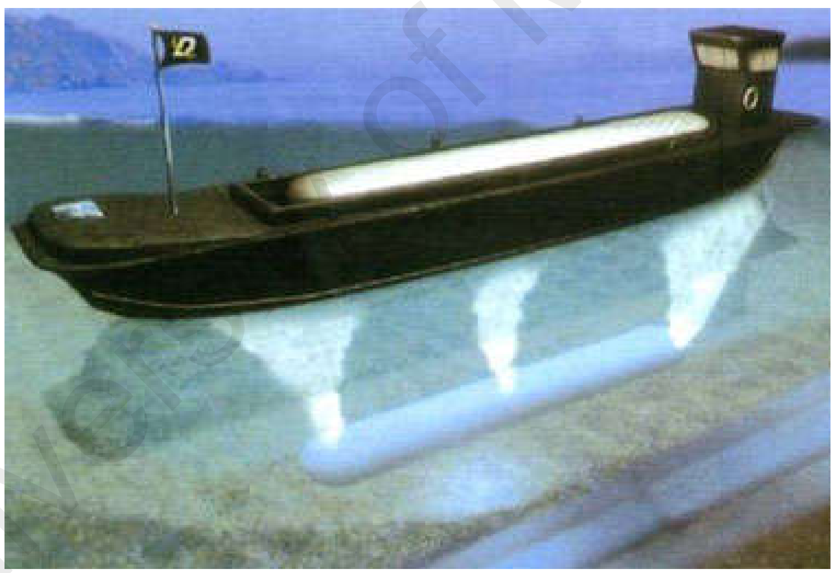


Figure 2.3: Geocontainers drop from split barge to designed location

Split barge is used to carry and dump the geocontainers to desired locations. Geocontainers were used to nourish and stabilize beaches in many Australian projects such as in North Kirra Groin, Russell heads and Maroochy River. Damage of these geosynthetic containers normally caused by the vandalism and coarse angular sediments or coral reefs abrasion. Hence the application of geocontainers required extra concerns from engineers to prolong their service period and performance (Restall et al., 2002).

2.4.2.3 Geomattresses

The geosynthetic mattress or the geomattresses are often utilized for bed protection and slope. Geomattresses are comprise of two layers interconnected geosynthetics. These geosynthetic units are filled with sand or concrete fills as shown in Figure 2.4. Two geosynthetic fabrics are sew into several compartments so that the filling materials can distribute evenly. The compartments will prevent the filling materials from great movement throughout the service period.

Figure 2.4: Concrete fill geomattresses for slope protection

The applications of geomattresses are often for the protection of river bed or canals, especially when the supply of rocks is limited. The mattress is normally placed directly on the subsoil and in some cases, another layer of soil will be added on it for extra protection (Pilarczyk, 2000).

2.4.2.4 Geotubes

Geotubes or geotextile tubes were developed and patented under Nicolon (Nicolon, 1995) and have been successfully applied in coastal protection projects since 1995. Geotubes are high strength polypropylene or polyester textile units that are permeable but able to capture soil within. Typical lengths of geotubes ranging from 30 m to 300 m. The dimension (circumferences, shape and length) can be customized for different projects.

Figure 2.5: Filling the geotube by pumping in slurry

Typically, geotube has minimum 40 kN/m of tensile strength, less than 20% elongation and seam strength of about half the tensile strength. The apparent opening size of the geotextile should be smaller than the 150 microns. The ports of inlet and outlet are normally 0.5 m in diameter and spaced at an interval of 30 m or more, depending on the length of geotube. Geotube design are dependent on the pumping pressure, filling material's properties and material's density. Normally geotube is filled up to 70% to 80% of its diameter (Oh & Shin, 2006). Figure 2.5 shows the pumping process during installation of geotube.

The weight of geotube is light as compared to the concrete or rock structures. The geotubes were normally rolled up and transport to the site. Installation of geotextile tube is simple, i.e. pump in the filling materials into the positioned tube to a desirable height. Pumping work can be done by inserting a dredge discharge pipe into ports or gravity-filled using a hopper (Jones et al., 2006). The geotubes will allow water to flow through while capturing the fills within. There are two ports, the inlet port and outlet port, where the slurry will be pumped into the inlet port. Generally, the slurry will be pumped into the geotextile tube with 20% to 25% of sediments so that the consistency of the slurry ensures the sediments able to pumped in to every corner of the tube (Bezuijen & Vastenburg, 2012). The most commonly height of tube is about 80% of its perfect diameter. Overly filled geotube which poses circular shape and significantly reduces frictional resistance to foundation.

Other considerations of a geotube include seam strength, ultraviolet radiation resistance, puncturing assistance and tube flattening due to consolidation. Besides, the alignment, placement, distance between two ports, permeability and consolidation of sediments are also important.

The speedy construction is favourable for projects under cost constraint. Oh and Shin (2006) mentioned time saving filling process of geotube which required less than one hour. Besides, they also reported the growth of seaweeds on the geotubes in Young-Jin beach, after one year of installation, which indicated that the polymer material was unlikely to cause negative impacts to the ecology.

In Russell Heads, a geotube groin was installed in 1993 as a coastal defence to the 1.0 m to 2.0 m wave climate. The geotextile tubes used were of 1.2 m diameter and 250 m length in total. The geotextile tubes were filled with home-made dredge pump to minimize the cost and showed good performance for over ten years (Saathoff, et al., 2007). However, damage caused during installation or vandalism will shorten the service life of geotextile tube used as coastal defence. Chew et al. (2003) carried out a series of cyclic flow tests on geotextile specimens with pre-cut holes in different sizes. Result shows geotextile damage affects strength of geotubes. However, higher flexural stiffness of the geotextile means able to perform normally with larger critical holes.

The main considerations in geotextile tube designs are as follows (Alvarez et al., 2006; Chu et al., 2011):

1. Strength of geotube directly affected by critical stress during pumping procedure. However, filling pressure does not significantly influence the final geometry of the geotubes.
2. Apparent opening size of geotextile is a crucial criterion to ensure good permeability and soil retain ability.
3. Potential geotextile damaging factors such as the chemical and biological degradation, insufficient seam strength and damage during installation also important in geotube design (Jeon et al., 2006). In Program GeoCoPS considered (Leshchinsky et al., 1996) the partial safety factors as follows:

$$T_{ult} = T_{work} (F_{s\text{-}id} \bullet F_{s\text{-}cd} \bullet F_{s\text{-}bd} \bullet F_{s\text{-}cr} \bullet F_{s\text{-}ss}) \quad (2.1)$$

where,

T_{ult} = Ultimate strength required (kN/m)

T_{work} = Tensile force under load conditions (kN)

$F_{s\text{-}id}$ = Reduction factors for installation damage

$F_{s\text{-}cd}$ = Reduction factors for chemical degradation

$F_{s\text{-}bd}$ = Reduction factors for biological degradation

$F_{s\text{-}cr}$ = Reduction factors for creep damage

$F_{s\text{-}ss}$ = Reduction factors for seam strength

4. Wave action, ultraviolet degradation and frictional effect of littoral drift are also important considerations but these matters require special monitoring on site for a long period.

Cantré (2002) described several important considerations in geotube design, such as height of geotube, filling ratio, consolidation effect, and the necessity of refilling after the consolidation and arrangement of geotextile tubes. Cantre also verified through his numerical modelling, critical tension on the geotextile tube occurred during the filling process. Unlike the conventional coastal protection structures, geotube design have limited design guidelines and references, despite the increase in demand for geotextile systems.

2.5 Application of Geotubes in Malaysia's coast

2.5.1 Hydrodynamic and Geomorphology of Peninsular Malaysia's Coasts

Sandy beaches and muddy beaches are two main types of geotechnical stratums of the coasts. They have very different sediment characteristics due to the variation in wave propagation towards the beach. Sandy beaches continuously are reshaped by higher intensity waves and tides. Fine sand will be washed away and leaving coarse sand to withstand the wave forces. On the other hand, mudflat which is composed of sand, clay or fine silt occurs at the coastlines which are protected from strong waves. Lower wave energy allows the deposition of finer particles and form mudflat. Plants such as algae, sea grass and mangroves are plants that able to grow well in mudflat. These plants, especially the mangrove forests are unique natural coastal defences found in tropical countries.

Mangroves are valuable assets for coastal communities. Mangrove forests play important roles in marine species nursery, dissipating wave energy approaching the shores and provided support for the low-lying agricultural land. However, due to natural coastal processes and human activities, erosion is inevitably to occur along the muddy beach. Sediment erosion cause the mangroves to topple over as the roots losses grip on the substrate. The number of mangroves reduced drastically and mitigation measures are in need to protect the precious assets Hence, the muddy coast protection are usually aim for both erosion protection and mangrove rehabilitation.

Generally, coastal management in these two-different coastal environments have different aims. Coastal management for the sandy beaches focus in erosion prevention and encourage beach nourishment. While for the mudflat, management method will be based on coastal protection and mangrove forest regeneration. The two different coastal environments might affect the effectiveness of geotubes application in coastal management.

Coastlines in Malaysia are mainly sandy coasts and muddy coasts with the total length of 4,800 km approximately. The east coast of Peninsular Malaysia is predominant by sandy beaches. However, in the west coast mudflats and mangrove forests are commonly be seen (Sharifah, 1992). The different coastal processes in Peninsular Malaysia are greatly influenced by the winds, monsoons, and tides. Winds will directly affect the sediment transportation, waves and currents; while tides caused rise, and fall of the water-level with tidal currents (Yang et al., 2013; Das & Crépin, 2013). Table 2.7 shows the hydrodynamic conditions in Peninsular Malaysia' coasts.

East coast experienced the higher wave energy compared to west coast. This is because shelter from Sumatera Island, Indonesia greatly reduce the dynamic forces hitting the west coasts. The reduced dynamic force allows the silt sized materials to remain near the shore and form the mudflat. The muddy environment is suitable for the growth of mangroves. While east coast was attacked by the stronger waves from South China Sea and affected by Northeast Monsoon annually (November to March). Fine sediments were transported away, leaving the heavier sandy materials. Thus, in east coast, sandy beaches are commonly seen.

Table 2.7: Hydrodynamic conditions in Peninsular Malaysia's coasts

Peninsular Malaysia's Coasts	**East Coast**	**West Coast**
Coast Type	Sandy beach	Mudflat
Monsoon Winds	North-East Monsoon	South-West Monsoon
Wave Height	< 1.8 m High wave energy	0.5 - 1.0 m Moderate wave energy
Maximum Wave Height (during storm)	2.7 - 4.8 m	3.0 m
Wave Period	6 - 9 seconds	6 - 9 seconds
Tidal	1 - 2 m Micro-tidal (< 2 m)	2 - 2.5 m Meso-tidal (2 - 4 m)

Natural factors such as winds, tides, storms and sea level rise directly influence the natural coastal processes, i.e., erosion and accretion of sediments. However, in Malaysia, the coastal retreat was accelerated by the tremendous development in agriculture and tourism industries. Constructions, sand dredging, and mangrove forests clearing significantly disrupts the coastal ecological environment and caused severe coastline erosion. Figure 2.6 shows the critical erosion areas in Peninsular Malaysia.

The awareness of Malaysia's government in coastal management increased especially after the tsunami incident in December 2004 that affected many Asia countries. Various of coastal defence structures were installed to protect the coastlines, infrastructures and human lives. The geotubes become a competitive alternative for the conventional hard solutions due to the implementation of eco-friendly concepts. Case studies regarding the application of geotubes in Peninsular Malaysia were carried out. Study cases were separated into two categories according to the coast types, i.e. sandy coast or mudflat.

Figure 2.6: Critical erosion areas in Peninsular Malaysia

2.5.2 Sandy Coasts Protection with Geotube Breakwaters

Sandy beaches are commonly seen along the East coast of Peninsular Malaysia. The coastal recession issue was always severe along the East Coast and was intensified by the North-East Monsoon. Without proper coastal management, the erosion of the coastlines threatens human lives and infrastructures nearby. Hence the authorities carried out coastal management in east coast with the main objectives to protect the beach from further erosion while encourage nourishment. Project in Teluk Kalong and Pantai Batu Buruk, Malaysia were reviewed, as the cases for the application of geotubes in sandy coasts.

2.5.2.1 Teluk Kalong

Sediments along the coast of Teluk Kalong are mainly granular sandy materials. Insufficient coastal protection and attack of the dynamic wave cyclic have led to the erosion of these sediments. Sediment erosion were experienced along the coastline and it has caused the structural instability of the existing precast concrete seawall. Uneven settlement occurred when the sandy materials behind the seawall panels drifted (Lee & Douglas, 2012).

Geotextile tubes of 500 m total length were used as a submerged dyke at 150 m offshore with the objectives to prevent further recession of coastline and encourage the sediment accretion to nourish the beach naturally. Figure 2.7 shows the condition of the beach before and after the installation of geotubes.

Figure 2.7: Condition of coastline before (a) and after (b) installation of geotextile tube in Teluk Kalong (Lee & Douglas, 2012)

During low tide condition, the geotextile tube must be fully submerged with a freeboard of 1 m. Performance of geotextile tubes was not affected much by the current and wind factors as they are submerged. The thickness of sediments accumulated after installation of the submerged dykes exceeded 1.8 m, with an estimated volume of 87,317 m^3. Nourishment of beach by submerged geotextile tube successfully created a gentler beach profile which has a higher potential for recreational purpose. Wave energy is lowered when the depth of water is reduced. This project increased the overall potential value of beach front at a low cost.

2.5.2.2 Pantai Batu Buruk

As a remedy to protect Pantai Batu Buruk from severe erosion, a 5 kilometre stretch of the beach was installed with the geotextile tubes as submerged breakwater, at 150 m offshore. The concept was very similar to the project at Teluk Kalong. The height of the geotextile tube used in Pantai Batu Buruk is 2.5 m and is placed above a geotextile scour apron. Estimated low tide water depth was 3.0 m and the submerged breakwater was required to be totally submerged with a free board of 1 m at low tide condition. Comparison between the pre-construction survey and post-construction survey at Pantai Batu Buruk showed positive impacts to the shoreline. The shoreline was prevented from further erosion and accumulation of sand was observed. The beach profile became gentler as compared to the previous steep scarps as shown in Figure 2.8

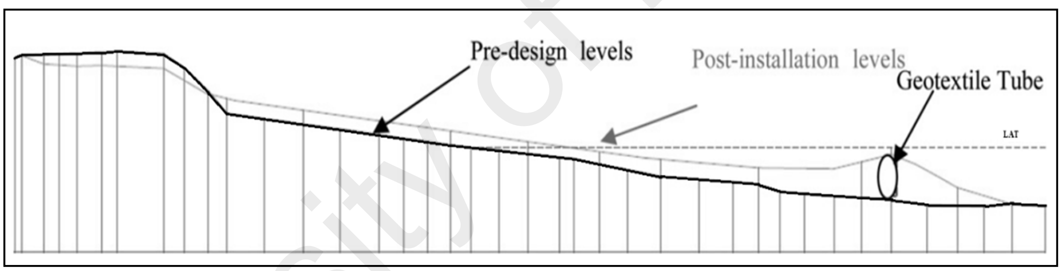


Figure 2.8: Cross section showing accretion of sediment in Pantai Batu Buruk (Lee & Douglas, 2012)

2.5.3 Muddy Coasts Protection with Geotube

Protection by Sumatera Island reduces wave energy approaching the west coast of Peninsular Malaysia. Lower dynamic force allows the silt sized materials to remain near the shore and form the mud flat which is suitable for the mangrove plantation. Inevitably, erosion occurs along the mangrove belts, despite the typical wave heights on the west coast being less than 2 m.

2.5.3.1 Tanjung Piai

Coastline erosion has occurred in Tanjung Piai since 1992 and was worsened due to the extensive port development nearby. However, as Tanjung Piai area had been declared as a national park, any shoreline protection measures proposed shall fulfil the requirements regarding the impact of construction to the environmental, aesthetic and tourism functions. Besides, work site must be accessed from the sea as it is not allowable to create roads in the park for equipment and materials transportation.

The National Hydraulic Institute of Malaysia used the mud-filled non-woven geotextile bags and brush fascine to stabilize the mangrove rehabilitation in year 2000. The geotextile breakwaters were placed 20 m from the escarpment with the main function being dissipate the wave forces approaching the shoreline (Ghazali et al., 2006). Hence a calmer water surface for sediment accretion and assist in the rehabilitation of mangroves is provided as shown in Figure 2.9. Department of Irrigation and Drainage (Department of Irrigation and Drainage Malaysia, 2012) monitored the performance of the geotextile breakwater through surveys and results showed the sediments had accumulated shoreward of the breakwater.

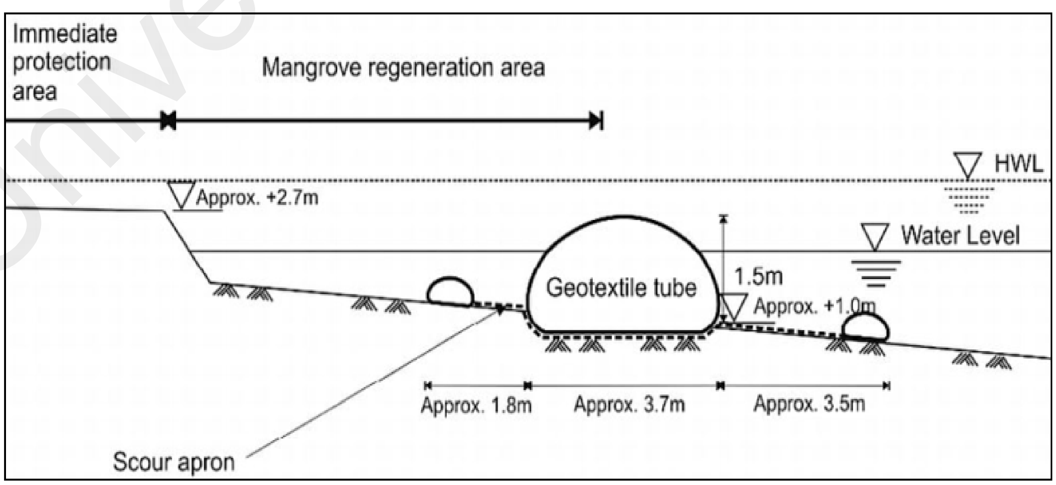


Figure 2.9: Geotextile tubes installed in Tanjung Piai to assist mangrove regeneration (Ghazali, et al., 2006)

2.5.3.2 Sungai Haji Dorani

An integrated method through mangroves regenerating with the assist of engineering structures to protect the coastal erosion was introduced in Sungai Haji Dorani (SHD), Sungai Besar, Selangor, Malaysia. In SHD, the seedlings of mangroves were difficult due to poor anchorage between tree roots and the mud flat in liquid form.

In 2007, a project was carried out by the Department of Irrigation and Drainage, to install geotextile tubes as offshore breakwaters to reduce the wave energy hitting the shoreline while promoting the deposition of sediments for mangrove regeneration (Raja Barizan et al., 2008). The area between the geotextile tube breakwater and the shoreline served as a mangrove plantation area (Jeyanny et al., 2012). Four geotextile tubes were installed at the beach front of the D'Muara Marine Park Resort in SHD due to the suitability of the study site with extensive open mud flat areas. The four high strength woven geotextile tubes with the dimensions 1.8 m x 3.7 m x 50.0 m were filled with sand slurry and placed at a 0.5 m gap between each other. The main aims of the installation of geotextile tubes were to dissipate approaching wave energy, to encourage sediment accumulation and to assist the mangrove regeneration. Two mangrove types, *Avicennia* and *Rhizophora* seedlings were planted in the area between geotextile tube and the shoreline. *Avicennia* could stand shallow mud level and *Rhizophora* grows well on thicker mud.

Four lines of measuring pins were implanted along the area behind geotextile tubes and in areas without geotextile tubes as shown in Figure 2.10. The 0.3 m exposed pins were plastic pipes implanted at 20 m interval along the baseline (Rasidah et al., 2010). The performance of geotextile tubes as breakwaters were monitored once a month by measuring the implanted pins for sediment accretion data, and by monitoring the surviving number of mangroves. Data taken from the area behind geotextile tubes and at area without geotextile tubes were compared and analysed.

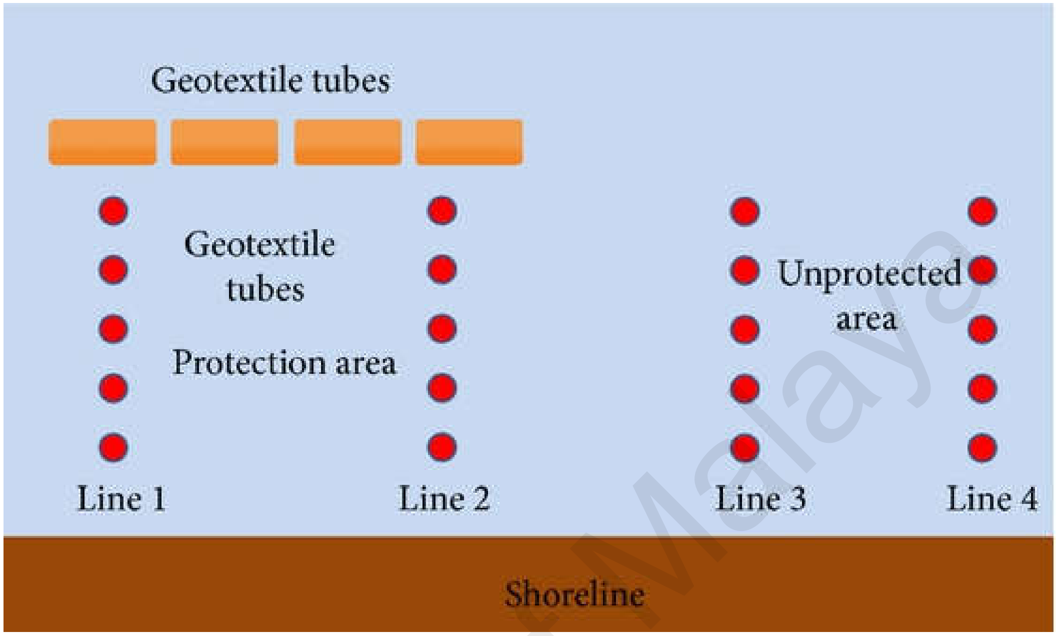


Figure 2.10: Location of measuring pins (adapted from Rasidah, et al., 2010)

Deposition and transportation of sediment disturbed the growth of *Rhizophora* but allow *Avicennia* to grow well. The project in SHD showed that the geotextile tubes provided temporary protection for the existing mangrove belts from further degradation, while stabilizing the shoreline through sediment accumulation (Lee et al., 2014).

2.5.4 Feasibility of Geotubes for Coastal Management

Application of geotubes as coastal protection in Malaysia can be a good lesson for the other tropical countries with the similar coastal geomorphology and climate. From the studies, application of geotubes were observed to be effective in coastal protection and able to assist mangroves regeneration. Geotubes are effective in nourishing eroded beaches naturally and assist mangroves growth. Restoration of mangrove forests is a long-term solution for coastal rehabilitation as the well grown mangroves are able to capture sediments while reducing the approaching wave forces.

However, the muddy coast which is formed by high proportion of silt and clay does not have high substratum strength as compared to sandy coast. Thus, uneven settlement of coastal defence structures which is installed on the mudflat, might be observable after certain period. The challenges, objectives, solutions and outcomes in the case studies are as shown in Table 2.8.

2.5.5 Advantages and Disadvantages of Geotubes

From the literature review (Alvarez et al., 2006; Saathoff et al., 2007), geotube is observed to be a good alternative to hard engineering coastal defence structures. These are the advantages of geotubes for the application in coastal protection.

1. Effective in coastline protection and nourishment
2. Easy to obtain, available in different dimensions
3. Can be customized according to needs
4. Light weight, easy to transport
5. Simple instrument and equipment needed
6. Required only low-skilled workers to install
7. Speedy execution time
8 Lower cost compared to hard engineering structures
9. Can be removed anytime, versatile
10. Does not involve rocks exploitation and concrete production

However, there are several disadvantages of utilizing geotubes for coastal protection in Malaysia, especially regarding the cost. As reported by Howard et al. (2012), the filling materials for geotube in Tanjung Piai were purchased with freight distance and hence the cost was less economically favourable. According to Russel and Micheals (2012), the cost for the geotextile tubes breakwater in Malaysia was approximately USD 700,000 for one kilometer of coast, which was higher than other tropical countries like Vietnam, USD

300,000 per kilometer. The difference in the installation price is very much dependent on the availability, dimensions, equipment cost and personnel cost.

Besides, the damages during the installation and the vandalism of geotubes need to be of concerned as severe damage can affect the performance. Geotubes installed along the mudflat in Chachoengsao's coastlines, Thailand in the year of 2005 was a good lesson for the researchers (Saengsupavanich, 2013). The geotubes installed experienced 0.6 m settlement after five years and were vandalised. Leakage of filling materials killed the marine animals, while damaged geotubes were not able to serve as coastal defences. Vandalism damage also experienced by geotubes installed along Gold Coast, Queensland, Australia (Restall, et al., 2002). The vandalism experienced in Gold Coast was then led to suggestions of geotextile coating with bitumen and early patching techniques. The disadvantages of geotubes application in the coastal protection in Peninsular Malaysia are as followings:

1. Higher cost compared to other countries.
2. Geotextile can be damaged by vandalism and shorten the service period.
3. Geotubes installed on mudflat might experience settlement without proper planning and design.

Table 2.8: Challenges, objectives, solutions and outcome of case studies.

Project Location	Challenges	Solution	Objectives	Outcome
Teluk Kalong (sandy coast) Year 2006	Surface of the existing seawalls experienced uneven settlement due to sediment erosion.	Installed 500 m geotextile tube as submerged dyke.	Coastal protection and beach nourishment.	87,317 m^3 sediments nourished naturally. Increased potential value of beach front.
Pantai Batu Buruk (sandy coast) Year 2008	Profile of sandy beach was steepened and affected recreational facilities and structures.	5 km geotextile tubes were installed, 150 m offshore along the beach.	Coastal protection and beach nourishment.	Beach nourishment contributed to gentler beach profile.
Tanjung Piai (muddy coast) Year 2003	Located in National Park, no road can be created for equipment and materials transportation. Mangrove roots lost grip and toppled. Soft substrate can cause settlement of structures.	Geotextile tubes installed as breakwater to encourage regeneration of mangroves.	Provide calmer water surface for mangroves rehabilitation and encourage beach nourishment.	Sediments accumulated shoreward of the breakwater and regeneration of mangroves succeed.
Sungai Haji Dorani (muddy coast) Year 2007	Erosion of coastline due to wave actions. Mangroves toppled due to poor anchorage between mangrove tree roots and the mudflat in liquid form.	Four geotextile tubes of 30 m length were installed with a gap of 0.5 m between each other. Mangrove species, the *Avicennia* and *Rhizophora* seedlings planted at geotextile tubes protected area.	Prevent further beach erosion, encourage beach nourishment and provide calmer water surface for mangroves rehabilitation.	Maximum sediment accretion of 60cm height was recorded. Observed erosion at certain points. *Rhizophora* could not survive due to sediment movement and deposition. *Avicennia* survived for two years healthily, before the roots covered by thick mud.

2.6 Geotubes Design

The design codes or references for geotubes are not well established as the they were only used in recent decades, and with limited considerations or researches. Typically, the design of the geotube starts with the planning or exploration phase, just like any other civil projects. At this phase, the environment conditions, i.e. the hydraulic and geotechnical information are established. While, the choice of geotubes is made based on the design data like water depth, structure's height and volume, wave pulsating forces, etc.

Then, the design procedure is follow by the design phase, where the geometric design, safety considerations, and stability design were carried out. First, the dimensions of structure need to be determine, together with the construction methods. Then, the potential failure mechanism and the stability of structures for waves and structure stability need to be assessed. Strength and durability of geotube need to be assessed as well. Design considerations can affect the successfulness of the application of geotextile tubes as coastal defence structures. There are a few concerns that engineers need to take into consideration during the geotextile tube design and will be discussed in the following sub-topics.

2.6.1 Design Methods

Liu and Silvester (1977) described the method to estimate the shape of geotubes by employing the elliptical integrals. The geometry of the filled geotubes can be predicted according to the present of several information which include the pumping pressure, geotube's circumference, slurry density and maximum tensile strength of geotube. Geometry of geotube after consolidation can also be predicted. Kazimirowicz (1994) reported the differential function which can predict the relationship between geotube's height, tensile force in geotube and the hydrostatic pressure.

Carroll (1995) and Miki et al. (1996) employed the analytical methods to describe the geometrical solution to estimate the geometry of filled geotubes. While, by assuming the present of hydrostatic condition in geotube, Plaut and Klusman (1999) presented the formulation for stacked geotubes design.

On the other hand, the advancement in technology allow us to simulate or study the behaviours of geotube under different conditions with appropriate inputs to computer programs. For instance, Cantre (2002) described the application of Finite Element Method (FEM) to simulate the consolidation process of stacked geotube. Malik and Sysala (2011) described the FEM analysis of geotubes filled with several liquids of different densities. While, Iryo and Rowe (2003) reported the FEM analysis of unsaturated non-woven geotubes design, focusing on the hydraulic behaviour of geotubes. Isebe et al., (2008) simulated the optimum shape of geotubes for the protection of sandy beaches by using FEM analysis.

Oh and Shin (2006) described the submerged geotubes used in Korea to protect the shoreline from erosion and described the limit equilibrium method to calculate the factor of safety. While, Leshchinsky et al. (1996) developed the computer program, GeoCoPS which employ the differential equations to describe the relationship between the tension in geotube, pumping pressure, geometry of filled geotube and the consolidation of the filling materials. For minor change in height, the shape of the geotube will remain elliptical. However, the greater change in height of these structures will consequent in a rectangle or flatten tube.

Sometimes, to assist the numerical designs, experiments are carried out. For instance, Seay & Plaut (1998) investigated the impact of length and width to the strength and durability of geotube. While Chew et al. (2003) pre-cut the geotextile, to investigate the reduction in strength of geotube under cyclic forces from different directions.

2.6.2 Design Considerations

There are many aspects need to be taken care during the design of geotubes for coastal protection. Several pre-design considerations include the geological data of site, wave climates, function requirements, construction time, budget, availability of materials, feasibility and impact to the environment. Besides the pre-design considerations, the considerations for the design of geotubes need to be taken care. This study mainly focusses on numerical modelling, hence several considerations in numerical modelling design will be explained in detail.

Numerical modelling allows an insight to the behaviours of geotube acting as the breakwaters. Simulation of the behaviours of structure under different circumstances can assist the engineers in design judgment. The primarily concerns in the design of geotube breakwaters include properties of geotubes, properties of filling materials, characteristics of foundation and wave climates (Cantre, 2002).

2.6.2.1 Properties of Geotube

The mechanical properties of the geotube directly influence the integrity of the geotube breakwater. There are various of dimension and properties of geotubes in the market (Lee et al., 2014). For the application in coastal protection, the retention, permeability, strength and durability are important. Retention ability ensure the retention of the filling materials inside the geotubes and maintain the height and shape of the structures. Permeability allow the water to flow through the geotubes and filling materials. While strength and durability will ensure the longer service period of the defence structures and minimize potential of damage.

The erosion or leakage of the sediment through the geotextile in coastal structures is very crucial consideration in the design. The excessive wash out of these sediments will result in the damage of structures or the excessive deformation of the structures (Cantre, 2002). Hence, the opening of geotextile must be smaller than the sediment size of filling material to effectively retain the sediment inside the tube. In many cases, sediment leak out during the service period instead of during the hydraulic filling process. Sediment leakage lead to severe deformation of geotube and cause the loss of its function to protect the shoreline.

The recommended design for the retention criteria of the closed geotextile structures are, $O_{90} < 5\ D_{10}\ C_u^{1/2}$ for stationary hydraulic load or current; and $O_{90} < 1.5\ D_{10}\ C_u^{1/2}$ for dynamic hydraulic load or wave impact (Pilarczyk, 2000). O_{90} is the average pore size of geotextile where 90% of the sand (> 60 μm) remain on it, D_{10} is the sieve size where 10% of sand materials passes, and C_u is the uniformity coefficient of the sand (D_{60}/D_{10}).

The geotextile must have sufficient strength and strong seam in order to resist the great pressure during the slurry pumping process that exert the most pressure on the tube (Bezuijen & Vastenburg, 2012). Generally, the woven geosynthetic have a higher tensile strength than strain; while non-woven geosynthetic structure opposite. The Young's Modulus, E or the tensile stiffness, J of the geotextile can be derived from its maximum allowable tensile load, T_{max} and corresponding strain, as shown in Equation 2.2 and Equation 2.3. Table 2.9 shows the tensile strength and corresponding strain for various geotextile types.

$$E = \frac{T_m}{\varepsilon_m \cdot t_g} \tag{2.2}$$

$$J = E \cdot t_g = \frac{T_m}{\varepsilon_m} \tag{2.3}$$

where,

t_g = Thickness of the geotextile fabric (m)

ε_m = Maximum strain of the geotextile

T_{max} = Maximum allowable tensile strength of the geotextile (kN/m)

Table 2.9: Tensile strength and corresponding strain for various geotextile types

Geotextile Types	T_{max} (kN/m)	ε_m (%)	J (kN/m)	$T_{max} \bullet \varepsilon_m$ (kN/m)
Polyester (PET)	100-1600	8-15	870-16000	8-210
Polypropylene (PP)	40-300	10-15	320-2400	4-45
Polyethylene (PE)	20-50	20-30	80-200	4-15

The durability of geotextile structures governs the lifespan of the geotube. The lifespan of these coastal structures is greatly affected by several common factors, include UV radiation, chemical and biological damage and mechanical damage. Hence, to select the appropriate geotextile materials, there are several aspects the engineers should look into. These includes the construction method, duration of the service period, stress or impacts during the service period, exposure to the ultraviolet (UV) radiation and UV stabilizer used, and the aggressiveness of environment (pH of environment, organism in soil, etc.).

2.6.2.2 Properties of Filling Materials

The physical properties of filling material are also very crucial in geotube breakwater design. The soil types, density and particle size are several main factors that directly affect the final geometry of structure. The particle size of filling material should be greater than the apparent opening size of the geotextile to ensure the sediments are encapsulated inside the tube and maintain the geometry (Leshchinsky et al., 1996; Pilarczyk, 2000). Besides, the filling materials used must be of the appropriate size and shape. If the filling materials have sharp edges, the abrasion between the materials and the geotextile will lead to damage of the structure, followed by spilling of fills and structure failure. In majority of the geotube breakwaters project, river sand or dredged beach sand are popular choices to fill the geotube breakwaters. Particle size gradation curve is important to determine average size of the fill sediments (Bezuijen and Vastenburg, 2012).

2.6.2.3 Properties of Foundation

In many numerical studies (Leshchinsky et al., 1996; Mike et al., 1996; Cantre, 2002; Chu et al., 2012), foundation properties are neglected in order to simplify the models. For example, the geotubes often assumed to be rested on rigid and not deformable foundation. However, in reality, there are different types of subgrade in coastal areas and some of them are weak foundation with low bearing capacity, especially for the muddy beaches.

When coastal protection structures (such as breakwater or groins) are construct on these weak subgrade, excessive settlement can happen and lead to the reduction of structure's height or even structural failure. Thus, assuming the foundation are very stiff or has limited movement can overlook the foundation factors that influence the performance of geotube breakwaters. In our study, field measured parameters of soft and deformable muddy shore were employed to simulate the soft foundation.

2.6.2.4 Stability

The stability of structure and the foundation settlement are important considerations in geotube designs. However, there are currently insufficient design standards available with regard to structural stability of geotubes on soft muddy beaches. Oh and Shin (2006) suggested factor of safety calculation formula to assess the stability of the geotube under different wave heights and foundation conditions. The factors of safety against sliding, overturning and bearing capacity are described in detail in Section 2.6.4.

In many cases, geotubes are placed nearshore as breakwaters to protect the shorelines from erosion. However, when incoming waves hit on the geotubes, there are high intensity forces exert on the tube. These hydrodynamic pulsating forces, F_{hp} can be calculated from the Hiroi's empirical formula (Goda, 1995). Hiroi's formula assumed rigid, rectangular and uniform cross-sectional area of coastal structure and zero initial water level. However, geotube has elliptical shape instead of rectangular.

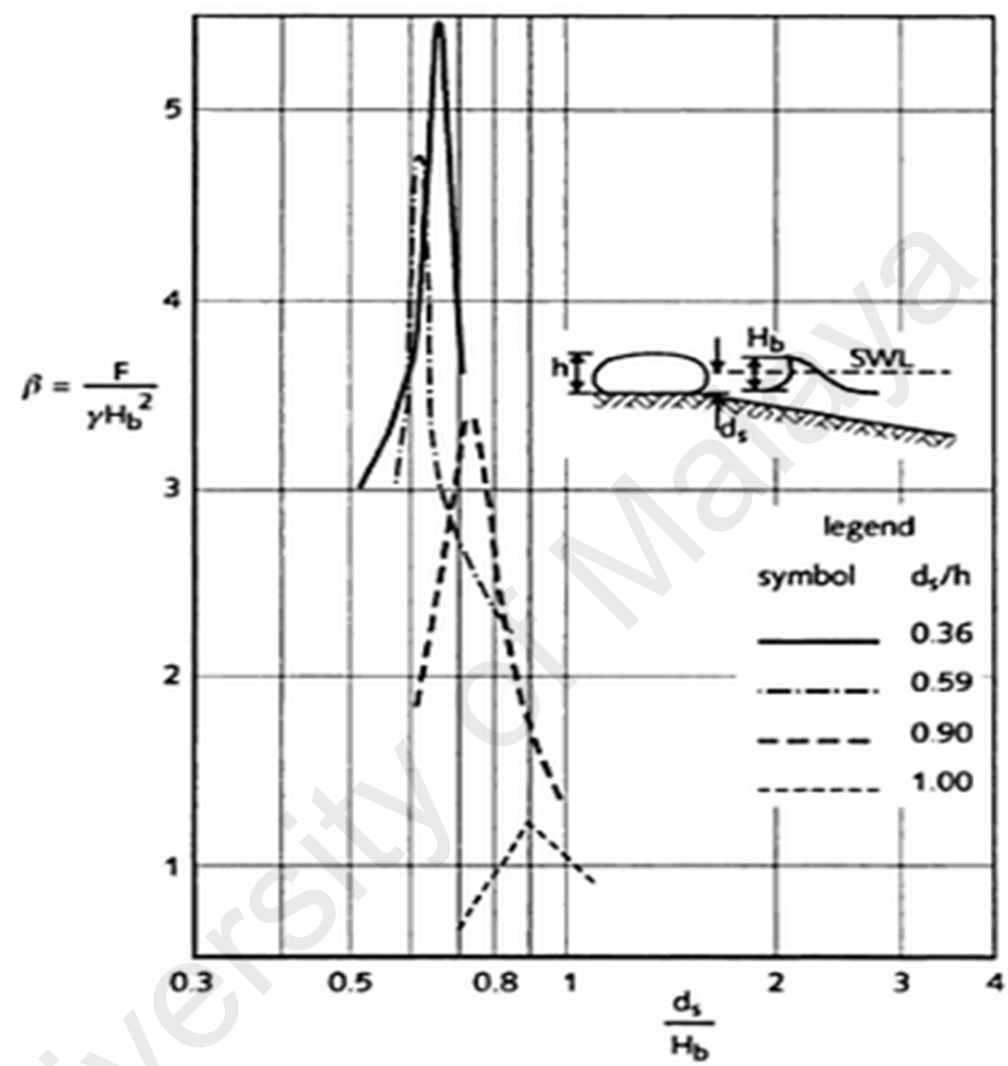


Figure 2.11: Coefficients for wave forces calculation (Liu, 1981)

Liu (1981) modified the Hiroi's equation to suit the application to geotube, where oval shape of structure was considered (Pilarczky, 2000; Koffler et al., 2008). The coefficient of the impact forces hit on the geotube, under different water depth and waves were investigated. Equation 2.4 expressed the modified equation. Figure 2.11 shows the plot of results, which can help us to determine the wave coefficient for the wave forces calculation.

$$F_{hp} = \beta \gamma_w (H)^2 \tag{2.4}$$

where,

F_{hp}	=	Hydrodynamic pulsating force (kN)
H	=	Height of geotube (m)
γ_w	=	Unit weight of sea water (kN/m^3)
β	=	Empirical coefficient related to the height of geotube, wave height and initial water height

2.6.3 Failure Mechanisms of Geotube Breakwaters

Potential failure mechanisms of the geotubes are very crucial aspects to be considered. Most of the time, the failure of the structures cause by overlooking the failure mechanisms during the design phase. The reasons for geotube failures include the insufficient and inaccurate data, mistakes in modelling or simulation, improper construction procedures, inappropriate materials used, vandalism, etc. (Khalilzad & Gabr, 2011).

