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FLOOD FLOW BEHAVIOUR IN RIPARIAN VEGETATED SANDY BED CHANNEL

Mohamad Hidayat Jamal¹, Mazlin Jumain¹, Zulhilmi Ismail¹, Zulkiflee Ibrahim¹

ABSTRACT

Riparian vegetation or trees on floodplain are important riverine features and essential in hydraulic engineering including flood management and river bank erosion protection. Mismanagement of vegetated floodplain has been pointed out as a contributing factor to the severity of flood damages including financial losses and even fatalities. Clearing of riparian vegetation might affect the hydrodynamics or hydraulics and morphology of rivers. Researches on the impact of riparian vegetation on compound channel hydraulics have been carried out for many years. In order to prevent scouring, sediment problem and flood mitigation, the effect of riparian vegetation on river flow must be understood. Experimental investigations on the flood flow behaviour in riparian vegetated sandy bed compound straight channel had been undertaken. The effects of two-line emergent vegetation along the edge of floodplain were studied by using flume in the Hydraulics Laboratory, School of Civil Engineering, Faculty of Engineering, Universiti Teknologi Malaysia (UTM). The findings on relative flow depth-discharge relationship, Manning's n , stream wise velocity, vorticity patterns and bed morphology changes during shallow and deep flood inundations in the riparian vegetated sandy bed channel are presented in this paper. The findings prevailed that riparian vegetation retarded the compound channel flow capacity which increased the relative flow depth up to 8.5%. The drag force induced by riparian vegetation combined with variation of bed morphology patterns had increased the Manning's n by 16.7%. The riparian vegetation also altered velocity distribution and limited the movement of secondary currents between main channel and floodplain. The dramatically changes of bed morphology had a strong influence on the flow structures in the channel.

Keywords: Physical model, riparian vegetation, flood plain, bed morphology

Introduction

Flood is one of natural disasters in Malaysia and many part of the world. The flood events in end of 2014 experienced in Kelantan and Pahang states really opened everybody's eyes on the disaster due to nature and human activities. The clearing of trees or vegetation along river banks has been pointed out as a contributing factor to the severity of flood damages. Disaster statistical data from 1990 to 2014 indicated that floods were the highest hazard contributor of 98.7% to Average Annual Loss (AAL) to Malaysia. The floods in Malaysia contributed to 62.5% hazard frequency, 24.1% mortality and 60.0% of economic issues (UNISDR, 2014).

Riparian vegetation is pointed out to play important roles for river banks stabilisation, erosion prevention, habitat creation and riparian buffer zones development for flood protection (Palmer et al., 2007). Uncontrolled human activities on floodplain areas for agricultural, commercial or housing purposes have great impact on the issues of flood damages and also soil erosion problems. Clearing up trees along river banks has been pointed out as a contributing factor to the changes in rivers system. The hydraulic of flood flow in open channel is characterised by a complex flow structure due to the interaction between the main channel and floodplain flows, lateral momentum transfer and secondary flows (Wang et al., 2011). River flow, sediment transport and morphological processes are among the most complex and least understood processes or phenomena in nature (Sirdari et al., 2014). Floodplain and main channel have their own characteristics such as roughness which depends on their type of surface. The vegetation's on floodplain which might grow naturally or planted for erosion prevention or for landscaping purposes. The behaviour of vegetated rivers is complex and its

influence is not yet fully known.

Vegetation significantly influences the increase of flow resistance, alters velocity distribution, affects the conveyance of rivers, raises water levels, increase in the shear stress near the vegetation and also influences transportation rate of sediment in rivers (Aberle and Jarvela, 2013; Kang et al., 2018). Vegetation characteristics such as height, density, distribution, stiffness, and type of vegetation have remarkable impact on the flow velocity as well as flow resistance. The reduced velocity will then lowered the forces necessary to cause scour and erosion processes that influenced the changes in channel morphology (Curran and Hession, 2013; Vargas-Luna et al. 2015). The presence of vegetation may also increase sediment deposition and modify the risk of flooding by the combined effects of increased roughness and decrease of flowing area of the river main channel (Romdhane et al., 2017; Romdhane et al., 2018). The flow structure and the position of scour bed depend on the flow conditions.

Various kinds of bed profiles are directly influenced to the variability of velocity distributions in the main channel. Ishigaki and Muto (2001) reported the secondary streamwise vorticity and bed profiles affected each other, where the secondary currents tend to form sandbars due to erosion and deposition processes. Therefore, it is important to further understand the riparian vegetation influence on the river hydraulics.

Experimental investigations on the flood flow behaviour in riparian vegetated sandy bed channel were undertaken in the Hydraulic Laboratory, Universiti Teknologi Malaysia (HLUTM). The study was carried out in an asymmetrical straight channel which covered shallow and deep flood inundation cases.

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Experimental Methodology

The experiments were carried out in a 12 m long flume with 0.5 m wide and 0.1 m deep rectangular main channel and a 0.5 m wide floodplain at HLUTM. The flume bed was set at a gradient of 1/1000. Uniform graded sand with mean diameter, d_{50} of 0.8 mm was filled in the main channel to form as bed material. A similar uniform sediment size was also used by many researches including Knight and Brown (2001), Atabay et al. (2005), Tang and Knight (2006) and Bousmar et al. (2006). The main reason of using uniform graded sand in the research was to minimise the “sheltering” and “hiding” effects. As bed forms propagate to the downstream, sediment moves from the ridge of the bed forms to the trough. In the trough, the sediment is sheltered and overlaid by the advancing grains from the upstream bed forms (Ismail, 2007). Steel rods with 5 mm diameter were used as artificial resistance elements in this study. The two-lined steel rods were placed in staggered arrays along riparian zone to simulate as closed spacing emergent floodplain vegetation of $2d$ where d represented the rod diameter. The resistance elements were placed at the edge along the floodplain at a distance between 4 m to 8 m from the channel inlet. Figure 1 and Figure 2 illustrate the layout of experimental set-up and the cross-sectional configuration of the channel. The experiment was left to run continuously for more than 24 hours and the water surface level evaluation was checked regularly until representative bed forms have been developed. The development of bed forms varied dramatically with time. A re-circulating flume system was used in the study and the discharge was measured using a Portaflow PF330 flow meter. Flow depth and bed morphology levels were measured

using a digital point gauge or water surface profiler with an accuracy of ± 0.1 .

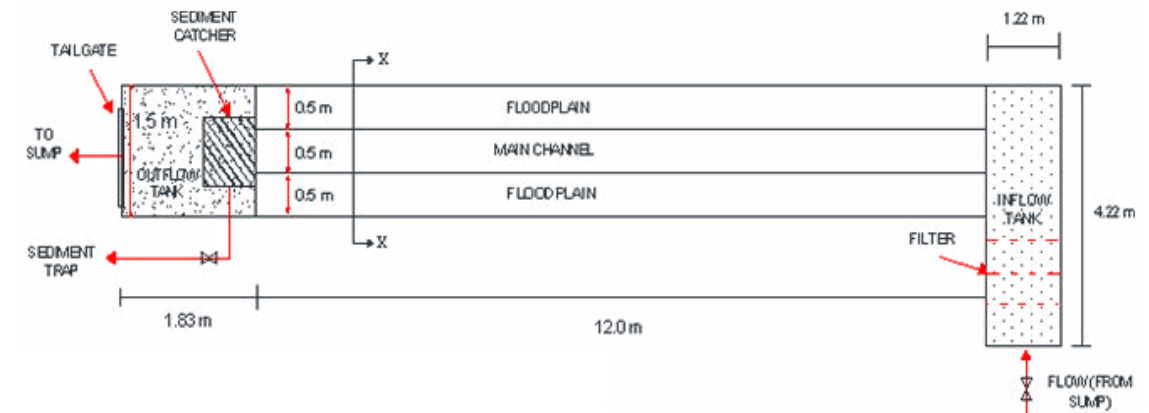


Figure 1: Plan view of the channel

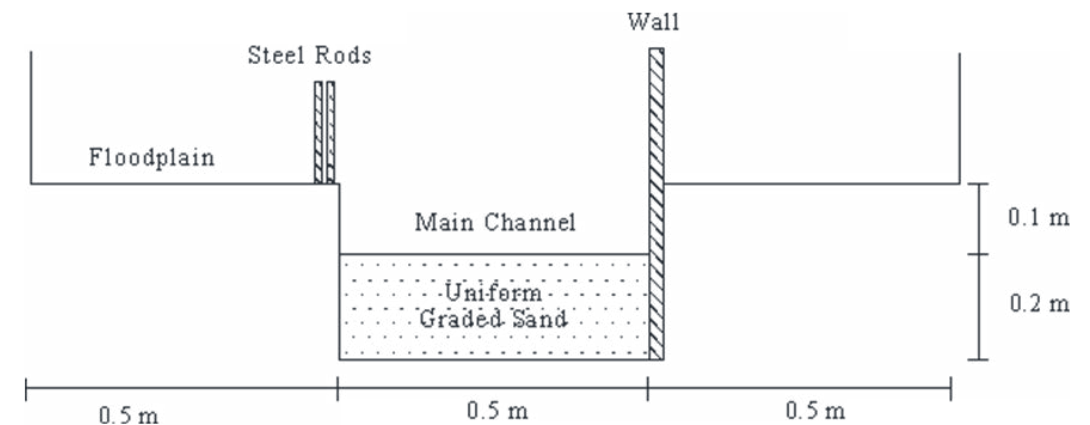


Figure 2: Cross-sectional view of the channel

mm attached on a mobile carrier. The 3-dimensional velocities were measured using a Nortek Vectrino + Acoustic Doppler Velocimeter (ADV) with a sampling rate of 100 Hz. The transverse interval distance for velocity measurement was 2 cm and varies vertically.

Results and Discussion

The experimental investigations were conducted under uniform flow condition in order to apply uniform flow theory in the analysis. The results were discussed for 2 different relative depths: DR = 0.30 and DR = 0.50. DR was the relative depth = $(H - d)/H$. For both relative depths, the calculated Reynolds number (Re) exceeds 2,000 and the Froude number (Fr) less than unity. Therefore, the regimes of flows were classified as sub-critical with turbulent condition.

Relative flow depth-discharge relationship

The experimental relative flow depth-discharge curves are illustrated as in Figure 3. It is apparent that the curves gradually increased with discharge. As mentioned by Fathi-Moghadam and Drikvandi (2012), the existence of vegetation along the river banks cause the decrease of water flow capacity and increase the water level. The mean increments of DR due to riparian vegetation is 8.5% compared to non-vegetated case. It was noticed that the main channel morphology also led to the increase of the flow resistance and subsequently reduce the discharge and raise the DR in the compound channel as reported by Khademishamami et al. (2014).

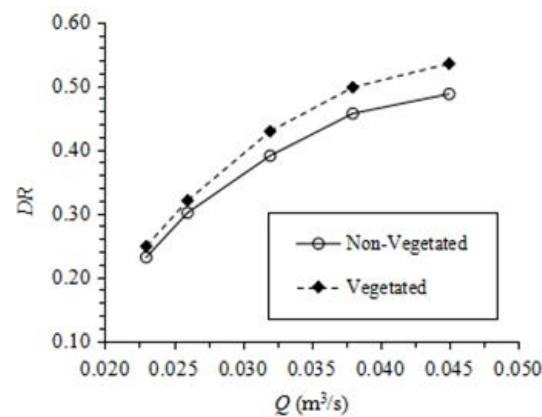


Figure 3: Relative flow depth-discharge curves

Manning's n

The flow resistance is among the important concern in the hydraulics of vegetated channels. The standard Manning's equation as shown in Equation (1) was applied to determine the Manning's n of compound straight channel.

$$n = \frac{R^{2/3} S_a^{1/2}}{U} \quad (1)$$

where R is hydraulic radius, S_a is channel bed slope and U is mean stream wise velocity. Figure 4 displays the Manning's n for non-vegetated and vegetated floodplain in compound channels. The Manning's n increased as the DR increased until the discharge was 0.38 m³/s and then started to decreased.

This is due to combined effect of momentum transfer and differential roughness between the floodplain and the main channel, as mentioned by Ibrahim et al. (2016). The Manning's n for non-vegetated was 0.018 to 0.024. Meanwhile, the Manning's n for

vegetated floodplain was 0.019 to 0.028 where 16.7% higher than in non-vegetated floodplain case. Zhang et al. (2010) and Ali et al. (2012) were also found a similar trend on the roughness effects of vegetation in the compound channel.

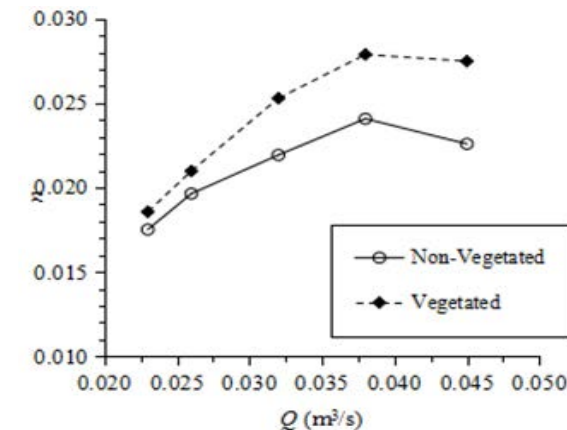
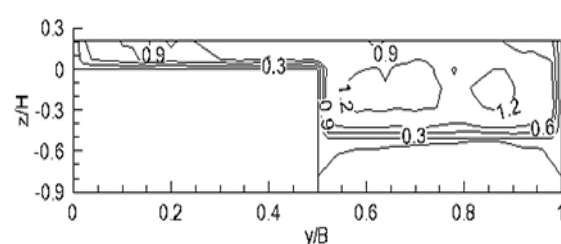


Figure 4: Manning's n-discharge relationships

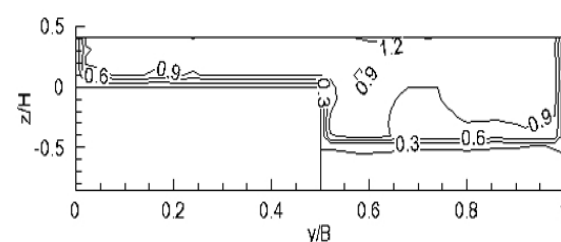
Streamwise velocity distribution

The cross-sectional distribution of stream wise velocity in the straight compound channels were plotted in order to understand the interaction between main channel and floodplain flows. The measured stream wise velocity U was normalised by the mean sectional velocity U_s to provide a general view of flow patterns in the channel. The distributions U_d were plotted at cross-section x/L of 0.50. x is longitudinal distance and L is total length of the channel. Meanwhile, the symbols of y , z , B and H represent the transverse distance, vertical distance, total channel width and total flood depth in the straight channel, respectively. Figures 5 and 6 display the normalised U/U_s in non-vegetated and vegetated floodplain straight channels at DR of 0.30 and 0.50. The phenomenon of "velocity dip" in which a maximum velocity occurs below

free surface takes place in the main channel. The velocity dip phenomenon can be clearly seen in both non-vegetated and vegetated cases. The maximum U/U_s for the shallow DR of 0.30 was 1.2; meanwhile for deeper DR of 0.50 the maximum U/U_s was smaller at 0.9. The variability of velocity observed in the main channel was due to the presence of the bed features which resulted from erosion and deposition processes. It was similar as mentioned by Khademishamami et al. (2014). For vegetated case, the maximum velocity cell U/U_s of 1.2 were also observed in main channel but its location were different where the bulging moves further toward the centre of the main channel as illustrated in Figure 6. The floodplain vegetation will reduce the floodplain velocity and increase the main channel velocity. The observation on velocity distribution shows that the maximum velocity occurs in the central of main channel region. This indicates that vegetation has generated higher resistance to the water flow and retards the penetration of main channel flow into floodplain as informed by Amel Sadeghi et al. (2010). The "free shear" is the situation where velocity stratification does not occur in the vertical direction. The free shear can be seen in vegetated zone where less momentum transfer between main channel and floodplain flows takes place. The results agreed to previous findings by Ibrahim et al. (2016). Similar as highlighted by Kang et al. (2018) that during the inundation of floodplain, the vegetation interacts with the flow, slowing it in the vegetated areas and reducing the overall channel conveyance. It can be said that maximum velocity region moves towards the bottom. The influence of the shrubs is felt in whole main channel.

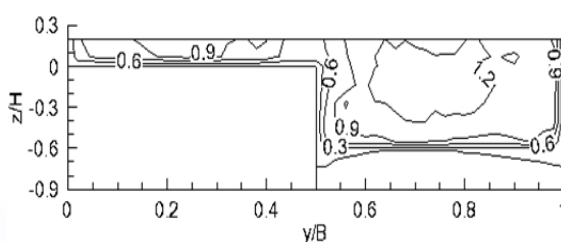


(a)

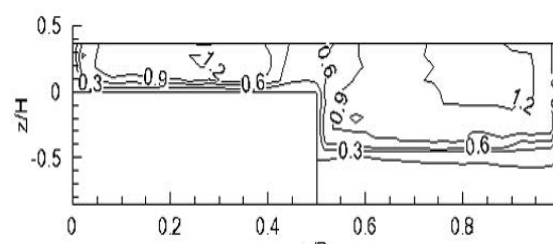


(b)

Figure 5: Distributions of U/U_s in non-vegetated case at DR of (a) 0.30 and (b) 0.50



(a)

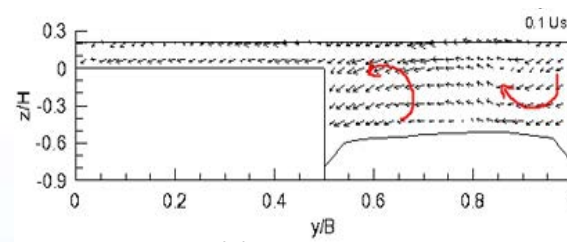


(b)

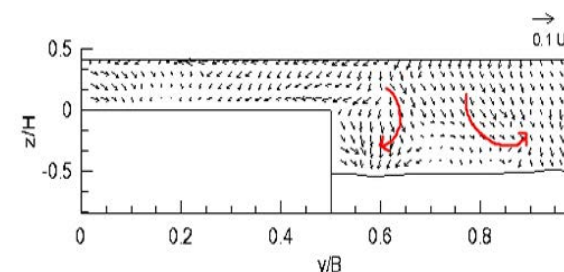
Figure 6: Distributions of U/U_s in vegetated case at DR of (a) 0.30 and (b) 0.50

Vorticity patterns

Flow circulations are important in altering the stream-wise velocity pattern, bed shear stress, turbulence and sediment transport. The secondary flow which is the resultant of V and W velocity components is normalised by the mean sectional stream-wise velocity, U_s . Figures 7 and 8 present the patterns of normalised stream wise vorticity in non-vegetated and vegetated compound straight channel at DR of 0.30 and 0.50. As the DR increased, the strength of secondary currents on the floodplain increased. The circulation pattern on the floodplain section was invisible due to shallow floodplain flow depth at DR of 0.30. The observed vortices magnitude in all cases can attain to 10% of mean sectional stream wise velocity. It can be said that the strength and direction of vortices in main channel were largely dependent upon the channel geometry as highlighted by Le Bouteiller and Venditti (2014).

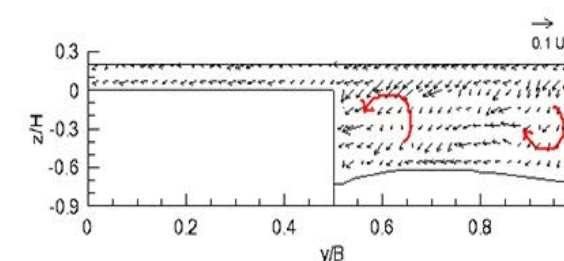


(a)

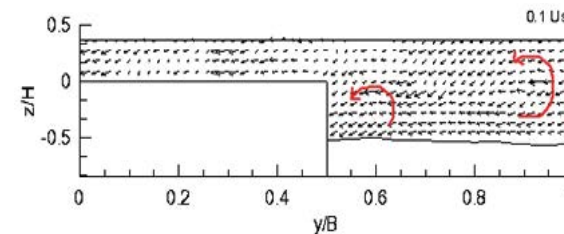


(b)

Figure 7: Vorticity patterns in non-vegetated case at DR of (a) 0.30 and (b) 0.50



(a)



(b)

Figure 8: Vorticity patterns in riparian vegetated case at DR of (a) 0.30 and (b) 0.50

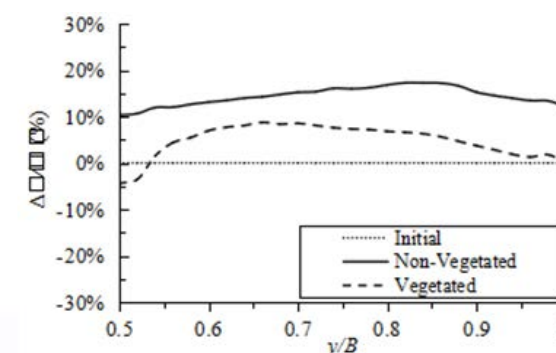
It can be noticed that riparian vegetation imposed flow resistance in the compound straight channel. The secondary currents were limited to pass through the vegetation in the riparian zone. A stroger anti-clockwise vorticity cell was observed near to the main channel-floodplain interface. It shows that vegetation had significant influence on turbulent flow structure and mass exchange of the compound channel flows. The variability of main channel vorticity

patterns in mobile bed cases was due to the bed morphology (Yang et al., 2012). Hence, the bed morphology features play important roles on flow structures in this study.

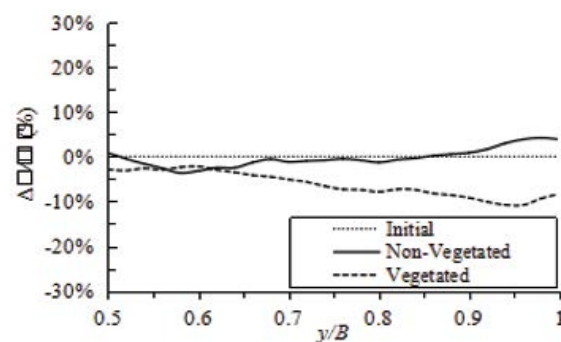
Bed morphology

In order to quantify the bed level changes in the channel, the analysis on normalised or relative main channel bed level changes ($\Delta Z/Z$) in non-vegetated and vegetated floodplain were carried out. Figure 9 shows the normalised bed level changes at location x/L of 0.50 for DR of 0.30 and 0.50. ΔZ is the bed level changes, Z is initial or original bed level.

It can be seen that the sand was deposited for both cases at DR of 0.30. Small sand scour can be observed at y/B 0.50 to 0.55 in Figure 9(a) which was due to the flow retardation effect on bed profiles by the riparian vegetation in the channel. The maximum deposited sand ($\Delta Z/Z$) in non-vegetated and vegetated cases were 17.3% and 8.7%. This implies that the influence of flow resistance which leads to the velocity reduction of the channel flow.



(a)



(b)

Figure 9: Bed morphology profiles for non-vegetated and vegetated cases at DR of (a) 0.30 and (b) 0.50

At a higher *DR* of 0.50 in Figure 9(b), it was observed that an erosion of sand on the bed occurred for both non-vegetated and vegetated cases. The maximum sand bed erosion ($\Delta Z/Z$) was 10.8% that can be observed in vegetated case. This indicates the role of secondary currents near the main channel-floodplain interface wall which caused a lateral migration of sediment toward centre of the main channel. This means the vegetation has forced the flow to concentrate in the main channel, similar as reported by Le Bouteiller and Venditti (2014).

The flood flow behaviour in riparian vegetation compound straight channel has been experimentally investigated. The conclusion can be drawn from the findings are: (i) the presence of riparian vegetation cause the decrease of water flow capacity and increase the water level, (ii) drag force induced by riparian vegetation combined with variation of bed morphology patterns have increase the flow resistance, (iii) riparian vegetation affects the velocity structure where shear layer and free shear took place near the main channel-floodplain interface and also vegetation zone, (iv) the

present of vegetation also limit the movement of secondary currents between main channel and floodplain, (v) the distribution of flow velocity and secondary currents of the channel influence to the changes and formation of main channel bed profiles.

Acknowledgements

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WAVE ENERGY HARVESTING SYSTEM DEVELOPMENT

Icahri Chatta¹, Safari Mat Desa¹, Noor Azme Omar¹, Suriani Othman¹,
Mohd Baharum Muhammad Din¹,

ABSTRACT

Digital live style and electrical transportation had been latest trend worldwide, contributing electric tariff and energy crisis factor for the next incoming years. Nowadays, renewable energy had become popular replacing hydropower and fossil fuel based power plants. Wave Energy Harvesting (WEH) from sea has potential to be an alternative natural resources to the electricity production for tomorrow renewable energy. Since 2015, prototype named NAHRIM's Coastal Defence and Energy Generator (N-CODE) has been invented and tested at laboratory as well as sites. In this study, few designs were tested to monitor the WEH system for electric generation and to identify design ability in 2-4 Ampere per second in Direct Current (DC), capable to charge power bank and lighted up LED. Series of study found that N-CODE could possible dissipated energy from water wave producing Direct Current (DC) and to be converted into alternatives current (AC).

Keywords: Electricity Tariff, Renewable Energy, Wave Energy, Kinetic Energy, Wave Energy Harvesting System.

Introduction

Many of conventional design for water energy harvesting was created from natural resources such as rivers and from manmade reservoir. Energy consumption will be continue increasing over the years ahead, in world competed economic growth, given potential interruption of natural resources. According to United Nation, on April 2010, about 1.5 billion have no access to electricity (Alla C.S. et al., 2014). At about 1.8 billion in the region use solid fuels for cooking and still not using electrical household for cooking. WHO estimates that more than 1.45 million people die prematurely each year from indoor air pollution caused by burning solid fuels, traditionally without using electric cooking burner (Askari M.S. et al., 2015; Alain C. et al., 2002; Adrien C. et al., 2015).

Nowadays, energy harvesting from ocean had been taken place worldwide in commercial scale. Ocean wave is an energy contain tremendous energy potential. Studies had found ocean wave and current could be dissipated and converted into electric generation. Ocean tide is a natural resources and it create natural gravitational effect in a day. Tides occur twice in 24 hours at different location caused by the gravitational effect of the moon (B.Sacco and A.K Tomilin; B Drew et al., 2009; Dina S et al., 2013; Eugen Rusu, 2014). Many design of wave energy harvesting (WEH) system had been made to generate electricity from ocean wave. There are variety of technologies of WEH in differences technical concept and design to capture the energy from ocean waves (David Morin ; Eduard M. et al., 2016; Ernest Weibel; Geoff Klempner and Isidor Kerszenbaum). Currently, there are commercial WEH system had been used in the world. The WEH system had been deployed currently at United Kingdom and France since the

11th Century. At the average tidal range of 8 meters, the WEH system generating power at 240MW deployed at the mouth of the La Rance estuary in Brittany France in 1966. The tidal change in the range of 10.8 meter had generating WEH system about 16MW and distributing 30 million kWh of electricity per year at Fundy Bay, Canada (Guilherme M.P., 2014; Jenny Hayward and Peter Osman, 2011; J.Khan and G. Bhuyan; Junhua Wu et al., 2013). As Malaysia surrounded by 4,675 kilometers coastline with islands, it gave advantage that Malaysia coastline have it natural resources for the continuous research and development of WEH system.

Objectives

The objective of this research is to study suitable design of wave energy harvesting system from natural resources of Malaysian east coast wave for generating electricity to the island community as renewable energy resources. As to fulfil the objectives, research scope had covered some WEH designs conceptual will be tested to fulfil the transformation of conceptual all WEH design into final design named as N-CODE. Site installation, data collection of energy harvested and others design factor influenced will be as critical observation during the study.

Methodology

In this study, all WEH consist of: (a) floatable device system – to dissipate kinetics energy continuously from simulated water wave; (b) electromagnetic system - for converting the kinetics energy to electricity current; and (c) electrical system – to manage electricity output such as an ammeter, LED light, AC-DC converter, light censored and others, according to WEH design. Material for floatable device system will be using hollow PVC pipes,

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attached with joints and PVC adhesive in order to build the concept of buoy. Setting of simulated water wave at Port and Harbour wave basin (P&H) are: (a) wave height at 0.65 meter; (b) wave frequency at 0.1 Hertz; and with water level of 0.60 meter. Below are the methodologies of each design of WEH inside the laboratory:

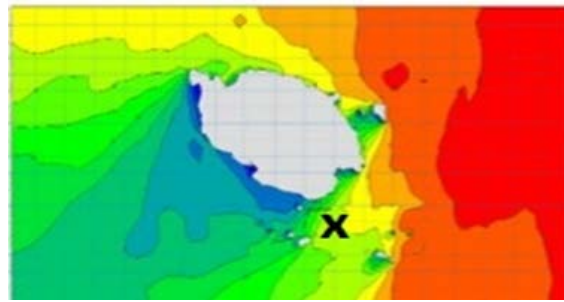


Figure 1: Wave Spectrum Model at Pulau Tinggi, Johor in 2016 for Site Test at Location 'X'.

Vertical Movement (VM)

The design material consist of: bicycles dynamo, bicycle gears and chain, mild steel support frame, floatable structures, led light, simple wiring for electricity measurement and electrical instrument devices.

Horizontal Movement (HM)

The design material consist of: 1 inches Hollow PVC Support Frame, nylon rope, magnet bars and copper coils. The HM will be submerged in the PHWBF and wave will be simulated for some testing scenario.

Multi Directional Movement (MDM)

The design material consist of: two inches Hollow PVC Support Frame, two kilogram of artificial reef of WABCORE, capacitors, nylon rope and magnet bars.

Site Testing of NAHRIM's Coastal Ocean Defence and Energy Generator (N-CODE) .In this study, Pulau Tinggi had been selected for the testing location. The exact location for the N-CODE deployment was at 300 meters off from the Pulau Tinggi, located at 2.278436 East, 104.12899 North.

Results and Discussion

The type of waves in this study is regular waves of Johnswap Equation with constant forces of wave frequency. Irregular waves had been tested once and resulting total mechanical operational failure for all WEH design. Below are findings of each WEH designs:

Vertical Movement (VM)

The testing scenario shows that the VM does not dissipated energy at all. Some modification of floatable unit design had been go through but does not performed better results. In this study, the size, type and numbers of gears and chain does not influence the VM.

Horizontal Movement (HM)

Observation found that the HM movement is not frequent in order to generate electricity. Some improvement had been suggested to add on additional elements in order to increase the HM movement.

Multi Directional Movement (MDM)

The MDM as Figure 2, had resulting stable design in energy dissipation as result as in Table 1 and had meet the study objectives. The MDM had performed stable in power generating for about 30 minutes. The

Table 1: The performance of MDM

Wave Height Simulated (Meter)	Period Of Charging (minutes)			
	1	10	20	30
	Power Bank Charged (Volt)			
0.15	1.19	3.18	4.69	5.22
0.17	1.1	3.25	4.03	4.4
0.23	0.8	1.5	1.91	2.27
0.25	0.72	1.22	1.57	1.83
0.27	0.5	1.02	1.49	1.74

maximum power generated at average of 4 Volts at constant wave height.



Figure 2: Final Design of MDM Testing at Laboratory and at site

N-CODE Site Testing

Two day of data collection had been recorded in October 2016 due to the monsoon season limiting days of site testing as in Table 2. From the Table 2, the N-CODE had succeed generating power in a small amount of quantity, at minimum of 2.6Volt at the beginning of high tide (or at the end of low tide). These shows that N-CODE could be possible generating direct current.

Table 2: The second day performance of n-code power generating vs. estimated wave height and tide time

Time	Estimated Wave Height (Meter)	N-CODE Power Generated (Volt)
1500	0.2	5.58
1515	0.3	6.7
1530	0.2	5.7
1545	0.2	5
1600	0.3	5.9
1615	0.3	5.3
1630	0.3	7.3
1700	0.3	5

Conclusion and Recommendations

In this study, it shows that the wave energy harvesting (WEH) system design could be further study for efficiency improvement. All five prototypes design had discover that water waves could be dissipated electricity. The main factors of the WEH design which are material selection and size. From the study that, size should not be oversized and undersized. Others main factors are floatable structure design, suitable magnet strength, the magnet movement frequency and anchoring technics. In this study also, N-CODE prototype had achieved the objectives of the study, that is power is possible could be generating from Malaysia East Coast include all area of its coastline and Malaysia's Island under the coast facing South China Seas. Finally, further studies could be done for developing N-CODE Installation Calculator (NIC) to calculate N-CODE at optimum cost for a specific deployment or calculating numbers of N-CODE at certain size of N-CODE. By this, N-CODE could possibly be commercialized and preliminary for lifetime operational and calculated maintenance cost within the commercialization deployment.

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INTEGRATED TIDAL MARINE TURBINE FOR POWERGENERATION WITH COASTAL EROSION BREAKWATER

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ABSTRACT

Malaysia experienced a predictable nature of tides all of the year. The most potential areas in Terengganu and Sarawak waters with average annual power that can be generated is between 2.8kW/m to 8.6kW/m. This condition gives excellent opportunity to explore power generation using tidal energy converters by utilised the stand-alone marine facilities such as breakwater with the tidal stream energy. The tidal energy converter is a device that converts the energy in a flow of fluid into mechanical energy by passing the stream through a system of fixed and moving fan like blades. The power output is dependent on its design characteristics, which covers the turbine specification and the met-ocean environmental condition. Hence, this paper focused on the conceptual design of the integrated marine turbine mounted on wave breakwater called WABCORE. The proposed marine turbine installed in the breakwater and the generated energy was estimated based on the performance analysis through Finite Element Analysis (FEA) and ANSYS Fluent Computational Fluid Dynamics (Fluent CFD) simulations. It was found that a maximum power output of 30 Watts could be generated by horizontal-axis axial-flow marine turbine with excellent venturi-effect of piping design that provided significant contribution on power generation.

Keywords: Integrated Breakwater; Marine Turbine; Tidal Energy, Marine Renewable Energy

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Introduction

Energy sources is crucial in developing country as it acts as a catalyst for the growth and development of a country. The demand for energy is expected to increase due to the growth of economic, population, income and improvement in life style. Currently, energy in Malaysia are mainly depends on conversion of fossil fuel sources; coal, natural gas and fuel-oil (Samsudin et al., 2016). The combustion of fossil fuel and coal for energy releases massive emission of Greenhouse gases (GHGs) (Safaai et al., 2011; Steen, 2000) that can endanger the environment and cause climate change/global warming. The reliance on fossil fuels/non-renewable energy resources should be reduced, because with current production of oil and natural gas reserves rate, the resources are expected to be depleted over time and cannot be sustained for future power generation (Haiges et al., 2017; Riti & Shu, 2016; Islam et al., 2009; Yaakob et al., 2006). As a developing country, Malaysia is pursuing towards sustainable development to achieve its developed country status. In parallel with the rapid economic development, concern on environment issues and security of its energy fuels supply, this has encouraged Malaysia's energy sector development to seek for alternative on greener path in harvesting energy by using renewable energy sources (Tan et al., 2013; Shamsuddin, 2012). The use of renewable energy promotes clean and environmentally friendly energy resources, ensure security for national energy supply with diversification of energy resources. Besides, shifting towards using renewable resources to garner energy for power generation allow the country to alleviate the reliance on fossil fuels. Malaysia is blessed with renewable resources to meet up its energy need such as solar, hydropower, wind, biomass, biogas and ocean energy (Chong

& Lam, 2013; Islam et al., 2009). Among of the resources available, hydropower and solar photovoltaic are the most popular and commercially installed, with total potential energy that can be produced are 22,000MW and 6,500MW respectively (Solangi et al., 2011). Based on National Renewable Energy Policy and Action Plan 2010, Malaysia aim to increase share of renewable energy mix in total power generation up to 24 percent by 2050, approximately 21.37GW cumulative capacity (Kardooni et al., 2016; Hashim & Ho, 2011). In accordance to that target, Malaysia surely has long way to go, thus the Malaysian Government has initiated several funding for universities and institutes to work on research and development for other renewable energy project to be explored in Malaysia, and this include research on ocean energy (Samrat et al., 2014). With total coastline of 4,675 kilometres (Samrat et al., 2014), Malaysia is endowed with valuable national asset where the coastline has provided abundance of opportunities to generate energy apart from being vital support for the development growth in littoral states along the coastline of Malacca Straits and South China Sea. Ocean energy may not yet commercially viable in Malaysia due to its technological challenges, nevertheless, initiatives using ocean energy as one of alternative energy has been explored and successfully pursued by other developed country namely Japan, United Kingdom, European Union, United States, Australia, etc. (Magagna & Uihlein, 2015; Yaakob et al., 2006). Ocean energy is one of renewable energy generated from ocean wave where it can be converted into vital sources of low-carbon electrical generation in the form of wave energy, tidal current energy, marine current energy, tidal range energy, ocean thermal energy conversion (OTEC) and salinity gradient (EMEC, 2017; Lim & Koh, 2010; Yaakob et al., 2006).

Utilizing ocean as an alternative energy sources are more dependable compare to solar and wind energy as it only present 20-30 percent of time (Pelc & Fujita, 2002), whereas the ocean energy can be harvested day and night.

The potential of generating electricity using ocean waves is very promising and economically viable with great impact on environment and society. Several studies on the prospect of harnessing ocean energy in Malaysia (Nasir & Maulud, 2016; Samrat et al., 2014; Azman et al., 2011; Lim & Koh, 2010) shows that tidal current energy has great potential and very reliable in compare to other ocean energy to be exploited for sustainable energy. Tidal current energy is predictable as it is natural phenomenon as result of interaction between gravitational forces of moon, sun and earth. Hydrokinetic energy present in flowing ocean wave generated by tides can be directly converted into electricity using a marine turbine (Güney & Kaygusuz, 2010). Marine turbine is a device that converts the energy in a stream of fluid into mechanical energy by passing the stream through a system of fixed and moving fan like blades. The fundamental concept is based on the aerodynamic force of lift to produce a net positive torque on a shaft, which is rotating due to the blades. This mechanical energy is able to produce electricity in the generator. It applied the same concept as wind turbine but with different physical fluid in which marine turbine explore the potential of electricity using tidal current of the seawater. The marine turbine can take advantages of marine facilities such as breakwater for cost sharing on construction, installation, maintenance and operation by integrating it into one composite structure.

Problem Background

The power output from marine turbine is dependently on its design which covered the type of turbine used, size and the site selection (Nilsson, 2009). In general, the turbine can be classified either axial-flow or cross-flow. Axial-flow turbines sweep through a circular area of water by rotating about an axis that is parallel to the flow direction. Cross-flow turbine sweep through a rectangular area by rotating about an axis that is perpendicular to the flow, with water flowing across each blade twice. Both can be added with a duct features to increase the mass flow rate over the rotor, allowing a given power output to be achieved from a smaller diameter turbine (Shives and Crawford, 2010). However, cross-flow turbine can produce more power for their size and more productive for shallow water compare to axial-flow (Roberts et al., 2016). The commercialized marine turbine can be classified based on it features as illustrated in Figure 1.

Even the tidal current is typically much slower than wind, a much smaller turbine size is sufficient to generate almost equivalent electricity output as those produced by wind turbine as seawater have higher fluid density (Bahaj and Myers, 2003). Tidal current energy has a great potential as a source of renewable energy since Malaysia experienced predictable nature of tides all the year. Based on previous study on potential wave energy in Malaysia, the analysis shows that Terengganu waters has largest potential area about 52,351,809.9m² and Sarawak waters about 6,875,901.81 m² (Nasir & Maulud, 2016).

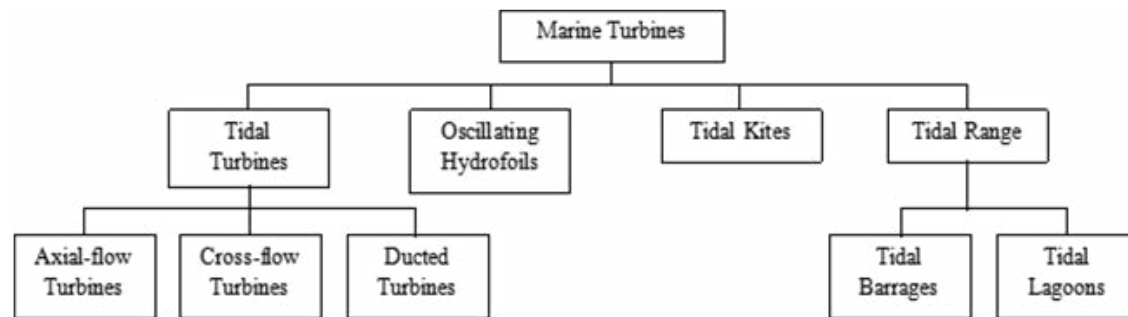


Figure 1: Current technology on marine turbine (Robert et al., 2016).

Facing the wide and vast South China Sea, Terengganu coastline is highly exposed to coastal erosion due to effect of northeast monsoon season, which occurs annually from November to February. Coastal erosion in this area has resulted loss of valuable agriculture land, affect aquaculture activities, and cause damage to beach amenities and infrastructures. Interaction between ocean waves, currents and tides exert influence on the shoreline profile, where the tidal currents can erode and transport sediments. For this study, Terengganu waters is selected, as it is known for mixed semi-diurnal tides which has two tidal cycle in a day, but one cycle is much stronger than the other, with maximum wave height over 0.5 meters during spring tides and 0.1 meters during lowest tide (NAHRIM, 2014). At the same time, the site also experienced severe coastal erosion which explained the need of NAHRIM wave breakwater to be installed at the study area. Therefore, the aim of this study is to propose a conceptual design of an optimum micro marine turbine mount on NAHRIM wave breakwater (structure NB4ERS) with intention of producing energy from tidal current. In parallel the objectives are as follow:

- i. To identify an appropriate marine turbine
- ii. To develop the conceptual design

of the integrated marine turbine mount on wave breakwater
iii. To perform a performance analysis on proposed design.

Methodology

The study begins with the literature review on marine turbine technology and potential site location. The current technology on the marine turbine has been addressed with various features producing different output power. Potential area with strong influence of tidal current energy was recognized using Bottom Deployment Method by installing Acoustic Doppler Current Profiler (ACDP) and Acoustic Waves and Currents (AWAC). Then, the most suitable marine turbine with maximum output power was identified providing its own specification. Based on the selected marine turbine, design concept of integrated breakwater with micro marine turbine was initiated with the modification of venturi-effect ducted turbine on WABCORE breakwater.

Follow by performance analysis on the design of venturi-effect ducted turbine and breakwater. The designs were analysed numerically using Finite Element Analysis (FEA) and ANSYS Fluent Computational Fluid Dynamics (Fluent CFD) software to study the durability of the integrated

breakwater. The performance analysis result for each design were compared and duct design which produce optimum power output was selected for actual scale prototype fabrication. Laboratory verification and parametric studies were conducted at NAHRIM Multi-Purpose Flume Lab using the actual scale prototype. Then, laboratory test results were discussed, and final modification was done before the WAB-CORE block is installed at the selected location. The study ends with the report compilation that summarised overall finding of this project. Figure 2 illustrates the flowchart of research activities.

Potential Site Location

Earth's surface is covered by more than 70 percent of ocean (Yaakob et al., 2006). To date, ocean waves is known as one of the alternative sources for sustainable energy, and by using wave energy converter devices it can be converted into electricity. The integration of the wave energy converter devices with breakwater was initiated since 1980's in Japan (Ojima et al., 1984). The first prototype of this integration was successfully installed at Sakata Port in 1989 (Takahashi et al., 1992) with an aim to utilise the available breakwater structures and reduce power generation cost. Nevertheless, a suitable site selection is crucial to obtain high performance of electricity production. Mustapa (2017) has listed out the availability of wave energy resources in various countries including tropics, North Sea, the Atlantic Ocean and Southern Pacific Ocean. Compared to the others, tropics countries have low energy potential, with the lowest part in West Peninsular Malaysia. Suitable potential location to harness.

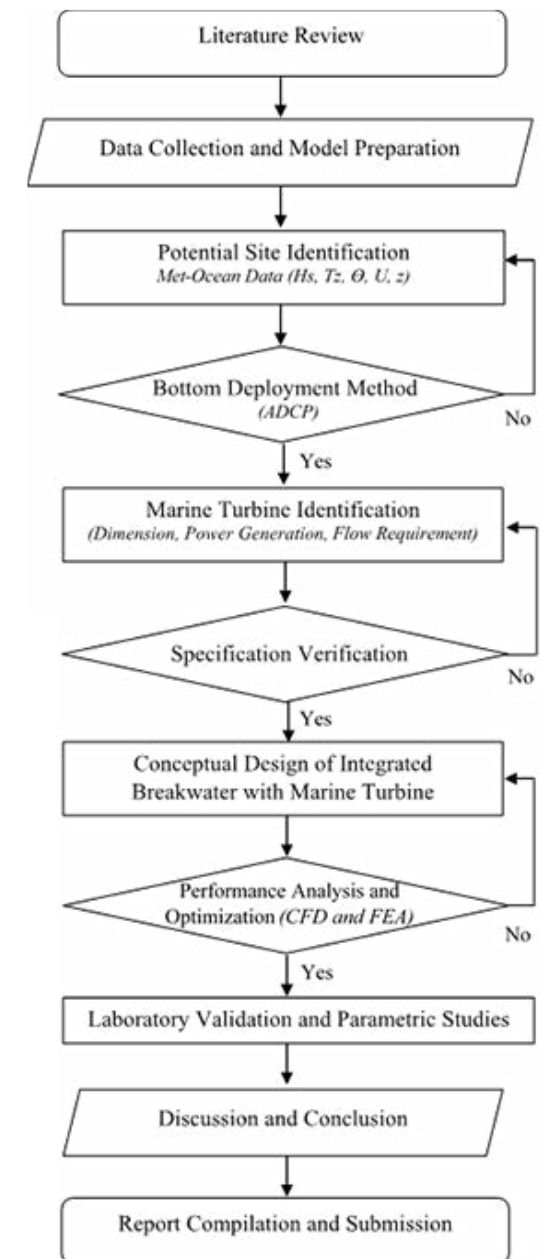


Figure 2: Flowchart of project activities.

wave energy is evaluating based on sufficiency of wave power to generate electricity. Wave power can be induced when ocean waves, currents and wind are in a steady rate, nevertheless the wave power that can be generated also differ based on weather condition and efficiency of the marine energy converter device used. Aforementioned, tidal current energy has great potential to be developed as an alternative energy for power generation. Tides are happened due to the combined effect of rotational earth towards the gravitational forces of moon and sun. It causes a rise and fall of sea level that creates potential energy, while the flows due to flood and ebb currents produce kinetic energy. Both energies can be utilised by the technology of tidal energy using tidal energy converter devices as renewable energy. The concept of integration of tidal energy converters with marine facilities was invented by Dutch coastal engineer in 1997.

Supported by the Dutch government, a pilot project known as Dynamic Tidal Power dam along Chinese coast was successfully installed in 2015 (Kempener and Neumann, 2014; Steijn, 2015). The 30-60 km long dam installed with a bi-directional turbine able to generate 5 GW or more of installed capacity. This is due to the availability of tremendous potential tidal energy in China's four coastal waters (Zhang et al., 2014).

In Malaysia, based on study by Muzathik et al., (2010), results show that east coast of Peninsular Malaysia by latitudes 3.5o N and 6.5o N and longitudes 102o E and 104.0o E, has more than 60 percent of annual wave energy provided by significant wave height less than 1.2 meter with higher total wave energy potential during northeast monsoon season. This supported with recent study by Nasir & Maulud (2016), Terengganu waters is one of hot-spot area with total area of 52,351,809.9m² and positive wave energy value to be exploited for ocean energy (refer Figure 3).

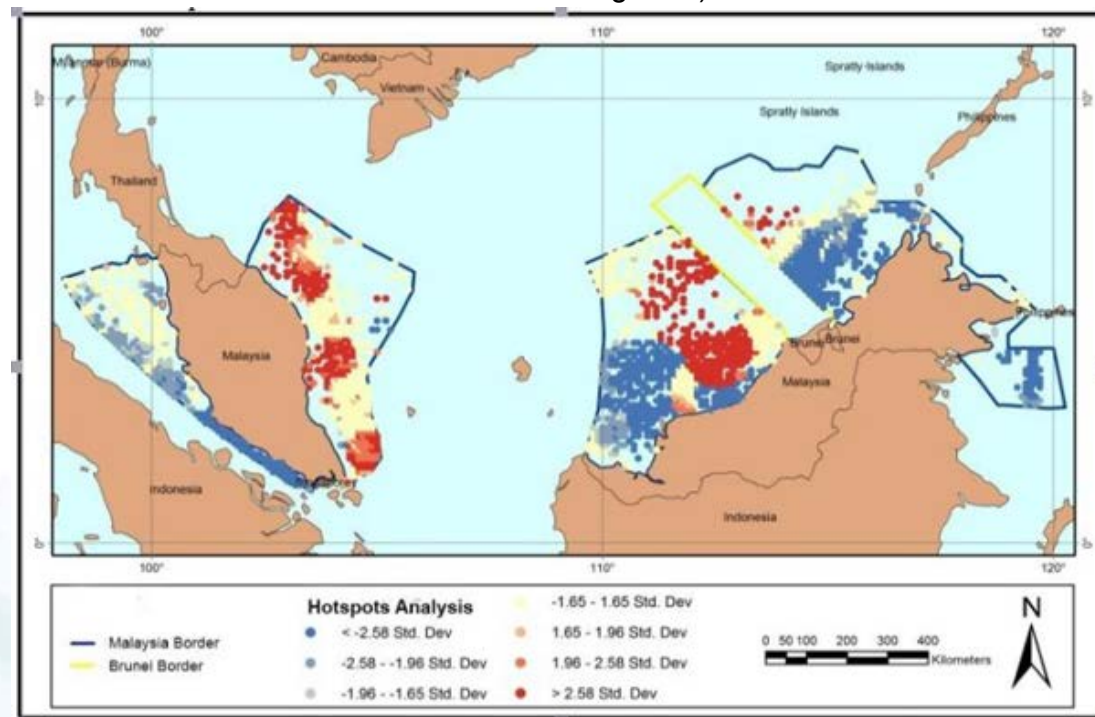


Figure 3: Hot-spot area with wave energy potential (Nasir & Maulud, 2016).

Terengganu area is known for mixed semi-diurnal tides which has two tidal cycle in a day but one cycle is much stronger than the other (NAHRIM, 2014; Lim & Koh, 2010). For this paper, the location of study area is selected based on data collected by NAHRIM (2014) which located in coastal area at Pantai Rhu Muda, Marang, Terengganu, nearby Kapas Island (refer Figure 4). Data collected by NAHRIM show strong influence of tidal current and tidal waves with current speed decrease steadily from surface to bottom of sea. Current speed at the surface reach maximum speed of 0.70m/s, but average speed is 0.30m/s. Strong tidal movement can be seen at middle depth and bottom shares same average current speed, maximum speed is 0.35m/s with average of 0.20m/s.

Maximum wave height at the study area is over 2.0m while minimum wave height recorded at 0.5m.

Even though Malaysia has low wave energy potential due to its location of sheltered seas, but it experienced a predictable nature of tides all of the year. This gives an excellent opportunity to explore the energy generation using tidal energy converters by utilised the stand-alone marine facilities such as breakwater. The breakwater acts as the protection of tidal energy converters against severe wave impacts. It is, however, should maintain its primary function to control longshore sediment transport and protected desired area.

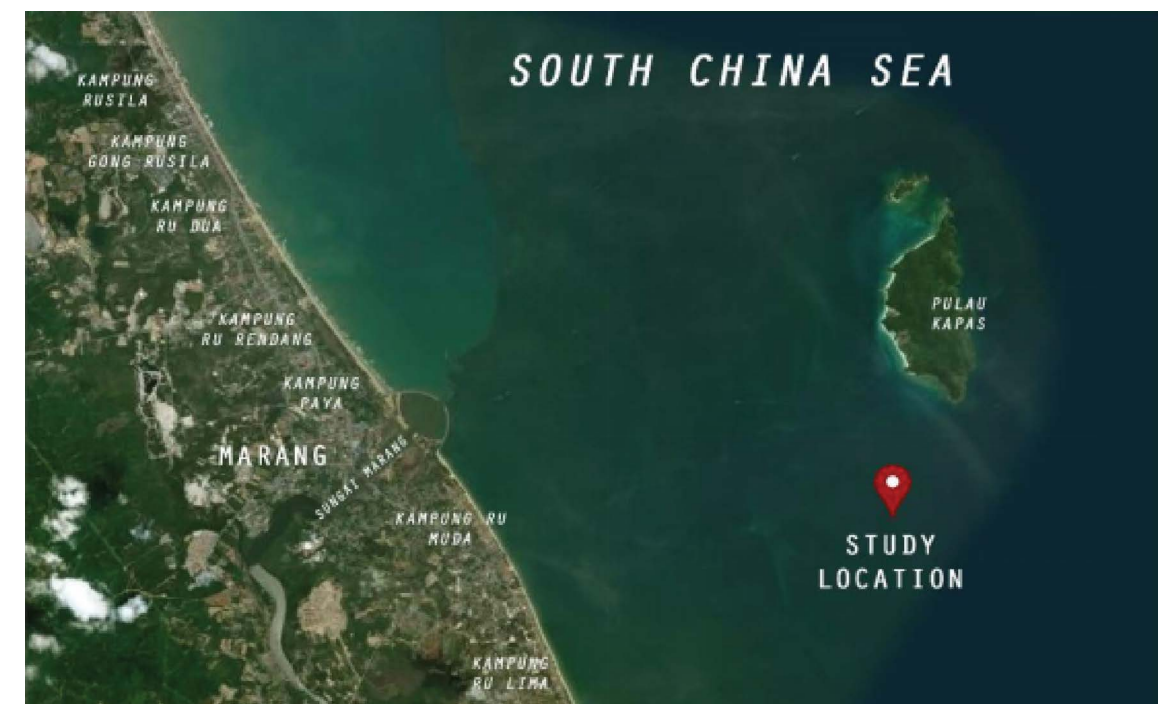


Figure 4: Location of study area

Conceptual Design of Integrated Breakwater Marine Turbine

The conceptual design of integrated NAHRIM wave breakwater, named WABCORE with marine turbine has been successfully developed for the condition of the low-speed tidal environment. Providing with 1 meters (H) x 1 meters (L) x 1.5 meters (W) of the concrete breakwater (refer Figure 5), four model of horizontal axis axial-flow ducted marine turbine has been proposed.

Design Development

In this study, the ducted turbine is designed based on axial-flow turbine with venturi shape duct. Axial-flow turbine was used as it is commercially obtainable in the market compared to cross-flow turbine (Robert et al., 2016). The propose systems feature three components (refer Figure 6), where

Component 1 captures a large area of the tidal stream and accelerates the flow through a narrowing channel (Component 2) into the micro-marine turbine and exit through Component 3. The venturi shaped duct reduced the fluid pressure which results velocity of the tidal current to increase towards the micro marine turbine. Even with small diameter of micro marine turbine, it can capture more energy from same amount of seawater compare to stand-alone turbine. Besides, the venturi shaped duct would provide some protection for the blades of turbines against debris and biofouling. In addition, the proposed ducted turbine is attached with the WAB-CORE breakwater by positioning its centre at 70 percent of breakwater height from the seabed for the same protection against debris and biofouling.

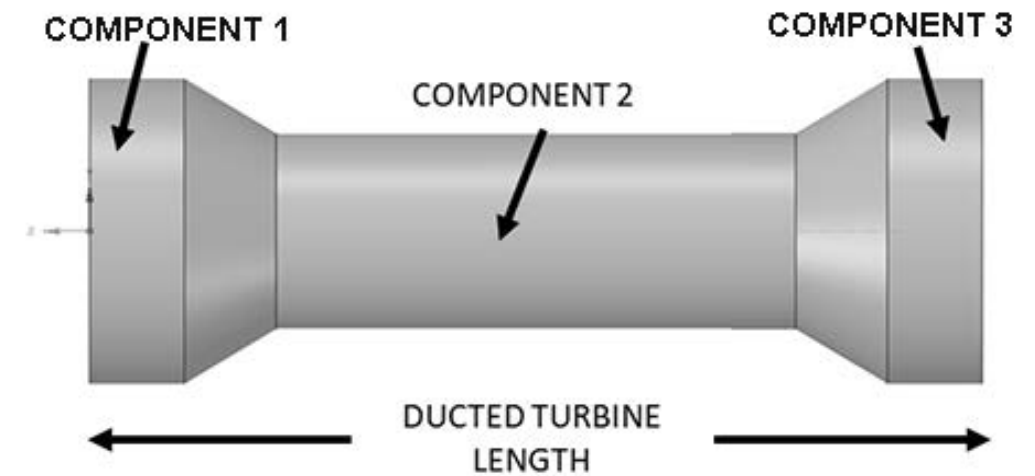


Figure 6: Components of Proposed Ducted Turbine

Four comparative models have been developed according to the proposed turbine design as shown in Figure 7. All Models 1, 2, 3 and 4 are designed with equivalent length and the micro marine turbine is positioned at the centre of Component 2. The significant difference between Model 1 with Models 2, 3 and 4 is on the diameter and design of venturi shaped duct. Model 1 is designed as normal open channel duct shape with same diameter. Meanwhile, Model 2 and Model 3 are designed with dissimilar narrow design of venturi shaped duct. This to discern which design can capture higher velocity of tidal current into the duct pipeline and supply enough current speed to rotate the micro marine turbine. Model 4 is repetition of Model 2 with bigger diameter of venturi shaped duct. Details on the proposed models can be refer in Table 1.