The common failure mechanisms of the geotubes include settlement, sliding, and overturning. Settlement of geotube is the lowering of the structure's height due to foundation settlement or filling materials consolidation. Structural settlement is caused by large structure's weight and hydrodynamic loads. Sliding or overturning of geotube are displacement of structure, caused by the inadequate resistance of the geotubes to the hydrodynamic loads. Vandalism of geotubes that caused the filling material spillage will consequence in the damage of geotubes and loss the function as coastal protection (Alvarez et al., 2007).

Generally, the failure mechanisms of the geotubes can be grouped into three main categories, which is inadequate stability, inadequate strength and loss of filling materials. The inadequate stability is mainly due to the wave impacts and the geotechnical properties of the stratum. The inadequate strength is normally related to the mechanical properties of filling materials, the durability of the geosynthetic fabric and additional protection of the geosynthetic structures. While the loss of filling materials is caused by the size of the filling sediments or the pores size of geosynthetic fabric.

2.6.4 Safety Factors

Due to the insufficient experiences in the application geosynthetic structures, probabilistic method cannot be used reliably. However, we can always assess the factor of safety of the structures regarding load and strengths. Since there is no reference or guideline to recommend the overall factor of safety, the industry normally adopt FoS of 1.2 for short term prediction and 1.4 for long term prediction. Equation 2.5 to Equation 2.9 are several formulations to assess the stability of geotube under waves, suggested by Oh & Shin (2006) and Bezuijen & Vastenburg (2012).

(a) ***Stability in wave (Bezuijen & Vastenburg, 2012)***

$$\frac{H_s}{\Delta_t \times D_k} \leq 1.0 \tag{2.5}$$

where,

H_s = Significant wave height (m)

Δ_t = Relative density of geotube (kg/m3)

D_k = Length of geotube if the geotube is parallel to the wave direction, or width of geotube if the geotube is perpendicular to the wave direction (m)

(b) ***Factor of safety for sliding (Oh & Shin, 2006)***

$$\mathrm{SF_{sliding}} = \frac{\mathrm{F}}{\mathrm{P_h}} = \frac{\mathrm{P_v} \times \tan\emptyset'}{\mathrm{P_w} \times \mathrm{h_{GT}}} \tag{2.6}$$

where,

F = Vertical force (kN)

$P_{horizontal}$ = Horizontal force (kN)

P_v = Overburden pressure and gravity weight of geotextile tube (kN/m^2)

P_w = Hydrodynamic pulsating load (kN/m^2)

h_{GT} = Effective height of geotextile tube (m)

ø' = Interface friction angle between geotextile and base sand (˚)

(c) ***Factor of safety for overturning (Oh & Shin, 2006)***

$$SF_{overturning} = \frac{M_R}{M_o} = \frac{P_v \times \frac{B'}{2}}{P_w \times \frac{h_{GT}}{2}} \tag{2.7}$$

where,

M_R = Moment preventing rotation (Nm)

M_o = Moment causing rotation (Nm)

B' = Width of equivalent rectangular shaped tube (m)

(d) ***Factor of safety for bearing capacity (Oh & Shin, 2006)***

$$SF_{bearing\ capacity} = \frac{Q_u}{Q_a} = \frac{cN_c + (\frac{1}{2})\gamma_s B' N_\gamma}{\frac{P_w}{B' - 2e'}} \tag{2.8}$$

$$e' = \frac{P_w \times h_{GT}}{3F} \tag{2.9}$$

where,

c = Cohesion of base soil (Pa)

N_c, N_γ = Bearing capacity factors by the internal friction angle of saturated base soil

γ_s = Submerged unit weight of base soil (kN/m^3)

e′ = Eccentricity of the hydrodynamic pulsating load

2.6.5 Beach Responses to Geotube Breakwaters

Coastal defence structures such as breakwaters, groynes and sea walls have been utilized as protective structures to avoid shoreline erosions and encourage natural nourishment of beach sediment (Seiji et al., 1987; Silvester and Hsu, 1997; Dean and Dalrymple, 2001; Lamberti et al., 2005; Ranasinghe and Turner, 2006; Burcharth et al., 2007). Prior construction of these coastal structures, it is important that the designers or engineers have an understanding or estimation of the shoreline responses to the structures. There was case (Khalilzad and Gabr, 2011) where coastal structures led to unintended erosion issue due to the lacking in the understanding of magnitude and mode of shoreline responses. Therefore, understanding in the shoreline responses can assist the design of the coastal defence structures.

The shoreline response to the coastal structures can be simulated with reliable accuracy, adapting the depth-averaged (2DH) morphodynamic models with both hydrodynamic and morphodynamic solutions. (Zyserman et al., 2005; Martinelli et al., 2006; Zanuttigh, 2007).

The shoreline responses or processes under the complete beach profiles, wave climates and other environmental parameters can be simulated by using the morphodynamic models. Nevertheless, the computational cost of these detailed and precise analyses is high and is not feasible or practical to simulate a large number of simulations. (Ranasinghe et al., 2006) reported the feasible approach where the circulation patterns in the lee of the coastal structures are used to represent the shoreline responses like accretion and erosion. This hydrodynamic modelling approach requires less computational cost as compared to the morphodynamic modelling. Therefore, a greater number of simulations can be carried out.

There are many parameters that can influence the shoreline response to the coastal structures. For example, the geometry of structure, the distance from the shoreline, the gap between structures, wave properties, tidal range, just to name a few. Hence, if every variable mentioned need to be tested with different values, the total simulation cases are too many and not practical to compute by using computational expensive morphodynamic model.

Ranasinghe et al. (2006) described the simpler approach by using hydrodynamic model to compute the wave circular patterns in the lee of the coastal structure, to indicate the potential sediment erosion or accretion causes by the structure. His study shows nearshore current circular patterns can be used as indicators to identify the shoreline erosion or accretion.

2.6.6 Construction Concerns

Extra care and concerns have to be taken during the installation of the geotubes to avoid damage of structures during the installation. The construction of geotube as coastal structures required additional care in two aspects, the site preparation and the filling process.

Geotubes expose to many environmental influences such as the ultraviolet radiation, chemical attack, vandalism just to name a few. The designers should be aware that these factors might affect the performance of geotextile tube and hence have to pay attention on these factors. Several limitations of the application of geotubes are included abrasion resistance, puncture resistance, degradation of geotextile under marine environment and ultraviolet radiation and lacking of design guidelines. Table 2.10 shows the precautions needed during the site preparation and installation works.

From the experiences in previous projects (Rajabian et al., 2012), the high tensile strength geotextile tubes were often placed nearshore as nearshore breakwaters and the public is able to access to the tube. Therefore, there are many cases were the geotubes were vandalized by using sharp edge objects like knife. While, some geotubes were torn during the delivery to the site, or during the installation work.

Another factor that affect the geotextile tubes is the ultraviolet radiation. The laboratory experiment suggested that the geotextile tubes have ultraviolet radiation resistance up to 50 years. However, there is no field data that can support the laboratory results so far. Therefore, it might be more appropriate to design the geotextile tubes for a period of 10 to 20 years. Nevertheless, the placement and planning of the geotubes will greatly increase the service life of the structures. For instance, submerged tubes or cover the tubes with stones, sand or marine growths, can effectively reduce the threat from the ultraviolet radiation.

Table 2.10: Precautions during the constructions of geotube breakwater

Construction Phase	**Precautions**
Site preparation	• Removed potential dangers (sharp edge objects) for the geotextile fabrics. • Secure the tubes with weight or ropes before pumping in fills. • Place temporary guide near the placement location of the tube
Filling process	• Filling materials must be greater than the opening of fabric. Fills that are too fine will be difficult to settle in the tube. • Shorter filling time can avoid mishaps. • Allow water to flow out efficiently through outlet port to prevent excessive pressure in geotube that will causes failure. • The pumping pressure must be applied carefully as after certain height; the slurry will transport at the top part of the tube. If the velocity of slurry drop, sediments might settle and block the transport path. Hence, the geotube will experience non-uniform cross section.

There is no standard to shows the best method of filling geotextile tubes. However, there are increasing numbers of contractors and labours have the experience in the installation works of geotextile tube. The quality of the construction works is very dependent on the skill and experiences of contractors. For example, the alignment of the geotube need to be secured by holding the tube along the placement location during filling work. This is because the filling process might cause the tube to twist, or the insufficient weight of tube will be displaced due to the waves. Filling materials that will consolidates over time will cause the reduction structure's height and consequent in insufficient resistant to the waves and currents. If the sand in the tube stabilized or consolidated over time, the tube will be flattened and it will be difficult to pump the tube higher. Hence the installation works shall not be stopped prematurely.

2.7 Summary

Development accelerates coastline degradation. Coastal protection is necessary when the erosion of coastline affected public and private properties. Conventional and well-established coastline protection methods include hard engineering structures such as concrete breakwaters, dikes, revetment and dunes. Currently, soft engineering approaches received higher demands recently as they are said to be more environmentally friendly. However, there are room for improvement for soft engineering structures and there is a need to enhance the field practice.

In many literature, deformable characteristics of foundation and impacts from waves were ignored in simulation models, in order to simplify analysis. However, geotube coastal structure's failures were reported and majority happened due to instability issues. It is necessary to consider the properties of foundation and wave in the design of geotube breakwaters. Nevertheless, typical filling materials for geotubes are river sand. Further studies on alternative filling materials can reduce reliance on the use of sand to fill geotubes. Thus, improve sustainability and environmentally friendly aspects of the application of geotube breakwaters.

The layout of geotube breakwaters affect sediment activities and hydrodynamic responses. Simulation of hydrodynamic and beach responses after installation of breakwaters can be carried out. However, a precise simulation model requires very detailed inputs including bathymetry, wave conditions, wind climates, tidal data, temperature, humidity, sediment size, amount and sources of sediment supply. The incorporation of all relevant data will affect the economics of a project and complete the design. Therefore, development of a simple yet effective prediction model is of high interest, where minimal parameter inputs can provide the required beach responses for design and construction purposes.

CHAPTER 3: METHODOLOGY

3.1 Introduction

The study looks into application of geotube as a muddy coastline breakwater to prevent beach erosion while encourages natural sediment nourishment. After choosing the study site, field study and site investigation were carried out in order to collect information on geotechnical, hydrodynamic and wind. Relevant document, photo and data such as sediment elevation before and after the installation of geotube breakwaters, mangroves growing conditions, settlement and damage of breakwaters, were collected from Forest Research Institute Malaysia, FRIM and University Malaya, UM. Long term wind and wave data were collected from Malaysian Meteorology Department.

Properties of geotube and filling material, i.e. coastal mud, sand and quarry dust were determined through laboratory tests. Results obtained were used as input parameters for computer modelling works. The suitability of three sediment types as geotube breakwater's filling materials were determined. Detailed descriptions of the laboratory experiments are stated in Section 3.3.1.

Internal stabilities of geotube breakwaters were analysed through a differential calculus program. Influencing factors of the internal stability were investigated. These factors included type and concentration of filling material, properties of geotubes, dimension of geotubes, tension, pumping pressure, just to name a few.

External stabilities against sliding, overturning and bearing capacity were investigated through numerical analyses by adopting Finite Element Method, FEM. FEM models were used to simulate failure mechanisms and the results were analysed to find the safety factors. Deformable characteristics of muddy foundation was considered.

Change in wave speeds and currents after geotubes installation were studied through hydrodynamic models. Prediction of beach responses such as sediment accretion and erosion, were done through the simple hydrodynamic models. Results were verified with field measured sediment elevations to appraise the sturdiness of the results. Accretion and erosion of sediment in study site were monitored every two months from September 2012 to November 2015. Previous sediment elevation data were collected from FRIM.

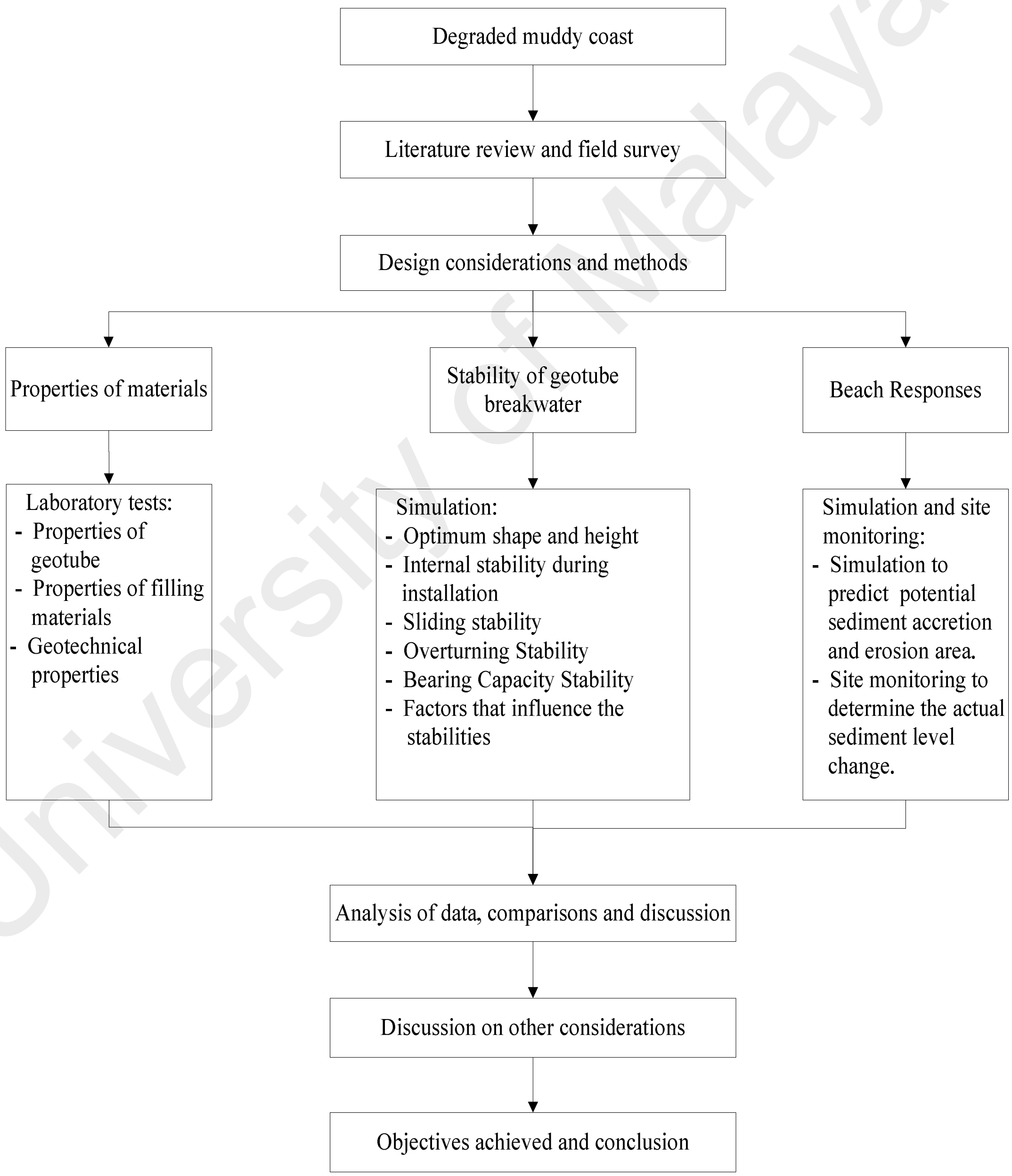


Figure 3.1: Flow of methodology

Considerations on environmental, cost and feasibility of utilization of geotube breakwaters on muddy coast were discussed. Conclusions and recommendations for future research were suggested in Chapter 5. Figure 3.1 shows the overall methodology flow of this study.

3.2 Site Description of Pilot Project

A severely degraded muddy beach along Sungai Haji Dorani, SHD located on west coast of Peninsular Malaysia was chosen as study site. SHD located at the distinct of Sabak Bernam, Selangor, Malaysia with coordinate 3°38' N, 101°01' E. SHD's beach is about 120 km from Kuala Lumpur and situates along the narrower channel of the Strait of Malacca, with a length of approximately 2.7 km. There are about 23,000 populations staying nearby. Most of the communities are fisherman and farmers. Mangrove forests used to be the most precious asset for the nearby coastal communities (Hashim et al., 2010). The mangroves able to protect the coastal areas from intensive wave forces, and act as natural coastal barriers and habitats for marine lives.

However, during recent decades, erosion of coastline was accelerated due to development in agriculture, aquaculture, fishery and tourism industries. The mangrove forests experienced the worst scenario, i.e. toppled and died due to sediment erosion. Thus, SHD is a good study site for the investigation and evaluation of the geotube breakwater's performance. Currently, the SHD muddy beach is classified under the critical category of eroded coast by DID (2006), this indicates the near shore facilities are in threat of loss or damage due to the erosion. Besides anthropogenic factors, natural factors such as storm surge, waves and tides also contribute to the destruction of mangroves and severe erosion of the muddy coastline because soft sediment on muddy coast are very vulnerable to erosion.

SHD is a good research site for coastal management study due to the critical coastal degradation. Coastal protection structures are the typical approach for the protection of shoreline from further erosion and reduce the degradation of mangroves. During the 1970's, first mitigation method carried out by Department of Irrigation and Drainage Malaysia was to constructed coastal dikes to counter tidal inundations and intensive hydrodynamic waves (Hashim et al. 2010). However, mangroves protection and rehabilitation were not emphasised in the earlier coastal protection projects as authorities generally did not appreciate their values.

Figure 3.2: L-block concrete breakwaters in SHD

In the year of 2008, University of Malaya introduced an innovative shoreline protection named L-block concrete breakwaters to assist the rehabilitation of mangroves. The concrete breakwaters were 30 m in length for each segment and were installed at 2.5 m gap between segments as shown in Figure 3.2. Total length of the breakwaters is about 70 m. Mangrove seedlings required half a year of nursery before being planted into the protected area. Protection from the L-block concrete breakwaters encouraged active sedimentation, approximately 0.7 m elevation after two years of construction. However, the rehabilitation of mangroves was not successful, more than 90% of the mangroves were dead or displaced by the wave currents within a year.

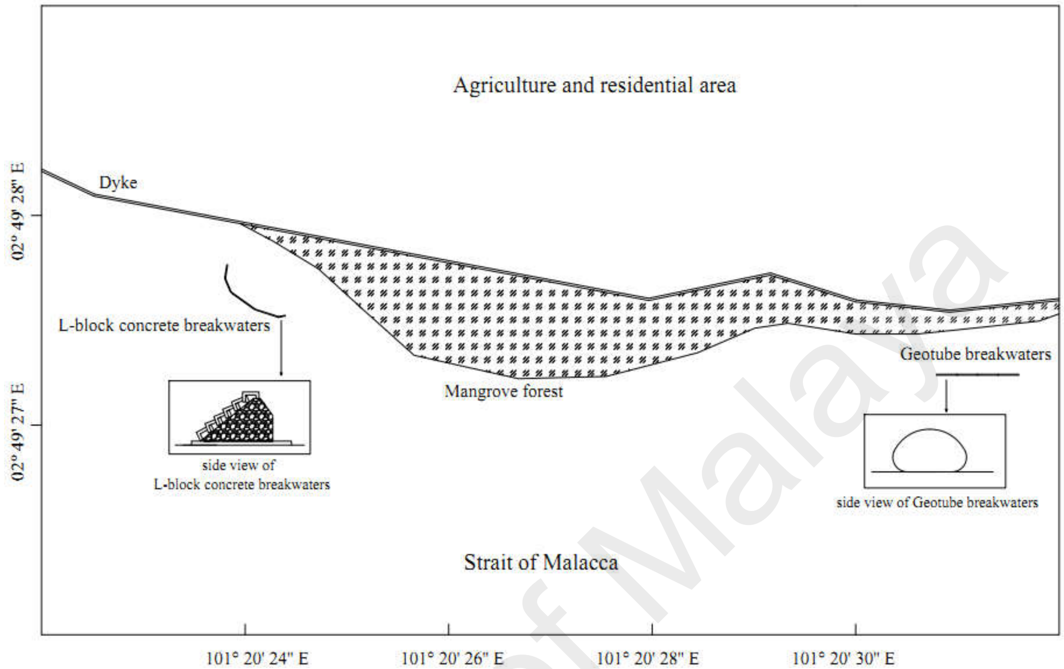


Figure 3.3: Geographic positions of geotube and L-block breakwaters

While, geotube breakwaters installed by FRIM is focus of this study. In year 2007, FRIM conducted a pilot research to rehabilitate degraded mangroves and coastline by using geotube breakwaters. The purposes of the geotube breakwaters are to reduce wave impacts and assist the grow of mangroves seedlings. Four stretches of geotubes, each 50 m in length were installed parallel to the coastline. The structures were located 100 m from the beach, with a gap of 0.5 m between geotubes. After one year of geotubes installation, FRIM adapted control planting technique to plant three species of mangroves (Avicennia, Mucronata and Rhizophora). Mangrove seedlings were raised in nursery, each inside a coir log before planting on the mudflat. Figure 3.3 shows the locations of geotube breakwaters and L-block concrete breakwaters.

Biggest difference between the L-block concrete breakwaters and geotube breakwaters are structure's layouts, gap in between stretches and the mangrove planting and monitoring techniques. Geographic positions and cross-sectional view of the geotube breakwaters and L-block concrete breakwaters are as shown in Figure 3.3.

3.2.1 Geotechnical Information

Coastal sediments in SHD are mainly greenish grey deposits consist of high silt contents. According to Hashim et al. (2010), these sediments sourced from rivers nearby such as Bernam River and Perak River. Chuan and Cleary (2005) stated that construction of Temenggor dam reduced sediment discharge from Perak River to coastal regions down drift, which included the SHD's coast. The available scanty data do not directly indicate the actual causes of coastline recession in SHD. However, it can be noticed that the coastline erosion has led to sediment erosion along the muddy beach and resulted in the death of mangroves. Mangroves toppled and dead due to the sediment loss. As a consequence, the beach was unprotected by mangrove forest and the muddy beach was exposed to direct wave cycles. In order to prevent, stop and solve the erosion issue, SHD needs proper coastal management to minimize threat and damage from coastline degradation to socioeconomic properties.

3.2.2 Wind and Wave Information

SHD's beach is sheltered by Sumatera Island from high intensity waves incoming from Indian Ocean. According to Meteorological Department of Malaysia, waves in SHD are mainly generated by winds. Northeast monsoon and Southwest monsoon are two main monsoons that influence wave characteristics, from November to March, and from May to September, respectively. While, transition period of the two monsoons (April to October) heavy rainfalls normally occur (Jamaludin and Jemain, 2007). The annual rainfall in SHD is approximately 2600 mm. Diurnal temperature ranges between 23°C to 33°C, and relative humidity ranges between 75 to 95%.

The wind observation data from 2012 to 2015 were collected from Meteorological Department of Malaysia are as shown in Figure 4.2 to Figure 4.5. Results show the most frequent generated waves are from the SW direction and WNW direction, which appear during Southwest monsoon and Northeast monsoon respectively. The winds from SW direction are not more than 15 knots (or 7.71 m/s); while winds from WNW direction have the greatest magnitude, range between 10 to 20 knots (5.14 to 10.29 m/s). The observation results also show that significant wave height range between 0.5 to 1.0 m throughout the year, with maximum tidal range of 3.2 m and wave periods of 3 s to 9 s.

3.3 Simulation of Geotube Breakwaters for Muddy Coastline Protection

3.3.1 Determination of Model's Input Parameters

Environmental and geotechnical conditions in every coastal region are unique. Different mitigation measures adapted for coastal erosion protection, in different regions, show different performance and effectiveness. Design of geotube breakwaters for muddy coastal protection involves several important considerations. For example, properties of foundation, filling materials and geotube. These properties or parameters need to be determined before other design or simulation phases, through field measurements or lab tests.

3.3.1.1 Geotechnical Information

Geotechnical data and parameters such as foundation sediment size, types, moisture contents, Atterberg Limits, consolidation tests and Vane Shear Test were determined. The Atterberg Limit Test were performed as a measure of the critical water contents and to evaluate the potential changes in behaviour and consistency. Due to the high water content in the muddy soil, the investigation of Atterberg limits is crucial. The Atterberg Limit test is performed in accordance with ASTM D4318-10e1-Standard test methods for liquid limit, plastic limit, and plasticity index of soils. Then, types of the soil sample were classified according to the ASTM D2487-11, Standard practice for classification of soils for engineering purposes (Unified Soil Classification System).

The shear strength of muddy foundation is important. Low foundation shear strength had higher risk for excessive foundation settlement and structure failure. However, muddy beach in study site was too soft for a proper undisturbed triaxial test's sampling. The alternative approach, namely Vane Shear test was carried out instead. Vane Shear Test can determine the in situ undrained shear strength. The vane shear test is a geotechnical investigation technique for estimation of undrained shear strength of fully saturated clay soil without disturbance. Vane shear test was carried out in accordance with ASTM D2573/D2573M-15-Standard test method for field vane shear test in saturated fine-grained soils. The advantages of vane shear test are that the test is quick and simple to provide the shear strength of soil needed as a parameter for simulation. Besides, this method is convenient to test the sensitivity of soil, undrained shear strength of clay, and can be conducted on soft clay area where sampling for laboratory experiment is difficult. Undrained shear strength Su can be calculated by using Equation 3.1.

$$Su = \frac{6\,T_{max}}{7\,\pi\,D^3} \tag{3.1}$$

where,

S_u = Undrained shear strength from the vane

T_{max} = Maximum value of measured torque

D_v = Vane diameter, 50.8 mm.

Measuring the consolidation characteristics of clayey soil in study site is not practical due to the long procedure time. Therefore, disturbed samples of coastal mud were collected using piston sampler from study site and tested in laboratory. One dimensional consolidation test was carried out in accordance with ASTM D2435/D2435M-11, Standard test methods for one-dimensional consolidation properties of soils using incremental loading. This test allows an insight of the magnitude and rate of settlement of the structure or foundation. The consolidation test is important as the result can assist the design of the coastal protection structure and ensure a satisfactory performance. The geotechnical reports are presented in Appendix C.

3.3.1.2 Properties of Geotubes

Basic physical properties of geotube such as the mass, thickness, tensile strength and puncture resistance of geotextile were determined through several index tests. The results from these tests are important parameters for the development of analysis models.

The unit mass of geotube was determined in accordance with the ASTM D5261-10-Standard test method for measuring mass per unit area of geotextiles. The geotextile was sampled from ten different and random locations of the geotube with surface areas of 100 mm^2 for each sample. Then the samples were weighed to obtain the mean average of the mass per unit area.

While, in order to assess the nominal thickness of geotextile, the ASTM D5199-01-Standard test method for measuring the nominal thickness of geosynthetics, was referred. Ten geotextile specimens were cut from various locations of the geotube in order to obtain mean thickness. Specimens were placed on the flat planar surface and a circular presser foot was lowered onto the geotextile for pre-determined time.

Tensile strength of geotube is very important as insufficient tensile strength causes damage of geotube during installation or service phase. In more serious scenario, the loss of integrity of the geotube breakwater or structural failure. Tensile strength of geotextile was determined in accordance with the ASTM D4595-11-Standard test method for tensile properties of geotextiles by the wide-width strip method. Tensile strength and the elongation of the geotextile specimens can be determined through this method. Majority of the geotextile types can be tested by this method. For instance, woven fabrics, non-woven fabrics and layered fabrics.

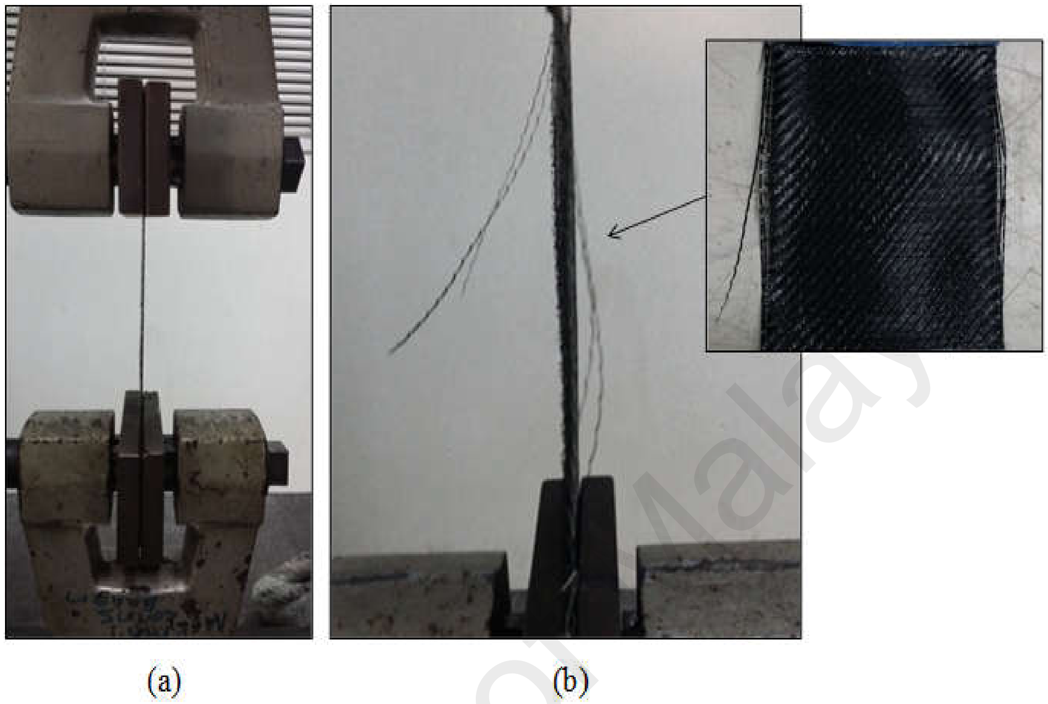


Figure 3.4: Wide-width strip test method for geotextile: (a) specimen before load was applied, (b) specimen ruptured

The apparatus of the wide-width strip tensile test includes a constant rate extension type of tensile testing machine described in ASTM D76-Specification for Tensile Testing Machines for Textiles, clamps that has sufficient width to grip the specimens without specimens slipping or damaging, and jaw faces that has jaw face wider than specimens. Six specimens were taken from different locations of the geotextile fabrics for machine direction test, and same number of specimens for cross-machine direction test. Test specimens were prepared to have a finishing of 100 mm wide by 200 mm long. 50 mm from top and bottom of the fabric were drawn a reference line to indicate the location of the jaw faces' edge. For woven geotextile fabric, the width shall be drawn greater than required width and trim to exactly 100 mm just before the test is carried out.

The distance between the clamps of tensile machine at the start of the test was 100 ± 3 mm. The tensile test machine was turned on after the specimen was properly attached to the jaw faces as shown in Figure 3.4. If specimen breaks at the edge of the jaws, damage in the jaws or slip in the jaws, results shall be discarded. However, according to ASTM D4595, geotextile specimens that experienced slippage in jaws or more than 24% of the specimens break at the location within 5 mm from the edge of jaw, then the jaw can be modified in three ways. First method is to pad the jaw faces, while second method is to modify the surface of the jaws. Thirdly, geotextile can be coated at the jaw areas. For our test, third method is used where geotextile was coated at the jaw face areas.

Tensile strength is the maximum force per unit width which causes the specimens to rupture, or as shown in the Equation 3.2

$$\alpha_f = F_f / W_s \tag{3.2}$$

where,

α_f	=	Represents tensile strength (kN/m)
F_f	=	Maximum force applied to break the specimen (kN)
W_s	=	Specimen's width (m)

Elongation of the geotextile specimens was calculated according to the Equation 3.3 or Equation 3.4.

$$\epsilon_p = (E \times R \times 100) / (C \times L_g) \tag{3.3}$$

$$\epsilon_p = (\Delta L \times 100) / L_g \tag{3.4}$$

where,

ϵ_p	=	Elongation (%)
E	=	Distance along the zero-force axis from the point curve leaves the zero Force axis to a point of corresponding force (mm)

R	=	Tensile test speed rate (m/min)
C	=	Recording chart speed (m/min)
L_g	=	Initial gauge length (mm)
ΔL	=	Change in length of specimens to the corresponding measured force (mm)

Wide pieces of specimens were gripped with the clamps of constant rate of extension type tensile testing machine as shown in Figure 3.4 (a). The longitudinal force was applied until the specimens ruptured as shown in Figure 3.4 (b). The tensile force, elongation, and Young's modulus were recorded and reported in Section 4.2.2.

(a) (b)

Figure 3.5: Test for static puncture strength of geotextiles using a 50-mm probe; (a) applying load on specimen, (b) specimen ruptured

Static puncture strength indicates the force required to puncture the geotube material. Static puncture strength of the geotextile used in this study was determined in accordance with ASTM D6241-14-Standard test method for static puncture strength of geotextiles and geotextile-related products using a 50-mm probe. CBR test is commonly used to assess the resistance of geotextile to aggregate penetration, particularly in separation applications. The specimens were clamped on circular plate without tension. Force was exerted in the centre of specimen by using the steel plunger until rupture. The large size of the plunger exerted multidirectional loads on the specimens. Maximum force recorded is the puncture strength of geotextile. Figure 3.5 (a) shows the apparatus set up and Figure 3.5 (b) shows the rupture specimen after puncture force was exerted by steel plunger.

Retaining the filling material inside the geotube containment unit ensure the geotube breakwater maintain the shape and height as designed for a longer duration. For coastal defence application, geotube is required to retain the fills while allowing water to pass through. In order to fulfil this requirement, the apparent opening size of geotube was assessed. ASTM D4751-12-Standard test method for determining apparent opening size of a geotextile, was referred to obtain apparent opening size of the geotextile. The apparent opening size distribution is plotted on graph and maximum allowable opening size, O_{95} was determined. The maximum allowable opening size is an important parameter to be used to assess the filtration capability of the geotube. Size of filling materials should be greater than O_{95} for effective filling.

Water permeability of geotube was assessed in accordance with ASTM D4491-15-Standard test methods for water permeability of geotextiles by permittivity. This experiment determined the water pass through geotextile sample in normal plane. High internal pressure can be exerted on geotube breakwater when water cannot low out effectively. The internal pressure can cause structure failure which need to be prevented. The summary of the geotube's physical properties is stated in Table 4.1.

3.3.1.3 Properties of Filling Materials

One of the major concerns in geotube design is the determination of the properties of filling material. The sizes, shapes and density of the sediments directly influence the geometry and performance of geotube breakwaters. For example, sharp-edged particles such as gravel or rock could be abrasive to the geotextile. Damage of the geotextile can directly shorten the survivability of the geotube breakwaters.

The sediment size of filling materials should be greater than the apparent opening size of geotextile to prevent leakage of sediment through the geotextile opening. Besides, larger particles have good permeability and do not easily clog the textile. With good permeability, sediment consolidates quicker during installation procedures. Finer sediment might take several months to fully consolidate due to their low permeability and clogging of geotextile openings. Studies (Fowler et al., 1996; Lawson, 2008; Kriel, 2012) showed it is not practical to refill the consolidated geotube due to high cost and time consumptions. Besides, refilling the geotubes, especially if the filling material consolidates at a very slow rate can cause reduction of final height that occurs only after several months. Excessive reduction in the geotube's height causes loss of its function as breakwaters.

The filling materials tested in this study were included the coastal mud, sand and quarry dust. Coastal mud samples were obtained from study site in SHD, representing the commonly used on site dredged sediments. While, sand samples were river sand obtained from local supplier near SHD, representing the most typically used filling material. While, quarry dust samples were obtained from a local quarry, representing the alternative filling material suggested in this study. Quarry dust is the dust or by-product generated from rock crushing process to produce aggregates, in our case, granite crushing. Aggregates are used for construction purposes but quarry dust is often treated as waste materials. Analysis of the particle sizes of these materials was carried out in accordance with the ASTM D422-07-Standard Test Method for Particle-Size Analysis of Soil. The particle size distribution curves of the filling materials were as shown in Figure 4.1. Density of soil samples was determined in accordance with ASTM D7263-09-Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens. Detailed laboratory reports of the determination of physical properties of sediments can be found in Appendix A1.

3.3.2 Analysis of Internal Stability of Geotube Breakwaters

Recent schemes of coastal defence structures encourage solutions that bring minimal negative impacts, damage or disturbance to the surrounding environment. Geotube breakwaters are good alternative to conventional hard engineering breakwaters. Installation procedures of geotubes are simple and speedy. The light weighted, flat and empty geotubes are easy to transport to desired location. Slurry is pumped into geotube to fill up the structure to desired height. However, design guidelines for the application of geotubes as breakwaters are not well developed. There are very limited projects and observations were reported or published. Many projects using geotubes as breakwaters were carried out based on judgment of experienced engineers without proper guidelines. Therefore, foretelling performance of geotube breakwaters is not a simple task, especially when these structures are placed on deformable foundation, such as SHD's mudflat.

The advancement in computer technology allows the engineers to predict and foresee the behaviours and performance of geotube breakwaters through simulation. In Section 3.3.2.1, prediction of internal stability of filled geotube was done by using GeoCoPS program which employs differential calculus. Several internal stability aspects included tension in geotube, pumping pressure and final geometry of geotube can be predicted.

3.3.2.1 Formulation and Description

The GeoCoPS 3.0 program adapts differential calculus in the determination of geotube's geometry, pumping pressure and tensile strength (Folwler, 1995; Pilarczky, 1994). The behaviours or geometry of geotube breakwater can be calculated when different slurry types were filled into it.

The formulations employed capable to solve relationship amongst geotube's circumference, L, unit weight of slurry, γ, height of the tubes, h, circumferential tensile force, T, and pumping pressure, P_o. The γ and L of geotubes can be easily determined, hence reduces the number of variables in the models. By using Equation 3.5, γ can be calculated. While, L is decided by engineers at the initial stage of design, normally based on required design height and readily available size of the geotubes. Geotubes are made in various sizes and length. Hence, engineers can select geotube's dimension based on the needs for the project and available sizes. Cost for custom made geotubes is higher than the ready-made or readily available geotubes.

$$\rho_m = 100/[c_w/\rho_s + [100 - c_w]/\rho] \tag{3.5}$$

where,

ρ_m	=	Density of slurry (kg/m^3)
c_w	=	Concentration of solids by weight in the slurry (%)
ρ_s	=	Density of the solids (kg/m^3)
ρ	=	Fluid density (kg/m^3)

Besides the parameters mentioned above, consolidation and final height of geotube are also important. Excessive consolidation of geotubes affects the structure's final dimension and function as coastal breakwaters. Final height estimation after consolidation requires specific gravity of sediments, Gs and saturated unit weight of consolidated fill, γ_{final}. Formulation adopted in the program is based on balanced state of pressurized and filled geotube (Leshchinsky, 1996). The formulations were adapted from Liu (1981), and Kazimierowicz (1994) where the geometry, P_o and T in the geotube were solved. Several assumptions adapted were included:

1. Solutions assumed two dimensional problems with uniform material and cross section along the geotube structures.
2. Weight per unit length of empty geotubes were negligible, as compared to the weight of filled geotube.
3. Geotextile fabric was flexible and thin.
4. Hydrostatic stresses were considered as slurry were used to fill the geotubes.
5. No shear stresses were considered between geotube and filling material.

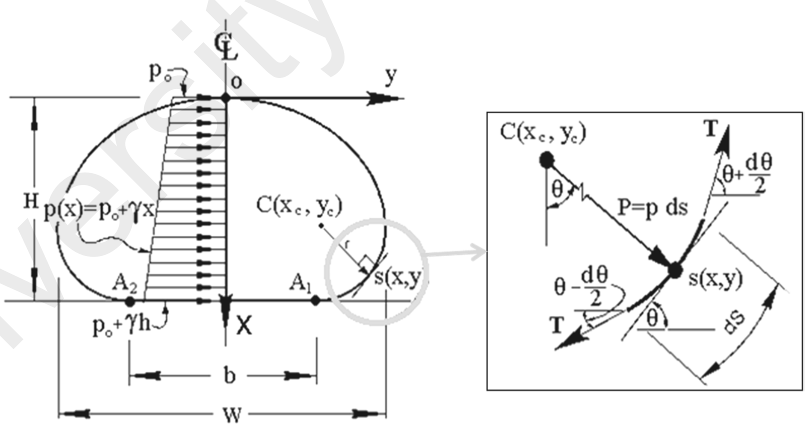


Figure 3.6: Cross sectional view of geotube with convention and notation (adapted from Leshchinsky; 1996)

Figure 3.6 shows the symmetrical geotube's cross section with maximum height, H and maximum width, W. Contact area between muddy foundation and structure is called width of contact, b. Unit weight of slurry is γ, while P_o is the pumping pressure. L is circumference of geotube and r is radius of curvature. Equation 3.6 shows the calculation of hydrostatic pressure at any depth, x which caused by slurry.

$$p\ (x) = P_o + \gamma_x \tag{3.6}$$

The function y = f(x) represents geotube's geometry. As shown in Figure 3.6, the contact point, S (x,y) has the radius of curvature, r from the centre point, C (xc, yc). The value of r and C differ along the y(x). Consider the forces on the arc length, d_s of geotube at S as showed in Figure 3.6. Assuming the solution is two dimensional and the shear stress between the slurry and geotube were ignored, T is constant along the circumference of the geotube.

$$r\ (x) = T\ /\ (p\ (x)) \tag{3.7}$$

Equation 3.7 is valid for any point along the A_1OA_2. T was assumed not transferred to foundation as a result of shear along the geotextile and foundation sediments. Hence T from Equation 3.7 is carry by the geotubes along b. Through differential calculus, radius of curvature, r can also be written as Equation 3.8.

$$r(x) = \frac{[1+(y')^2]^{\frac{2}{3}}}{y''} \tag{3.8}$$

where y' = dy/dx and y"=d^2y/dx^2.