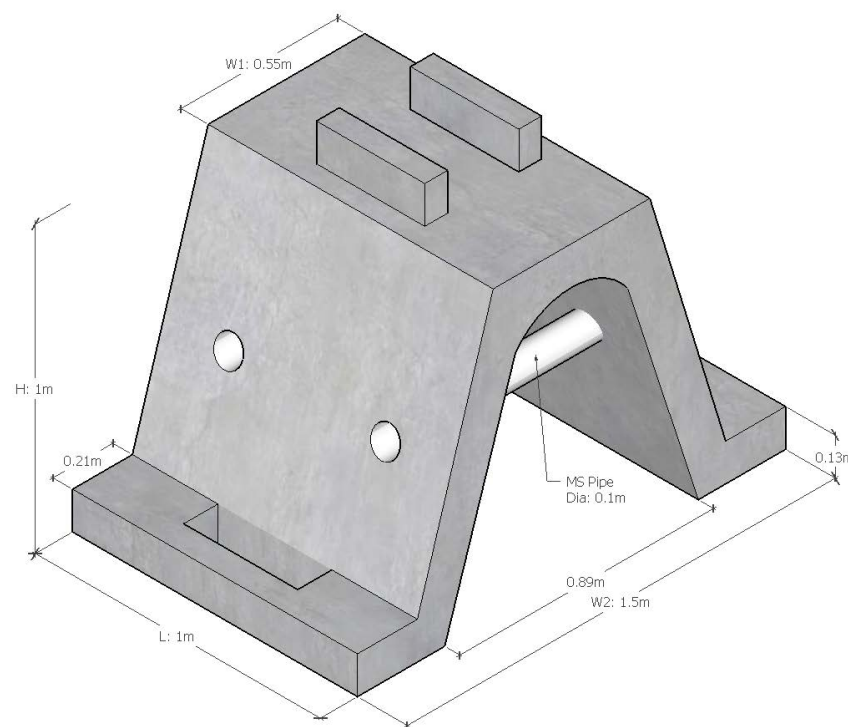


Figure 5: NAHRIM breakwater structure (WABCORE)

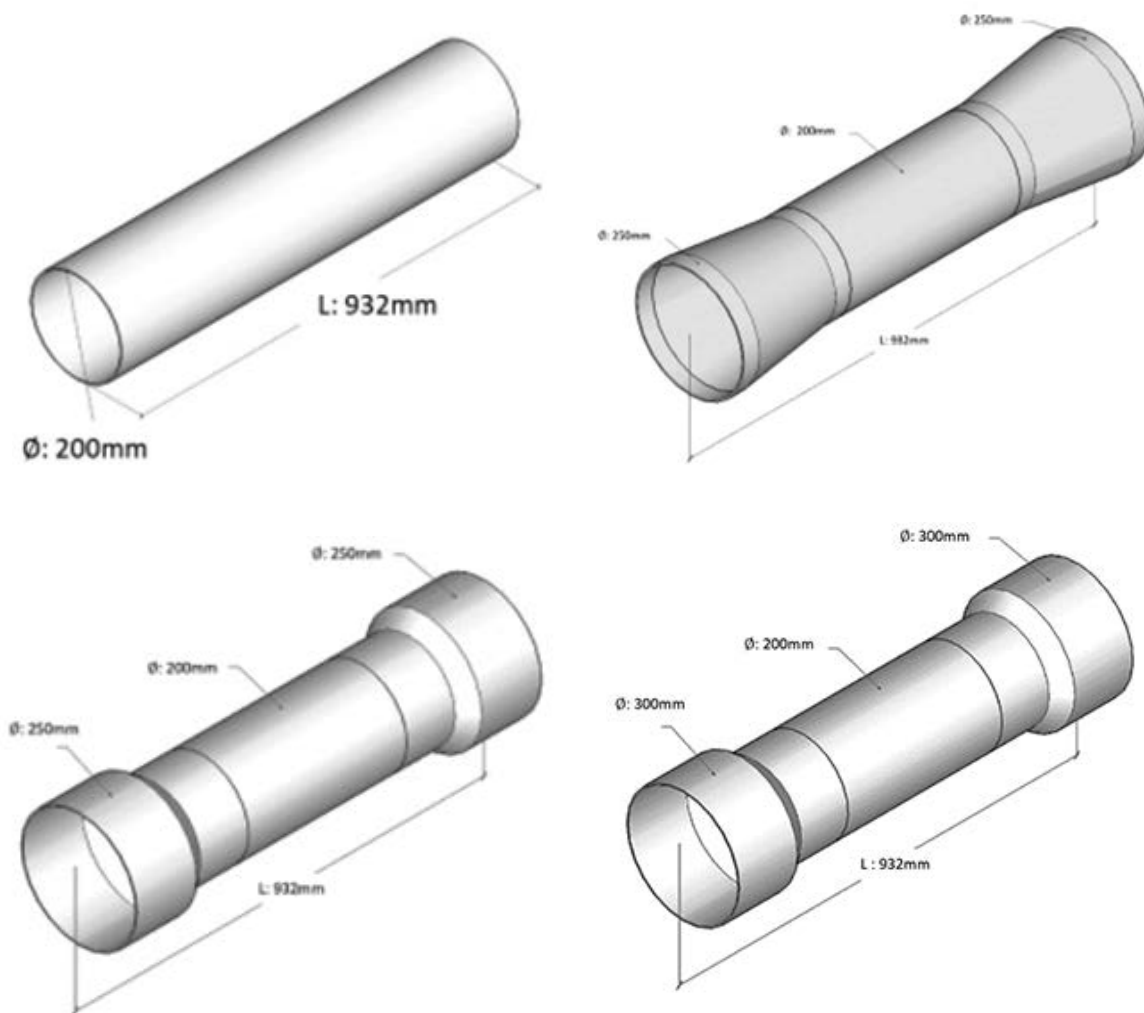


Figure 7: Proposed design for horizontal axis axial-flow marine turbine

Design Optimization

In the design process, ANSYS Fluent Computational Fluid Dynamics (Fluent CFD) software was used for validating and optimizing the ducted turbine design prior to prototype production. The key point of Fluent CFD model testing is to analyse which ducted turbine design that can provide highest velocity inside the stream flow pipe. The ducted turbine has been analysed with inflow water speed at 1.0m/s. During the simulation, the characteristic of fluid followed properties of sea water; and it was assumed that the turbulence level inside the ducted turbine is about 5 percent. Based on the result of Fluent CFD simulation (refer Figure 8 and Figure 9), design of Model 2 and Model 4 shows greater fluid velocity attained at the centre part of Component 2 with 1.67m/s and 2.35m/s respectively. For the selection of an optimum ducted marine turbine design, design of Model 4 is selected as it has a great potential to obtain the highest power output. Higher velocity attained at the centre part of duct pipeline (as illustrated in Figure 9) allow the micro-marine turbine to generate more power output. Hence, it is clearly shown that the suitable venturi shaped duct plays a primary role in maximising the power generation.

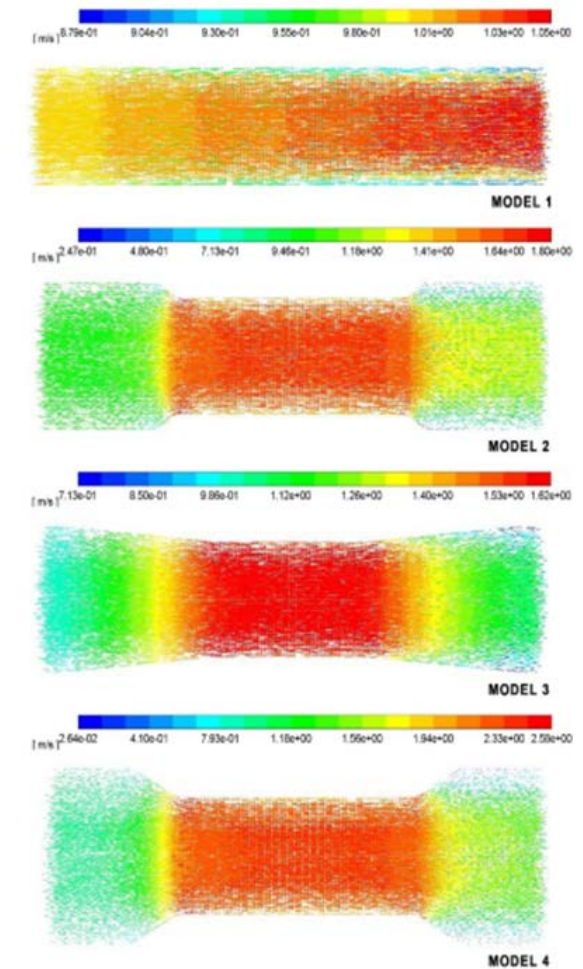


Figure 8: Fluent CFD Simulation Results

Table 1: Proposed Micro Marine Turbine

Turbine	Length, L (m)	Diameter, ϕ (m)		
		Component 1	Component 2	Component 3
Model 1	0.93	0.2	0.2	0.2
Model 2	0.93	0.25	0.2	0.25
Model 3	0.93	0.25	0.2	0.25
Model 4	0.93	0.3	0.2	0.3

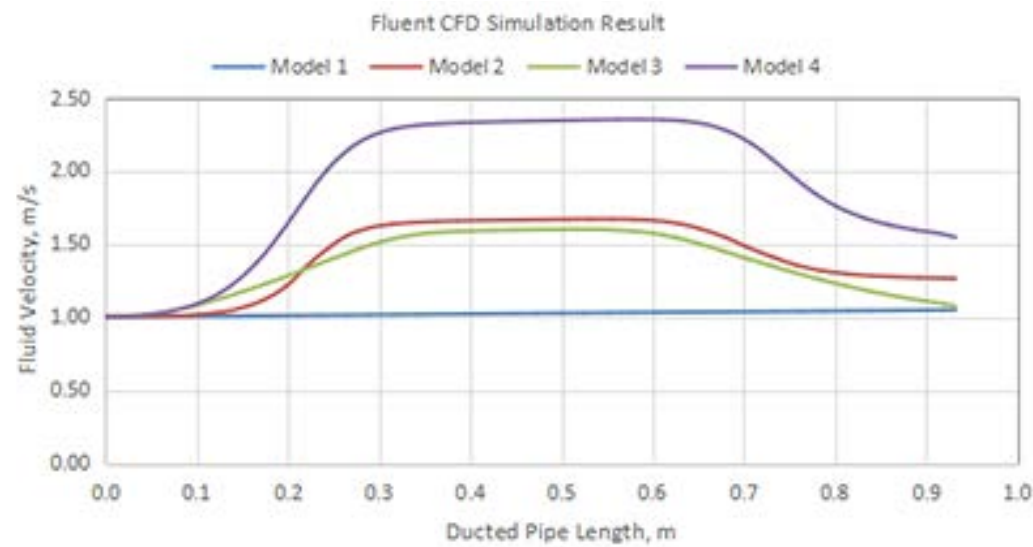


Figure 9: Fluent CFD Simulation Results

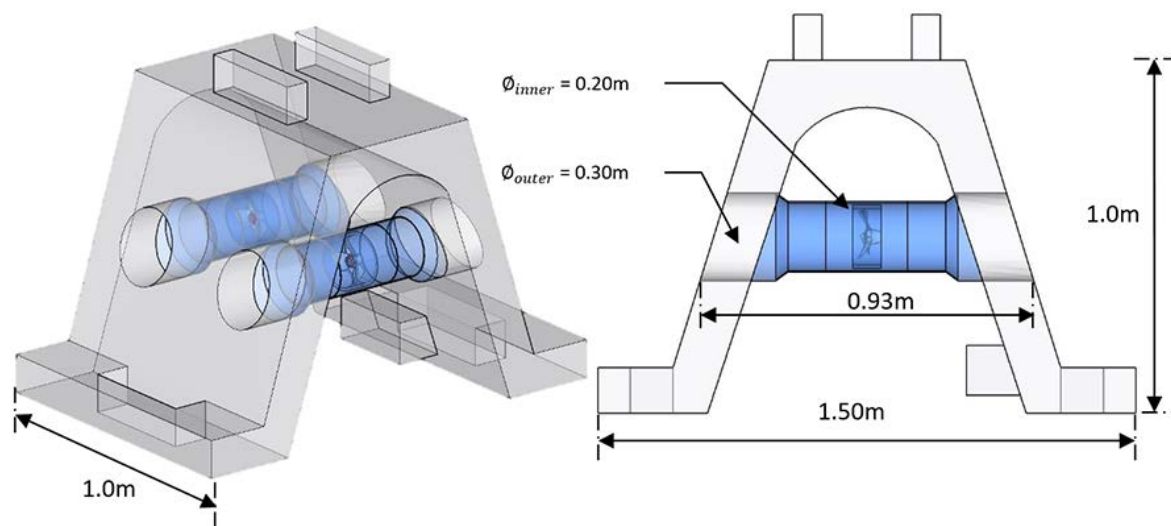


Figure 10: Schematic Diagram of Turbine Model 4

Design Prototype

Model 4 was selected for prototype fabrication to be integrated with breakwater structure WABCORE. The perspective view

of the integration breakwater with axial-flow marine turbine was illustrated in Figure 10, Figure 11 and Figure 12.

A laboratory test on the full-scale prototype for the model validation was conducted at Multi-Functional Flume NAHRIM. During the validation testing, the flume was filled with water up to 1.2 meter and wave speed generated are based on current velocity at study location, Pantai Rhu Muda, Marang. An electro-magnetic current meter was used to measure the velocity intake for the ducted turbine. Figure 13 below shows illustration of the laboratory test.



Figure 11: Perspective View of Tidal Current Flow across Ducted Turbine

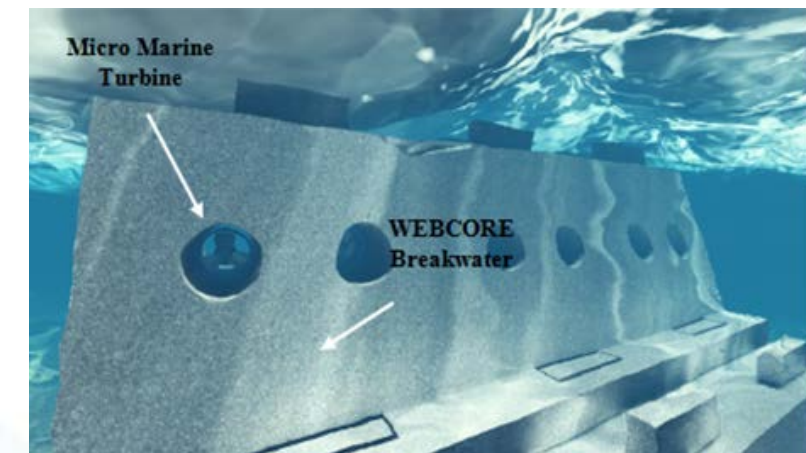


Figure 12: Perspective View of Integrated Structure WABCORE Breakwater and Micro Marine Turbine

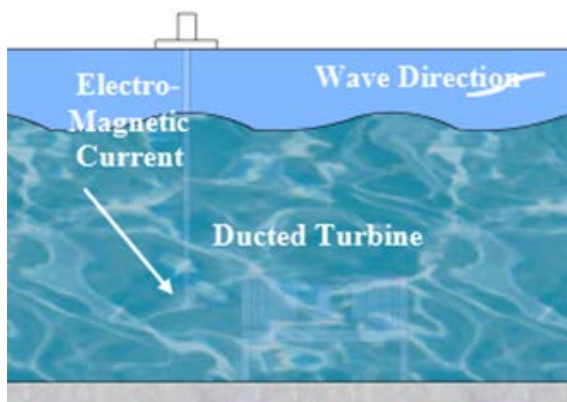


Figure 13: Illustration of Laboratory Test at NAHRIM

Performance Analysis

Finite Element Analysis (FEA)

Finite Element Analysis (FEA) was conducted on both original WABCORE block and modified block to investigate the effect of design modifications towards the mechanical performance of the blocks. The blocks were initially modelled using AutoCAD while FEA was conducted using Abaqus. Material properties of concrete grade 30 were used in the FEA model with nominal values as tabulated in Table 2. Applied loading is also detailed in the same table.

Table 2: Material properties and load definition

Material Properties	
Material	Concrete Grade 30
Young's Modulus	30 GPa
Poisson's Ratio	0.15
Density	2400 kg/m ³
Load	
Type	Uniform Pressure
Magnitude	150 kPa

Outputs of the FE analysis are explained as follows:

Stress Distribution

Figure 13 shows the stress distribution of original WABCORE block (left) and modified block (right). Based on the contour plots, modifications made on the block (i.e., diameter of holes) does not significantly affect the stress distribution on the block where the stress value is still within safe tolerance (maximum of 0.37 MPa).

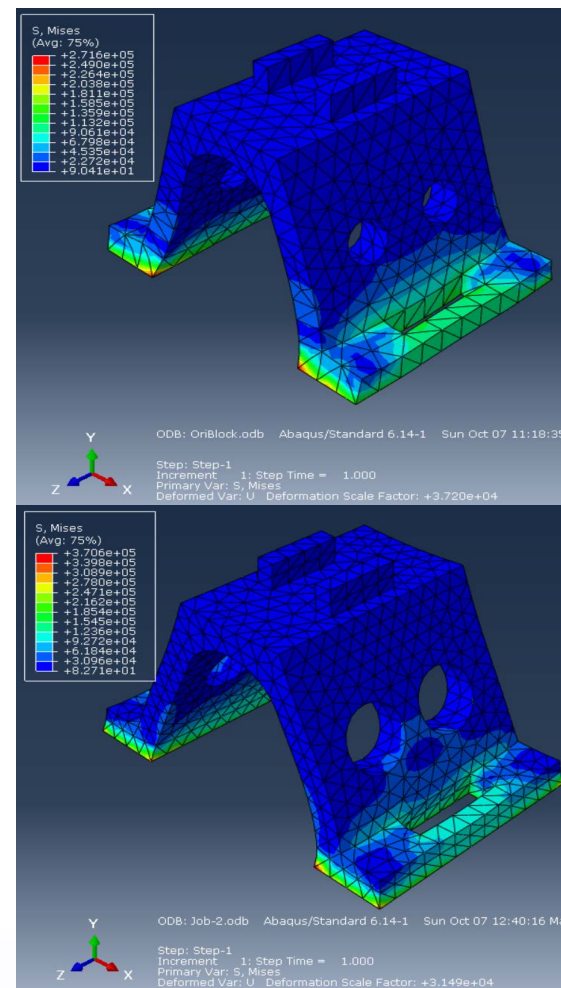


Figure 13: Stress distribution on original NABERS block (above) and modified block (below)

Displacement

Figure 14 shows the displacement of original WABCORE block (left) and modified block (right). Based on the contour output, very small changes could be observed in terms of displacement. It could be concluded that the modified block is performing as well as the original WABCORE block.

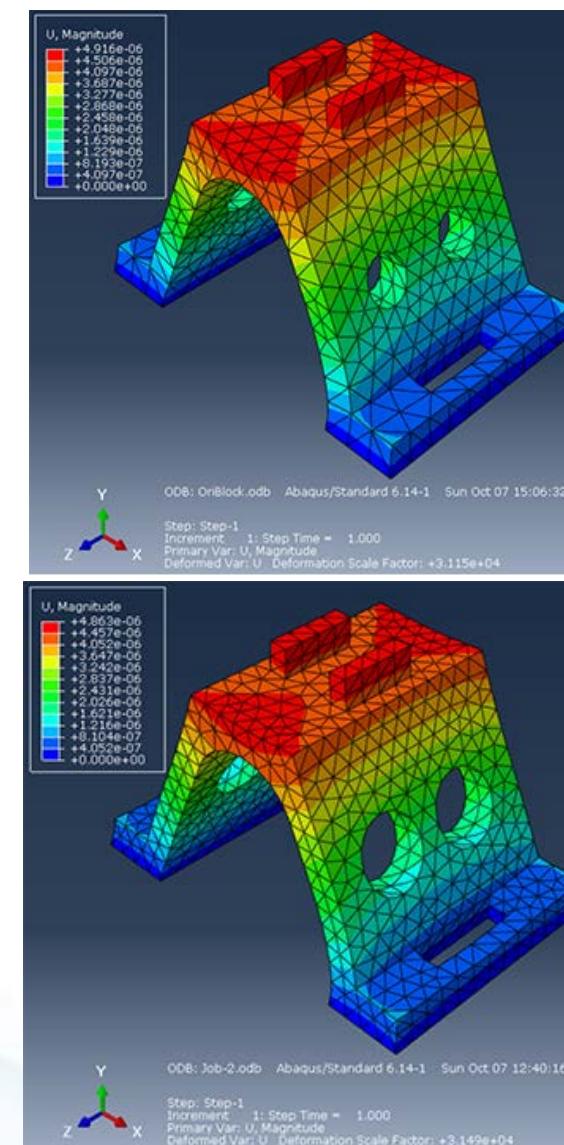


Figure 14: Displacement on original NABERS block (above) and modified block (below)

ANSYS Fluent Computational Fluid Dynamics (Fluent CFD)

The total kinetic power in a marine current turbine has a similar dependence as a wind turbine and is governed by the following equation (Hammons, 1993; Couch and Bryden, 2006; Benelghali et al., 2007):

$$P = \frac{1}{2} \rho A C_p v^3 \quad (1)$$

where ρ is the fluid density, A is the swept area of the turbine and v is the fluid velocity. C_p is known as the power coefficient and is essentially the percentage of power that can be extracted from the fluid stream and takes into account losses due to Betz law and those assigned to the internal mechanisms within the converter or turbine. For wind generator, C_p has typical values in the range 0.25–0.3. The upper limit is for highly efficient machines with low mechanical losses. For marine turbines, C_p is estimated to be in the range 0.35–0.5 (Myers and Bahaj, 2006).

By apply Eq. (2) for the selected model of ducted turbine (Model 4), the expected power output can be determined. Table 3 states the parameter values for the corresponding model and Figure 13 illustrated the power output graphically. The line graph of Figure 15 shows the expected power output of proposed micro marine turbine varies with the inlet velocity from 0.1 m/s to 1.0 m/s. The inlet velocity was set with corresponding to the standard current velocity of seawater at Peninsular Malaysia (Idris et al., 2014). It should be aware that the inlet velocity that passes through the turbine will accelerate as it enters the narrowing channel of the duct space, hence producing more power output. The value of power output for the ducted

pipeline from the Fluent CFD simulation were compared with laboratory test result and power output given by the manufacturer (WaterLily).

Table 3: Value of parameter for the corresponding model

Model	Inlet Fluid Velocity, (m/s)	Fluid Density, $p (kg/m^3)$	Swept Area, $A (m^2)$	Power Coefficient, C_p	Fluid Velocity, $v (m/s)$
4	0.1	1025	0.20 x 0.15	0.59	0.24
	0.2	1025	0.20 x 0.15	0.59	0.47
	0.3	1025	0.20 x 0.15	0.59	0.71
	0.4	1025	0.20 x 0.15	0.59	0.94
	0.5	1025	0.20 x 0.15	0.59	1.17
	0.6	1025	0.20 x 0.15	0.59	1.41
	0.7	1025	0.20 x 0.15	0.59	1.64
	0.8	1025	0.20 x 0.15	0.59	1.87
	0.9	1025	0.20 x 0.15	0.59	2.11
	1	1025	0.20 x 0.15	0.59	2.34

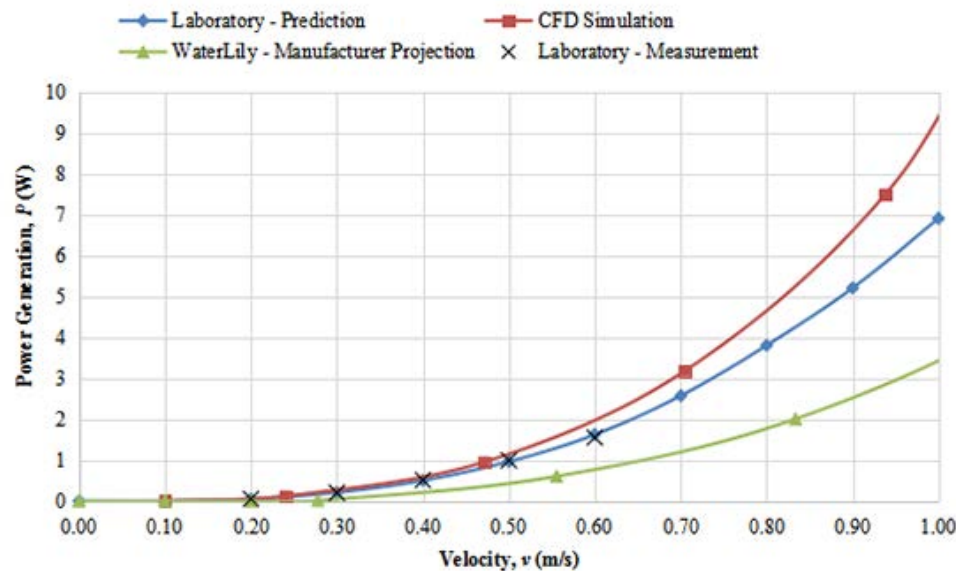


Figure 13: Expected Power Output of Proposed Horizontal Axis Axial-flow Marine Turbine

Conclusion and Recommendation

In this study, the conceptual design of integrated NB4ERS NAHRIM wave breakwater with axial-flow marine turbine was successfully developed for the condition of the low-speed tidal environment. Four models have been proposed corresponding to its fluid velocity. The model has a significant difference in terms of the venturi shaped design of ducted axial-flow marine turbine that provided an enormous impact on power generation. Maximum power output of 15 Watt/turbine was obtained adopting integrated structure of Model 4, positioned in coastal area at Pantai Rhu Muda, Marang, Terengganu, nearby South-West of Kapas Island.

Acknowledgement

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THREE-DIMENSIONAL FLOW MEASUREMENT WITH ACOUSTIC DOPPLER VELOCIMETER (ADV)

Mohd Sofiyan Sulaiman¹, Ahmad Jusoh¹, Goh Qiu You¹, Xiang Zhi Chung¹

ABSTRACT

It is crucial to obtain high quality data of three-dimensional flow at field scale as previously demonstrated at laboratory scale. The use of Doppler system is useful in capturing fine scale of flow and turbulence data. However, careful filtering and despiking process must be employed prior to data analysis. It is proposed that CORcr = 60 and signal-to-noise ratio (SNR) of 15 gave better filtered results to eliminate noise and aliasing in Acoustic Doppler Velocimeter (ADV) data. Three dimensional flow of lateral (y-axis), vertical (z-axis) and stream wise (x-axis) are crucial to obtain mean velocity, turbulent intensity, turbulent kinetic energy, Reynolds stresses, horizontal-vertical interaction, local bed shear stress, quadrant analysis and determination of eddy periodicity. These technical data are crucial for sediment transport, hydrodynamic and flow field study. As such, incipient motion at local scale can be determined by using local bed shear stress instead of reach averaged bed shear stress. To do so, the data input of lateral (y-axis), vertical (z-axis) and stream wise (x-axis) are crucial for computation.

Keywords: Acoustic Doppler velocimeter, Despiking, Three-dimensional flow, Turbulence, Flow field

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Introduction

Since the inception of 3D-Acoustic Doppler Velocimeter (ADV) in 1994 by the U.S. Army Engineer Waterways Experiment Station (WES), the application of Doppler system has become a prominent device to measure flow and turbulence data at both laboratory and field scale (Sulaiman et al., 2013). Past works on flow and turbulence measurement mostly relied on the Laser Doppler Velocimeters (LDV), micropropellers, time-of-flight acoustic type current meters, electromagnetic current meters and Pitot tubes (Kraus et al., 1994). The 3D-ADV can withstand hostile and robust environment at both oceanic and mountainous regions, and can maintain high quality data as previously obtained at laboratory scale. The fluctuation data is readable through acoustic detection from ADV beam. Although LDV uses the same principle, ADV is capable of capturing the finer-scale of turbulence. Voulgaris and Trowbridge (1998) hypothesized that ADV is invulnerable to corrosive issues and is able to avoid deterioration of acoustic transmission due to high level of turbidity. Although, to date, ADV has been regarded as reliable instrumentation, evaluation has to be made to ensure the quality of the data and the trustworthiness of the results. As such, Voulgaris and Trowbridge (1998) provided discussion on the accuracy of ADV data as opposed to LDV data by the means of looking at the consistency between measured parameters and theoretical parameters. Nikora and Goring (1998) and Groing and Nikora (2002) provide insight formulation to despiking the noise data from time series. In addition, McLelland and Nicholas (2000), Blanckaert and Lemmin (2006) offer alternatives to detect and remove noise from ADV data. However, for estuary

application, Cea et al. (2007) and Chanson et al. (2008) proposed the post-processing technique for application purposes which slightly differs from natural river algorithms. Past reports have been made available on both field scale and laboratory scale using Doppler system to capture the flow and turbulence data (e.g: Song and Chiew, 2001; Ferro, 2003; Garcia et al., 2005; Kim et al., 2000; Nikora and Goring, 1998; Sukhodolov and Rhoads, 2001; Carollo et al., 2005; Tritico and Hotchkiss, 2005; Stone and Hotchkiss, 2007; Strom and Papanicolaou, 2007; Lacey and Roy, 2007). A few attempts have been made to relate the turbulence effect on sediment entrainment and transportation via ADV instrumentation (Sulaiman et al., 2013; Wren et al., 2014; Grams and Wilcock, 2014; Curran and Tan, 2014). However, the application of Doppler system in capturing flow and turbulence data at Malaysian rivers received little attention except previous works by Sulaiman et al. (2013). This paper summarizes the application of ADV in collecting raw data and performing despiking process using Nikora and Goring (2002) method with a special reference to gravel bed river. In addition, the eddy taxonomy methodology is presented to discriminate small scale eddies from large scale eddies via visual interpretation of time series and skewness diagnosis. Finally, the relationship between local scale shear stress and bed load transport is shown to observe the application of three-dimensional flow data in fluvial research.

Acoustic Doppler Velocimeter System Description

The standard 3D ADV comprises of an acoustic transmitter, three acoustic receivers, a signal condition module and a data processing module. There are two

types of ADV versions (manufactured by Nortek) available; namely Vectrino (laboratory version) and Vector (field version). The Vector uses the Doppler shift principle to measure current velocity by transmitting sound pulses, listening to their echoes and measuring the change in pitch or frequency of the returned sound (Nortek, 2005). In contrast to Doppler profiler, the receiving and transmitting beams for ADV are operated separately. The central beam will transmit the sound wave and the three receivers will receive the signal. Figure 1 show the principle of ADV operation and the simple mounting of the ADV on a temporary frame for experimentation works. Three receivers intersect each other at 157 mm from the transmitter. The measurement volume is defined by the intersection of these three receiver beams. Thus, the actual sampling volume is located at an approximate 15 cm beneath the transmitter. The transmit transducer sends a short pulse that covers only about 4 mm vertically, and the receivers listen to an echo that corresponds to about 14 mm vertically (Gordon, 1999). As the short pulse is transmitted along the transmit beams, the suspended particles will scatter the acoustic energy back to the receiver beam (Nortek, 2005). The emitted and received ADV frequency can be defined as $f_e = c/\lambda_e$ and $f_r = c/\lambda_r$ respectively where f = frequency, c = speed of sound and λ = wavelength. The Doppler shift between the outgoing and incoming pulse is obtained from

$f_D = f_r - f_e$	-1
$f_D = \frac{f_e}{c} \vec{V}(\vec{e}_e + \vec{e}_r)$	-2

where $\vec{V}$ = velocity of particle $\vec{e}_e$ = unit vector along emitter and $\vec{e}_r$ = unit vector along receiver. Thus, the velocity along the transmitter and receiver is defined as

	$u_b = f_D \frac{c}{f_e}$	-3
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The velocity along acoustic axis is typically referred to as "beam" axis and represented as

	$u_i = T_{ij} V_{ij}$	-4
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where V_{ij} = beam velocity (V_1, V_2, V_3, V_4), u_i = orthogonal velocity (u, v, w) and T_{ij} = transformation matrix (K. Strom, personal communication, July 4, 2011). Although the use of ADV is wide spread and quite effective, there were some circumstances where the ADV could not produce the quality data. Strom and Papanicolaou (2007) commented that the application of ADV at coarse bed rivers should require additional care and advance processing from the researchers. The interference of sample volume from the boundary, high level of turbulence, large mean shear gradient and acoustic signal interference between boundary reflection and sample volume are some of the sources of ADV problems at coarse bed rivers. Therefore, initial filtering and despiking process must be executed by the researchers in order to produce high quality data.

Mathematical Expression

The ADV probes will read the velocity in three mutually perpendicular directions: stream wise flow (u), vertical flow (normal to boundary) (w), and lateral flow (v). If the flow is turbulence, the velocity in any of the three directions is not constant. The example of velocity time series can be extracted from ADV data processing module as shown in Figure 2.

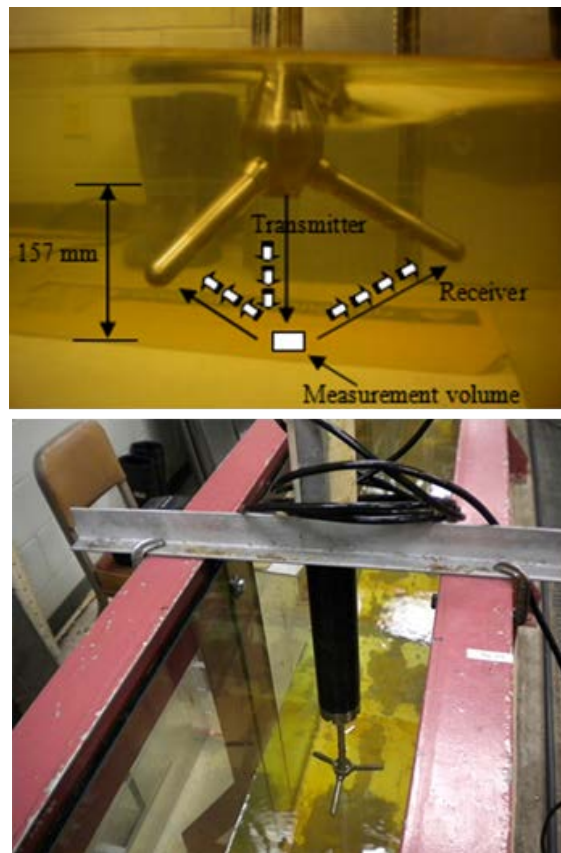


Figure 1: Principal of transmitting and receiving ADV signal (above) and ADV probe mounted on temporary frame for laboratory experimentation (below)

The irregular velocity fluctuations as shown in figure 2 indicating the turbulence motion in an open channel flows. This turbulent velocity record (u) can be summarized into an averaged and random field such that

$$\hat{u}_i(\underline{x}, t) = \underbrace{\overline{u_i(\underline{x})}}_{\text{average component}} + \underbrace{u'_i(\underline{x}, t)}_{\text{fluctuation component}} \quad (5)$$

Using mathematical treatment, it is always useful to separate the instantaneous turbulent velocity into mean motion (notation with overbar) and fluctuation component (notation with prime).

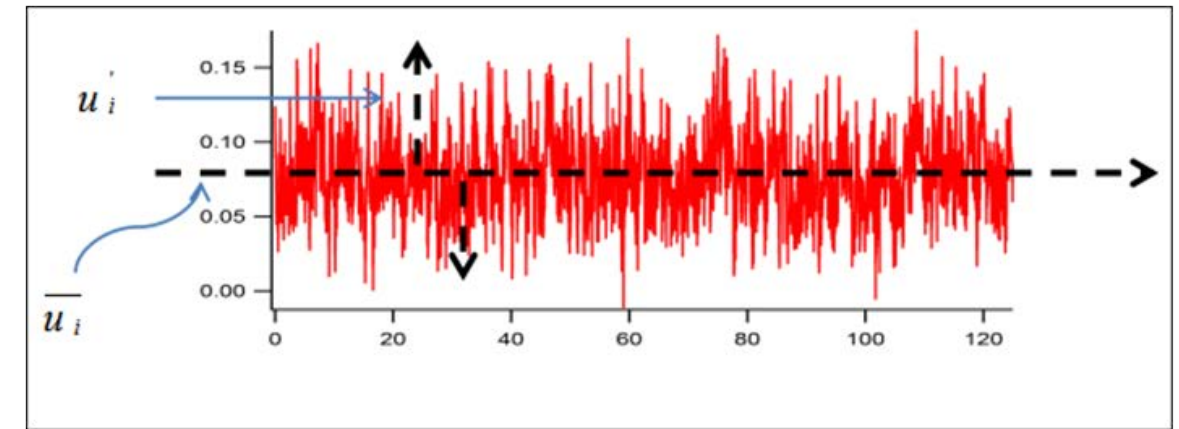


Figure 2: Time series of velocity data

The time-averaged value at fixed point are given by

$$\overline{u_i} = \frac{1}{t_1 - t_0} \int_{t_0}^{t_1} \hat{u}_i dt \quad (6)$$

where i = velocity component in x, y, z directions. The deviations (fluctuations) from the mean velocity are either positive or negative. The average magnitude of the deviation from the mean reveals the 'intensity' of the turbulence for each velocity component. The standard deviation of the velocity distribution or root-mean-square (RMS) is best to describe turbulence intensity. RMS is defined as

$$\text{RMS}_u = \sqrt{\left(\sum u'^2 \right) / N} \quad (7)$$

where N is the total number of observations in a given series. The similar expression can be defined for both lateral and vertical flow components. The three expressions can be combined to obtain the turbulent kinetic energy (TKE) as

$$\text{TKE} = 0.5 \rho (\text{RMS}_u^2 + \text{RMS}_v^2 + \text{RMS}_w^2) \quad (8)$$

The TKE represents the energy extracted from the mean flow by turbulent eddies (Robert, 2003). The fluctuation terms from equation 5 are substituted into the Navier-Stokes equation which yields the turbulent term in the normal stress $(-\overline{p u'_x u'_x} - \overline{p u'_y u'_y} - \overline{p u'_z u'_z})$ and shear stress $(-\overline{p u'_x u'_y} - \overline{p u'_x u'_z} - \overline{p u'_y u'_z})$ formulation known as Reynolds stress. Reynolds stress represents the degree of momentum exchange at a given point in the flow (Liu, 2001; Robert, 2003). In fluvial dynamics studies, this parameter is very important to calculate the transport field and the scour, deposition and channel change (Biron et al., 2004). Besides, the eco-hydraulics study requires the flow parameters which are capable to describe the linkages between hydrodynamic and ecological process (Stone and Hotchkiss, 2007). It is important to describe the spatial heterogeneity of hydrodynamic in natural rivers by analyzing the mean velocity, Reynolds shear stress, turbulent intensity (TI) and turbulent kinetic energy (TKE).

Pre- / Post-Processing Technique of ADV Data

The pre- and post-processing technique inferred the steps taken at ADV setting, collecting the data, reading the data, filtering the data and interpreting the data. The framework for applying ADV is deduced in Figure 3.

Pre Processing Technique

At the pre-processing (prior to processing the data), the pitch and roll must be set at 0 values (or closer to 0). This setting is crucial to make sure that tilt issue is solved and the transmitting transducer is perpendicular with water column. The leg of ADV receiver (stream wise receiver) must face the flow at opposite direction regardless of obstruction at upper part. If the secondary current exists, the receiver leg must be adjusted to be in line with the stream wise direction. It is crucial to avoid any interference induced by the mounting leg to the ADV receiver which in turn propagate noise and error to ADV reading. The sampling rate sets the output rate from transducer for velocity, amplitude and pressure reading. It is recommended to increase the sampling rate in clear water since the Doppler system is more effective at greater presence of suspended particles. Velocity range is the maximum expected velocity from the deployment. It is a nominal velocity range as the uncertainty is expected for different sampling rate. Thus, the user must set the highest possible velocity range prior recording the data. Three coordinate systems can be selected from the vector software: Beam, XYZ or ENU (East, North, Up). It is convenient to use xyz coordinate because it is readable without further transformation. X, Y and Z represents stream wise, lateral and vertical direction respectively. Sampling volume is directly

related with the proximity of ADV transducer to the boundary. When the transducer is close to the boundary, it is useful to reduce the sampling volume to limit the vertical extent of transmitting pulse. Transmit length is directly proportional to sampling volume. It is advisable to increase the transmit length in clear water due to weak backscatter for transmitting pulse. Reading and recording the ADV data can be made using two approaches; real time monitoring and autonomous deployment. In ADV vector, data acquisition mode offers two schemes; continuous data acquisition offers collecting the data without breaks whereas burst data acquisition samples the data with breaks at regular intervals. The retrieval data must be converted into ASCII format for further filtering and analysis. ASCII data are readable and easy to import into most available analysis program.



Figure 3: Flowchart of ADV execution

Post Processing Technique

Post-processing techniques entails the process of filtering the data and interpret the results for physical reasoning. The sampled ADV data consist of spikes and noise due to slow moving water, clear water or obstruction element. Thus, the data must be filtered using three approaches; Signal-to-Noise Ratio (SNR), Correlation (COR) and despiking approach. Low SNR could mean bad data due to inability of receiver to distinguish the noise and velocity signal. Thus, SNR should be in the

region of 10-15. Correlation (COR) implies how close the pulse pair is (transmitting and receiving pulse). The value for COR should be in the region of greater than 70%. However, these values varied and subject to flow function (Wahl, 2000; Strom and Papanicolaou, 2007). The threshold value for COR can be examine through Shapiro-Wilks test statistic for normality of a distribution. The true value is assumed to be normal than its contaminated parts (Strom and Papanicolaou, 2007; Sulaiman et al., 2013). From Figure 4, it is clear

evident that Shapiro-Wilks statistic is started to decline at $COR=60$. For data which under-goes despiking process, Shapiro-Wilks statistic is close to 1 opposed to data which not undergoes despiking process. The threshold correlation is best equipped at $COR_{cr}=60$ for the whole ADV data.

Another problem with ADV data is spikes due to contamination between the outgoing and incoming pulses reflected from the complex geometries of a stream (Sulaiman et al., 2013). The Phase-Space Thresholding Method (Goring and Nikora, 2002) emulates the concept of three-dimensional Poincare map or phase-space plot where the points outside ellipsoid are defined as spikes. After the spike detection, the

extrapolation is commenced to replace the contaminated points (Figure 5). After the filtering process, the clean data must be assessed prior to physical reasoning/interpretation. The visual distribution of a histogram and probability density function (pdf) provide evidence of cleaned data and ready for further analysis. The histogram for stream wise, lateral and vertical direction must well fit with Gaussian distribution as shown in Figure 6. It is observed that the data prior to filtering process is positively skewed and considerable spikes are observed for time series data. After the filtering and despiking process, the histogram is normally distributed and well fit with Gaussian bell curve.

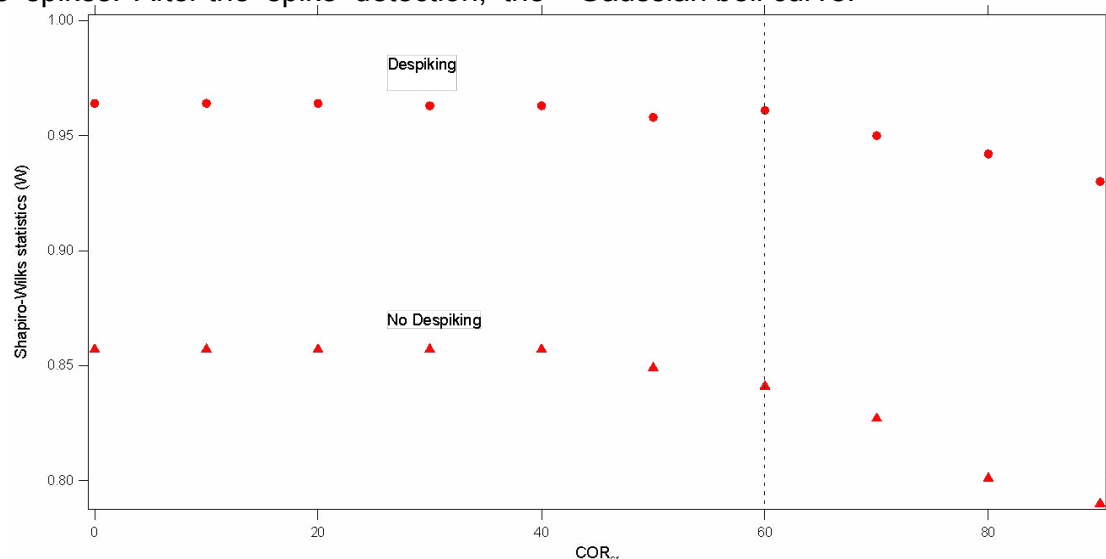


Figure 4: Shapiro-Wilks test for normality

Interpretation of ADV data

Morphodynamic feedback of a fluvial system is largely relying of these three interactions: flowing water, deformable boundary and transported sediment. These three themes are inter-related and provides avenue for further observation. Flowing water relates the flow states whether it is laminar or turbulent; depending on the Reynolds number and attributes of turbulent flow. The impacts of turbulence on sediment

transport can be observed on the nature of incipient motion, bed load transport and suspended load transport. Prior to the interpretation of turbulence on sediment transport, it is crucial to deduce the attributes of turbulence by observing the time series from ADV data. Besides, the presence of a hierarchy of eddies is an example of attributes of turbulence in an open channel.

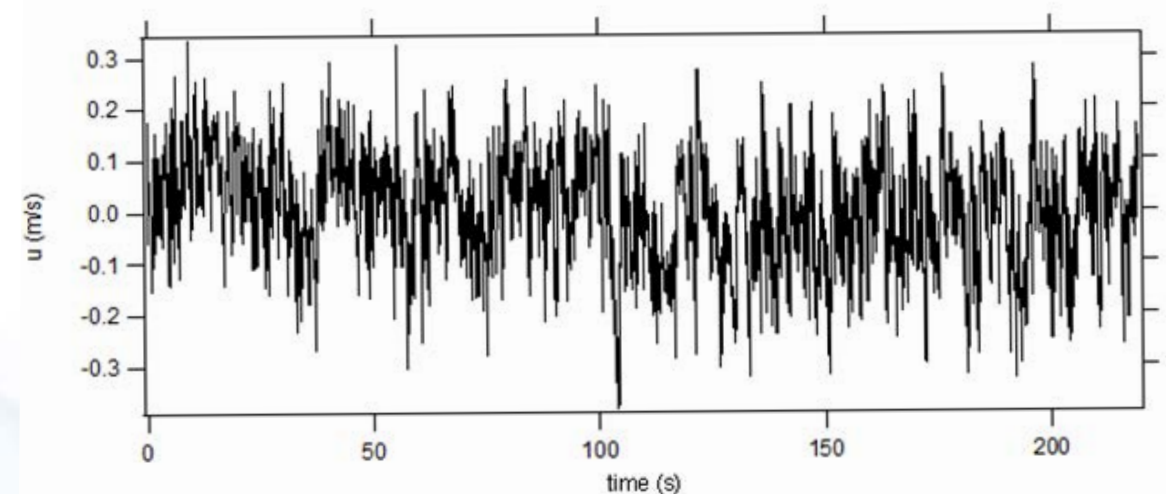
Eddy Taxonomy Methodology

Presence of turbulent structure in the form of eddies is an example of turbulence attributes in an open channel. The growth and decay in eddies is an intermittent process, where the interaction between flowing flow and its boundaries produce secondary current and high shear stress (Fox et al., 2005). Large- and small-scale eddies correspond to the slow and fast fluctuations within the time series as portrayed in Figure 2. Past researchers such as Schlichting (1968), Buffin-Belanger and Roy (1998), Roy et al. (1999), Tamburrino and Gulliver (1999), Papanicolaou and Hildale (2002) and Fox et al. (2005) has been monumental in advancing fundamental knowledge of turbulent eddies and essential development of eddy taxonomy over the past decade. The strict method of Fox et al. (2005) is applied where the moving-average time step, T_s , is identified to isolate macro-turbulence or large scale eddies via visual observation from time series and the moving-average

skew diagnostics. In ExploreV application, digital filtration process offers linear transformation in time series using the following equation:

$$u(t) = \sum_{l=-\infty}^{\infty} h_l u(t+l) \quad (9)$$

where h is weighting function and generally used to smooth the data and exclude high frequency fluctuations or exclude the low frequency trends from time series. Digital filters in ExploreV consist of three methods: low pass, high pass and band pass. The low-pass filter removes the high frequency components from the time series; the high-pass filter leaves only high frequency components in the time series and band-pass filtration allows the removal of both low and high frequency components (Nortek, 2006). The following expressions deduce on how to separate fast- or slow-fluctuation from time series data (see Table 1).



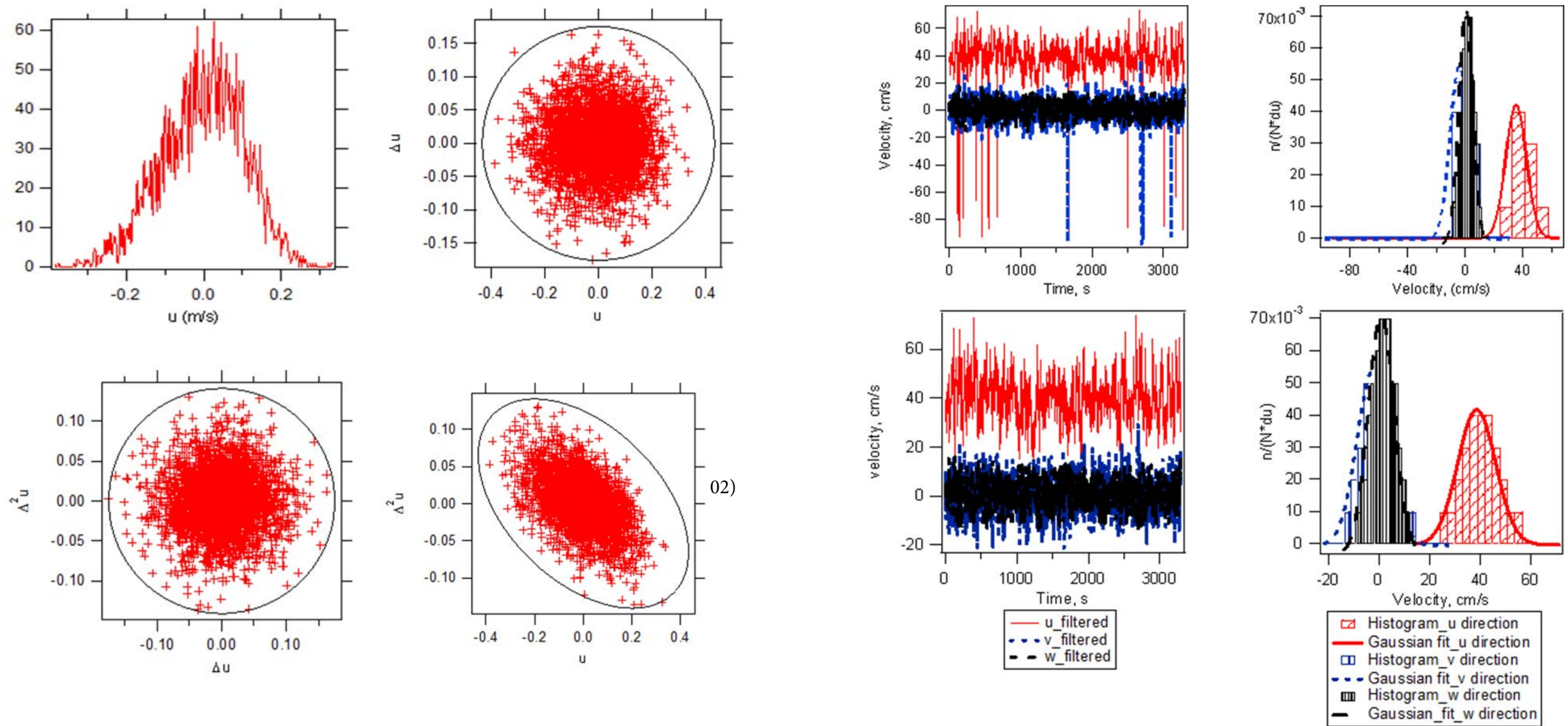


Figure 6: Histogram plot for time series before and after filtering

Table 1: Digital filtration in eddy taxonomy methodology (Nortek, 2006)

Filter type	Title	Expression
Low pass or high pass	Gaussian curve	$h_k = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(-\frac{k^2}{2\sigma^2}\right), k \leq M$ $h_k = 0, k > M$
	Turkey's filter	$h_k = \frac{1 + \cos(2\pi k/2M+1)}{2M+1}, k \leq M$ $h_k = 0, k > M$
Band pass	Bell-shaped	$h_k = \frac{\sqrt{2}}{M} \exp\left(-\frac{\pi k^2}{2M^2}\right) \cos\left(\frac{2\pi k}{M}\right), k \leq M$ $h_k = 0, k > M$

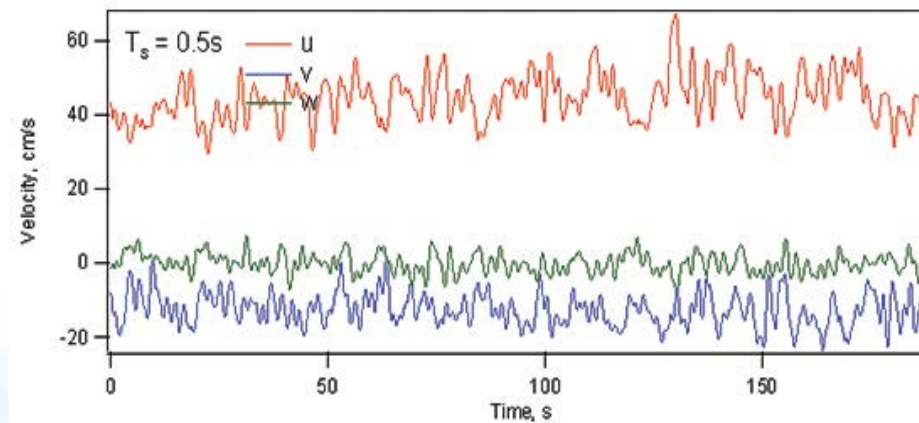
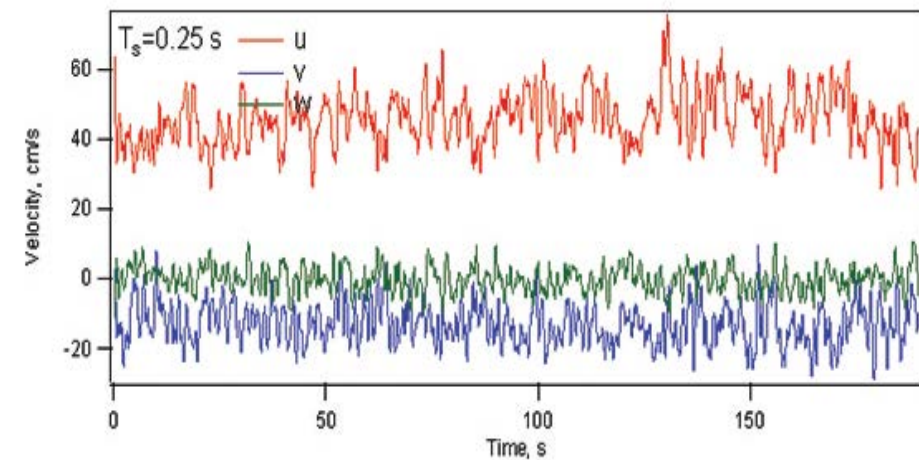
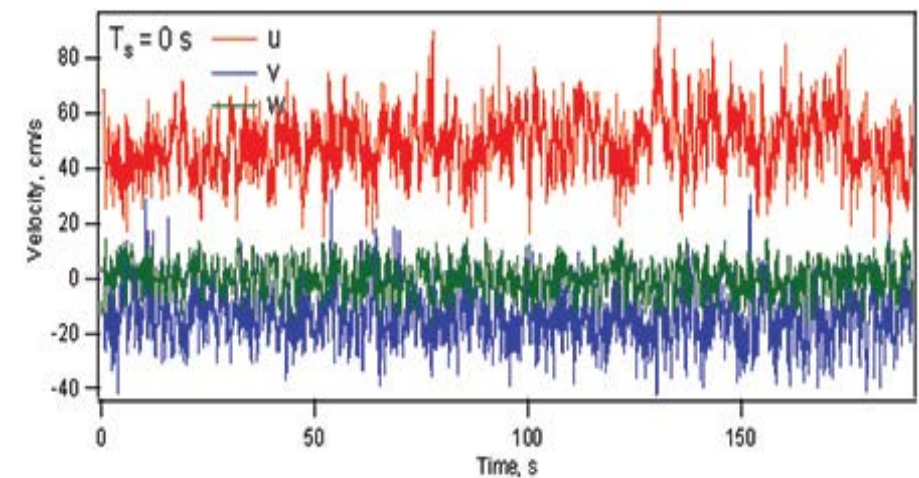
*= half width of the filter

The elimination of fast fluctuation within the time series are executed via visual interpretation of multiple slow fluctuating time series plots with successive values of moving-averaged time steps, T_s as previously demonstrated by Tamburrino and Gulliver (1999) and Fox et al. (2005). They believed that there is a physical meaning attached to T_s . Time steps, T_s is a multiple of the small scale eddy bursting period which reflects the elimination of fast fluctuations and corresponding explicitly to a multiple of the small-scale eddy periodicity (Fox et al., 2005). Figure 7(a-e) illustrates the constructed time series for point 2 at time steps $T_s = 0$ s to $T_s = 1$ s. Moving-average calculation for stream wise, lateral and vertical direction at $T_s = 0$ is the original time series prior to successive moving-average procedure. It is observed that “sharp jumps” exists at time steps $T_s = 0$ to $T_s = 0.75$. These “sharp jumps” indicate the presence of more than one small scale eddy or fast fluctuation in time series. However, as time steps reach $T_s = 1$, the time

series is “smoothed” which correspond to the small-scale eddy periodicity. This time steps ($T_s = 1$) correspond as the threshold time where the impact of small scale eddy is equally accounted for each point (Fox et al., 2005). At $T_s = 2$ s, once again the impact of small scale eddies is accounted for at each point where the moving average is calculated. The visual interpretation of defining time steps T_s for small scale periodicity is complimented via skew diagnostics as previously proposed by Fox et al. (2005). Skewness is the third statistical moment of the velocity distribution given by:

$$S_k = \frac{n}{(n-1)(n-2)} \sum_{i=1}^n \frac{(u_i - \bar{u})^3}{s^3} \quad (10)$$

where n = stream wise velocity; s = standard deviation of stream wise velocity; n = number of time series data.



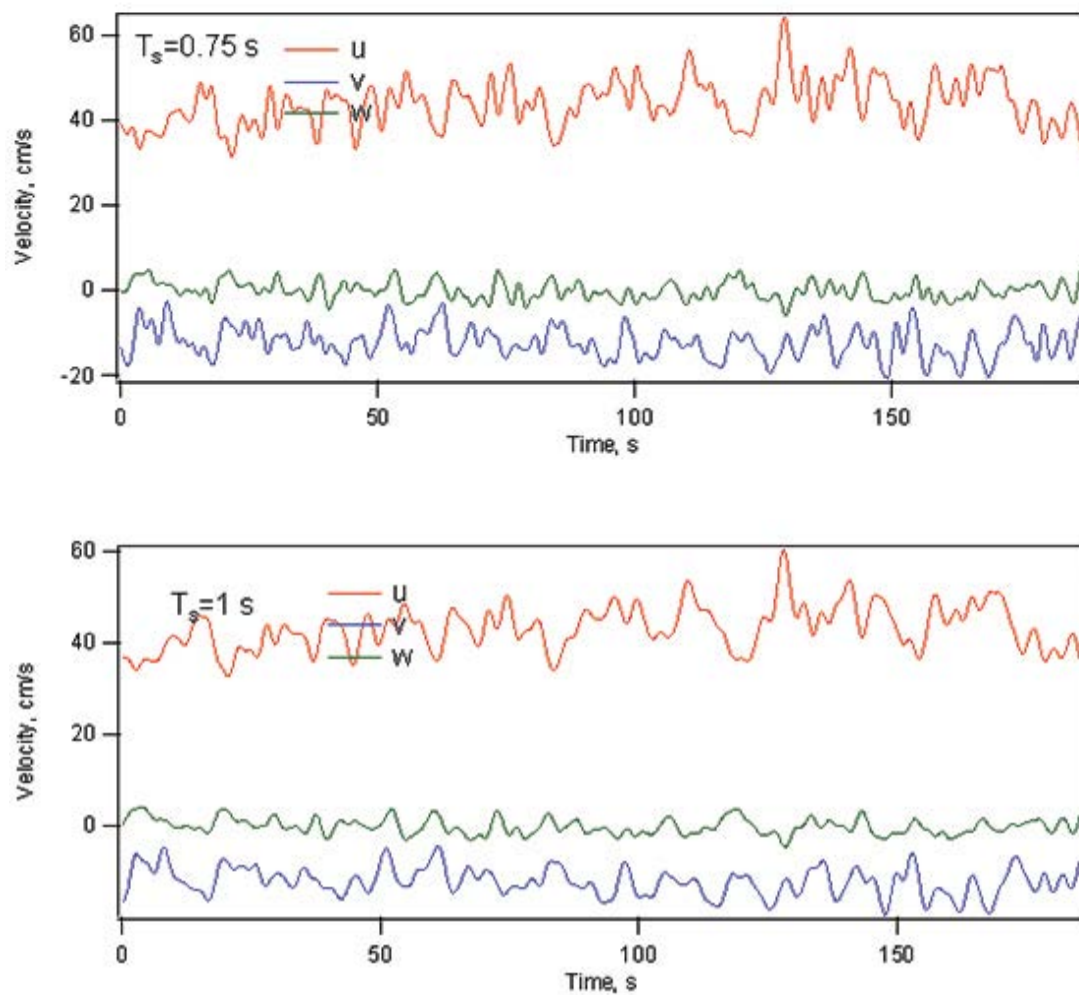


Figure 7: Moving average calculation for (a) $T_s = 0$ s; (b) $T_s = 0.25$ s; (c) $T_s = 0.5$ s; (d) $T_s = 0.75$ s and (e) $T_s = 1.0$ s.