By substituting the Equation 3.6 and Equation 3.8 into Equation 3.7, the Equation 3.9 was obtained. Equation 3.10 is not a closed form formula, hence needs to be solved numerically. Equation 3.9 depicted the relationship between the tube's geometry y(x), T, γ, p_o, and h. Where the x varies between zero and h.

$$Ty'' - [po + \gamma x][1 + (y')^2]^{\frac{3}{2}} = 0 \tag{3.9}$$

$$y = f(x|T, p\,o, h, \gamma) \tag{3.10}$$

In normal practice, unit weight of slurry γ can be determined easily. Therefore, Equation 3.6 depicted the relationship between y and three remaining unknowns, T, P_o and h. There are three types of solution can be carried out. By providing the circumference and T_{ult} of geotube, geometry of tube and the P_o can be found. By providing the desired H_f, T_{ult} and P_o are solved. Meanwhile, providing P_o allows the T_{ult} and geometry of geotube to be solved.

Equation. 3.11 and 3.12 are two constraints, i.e. geometrical boundary condition at point O and flat base length, b.

$$\frac{1}{y'(0)} = 0 \tag{3.11}$$

$$b = \frac{W}{po+\gamma h} \tag{3.12}$$

where W = weight per unit length of the slurry,

$$W = 2\gamma \int_0^h y(x)dx \tag{3.13}$$

Combination of Equation 3.12 and 3.13 gives Equation 3.14.

$$b = \frac{2\gamma}{po+\gamma h} \int_0^h y(x)dx \tag{3.14}$$

Therefore, the circumference of geotube, L can be obtained when b and a single parameter either T, p_o or h are given. Nevertheless, it is more practical to obtain L from the manufacturer. By providing the L, the outcome of the analysis is b. However, practically, it is easier to obtain the L from the manufacturer of geotextile tube. If L is provided, b will be the outcome of analysis.

$$L = b + 2\int_s ds \tag{3.15}$$

where d_s is arc length and ds = $[1+ (y')^2]$ dx. Combining the Equation 3.14 and 3.15 will gives Equation 3.16

$$L = \frac{2\gamma}{po+\gamma h}\int_0^h y(x)dx + 2\int_0^h [1 + (y')^2]^{\frac{1}{2}}\,dx \tag{3.16}$$

In a nutshell, by providing L and either one parameter from T, h and p_o, tube geometry and the other two parameters are found. Lastly, it is necessary to find the axial tensile force per unit length, T_{axial}. Figure 3.7 showed the definition of T and T_{axial}.

Equation 3.17 is formulation to find force P which act on vertical plane signifying the end of a tube resulting from pressurized slurry. The force P is carried by tube in the z-direction, hence the force T_{axial} per unit length is P divided by circumference, L of tube, as showed in Equation 3.18.

$$P = 2\int_0^h (po + \gamma x)\, y(x)\, dx \tag{3.17}$$

$$T_{axial} = \frac{2}{L}\int_0^h (po + \gamma x)\, y(x)\, dx \tag{3.18}$$

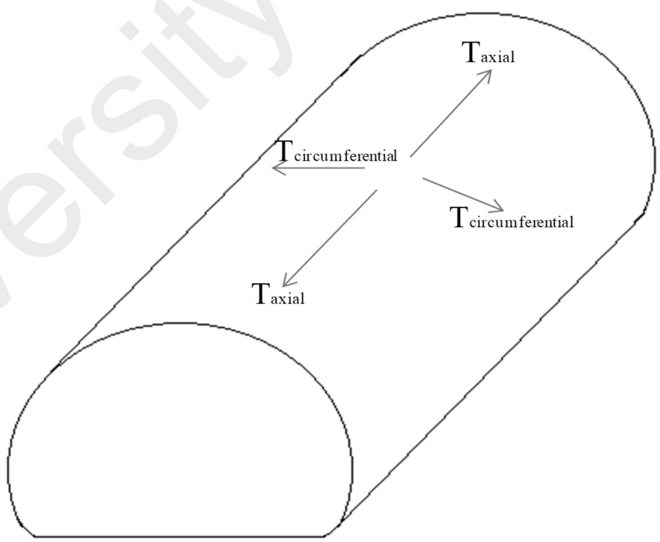


Figure 3.7: Tensile forces in geotube

Generally, T_{axial} is smaller than $T_{circumference}$. Therefore, T_{axial} can be ignored if isotropic strength of geotextile is considered. But in real cases, geotubes are anisotropic because their strength in fill and warp directions are always vary. Many geotubes are anisotropic because during manufacture, different types and number of yarns per unit width are used in each of the principal direction. Thus, it is possible that the $T_{circumference}$ is lower than T_{axial}. T_{axial} should always be considered to ensure the safe design of structures with economic selection of geotextile.

3.3.2.2 Input Parameters

Input parameters are required, in order to solve a simple problem needed to solve the problem. Standard International Units were employed for all inputs. The basic input for this study is listed in Table 3.1 and Figure 3.8. Reduction factors for ultimate tensile strength, seam strength, installation damage, creep, biological and chemical degradation were all set to 1. In this study, three sediment types were considered as the options for geotube's filling materials, i.e., sand, coastal mud and quarry dust. The practicality of geotube as breakwaters is debatable as dredged natural sand is the most commonly used fill for geotube breakwaters. The over-exploitation of natural sand resources through dredging activities caused erosion of the dredging areas and depleting resources of natural sand in the long term (Martinelli et al., 2011, Roberts and Wang, 2012). Hence, in this analysis, quarry dust was considered in terms of the suitability to be used as the alternative filling material for geotubes.

Quarry dust is a by-product of coarse aggregates production. Increasing demand for the coarse aggregates in construction projects had resulted in the larger amount of quarry dust. This material is rarely used in civil engineering projects and yet its' monetary worth has not been fully considered. Therefore, quarry dust is often looked at as a waste and is mishandled to save cost, i.e. dumped in landfills (Erftemeijer et al., 2012). Using the quarry dust as alternative fill materials is able to increase the commercial value and application of this waste, while reducing the demand for natural sand. Hence, utilization of quarry dust to fill the geotextile breakwaters promotes the concept of sustainable material management (Ljungberg, 2007, Sivakumar et al., 2012).

Besides the type of sediment, the sediment concentration of slurry is another factor that affects the final geometry of the geotube breakwater. Generally, the slurry consolidates after being pumped into the breakwaters. If significant reduction in structural height happens, several times of pumping process are required in order to pump the tube to the designed height. The repetition of pumping process is not favourable as it is time and cost consuming. There is no published research work on the influence of sediment concentration in slurry to the geotubes. However, practitioners usually utilize 20% to 40% sediment concentration in the design stage based on the judgment and experience (Leshchinsky et al., 1996).

The program was used to analyse relationship between the sediment concentration to the T, h and p_o of slurry filled geotube breakwaters. During the analysis, three inputs are to be provided; γ, L and h (as structure's height is normally decided according to the site's conditions). The h employed in the model is 1.8 m, and sediment concentration inputs used ranged from 0% to 80%. The program solved and computed the other two parameters, T and p_o, as the solution. The configurations and parameters are as shown in Table 3.1.

Table 3.1: Constants parameters used in the analysis

Parameters	Values/Descriptions
Circumference of geotextile tube	8.6 m
Length of geotextile tube	50.0 m
Safety factor for seam strength	1.0
Safety factor for installation damage	1.0
Safety factor for degradation	1.0
Safety factor for creep	1.0

3.3.2.3 Analysis Models

The analysis study required several parameters inputs in order to produce the results, as shown in Figure 3.8. The circumference and length of geotube which were set constant throughout the analysis, by adapting the SHD's geotube dimensions. The filling materials adapted in the study were included the coastal mud, sand and quarry dust, with dry density of 380 kgm^3, 1400 kgm^3 and 1650 kgm^3 respectively. While submergence conditions employed were (1) no water surrounding at all, and (2) semi submerged to a height of 1.5 m. The sediment concentration in slurry ranged from 0% to 80% in order to determine the influence of the concentration of slurry to the behaviours of geotube breakwaters.

Other than the inputs mentioned above, there were three important inputs which including H, T and P_o. By providing two of these parameters, the third parameters were calculated as the solution. Hence, the discussion on the internal stability were then discussed. The results were representing the geotube breakwaters in study site, muddy coast in Peninsular Malaysia. Therefore, the results can be adapted as a reference to help in engineer's judgment during the design of the similar projects.

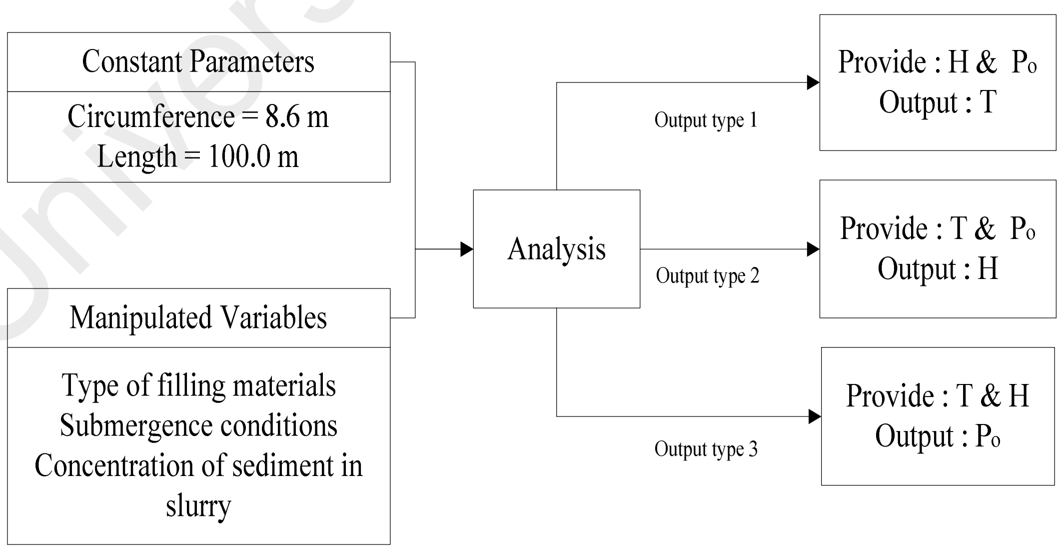


Figure 3.8: Flow chart showing parameters used in the analysis model

3.3.2.4 Verification of Analysis

Verification of the sturdiness of the solution were carried out by comparing the calculation results with an experiment result reported by Liu (1981) and the numerical analysis results reported by Silvester (1997).

Liu (1981) carried out observation of mortar-filled geotubes and traced the geometry of geotube after filled. Circumference of the geotube used by Liu was 1 m. The tube was filled with water and there is no water outside the geotube. The H, b and W obtained were 0.16 m, 0.31 m and 0.38 m respectively. Silvester (1997) carried out numerical analysis to investigate the geometry of filled geotubes and stated that the results were verified with physical model. In the numerical modelling, the circumference used was 3.6 m. Pressure at the bottom of tube were calculated by using Equation 3.2. The unit weight of water and mortar used was 1 and 2 respectively. The agreement between the experiment results from Liu (1981), Silvester (1997) and results from GeoCoPS calculation indicated the reliability of the differential equation solution.

3.3.3 Analysis on External Stability of Geotube Breakwaters

In most of the literature, internal stability was widely discussed and analysed. However, there are very limited published materials regarding prediction or assessment of the external stability of geotube breakwaters subjected to waves. Besides, majority of the studies assumed non-deformable or very stiff foundation. This study analyses external stability of geotube such as stability against sliding, translation, rotation and settlement, by using the Finite Element Method, FEM with respect to the influences of filling materials, foundation, geotubes and wave loads. The deformable foundation, or the muddy foundation were adapted to evaluate the influence of the foundation characteristics to the behaviour of geotube breakwater.

There were various of analysis carried out to assess influence of Young's modulus, cohesion, material's density of materials, the filling percentage, the height of significance waves, foundation properties, etc. Results obtained were compared and discussed with the measured site data.

3.3.3.1 Finite Element Model Development

(a) ***Part module***

There are many modules need to be setting up before simulation of sturdy results. The part module is where the models were sketched out. Due to the great length ratio to width and height of geotube, two dimensional analyses were employed out as it is sufficient to generate the result yet is less computational expensive.

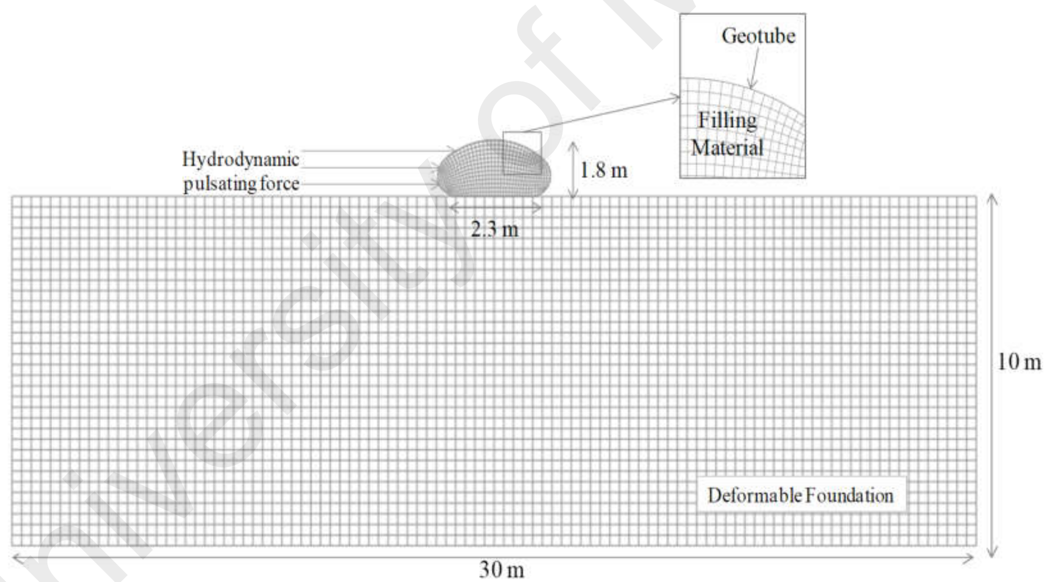


Figure 3.9: Finite element model - parts and meshing

There are three main parts in the simulation model i.e., geotube, filling material and foundation. These parts were modelled and partitioned according to locations of applied load. The element type of geotube is assigned as three nodes quadratic beam in a plane, B22, while the other two parts are assigned eight nodes biquadratic plane strain quadrilateral elements, CPE8. The mesh configurations for these three parts are as shown in Figure 3.9.

(b) ***Property module***

Properties of parts were assigned in the part module. There are three parts in the simulations, i.e. geotube, filling materials and foundation. The geotube's properties were obtained through manufacturer and lab experiments. Properties of foundation were collected and measured on-site. In the models, the assumed saturated weak foundation was assigned cohesion of 20 kPa and dilation angle of 0.1°. Besides, there were two types of filling materials used in the analyses which included river sand and quarry dust. Sand is the most widely used dredged materials to filled the geotubes. While, the quarry dust is a waste material proposed in this study to be used as the alternative options for the conventional fills. The general properties of the filling materials and foundation materials are described in Table 3.1.

The properties of geotube were constant throughout the simulation and were same with the geotube used in study site. The geotube had circumference of 8.6 m, 50 m in length and 0.001 m thickness. The properties of the geotube were described in Table 3.2.

(c) ***Assembly and interaction modules***

After assigning properties to the parts, the parts were assembled. Frictional force of 0.5 was assigned between the geotube and the foundation. While, the filling material and geotube adapted the 'tie' function, assuming they are integrated and will remain touched even if deformation happens.

(d) ***Step module***

In the step module, forces were defined in different steps. The first step was the gravity which ensured the equilibrium state before the exertion of the hydrodynamic pulsating forces. Second step was the hydrodynamic step, where the foundation was imposed with hydrodynamic pressure, imitating the initial sea water level. Then the geotube was applied with the hydrodynamic pulsating forces to the height of wave height. Wave height in SHD range from 0.5 m to 1.0 m throughout the year. Hence, wave height of 0.5 m, 1.0 m and 1.5 m (representing critical wave height) were adapted to study the stability and the effectiveness of geotube as coastal breakwater.

(e) ***Load module***

Gravity force is applied on all objects while hydrodynamic force is applied onto the geotube, to a level same with the significant wave height. Hydrostatic pressure, P_h representing the water level is applied over the foundation surface, calculated from Equation 3.19.

$$P_h = \rho g h \tag{3.19}$$

where,

P_h	=	Hydrostatic pressure applied on foundation (kN/m^2)
ρ	=	Fluid density (kg/m^3)
g	=	Gravity acceleration (m/s^2)
h	=	Water level from foundation (m)

Hydrodynamic pulsating force is adapted as it is the most significant forces that imposed on a near shore coastal structure. The formula for the wave equivalent force, proposed by Liu (1981) is expressed by Equation 3.20. This formula considers forces applied onto the oval cross section shape which is similar to the geometry of a geotube.

The boundary conditions were also applied in the load module where the bottom of the foundation is restrained from movement in both the vertical and horizontal direction;

while, the right and left sides of the foundation part is restrained from the horizontal movement.

$$F = \beta \gamma_w {H_b}^2 \tag{3.20}$$

where,

F = Equivalent wave load

β = Coefficient depending on the ratio of d_s/H and d_s/H_b (d_s, H, H_b are initial wave height, geotube height, and wave height)

γ_w = Unit weight of sea water.

(f) ***Mesh module***

The mesh module is one of the most important module where the appropriate mesh is applied on whole model and elements are assigned to the meshes to ensure the realistic analysis. For example, smaller meshes give more accurate results but required long computational time. Therefore, many different meshes were tried and compared to obtained the mesh size which produce sturdy results yet required optimum running time. For this study, geotube was modelled with 3-node quadratic beam in a plane, B22 element, while filling materials and foundation were modelled with 8-node biquadratic plane strain quadrilateral, CPE8. Figure 3.9 shows the meshes applied on whole model.

(g) ***Job and Visualization module***

Analyses were submitted in job module and results were extracted from the visualization module. Generally, the inputs used for the model development included the properties of geotube, properties of fills, properties of foundation, boundary conditions, and loads. While, the expected outputs of the simulation models were the displacement or deformation of geotube, foundation settlement, tension experienced in geotube.

Figure 3.10 shows the foundation settlement results in the FEM analysis while Figure 3.11 shows the sliding movement of the geotube. Several assumptions when using the model were included:

1. Two dimensional solutions were chosen because the length of geotube is very long in ratio as compared to cross section. Cross section of is assumed uniform along the length.
2. Results only show critical moment when wave hit the geotube breakwaters. In real situation, after one year of geotube installation, mangroves were planted and grow well.

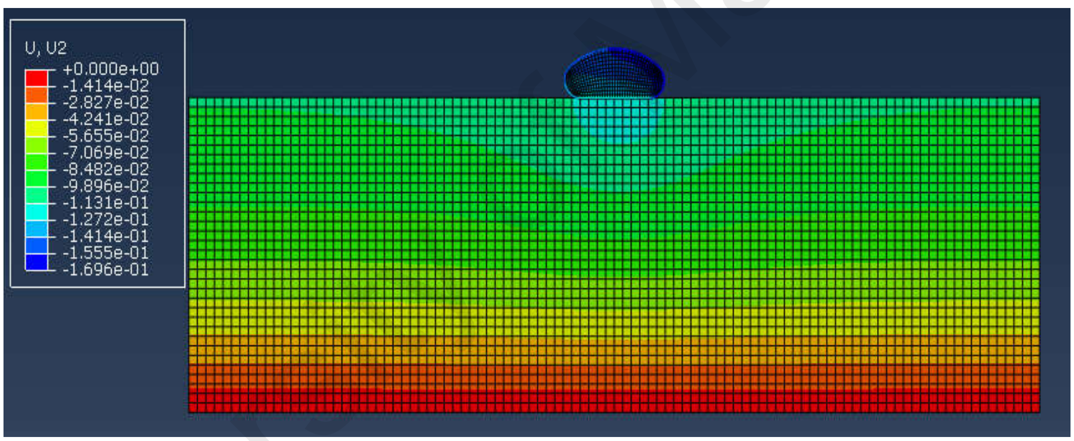


Figure 3.10: The settlement of foundation caused by the structure

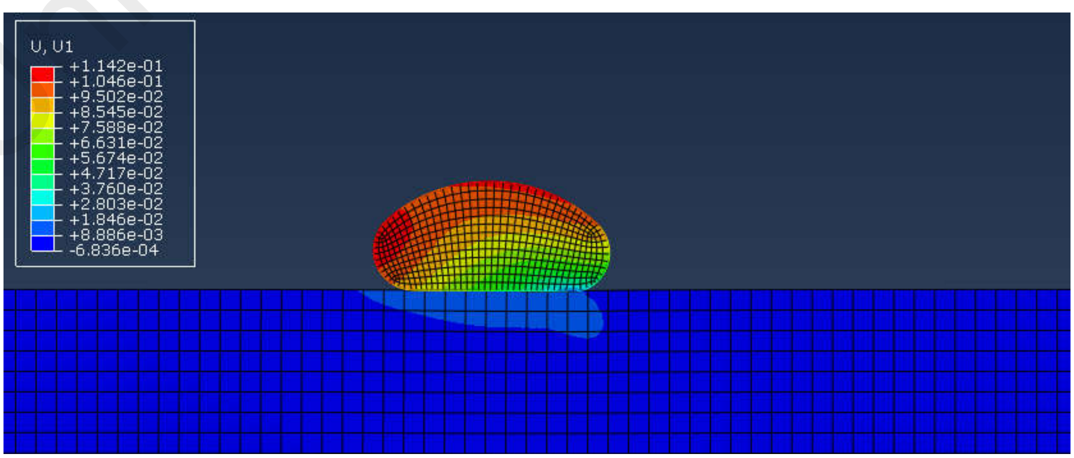


Figure 3.11: The sliding of structure due to hydrodynamic pulsating force

3.3.3.2 Input Parameters

Important input parameters adapted during the development of FEM models were stated in Table 3.2 and Table 3.3.

Table 3.2: Properties of the fill materials and foundation

Type of filling materials	Mud	Sand	Quarry Dust
Saturated density (kg/m^3)	1200	2000	1900
Elastic modulus (Pa)	4×10^6	4×10^7	6×10^7
Poisson's Ratio	0.3	0.3	0.3
Angle of friction (°)	27	32	34

Table 3.3: Properties of the geotube

Properties of geotube	Value
Density (kg/m^3)	900
Elastic modulus (GPa)	2.35
Poisson's ratio	0.4

3.3.3.3 Safety Factor Calculation

There were several formulas used and reported by Oh and Shin (2006) to estimate the factor of safety, FoS of geotube against the sliding, overturning, and bearing capacity. There was literature reported the used of LEM to obtain FoS. These methods were quick, but only able to provide a brief prediction of FoS without any insight on the extent of the deformation or displacement. Furthermore, the LEM are normally calculated conservatively.

It is possible that the geotube experienced some movement or displacement yet able to maintain its function as a coastal breakwater. Therefore, through the FEM analysis, we can determine the ultimate force needed to cause significant displacement or movement to the geotube or foundation. The ultimate force can then be used to calculate the FoS for other model of similar properties and environment conditions. Besides, by using FEM models, the mode of failure can be simulated.

The FoS developed from the FEM models were then compared with the FoS calculated from the equations (Equation 2.6 to Equation 2.9) adapted by Oh and Shin (2006) to evaluate the robustness of the method used in this study.

3.3.4 Analysis on Beach Responses after Installation of Geotube Breakwaters

The beach responses (accretion or erosion of sediment) to the installation of geotube breakwaters are very important. Inappropriate placement of the breakwaters can cause further coastal erosion instead of beach nourishment. In order to simulate the beach responses after the installation of the geotube breakwaters, the MIKE 21 software developed by the DHI Water and Environment was used. The software can be applied in the hydraulic simulation of lakes, estuaries and coastal areas. The MIKE 21 Flow Model, Hydrodynamic module was adapted. The main inputs of the model include the grid setting, bathymetry, wave and wind properties and open boundaries. While the major model outputs include the wave current speed, wave current direction and water depth just to name a few.

3.3.4.1 Hydrodynamic Model Development

The study aims to predict the shoreline response after the installation of the geotube breakwater along the SHD muddy beach. The bathymetry of SHD is created according to the rectified map, field measurements and MIKE 21 software. Besides, the tidal variation and wind generated waves were accounted in the hydrodynamic simulations. The maximum tidal range recorded from 2012 to 2015 was 3.2 m and hence adapted as the tidal input for model.

While, wind data of the same time frame were collected from the nearest station in Setiawan. In SHD, the typical strong winds are from Northwest and Southwest direction with wind speeds not more than 20 knots and 15 knots respectively. Wind data was collected from 2012 to 2014 and were presented in wind rose plots, as shown in Figure 4.2 to Figure 4.5

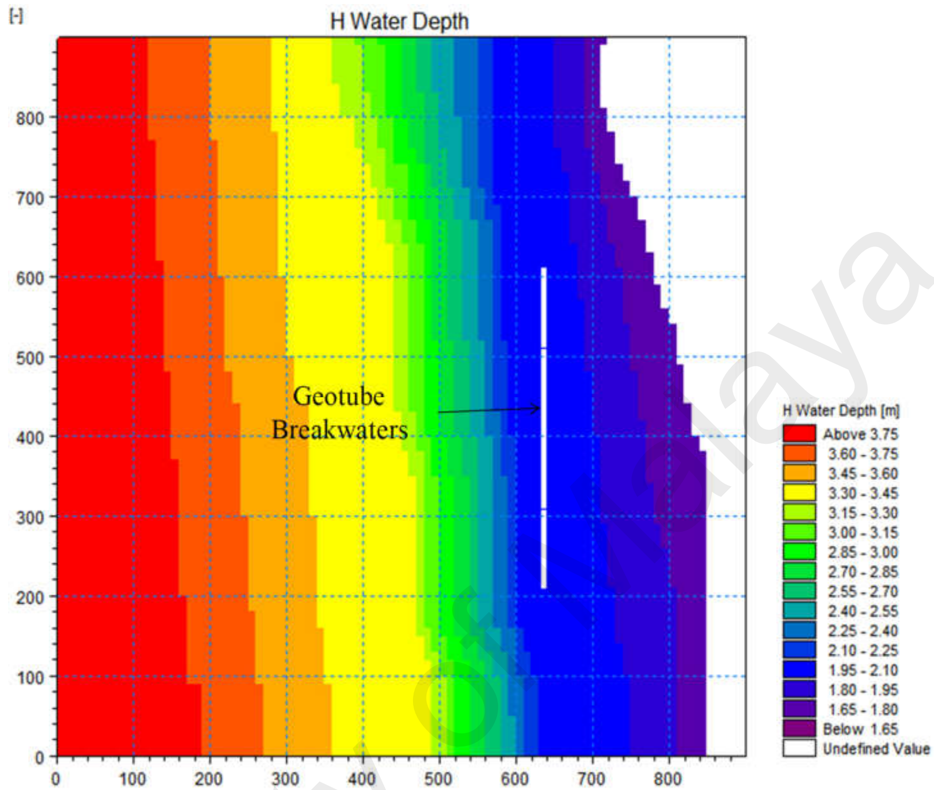


Figure 3.12: Simulations of the water depth and geotube breakwaters in SHD

3.3.4.2 Input Parameters

The model used in the study consist of geotube breakwaters which are parallel to the shoreline as shown in Figure 3.12. Map projection used was UTM-47, covering the area of longitude 101° 0' 50" and latitude of 3° 38' 8". The analysis model represents 450 m both in the long-shore and cross-shore direction of the study site, with cells of 0.5 m x 0.5 m, or a total cell of 810,000. Each model was run for a time step of 24 hours to observe the shoreline responses influenced by the tidal forces. The sinusoidal tidal signals with 3.2 m range were considered in all models. The influence of the distance of geotube from shoreline, gaps between geotubes, and length of the geotubes were also analysed and discussed.

3.3.4.3 Analysis Models

The outputs of the model are the speed and direction of wave currents without updating the sediments changes. The results were analysed and used as the proxy to predict the beach responses to the geotubes such as sediment accretion and erosion and discussed in Section 4.4.1. There are several assumptions and limitations made in the simulations:

1. The water depth and beach profile are modelled according to beach profile in SHD at local scale.
2. Wave reflection caused by the single line geotube breakwaters is neglected.
3. Models are depth average simulation, representing the hydrodynamic changed without the morphological updating.
4. The beach responses (erosion or accretion) to the geotube breakwater is predicted based on the change in wave current direction, pattern and speed.

The on-site measurement of sediment level changes after one year of installation were obtained from FRIM's reports and documentation. While after one year, beach started to nourished naturally and mangroves were planted at the protected area. Mangroves planting aims to further reduce the wave impact to the beach, encourage more sediment accumulation and rehabilitate the natural habitat. Monitoring were carried out from 2012 to 2015 for this study, and the change in beach profile after 8 years of geotube installation was reported in Section 4.4.2. These on-site monitoring measurements were compared with the result from MIKE 21 simulation to evaluate the sturdiness of the simulation results for sediment activities prediction.

3.4 Field Monitoring of Rehabilitated Study Site

Monitoring of geotube breakwater was carried out in order to compare the on-site observations with the simulation and prediction results. Study site of this project is located at the muddy beach in SHD, Selangor, Malaysia. The shoreline in SHD experienced great degradation and mangroves belt recession to an extent that the ecosystem failed to self-correct. Four stretches of geotubes were installed along the SHD beach by FRIM to assist the nourishment of beach and to protect the mangrove seedlings. Location of the study site is as shown in Figure 3.13.

The performance of the geotube breakwater in terms of protecting the eroded shoreline from further erosion and nourish the area naturally were observed. Conditions of the geotubes such as damaged, settlement or displacement were also recorded. These data are useful as a reference to assist design judgment in the application of geotube breakwaters on muddy beach. Discussions were made in regards to the sediment erosion and accretion, and the limitation and strength of geotube breakwater as compared to the conventional concrete breakwaters.

3.4.1 Sediment Elevation Monitoring

The on-site observation works included the monitoring on accretion and erosion of sediment, the deformation, displacement or settlement of geotubes on the muddy foundation. Previous monitoring data since the installation of the geotubes in 2007 were as well collected to complement the recent observations.

Monitoring works were carried out by installing monitoring pins around the geotube breakwaters and L-block concrete breakwaters. These pins were installed leeward and seaward of the structures and been monitored every 2 months since July 2013 to November 2015 to monitor for the change of sediment level.

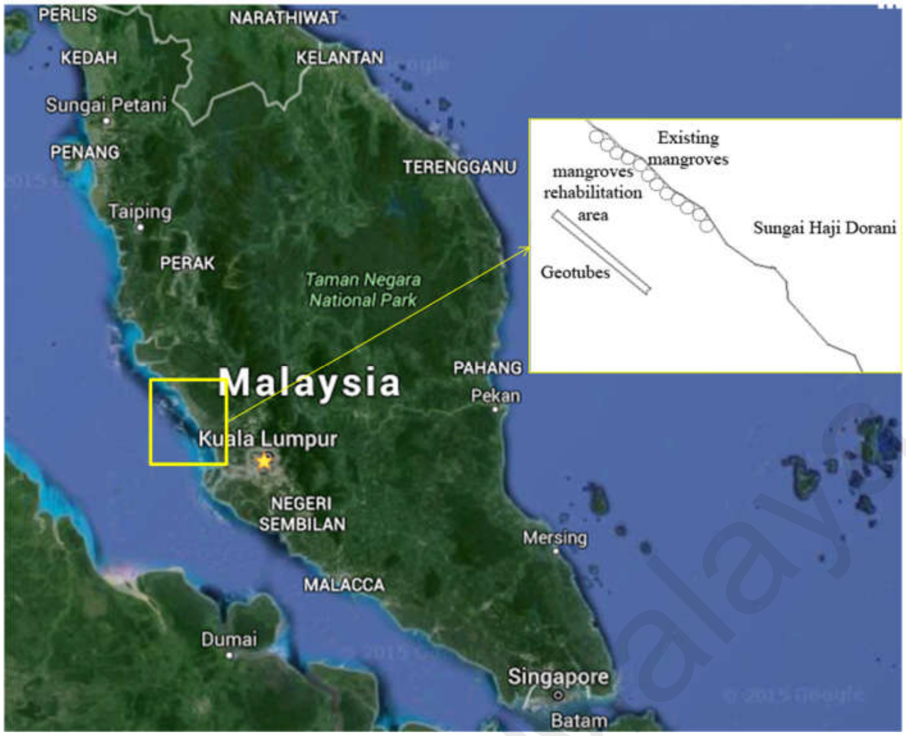


Figure 3.13: Location of the study site (SHD) where geotube breakwaters were installed (adapted from Google map)

Figure 3.14: Installation of monitoring pins

The monitoring pins installed were the PVC pipes of nominal size 40 mm, followed the standard in MS 628, part 1: 1999. Each pipe was cut into 4.0 m long and holes were drilled. Steel rod was inserted into the drilled holes as handle to assist the installation work, as shown in Figure 3.14 and Figure 3.15. Monitoring pins were push 2.5 m into the mudflat while exposing 1.5 m above the ground. Figure 3.16 shows the locations of geotube breakwaters and L-block concrete breakwaters in SHD.

Installation location of the monitoring pins were as shown in Figure 3.17 for geotube breakwaters, and Figure 3.18 for L-block concrete breakwaters. The L-block concrete breakwaters protect a smaller area 20,000 m^2, hence the monitoring pins were installed at 25 m distance to each other's. While, surrounding the geotubes, monitoring pins were installed at 40 m distance to each other's. The monitored pins covered approximately 44,800 m^2 area around the geotube breakwaters. The monitoring measurements were used to verify the simulation results. Besides, the measurements from these two different coastal protection structures were compared and discussed in terms of advantages, limitations, cost, construction time, environmental influences, etc.

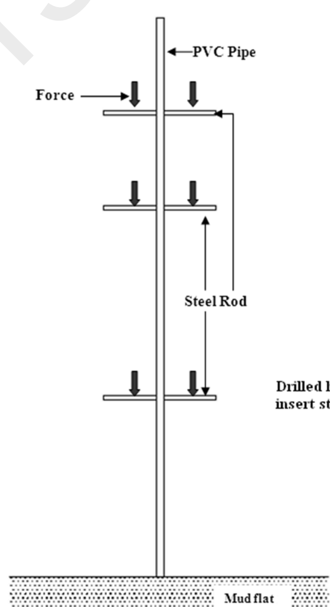


Figure 3.15: Monitoring pin installation method

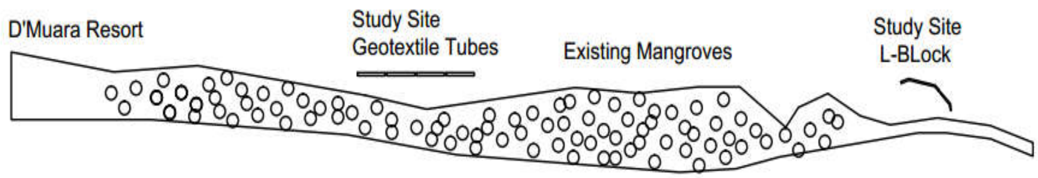


Figure 3.16: Site plan the coastal defence structures along Sungai Haji Dorani

Geotechnical investigation and survey were carried out before the installation of monitoring pins. After installation of the monitoring pins, the coordinate of the pins and their exposed length were located and recorded. Measurements were carried out during the low tide period. While the length of pins exposed were recorded to determine the accretion or erosion of the sediments. Due to the soft muddy foundation, measuring each pin with metre rule is not practical. Hence, the total station (TOPCON GPT 3100 series) was used to obtain the length of exposed pins from far. Several temporary bench marks were prepared to serve as reference points.

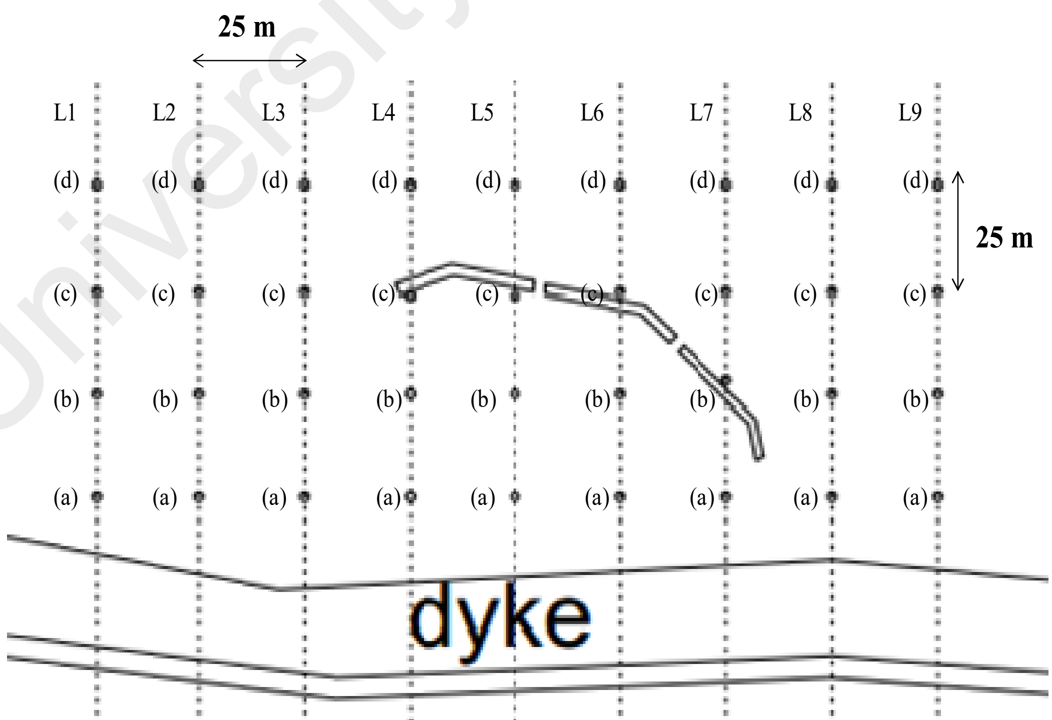


Figure 3.17: Location of the monitoring pins around the L-block breakwater

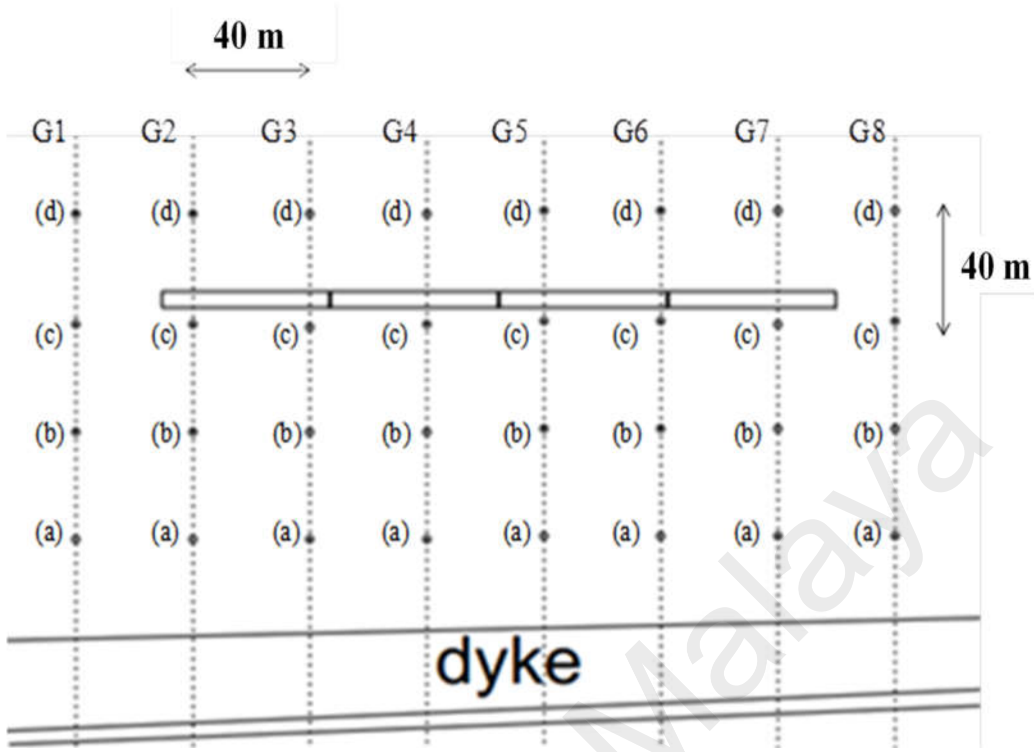


Figure 3.18: Location of the monitoring pins around the geotube

The changes in the exposed length of pins will indicate the accretion or erosion happened. The Equation 3.21 will be used to calculate the changes of sediment.

$$L_i - L_f = + L \text{ or } - L \quad (3.21)$$

where,

L_i	=	Initial exposed length of monitoring pins (m)
L_f	=	Final exposed length of monitoring pins (m)
+ L	=	Accretion of sediment (m)
- L	=	Erosion of sediment (m)

The measurements of sediment activities were then being presented in the cross-shore profile. As shown in Figure 3.17, there are nine profile lines around the L-block concrete breakwaters and eight profile lines around the geotube breakwaters as shown in Figure 3.18. Besides the accretion and erosion of the sediment, geotube breakwaters were also measured for changes in height, displacement and settlement. The physical conditions (such as damage or spillage of fills) were also observed. The damage of the structures, behaviours and performance in coastal protection and beach nourishment were discussed. The physical condition of coastal protection structures after few years of service period, after the continuous attack of waves, are very useful supplementary references in future research to assist coastal structure designs.

3.4.2 Conditions of Mangrove Rehabilitation

In year 2007, FRIM conducted a pilot research to rehabilitate the degraded mangroves by using geotube breakwaters to reduce the wave impacts. After one year of geotubes installation, FRIM adapted control planting technique to plant three species of mangroves (Avicennia, Mucronata and Rhizophora). Mangrove seedlings were raised in nursery, each inside a coir log before planting on the mudflat.

Maintenance, care and observation works were carried out after planting to the breakwater protection area. For instance, regular observation, remove and replace dead plants and pests. The conditions of mangroves rehabilitation were observed and discussed in Section 4.6.3.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Introduction

This chapter focuses on results and discussion of the study. Section 4.2 describes the properties and parameters obtained through tests and experiments and be used in simulation models. Section 4.3 and 4.4 discuss the internal and external stabilities of geotube as breakwater on muddy foundation. Section 4.5 describes the prediction of beach responses after installation of geotube breakwaters through simplified hydrodynamic models. Section 4.6 discusses the comparison between geotube and concrete breakwaters on muddy coast, based on different aspects, such as effectiveness, feasibility, environment and cost.

4.2 Properties and Parameters

Reliability of analysis models directly depends on input parameters. In this study, several important parameters such as properties of filling materials, properties of geotube, geotechnical data and the wind-wave information were determined. Section 4.2.1 describes properties of filling materials such as coastal mud, river sand and quarry dust. While, Section 4.2.2 and Section 4.2.3 describe properties of geotube and geotechnical information of SHD. Wind and wave information is described in Section 4.2.4.

4.2.1 Properties of Filling Materials

Properties of filling material are important considerations during the design of geotube breakwaters. Three different filling materials were considered in this study, which included coastal mud, river sand and quarry dust. Coastal mud represents on-site available sediments. Many geotube breakwaters installation involve on-site sediment dredging to pump into geotubes as filling material. However, sediment dredging on-site is not ecological friendly. Sand dredging activities consequence in local sediment erosion. On the other hand, natural river sand samples which represent the most typical sediment used as slurry, were obtained from local supplier near SHD. The natural river sand was also involving dredging of sediment from rivers. While, quarry dust samples were received from a quarry in Selangor. Quarry dust is the by product from granite crushing process and has low economic value due to limited applications. This work assesses the potential of the quarry dust as filling material for geotube, in order to widen the applications of this waste material.

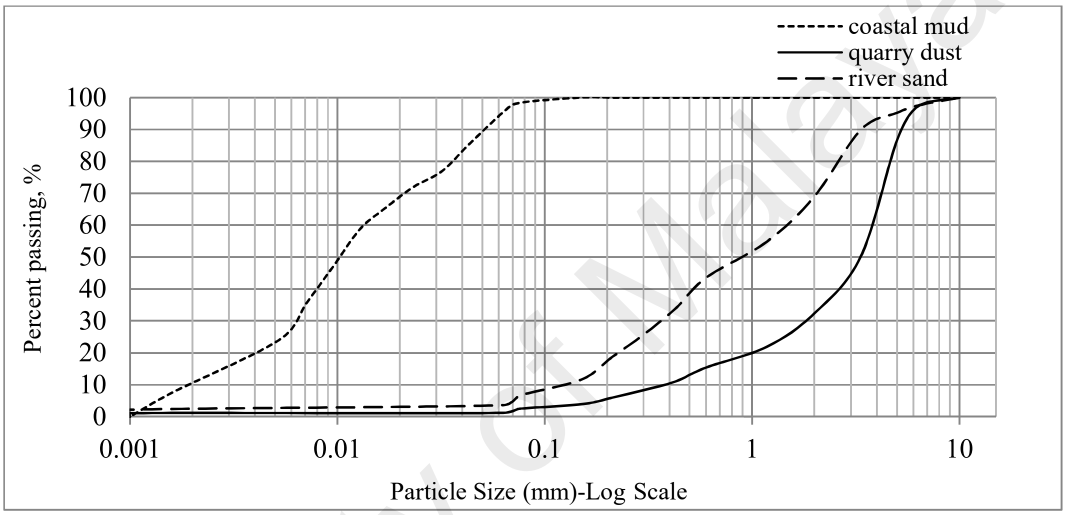


Figure 4.1: Particle size distribution curve of coastal mud, river sand and quarry dust

Result in Figure 4.1 shows coastal mud has finest sediment size amongst the three sediment types. D_{10}, D_{30}, D_{60} and D_{85} of coastal mud are 0.0019 mm, 0.0064 mm, 0.0135 mm and 0.0410 mm. Coefficient of uniformity, Cu and coefficient of curvature, Cc for coastal mud are 7.89 and 1.60 respectively. The river sand has greater sediment size compared to coastal mud. D_{10}, D_{30}, D_{60} and D_{85} of river sand are 0.136 mm, 0.368 mm, 1.613 mm and 2.900 mm. Hence the river sand has Cu of 11.86 and Cc of 0.857. However, the quarry dust has the greatest particle size. D_{10}, D_{30}, D_{60} and D_{85} of quarry dust are 0.395 mm, 1.760 mm, 3.750 mm and 4.900 mm. Cu and coefficient Cc for quarry dust are 9.24 and 2.09 respectively. The density of coastal mud, river sand and quarry dust are 1201 kgm^{-3}, 1978 kgm^{-3} and 1890 kgm^{-3} respectively.

4.2.2 Properties of Geotube Breakwater

The physical properties of geotube are important to ensure the effectiveness of geotube breakwaters. Structural failure due to inappropriate design can be prevented. The high strength polypropylene woven geotube samples used in this study are same with the geotube breakwaters installed in SHD. The length of the geotube is 50 m with a circumference of 8.6 m. Properties of the geotextile samples cut from the geotube are stated in Table 4.1, while laboratory test reports can be found in Appendix B.

Table 4.1: Specification of high strength polypropylene geotextile in this study.

Descriptions	Magnitude	Unit
Thickness of geotextile	1	mm
Mass per unit area	900	gsm
Tensile strength (MD/CD)	120/120	kN/m
Maximum tensile elongation (MD/CD)	21/20	%
CBR puncture strength	6500	N
Apparent opening size	≤ 354	micron
Permittivity of geotextile	0.8	s^{-1}

*MD = Machine direction
*CD = Cross direction

The apparent opening size, O_{95} of the geotextile is ≤ 354 microns. According to Leshchinsky et al. (1996) and Pilarczyk (2000), filling materials must be greater than apparent opening size of geotextile, in order to prevent sediment leakage through the openings and causes reduction in height and loss of functions. According to the literature, height of geotube structure reduced by more than 50% when fine sediment was used as filling material. The consolidation process of fine material takes longer time frame as the permeability is lower. Hence, second pumping procedure can only be carried out after several months when fine sediment in geotube is consolidated and this practice is not realistic. Besides, finer sediment can result in clogging of geotextile openings and lead to excessive hydraulic pressure inside geotube. Therefore, the coastal mud is not suitable to be used as the filling material. Sand and quarry dust are more suitable options.

From Table 4.1, maximum tensile strength of the geotextile is 120 kN/m. Hence, pumping pressure during the installation and the dynamic forces during the service period of geotube breakwaters should be lower than 120 kN/m to prevent structural failure.

Table 4.2: Several published filtration criteria of geotextile tube

Institute	Equations and criteria	Sand	Quarry dust	Coastal mud
Carroll (1983)	$O_{95}/D_{85} \leq 2$ to 3	$O_{95}/D_{85} =$ 0.126 (≤ 2 to 3) OK	$O_{95}/D_{85} =$ 0.075 (≤ 2 to 3) OK	$O_{95}/D_{85} =$ 8.902 (> 2 to 3) NOT OK
Christopher and Holtz (1985)	$O_{95}/D_{85} \leq 1$ to 2	$O_{95}/D_{85} =$ 0.126 (≤ 1 to 2) OK	$O_{95}/D_{85} =$ 0.075 (≤ 1 to 2) OK	$O_{95}/D_{85} =$ 8.902 (> 1 to 2) NOT OK
ASSTHO (1986)	For soil < 50% passing the No.200 sieve, O_{95}<0.59 mm; (i.e. AOS of the fabric > No. 30 sieve) For soil > 50% passing the No.200 sieve; O_{95}<0.30 mm; (i.e. AOS of the fabric > No. 50 sieve)	6% river sand passing No.200 sieve (< 50% passing) (required O_{95} < 0.59 mm) $O_{95} = 0.354$ mm OK	2.5% quarry dust passing No.200 sieve (< 50% passing) (required O_{95} < 0.59 mm) $O_{95} = 0.354$ mm OK	98% coastal mud passing No.200 sieve (>50% passing) (required O_{95} < 0.297 mm) $O_{95} = 0.354$ mm NOT OK

In order to investigate the influence of sediment type to the geotube breakwater, three main sediments were used in tests, i.e. coastal mud, river sand and the quarry dust. However, the on-site sediment, coastal mud did not fulfil primarily requirements as geotube's filling material, due to their very fine sediment size. Hence coastal mud excluded in simulation models.

The sediment size of silt loam (coastal sediment in SHD) is too fine (D_{85} = 41 micron) as compared to the O_{95} of the geotextile (354 micron). According to Bezuijen and Vastenburg (2012), there are several criterions suggested for filling material and are stated in Table 4.2. The coastal mud particles do not comply with any of these design criteria.

Hence, the on-site sediment is not suitable to fill the geotubes as sediment size is too fine and will leak out from geotextile openings.

4.2.3 Geotechnical properties of study site

The construction of coastal protection structures on SHD's muddy beach is an expensive and difficult task. This is because the mud deposits are very weak and soft due to the unconsolidated soil which has high water content. Weak, deformable and compressible foundation in SHD has high potential for structural settlement or sliding.

Table 4.3: Geotechnical information

Descriptions	Magnitude	Unit
Soil properties:		
Percentage:		
Clay	8	%
Silt	87	%
Sand	5	%
Soil classifications:		
USCS classification	CH	-
USDA classification	Silt	-
Dry density	380	kgm^{-3}
Wet density	1201	kgm^{-3}
Water content	97.63	%
Atterberg Limits:		
Liquid Limit	169.50	-
Plastic Limit	62.31	-
Plastic Index	107.19	-
Compression index, C_c	0.68	-
Swelling index, C_u	0.16	-

Construction of geotube breakwater commonly carried out after a layer of geomembrane was layered on the weak foundation. The layer of geomembrane functions as anti-scouring protector and helps to reduce the potential of sliding and settlement due to scour.

Table 4.3 shows several important parameters obtained from laboratory experiments or site investigations and employed as inputs in simulation models. Other relevant laboratory reports can be found in Appendix C.

4.2.3 Winds, Waves and Tides Information of Study Site

In SHD, waves are mainly wind generated, thus wind data is important parameters for hydrodynamic simulation of coastal protection structures. Wind data from year 2012 to 2015 were collected from nearest wind station Setiawan station. The data from these four years were presented as wind rose plots in Figure 4.2 to Figure 4.5. From the results, two most common wind directions are from SW and WNW directions. Wind from these two directions are generally caused by the southwest monsoon and northeast monsoon. The strongest wind from SW and WNW directions are not more than 15 knots and 20 knots respectively.

The maximum tidal range recorded from 2012 to 2015 was 3.2 m and hence adapted as the tidal input in simulation models. While, the significant wave height recorded was lower than 1 m (majority between 0.5 m to 1.0 m). Figure 4.6 shows tide water level data of November 2015 in SHD.

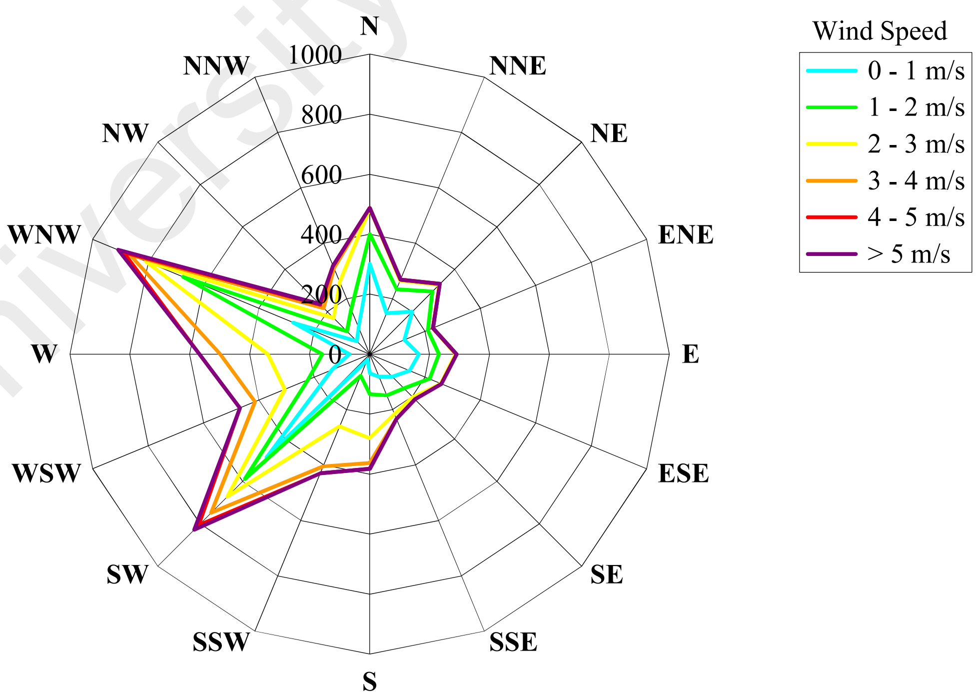


Figure 4.2: Wind rose plot for year 2012

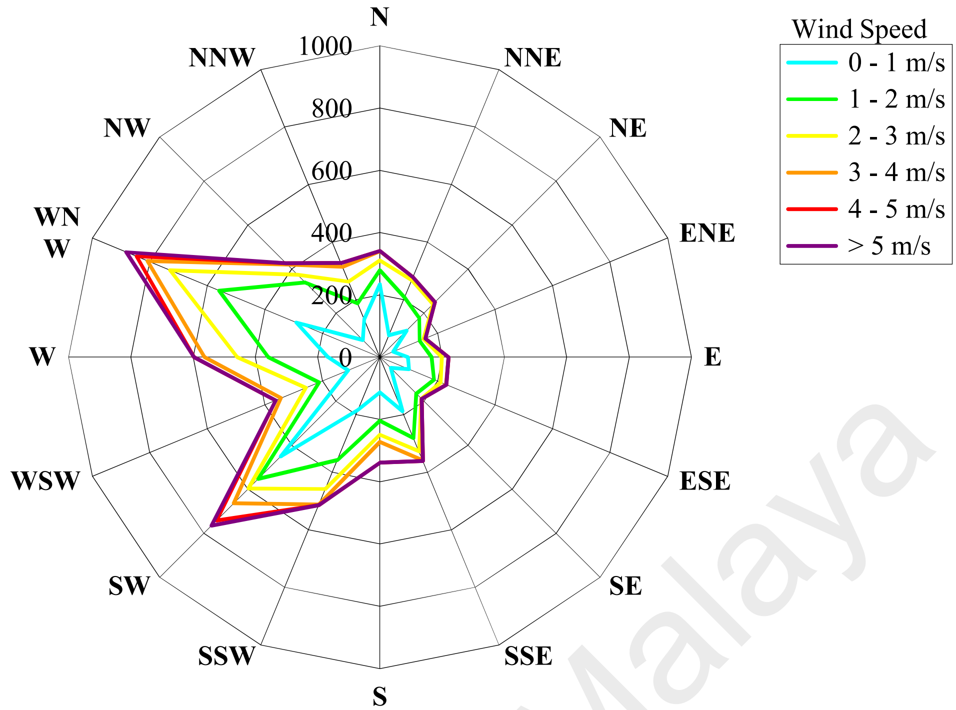


Figure 4.3: Wind rose plot for year 2013

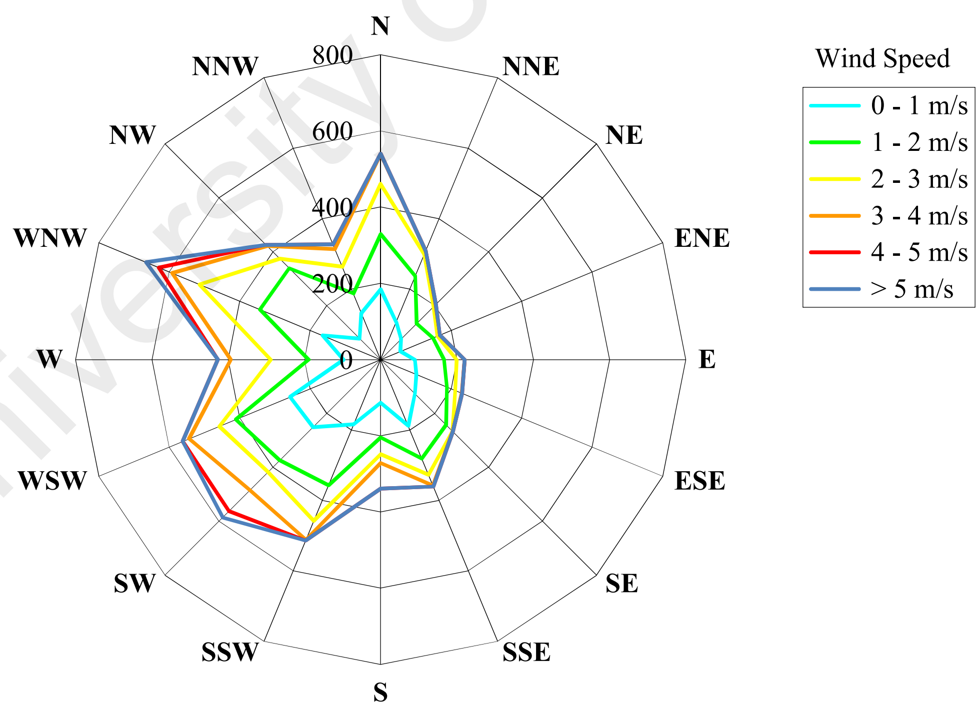


Figure 4.4: Wind rose plot for year 2014

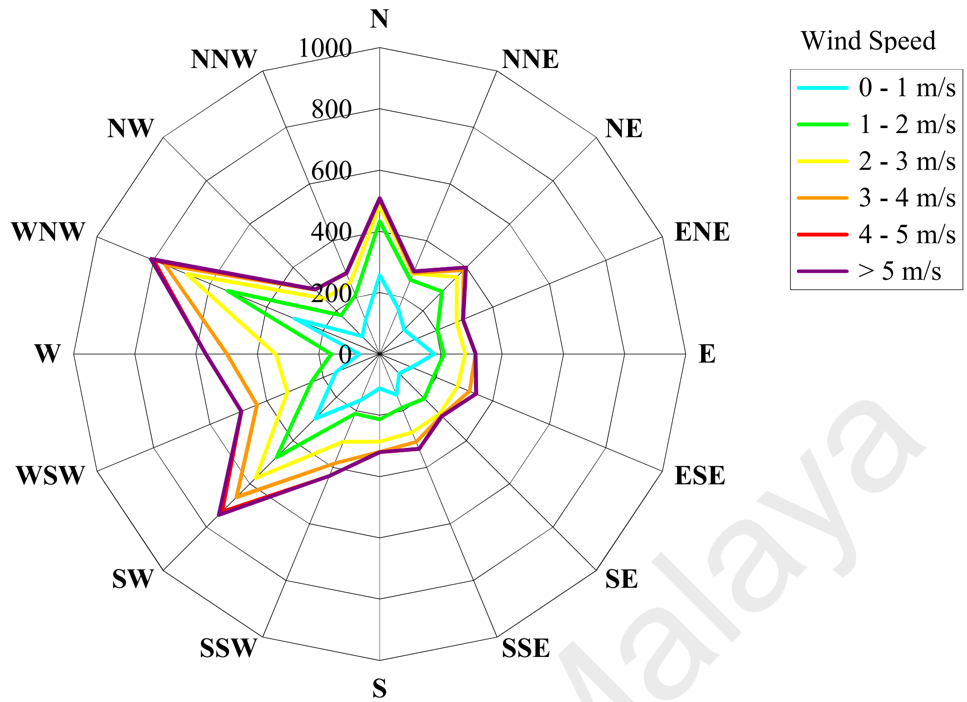


Figure 4.5: Wind rose plot for year 2015

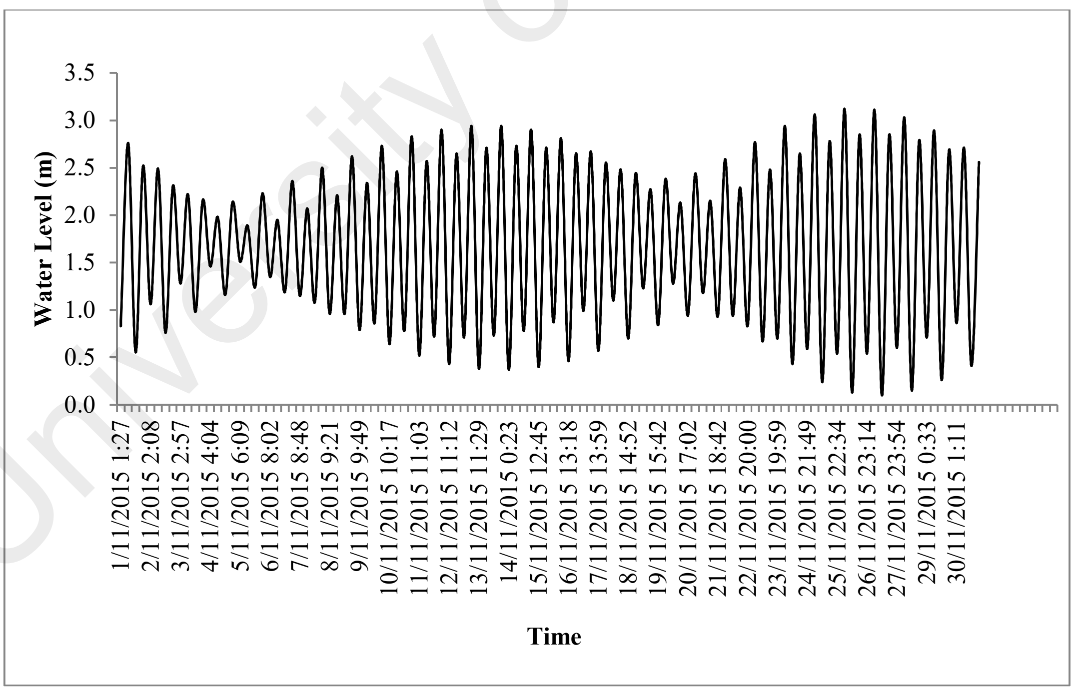


Figure 4.6: Tide water levels in November 2015

4.3 Internal Stability of Geotube Breakwater

4.3.1 Relationship between T, H, P_o and Geometry of Geotube

Influences of parameters (T, H, and P_o) to the geometry of geotube was studied by employing differential calculus formulation with GeoCoPS program. All cases were conducted with the geotube circumference of 8.6 m, which is same with the geometry of geotubes in SHD. Unit weight of slurry to water was taken as 1.2.

Figure 4.7 shows the effect of specified geotextile tensile strength, T_{ult} on the geometry of geotube. Theoretically, in order to achieve perfect circular cross section with diameter equal to theoretical height, $D = L/\pi = 2.74$ m, geotube's T or P_o are significantly high. As shown in Figure 4.7, when given T_{ult} 25 kN/m, the maximum H can be obtained is 2 m, or 69% of D. Increase T_{ult} to 50 kN/m achieved maximum H of 2.3 m, which is 84% of D. If given T_{ult} is 200 kN/m, the maximum H obtains is 2.7 m, with significantly high P_o of 139 kPa. However, T_{ult} of geotube available in market typically range between 50 to 150 kN/m. Thus, in order to achieve desirable structure's height without experiencing significantly high tension and P_o, a geotube with greater circumference can be used. Cost of geotube is based on its mechanical properties. Geotube with suitable properties should be chosen during the design phase, in order to fulfil the geometry requirements at optimum project cost.

Figure 4.8 illustrates the influence of H on geometry of geotube. In order to achieve H = 0.5 m (about 18% of D), P_o experienced in geotube is almost zero and the T in geotube is approximately 0.8 kN/m, which are both insignificant. However, in order to achieve H = 2.5 m, which is 92% of D, P_o experienced in geotube is 26.9 kPa and T is 54.0 kN/m. The closer the height of geotube to the perfect diameter, the greater the P_o and T in geotube. Therefore, to prevent extremely high P_o and T in geotube, design height should be maintained approximately between 75% to 90% of D.

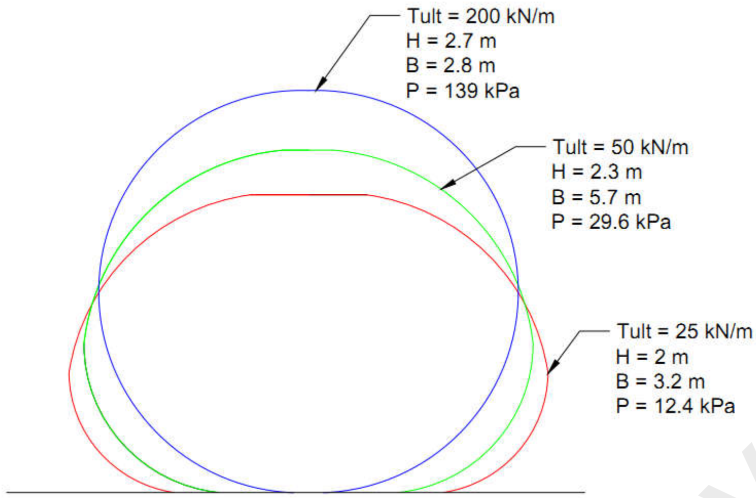


Figure 4.7: Influence of T_{ult} on geometry of geotube

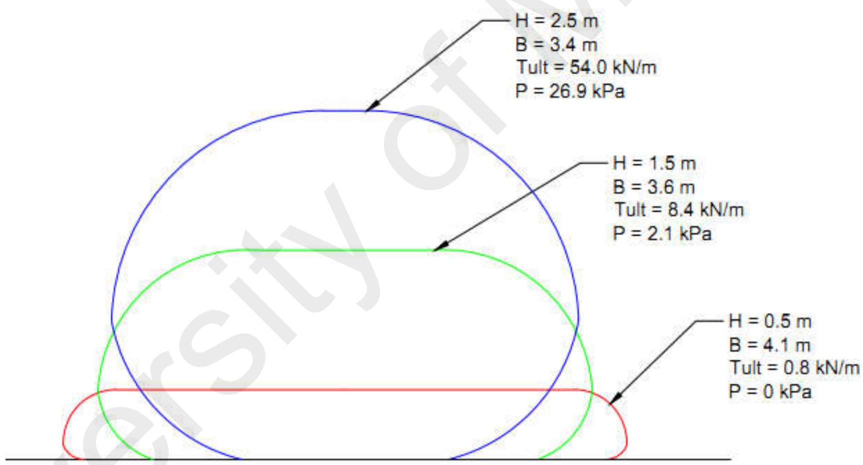


Figure 4.8: Influence of H on geometry of geotube

Figure 4.9 shows relationship between P_o and geometry of geotube. At lower pressure range, small increase in the P_o results in significant increase in H. For instance, increment of H from 2.3 m to 2.6 m involves 15.0 kPa increment in P_o. However, beyond approximately 90% of D, increment in H becomes insignificant even though P_o constantly increased. While, T increases exponential and directly with P_o. Without good field control, high P_o can rupture or damage the geotube especially around inlet area during pumping procedures.

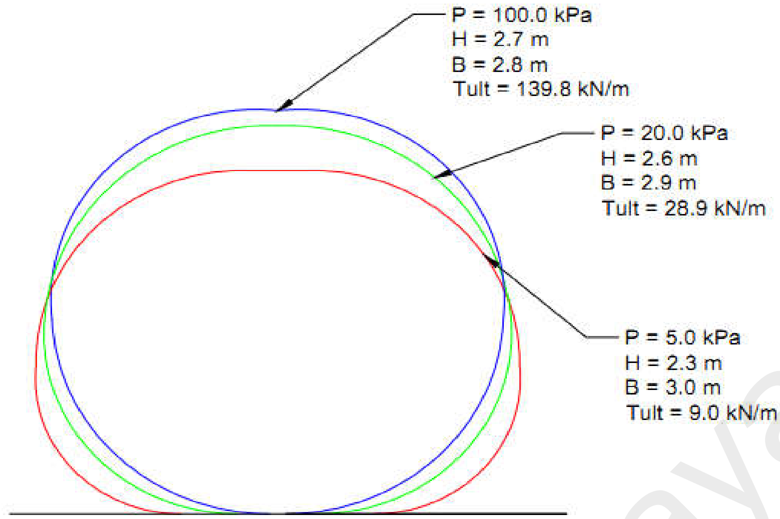


Figure 4.9: Influence of P_o on geometry of geotube

Figure 4.10 depicts the relationship between P_o and H of geotube. Influence of P_o to H is most significant at the initial filling stage. As pressure increase throughout the filling process, influence of P_o to the H of geotube become insignificant. Height of geotube achieves perfect height of 2.74 m when pumping pressure is at infinity, and this is not realistic. Graph in Figure 4.10 shows that, for our selected geotube, 2.5 m is the maximum H it can achieve (about 90% of D), before P_o become insignificant in increasing the height of geotube.

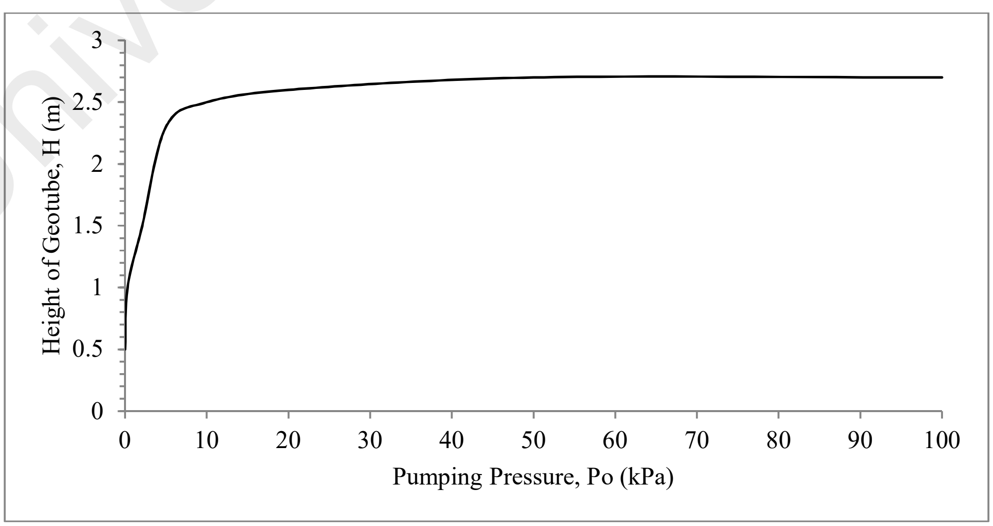


Figure 4.10: Relationship between the P_o and the H of geotube

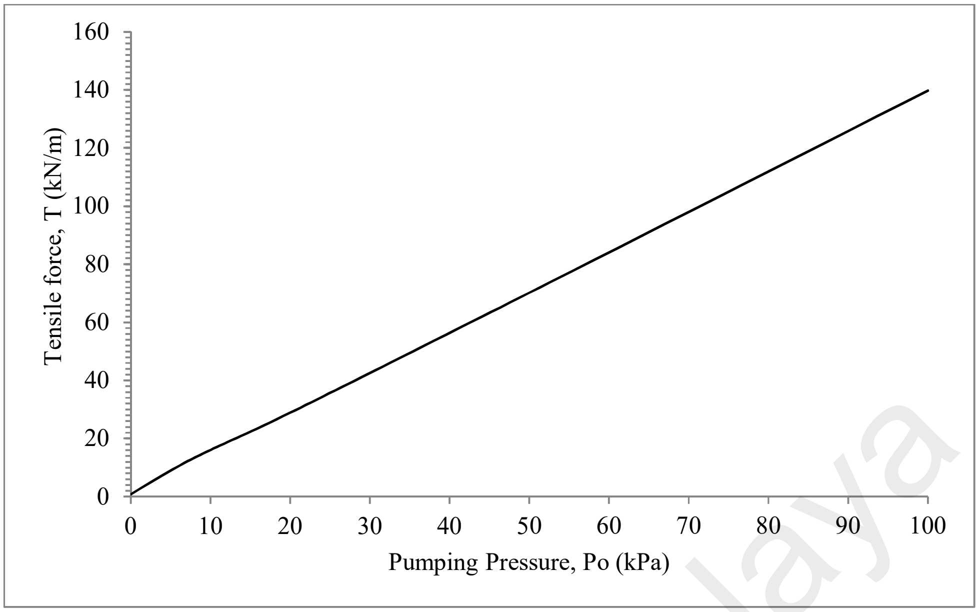


Figure 4.11: Relationship between the P_o and the T in geotube

Figure 4.11 illustrates relationship of P_o and T in geotube. T in geotube linearly increase with P_o. Understanding the relationship between P_o and T is important in the design and selection of geotube breakwater to avoid structural failure due to insufficient tensile strength, especially during the pumping procedure where P_o is highest around the inlet of geotube. The selected geotube for this study has tensile strength of 120 kN/m. Hence the maximum P_o before geotube rupture is approximately 88 kPa.

Figure 4.12 shows the relationship between H and T of geotube. When H of geotube closer to its perfect diameter (D = 2.74 m for geotube used in this study), T in geotube increases significantly. Continuous filling process do not increase the H significantly, but T significantly increases and exceed tensile strength of the geotube. Therefore, continuous pumping procedure to fill the geotube after it reaches 2.45 m, can causes excessive T in geotube and lead to rupture of the unit.

In a nut shell, this study shows the influence of the three parameters T, H and P_o to the geometry of geotube. Determinate one of the three parameters can assist the determination of the other two parameters.

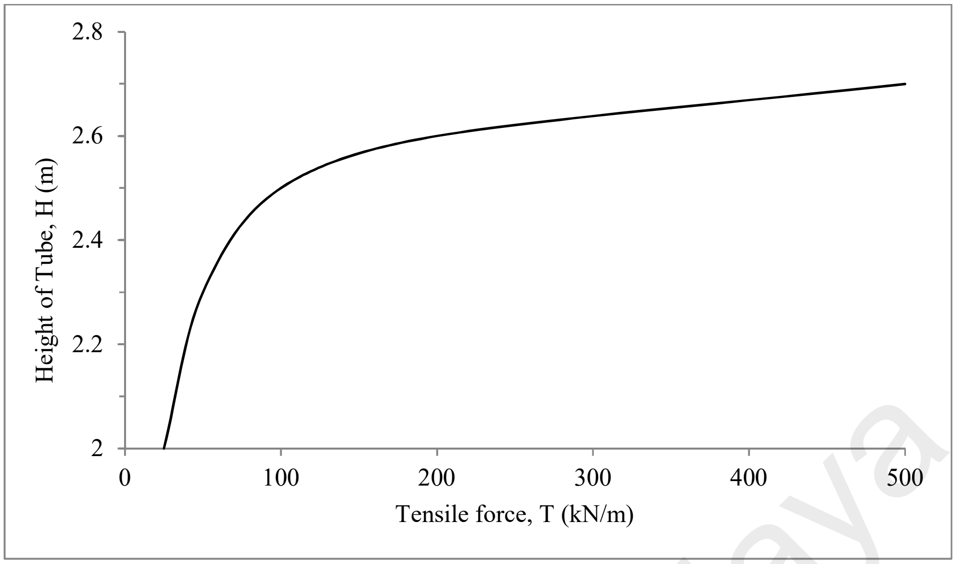


Figure 4.12: Relationship between T and H of geotube

4.3.2 Factors that Influence the Internal Stability

Figure 4.13 to Figure 4.15 illustrate relationship between sediment concentration of slurry and P_o, H_f and T in geotube. In order to investigate other influencing factors of geotube's internal stability, different conditions were considered in analysis model. For instance, sediment types, sediment concentration in slurry, pumping pressure, geotube's submergence conditions, design height of geotube, and maximum tensile forces experienced in geotubes.