Positive and negative skewness represents physical meaning in fluid mechanics. Buffinger and Roy (1998) deduced that negative skew for stream wise is caused by the eddies generated from ejection type process or shedding motion. Meanwhile, sweep motion is the main cause of observed positive skewness from velocity distribution. Fox et al. (2005) postulate that skew diagnostic will change as small-scale eddies are removed. As the skew value reaches a local absolute minimum or experience a significant change in slope, this suggests correspondence to the threshold T_s where fast fluctuation is

removed from the time series (Fox et al., 2005). Figure 8 shows the skew curves for point 2 & 8 as a complimentary diagnostic to determine thresholds time steps, T_s . For stream wise direction, a significant change of skew slope occurs at $T_s \sim 1$. For transverse and vertical direction, a local minimum skew value and significant change in skew slope occurs at $T_s \sim 1.25$.

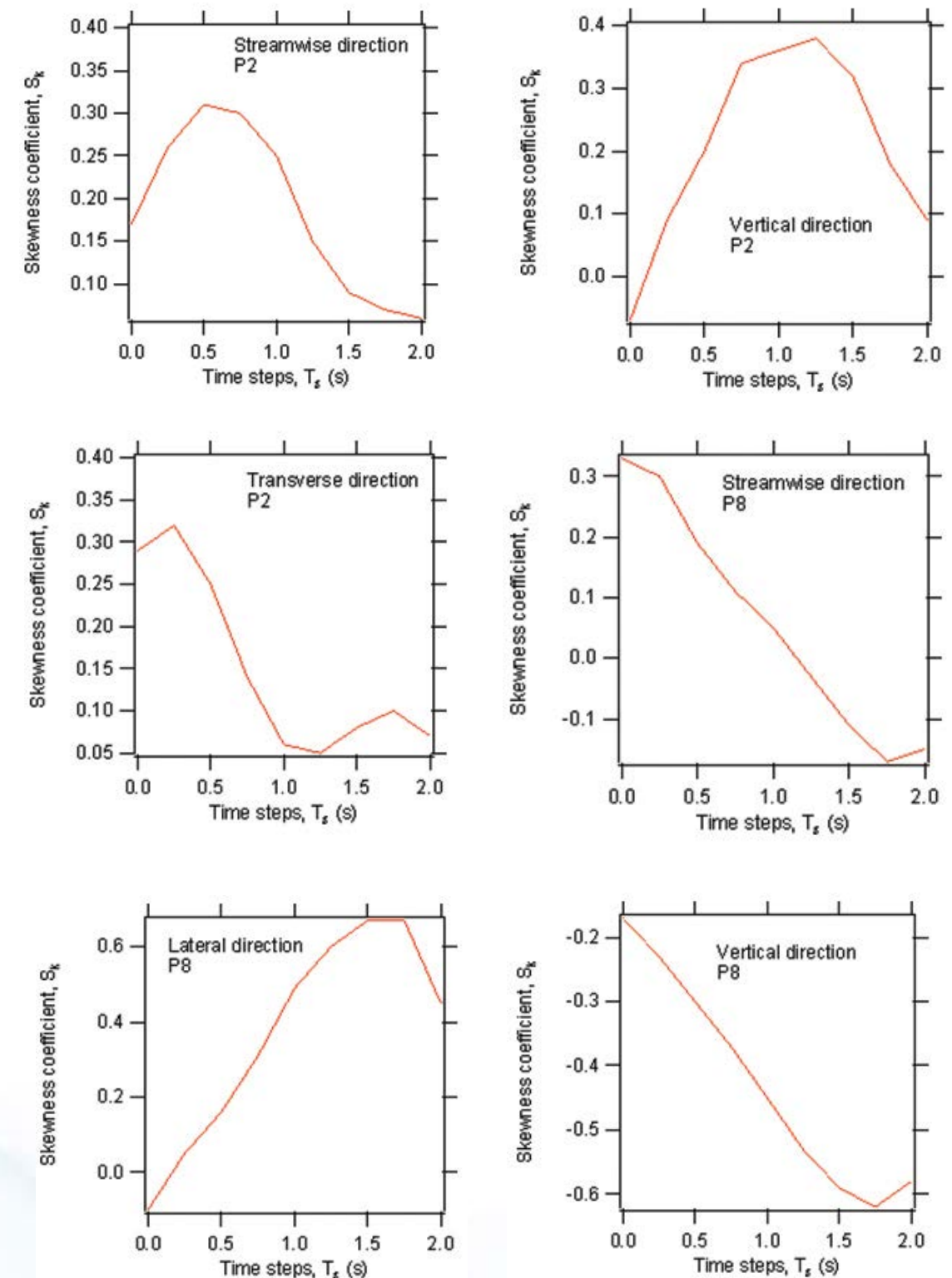


Figure 8: Local minima and changes in slope for skewness coefficient

Relationship between local bed shear stress and bed load transport

Local bed shear stress is estimated by using 5 methods namely log law method, Reynolds extrapolation, Reynolds stress near bed, TKE value near bed and TKEw' near bed. For log-law approach, local bed shear stress is back-calculated from the friction velocity as indicate in the standard log-law formulation. Friction velocity (u_*) is sought through the least square method by creating new variable x and y (K. Strom, personal communication, July 4, 2011). New variable x and y were created as follows:

$$u = u_* \left[\frac{1}{k} \ln \left(\frac{z + z_o}{e^{-kB} k_s} \right) \right] \quad (11)$$

$$y = m[x] \quad (12)$$

Table 2: Bed Shear Stress Value Estimated with Different Methods

Reach	Zone	Sampling Point	Bed Shear Stress (N/m ²)				
			Log-law	Reynolds Extrapolate	Reynolds	TKE	TKEw'
Location 1	Stoss	P1	0.4285	25.5366	-4.6708	3.7208	0.8988
		P2	0.1277	5.3158	1.5734	0.278	0.4695
		P3	0.1796	3.01	0.1098	0.2057	0.0071
		P4	0.0144	0.1859	0.0944	1.7158	0.0222
		P5	0.3881	1.8145	1.043	4.4402	0.2002
	Side	P6	0.1513	3.5	2.3294	1.5604	0.4935
		P7	0.3648	3.4259	0.8102	3.7208	0.227
		P9	0.1082	0.7152	0.4966	0.5765	0.0676
		P10	0.2958	0.7917	0.443	2.295	0.1648
		P8	0.0384	1.6522	1.0139	0.9651	0.0279
Location 2	Lee	P11	0.228	1.0476	0.6032	1.1748	0.0996
		P12	0.3386	1.2818	0.5341	1.0356	0.0938
	Downstream	P1	0.404	2.3096	0.1849	5.4498	0.1447
		P2	0.6708	2.8746	2.3261	2.4364	0.2476
		P3	0.3204	6.3437	-0.0014	0.0485	0.0009
		P4	0.8123	4.4351	1.1708	10.5194	0.2576
	Side	P5	0.2756	2.7988	2.4584	2.2226	0.2883
		P6	0.2822	7.2956	0.6861	2.5972	0.2388
		P7	0.2074	12.2373	-1.3588	3.9496	0.3097
		P9	0.1277	4.1665	3.4227	16.8325	0.2801
		P10	0.818	4.3094	-0.4361	1.5256	0.06
	Lee	P8	0.013	1.6969	0.3095	2.0723	0.2055
		P11	0.0292	6.1179	0.0012	0.0304	0.0004

where k_s is equivalent to $d_{50}; Z_0; = 0.25 d_{50}; k = 0.4$ and $B = 8.5$. The slope for straight with intercept of zero is equivalent to u_* . The same approach goes for Reynolds extrapolation method. A plot of $-\overline{pu'w'}$ vs z is made for the whole profile and a straight line is fitted to the linear portion of $-\overline{pu'w'}$. The straight line is extrapolated to y -axis ($z = 0$) to obtain the value at intercept x -axis which is equivalent to u_* . For TKEw' method, the steps are quite straightforward where the value closest to the bed is considered. This method is further expanded by considering fluctuation in vertical element, herein known as TKEw'. Sample of data for local bed shear and local bed load transport are shown in Table 2 and 3 respectively.

Table 3: Bedload Transport Data

Zone	Sampling Point	Unit Transport rate (g/ms)	
		Downstream	Upstream
		Location 1	Location 2
Stoss	P1	0	0.04327
	P2	0	0.06714
	P3	0	0.0008406
Side	P4	0.008808	0.06235
	P5	0	0.0455
	P6	0.019262	0.008553
	P7	0.012646	0.1154
	P9	0	0.0364
Lee	P10	0.001535	0.01144
	P8	0.000585	0
Downstream	P11	0	0
	P12	0	0

Table 4: Pearson correlation between local bed shear stress and unit transport rate

		Bedload_2	Loglaw_2	RevExtrap_2	Reynolds_2	TKE_2	TKEw_2
Bedload_2	Pearson Correlation	1	0.243	0.401	0.012	0.309	.677*
	Sig. (2-tailed)		0.472	0.222	0.972	0.356	0.022
	N	11	11	11	11	11	11
Loglaw_2	Pearson Correlation	0.243	1	-0.177	0.025	0.056	0.001
	Sig. (2-tailed)	0.472		0.603	0.941	0.87	0.997
	N	11	11	11	11	11	11
RevExtrap_2	Pearson Correlation	0.401	-0.177	1	-0.541	-0.106	0.063
	Sig. (2-tailed)	0.222	0.603		0.086	0.755	0.853
	N	11	11	11	11	11	11
Reynolds_2	Pearson Correlation	0.012	0.025	-0.541	1	0.562	0.447
	Sig. (2-tailed)	0.972	0.941	0.086		0.072	0.168
	N	11	11	11	11	11	11
TKE_2	Pearson Correlation	0.309	0.056	-0.106	0.562	1	0.514
	Sig. (2-tailed)	0.356	0.87	0.755	0.072		0.106
	N	11	11	11	11	11	11
TKEw_2	Pearson Correlation	.677*	0.001	0.063	0.447	0.514	1
	Sig. (2-tailed)	0.022	0.997	0.853	0.168	0.106	
	N	11	11	11	11	11	11

*. Correlation is significant at the 0.05 level (2-tailed).

Those information (Table 2 and 3) are crucial to study the inter-relationship between local bed shear and bed load transport. Bed shear stress initiate particle motion in the river system. Pearson correlation is fitted between local bed shear (from Table 2) with unit transport rate (from Table 3).

Table 4 clearly depicted that local bed shear stress using turbulent kinetic energy (TKE) at vertical flow (w direction) is having high correlation with bed load transport rate (0.677). Figure 9 justified that the increasing or decreasing of transport rate at different point location proportionate with the increasing or decreasing of local bed shear.

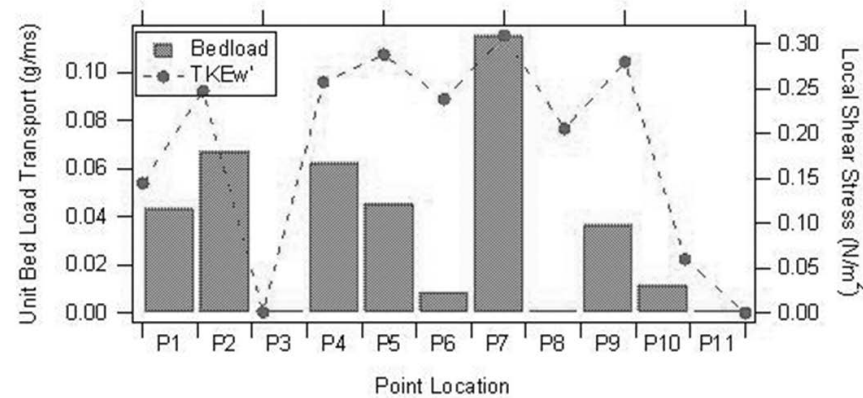


Figure 9: Relationship between local bed shear and unit transport rate

In general, this paper provides brief technical view of application of Doppler system at measuring flow and turbulent data. Prior to Reynolds decomposition, the ADV data must be transformed into Cartesian unit for filtering and despiking purpose. The Nikora and Goring (2002) method is successfully adopted to eliminate noise and aliasing due to contaminated pulse between outgoing and incoming signal. The statistical analysis of Shapiro-Wilks information and histogram plot provide useful indicators for ADV performance prior to Reynolds decomposition. The turbulent statistics on intensity, kinetic energy and Reynolds stress are obtained using Reynolds-Averaged Navier Stokes (RANS) formulation. Besides turbulent statistics, elimination of fast fluctuation or small eddies from time series are shown in this paper via eddy taxonomy

approach. The small-scale eddy periodicities are sought through time steps moving-average calculation. The threshold for small-scale eddy periodicity is complemented through the observation on skewness coefficient values. It is observed that at the location without any obstruction (e.g; P2), the small-scale eddy periodicity occurs faster than the location at the lee of obstruction element (e.g: P8). As a general conclusion, the usage of Doppler system in capturing fine scale turbulence statistics is successfully demonstrated in this paper. In addition, the relationship between local bed shear generated from ADV data and the actual bed load data can be illustrated successfully with the help of correlation analysis and graphical relationship. Thus, cleaned ADV data are helpful towards solving flow and sedimentation study in fluvial system.

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A PHYSICAL MODELLING OF WAVE TRANSMISSION DUE TO LOW DENSITY POROUS SUBMERGED BREAKWATER

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ABSTRACT

Wave transmission due to low density porous submerged breakwater was studied through a laboratory scale modelling. The purpose of the invention of low density porous submerged breakwater is to exploit the eco-friendly material for the construction of breakwater at lower cost as well as to dissipate energy under varying wave conditions. The submerged breakwater units were provided by the National Hydraulics Research Institute of Malaysia (NAHRIM) and series of tests were performed at the Hydraulic Laboratory of Civil Engineering Department, Universiti Putra Malaysia (UPM). Tests taken were varied from lower to higher wave heights, shallower to deeper water depths, and shorter to longer wave periods. The breakwater is setup at a formation of 3(base):2 (middle):1(crest) and the model structure was placed at the middle of the wave flume. To increase the stability of the breakwater, the model units were tied up simultaneously. A total of eight wave probes were installed, four at the front and another four at the back of the breakwater to capture the incident and transmitted wave heights, respectively. Results showed that high wave steepness contributes to the lower transmission coefficient (K_t) and higher relative freeboard causes the K_t to increase.

Keywords: Low density porous submerged breakwater, Wave Breaker Coral Restoration, wave steepness, wave transmission

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Introduction

Breakwaters or groynes usually serve their function of protecting land from erosion and enabling safe navigation into harbours and marinas. Breakwaters can be permanently submerged, permanently exposed or intertidal. In each case, the depth, size and position of the structure relatively to the shoreline determine the coastal protection level provided by the structure (Taveira-Pinto, 2005). Malaysian coast has suffered coastal erosion since past. According to National Coastal Erosion Study report, coastal erosion in Malaysia is a major concern rather than economic problem. Coastal erosion has resulted in damage and loss of agriculture land, mangrove forests, houses, roads and recreation beaches. Along the west coast of Peninsular Malaysia, most of the agricultural land is seriously threatened. Tanjung Piai, Johor, for example was suffered intensive erosion due to strong current with a speed of 0.5 m/s to 0.8 m/s. Nor Aslinda & Wan Hasliza (2014) stated that the local current pattern is formed near the tip due to the presence of a small island as the flow entering the study area during flooding were reflected to the east while the flow during the ebb tide tend to move toward the opposite direction. Some parts of Malaysian coastline are protected by hard engineering structures. Along this seaside lines, a few breakwaters were developed to diminish the shoreline disintegration. In Terengganu for example, three submerged breakwaters were built in Kerteh Bay that concealed to 2100m long. In other places, the local newspaper reported that breakwaters were constructed in Kampung Pasir Pandak, Kampung Buntal and Kampung Santubong, Sarawak in year 2004 and at Kampung Pasir Panjang in year 2007. The construction of breakwaters in Malaysia has shown the importance and needs of structural protection to protect our shore from continuous erosion. In

Malaysia, progressive researches through several agencies and universities have been made to study the effectiveness of artificial coastal structural protections in protecting the Malaysian coast. Universiti Teknologi Malaysia is pioneered to conduct a research on the various mangrove structures through the artificial root grip (Eldina et al., 2005). On the other hand, Jabatan Perikanan Malaysia (JPM) and Lembaga Kemajuan Ikan Malaysia (LKIM) were developed various artificial reefs and there were classified as giant structures that can be up to 3.5 m height with the weight of up to 42 metric tons. The recent development of coastal structural protection in Malaysia was conducted by the National Hydraulics Research Institute of Malaysia (NAHRIM) through the development of Wave Breaker Coral Restoration (WABCORE). The invention of WABCORE was patented under Reka bentuk Industri dengan Perbadanan Harta Intelek Malaysia (MyIPO), reference number 12-00200-101. WABCORE has a characteristic of submerged breakwater that functioning as an erosion control structure and beach stabilization with wave breaking as well as an artificial concrete reef, a luxury home for aquatic life.

Wave Breaker Coral Restoration (WABCORE)

WABCORE is a composite system unit developed by National Hydraulics Research Institute Malaysia (NAHRIM). Pre-fabricated concrete structures are stacked together to form a simple trapezoidal design which then called as one unit. The WABCORE model is arranged in manner of bottom up with wide bottom and narrow hat of 4, 3, 2 and 1 units respectively from the bottom, second and third middle layers followed by the top hat and in narrow crested shape. Mass weight of the prototype structure is

371.4 kg; therefore, model weight has to be at 5.8 kg complying the Froude scaling for mass weight of 1: x3. Detailed drawing was then produced and certified to comply with the geometric method of similarity with the base width, crest width and structural height are 200 mm, 100 mm and 125 mm, respectively. The first prototype model was successfully constructed at Pulau Tioman in Endau-Rompin, Pahang and Pulau Tinggi in Johor (Safari et al., 2016; Fauzi et al., 2013). The positive performance of WABCORE for wave transmission when subjected to different sea state conditions such as water depths (m), wave periods (s), incident wave height (m) and wave length (m), indicated high wave suppression and capability of WABCORE that promotes wave breaking and structural friction to wave motion. Previous studies (Safari et al., 2016; Safari, M.D. 2017) showed that the smallest transmission coefficient is lowest when the water depth is the smallest. The value increased as the water depth became larger. For the effect of incident wave height, it was concluded that the smallest water depth and small amplitude of significant wave height, the value of wave transmission coefficient is high. On the other hand, when the smallest value of water depth with bigger wave height, the value of wave transmission coefficient is reduced which shows that incident wave height plays a role in wave dissipation. High wave period which resulted into longer wavelength leads to smaller transmission coefficient due to potential of wave passing over the structure. Transmission coefficient become smaller when the value of H/L at 0.0889 which shows that the wave period is short. Transmission coefficient increased when the h/d at 0.5 and 0.4. The effect of wave period become major factor when water depth is at structural crest level. Physical sea state condition such as water depth and incident wave height shows correlation to the

performance of WABCORE. For coral restoration, observation of deployed WABCORE at site recorded coral growth increased rapidly from 8.66% to 29.75% from the whole surface of WABCORE in three years (Fauzi et al., 2013).

Problem and purpose of study

The stability of the present WABCORE system is governed by the heavy self-weight of the breakwater unit. The present model was developed by using steel as connection and high-density concrete. Therefore, the model construction cost is high due to its material. Realizing the importance of preservation of environment as well as to cater for the sustainable green construction, a new version model of WABCORE was developed. In order to reduce the cost, the present WABCORE model was modified by using concrete with reduced density and fibre as material without using any steel as connection. The model dimension was kept constant similar to the present model. The concrete density was reduced to 1600 kg/m³, thus reducing its model weight to merely 3.5 kg each unit. Froude scaling (1: x3) is used to obtain the model weight relative to prototype weight with ratio 1 to 4. The present study explores the performance of the new WABCORE model in dissipating wave energy via physical modelling. The results presented in this paper is only part of physical modelling works done. The paper is organized by introducing the needs for a new submerged breakwater for structural coastal protection in Malaysia. The literature review discusses, in general, the role of submerged breakwaters in dissipating energy. The physical modelling work is presented in research methodology with the experimental setup of the laboratory works and test scenarios. In results and discussion, the physical modelling results are presented in term of the effect of wave steepness and relative freeboard on wave

transmission coefficient and discussion on how these parameters affect the transmission of waves are elaborated. Conclusion summarized the overall work done up to date and outputs in general as well as future works.

Literature Review

The general function of submerged breakwater

Breakwaters are used around the globe as a mean of protection against beach erosion by reducing the height of incoming waves into smaller transmitted waves. Based on Lian et al., (2015), submerged breakwaters are commonly built in coastal areas to protect vessels, facilities, and coastal buildings against wave erosion. Due to the ability of submerged breakwaters to debilitate waves, they have great demand in coastal areas, especially in regions of tourist attraction. Thus, it is important to investigate the wave transformation processes and its dynamic characteristics around submerged breakwaters. In addition, breakwaters can efficiently be used to preserve tranquillity inside the harbour basin, to decrease the deposition of sediments that can lead to water pollution at the harbour entrances, and to be a sanctuary for marine habitats. There are several types of submerged breakwater and the most conventional one is the rubble mound structures. Rubble mound structures are the oldest method where it can be traced back to 2000 BC. Takahashi (1994) as cited in d'Angremond and Roode (2004) claimed that the stone masonry structure were discovered in Alexandria, Egypt. As the submerged breakwaters act to absorb some of the incoming wave energy by causing the wave to break prematurely thus diminishing the transmitted wave energy, these structures are viable to be constructed

as the coastal protective function without spoiling a landscape. In addition, being submerged, they are more aesthetic than the exposed or emerged breakwaters.

Wave transmission coefficient

Numerous studies were done on submerged breakwaters, looking at various degree of inspections. Liao et al., (2013) studied on wave breaking criteria and energy loss caused by a submerged porous breakwater on horizontal bottom. Dattatri et al., (1978) showed that wave transmission is mainly affected by the structure crest width (B) and the freeboard of the structure below the sea surface (R). The studies from Van der Meer et al., (2005), D'Angremond et al., (1996) and Seabrook and Hall (1999) have resulted in some experimental formula for transmission coefficients (KT). Furthermore, Vander Meer et al., (2005) developed a transmission coefficient formula from previous experimental tests for waves passing a low crested structure with a wide range of incident wave conditions and structure geometry conditions. Rojanakamthorn et al., (1990) derived a modified mild-slope equation for modelling wave breaking on a submerged permeable breakwater, and a breaking index was used to be a criterion to find the incipient breaking point of breaking waves traveling over a permeable structure. Wave transmission coefficients have been used in most design considerations in constructing a submerged breakwater. For examples, Shirlal et al., (2007) suggested that a submerged structure is constructed at a water depth of 1.5 -5.0 m with a front slope of 1:2-1:3 and a height exceeding

0.7 times the water depth. On the other hand, submerged structures are designed generally for a KT value of 0.6 (Goda, 1996). Shore Protection Manual, SPM (1984) stated that wave transmission coefficient can be simply described as the ratio of transmitted wave height to incident wave height, as given in equation 1 i.e.,

$$K_T = \frac{H_t}{H_i} \quad (1)$$

The stability of breakwater

The stability of a rubble mound breakwater is usually described by the "zero damage" design sea state, allowing some small armour movements if they do not exceed some limits. According to a number of regular wave tests with no overtopping, that stability is a function of the dimensionless Hudson stability number. Apart from Hudson stability number, there are many other methods to determine the stability of the submerged breakwater depending on the characteristic of structures and the waves. A study performed by Kiran A.S et al. (2015), the stability of sand filled geo-synthetic tubes applied as submerged breakwater can be determined by using equations derived by Pilarczyk (2000) and Kriel (2012). Furthermore, study by Matsumoto et al., (2011) found that the method to find stability is based on a wave induced flow field around a breakwater, which is calculated by a numerical wave flume CADMAS-SURF (Isobe et al., 1999), and successive wave force calculation based on the Morison formula. Muttray (2012) in his study used observation to determine the structural stability where the rocking of the armour part and the displacement of the structures are observed. It is noted that the stability of the submerged breakwater of this study is not covered in this paper.

Innovation of submerged breakwater

An innovation of the submerged breakwater using the concrete prefabricated submerged structures was presented in Tamrin et al., 2014. These units are useful in areas where natural rocks are not readily available and can be less costly than a rubble mound structure (Morang et al., 2014). In designing a conventional breakwater, the armour layer which is supported by the core, the under layers and the toe protection is an important consideration. The outer armour slope consists of a single or double layer of armour units, depending on type of armour units (rocks or concrete units) and the placement method (randomly or interlocking). The type of armour units can be made of rocks, cubes, tetrapods, accropods, corelocks, or xblocks (Van Rijn, 2016). The first three-unit types have no interlocking system where units are either randomly or orderly placed, the tetrapods are randomly placed but has fair interlocking system. The last three units have high interlocking system and units are strictly patterned. Another type of submerged prefabricated breakwater is made up of Reef Ball units (Barber and Barber, 1996) deployed near the beach. Reef Balls come in various size and weights and have holes in half-ball-shaped concrete shells. They are designed to serve dual function, as a wave attenuator and an artificial reef. Studies related to reef ball units have been made by several authors e.g. Del Vita (2016), Harris (2003), Harris (2009), Arnouil (2008), Armano and Hall (2003) and others. In Malaysia, research and innovation in submerged breakwaters is progressing. The design and development of Artificial Mangrove Root System (ArMS) submerged breakwater (Fatimah et al., 2005) marks the initial invention of submerged breakwater in Malaysia, to the knowledge of the author. The ArMS model can act as protection

to the mangrove seedlings by allowing sedimentation build-up in soft muddy coast due to its performance in dissipating wave energy (Hadibah et al., 2005). The numerical model was developed to investigate the capability of the ArMS structure as a wetland habitat protector (Fatimah et al., 2008). The application of geotextile tube as a submerged break-water is evidence at Teluk Kalung and Pantai Buruk, on the east coast of Malaysia. This imported technology had improved the beach conditions at the studied areas. The significant quantities of sand had accumulated, increasing the beach width as the shoreline had migrated seawards (Lee and Douglass, 2012). The recent innovation of submerged breakwater in Malaysia is WABCORE model. As previously mentioned in Introduction section, the first prototype model was successfully constructed at Pulau Tioman in Endau-Rompin Pahang and Pulau Tinggi in Johor (Safari et al., (2016) and Fauzi et al., (2013)). The brief description of innovation of submerged breakwater can be found in Ibrahim et.al (2018).

Experimental Methodology

The experiment was conducted in the Hydraulic Lab of Civil Engineering Department, UPM. The WABCORE model was provided and supplied by National Hydraulics Research Institute (NAHRIM). The WABCORE was stacked in the pyramid shape where the width base is made up of three models and followed by two in the middle, and one model as the top crest as seen in Figure 1. The total height of the structure is 0.38 m.



Figure 1: The formation of the WABCORE inside the wave flume

The setup of this experiment is illustrated in Figure 2. The model was placed in the middle of the wave flume. Eight wave probes were used for this experiment i.e. four is placed in front of the model while another four is placed behind the breakwater model to capture the incident and transmitted wave heights, respectively. Wave absorber was placed at the far end of the wave flume to dampen the transmitted wave and to absorb dissipation of wave energy so that it behaves like a natural beach. The generation of a series of regular waves with specified heights and periods was done using a wave generator equipped within the wave flume. Calibration setup prior to model testing was conducted to ensure the accuracy of the results obtained. The model was tightened with cable ties to imitate the grouting process in real application as showed in Figure 3.

To determine the wave coefficient transmission, six waves with different heights ($H_s = 0.05\text{m}$, 0.1m , 0.15m , 0.2m , 0.25m , 0.30m) were generated and the freeboard were varied ($R = 0.02\text{m}$, 0.07m , 0.12m , 0.17m , 0.22m). In addition, three wave periods (1.43s , 1.68s and 2.0s) were also tested as parameters. Transmission coefficient, K_t can be determined using equation 1. Two types of preliminary

analyses were performed based on the study by Dattatri et al., (1978) which are: (i) the effect of wave steepness on wave transmission; and (ii) the effect of relative

freeboard on wave transmission. Experimental results are compared with the modelled results where the model is developed based on the linear wave theory.

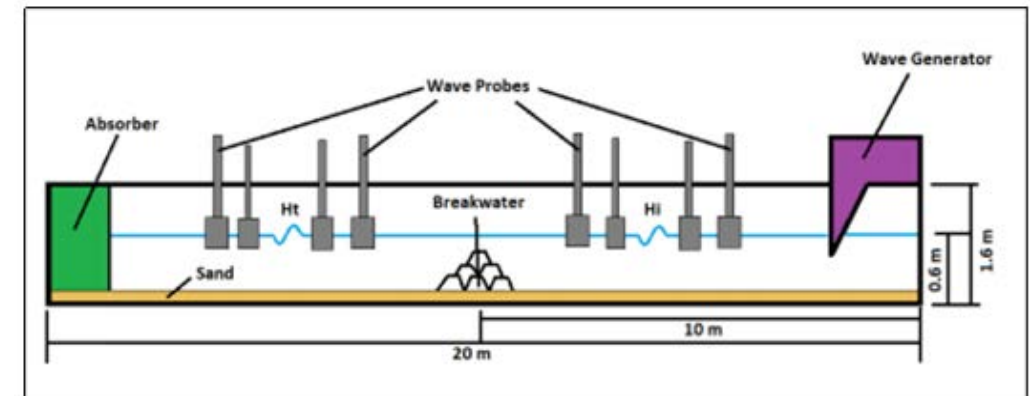


Figure 2: The experimental setup of the experiments



Figure 3: WABCORE model structured by cable ties for stability purpose

Results and Discussions

Effect of dimensionless wave steepness (H_i/L) on wave transmissions coefficient (K_t)

Figure 4 shows the effect of wave steepness on K_t . For $T_p = 2.0\text{s}$, the highest K_t is 1.03 when H_i/L is 0.048 and $R = 0.12\text{m}$. The lowest K_t is 0.35 when H_i/L is 0.008 and $R = 0.02\text{m}$. For $T_p = 1.65\text{s}$, the highest K_t is 0.925 when H_i/L is 0.023 and $R = 0.17\text{m}$ while the lowest K_t is 0.5 when H_i/L is 0.011

and $R = 0.02\text{m}$. For $T_p = 1.43\text{s}$, the highest K_t is 1.04 when H_i/L is 0.02 and $R = 0.02\text{m}$ while the lowest K_t is 0.41 when H_i/L is 0.02 and $R = 0.02\text{m}$.