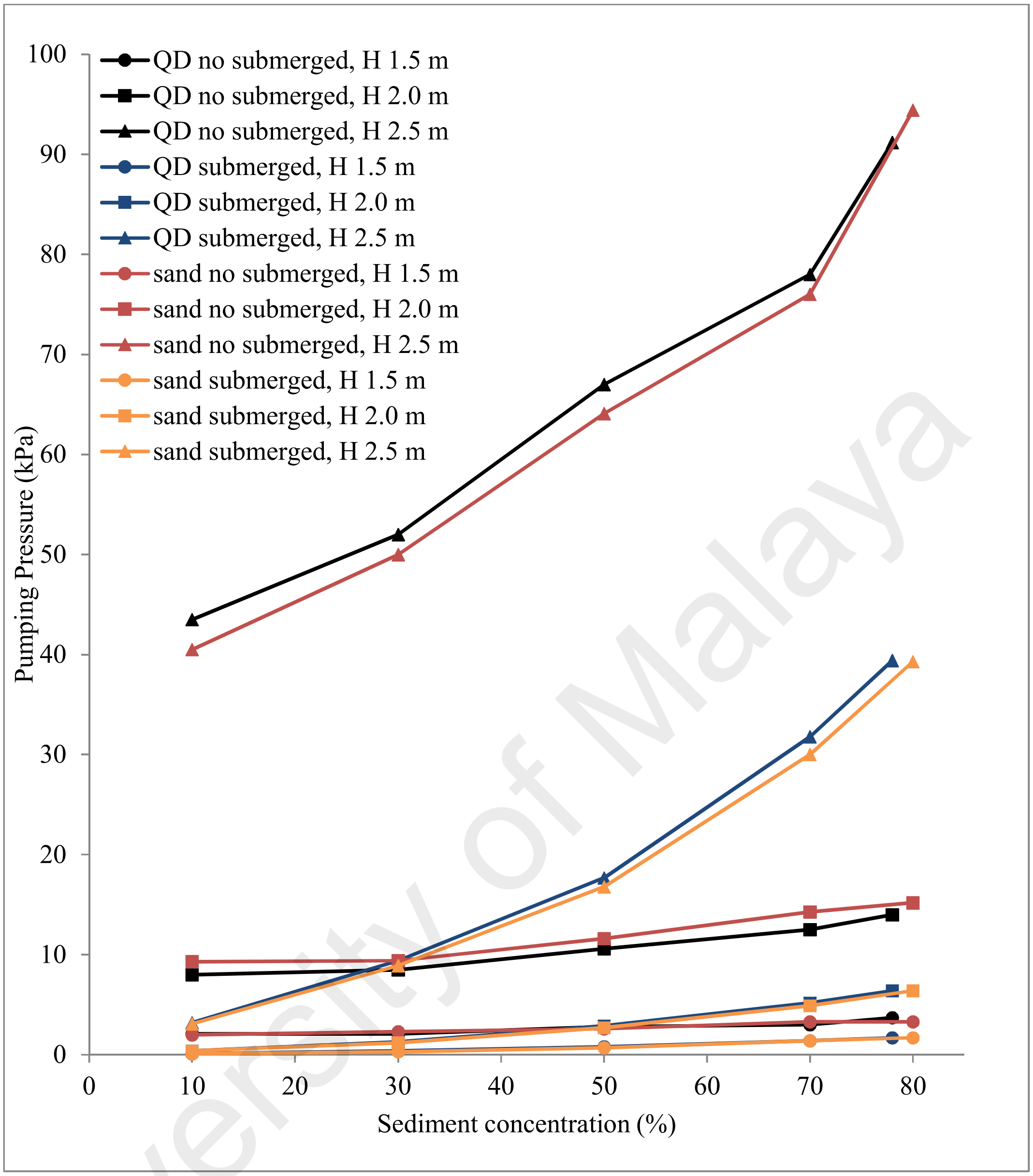


Figure 4.13: Relationship between sediment concentration of slurry and P_o

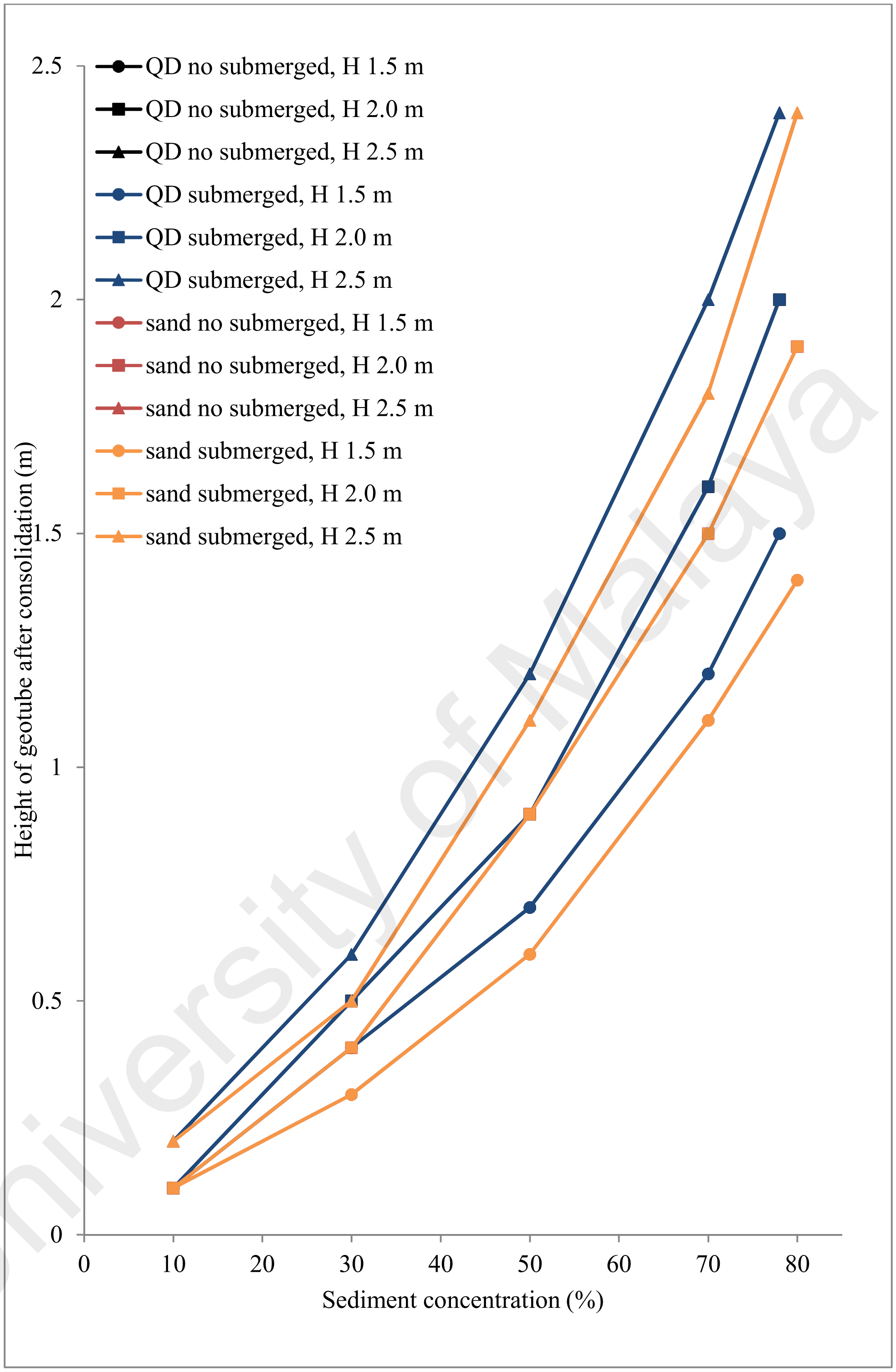


Figure 4.14: Relationship between sediment concentration of slurry and the height of geotube after consolidation

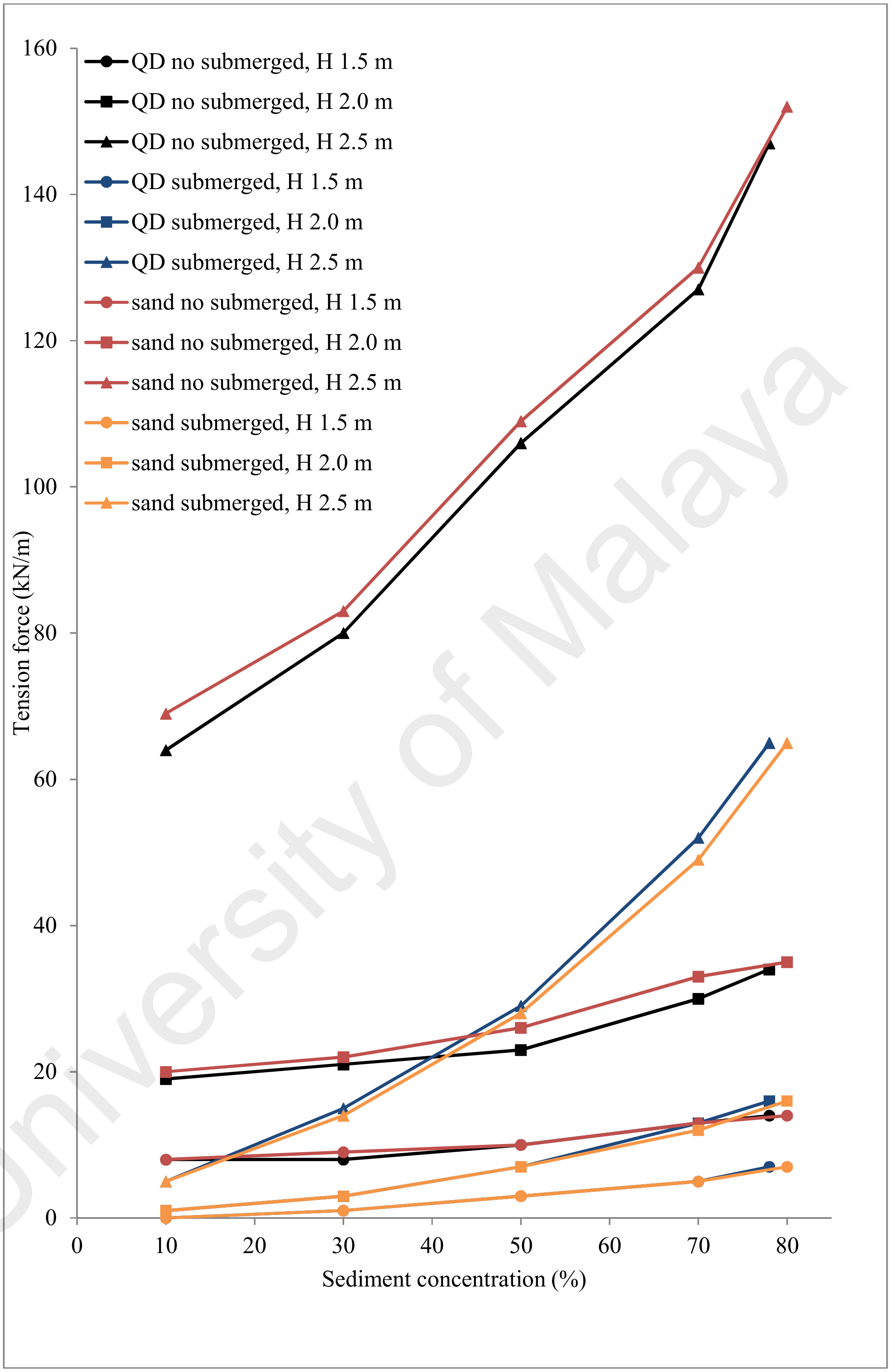


Figure 4.15: Relationship between sediment concentration of slurry and the tensile force in geotube

4.3.2.1 Influence of Submergence Condition

The submergence condition (submerged or non-submerged) is important consideration in geotube breakwater design. Results from Figure 4.13 to Figure 4.15 illustrates the different behaviours of geotube breakwater under different submergence conditions. Non-submergence condition is the more critical conditions because there is no external load from the water to balance the forces exerted by the slurry. For example, referring to Figure 4.13 and Figure 4.15, a non-submerged geotube of H = 2 m, filled with quarry dust slurry of 30% sediment concentration, experienced 5.5 times greater P_o and 6 times greater T in geotube, as compared to submerged geotube. At the mentioned condition, P_o of submerged geotube is 1.4 kPa, and non-submerged geotube is 7.8 kPa; while, T of submerged geotube is 3.4 kN/m, and non-submerged geotube is 20.5 kN/m. Thus, during the design of geotube, the non-submergence conditions shall be adapted.

4.3.2.2 Influence of Sediment Concentration of Slurry

Figure 4.16 illustrates relationship between sediment concentration of slurry and T in geotube under non-submergence condition. Higher sediment content in slurry results in higher P_o. This is because higher sediment content reduces the workability of slurry. Hence, larger P_o is needed to pump slurry into geotube. Increasing the sediment content in slurry from 10% to 80 % can result in up to two times increase in P_o.

In normal practice, although there is no standard references or codes, sediment concentration of 20% to 40% were often applied for smooth pumping process. Figure 4.16 shows that both quarry dust slurry and sand slurry showed very similar trends in the change of P_o. Sand is the most commonly used filling materials for geotube. Quarry dust slurry can be alternative filling material for geotubes, substituting sand as both materials give similar result. As compare to the coastal sediment, both quarry dust and sand sediments have bigger size than the geotextile openings. Therefore, excessive water flow out from the geotube effectively yet retaining sediments inside the geotube.

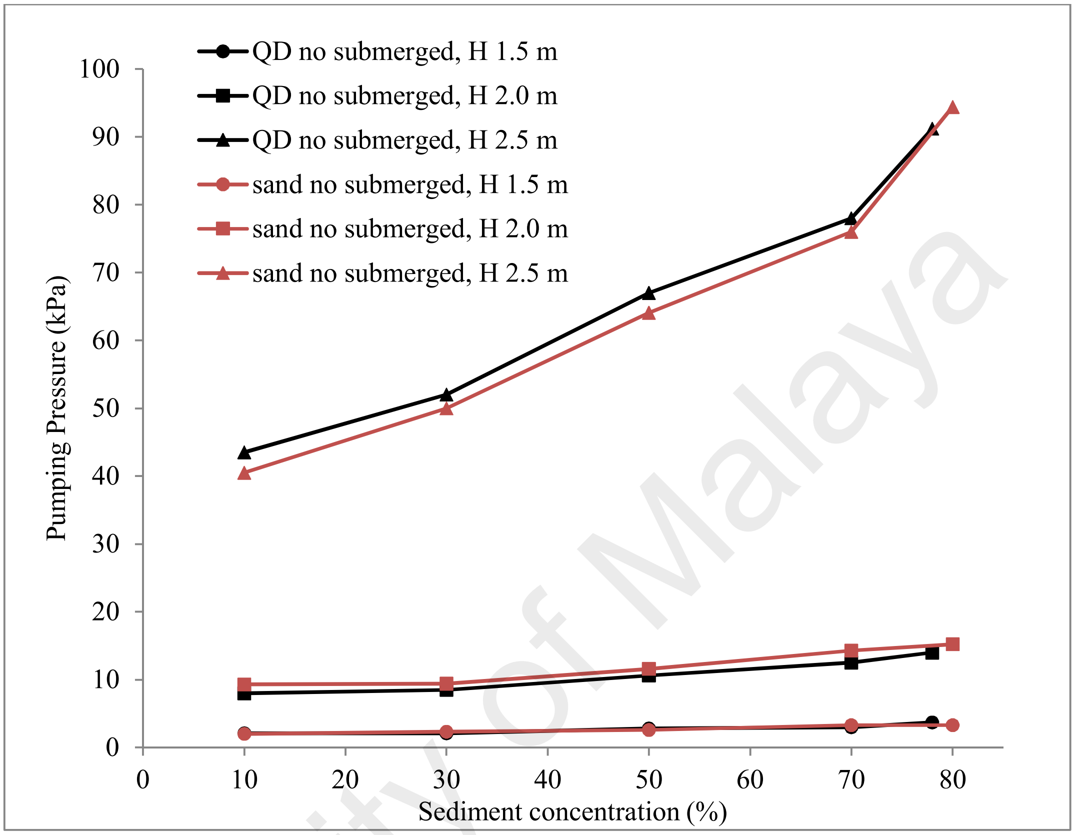


Figure 4.16: Relationship between sediment concentration of slurry and T in geotube under non-submergence condition

Greater sediment concentration in slurry form a geotube to designed height at shorter time. Results from Figure 4.17 shows that H is not directly influenced by the submergence conditions of geotube but sediment concentration. Greater sediment concentration in slurry fill the geotube structure and consolidate to desired height at quicker rate. For instance, approximately 80% sediment concentration in quarry dust and sand slurry can achieved 2.4 m geotube's final height, with one pumping procedure. However, 80% sediment concentration in slurry is not practicable as the workability of the slurry is too low. Therefore, in geotube breakwater design, engineers need to compromise between sediment concentration and workability of slurry.

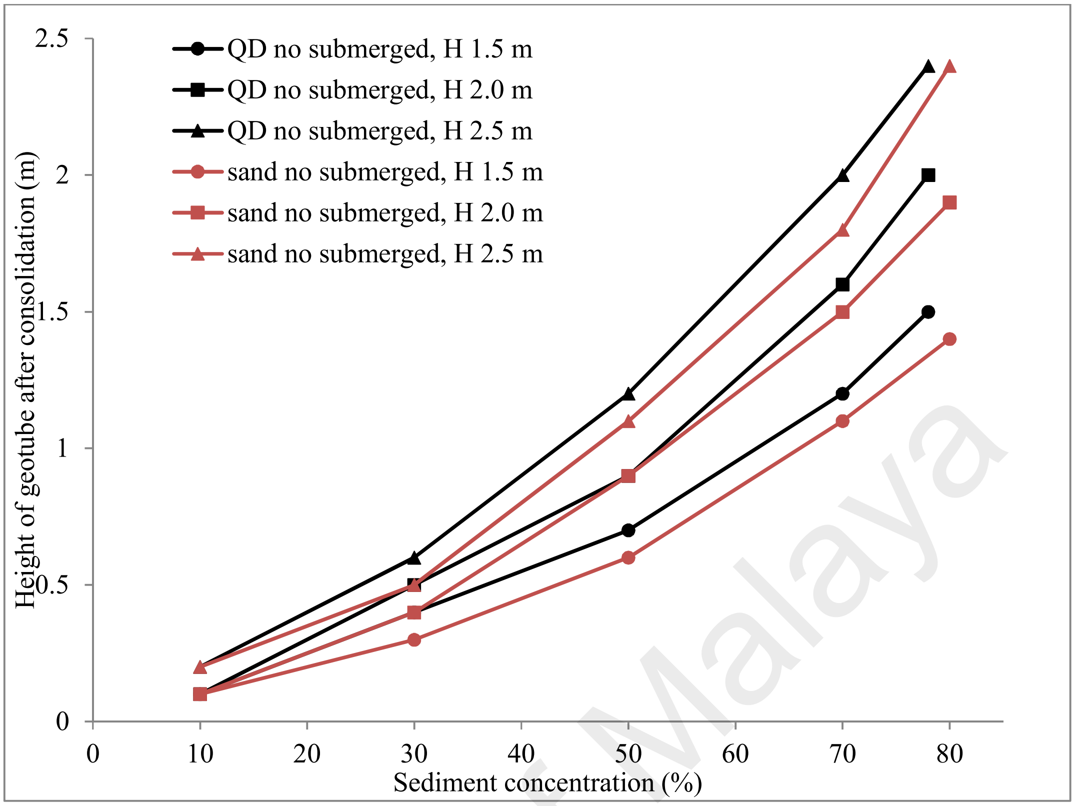


Figure 4.17: Relationship between sediment concentration of slurry and the height of non-submerged geotube after consolidation

Results in Figure 4.17 depicts that sediment concentration of 50% in slurry can be adapted instead of adapting the conventional 20% to 40% sediment concentration. First of all, the slurry with 50% sediment has acceptable workability and do not cause high P_o, as shown in Figure 4.16. Besides, H of geotube after one-time consolidation ranged from 35% to 50% design height. Quarry dust and sand slurry showed similar trend and results. Nevertheless, H_f of geotube filled with quarry dust showed 5% greater than sand filled geotube. Quarry dust and sand sediments have great permeability due to their larger size and edged shape. Higher permeability of these sediments ensures water flow out quicker through the geotube's openings and consolidate quicker than coastal muddy sediment (silt loam). Speedy sediment consolidation allows second pumping procedure to be carried out quicker, as soon as on the same day. Therefore, this reduce the construction time and greatly minimize the project cost.

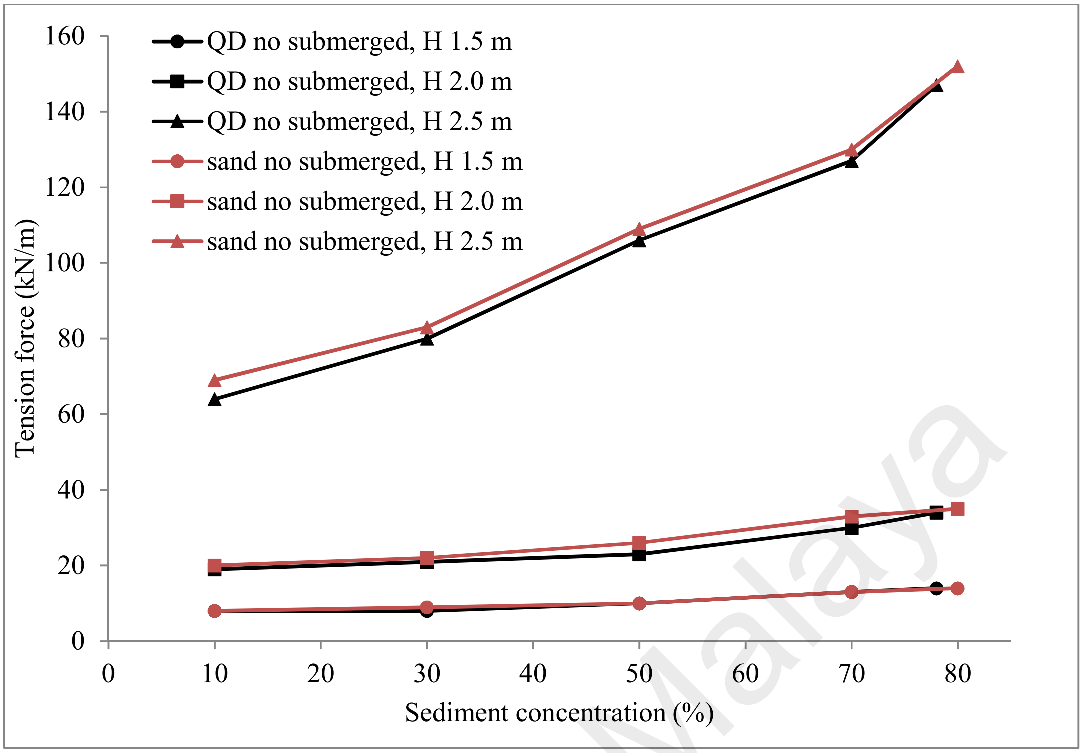


Figure 4.18: Relationship between sediment concentration of slurry and the tensile force in non-submerged geotube

Relationship between sediment concentration of slurry and T in non-submerged geotubes is presented in Figure 4.18. The greater the sediment concentration in slurry the greater the T in geotubes. The T exerted on geotubes should be lowered than tensile strength of the geotube, in this study, 120 kN/m. This can prevent structural damage or failure. Analysis shows pumping in quarry dust or sand slurry with sediment concentration more than 60%, in order to achieve 2.5 m structure's height, exerted T more than 120 kN/m. Hence in this study, 50% is the upper limit for sediment concentration of slurry. The T was significantly higher when the design geotube's height is 2.5 m as H is very close to the ideal diameter of the geotube (D = 2.74 m). For geotube's height 1.5 m and 2.0 m, T in geotubes is smaller than 120 kN/m.

From the study, sediment concentration of the slurry influences all the important parameters, P_o, H, and T. 50% is the upper limit for sediment concentration in slurry because the P_o and T exerted on geotube do not exceed allowed magnitude. Besides, design height of geotube can be achieved in one-time pumping procedure and consolidation. High permeability of sand and quarry dust allow water to flow out fast and allow second time filling procedure to be carried out in short time frame.

4.3.2.3 Influence of Sediment Type

In order to investigate the influence of sediment type to the geotube breakwater, there were three main sediments considered in this study, i.e. the coastal mud, river sand and the quarry dust. However, the coastal mud was not fulfilling the primarily requirements and criterion for size, as discussed previously in Section 4.2.3. Hence, coastal mud sediment is excluded from simulation models. Sediment with size finer than geotextile's openings results in sediment leakage and structural failure. Although the finer particles might slowly bind pores and prevent further sediment leakage, such blockage of opening could cause significant increment in T in long term fur to poor permeability. Besides, formation of fine sediment filter cakes, consolidation and refill process take up very long time, and this is not time and cost effective.

Lower permeability slows down water flow out from geotube. Hence create high T in the geotube, especially when the structure is non-submerged. High T increases the potential of damage of geotube. Besides, low permeability increases the consolidation time. According to Leshchinsky et al. (1996), using very fine sediment to fill the geotube, takes about 1 month for the fills to fully consolidate into a flat layer of filter cake. The change in the geotube height can be very significant and required second filling procedure. They also reported that consolidation of a slurry which lead to 9% increase in density, caused 50% reduction of height. Refilling the consolidated geotube is time consuming, involves complex procedures and expensive. Most importantly, there is a risk that the final height of geotube breakwater formed is uneven.

Geotube breakwaters in SHD were filled with river sand transported from nearby supplier. Traditionally, dredged sand was used to filled the geotube breakwaters. However, increasing demand for river sand resulted in more sand dredging activities and causes erosion at those areas. The major aim to construct geotube breakwater is to protect coastline from erosion. Hence, application of geotube is debateable, if filling up the structure with sand causes erosion to other regions. Therefore, in this study, quarry dust is recommended as an alternative filling material for geotube breakwaters.

Quarry dust is a by-product of coarse aggregates production, in this study, quarry dust is the by-product from granite crushing production. Increasing demand for the coarse aggregates in construction projects resulted in greater production of quarry dust. This material is rarely used in civil engineering projects and yet its' monetary worth has not been fully considered. Therefore, quarry dust is often looked at as a waste and is mishandled to save cost, i.e. dumped in landfills (Erftemeijer et al., 2012). Using the quarry dust as alternative fill materials is able to increase the commercial value and application of the waste, while reducing demand for natural sand. Hence, filling geotube breakwaters with quarry dust promotes concept of sustainable material management (Ljungberg, 2007, Sivakumar et al., 2012).

From Figure 4.15 to Figure 4.17, quarry dust and sand slurry showed the similar results in terms of P_o, H and T. This is because the similar sediment size and properties of the two sediments. Overall, the quarry dust slurry creates higher after-consolidation-height of geotube and causes slightly lower tensile force in geotube as compare to sand slurry. Sediment of quarry dust is bigger in size hence creating more pores in between each particle. Therefore, the final height of geotube filled with quarry dust is slightly higher compared to sand filled geotube. Greater particle size allows the water to flow through faster and lower density cause less force on the geotube, thus explained the lower T in quarry dust filled geotube. Figure 4.19 depicts that higher water content in the fill causes greater change in final height.

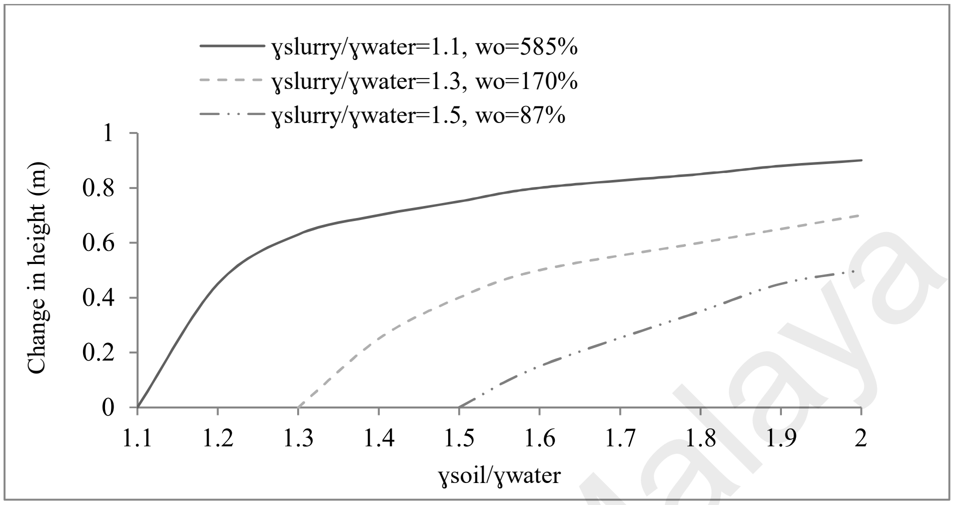


Figure 4.19: Change in H against the density of soil

4.3.2.4 Influence of Designed Height of Geotube

The influence of geotube's designed height to the P_o and T were evaluated. Figure 4.20 to Figure 4.21 show relationship between geotube's H, P_o and T, with sediment concentration of 30% and under non-submerged condition.

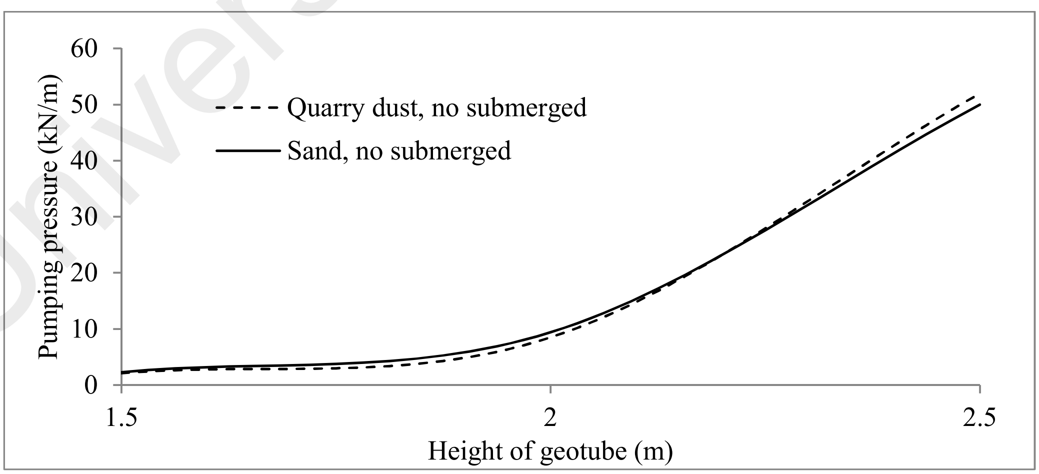


Figure 4.20: Relationship between geotube's H and P_o (sediment concentration of 30% and non-submerged condition)

Figure 4.20 shows increment of H from 1.5 m to 2.0 m, causes 3 times greater P_o in sand filled and quarry dust filled geotubes. However, when H increases from 2.0 m to 2.5 m, increment in P_o is 5 times. This happened because the H is close to D. At this stage, continuous pumping process increases P_o, but do not affect H of geotube significantly.

Figure 4.21 shows the T in geotube when the designed height is varied. The trend of graphs in Figure 4.21 is similar to the Figure 4.20. This is because when the P_o is significant, geotube experiences greater tensile forces exerts from inner side of geotube. Geotube experiences T from two directions (circumference and axial). Forming geotube with a height closer to ideal height requires high P_o and hence exerts high T in the geotube.

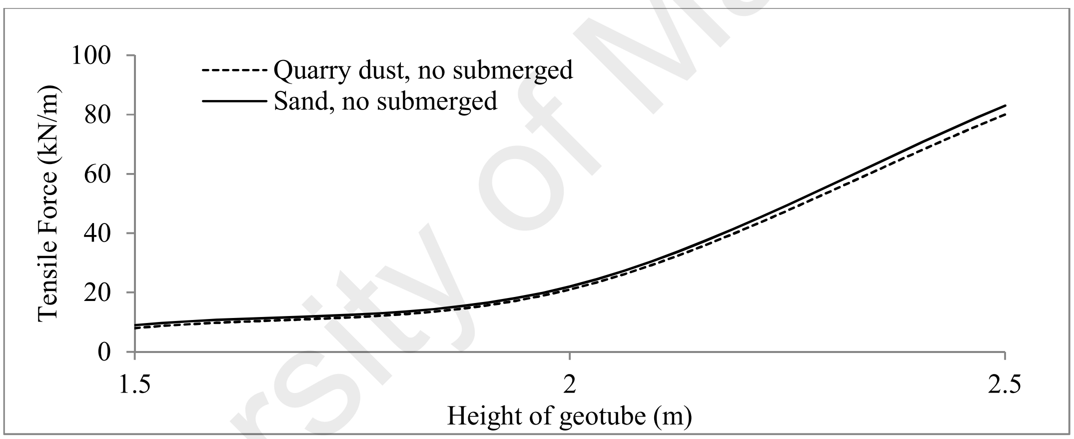


Figure 4.21: Relationship between designed height of geotubes and tension forces experienced in geotube (sediment concentration of 30% and non-submerged condition)

4.3.3 Verification of Results

Reliability of outputs were verified by comparing analysis results with observed geometry of geotube in SHD. The circumference of the sand-filled geotube on-site were 8.6 m. Average final height of geotube breakwaters on 2015 is 1.71 m, with a base width of approximately 2.64 m. The maximum width of the geotube was 3.48 m and located 0.68 m from base. The five observation points were plotted in Figure 4.22 and compare with the analysis result. Analysis result is closely agreed with observation result, with a difference range of 2% to 4%. Therefore, analysis results are satisfactory.

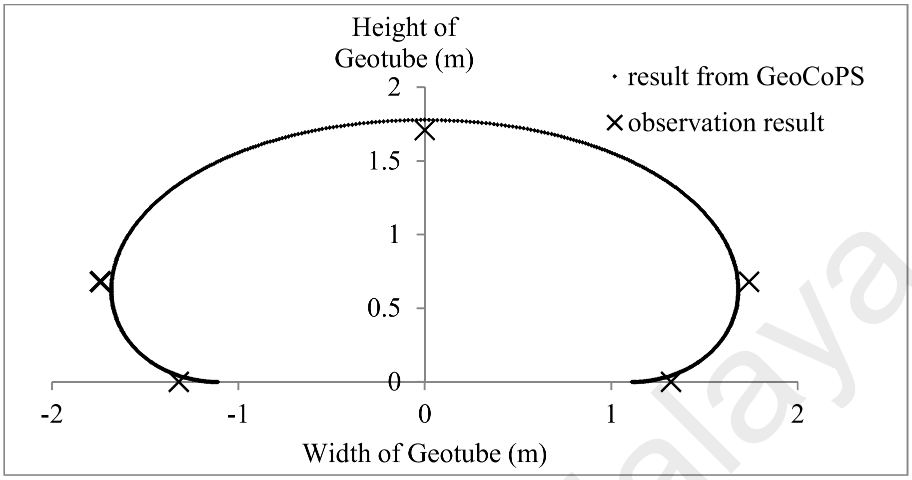


Figure 4.22: Geometry of geotube from calculation and on-site observation

4.4 External Stability of Geotube Breakwater

4.4.1 Hydrodynamic Impacts

The typical wave height, H_w in SHD range between 0.5 m to 1.0 m. For simulation, wave height of 0.5 m, 1.0 m and 1.5 m were adapted to study the behaviours of geotube under hydrodynamic impacts. Equation 3.20 is used to calculate hydrodynamic force hitting on geotube's curved surface. Calculated hydrodynamic pulsating forces are 9.0 kN, 31.0 kN and 62.5 kN, representing forces sourced from 0.5 m, 1.0 m and 1.5 m wave heights.

Figure 4.23 to Figure 4.25 show the typical reactions, or deformation of geotubes, and foundation due to the hydrodynamic impacts. Results show that the side of geotube hit by wave cycles tends to lift up, while the protected side tends to sink or settle. Therefore, the foundation at the protected area tends to settle greater. Data from all models were extracted and plot into graphs to show the influences of each parameter. Discussions on these data are covered in Section 4.4.2 to Section 4.4.6.

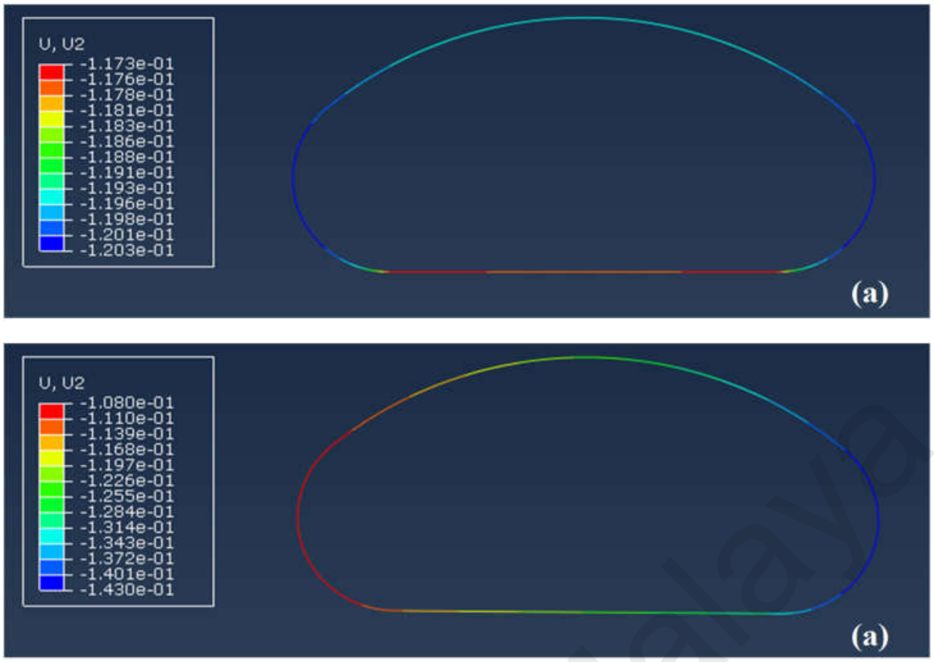


Figure 4.23: Deformation of geotube breakwater (a) without wave force; and (b) with wave force

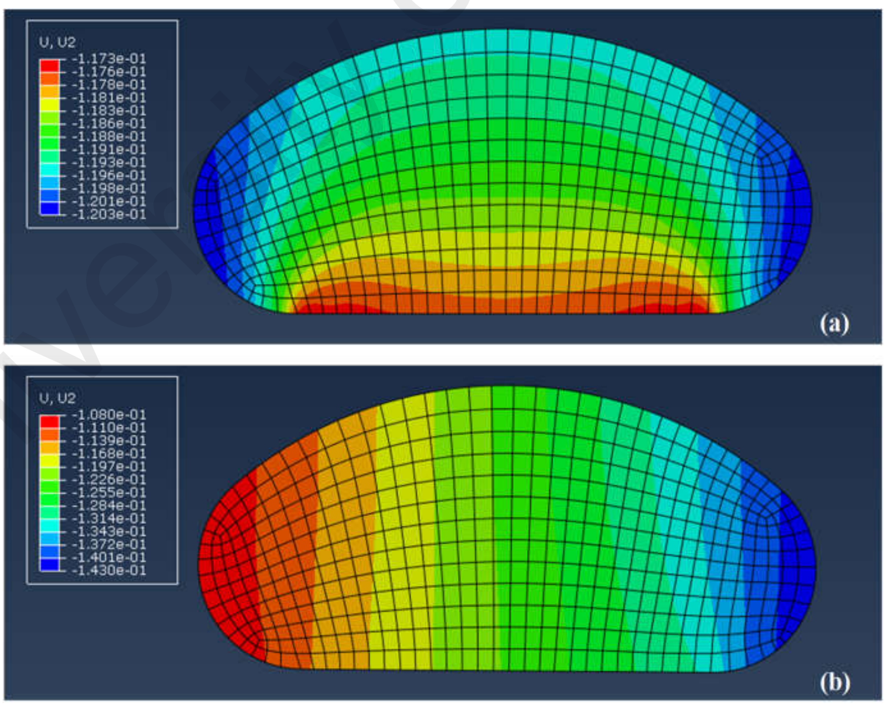


Figure 4.24: Deformation of filling material (a) without wave force; and (b) with wave force

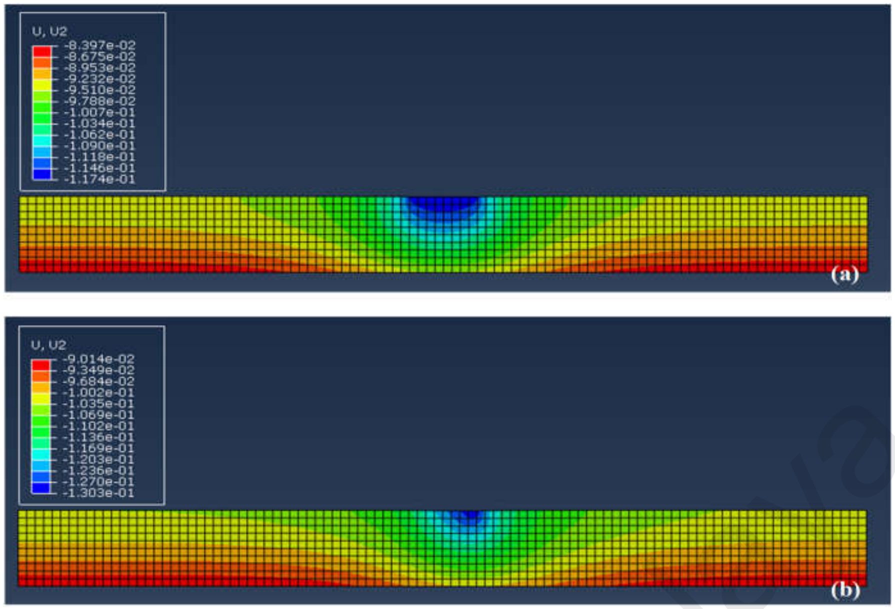


Figure 4.25: Deformation of muddy foundation (a) without wave force; and (b) with wave force

4.4.2 Influence of Properties of Foundation

Many research simplified simulation analyses by assuming very stiff or not deformable foundation. However, such assumptions can lead to underestimation of impacts from foundation deformation to the stability of geotube breakwaters. Moreover, this study focuses on simulation of geotube breakwaters on soft and deformable muddy foundation.

Figure 4.26 and Figure 4.27show influences of foundation stiffness to the behaviours of geotube breakwater under various wave forces. Properties of geotubes and filling materials were set to constant. Foundation properties adapted were soft and stiff foundation. The soft foundation properties employed in the simulation is based on real muddy foundation's properties from SHD. While, non-deformable foundation was simulated by employing high young modulus (i.e. E = 1 GPa) value in soil, as suggested by Khalilzad and Gabr (2011). The foundation properties are as stated in Table 4.3.