Figure 4(a) shows the relationship between the wave steepness and K_t for $T_p = 2.0\text{s}$ followed by 1.65s (Figure 4(b)) and 1.43s (Figure 4(c)). Generally, it can be seen that as wave steepness increases, the wave transmissions are decreased. This is due to the higher wave steepness contain lower energy compared to lower wave steepness.

Hence wave energy is easily dissipated when wave is steepened. The results from the study by Dattatri et al. (1978) show similar trend as compared to results presented in this paper. Authors claimed that the effect of wave steepness on K_t is insignificant for the range of higher wave steepness due to the higher wave steepness seems to merge into one. This can be seen in Figure 4(b) for H_i/L vs K_t for 1.68s wave period where the points are merging. Apart from that, study by Hamdani et al. (2015) on wave transmission on submerged breakwater with interlocking D-block armor also shows similar trend for effect of wave steepness on K_t . Authors concluded that the higher the value of wave steepness, the lower the K_t is. Moreover, there is also a study on the wave transmission on submerged breakwaters made of hollow hemispherical shape artificial by Armono and Hall (2003). A qualitative parametric analysis was performed to find the effects of the external and dimensional variables on the wave transmission through HSAR breakwaters. For relationship between K_t and wave steepness, authors concluded that the higher the wave steepness, the lower the K_t . Based on those studies, it can be concluded that similar curve is obtained, in agreement with the results obtained from this study.

Effect of dimensionless freeboard on wave transmissions coefficient (K_t)

Figure 5 shows the effect of relative freeboard on K_t . For $T_p = 2.0$ s, the highest K_t is 1.03 when H_i/L is 0.048 and $R = 0.12$ m. The lowest K_t is 0.35 when H_i/L is 0.008 and $R = 0.02$ m. For $T_p = 1.65$ s, the highest K_t is 0.925 when H_i/L is 0.023 and $R = 0.17$ m while the lowest K_t is 0.5 when H_i/L is 0.011 and $R = 0.02$ m. For $T_p = 1.43$ s, the highest K_t is 1.04 when H_i/L is 0.02 and $R = 0.02$ m while the lowest K_t is 0.41 when H_i/L is 0.02 and $R = 0.02$ m.

Figure 5 (lower) shows the relationship between relative freeboard and K_t for $T_p = 2.0$ s followed by 1.68s Figure 5 (upper) and 1.43s Figure 5 (middle). Based on the results, it can be seen that as relative freeboard increases, the wave transmissions is also increased. This is because the high relative freeboard carries more energy within it so the amount of energy dissipated is lower compared to the lower relative freeboard. Hence wave energy is easily dissipated when the relative freeboard is low. Based on previous study by Seabrook and Hall (1999), the parameters that highly influenced the K_t is d_s (freeboard), incident wave height (H_i) and crest width (B). The results on the relationship between K_t and d_s by the authors showed that the higher the freeboard (d_s), the higher the K_t . In addition, study by Yulastuti and Hashim (2015) on wave transmission over submerged rubble mound breakwater using L-blocks shows the relationship between freeboard and wave transmission. They mentioned that the higher the freeboard, the higher the wave transmission. Authors added that when the relative freeboard is greater than 40%, the efficiency of the wave attenuation is decreased.

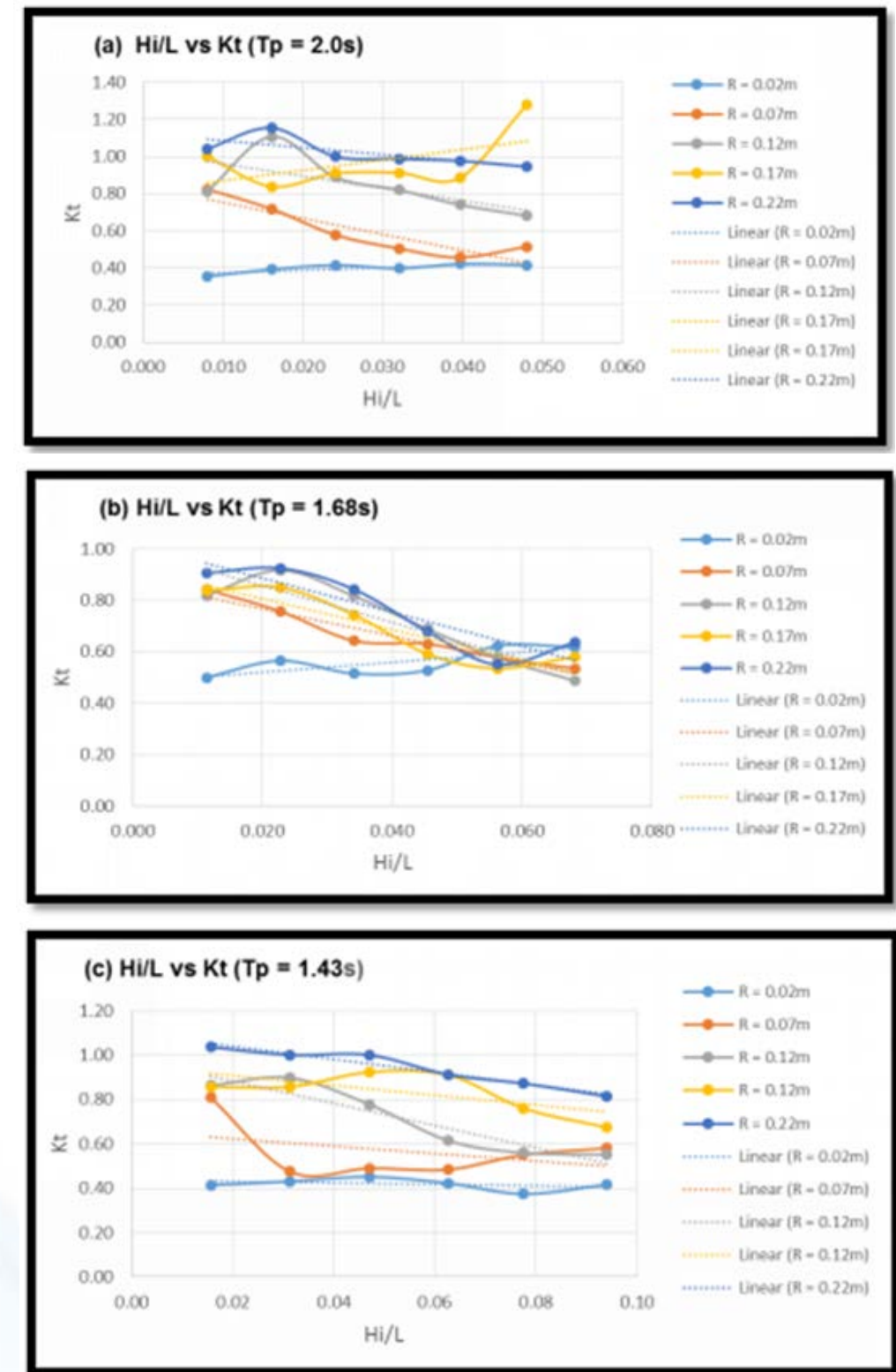


Figure 4: Dimensionless wave steepness versus wave transmission coefficient (K_t). General pattern can be observed where increasing wave steepness leads to decreasing wave transmission coefficient.

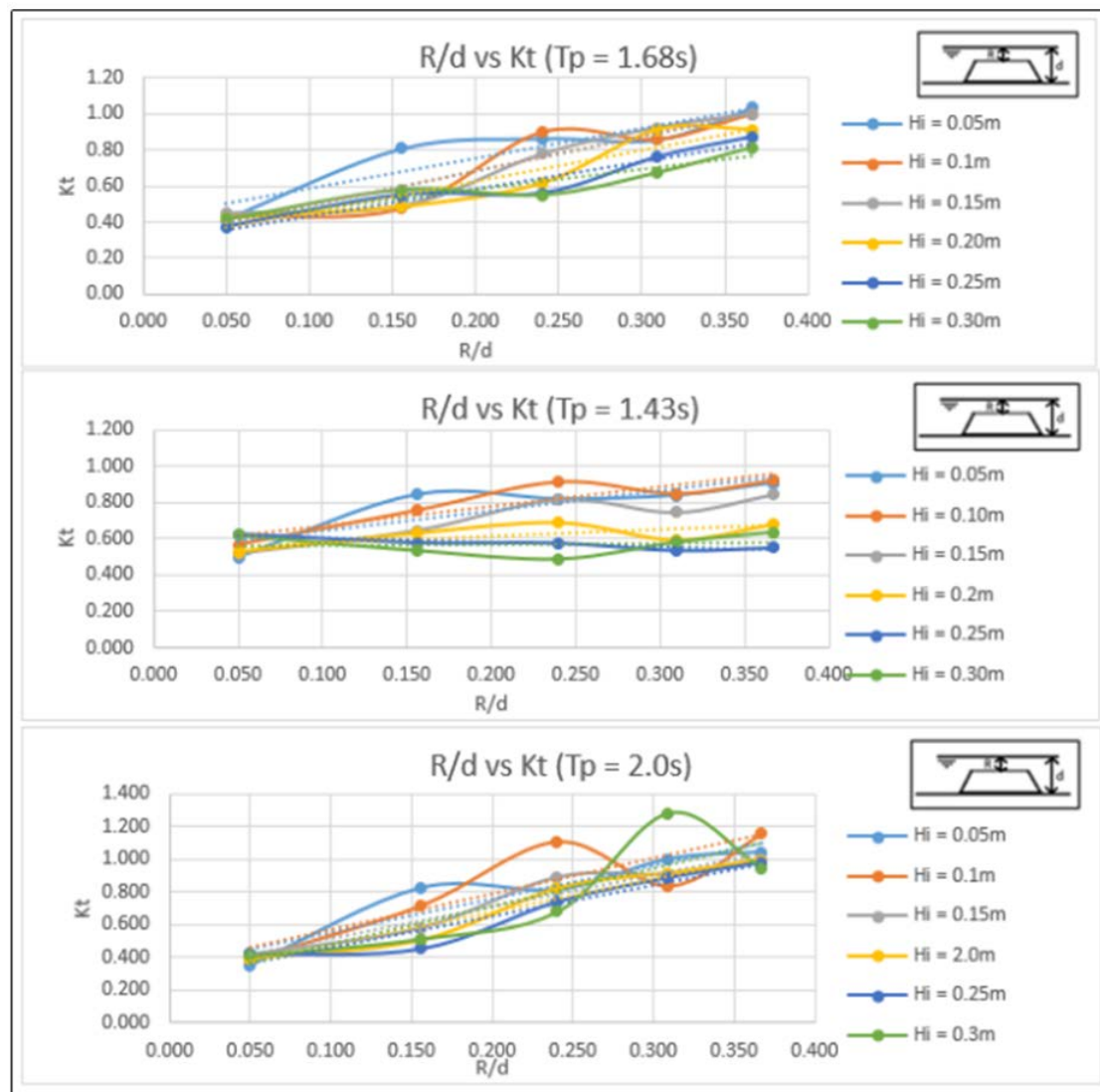


Figure 5: Relative freeboard vs wave transmissions coefficient (Kt). General pattern can be observed where increasing relative freeboard leads to increasing wave transmission coefficient.

Wave profile comparison between measurement and model

The comparison between measured wave profiles and modelled wave profiles is presented in Figure 6. Figure 6 shows six comparison results for different wave heights (a) $H_s = 0.05\text{m}$ (b) $H_s = 0.10\text{m}$ (c) $H_s = 0.15\text{m}$ (d) $H_s = 0.20\text{m}$ (e) $H_s = 0.25\text{m}$ (f) $H_s = 0.30\text{m}$ and wave period of 2.0 sec. The water level is constant at 0.4 m depth. This represents the case of freeboard 0.02 m.

The analytical model is developed based on the linear wave theory and breaking depth coefficient is set at 0.8, commonly applied for regular and stationary wave. Shoaling and refraction processes of wave are considered, but reflection of waves is neglected. An algorithm of the mathematical model is generated using a MATLAB programming language.

The simulated profiles show the smooth pattern of propagated and transmitted waves before and after the structure, respectively. Breaking of wave were observed as the wave approaches the breakwater and the process is evidenced at the crest of the structure. As energy releases, wave height decreases behind the breakwater. The trend of propagated and transmitted waves is observed in the physical experiment. Higher incident waves are measured by the wave

probes as recorded in wave probes 1,2,3 and 4. However, in all cases, wave probe 2 seems to give unacceptable result. Perhaps, this is caused by the malfunction of wave probe 2. Further investigation is required to justify the problem. On the other hand, lower transmitted waves are measured by wave probes 5,6,7,8. Comparing data between measurement and simulation, confidence is gained more on the measured data than simulated data.

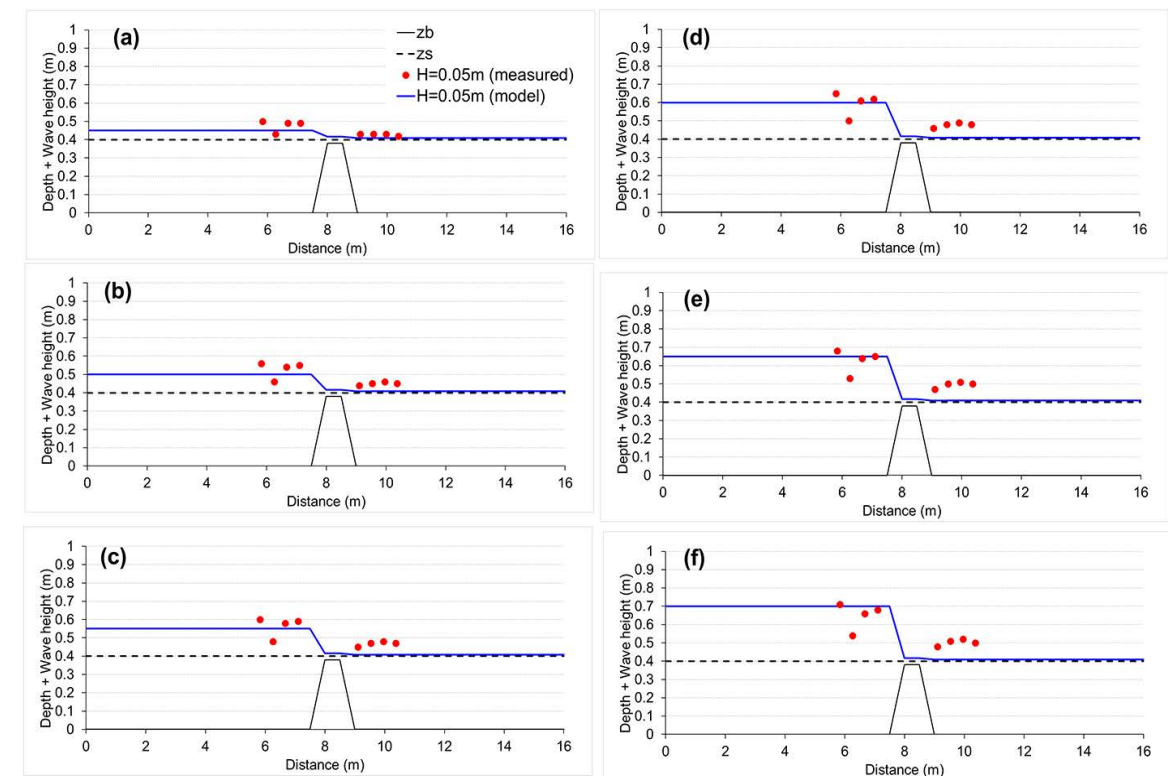


Figure 6: Wave and water level profiles between measurement and model for selected cases. Measured profiles are slightly higher than modelled profiles. Modelled profiles exhibit smooth lines as model is linear and stationary, neglecting the effect of reflection.

Conclusion and Recommendations

A study on physical modelling of wave transmission coefficient due to low density submerged breakwater was investigated. The submerged breakwater of WABCORE was placed in the middle of the wave flume with the arrangement of 3 (base):2 (middle):1

(crest). Two main parameters i.e. varying freeboards and wave steepness were investigated to observe the performance of the submerged breakwater in transmitting the wave height. Wave profile comparison between the simulated and measured data was done. The results from the experiments

confirm the objectives of this study. Higher wave steepness contributes to the lower K_t value and higher relative freeboard causes the K_t to increase. Based on the previous studies, similar trends are achieved. The irregularities in the results are probably caused mainly by the reflection. Further study will be continued to investigate the effect of varying crest widths and the stability of the structure.

Acknowledgements

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PERFORATED CYLINDER AS A POTENTIAL ARTIFICIAL REEFS FOR PROTECTION, MITIGATION, AND RESTORATION

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ABSTRACT

Arrays of perforated cylinders (PC) can be used as a wave breaker, an alternative system to conventional breakwaters or as an artificial reefs system. The PC has potential to act as a wave dampening system specially to ensure mild climates for mangrove sapling growth and sustainability, beach erosion attenuator and/or as a fish breeding ground. In this paper, an experimental test to study the parameters influencing wave transmission through the submerged PC is presented. Two sizes of cylinder were used, 100 mm and 200 mm diameters with a range of porosities, ε between 6.25% to 48%. The experiment was tested for one and two rows model configurations. The influences of submergence depth (y/d), wave steepness (H/L), relative width (W/d), relative spacing (B/d) and porosity (ε) on wave transmission was studied and mathematical equation was developed. Test results indicated a decrease in wave transmission as the wave length decreases. Whilst submergence depth plays a significant factor in controlling the wave energy losses with the loss coefficient increases as water level drop. And the magnitude of losses increased further when tested with wider cylinder with lower porosity ($\varepsilon = 6.25\%$ and 14%). Wider cylinder showed promising performance in reducing wave transmission than smaller model.

Keywords: Cylinder; experimental investigation; submergence depth; porosity; wave transmission

Introduction

Malaysia in general is surrounded by sea with beaches in the east and west coasts of Peninsular Malaysia and East Malaysia are known among the world's top tourist destinations for its wide white sands. Almost 90% of the Malaysian coasts is sandy beach which prone to erosion with 91% and 76% being along east coast of Peninsular and east coast of Sarawak (EPU, 1985) respectively. However, amid the coastal zone managements, guidelines and studies overseen by decision makers, respective authorities and researchers, the degrees of erosion outmaneuvered the end results of careful implementations and mitigations often overrules the expectations of coastal populace in saving their beaches. More than often the mitigation options are not what is expected but excepted as a way to impede the ongoing eroded forces, the waves.

Waves are the main force to be reckoned when dealings with the appropriate coastal defence to be proposed. The joint force of winds which resulted in storm surges and sea level rise will likely exaggerate the risk of structural failure if the design is compromised by ignoring these climate parameters. As a solution, higher crest and bigger boulders are used as the coastal defence material which is what most coastal engineers in general are normally opt for as the structural forefront materials. However, to cater for the short supply of hard rock preferably granite which comes in higher Malaysian ringgit, more and more alternative approach is opted to balance out the shortage. Among others is using concrete blocks such as tetrapod, accropode, CORE-LOC, reef ball, perforated pile, NEXC block etc. or other composite materials such as geotextile tube, concrete block with steel-fence-baskets, sea gabions etc. Over

the years, with coastal climatology getting rougher and frequent, the main objective of a protection structure is going towards handling bigger and higher wave forces during extreme event. This said, the selection of defence structure depends on the wave characteristics and the morphology of the coastal region. In view of the changing wave climates in future, more research and experiments were conducted featuring alternative coastal structures in hope that it will be employed at selected coastal areas whether shore-connected or detached, submerged or emerged and the position can be alongshore or oblique. In the early 20's the design of coastal defence has evolved and more researchers going towards an environmentally friendly approaches. A pilot projects using shore-connected NEXC Block, a hollow concrete unit along the coasts of Port Dickson and Marang, in Negeri Sembilan and Terengganu respectively has shown a positive outcome so far within a year it was commenced with accumulated beaches detected (Mohamed Rashidi et al., 2017). Black and Mead (2000) preferred submerged breakwaters over traditional rubble-mound breakwaters or vertical wall breakwaters due to their low environmental impact. These structures are still considered popular alternatives in solving coastal engineering problems. A successful approach by Oh and Shin (2006) on submerged geotextile tubes in protection of the East Korean shore shown a successful outcome of shoreline accumulation at immediate protected beach. Heibaum (2002) and Sprague (1995) presented similar end results using geotextiles as protection medium. In submergence conditions the structures resulted in less reflection and the porosity added dissipate the wave energy whilst control water tranquility conditions. Pilarczyk (2003) and (Rao et al, 1999) were

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in agreement with the submergence effect especially in wave disturbances control which also agreed by Stamos et al. (2003). In other type of wave breaker models, wave breaking tests on porous breakwaters specifically on pile has been carried out since the 60's. It has been considered a good medium of structural defence by many researchers and an added score in preferable environment water conditions such that in harbour. The latter is in agreement by many researches (Hayashi et al. 1966, Rao et al. 1999, Costello 1952, Nagai 1966, Van Weele and Herbich 1972, Khader and Rai 1981, Mani and Pranesh 1986, Mohd Anuar and Jaffar Sidek 2010, 2012). A single row suspended pile tested in various porosities has been researched by Rao et al. (2003). Their study indicated a parallel result with energy losses increase by 10% to 15% when porosity increase from 0% to 25%. Other than porous pipe, alternative type of breakwaters such as vertical thin barriers by Wiegel (1961), vertical slotted walls (Grüne and Kohlhasse, 1974), vertical slit type breakwater (Kakuno, 1983), vertical and horizontal wave screens (Thomson, 2000) and hollow hemispherical shape reefs (Armono, 2002, Armono and Hall, 2003) have been tested for their performance in transmitting waves. Porous and submergence are known to have an important role in breaking wave energy on the surface waves and the performance of a potential breakwater (Dick and Brebner, 1968). Twu et al. (2001) tested on a solid breakwater found that a solid medium shown less transmission but higher diminishing impact on wave energy and greater reflection. Apart from the traditional rubble mound breakwater (Losada et al., 1996, Dattari et al., 1978) and submerged wave filter systems (Clauss and Habel, 1999), Hattori (1972) in his research on perforated wall concluded that wave

attenuation depended on three elements, namely the ratio of wall thickness, hole diameter and the porosity of the wall. Losada et al. (1997) tested on submerged porous step breakwater with porosities between 0.521 and 0.62 indicated that percentage of transmission was highly dependent upon porosity with wave transmission decreased dramatically when porosity increased and vice versa. Clauss and Habel (1999) concluded that when the structural porosity is greater than 50%, nearly no reduction of wave height and energy was achieved, a reduction of 26% was achieved when porosity at 5% under full submergence. Thomson (2000) used various porosities in his wave screens tests agreed with Clauss and Habel (1999) but with contrast results between porosity and wave transmission. Twu et al. (2001) reported that the transmission coefficient was more significantly reduced particularly for thick or wide structures with lower porosity. Thus indicated that wider wave breaker causes multiple dissipation of surface waves coupled with its' low porosity. Whilst Thomson (2000) reported Hartmann (1969) investigated that through wire mesh structure the percentage of wave energy dissipation is higher when the wave steepness was greater and the mesh was thicker (low porosity). Similar conclusion was made by Ting et al (2004) from their test on a frame-type rectangular box under a series of porosities where wave energy loss decreased for a certain percentage of porosity which also agreed by Jaffar Sidek and Abdul Wahab (2007) with the energy losses decreased further as porosity reached above 0.8. In this work, we used the method of Rao et al. to perform a parametric experimental study to investigate simultaneously in various porosities, submergence depth and model widths the wave transmission character-

istics of a single and two-rows cylindrical breakwater models.

Experimental Methodology

Two-dimensional tests of the cylinders were carried out in an 18 m (length) x 0.95 m (width) x 0.9 m (height) wave flume tank at the Coastal and Offshore Engineering Institute (COEI) of Universiti Teknologi Malaysia (UTM). The tank is equipped with an L-shaped steel bar-screen which acted as a passive wave absorber to reduce wave reflection and a piston wave generator to generate waves. The test model was installed in the channel about 10.0 m from the wave generator board. Figure 1 shows details of the wave flume with five capacitance-type wave probes used to measure incident and transmitted waves, located in front and rear of the test model respectively. The model cylinder used in this study consisted of two diameters, 200 mm and 100 mm in various porosity (6.25, 14, 28 and 48% porous) with similar structural height of $h = 25$ cm. A single cylinder with a diameter of 200 mm is known as C1 and the 100 mm is known as C2. Whilst the double cylinder with an external diameter size of 200 mm and internal cylinder of 100 mm

in size is known as C3. Several series of four (4) unit cylinders (one-row arrangement) were constructed for each type of models and in four porosity configurations. The three cylinder models in different porosity is shown in Figure 2 and the one-row arrangement of the model in the flume is shown in Figure 3. The wave transmission investigation is conducted in two stages. First, the experiment is conducted for a C1, C2 and C3 one-row model in various porosities and test parameters. The models will be tested for wave transmission, K_t and energy loss coefficient, E_l . In second stage, the two models, C1 and C3 will be investigated for two-row model arrangement under similar test parameters. The two-row model arrangements were designed in three spacing sizes, $B = 0.02D, 0.5D$ and $0.75D$. The experiment was to investigate which model resulted in the least transmitted wave under specific porosity. In general, it is to determine which model configuration will show a significant percentage decrement of wave transmission. At the end of the experiment, a mathematical equation will be derived for each model C1, C2 and C3 in both one- and two-row configuration analyses.

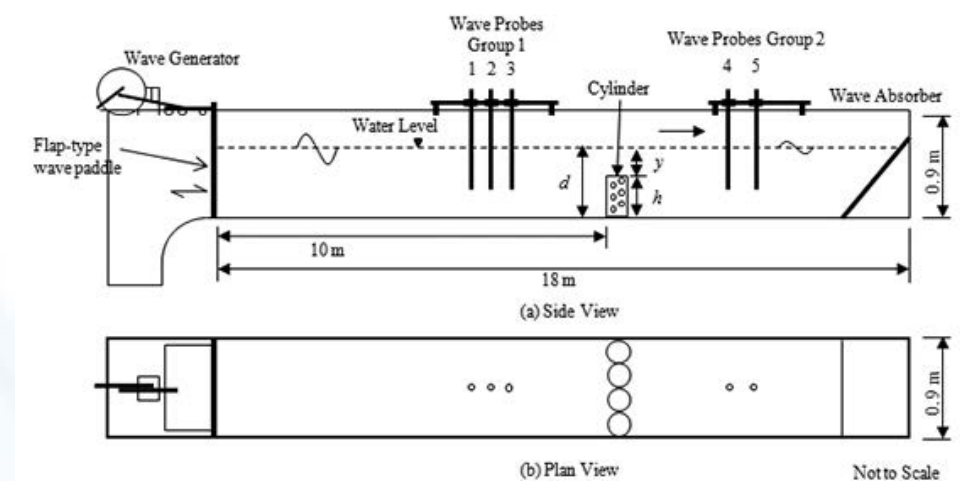


Figure 1: Side view (top) and plan view (bottom) of the experimental setup

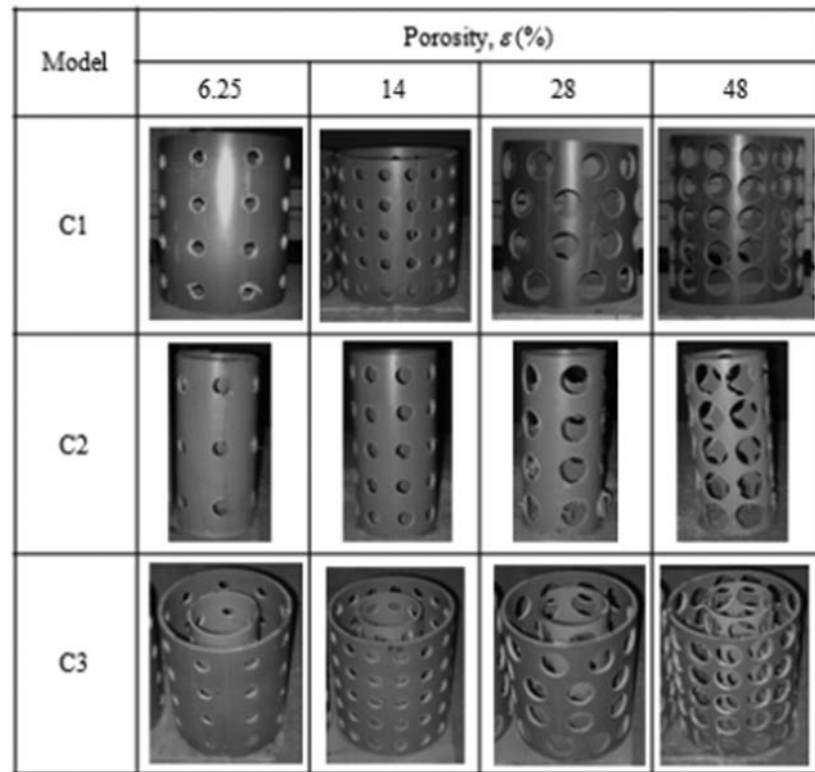


Figure 2: Three types of cylinder models in various porosities



Figure 3: One-row C1 model ($\epsilon = 6.25\%$)

A reflection and calibration test (without test model) on the wave paddle and wave probe was first conducted before the real tests on the model is carried out. Each wave probe was calibrated and incident and reflected waves were resolved using a least square analysis of 3 probes of the 5 probe array (Mansard and Funke, 1980). Several wave frequencies were generated

to get a series of suitable wave period combinations with respected to the selected water depths. The incident wave characteristics at probe groups 1 and 2 were used to define the transmission coefficient K_t such that:

$$K_t = \frac{H_t}{H_i} \quad (1)$$

where, H_t and H_i are the incident H_{m0} values at probe arrays 2 and 1 respectively. The transmission coefficient values were analysed with respect to the various incident wave and structure characteristics. This analysis involved a simple graphical trend analysis of the data, followed by a comprehensive statistical analysis of alternative design equation relating to K_t to the most important design variables. The trend analysis was conducted with dimensional and dimensionless variables considered to be important to the transmission process. The most important dimensionless variables were ascertained by dimensional analysis of the transmission process. It is generally accepted that the transmitted wave at a fully submerged breakwater is a function of a number of variables. Using Buckingham's Method and matrix, the Buckingham Pi Theorem was used to determine the number of dimensionless products formed as a symbol π (Sharp, 1981). A dimensionally homogeneous equation involving n variables of dimensionless products is arranged in Equation (2).

$$n_r = f(\pi_1, \pi_2, \pi_3, \dots, \pi_{n-r}) \quad (2)$$

where, r is the number of fundamental dimensions contained in the original variables. The required number of π terms is less than the number of original variables by the amount of r . In determining the number of fundamental dimensions in a set of variables, it can be eased by setting up a matrix of the variables and their fundamental dimensions. The significant quantities related to wave transmission, are H_i = incoming wave height; T = period of incoming wave; H_t = transmitted wave height; H_r = reflected wave height; y = height between top cylindrical model and water surface, d = water depth in vicinity of

tested models; L = wave length; B = spacing between rows of tested models; width of model, W and ϵ = porosity of tested models. The complete dimensionless equation for one row and two rows test models becomes,

$$\pi = f\left(\frac{H_i}{L}, \frac{H_t}{L}, \frac{y}{d}, \epsilon\right) \quad (3)$$

or

$$K_t = \frac{H_t}{L} = f\left(\frac{H_i}{L}, \frac{y}{d}, \frac{W}{d}, \epsilon\right) \quad (4)$$

for fully submerged one row and two-row model without spacing, and, for the case of two-row model with spacing

$$\pi = f\left(\frac{H_i}{L}, \frac{H_t}{L}, \frac{y}{d}, \frac{B}{d}, \epsilon\right) = 0 \quad (5)$$

or

$$K_t = \frac{H_t}{L} = f\left(\frac{H_i}{L}, \frac{y}{d}, \frac{B}{d}, \epsilon\right) \quad (6)$$

These basic dimensionless variables were used to develop alternative dimensionless variables which are more relevant to the physical processes affecting transmission at submerged breakwaters. Dimensional and statistical analysis of these relevant variables was undertaken to develop a design equation for transmission at fully submerged cylinder model.

The selected test parameters for this study is shown in Table 1 and the test was conducted under no-breaking wave conditions. Three water depths, d were used in the experiments, namely, $d = 0.27$ m, 0.30 m and 0.35 m. In each individual test parameters for the cylinder model, the mean transmitted wave height was determined by taking an average of at least 5 wave heights in the generated wave periods.

Results and Discussion

The relationships between wave transmission and wave steepness differentiated by various non-dimensional variables; the relative submergence depth (y/d), cylinder configurations (W/d), relative spacing (B/d) and porosity (ε) are presented and discussed in the following section.

Depth of Submergence (y/d)

To represent the influence of submergence depth on the one-row model of C1, C2 and C3 under porosity $\varepsilon=6.25\%$, the variations of K_t and energy loss, E_l are shown in Figure 4. From Figure 4 it is evident that lower submergence produced lower transmission and increase energy losses. Furthermore, wider cylinder with bigger diameter such as C1 and C3 transmitted wave less than smaller cylinder, C2. The

effect of submergence is likely less significant when tested with model C2. This can be explained due to the surface wave travelled over wider model, C1 and C3 as compared to smaller model C2 thus diminished higher percentage of wave energy. Hence, wider model has significant advantage in terms of wider footprints to diminished surface waves. It is similar like having a two-row model with closely fit arrangement which represents double width model, thus could resulted in an increased in energy losses.

To test this theory, Figure 5 shows the effect of two-row cylinder model C1 and C3 of similar porosities with spacing $B = 0.02D$ on the wave transmission and energy dissipation performance. The plot shows that closely fit tow-row model resulted in a significant decrement of wave transmission especially under lower submergence depth and at steeper waves (H/L). Larger

vortices are expected from high frequency waves or high wave steepness at the leeward side of the model thus resulting in greater energy dissipation. In between double cylinder model C3 and singular cylinder C1, there is not much improvement in the resultant K_t shown in all submergence depths tested with the variance is less significant.

Table 1: Summary of test parameters

Geometry of Test Models		Hydraulic Parameters		
Porosity, ε	Spacing between rows, B	Water depth, d (m)	Depth of Submergence y/d , (%)	Wave period, T (s)
6.25%	$0.02D$	0.27	7	0.85, 0.90,
14%	$0.5D$	0.3	17	0.96, 1.03,
28%	$0.75D$	0.35	29	1.12, 1.18, 1.25, 1.34, 1.49, 1.67
48%				

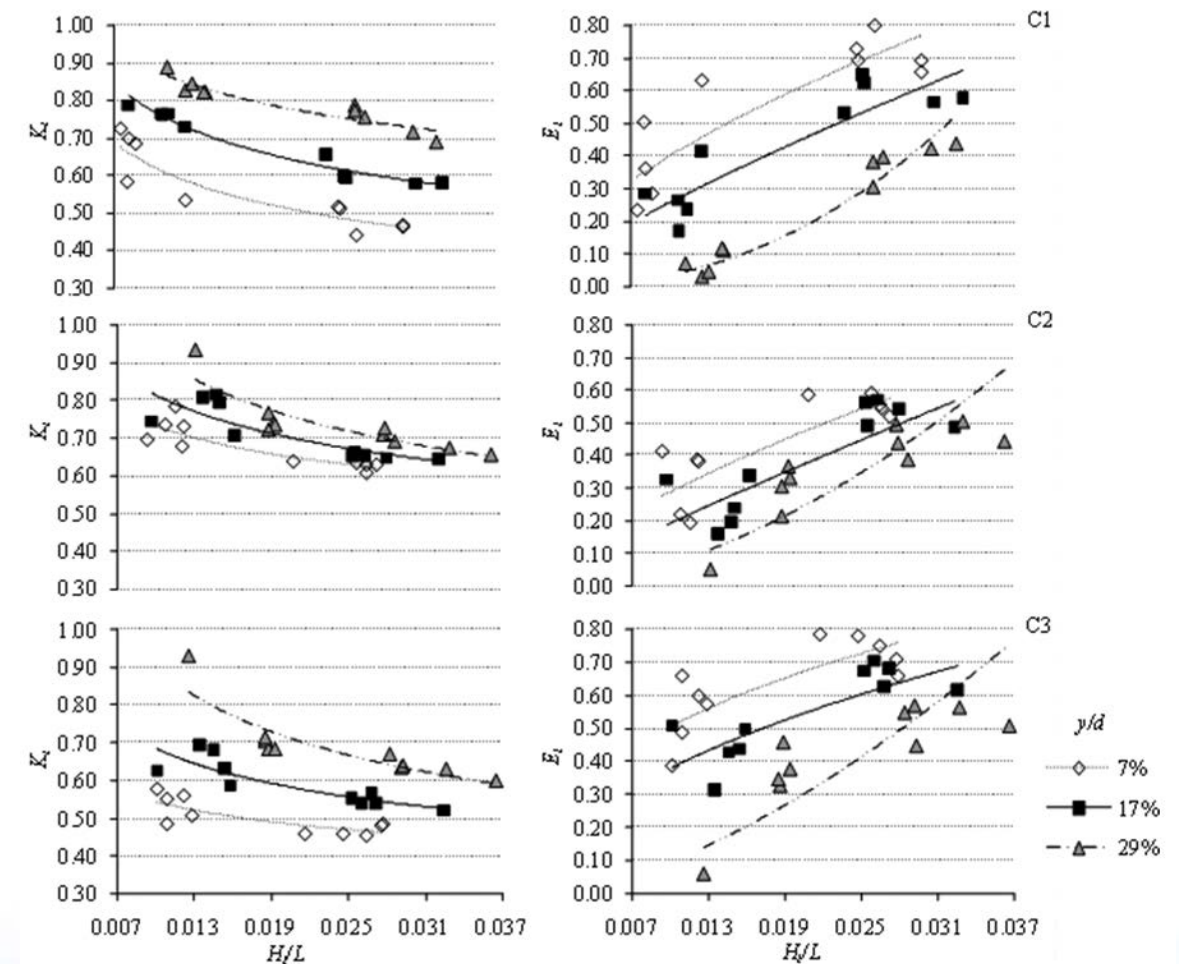


Figure 4: Wave transmission (left) and energy loss (right) coefficients due to surface waves travelling over one-row submerged cylinder C1, C2, and C3 with porosity $\varepsilon = 6.25\%$

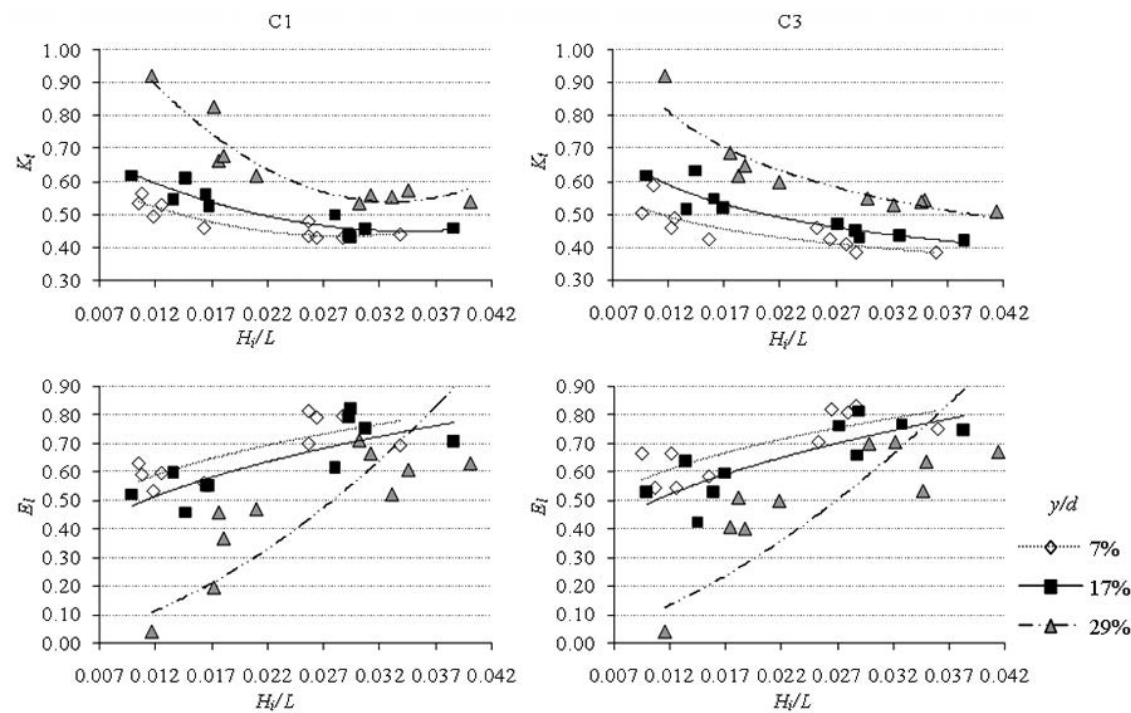


Figure 5: K_t (top) and E_I coefficients (bottom) for 2-row submerged cylinder C1 and C3 with $\mathcal{E} = 6.25\%$, $B = 0.02D$

Relative Width (W/d)

The effect of model crest width on the disturbance of surface waves is significant at higher wave frequency as shown in Figure 6. The steeper waves are subjected to extensive surface wave disruption under wider crest model (as in two-row closely fit arrangement) as compared to narrow model crest width (one-row model arrangement). Also, in low frequency or low wave steepness, the modulate momentum of the surface waves tend to glide on the crest of smaller model, C2 hence allowed more waves to be transmitted at the lee of the model. Furthermore, smaller porosity gave additional dissipation effect on the surface wave energy motion and resulted in

further decreased in wave transmission. The effect of porosity on the model's performance in transmitting or dissipating wave is investigated further in the following section.

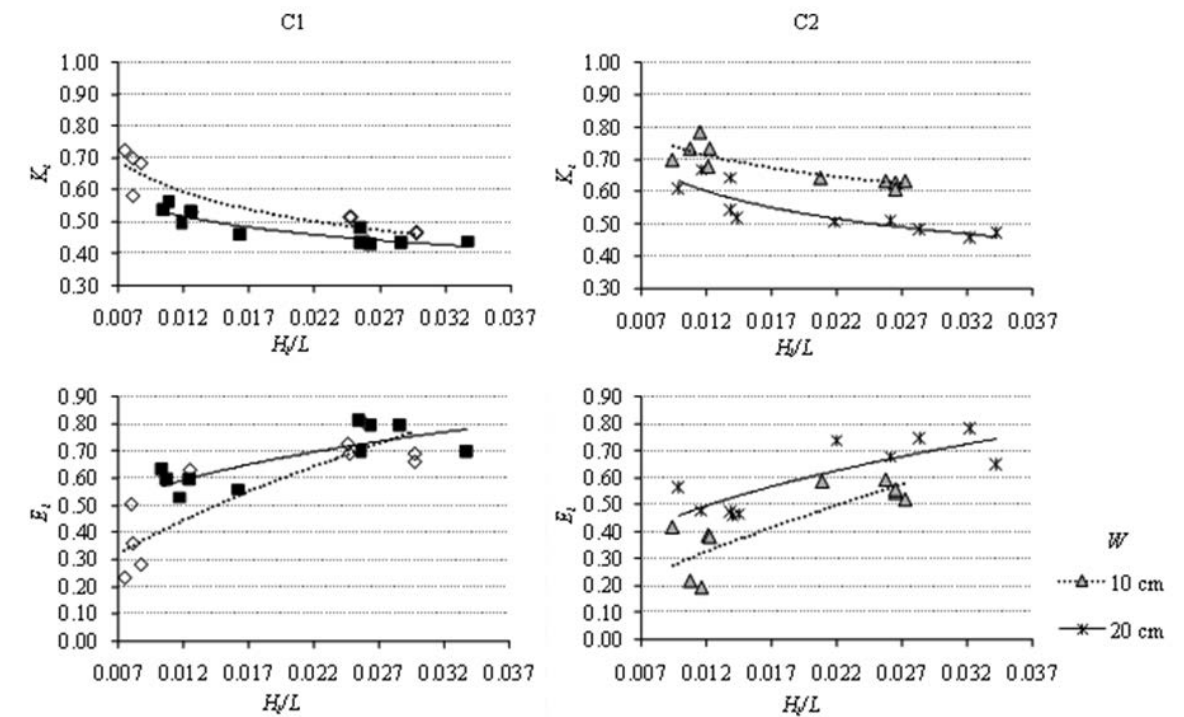


Figure 6: Influence of cylinder width for model C1 and C2 with $\mathcal{E} = 6.25\%$, $d = 0.27$ m on K_t and E_I coefficients

Percentage of Porosity ($\mathcal{E}$)

The performance of cylindrical models with different percentage of porosities is compared. Plots of H_i/L against K_t were drawn with the porosities as a third variable. From Figure 7, it can be clearly noted that at higher wave steepness more porous pile caused more wave transmission. It shows that for model C1, as the porosity increases from 6.25% to 48%, wave transmission K_t increases about 22% for H_i/L less than 0.010 and about 40% for H_i/L more than 0.025 for lower water depth $d = 0.27$ m. Hence, there is an average increased in transmission coefficient of about 30% between the lowest (6.25%) to most porous (48%) model. Whilst for model C2 and C3, the average increment of K_t between porosity 6.25% and 48% is approximately at 14% and 28% respectively. Studying the effect

of porosities in Figure 7, between porosity 28% and 48% all models shown almost uniform increment of K_t for all wave steepness. Lower porosity resulted in lower transmission as expected regardless of the type of cylinder tested. Nevertheless, model C1 and C3, indicated a significant variance in wave transmission in all wave steepness tested. It can also be observed that model C2 indicated an insignificant variance in wave transmission for model with 6.25% and 14% porosity respectively. It shows that the performance of smaller model with lower percentage of perforations is only marginal.

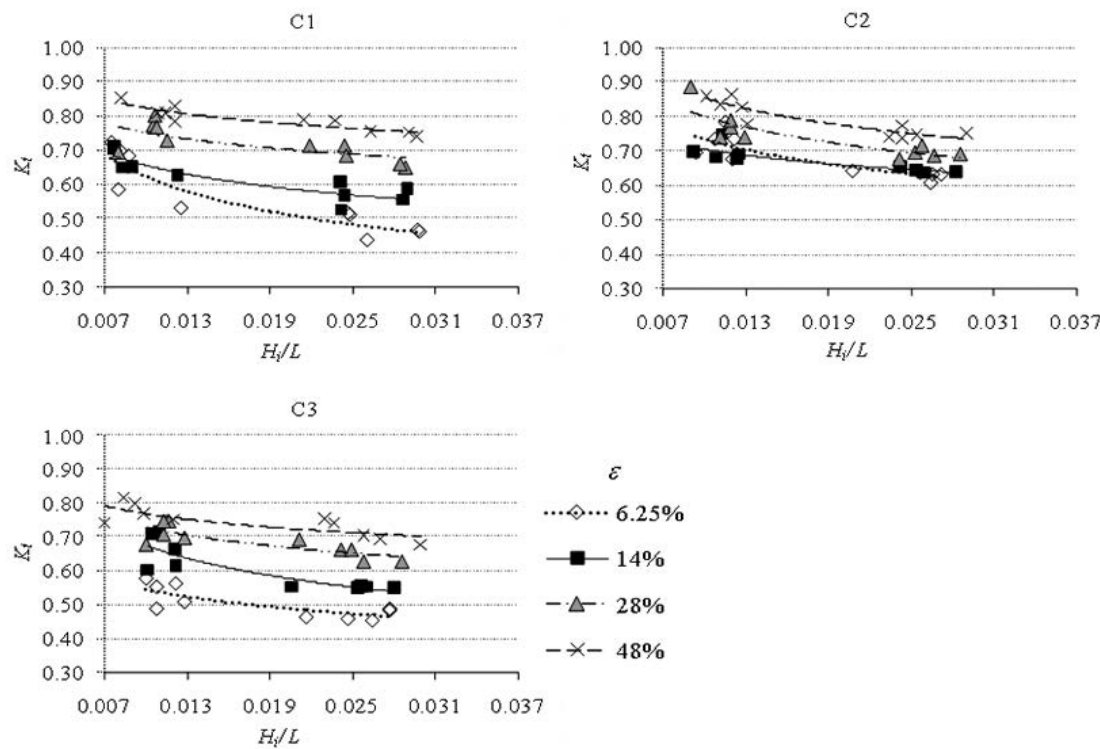


Figure 7: Influence of porosity ε on K_t at $d = 0.27$ m for one-row model C1, C2 and C3

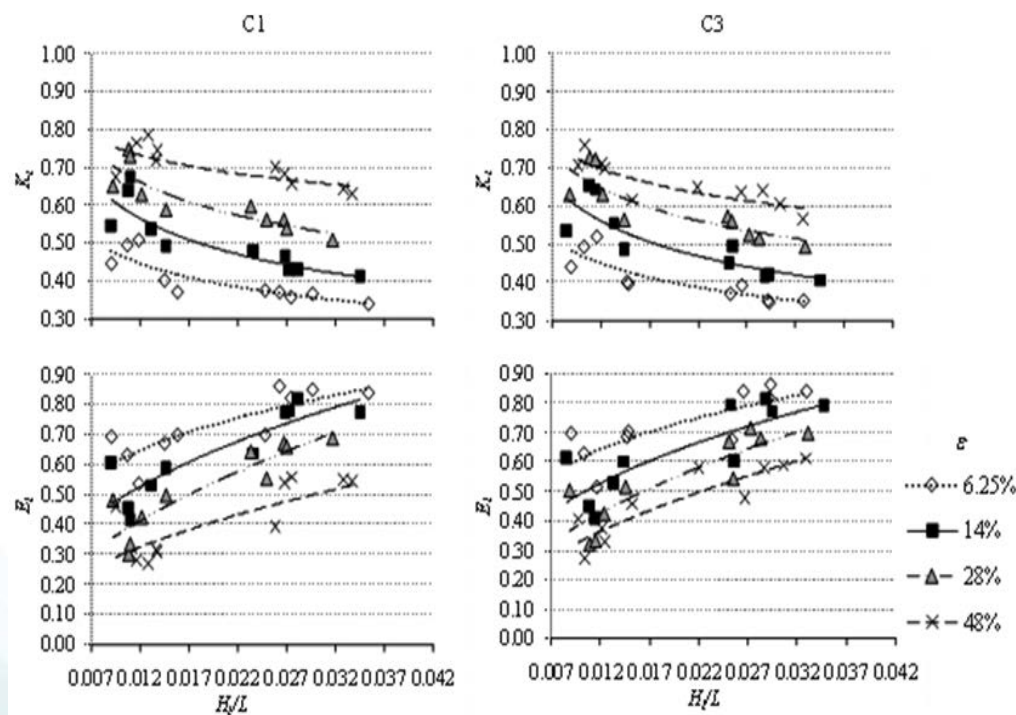


Figure 8: Influence of porosity ε on K_t at $d = 0.27$ m for 2-row model C1 and C3 at $B = 0.75D$

As for two-row model arrangements, the effect of porosity changes on K_t performance for spacing $B = 0.75D$ is investigated in Figure 8. From Figure 8, it is found that as the relative wave height increases, the wave transmission decreases. The increment of K_t between tested porosity in model C1 and C3, is almost homogeneous especially in lower percentage of porosity, the 6.25 % and 14% perforated models respectively. In more porous model, a comparison between porosity 28% and 48%, model C1 shown a significant variance in K_t performance with an increase of K_t from 52% to 65% at higher wave steepness. As for model C3, at higher wave steepness, model with porosity 28% and 48% shown slightly lower increment variance of about 9%. This can be explained by considering the water particle motions generated by double cylinder has likely produced higher dissipation in between the cylinders when water particle velocity and acceleration increases as wave steepness increases.

Relative Spacing (B/d)

Figure 9 illustrate the effect of relative spacing B/d on wave transmission under submerged water depth $d = 0.27$ m. From observations, the plot shown in Figure 9 indicated that the spacing between rows of tested models has been found to be an important parameter in attenuating waves. That as the relative spacing B/d increased more than 0.016, wave transmission coefficient K_t decreased below 40% at wave steepness range between 0.025 to 0.036. The wave energy loss is more than 50% for all row spacing tested on model C1 and C3. Thus concluded that regardless of the row spacing, B , and type of cylinder, an investigation of model with more than one row arrangement shown more than 50% wave energy reduction or wave damping capability.

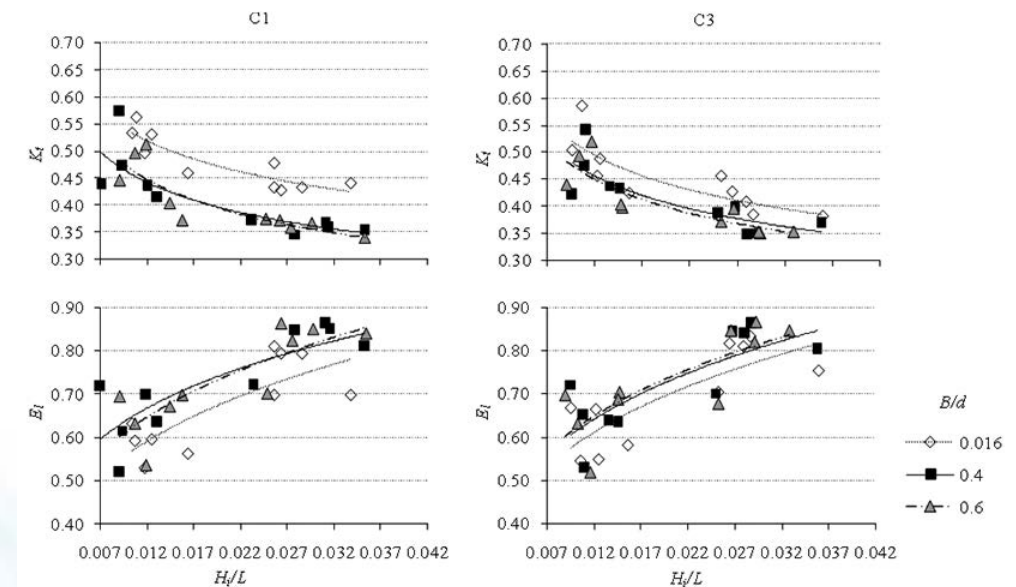


Figure 9: Influence of B/d on K_t and E_t at model porosity $\varepsilon = 6.25\%$, $d = 0.27$ m

Relative spacing $B/d = 0.016$ provides highest K_t compared to B/d of 0.4 and 0.6 in all wave frequency tested. Higher B/d with double dampening effect created by two rows model tend to create more surface wave turbulence hence lower the transmission and increased energy losses as compared to closely spaced model. The closely spaced model with $B/d = 0.016$ is likely functioned as a single unit model thus produce low vortices turbulence on the surface wave as it travels through the model hence resulted in higher wave transmission.

Analytical Functions

Multiple regression analysis was used to determine the relationship between the independent non-dimensional variable and wave transmission, K_t . A commercial statistical software package, SPSS Clementine Client V10.1 for Windows (SPSS Inc., 2006) has been used to develop the simplest and most viable equation to aid in estimating the wave transmission through various cylinder models. Equations 7 and 8 illustrate the relationship between each of the predictor terms with dependent variable K_t for one-row and two-row model configurations.

Table 2: Summary of Equations Derived for One-row Cylinder Models

Equations	
Model C1	$K_t = -27.647 - 6.555(H_i/L) + 29.152(y/d) + 35.351(W/d) + 0.365\varepsilon$
Model C2	$K_t = 80.54 - 7.303(H_i/L) - 79.32(y/d) - 199.438(W/d) + 0.265\varepsilon$
Model C3	$K_t = -24.333 - 6.04(H_i/L) + 25.705(y/d) + 31.124(W/d) + 0.455\varepsilon$

$$K_t = a_0 + a_1 \left(\frac{H_i}{L} \right) + a_2 \left(\frac{y}{d} \right)^2 + a_3 \left(\frac{W}{d} \right) + a_4 \varepsilon \quad (7)$$

where $y/d, H_i/L, W/d$ and ε are the independent non-dimensional variables for one-row model whilst for two-row model,

$$K_t = a_0 + a_1 \left(\frac{H_i}{L} \right) + a_2 \left(\frac{y}{d} \right)^2 + a_3 \left(\frac{B}{d} \right) + a_4 \varepsilon \quad (8)$$

Observation data from the laboratory tests were used to evaluate the proposed equations by examining the percentage of prediction errors, index of fit R^2 , standard error of estimate, residual normality and F statistics using the SPSS statistics package. The developed empirical equations for all three cylinder models C1, C2 and C3 are given in Tables 2 and 3 for one-row and two-row arrangement respectively.

Table 3: Summary of Equations Derived for 2-row Cylinder Models

Equations		
No Gap	Model C1	$K_t = -4.366 - 7.734(H_i/L) + 5.77(y/d) + 6.23(B/d) + 0.414\varepsilon$
	Model C2	$K_t = -6.683 - 7.4(H_i/L) + 8.027(y/d) + 9.068(B/d) + 0.434\varepsilon$
With Gap	Model C1	$K_t = 0.536 - 6.965(H_i/L) + 0.728(y/d) - 0.038(B/d) + 0.503\varepsilon$
	Model C2	$K_t = 0.523 - 7.103(H_i/L) + 0.758(y/d) - 0.002(B/d) + 0.448\varepsilon$

Conclusion and Recommendations

The size of perforations of the test models has been found to have a significant effect on wave transmission. The increase in the coefficient of wave transmission, K_t for the porous pile means that more energy is allowed to penetrate through the pile resulting in the increased height of the transmitted wave. A porosity value of $\varepsilon = 48\%$ was found to indicate the highest wave transmission characteristics.

The height of transmitted wave was also governed by water depth and wave steepness. The wave transmission coefficient was found to increase with the increment of water depth. With increasing wave steepness, the wave transmission tended to decrease for a given water depth for all porosity values tested. The increase in spacing (B) resulted in a decrease in wave transmission. Thus, this indicated that model spacing B has an influence on wave transmission coefficient. However, the effect of the number of rows is significant only when the rows were increased from one to two-rows which resulted in higher wave energy being dissipated. However, the increase of an additional row (three-row model, undisclosed) indicated an additional percentage of wave energy loss by 6 – 10% as compared with the 2-row model. Thus the increase in the number of rows, W to attenuate waves was shown to be less significant. porous

cylindrical type structure will likely make a better alternative mitigation medium to dampen wave energy at the lee of the structure as an offshore detached breakwater or as a nearshore artificial reefs cum wave breaker by creating calm environmental condition within the basin and at the same time gives good water circulation with the model surface permeability effect. The arrays of the perforated cylinders also has potential to ensure mild climates for mangrove sapling growth and sustainability, beach erosion abbeviator and/or as a fish breeding ground. The equations derived for K_t as summarized in Tables 2 and 3 have been generated based on four variables namely $y/d, H_i/L, B/d, W/d$ and ε to be inferred as a general wave transmission estimation under various hydraulic and cylindrical geometrical parameters.