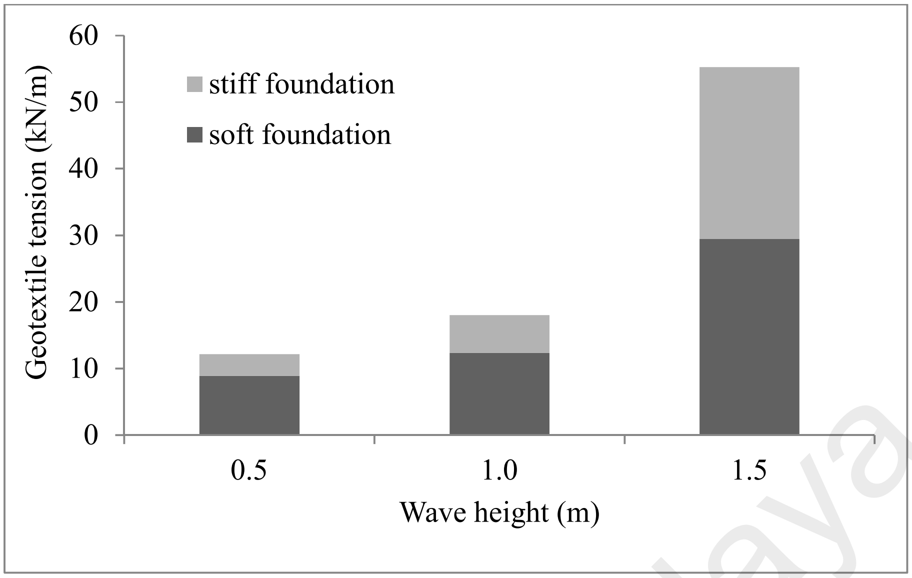


Figure 4.26: T in geotube under different foundation stiffness and wave height

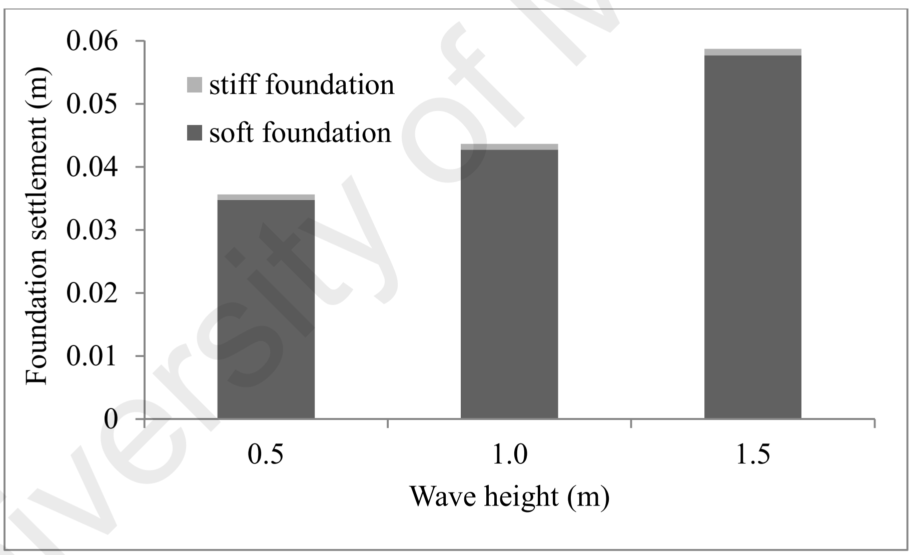


Figure 4.27: Foundation settlement under different foundation stiffness and wave height

Figure 4.26 shows T in geotubes for soft and stiff foundation. Result shows that placing geotubes on soft and stiff foundation exert different T in geotube. The difference in geotube's T are 63.8%, 54.3% and 12.2%, for H_w 0.5 m, 1.0 m and 1.5 m respectively. In the case of soft foundation, T is higher because the geotube deformed when the soft foundation settled. Deformation of structure exerts extra tension in the geotube. The T exerted in geotube must not exceed the geotube's tensile strength to prevent structural damage or failure.

Figure 4.27 illustrates soft foundation settlement range from 3.5 cm to 5.7 cm under different H_w. However, for the case of stiff foundation, settlements are negligible, or approximately 0.1 cm under all H_w. Therefore, assumption of non-deformable foundation can lead to underestimation of structural settlement which can possibly causes instability or failure of structure. Thus, assumptions of very stiff foundation are not recommended in simulation of geotube breakwaters on muddy beach, as this affect the reliability of results.

4.4.3 Influence of Filling Material's Properties

4.4.3.1 Density of Filling Material

Density of filling material directly related to total weight of geotube breakwater because the structure's weight is mainly from filling material. Figure 4.28 shows the typical location of greatest tension experienced in a geotube breakwater when the hydrodynamic forces were applied on the tube.

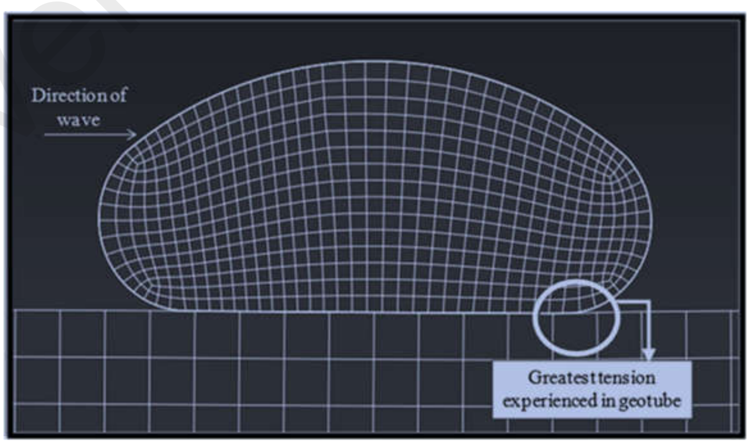
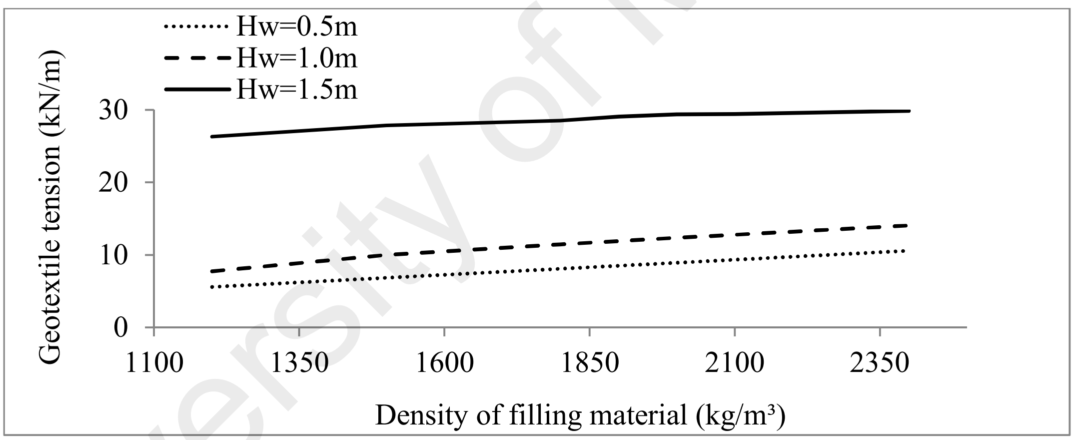


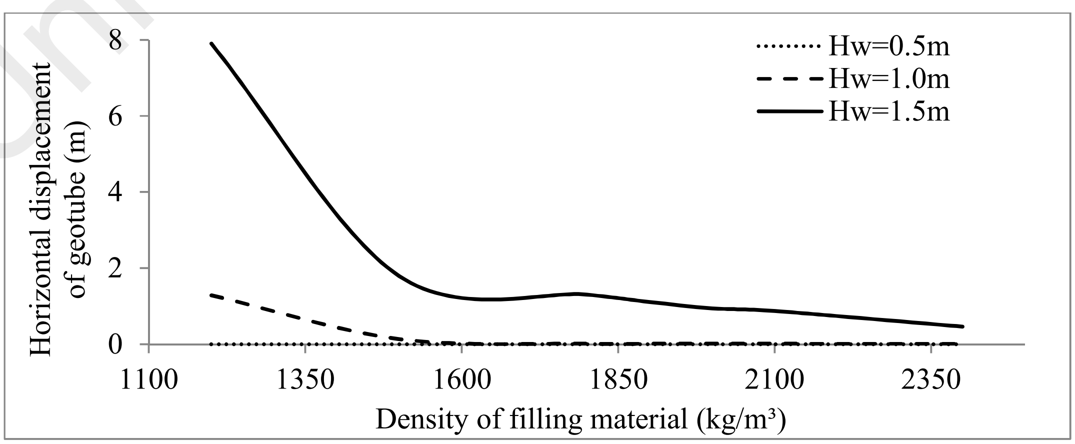
Figure 4.28: Location of greatest tension in geotextile during service period

Result in Figure 4.29 (a) indicates greater density of filling material directly causes greater T in geotube. Filling material is captured inside the geotextile unit. Therefore, larger weight of filling material exerts greater forces in the geotube. However, the impacts fill material's density to the T in geotube, is less significant as compared to wave height.

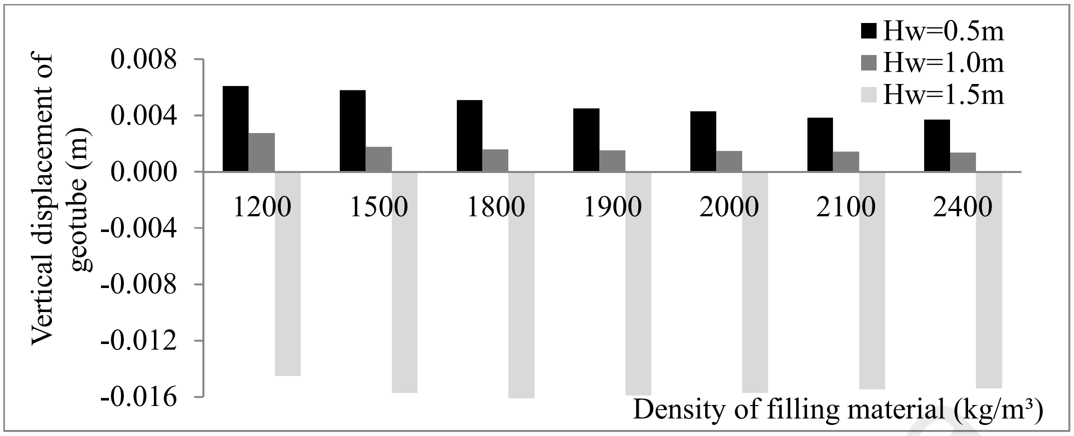
Wave cycles that hit on geotube breakwaters exert additional forces on the geotube. The additional force is also known as hydrodynamic pulsating force and causes increment of T in geotube. Result shows that T in geotube ranged between 5.5 kN/m to 10.5 kN/m when Hw is 0.5 m, ranged between 7.7 kN/m to 14.1 kN/m when H_w is 1.0 m, and ranged between 26.3 kN/m to 29.9 kN/m when H_w is 1.5 m. Increment of H_w from 0.5 m to 1.0 m causes an average 40% increment in T. However, when Hw increases from 1.0 m to 1.5 m, T shows significant increment of 150%. This indicates H_w has significant impact to the behaviours of geotube breakwaters.



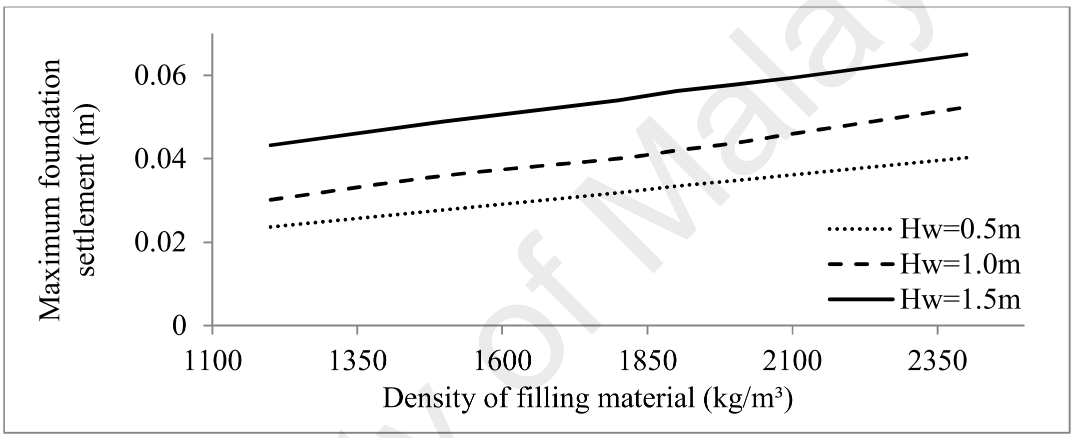
(a)



(b)



(c)



(d)

Figure 4.29: Influence of the density of filling material to (a) geotextile tension; (b) horizontal displacement of geotube; (c) vertical displacement of geotube; and (d) foundation settlement

Figure 4.29 (b) presents relationship between density of filling material to the lateral movement of geotube breakwater on muddy beach. Result shows H_w of 0.5 m causes very negligible horizontal movement on geotube breakwater (less than 2 mm). However, H_w of 1.0 m causes instability of geotube against sliding, especially when density of filling material is low. At 1.0 m H_w, if density of filling material is less than 1500 kgm^{-3}, resistance of geotube for lateral movement reduced drastically. Forces that causes sudden significant change in horizontal displacement were used to calculate geotube's factor of safety against sliding. Calculation of factor of safety are discuss in detailed, in Section

4.3.7. For example, density of filling material is 1500 kgm^{-3}, geotube can withstand hydrodynamic pulsating force of 28 kN, without significant lateral movement. While, when H_w is 1.5 m, geotube breakwater has insufficient resistance over the high hydrodynamic pulsating forces. Therefore, in all the cases with H_w = 1.5 m, geotube showed significant lateral movements ranged from 0.46 m to 7.90 m. In this case, geotube breakwater experiences sliding failure. In a nut shell, density of filling material is major factor that causes instability of geotube's against sliding or resistance over wave load.

Relationship between sliding movement and overturning movement of geotube breakwater under waves can be observed from Figure 4.29 (b) and Figure 4.29 (c). In Figure 4.29 (c), result illustrates insignificant geotube's vertical movement (uplifting movement of geotube), which ranged from -1.6 cm to 0.6 cm. Negative value represents settlement of geotube instead of being lifted up. Measurement was done by observing vertical displacement of geotube at location shown in Figure 4.30. Under H_w = 0.5 m and H_w = 1.0 m, geotube was lifted at a range less than 0.61 m. However, under a greater wave height, where the hydrodynamic forces are greater, the geotube were compressed and settled up to 0.016 m. From the results, we noticed greater H_w, geotube breakwater undergoes more significant sliding motion and less overturning motion.

Figure 4.29 (d) presents the settlement of muddy foundation. Results shows that higher density of fills directly causes greater settlement of muddy foundation. While greater wave height means greater hydrodynamic load exerts on geotube structure, and causes larger additional load on the foundation, consequent in greater foundation settlement. However, density of filling materials does not have noticeable effect on the change of height of geotube structure. The maximum change of height of geotube in all cases, under hydrodynamic pulsating loads is only 0.235 cm, which is negligible.

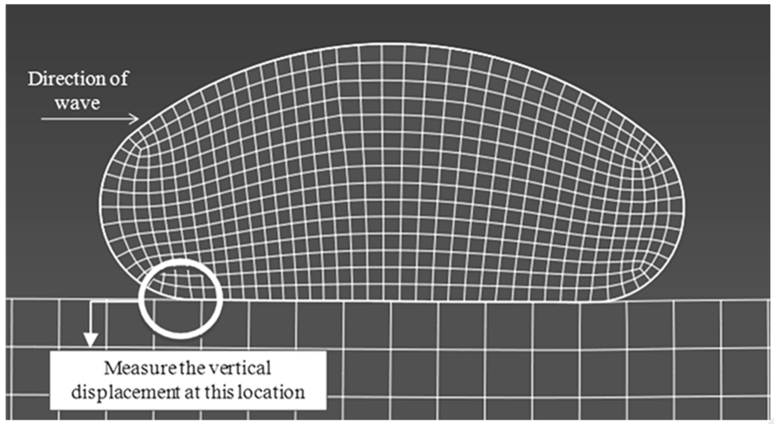


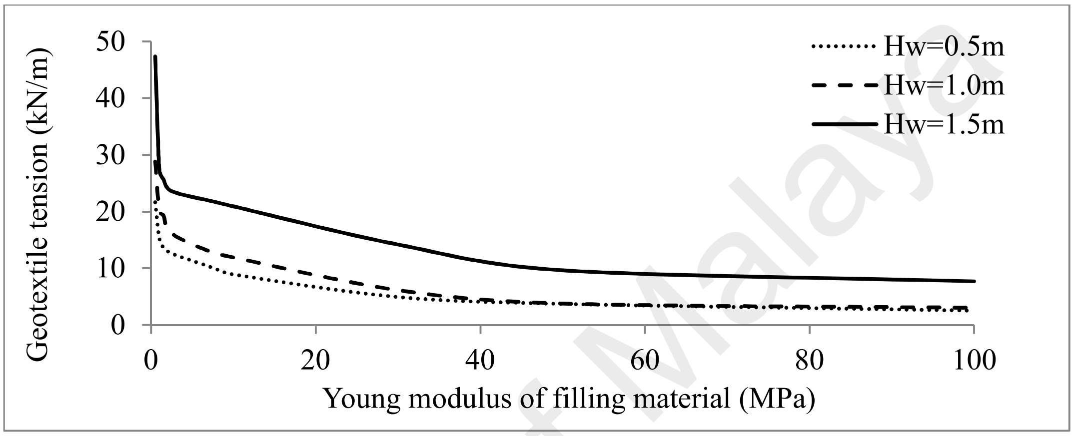
Figure 4.30: Location of geotube's vertical displacement measurement

Generally, density of filling material plays very vital roles for external stabilities of geotube breakwater. Greater fill material's density allows greater resistance over sliding and overturning displacement of the structure. But at the same time, heavier structure self-load might lead to excess settlement of soft foundation. Hence, it is important to evaluate the interaction of coastal soil and geotube breakwater before installation of structure on site, to prevent failure or damage due to excessive sliding, overturning or settlement issue.

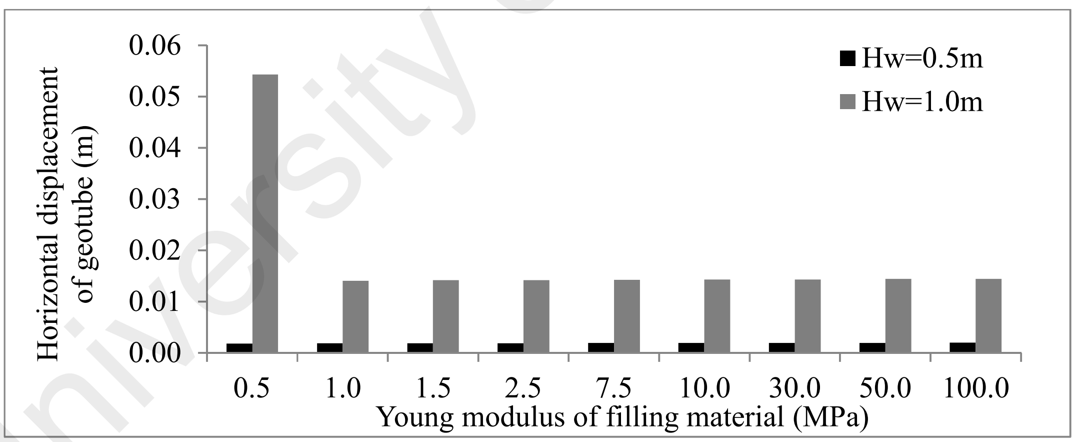
4.4.3.2 Stiffness of Filling Material

Figure 4.31 shows the influence of soil stiffness (Young's modulus, E) to behaviours of geotube breakwater and muddy foundation. Result in Figure 4.31 (a) depicts lower E (less stiffness) causes greater T in geotube breakwater. This happen because filling materials of lower E experience greater distortion under waves, which induce extra tension in geotube.

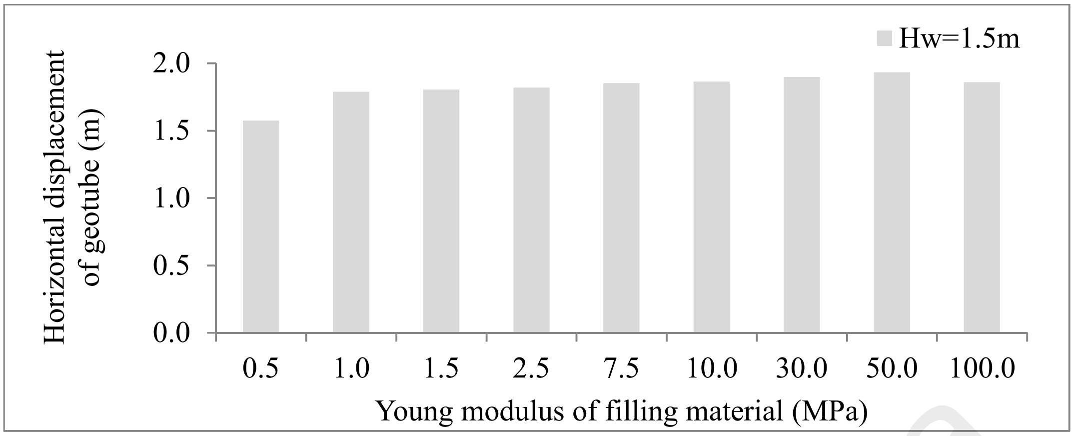
Figure 4.31 (b-i) and Figure 4.31 (b-ii) show sliding movement of geotube under different H_w. It is noticeable that under H_w of 0.5 m and 1.0 m (which represents the real H_w range in SHD), geotube breakwater experiences insignificant lateral movement. However, under H_w of 1.0 m, if stiffness of filling material is low, where E is less than 1MPa, geotube undergoes significant lateral movement. The great horizontal displacement is caused by combination of sliding motion of geotube due to instability against sliding, and distortion of filling material due to lower E (less stiffness). While under H_w of 1.5 m as shown in Figure 4.31 (b-ii), great hydrodynamic forces caused significant lateral movement of geotube breakwater, range from 1.58 to 1.93 m. Hence under H_w of 1.5 m, resistance of geotube against sliding is very low.



(a)



(b-i)



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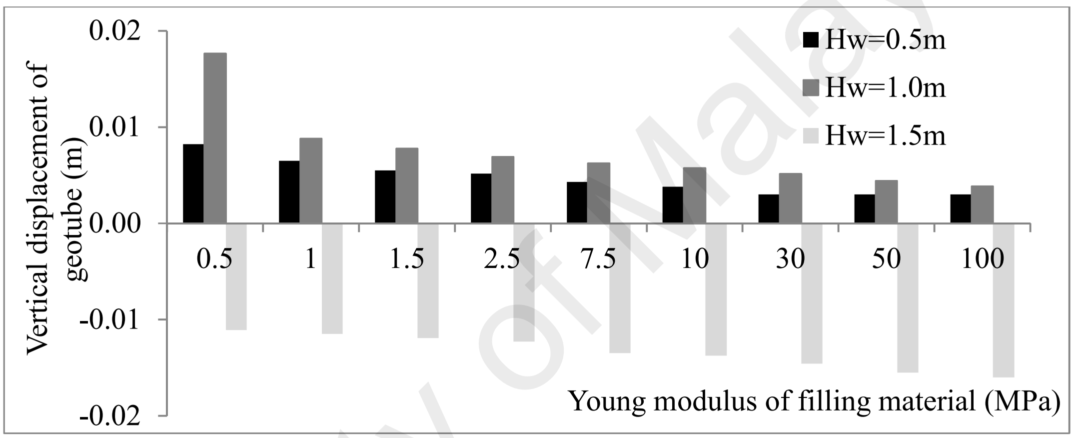
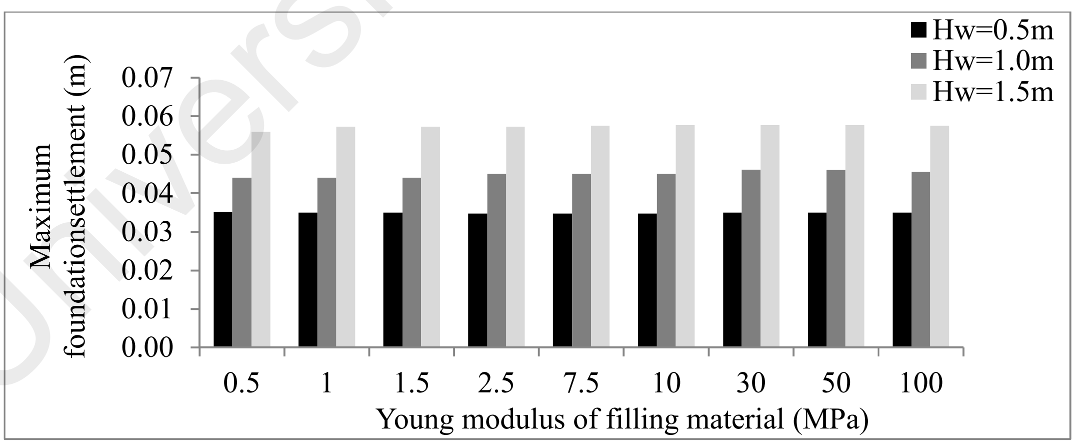


(c)



(d)

Figure 4.31: Influence of the young modulus of filling material to (a) geotextile tension; (b-i,ii) horizontal displacement of geotube; (c) vertical displacement of geotube; and (d) foundation settlement

Results from Figure 4.31 (c) shows the minimal influence of soil stiffness to geotube's uplifting movement under hydrodynamic force. At H_w = 0.5 m, uplifting displacement range from 0.003 m to 0.008 m. At H_w = 1.0 m, uplifting displacement range from 0.004 m to 0.017 m. Under H_w = 1.0 m, lower E of filling material causes greater vertical displacement of geotube. This happen due to combination of lifting displacement and geotube's distortion under greater wave forces. Filling material in geotube is less rigid if E is less than 1MPa, thus undergo change in shape. Under H_w = 1.5 m, greater hydrodynamic force was applied on geotube structure. Hence the geotube structure was compressed downward, towards the soft foundation, causing settlement to the muddy foundation as well.

Nevertheless, the Figure 4.31 (d) shows that the stiffness of filling material do not have significant influence to foundation settlement. The settlement of muddy foundation is similar for cases under the same H_w. However greater H_w induces greater hydrodynamic pulsating force on geotube and increase risk of soft soil settlement.

In a nutshell, stiffness of soil directly affect tension in geotube, and resistance of geotube over sliding and overturning motion. The stiffness of soil does not directly influence foundation settlement.

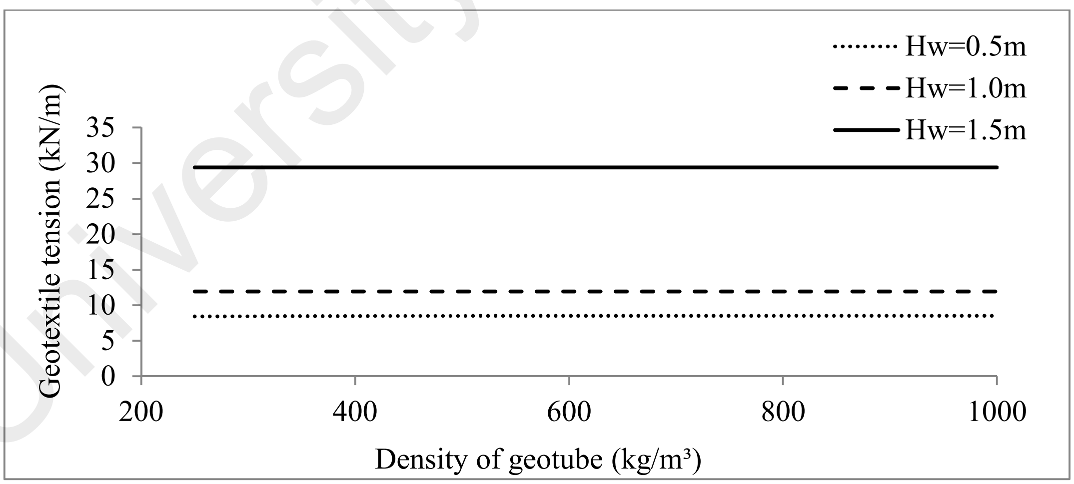
4.4.4 Influence of Properties of Geotube

4.4.4.1 Density of Geotube

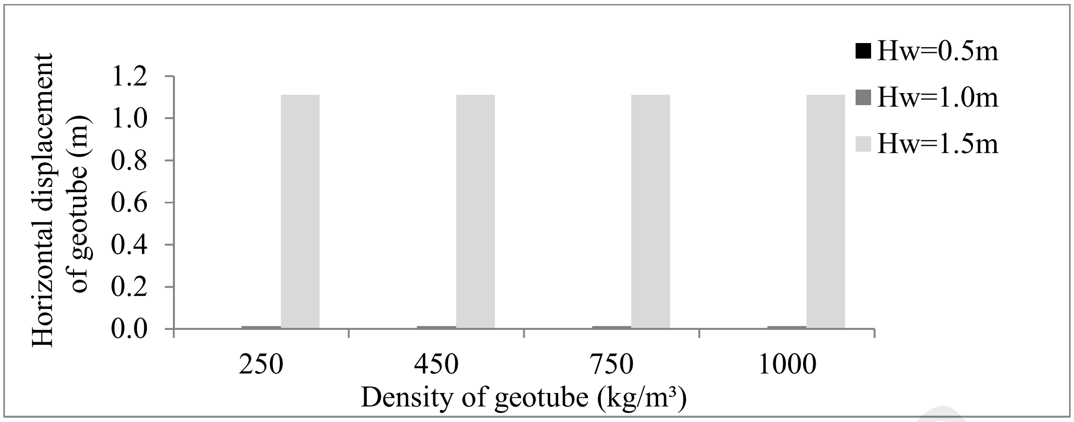
In order to analyse influence of geotube's properties to the behaviour or performance of geotube breakwater, filling material's properties were set constant with a density of 2000 kgm^{-3} and young modulus of 10 MPa. Figure 4.32 (a) to Figure 4.32 (d) show minimal influence of the density of geotube to overall behaviours of geotube breakwater. T in geotube, horizontal movement, vertical displacement and foundation settlement are not directly affected by density of geotube. However, the wave height greatly affects the behaviour of geotube, especially when H_w = 1.5 m.

From Figure 4.32 (a), T in geotube for cases where H_w = 0.5 m, are approximately 8.5 kN/m. However, under H_w = 1.0 m and H_w = 1.5 m, T increased by 40% and 145% respectively. Meanwhile, Figure 4.32 (b) illustrates instability of geotube breakwater against the 1.5 m wave height. Under H_w = 1.5 m, geotube breakwater experiences sliding failure. While, sliding motion of geotube breakwater under H_w = 0.5 m and H_w = 1.0 m ranged from 0.002m to 0.015m.

Figure 4.32 (c) and Figure 4.32 (d) show density of geotube do not influence geotube vertical displacement and foundation settlement. Results obtained from analyses are same with the results from Figure 4.29 (c) and Figure 4.29 (d), for case density = 2000 kgm^{-3}. Therefore, pointed out the insignificant influence of geotube's density to the overall behaviours of geotube breakwater.



(a)



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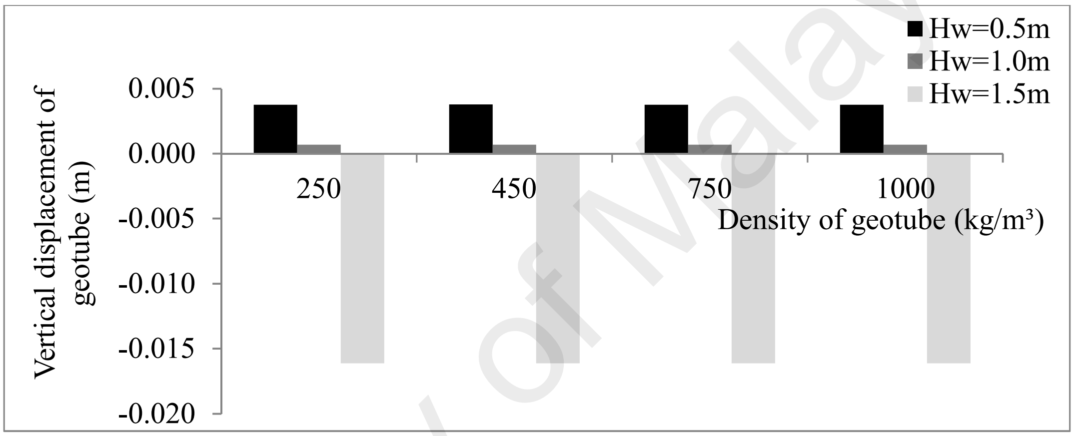

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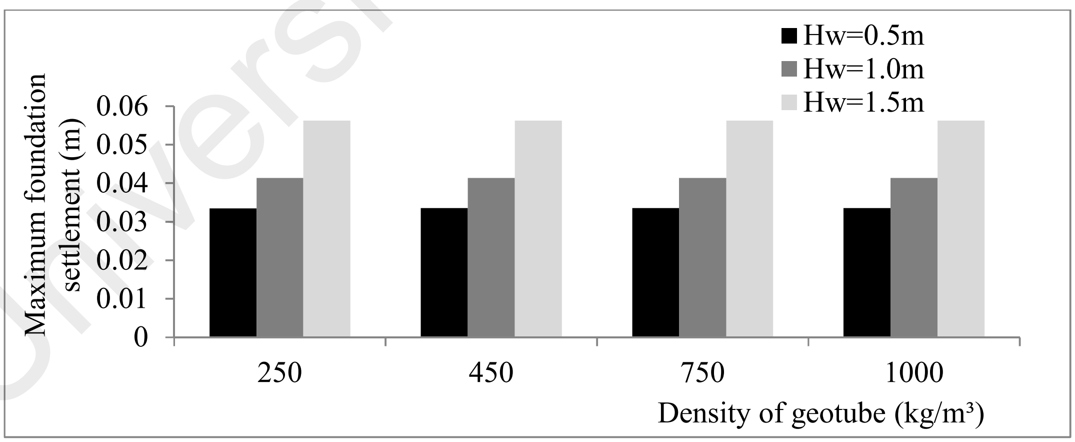
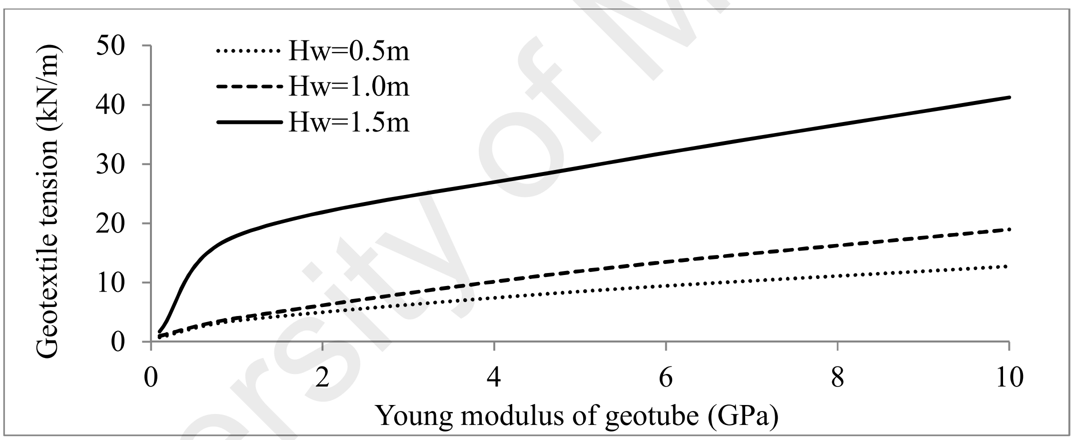


(d)

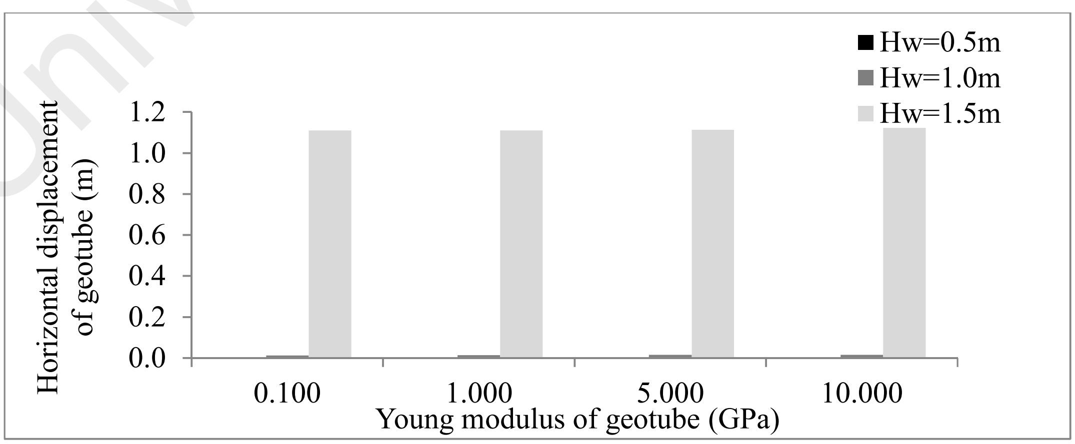
Figure 4.32: Influence of the density of geotube to (a) geotextile tension; (b-i,ii) horizontal displacement of geotube; (c) vertical displacement of geotube; and (d) foundation settlement

4.4.4.2 Stiffness of Geotube

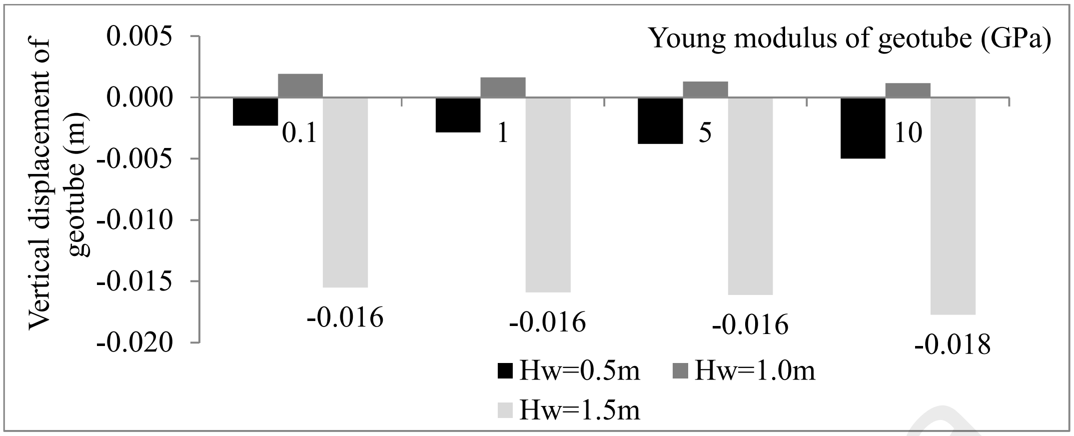
Higher young modulus of geotube represents stiffer properties of geotube. From Figure 4.33 (a), result shows T increase when E increase. The stiffer the geotextile, the more rigid the structure is. Hence when load apply on the structure, geotube tends to hold the shape in place, but this causes higher tension at the bottom of the structure. If stiffness of geotube is lower, the structure is more flexible and can be distorted when loads applied. Thus, T at the bottom of geotube is reduced. Figure 4.33 (b) to Figure 4.33 (d) show the insignificant influence of geotube's stiffness to sliding movement, vertical movement and foundation settlement.



(a)



(b)



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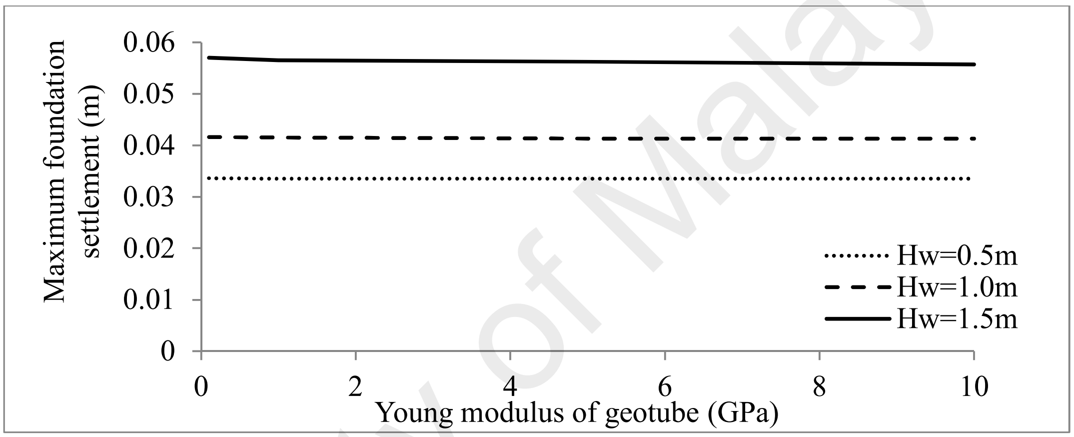


(d)

Figure 4.33: Influence of the young modulus of geotube to (a) geotextile tension; (b-i, ii) horizontal displacement of geotube; (c) vertical displacement of geotube; and (d) foundation settlement

4.4.5 Influence of Wave Height

Wave height, H_w is one of the major factors that affect external stability of geotube breakwater. Figure 4.29 to Figure 4.33 show increment in H_w caused less stability of geotube breakwater. The effect from H_w increment is very significant when H_w change from 1.0 m to 1.5 m, as compare to the change from 0.5 m to 1.0 m. For instance, comparing two cases where case A is the increment of H_w from 0.5 m to 1.0 m, while case B is the increment of H_w from 1.0 m to 1.5 m. Comparisons were carried out for all the results reported in Figure 4.29 to Figure 4.33.

For T in geotube, results obtained for case B is approximately 2 to 3 times the results obtained for case A. While, results for lateral movement obtained in case B is 8 to 11 times the results obtained in case A. The vertical displacement for case B is approximately 5 to 16 times greater than case A. However, the foundation settlement is not affected significantly by the increment in wave height. The results for case A and case B differ by ratio of 1. This means that the increment in wave height is almost directly proportional to the settlement of foundation.

4.4.6 Quarry Dust as Alternative Fills

4.4.6.1 Behaviours of Geotube Breakwater

FEM results show that properties of filling material, foundation and wave height are major influencing factors for the overall external stability of geotube. The FEM models employed real wave climates and foundation properties from SHD. In this section, alternative filling material, quarry dust was analysed and discussed.

In normal practice, river sand is the most commonly used geotube's filling material. However, there are studies described the weakness of using sand, such as sand dredging activities that lead to erosion of dredging site. Hence sustainability issue is debateable when human erode an area to protect another eroded area.

Suitability of quarry dust as geotube's filling material were analysed by employing properties of quarry dust in FEM model. The effectiveness of sand as filling material were also studied and reported. Comparison between quarry dust and sand fills were carried out. Properties of sand and quarry dust were stated in Section 4.2.1 and Appendix A. Foundation properties adapted were stated in Table 4.3 and Appendix C. While, wave heights employed were 0.5 m, 1.0 m and 1.5 m.

Result shows that under different wave heights (H_w = 0.5 m, 1.0 m and 1.5 m), T sand filled geotube shows 1.5 times to 2 times greater in T, compared to quarry dust filled geotube. This happened because higher stiffness of quarry dust holds the geotube in shape better and experienced less distortion under waves. Hence less T at the bottom of geotube.

Quarry dust has lower density compared to river sand. Thus, quarry dust filled geotube breakwater experiences greater lateral displacement under waves. Lower density indicates lighter structure weight, hence lower resistance against waves. As compared to sand filled geotube, quarry dust filled geotube shows 8% to 21% greater lateral displacements, under H_w range 0.5 m to 1.5 m.

Uplifting displacement or overturning displacement of sand filled geotube is approximately 50% less than quarry dust filled geotube. The uplift displacements are insignificant compared to lateral displacements, which ranged from 0.4 cm to 1.5 cm for sand filled geotube. While, uplifting displacements for quarry dust filled geotube ranged from approximately 0.7 cm to 3.0 cm.

The sand filled geotube has heavier self-load hence causing more settlement on the muddy foundation, ranged from 3.5 cm to 6.8 cm. While, quarry dust filled geotube caused settlement between 0.2 cm to 2.7 cm.

4.4.6.2 Monetary and Environmental Considerations

Geotube breakwaters were commonly used as coastal structures and were commonly filled with dredged natural sand. On a sandy beach, sand fills are normally dredge on-site to save project cost. However, sand dredging activities itself caused bathymetry change in the coastal area, at the same time affect the aquatic livings habitats. While on a muddy beach, sand fills are obtained from local suppliers due to the lacking of sand supply at muddy coasts. For example, geotube breakwaters in SHD were filled with sand supplied from local suppliers, but not dredged from the muddy beach.

On the other hand, quarry dust has very limited engineering application and often treated as waste. The quarry dust was always mishandled and dumped in open area. Using the quarry dust to fill the geotextile coastal structures can widen its applications, at the same time mitigate the mishandle issues of quarry dust. Besides, with quarry dust, demand for the exploitation of natural sand will reduce. Thus, the quarry dust filled geotextile tube is a more sustainable approach for coastal protection.

Apart from the environmental and logistical advantages, quarry dust also has advantages in term of cost. Quarry dust has very low economical value as it is not widely used for engineering applications. Especially on muddy beaches where supply of sand and quarry dust are lacking, the transportation of these sediments to the site is essential, as the extra fine mud sediments are not suitable to be used as filling material of the geotextile tubes. In Malaysia, the cost of supply and delivery to the pilot project site in Selangor is about MYR 400 per load (10-wheel lorry; 22 tonnes) for quarry dust and MYR 850 per load for sand. Hence, especially utilization of quarry dust instead of sand in filling the geotextile tubes, can cut down the fill material cost up to 53%.

4.4.7 Theoretical Derivation of the Stability of Geotube

Oh and Shin (2006) employed formulas to calculate factor of safety against sliding, overturning and bearing capacity for geotube breakwaters. The formulas are described in section 2.6.4. Factors of safety are often use as simplification method to predict stability of coastal structures. However, factor of safety calculated from formula show only approximate stability level of structure but the extent of deformation or displacement cannot be determined. FEM enable us to have an insight of deformation or displacement of a structure.

Figure 4.34 shows factor of safety against sliding of geotube, under H_w of 0.5 m, 1.0 m and 1.5m. The factors of safety were calculated by using Equation 2.6. Factor of safety against sliding can also be calculated from FEM by determining the force which causes sudden significant change in lateral movement. For this study, there is a sudden change in lateral movement when H_w is 1.0 m and density used is 1200 kgm^{-3}. The Figure 4.35 shows the comparison between factors of safety calculated from FEM analysis and calculated with Equation 2.6.

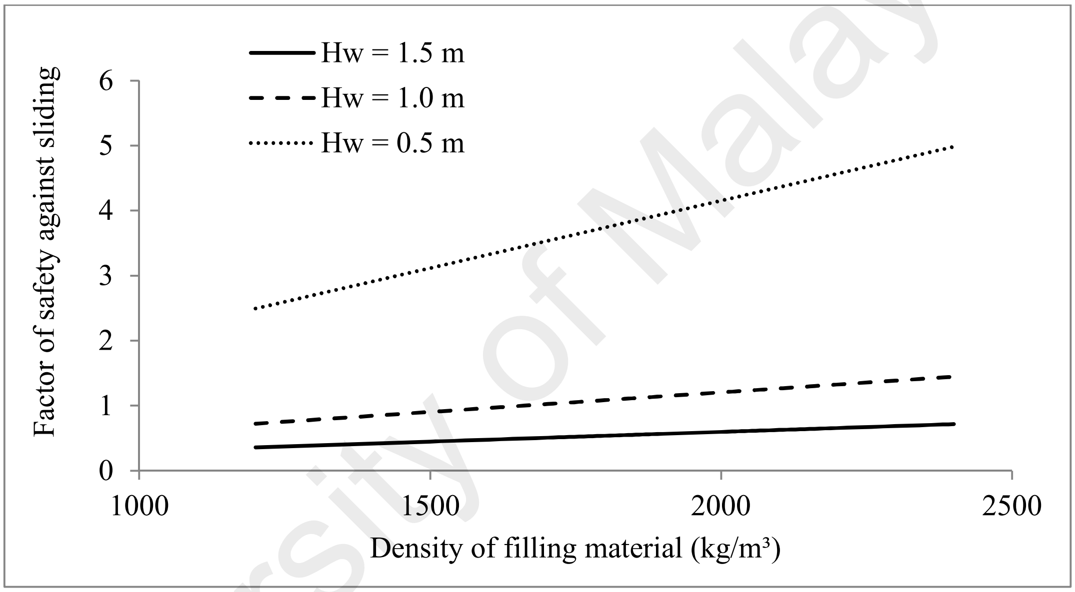


Figure 4.34: $FoS_{sliding}$ calculated using Equation 2.6

Figure 4.35 shows that results obtained from formula and results from FEM analysis are closely agreed. The R^2 value of the comparison graph is 0.9988 which represents very high accuracy of results. Therefore, FEM model used can give a good prediction of sliding resistance of geotube breakwater against hydrodynamic forces.

All analysis models in this study did not show significant overturning issue. The lifting displacements do not exceed 2 cm in all FEM analyses cases. The results from FEM tallies with factor of safety against overturning calculated from Equation 2.7. At H_w of 1.5 m, $FoS_{overturning}$ ranged from 1.45 to 2.90. While when H_w = 1.0 m, $FoS_{overturning}$ ranged from

2.93 to 5.85. The $FoS_{overturning}$ when H_w = 0.5 m are very high, ranged from 10.08 to 20.14. Therefore, overturning of structure is not the major concern in the design of geotube breakwater, under SHD's environment conditions. Geotubes experienced greater sliding motion than overturning in SHD's muddy coast.

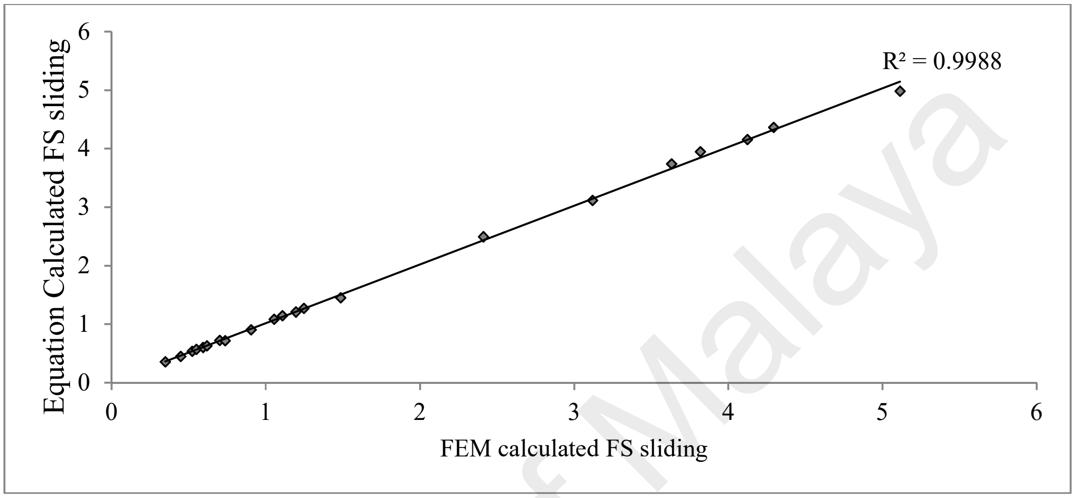


Figure 4.35: Comparison of the $FoS_{sliding}$ calculated from FEM and from Equation 2.6

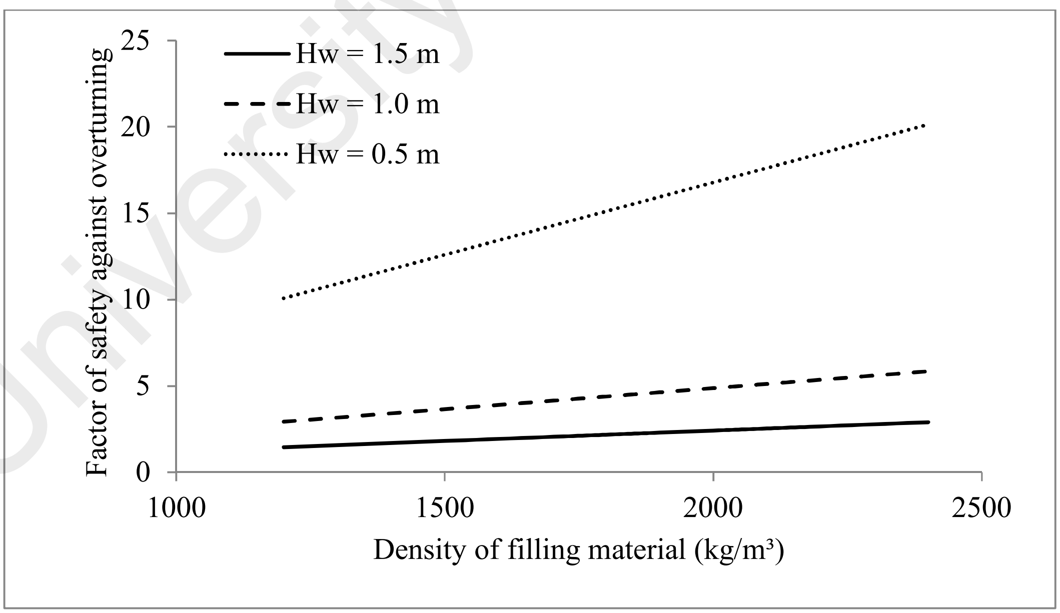


Figure 4.36: $FoS_{overturning}$ calculated using Equation 2.7

Figure 4.37 shows that results obtained from formula and results from FEM analysis are closely agreed. The R^2 value of the comparison graph is 0.9943 which represents very high accuracy of results. Therefore, FEM model used can give a good prediction of overturning resistance of geotube breakwater against hydrodynamic forces.

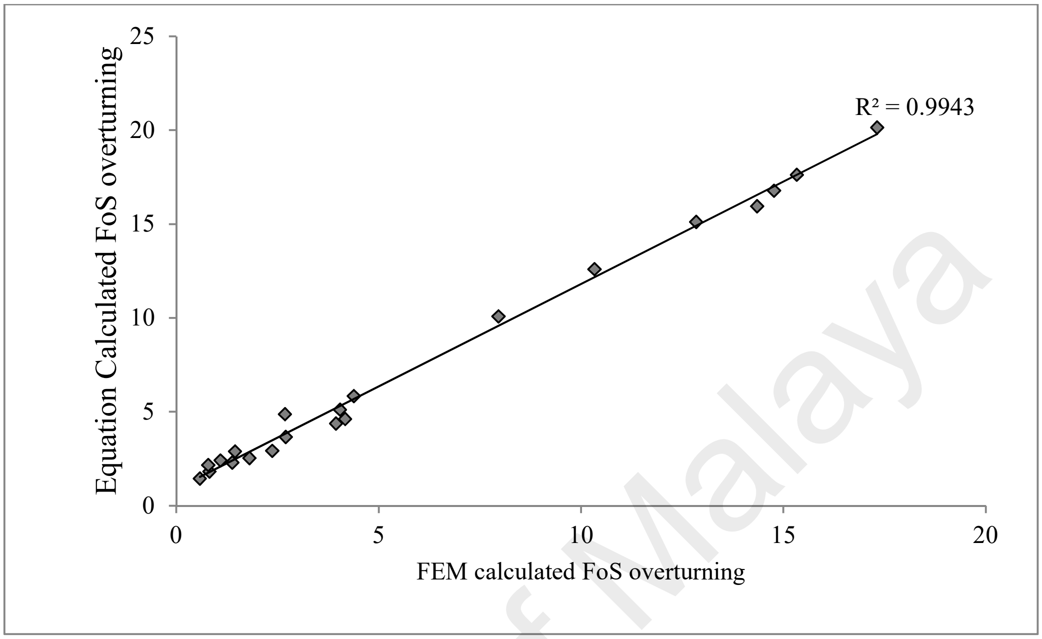


Figure 4.37: Comparison of the $FoS_{overturning}$ calculated from FEM and from Equation 2.7

Beside stability of geotube structure against sliding and overturning, factor of safety for bearing capacity is also very crucial especially for coastal protection structure design on muddy foundation. By adopting Terzaghi's method for bearing capacity calculation, predicted bearing capacity is 195.3 kPa. While by using FEM method, the bearing capacity can be obtained by continuously adding load over the geotube structure and observed foundation settlement under the structure. From load-displacement curve in Figure 4.38, the FEM calculated bearing capacity is 134.0 kPa.

The main factor that caused the difference in the results obtained from Terzaghi's method and FEM analyses is due to different assumptions used. For instance, the Terzaghi’s equation soil is assumed to be perfectly plastic. While in FEM analyses, the elasto-plastic properties were assumed for muddy foundation. Therefore, the

$FoS_{bearingcapacity}$ calculated from Terzaghi's method is higher than the $FoS_{bearingcapacity}$ calculated according to FEM method. The result in Figure 4.39 shows that trend or pattern of curves found in this study is in good agreement with results calculated from formula. However, it is noticeable that $FoS_{bearingcapacity}$ calculated from Terzaghi's method is more conservative as compare to the FEM results. Hence, the FEM model used in this study is able to give a result which considered the closer to real foundation conditions (soil is not perfectly plastic or perfectly elastic). The results obtain from FEM models has lower chance of underestimating the potential risk for excessive foundation settlement or bearing capacity failure.

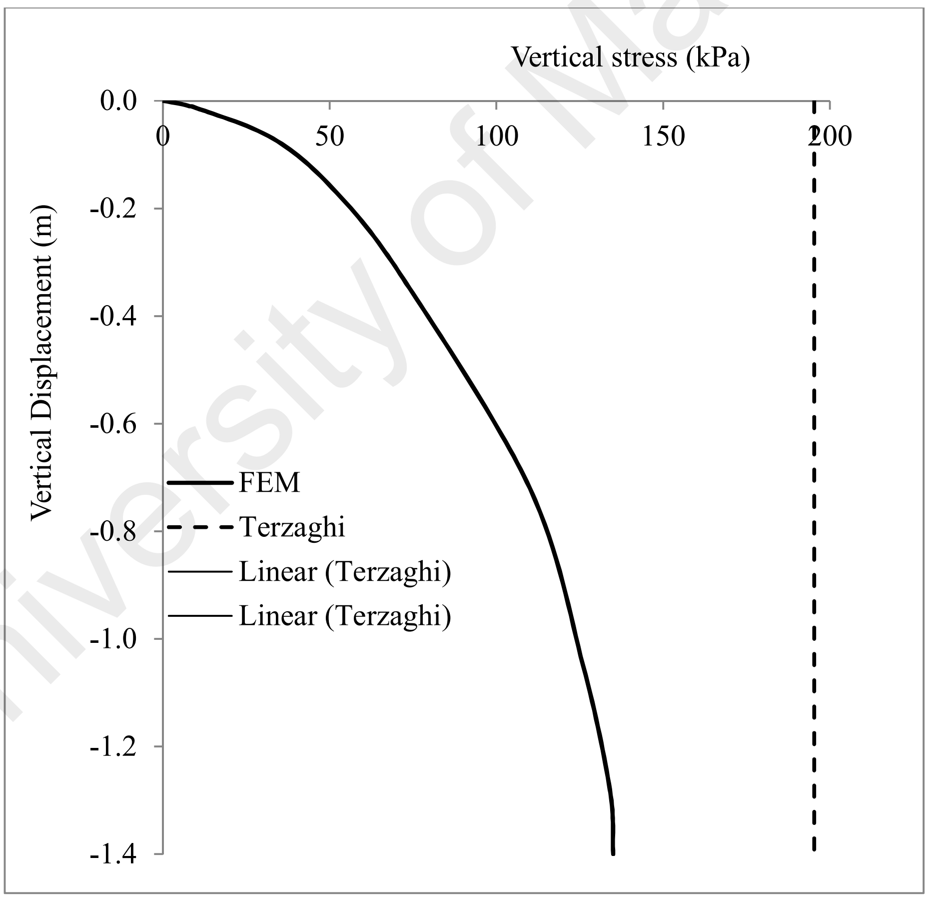


Figure 4.38: Comparison of the Load-displacement curve under centre of structure calculated from Terzaghi's equation and from FEM analysis

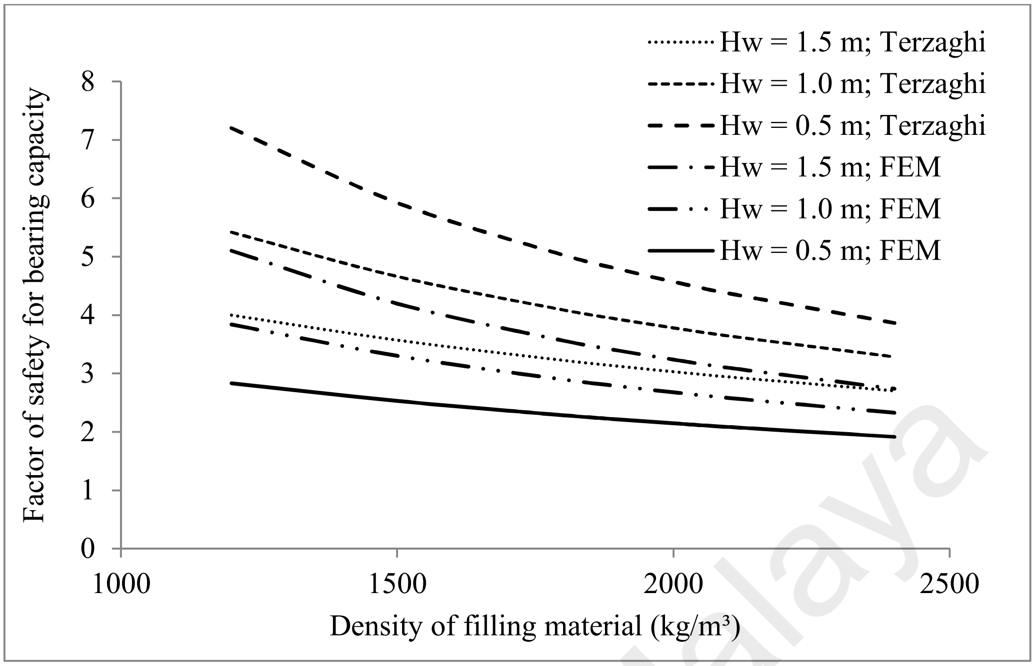


Figure 4.39: Comparison of the $FoS_{bearingcapacity}$ calculated from FEM and from Terzaghi's equation

4.5 Beach Response to the Geotube Breakwaters

The change in beach profile after installation of geotube breakwater is one of the most important aspects that engineers interest in. Installation of coastal structures aims to prevent further erosion along protection area while encourage natural beach nourishment. However, there were cases (Zanuttigh, 2007) where installation of coastal protection structures led to further erosion of coastline, due to wrong decision on structure's layout.

This section discusses beach responses in SHD such as sediment accretion and erosion, after installation of geotube breakwaters. The beach responses to the installed geotubes were investigated and compared by adapting two methods which included the hydrodynamic simulations and on-site monitoring. Besides, the influencing factors such as structure's layout, structure's dimensions and gap between the breakwaters were discussed.

4.5.1 Hydrodynamic Simulation Result

Hydrodynamic models were developed to simulate wave-current reactions after installation of geotube breakwaters on study site. The results can be then used as proxy to predict the beach responses, i.e., sediment accretion and erosion, to geotube breakwaters. The simulations involved analyses of small spatial scale, i.e., cover only area around the geotube breakwaters and beach.

4.5.2 Effect of Geotube Breakwaters to the SHD's Beach Responses

Hydrodynamic models were set up according to the SHD geological conditions and wind climates. Wind generated waves were employed, thus two main wind conditions were set up namely Case M1 (the SW direction with 7.5 ms^{-1}) and Case M2 (the WNW direction with 15.0 ms^{-1}).

Figure 4.40 and Figure 4.41 depict magnitude and direction of waves near SHD shoreline before any structural installation, for Case M1 and Case M2. While the changes of the wave current after the installation of the single line geotube breakwaters were as shown in Figure 4.42 and Figure 4.43.

The unprotected SHD shoreline of Case M1 experienced maximum wave speed of 0.25 ms^{-1} as shown in Figure 4.40. After the geotube breakwaters were installed the speed of the wave at the lee of geotube breakwaters is approximately 0.05 ms^{-1}, reduced up to 80.0% as shown in Figure 4.42. Besides, the Figure 4.42 also shows that the wave speed seaward the geotube structures are approximately 0.01 ms^{-1}. The slower wave speed indicates the greater potential for sediment accretion at the location. The wave speed for Case M2 was 0.29 ms^{-1} before geotubes installed (Figure 4.41) and reduced to the range of 0.07 $ms^{-1}$to 0.18 ms^{-1}leeward the beach (Figure 4.43). While wave speeds at the area seaward the geotubes range from 0.11 ms^{-1} to 0.25 ms^{-1}.

Low wave speeds indicate higher potential of sediments settlement at the location instead of being washed away to other locations. In both Case M1 and Case M2, the installation of geotube breakwaters are able to reduce the wave speed at the protected area, indicating potential accretion of sediment at the lee of breakwaters. Besides, certain locations at the seaward direction of geotubes also showed reduced wave speeds which encourage sediment accretion.

The wave speed was used as the proxy to predict the beach responses after the installation of geotube breakwaters. In Figure 4.44 and Figure 4.45, the potential area of sediment accretion was shown. These areas were predicted to have sediment nourishment because the wave speeds were reduced. Case M1 in Figure 4.44 shows greater sediment nourishment area as compared to Case M2 in Figure 4.45. Comparatively, Case M2 is the more critical conditions and should be employed as the critical design input in the other simulation models.

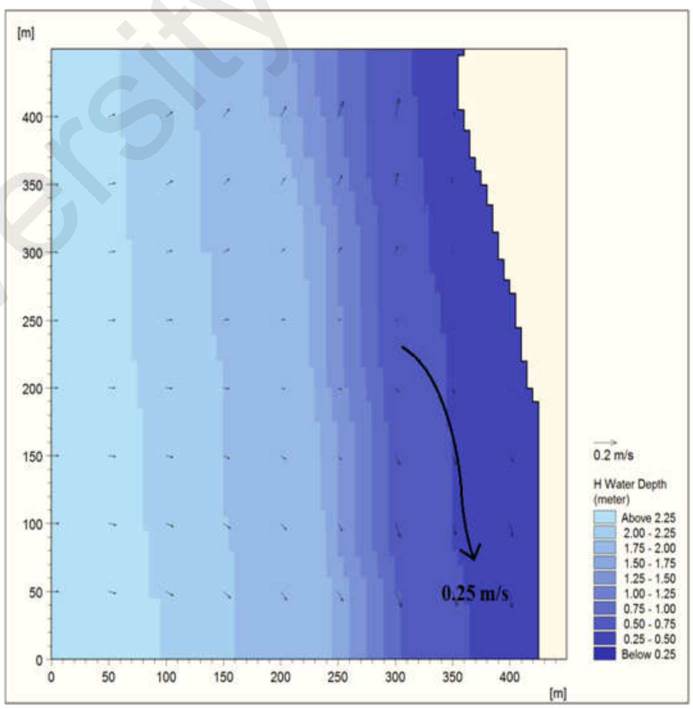


Figure 4.40: Current speed without the protection of geotube breakwater (Case M1)

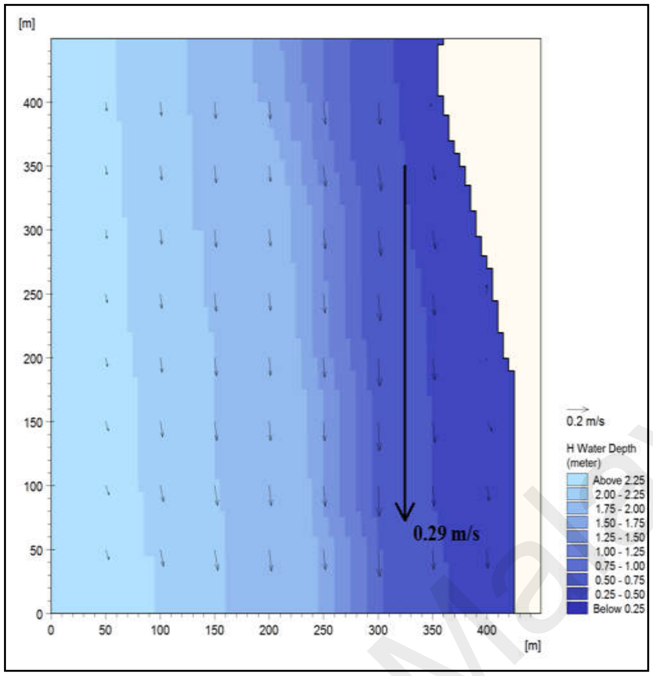


Figure 4.41: Current speed without the protection of geotube breakwater (Case M2)

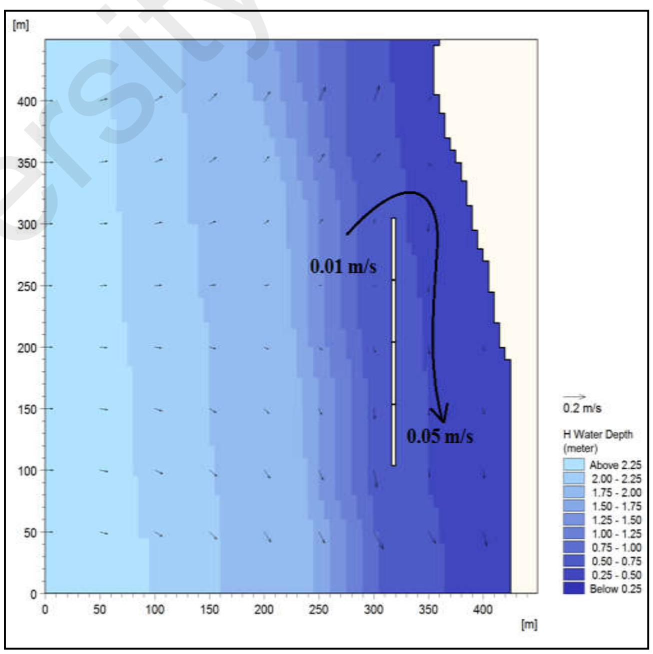


Figure 4.42: Current speed after the protection of four stretches of 50 m geotube breakwaters located 70 m from shore (Case M1)

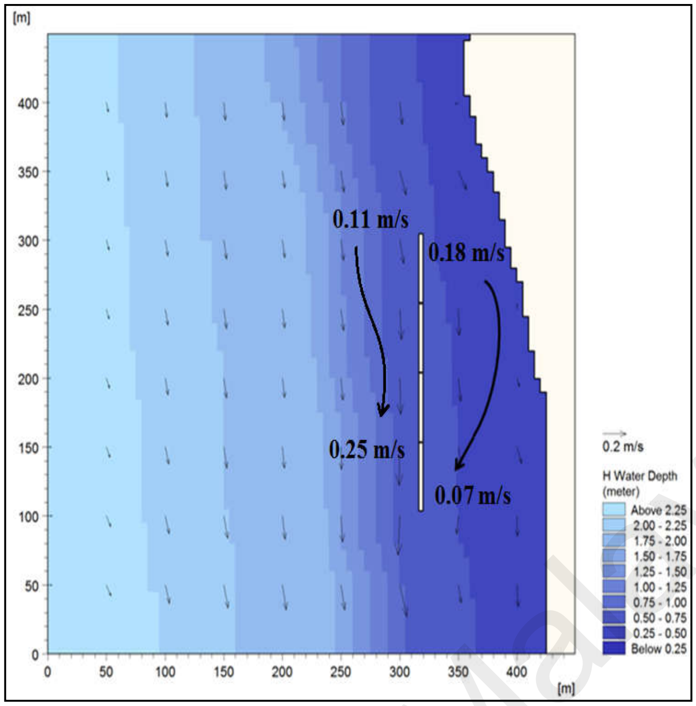


Figure 4.43: Current speed after the protection of four stretches of 50 m geotube breakwaters located 70 m from shore (Case M2)

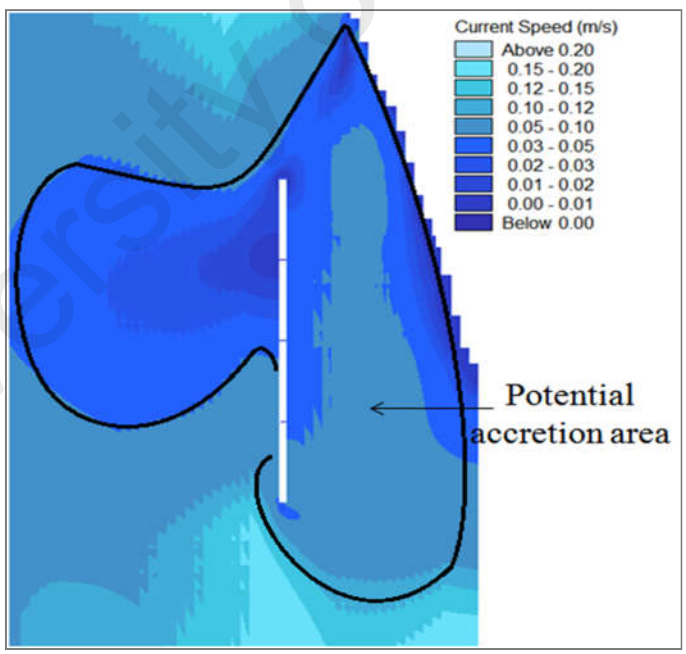


Figure 4.44: Current speed as proxy to estimate beach accretion area after installation of geotube breakwater (Case M1)

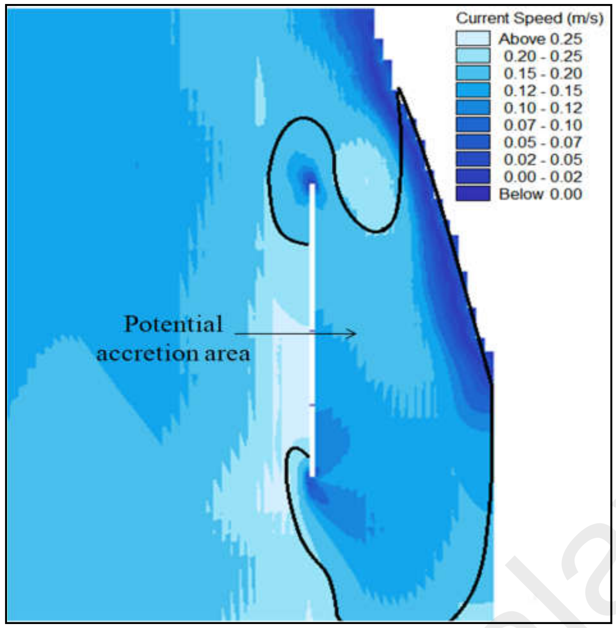


Figure 4.45: Current speed as proxy to estimate beach accretion area after installation of geotube breakwater (Case M2)

4.5.2.1 Influence of the Length of Geotube Breakwater

Influence of geotube breakwater's length was analysed through hydrodynamic simulation models, where length ranged 50 m to 200 m were used as inputs. The wind-generated waves were applied and employed the conditions in Case M2. Figure 4.46 depicts the relationship between length of geotube breakwaters and the wave speed. The highest wave speed without protection of geotubes was 0.29 ms^{-1}.

Result shows increment in the length of geotube breakwaters reduces the incoming wave speeds more effectively. Result also shows distance from shoreline influences wave speeds. The further geotube breakwaters are placed, less effective the structure in reducing wave speeds. The four stretches of geotube breakwaters installed in SHD were of 50 m length each, located at about 100 m distance from shoreline. Results indicated that wave speed at the lee of geotubes was expected to be reduced to approximately 0.09 ms^{-1}, providing a calmer area for the rehabilitation of mangrove trees.

Hydrodynamic simulation model is important to ensure the length of geotubes and the placement location were able to provide the optimum performance in protecting the beach.

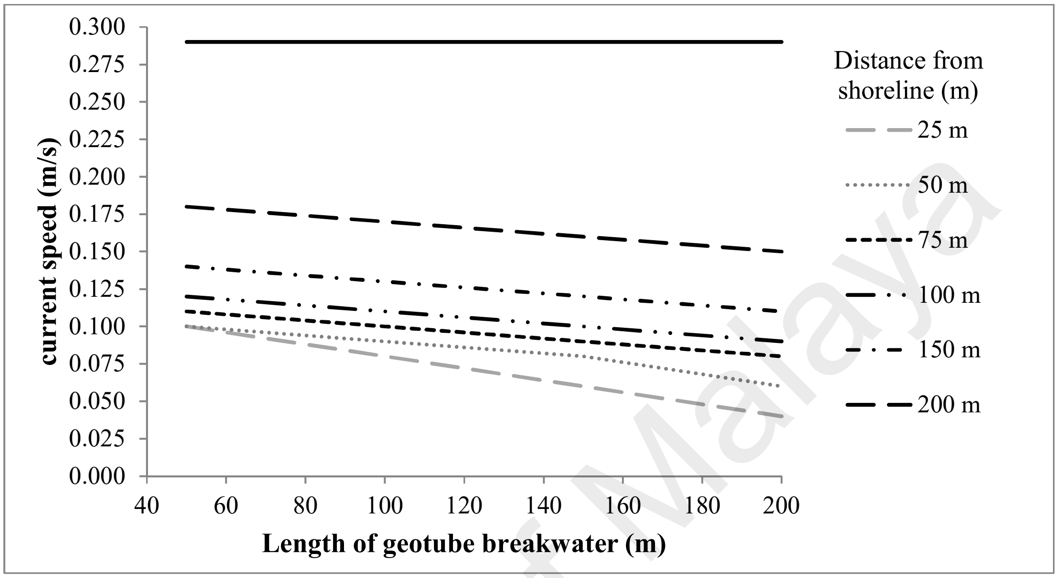


Figure 4.46: Relationship between length of geotube to the wave current speed at protected area

4.5.2.2 Influence of the Location of Geotube Breakwater

As mentioned, distance of geotube breakwaters from shoreline is one of the factors that influences effectiveness of geotubes in shoreline protection. Simulations were carried out with geotubes placed at a distance range of 25 m to 200 m from shoreline. Wind parameters employed was in accordance to parameters in Case M2.

From the Figure 4.47, result shows that placing geotubes of longer length nearer to the beach, provide the better performance in wave speed reduction. Reduction of the incoming wave speed reduce the potential of sediment transported away by the water. Therefore, beach is protected and experience less erosion issue. Besides, reduction in wave speed allows the sediments from other places to settle down at the protected area and nourish the beach naturally.

Result from Figure 4.47 shows that longer geotube breakwaters placed nearer to the beach are able to protect the beach from erosion more effectively. However, project cost and total area of protection might not be optimum. Using the geotube of 200 m in length can reduce the wave speed approximately 60% as compared to the 50 m geotubes when the distance to beach is 25 m. However, when the distance from beach is far, the influence of the geotube's length to the reduction in incoming waves is reduced. For instance, the geotube of 200 m in length reduce the wave speed approximately 16% as compared to the 50 m geotubes when the distance to beach is 200 m.

For our pilot study in SHD, the 200 m long geotube breakwaters were located approximately 100 m from the beach, the wave speed was reduced from 0.29 ms^{-1} to 0.08 ms^{-1}. Hence the sediment accretion at the protection area were noticeable with the presence of the geotube breakwaters.

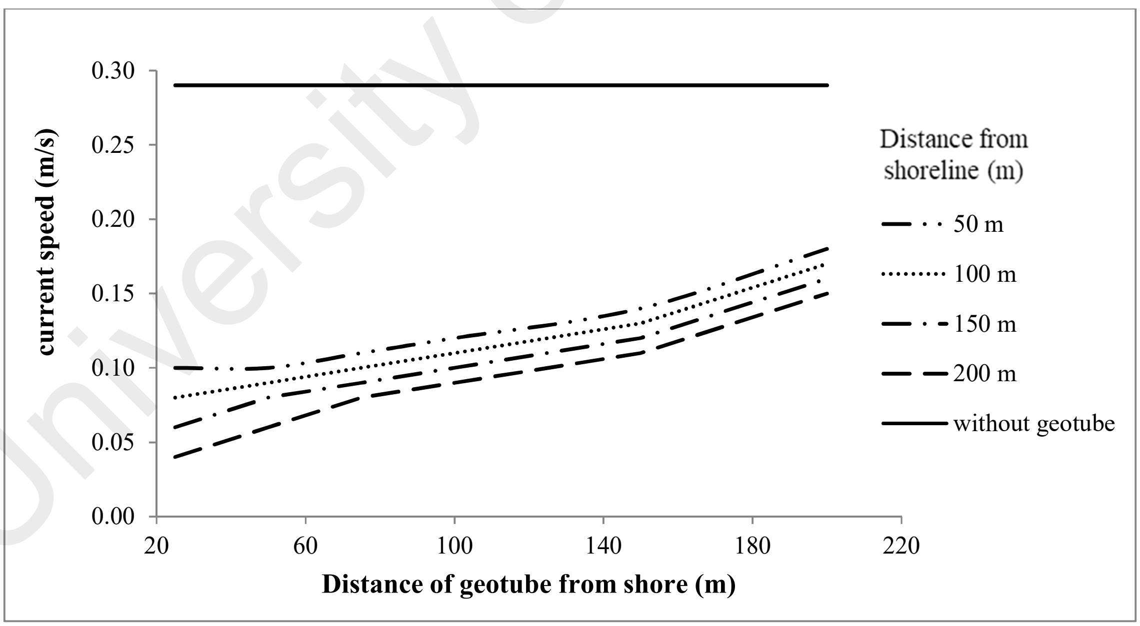


Figure 4.47: Relationship between distance of geotubes from shoreline to the wave current speed at protected area

In a nutshell, the hydrodynamic model can be utilized to assist the geotube breakwater design. The total area of protection, cost of geotubes (geotubes of different lengths have different prices), effectiveness in wave reduction and construction cost can be predicted. In order words, these simple hydrodynamic simulation models can assist the engineers in obtaining an optimum design in term of cost and effectiveness.

4.5.3 On-site Monitoring Result

Sediment nourishment and scouring measurements after first year of geotube installation were obtained from FRIM. Mangrove seedlings were planted at the protected area after approximately a year after the geotubes installation on study site. Continuous observation was carried out until eight years after the installation of geotubes where mangroves were all grown maturely. The beach conditions from 2007 until 2015 were shown in Figure 4.48.

The on-site monitoring measurements were used to verify the sturdiness of hydrodynamic simulation result in previous subtopic. Figure 4.49 (1a and 1b) showed the accretion and erosion of sediment around the geotube breakwaters protection area after one year of installation. This on-site measurement verified the hydrodynamic result (Case M2) as shown in Figure 4.45. Result in Figure 4.45 indicates the wave current speed with the presence of geotube breakwaters. The results were used as proxy to indicate or predict the major sediment accretion area which located at the middle part of protected area and two ends seaward the geotubes.

Results in Figure 4.49 (1a and 1b) also showed main accretion of sediment at the similar areas, with maximum accretion of approximately 0.2 m. The results from hydrodynamic simulations in Figure 4.45 were verified. Predicted sediment accretion area in Figure 4.45 is similar to the on-field measurement, where the sediment accreted at the lee of geotubes, and two ends seaward. Therefore, the simple hydrodynamic simulation model can be used to predict the accretion and erosion area after the installation of geotube breakwaters. During the design of geotube breakwaters, different layouts of geotube breakwaters can be simulated to obtain the optimum geotube breakwaters design.

Figure 4.48: Beach conditions in SHD study site (a) on the first year after geotube installation; (b) on the second year after geotube installation and newly planted mangrove seedlings; (c) on fifth year after geotube installation and growing mangroves; (d) on eighth year after geotube installation and matured mangroves

Figure 4.49 (2a and 2b) shows the accretion and erosion of sediment after eight years of geotube installation with maturely grown mangroves. The protected area was occupied by mangroves. Hence the sediment was greatly accumulated at the lee of geotubes. Accretion of sediment at protection area ranged from 0.7 m to 1.0 m. The impressive beach nourishment was due to the protection of geotubes and the contribution of mangrove trees.

The mangroves further reduced the incoming wave speeds and prevent the area from further erosion yet providing calmer area which encourage sediment settlement. Besides, the roots of mangroves captured the sediments around them and preventing the sediment to be washed out easily. After eight years, the protected area was nourished and experienced minimal direct wave actions. Other areas around the geotubes also showed accretion ranged from 0.3 m to 0.6 m as shown in Figure 4.49 (2a and 2b).

The installation of geotubes was able to protect the beach from further erosion and promoting sediment accretion at certain area around the structures. However, integrated approach of utilizing both geotube breakwaters and mangrove was able to enhance the performance of beach nourishment.

The hydrodynamic simulation was complement by the site monitoring to study more precisely, the morphological changes of site after the installation of geotube breakwater. The result form MIKE 21 simulation were compared with sediment level change results in SHD, of first year (before mangrove planting) and eight years (mangroves matured) after geotubes were installed.

The result from the simulations predicted the similar trend of sediment level changes observed on site. On first year after the installation of geotubes, it was noticed that there was deposition of sediments took place around the edge of the geotube breakwaters and some scouring happened at the inner of geotubes. This on-site observation is tally with numerical simulation which showed reduced wave intensity at the same location. This indicates higher possibility for sediments to settle down at the location. After eight years, the mangrove trees were matured and the protected area showed successful beach nourishment. However, the installation of the geotube breakwaters do not affect the current direction and intensity at the unprotected areas.

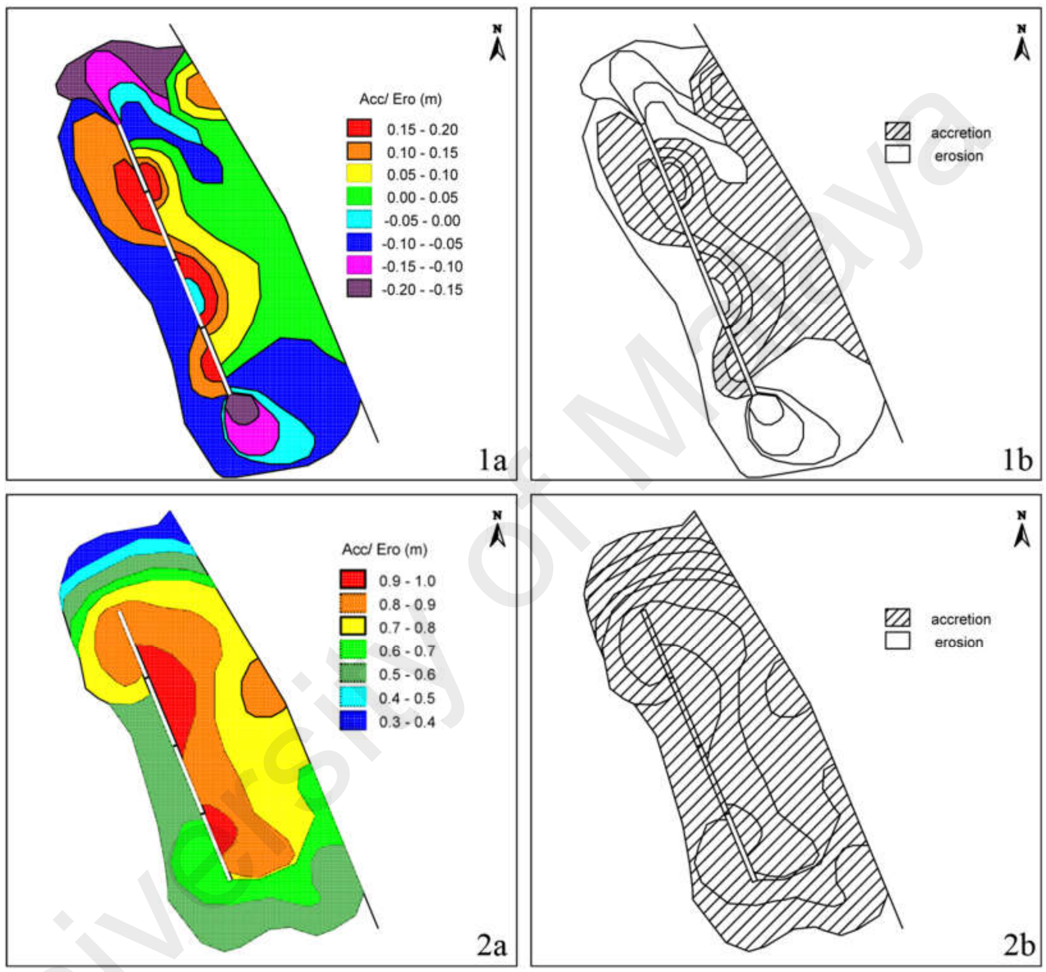


Figure 4.49: Accretion and erosion conditions around the geotubes protection area: (1a & 1b) after one year of geotubes installation without mangroves; (2a & 2b) after 8 years of geotubes installation with matured mangroves

On site monitoring allow the user to observe the real situations which inclusive many factors that can affect the results. For instance, the wave and wind climates, vandalism of geotube breakwaters just to name a few. Meanwhile, the numerical simulation only allows observant to get a resultant output based on the critical conditions modelled. Therefore, by employing both simple numerical simulation and on-site monitoring complement each other and allow a precise study on beach responses to geotube breakwaters. The detailed changes in beach profiles are presented in Figure 4.50 to Figure 4.57. The location of the monitoring pins is shown in Figure 3.18.

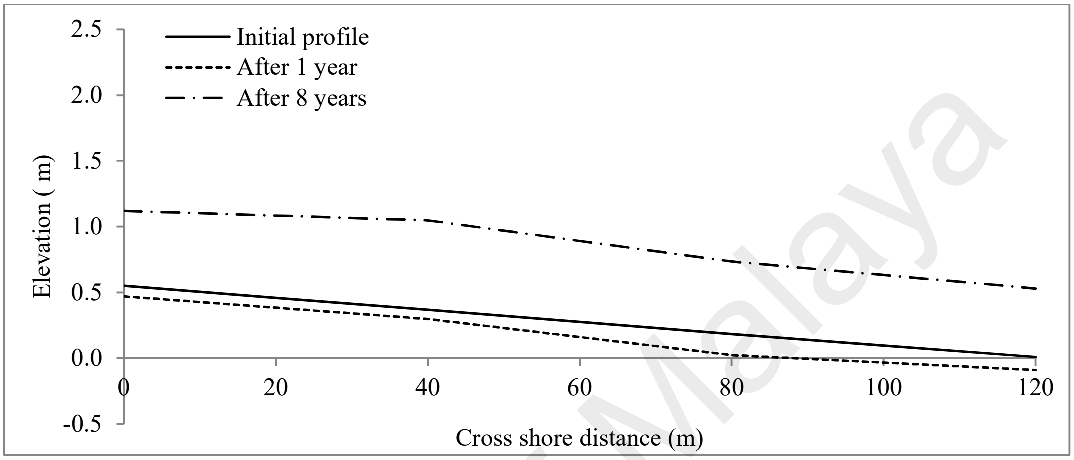


Figure 4.50: Changes in beach profile for line G1

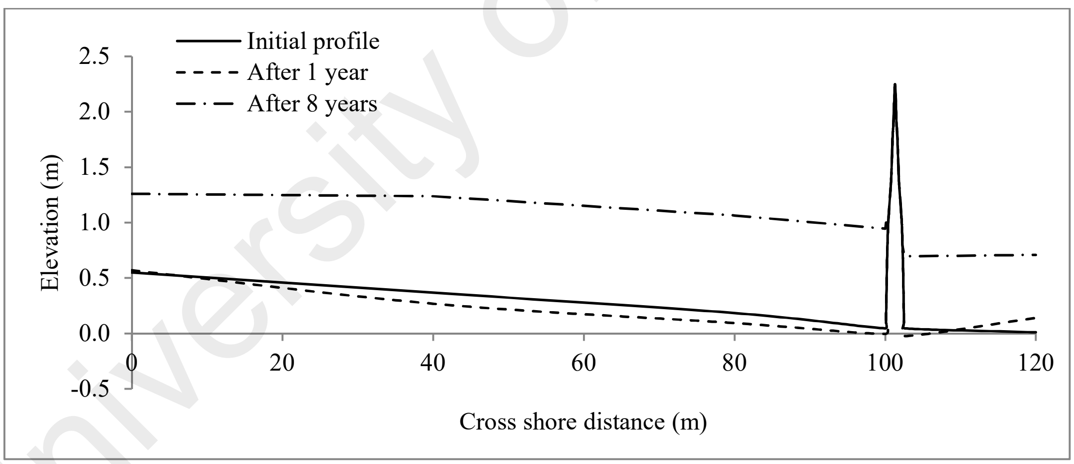


Figure 4.51: Changes in beach profile for line G2

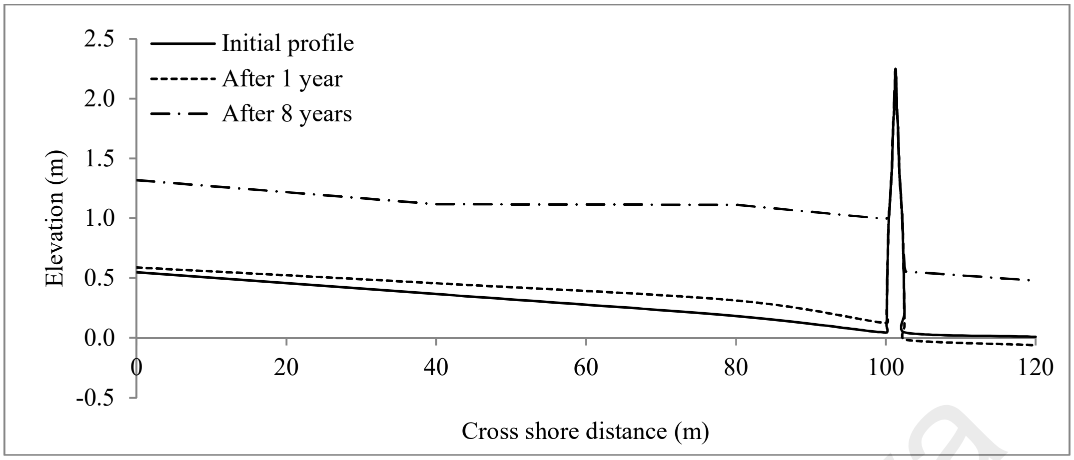


Figure 4.52: Changes in beach profile for line G3

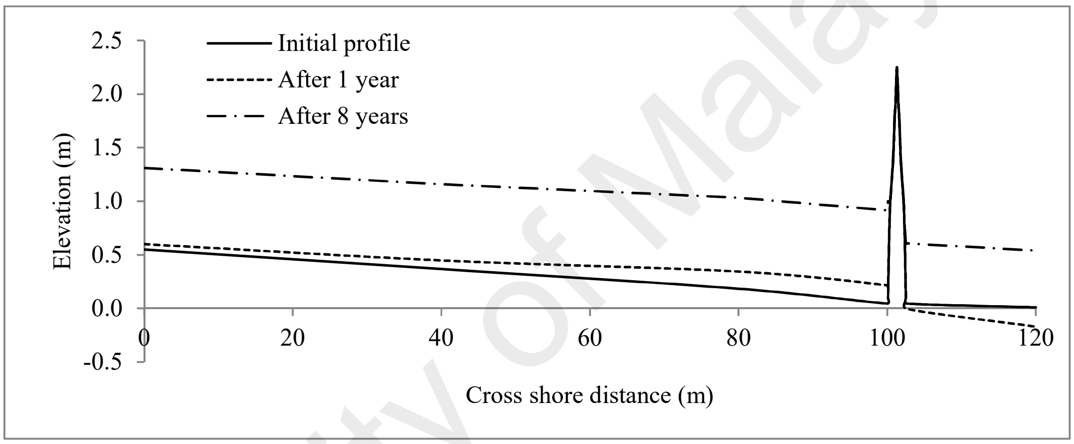


Figure 4.53: Changes in beach profile for line G4

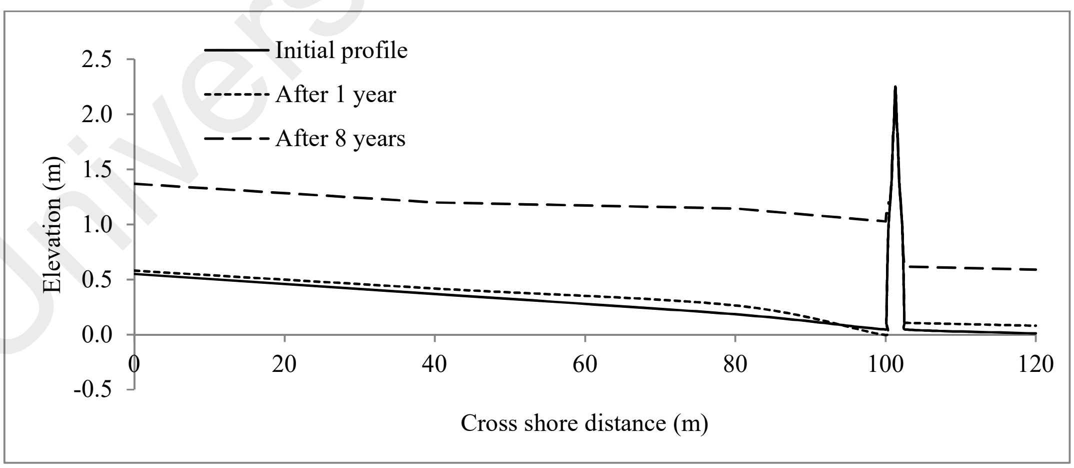


Figure 4.54: Changes in beach profile for line G5

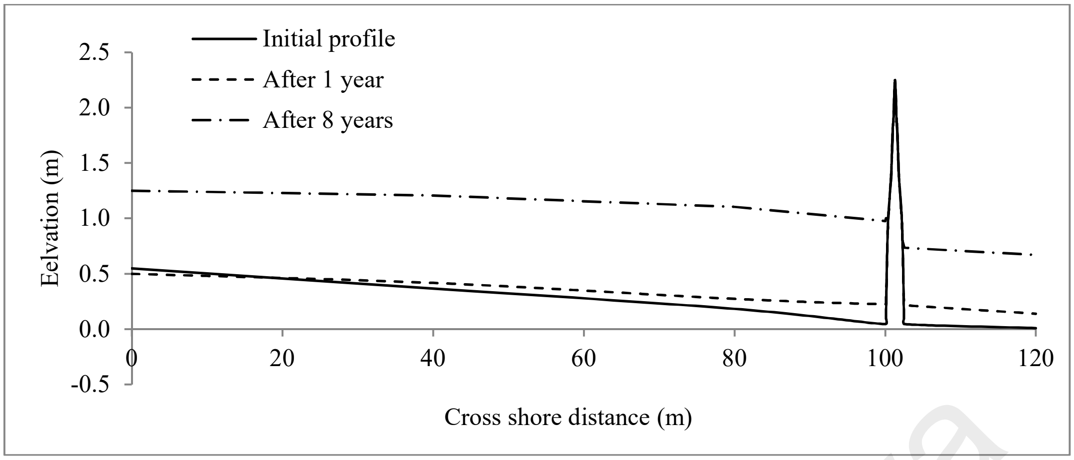


Figure 4.55: Changes in beach profile for line G6

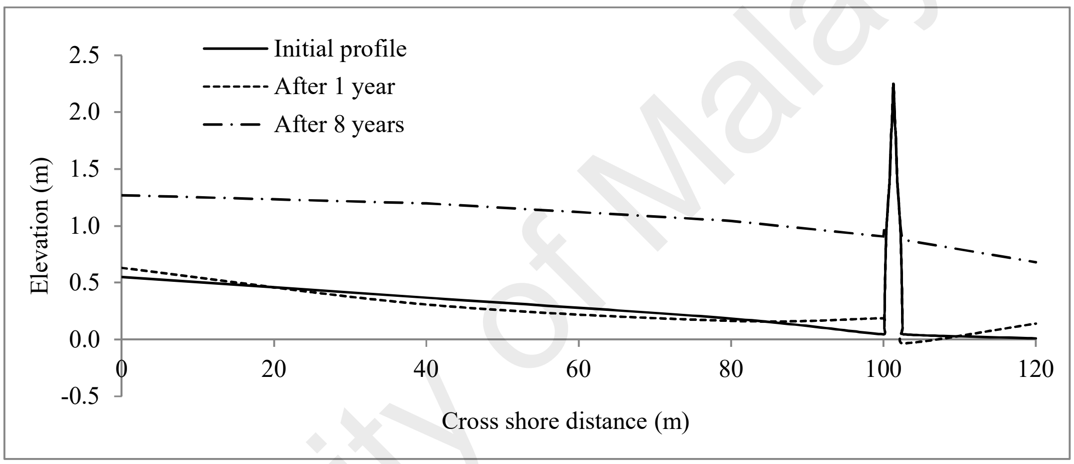


Figure 4.56: Changes in beach profile for line G7

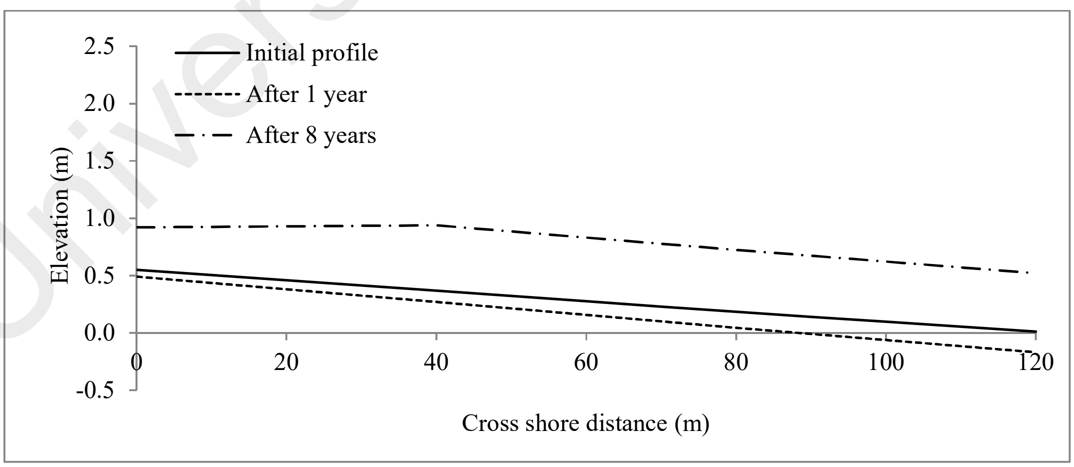


Figure 4.57: Changes in beach profile for line G8

4.6 Comparison between Geotube and Concrete Breakwaters

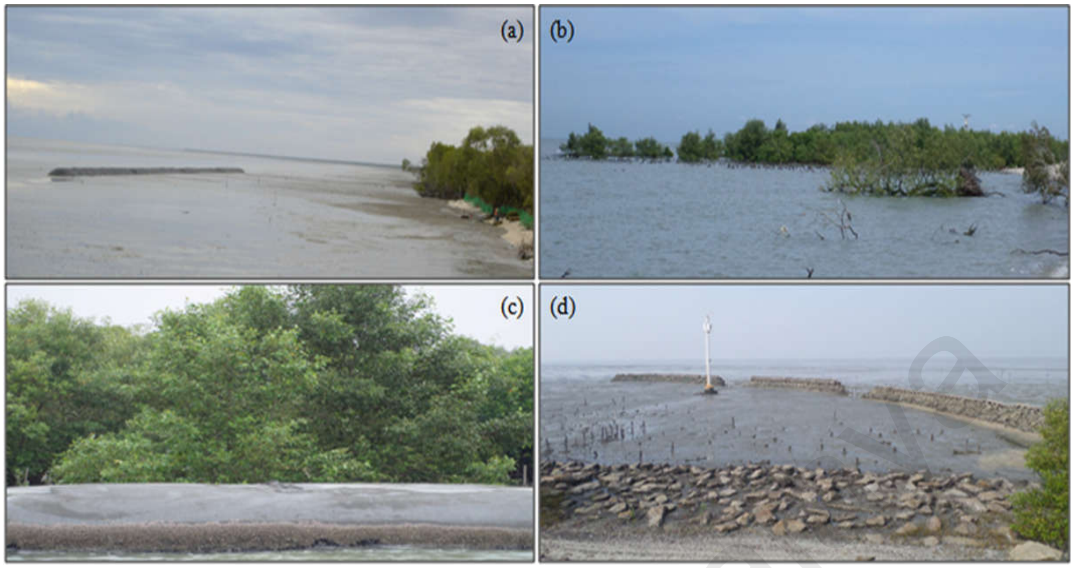


Figure 4.58: Conditions of area around (a) geotube breakwater on 2007, (b & c) geotube breakwater on 2015, and (d) L-block concrete breakwater on 2015

Construction of coastal protection structures on mudflat is more difficult compared to sandy coast, as the muddy foundation is weak and deformable. In SHD's muddy coast, there are two breakwaters, made of different materials, geotube and concrete units. Figure Figure 4.58 (a) to (c) show the condition of around geotube breakwaters while Figure 4.58 (d) shows the condition around the L-block concrete (LBC) breakwaters.

The focus of this study is the geotube breakwater which is categorized under the soft engineering method for coastal protection. While, LBC breakwater is categorized under the conventional hard engineering method, constructed by University of Malaya, located about 1 km away from geotube breakwaters, as shown in Figure 3.3.

Comparison between these breakwaters is carried out in this section. The change in beach profile, layout, cost, advantages and weaknesses are discussed.

Table 4.4: Descriptions of geotube and concrete breakwaters.

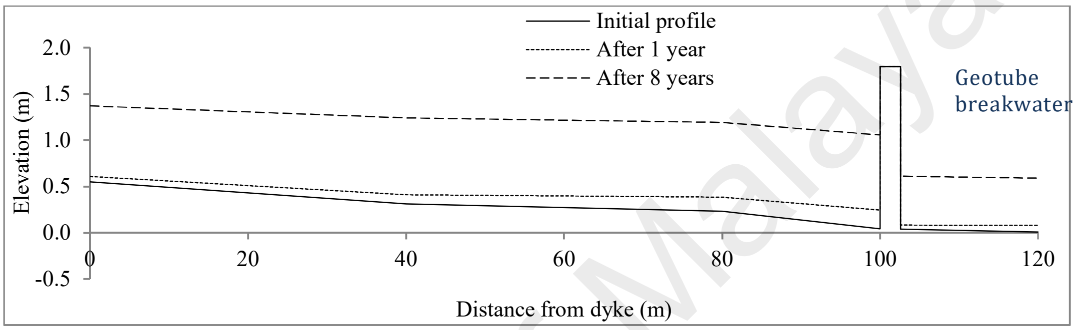
Descriptions	Geotube breakwater	L-block concrete breakwater
Number of stretches	4	3
Length per unit	50 m	30 m
Gap between stretches	0.5 m	2.5 m
Height of structure	1.80 m	1.75 m
Distance from beach	100 m	80 m
Average sediment elevation	0.90 m	0.70 m
Structure settlement	0.09 m	0.17 m
Mangrove species	Avicennia, Mucronata and Rhizophora	Avicennia and Rhizophora
Mangroves condition	Grown well and matured	Died before matured

4.6.1 Beach Profiling

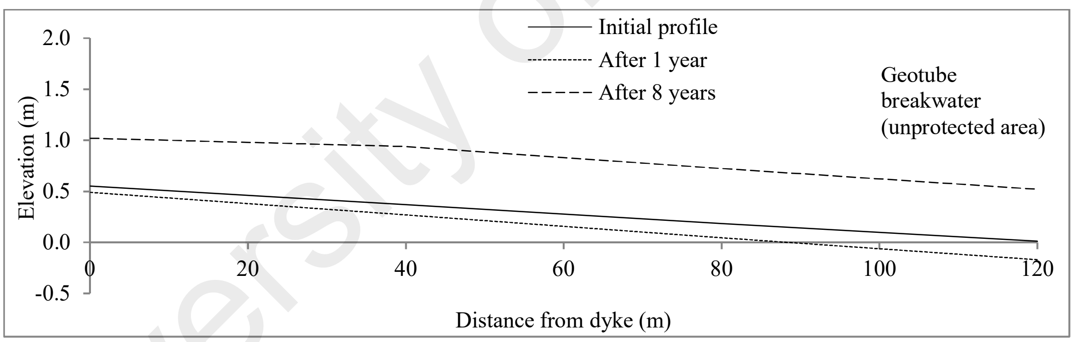
Figure 4.59 shows field observation results of sediment accretion and erosion after installation of both geotube and LBC breakwaters. Figure 4.59 (a) and (b) illustrates change in beach profile around geotube breakwaters' site, where monitoring result from line G5 and G8 represented protected area and unprotected area respectively. One year after the construction of breakwaters, there were sediment accretion along line G5 ranged from 0.04 m to 0.20 m. While, erosion experienced along line G8 ranged between 0.06 m to 0.18 m. Figure 4.59 (c) and (d) show results from Line L5 and L8 which depict change in beach profile around the protected and unprotected area by LBC breakwaters. Line L5 experienced sediment accretion between 0.06 m to 0.20 m, but the unsheltered area along line L8 experienced sediment level change ranged from 0.01 m accretion to 0.07 m erosion.

After 8 years, line G5 shows sediment accretion between 0.80 m to 1.01 m leeward, and 0.58 m seaward. While the unprotected line G8 shows 0.47 m to 0.56 m increment in beach profile. On the other hand, 0.66 m to 0.75 m of sediment increment were observed leeward the geotube structure and 0.58 m seaward the geotube. While the unsheltered line L8 experienced sediment accretion between 0.52 m to 0.57 m.

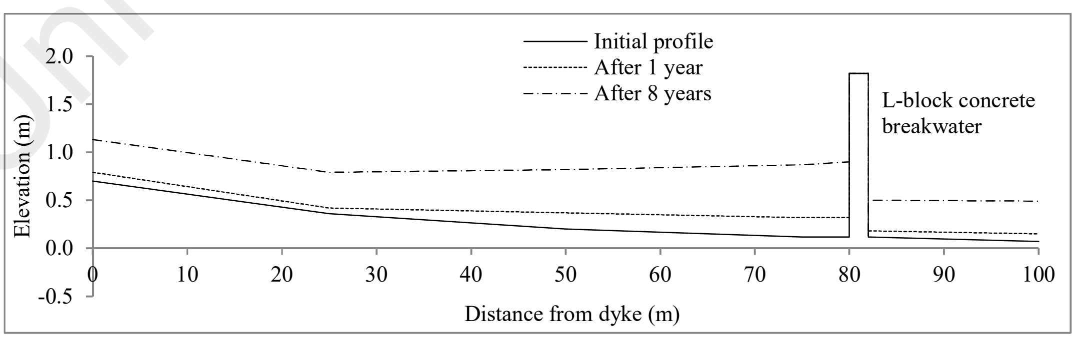
From the results, geotube and LBC breakwaters are able to encourage natural sediment accumulation behind the sheltered area of the structures. However, these structures do not directly promote beach nourishment at the unprotected area. The results also show that after one year of breakwater installation, erosion at unprotected area around the geotube breakwater was more severe as compared to LBC breakwater's site. This is because the geotube structures are located nearer to river mouth where the current speeds are higher.



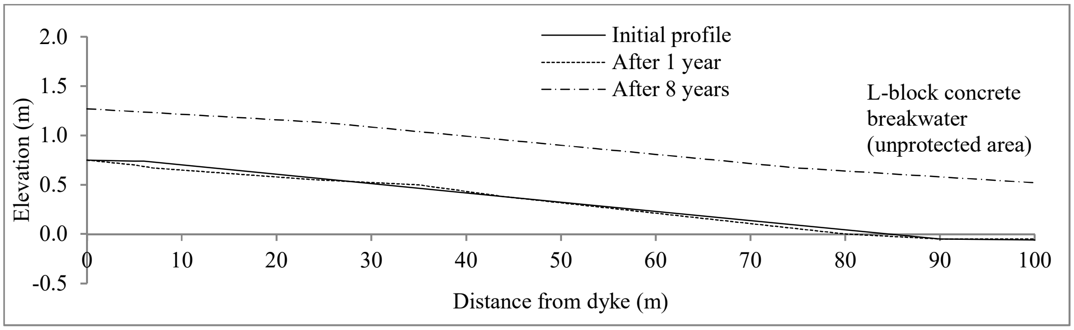
(a)



(b)



(c)



(d)

Figure 4.59: Sediment accretion and erosion at (a) area protected by geotube breakwater, (b) area unprotected by geotube breakwater, (c) area protected by L-block concrete breakwater, (d) area unprotected by L-block concrete breakwater

After one year of breakwater construction, mangroves were planted at both sites, at the structures' sheltered areas. However, the rehabilitation of mangroves was only succeeded at the geotube breakwater's site. Therefore, after 8 years of observations, it can be noticed that, the sediment accretion condition at geotube protected areas showed approximately 25% greater than LBC breakwaters. The integration of both mangroves and muddy coast protection structures is able to prevent sediment erosion, while encourages natural beach nourishment by reducing wave impacts and accreting the sediments.

4.6.2 Mangrove Rehabilitation Conditions

The mangroves around the geotube breakwaters grown mature and become the natural barrier that protect the beach from wave actions. While the mangrove seedlings around the LBC breakwaters were all dead before grown mature. This happened due to two major factors. Firstly, geotube breakwaters located close to each other at a gap of 0.5 m. While, LBC breakwaters were arranged with gap of 2.5 m. Mangroves seedlings which are not matured will be displaced when higher intensity waves attack. The water flows through wider gaps and carry the mangrove seedlings away. Besides, the mangrove in geotube breakwater's site received very good care, observation and maintenance. Dead plant was removed and replaced with healthy mangrove seedling constantly. Therefore, the mangrove conditions around the geotube breakwaters showed better growing conditions.

However, geotube breakwaters experienced minor damage due to vandalism activities as nearby communities are able to reach the geotubes through stepping on the roots of matured mangroves. Some part of the geotubes were cut and losses filling materials over the time (Figure 4.60). While, LCB experienced minor damage on the concrete units due to the sun, wind and water effects.

Figure 4.60: Spillage of filling material caused reduction of geotube's height and repaired with concrete units

4.6.3 Advantages and Limitations of Geotube Breakwaters for Muddy Coast Applications

Geotube breakwaters can be installed very quickly and with simple equipment as compared to LBC breakwaters. No skilled workers are needed for installation works. Besides, installation of geotube on site allows the geotextile units to be send to site easier and quicker as compared to shifting precast concrete units. The lighter overall mass of geotube breakwaters prevent the structures from excessive settlement. From observations, average settlement of LBC breakwaters was about 17 cm but geotube breakwaters experienced approximately 9 cm settlement in average.

Besides, installation of geotube breakwaters can be done by pumping in sediment such as quarry dust to prevent overly exploitation of natural resources. Reuse the by product from quarry not only able to overcome the mishandling issue of quarry dust, but also imbedded the sustainable concept into coastal management. Reduce the concrete production is able to reduce production of carbon dioxide which can cause the greenhouse effect. Geotubes can be removed any time when the functions or objectives achieved. Therefore, minimize the aesthetical impacts to environment.

Cost of the installation of geotube breakwater is cheaper overall, compare to LBC breakwater. Installation of 100 m of geotube breakwaters cost approximately MYR 90,000. The longer the total length of geotube breakwaters, the lower the cost. This is because major project cost is not from geotube and filling materials but machineries, site cleaning works and workers. While, LBC breakwaters installed in SHD costs about MYR 140,000 for a coverage of 70 m. The mentioned project cost was quoted by same contractor. The main reason of the higher cost of LBC breakwater construction is due to the longer construction time required.

Although geotube breakwaters are feasible for the application in muddy coastal, the structure has limitations. For example, geotube breakwaters are more fragile as compared to concrete breakwaters. Geotube can be damaged by sharp items and losses the function as breakwater after spillage of filling materials. Besides, single line geotube breakwaters is insufficient to withstand greater wave height and could lead to failure. Therefore, single line geotube breakwaters are suitable to be applied on coastal area with smaller hydrodynamic forces, as coastal erosion protection structures.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The application of geotube breakwaters as muddy coast protection structures was examined. Based on the laboratory experiment, simulation and on field monitoring results, these conclusions can be drawn.

5.1.1 Internal Stability of Geotube Breakwaters and the Influencing Factors

Major influencing factors for internal stability of geotube breakwaters include submergence conditions, physical properties of geotubes, slurry concentration, types of filling material and design height of geotube.

Non-submergence condition (low tide) is more critical as compared to submerged condition. Geotube experiences approximately 2 to 3 times greater pumping pressure around the inlets, P_o and tension, T in geotubes, under submergence condition. However, submergence conditions do not affect final height of geotube.

Higher sediment content in slurry reduces its workability, and results in greater Po and T in geotubes. From result, increasing sediment content in slurry from 10% to 80% can result in up to doubled P_o and T. The geotube used in this study has a tensile strength of 120 kN/m and upper limit of the sediment concentration in slurry is 50%.

Sediment size of muddy coastal sediment in SHD, 0.041 mm, is too fine to be used as filling materials as the sediment could leak out from geotube and lead to structural failure. Opening size of the geotextile sample used in this study is 0.354 mm. Therefore, the sediment for filling material must have greater size than the openings. River sand and quarry dust are good filling materials due to their larger size of D_{85} = 2.9 mm and D_{85} = 4.9 mm that allow good permeability and can be effectively retain in geotube. We recommend the use of quarry dust as an alternative for sand fills in geotubes, to widen its applications and economic value.

Designed height of geotube breakwater should not be close to its theoretical diameter as this significantly increase P_o and T in geotube. In our study, increasing H from 1.5 m to 2.0 m doubles the T and fourfold the P_o. While, increasing H from 1.5 m to 2.5 m (close to D = 2.74 m), shows 9 times greater in T and 25 times greater P_o.

5.1.2 External Stability of Geotube Breakwaters

Result shows that deformable foundation should not be overlooked. As compared to rigid foundation, T in geotube breakwaters which placed on deformable foundation are 63.8% (H_w = 0.5 m), 54.3% (H_w = 1.0 m) and 12.2% (H_w = 1.5 m) greater. Analysis shows settlements range from 3.5 cm to 5.7 m. While, settlements for rigid foundation are all approximate to zero.

For this study, lower limit for density of filling material is about 1500 kg/m^3 to prevent sliding failure. While, greater stiffness of filling material has greater resistance to structural deformation under waves and causes less T in geotube.

Density and stiffness of geotube breakwaters show minimal influences on its external stabilities. Simulation results show the significant influences of wave heights to the external stability of geotube breakwaters. Single line geotube breakwater installed at SHD are stable under normal wave climate (Hw = 0.5 m - 1.0 m). However, if the wave height reaches 1.5 m, the geotube breakwaters are predicted to fail against sliding and excessive settlement.

5.1.3 Quarry Dust as Alternative Filling Material for Geotube Breakwaters

Quarry-dust-filled-geotubes shows competitive result compared to sand-filled-increase their economic value and reduce exploitation of river sand. In our analysis cases, sand filled geotube shows up to 2 times greater in T, but approximately 50% less overturning displacement. While, quarry dust filled geotube shows 8% to 21% greater lateral displacements, under H_w range 0.5 m to 1.5 m. Filling geotube with quarry dust instead of sand reduce the sediment cost up to 53%.

5.1.4 Beach Responses with the Presence of Geotube Breakwaters

Prediction of the beach responses was done by observing the changes in wave speeds and directions from simulation results. Lower wave speed indicates a higher potential for sediment settlement and lower chance of littoral transportation.

The unprotected site under wind speed of 7.5 ms^{-1} in SW direction experienced maximum wave speed of 0.25 ms^{-1} and was reduced up to 80.0% with protection of geotube breakwater. While, condition where wind speed is 15.0 ms^{-1} in WNW direction, wave speed reduced from 0.29 ms^{-1} to 0.11 ms^{-1} with protection.

Result shows that the simulation model is important to have a quick insight on the beach responses, and compromise between length of geotube and distance placed from shoreline, in order to protect the eroded beach optimally. In SHD, the 200 m long geotube breakwaters are located approximately 100 m from the beach and wave speed reduced from 0.29 ms^{-1} to 0.08 ms^{-1}. Thus, sediment accretion at protection area were significant. Field observation also shows that more sediment accumulated after the mangroves were planted and grown mature.

5.2 Recommendations

This project shows the important considerations during application and design of geotube breakwaters for muddy coast protection, which is worth replicating in other eroded shorelines with similar characteristics. Several recommendations for further study are as follows:

1. The shoreline along SHD experienced severe erosion since several decades ago. Literature mentioned that construction of Temenggung dam was the main reason that reduced the littoral transportation to SHD (Cleary and God, 2000). Besides, the construction of dykes along the SHD site can also become the reason for the continuous muddy beach erosion due to the reflective effect of the dykes. Therefore, it is recommended to carry out detailed morphological and hydrodynamic study in a large spatial scale to determine the actual reasons of the degradation of the muddy beach. Detailed hydrodynamic study can help the authorities to manage coastal issue more effectively.

2. For muddy coast, mangroves are one of the natural barriers to protect the shorelines. However, different mangroves require different growing environment. Certain species required thicker sediment to grow, but some species cannot survive when there is excessive sedimentation. Therefore, it is recommended to carry out collaboration study with mangrove experts to find out the suitable mangroves species for rehabilitation purposes.

3. The soft and deformable mudflat is the most challenging part during the installation of breakwaters. Further study in ground improvement is recommended. As muddy beach has soft, wet and deformable foundation, construction works could be very challenging. Studies on ground improvement methods, feasibility, cost and time effectiveness, are important.

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