Acknowledgements

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A STUDY OF CASCADE AERATOR MODEL FOR GROUNDWATER TREATMENT

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ABSTRACT

Many technique have been study in groundwater treatment. Cascade Aerator system is one of the groundwater treatment technique to remove heavy metal such as Iron and Manganese. Cascade Aerator has been used because it is effective way to treat groundwater at low cost. In this study, Lattice Boltzmann Method (LBM) and experimental will be used to investigate the aeration process in Cascade Aerator model. In designing a new Cascade Aerator, a different dimension issued to determine the best design that can decrease the concentration of Iron and Manganese. Two sets of Lattice Boltzmann Method (LBM) simulations and two set of Particle Image Velocimetry (PIV) experiments are carried out and the flow results of the velocity and pressure are computed. Based on the findings, it was shown that LBM simulation and PIV data are in good agreement with each other in term of velocity distribution. Additionally, it was also found that the velocity of the water has a significant influence on the aeration efficiency. In addition the occurrence of negative pressure region in the cascade aerator, increases the possibility of cavitation to occur. The cavitation will damage the structure of the cascade aerator. Furthermore, it also founds that increasing in barrier thickness of the cascade aerator will improve the aeration efficiency and reduce the cavitation in cascade aerator.

Keywords: Cascade Aerator, Iron, Manganese, LBM, PIV

Introduction

The river banks can be defined as land by the banks of the river. The unstable river is often prone to rapid erosion. Many techniques are developed and used on river banks such as agile spills, rock clumps, structured soil cells and others to stabilize river bank and improve river flows. The structure of a riverbank is the same as a cassette aerator. The cascade aerator is the integrated part of the dam where it is usually used to remove excess water from the reservoir to prevent overflow and to release pressure on the dam from the increase in the amount of water. Modern cascade aerator containers are designed as prismatic rectangular chutes with horizontal steps. On the chute design step, the presence of steps can increase the higher energy dissipation on the overflow surface. In addition, the presence of air landing (i.e. aeration flow) helps to increase waste energy flow. Therefore, the energy level in the downstream dissipater structure from overflow can be reduced and the risk of cavitation is reduced. Stream on the slope of the step was widely studied using both laboratory experiments on scale models and analytical and numerical approaches. Studies have revealed three types of common flows over the overflow, namely the flow of nappe, partial nappe flow (transitional flow) and trimming flow and each flow occurs for certain conditions. Nappe flow usually occurs at low flow rates and extends altitude-related measures. The cascades flow over the steps, falling within a series of plunge from one step to another in a thin layer attached to each step. On the other hand, the emission stream occurs at a larger flow rate where water flows step as a coherent flow, slides over the steps and is offset by fluid flowing between them. Transitional flow is a flow

regime located in the region between the two streams. In this type of flow, the nappe does not completely block the surface and it spreads with great upheaval.

Aeration is the process that is often used by many factories and treatment plant in order to remove the constituents in water such as dissolved gas and oxidised dissolved metals and important to provide DO for aerobics organisms in BOD removal and nitrification (Alp et al., 2011). According to Alp et al., 2011, aeration is defined as using atmospheric air as the oxygen sources and oxygenation happened, this method are important process in water and waste water treatment to transfers oxygen from gases to the liquid (Moulick et al., 2010; Baylaret al., 2009). The turbulence of aeration process will cause the scrubbing process that will help to remove the dissolved gases from the water. Subsequently, the dissolved gas will escape into the surrounding air. The efficiency of the aeration is influenced by the amount of the surface contact between air and water. The size of the water drop will also affect the efficiency of the aeration. Iron and manganese minerals can be usually founded in the soil and they will be dissolved into the groundwater. The water will turn to dark brown colour due to the presence of iron and manganese. For this reason, the water is not suitable to be consumed by human. The aeration process can help to filter this water thereby increasing the quality of the water until it is safe for consumption. In this project, the function of the stepped spillway is same as the cascade aerator. Given the similarity between both problems, the stepped spillway is used to generally investigate the aeration process in this project. Figure 4 shows the process of stepped spillway setup.

In lab experiments, different configurations of built spillway are built. The 3mm

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Perspex thickness is selected in this project. The Perspex is cut to a certain dimension based on a cascade aerator configuration stepped. Then, the Perspex plate is connected to build a cascade aerator. The cascade aerator model is shown in Figure 1. After the cascade aerator is built, the experiment will run. First, the water is poured into the water tank, then the water will flow down the cascade aerator. The velocity and pressure on the stepping barrier located in the airless and aerated airflow area is determined to investigate the ventilation. In addition, overflow was also observed.

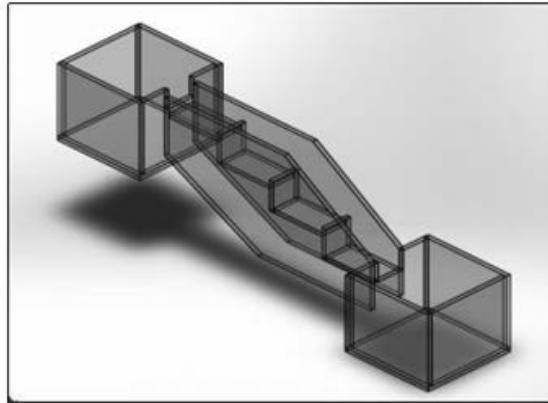


Figure 1: Schematic diagram of stepped spillway

Boltzmann Lattice Method (LBM) based solution is selected to simulate flow in the cascade aerator. LBM solutions have been found to be effective for high precision fluid simulation at calculated solutions (Abas et al., 2015; Abas et al., 2016). After that, Preview software is used to process results and analyse.

Governing Equations: Lattice-Boltzmann Formulation

The D3Q19 model is selected in this simulation. There are 19 velocity vectors in this model, Figure 2. Lattice Boltzmann Method, LBM is a computational

simulation method used for simulation of fluid flow (Abas et al., 2011). LBM also is a simulation technique, which use the distribution function to replace tagging each particle as in molecular dynamic simulations.

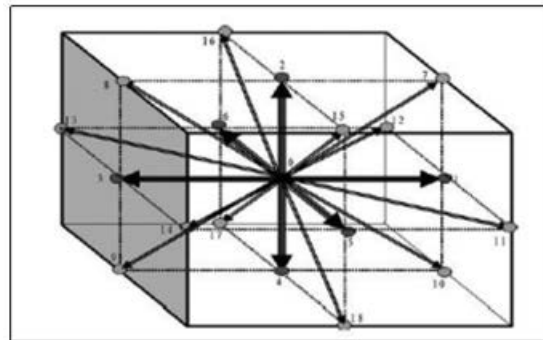


Figure 2: Lattice arrangement for D3Q19. (A.A. Mohamad, 2011)

The Boltzmann transport equation is applied in this simulation. Firstly, the statistical description is represented by the distribution function, $f(r, c, t)$. This distribution function can determine the number of the molecules with particular velocity between c and $c + dc$ within a particular range between r and $r + dr$ at a given instant of time (t). If the collision, Ω is applied to the system, the distribution equation become:

$$f(r + cdt, c + Fdt, t + dt) - f(r, c, t) = \Omega[f(r, c, t)] \quad (1)$$

If there is no any external force is applied to the system, the Boltzmann transport equation can be rewritten as:

$$\frac{\partial f}{\partial t} + c \cdot \nabla f = \Omega \quad (2)$$

The BGKW Approximation

However, the collision term is very complicated thus it is difficult to solve the Boltzmann transport equation. The outcome of two body collisions is not likely to influence significantly. Therefore, Bhatnagar, Gross and Krook (BGK) introduce a simplified model for collision operator in 1954 in order to solve it.

$$\Omega = \omega(f^{eq} - f) = \frac{1}{\tau}(f^{eq} - f) \quad (3)$$

The coefficient ω is called the collision frequency, a Maxwell-Boltzmann distribution function and τ is called relaxation factor. Besides, f^{eq} is the local equilibrium distribution function. The discretized Boltzmann equation along specific can be written as:

$$\frac{\partial f_i}{\partial t} + c_i \cdot \nabla f_i = \frac{1}{\tau}(f_i^{eq} - f_i) \quad (4)$$

The right-hand side of the equation is the advection (streaming) whereas the left-hand side term is the collision process. The above equation is the working horse of the lattice Boltzmann method. Besides, it can replace the Navier Stokes equation in CFD simulations. Thus, it is possible to derive Navier-Stokes equation from Boltzmann equation.

After the equation (4) is discretized,

$$f_i(x + e_i \Delta t, t) = f(x, t) + \frac{\Delta t}{\tau}(f_i^{eq}(x, t) - f_i(x, t)) \quad (5)$$

It also known as collision step. The fluid density and macroscopic velocity can be obtained from the moment of the distribution function as below.

$$\rho = \sum_i f_i^{eq} \quad (6)$$

$$u = \frac{1}{\rho} \sum_i f_i^{eq} c_i \quad (7)$$

Equilibrium Distribution Function

The equilibrium distribution function can be written as:

$$f^{eq}(\rho, u) = \rho w \left[1 + \frac{1}{c_s^2}(c \cdot u) + \frac{1}{2c_s^2}(c \cdot u)^2 - \frac{1}{2c_s^2}(u \cdot u) \right] \quad (8)$$

The weight functions for D3Q19 are:

$$i) w_0 = \frac{1}{3}$$

$$ii) w_{1-6} = \frac{1}{18}$$

$$iii) w_{7-18} = \frac{1}{36}$$

Streaming step

The solution domain in the LBM needs to be divided into lattices. The fictitious particles reside in each lattice node. Some of these particles will move along a specified direction to the neighbouring nodes in accordance with the velocity set, Figure 3.

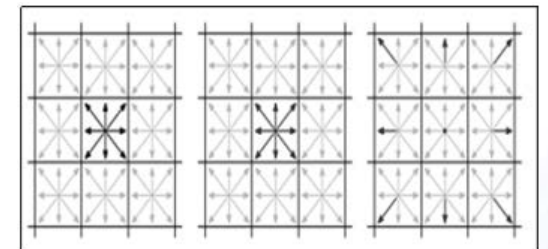


Figure 3: Collision-Propagate scheme for the D2Q9 model.

*Black color: Pre-collision values for populations

*Blue: Represent the post-collision status

Boundary Condition

Figure 4 shows the boundary conditions of the cascade aerator system. Periodic boundary condition is set at the start and the end of the simulation to simulate continuous flow of water. Additionally, the bounce back boundary condition are set at the barrier of stepped spillway to simulate wall boundary.

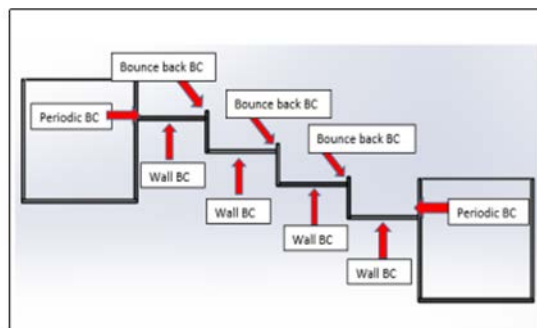


Figure 4: LBM boundary Condition Diagram

Results and Discussion

Based on Table 1, shows that the velocity of the main flow of water over the overflow step and the water velocity increases on the cascade aerator from overflow. The maximum velocity of the main stream of water over the cascade aerator is 7.188m/s. When the velocity increases, the pressure will decrease. The pressure is gradually descending along the main stream of water. The minimum pressure recorded is -7,962 Pa. Additionally, it also found that negative pressure occurred in the cascade aerator. This means there is a possibility of cavitation occurring there. Additionally, it also found that there was a lot of air trapped in step 4 of the jump overflow. Therefore, aeration occurs in step 4. Greater aeration efficiency is due to the greater amount of air trapped in the water. Therefore, it is found that when the length of the overflow step is extended (twice the size), the water

velocity will increase. As a result, the pressure will decrease due to the increase in water velocity. Next, aeration and cavitation phenomena will increase shown in Figure 5:

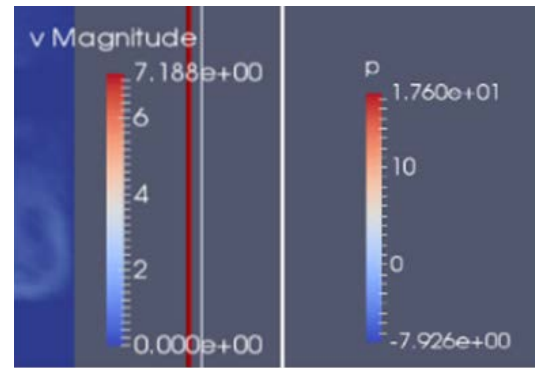


Figure 5: Water flow pattern, velocity and pressure profile

Table 1: Water flow pattern, velocity and pressure profile

Step	Flow Pattern	Velocity	Pressure
1			
2			
3			
4			

Conclusion

The relationship between the configuration of the stepped spillway and the aeration and cavitation phenomena had been investigated in this project. Three sets of LBM simulations and two sets of PIV experiment were carried out. In this project, the LBM simulation for the water flow over the stepped spillway is validated by the experiment. It proves that the LBM

simulation and the Para view software can simulate the water flow over the stepped spillway accurately. This proves LBM's mettle to determine the velocity and pressure result of the water in the stepped spillway with high accuracy.

It has been found that when the length of the step increases, the velocity of the water will increase as well. Thus, the energy dissipation rate per unit width and the

aeration efficiency increases. However, it was found that there was a negative pressure region in cascading coatings. Negative pressure that occurs is greater than negative pressure. Therefore, it has a greater possibility for cavitation to occur in a cascade aerator. Increased step length will increase both the formation of ventilation efficiency of cavitation area. If velocity is too fast, the efficiency of ventilation and cavitation will increase as well. Therefore, the velocity of water needs to be increased at an appropriate rate to avoid cavitation and increase the efficiency of the ventilation. It can be concluded that increasing the thickness of the coating of barbed coatings is the best design between the three sets. It can be used for river bank project and water treatment due to its high ventilation efficiency and low cavitation in the cascade aerator.

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ASSESSING THE STABILITY AND EFFICIENCY OF NEXC BLOCK AS AN INNOVATIVE COASTAL PROTECTION STRUCTURE IN A WAVE FLUME

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ABSTRACT

Physical model is one of the methods in assessing natural hydrodynamic changes due to multiple variables. In coastal region, the interactions can be visually assessed between structural, wave, water level and beach profile changes. NEXC Block or NAHRIM Coastal Erosion Protection and Beach Expansion Block has been developed and tested using this technique; the wave flume facility in NAHRIM. Main objective of the study is to assess the stability and efficiency of the structure as an innovative coastal protection structure. NEXC Block model has been geometrically downscaled to 1:10 and the experiment was carried in wave flume facility. The study includes simulation of multiple downscaled water levels (mean high water, mean sea level and mean low water) and wave heights (calm and extreme) parameters. Beach profile changes was evaluated by measuring sediment accumulation or loss for both before and after each simulation. Impacts on scour at toe and back scour were also determined. It was found that sediment loss is maximum during high water level with extreme wave condition. Sufficient scour protection would ensure structural intactness and stability of the model installation. During calmer period, the sediments will eventually returned back to the beach hence expanding the beach naturally. The result of the study was also compared to the analysis on beach profile changes at the existing NEXC Block pilot project site.

Keywords: physical model, coastal protection structure, erosion, scour, wave flume

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Introduction

Beach erosion and accretion are cycles of coastal processes which driven both naturally or by human activities due to hydrodynamic changes within the shore zone. Monsoon seasons, king tides and sea level rise are among the natural phenomenon which impacted the change of coastal hydrodynamic pattern in wave, current and water level (J.A. Comfort, M. S., 1997). Infrastructure development within coastal zone such as breakwater, jetty and groin directly disturbed existing sedimentation process hence causing erosion at the leeside. In term of beach stability, while sediment accretion is favored due to creation or expansion of more dry area, erosion however should be controlled in order to protect existing ecosystem, physical and socioeconomic activities within the area (Kamphuis, J. W., 2000). In general there are two methods of coastal erosion control measures; hard engineering and soft approaches (US Army Engineers, 1984). Hard engineering protection structures include breakwater, groin, seawall and concrete blocks namely ACCROPODS, TETRAPODS, SAUH, SINESLAB & LABUAN BLOCK. Some examples of soft approach techniques include beach nourishment, mangrove replanting, Beach Management System (BMS) and Pressure Equalization Module (PEM). NEXC (NAHRIM Coastal Erosion Protection and Beach Expansion Block) is an invention of a shore-based coastal block, placed on intertidal zone and considering the combination of both approaches. The advantages of coastal blocks are flexible design, easy, quick fabrication and installation works. NEXC Block provides ample shoreline protection against big waves during extreme condition, while encourages sandy sediment accumulation

during calmer period hence expanding the beach.

Research and development of NEXC Block has been conducted by researchers from National Hydraulic Research Institute of Malaysia (NAHRIM) and Universiti Teknologi Malaysia (UTM). The study includes testing of model structure in a wave flume facility located at NAHRIM. Physical model is considered as the most suitable approach to determine wave – water level – sediment – structure interaction in a controlled environment. The applied research was further expanded with the fabrication and installation of full scale NEXC Block in 2014. The severely eroded sandy beach at Pantai Batu Lima, Port Dickson, Negeri Sembilan and Pantai Rhu Muda, Marang, Terengganu are chosen as test sites. The objectives of this study are to assess the stability of NEXC Block as a coastal protection structure using physical modelling, to evaluate the mechanism of beach scour towards NEXC Block in a wave flume, and to determine the effectiveness of NEXC Block in reducing coastal erosion impact at site.

Materials and Methods

Study Area

Site A at Pantai Batu Lima, Port Dickson, Negeri Sembilan is threatened by coastal erosion each year due to strong winds, huge waves and high tides over the past five years especially during both Southwest and Northeast monsoons. Severe coastal erosion problem was reported within the vicinity of Tanjung Tuan Regency Hotel beach after the storm wave event in early October 2013 as shown in Figure 1. Length of the eroded shoreline was initially about 100 metres shoreward but since then the stretch of erosion increased each year. Based on observation it is also presumed that that specific area is experiencing lack of

sediment supply due to interference from emerged discharge drain on the north side and headland reclamation at the south side.

The bay is also a popular attraction for tourism and water sport activities throughout the year.

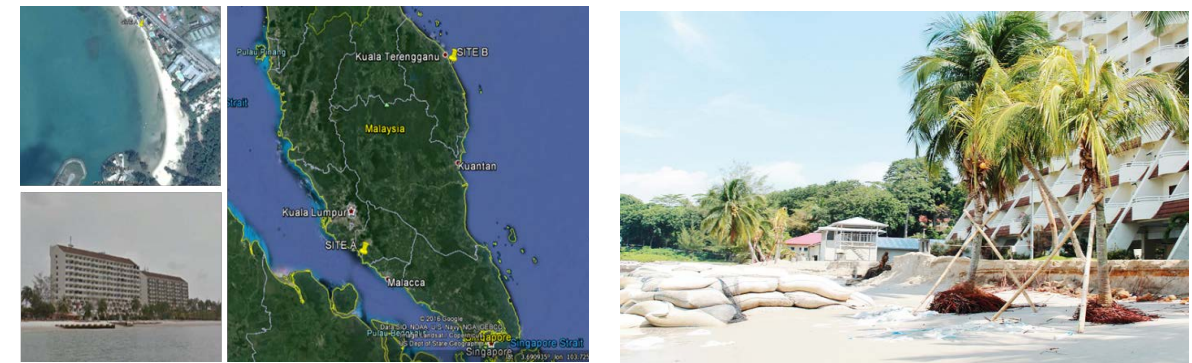


Figure 1: Coastal erosion within hotel compound at Site A, Port Dickson, Negeri Sembilan

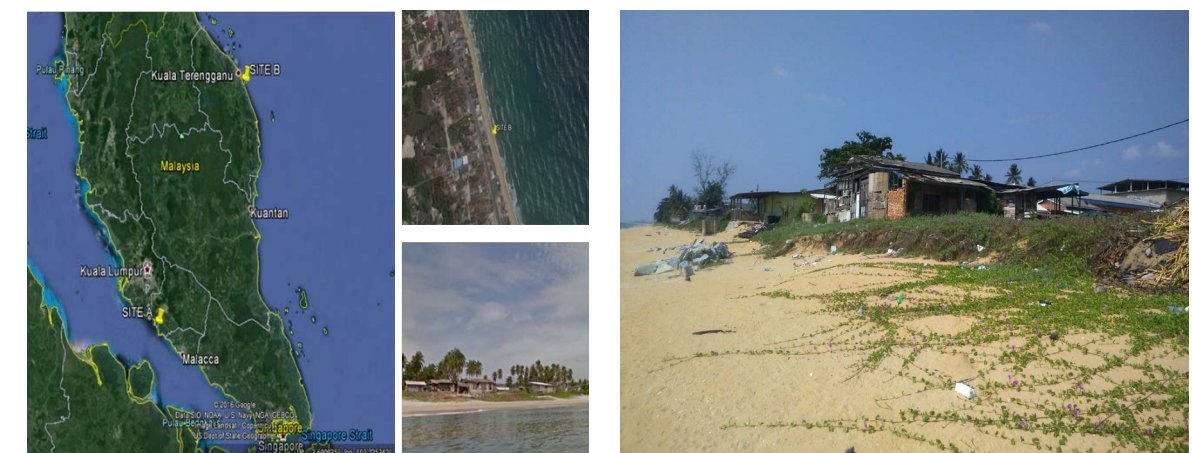


Figure 2: Coastal erosion within village houses at Site B, Marang, Terengganu

Whereas Site B is situated at the east coast of Peninsular Malaysia, facing South China Sea as in Figure 2. The shoreline is exposed to heavier wave and higher water level impacts during Northeast monsoon (November–February). The sandy beach of Pantai Rhu Muda, Marang, Terengganu is a traditional village community where fishing and small-medium scale fishing related-industry are the main socio-economic activities. The straight coast off Marang is usually exposed and severely eroded

during the monsoon but eventually stable and naturally nourished during calmer period (A.H.M. Rashidi, 2018).

Data Acquisition

Data acquisition involves of primary field data collection works and secondary data analysis. Primary data consists of tide level, current, wave, sediment sampling, bathymetry and beach profile survey is obtained directly from field works at both test sites. Equipment used include AWAC (Acoustic Wave and Current Profiler), tide gauge, sediment grabber, single beam depth scanner and RTK (Real Time Kinematic) satellite navigator by NAHRIM technical team. Secondary data namely tidal constituents, long term wind and wave data is gathered from other relevant authority departments such as JUPEM (Jabatan Ukur dan Pemetaan Malaysia), PHN (Pusat Hidrografi Nasional) and UKMO (United Kingdom Meteorological Office). Secondary data is later analysed to produce the Mean Sea Level, Mean High Water, Mean Low Water and maximum wave height parameters. For physical model platform setup, beach profiling survey data is used along with downscaled water level and sediment size. Data in laboratory refers to the beach profile measurement which was conducted before and after each simulation. Collected data in wave flume is later analyzed to evaluate beach profile changes due to hydrodynamic interactions.

Product Design

NEXC Block is a semi-trapezoidal shape solid structure made of reinforced concrete and purposely designed to control coastal erosion problem. The special features of NEXC Block are heavy and solid block to provide ample protection against big waves. Slanting hollow sections at surface is purposely designed to dissipate wave energy at the same time allowing flowing of water with sediment through the block. This hence encourages sediment trap leading to

natural beach accretion or expansion in a given time frame. The dimension of full size NEXC Block is 1.2m height x 1.2m width x 1.2m length. Surface block angle is 45° with minimum factor of safety 2.4 against sliding and overturning failures. Approximate weight is 2,500kg @ 2.5 tonnes with marine concrete Grade 40kN/m². The reinforcement bars are H and Y type with concrete cover of 50mm.

Reinforcement bars are essential to avoid structural failure during logistics and handling due to the weight. The system is also provided with interlocking system of male and female lock design to minimize the overturning and sliding impacts. Medium size armor rocks and crusher run wrapped in geotextile blanket are used as the base of the NEXC Block system. For protection against scouring impact, armor rocks are also placed as apron along toe of the structure.

For laboratory testing, the 1:10 downscaled dimension is 0.12m height x 0.12m width, where in length the structure models are arranged to fit the 2.5m flume's breadth. Concrete Grade 30kN/m is used with proportionate reinforcement bars. As given in Figure 4, surface angle of 45° is preferred as the most suitable for all scenarios at both proposed sites.

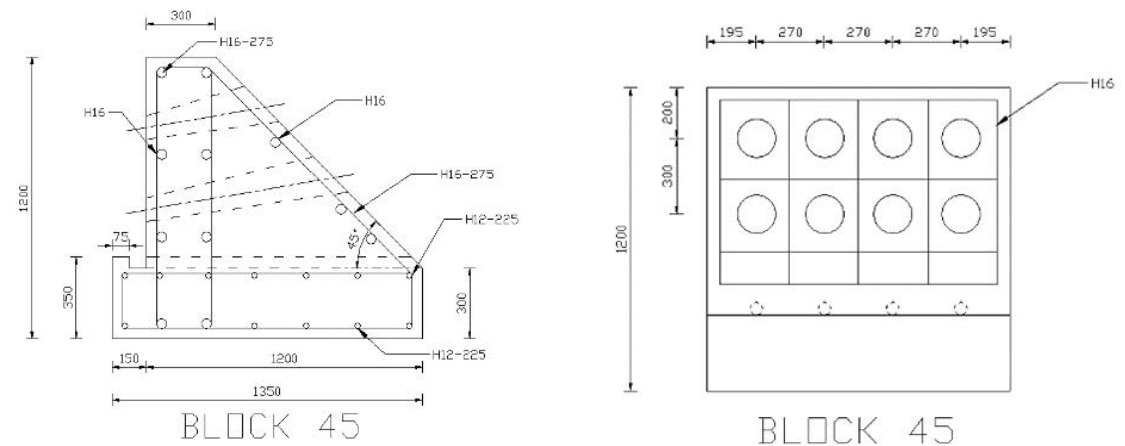


Figure 3: Side and front views of the 45° NEXC Block unit



Figure 4: Downscaled and full size model of the 45° NEXC Block unit

Test Parameters

In this experiment, geometrical downscaling for structure and sea level was set at 1:10. Dynamic downscale and Froude scaling were set as default as provided by the wave paddle equipment and software. Test parameters were decided based on the suitability in determining structural – wave – water level impacts towards the process of erosion and accretion within the installation area. Setting of water level for Mean Sea Level is based on the center line and existing dimension of flume facility. Difference in water level in the laboratory

relates to the actual tidal measurement and height of structure. Parameter of waves described the extreme and calm period, which also be referred to the Northeast and Southwest monsoon effect. Due to time constraint, downscaled D50 sediment size of 100 μ m and beach profile slope of 1:15 were determined from actual condition at site B. Simulation time was justified based on equilibrium time for significant beach profile changes in a given one peak period of tidal cycle at both sites (A.H.M. Rashidi, 2017).

Experimental Setup

Physical modelling experiment and product testing was carried out in two dimensional (2D) wave flume with the dimension of 100m length, 2.5m width and 2m height located at NAHRIM. The experimental design setup was shown in Figure 5.

Table 1. Summary of test parameters for physical modelling experiment

Parameter	Actual	Model
Geometric	1:10	
Length (m)	1.2	0.12
Height (m)	1.2	0.12
Width (m)	1.2	0.12
Mean High Water, MHW (m)	2.47	1.1
Mean Sea Level, MSL (m)	1.45	1.0
Mean Low Water, MLW (m)	0.43	0.9
Beach Slope	1:15	1:15
Dynamic Down Scale	Default	
Time (s)	180	60
Beach Material Size (m)	0.07	100 μ
Extreme Wave Height, HsE (m)	2.0	0.2, 0.1
Calm Wave Height, HsC (m)	1.0	0.1
Wave Period, Tp (s)	6	2

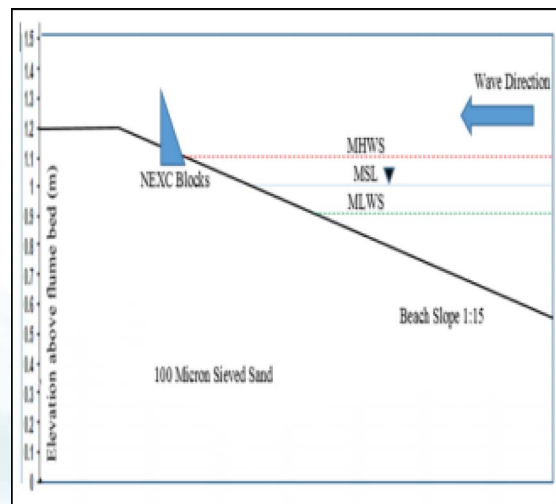


Figure 5: Flume set-up for physical modelling experiment

This paper discusses only three main simulation scenarios as summarized in Table 2. Each scenario involves 6 test runs and thus a total of 18 tests were carried out. Every test run took about 60 minutes which is equivalent to 192 minutes at site; which covers one tidal cycle peak period. In order to minimize the error, simulation was conducted until it reached the equilibrium phase. This refers to no significant change in the characteristics of scour dimension on the beach profile.

Table 2: Summary of three simulation scenarios

Scenario	Water Level (m)	Wave Height (m)	
Scenario 1	MLW		
(Vacant, No structure)	MSL	HsE	HsC
	MHW		
Scenario 2	MLW		
(Vertical Coastal Wall)	MSL	HsE	HsC
	MHW		
Scenario 3	MLW		
(NEXC Block model)	MSL	HsE	HsC
	MHW		

Beach profile measurement is conducted before and after each simulation for an area of 1.5m x 0.5m of front and back of model structure installation respectively. Manual measurement equipment namely theodolite, laser scanner and ruler are used. Changes in beach profile were analyzed quantitatively and compared qualitatively based on actual condition at site.

Site Monitoring Surveillance

Scheduled site monitoring was carried out regularly by NAHRIM technical team to ensure the integrity of the structure is continuously acceptable after the construction work completed. Any formation of failure, distortions, instability or deformation occurs to these structures were recorded, marked and measured for analysis. If the damage is considered critical, hence repair and recovery action must be taken immediately.

At the same time, beach profile survey is also conducted to determine any changes to the shoreline. Beach profile survey is an analytical method for quantitative comparison to determine the coastline changes. During the survey works, the coordinates of each point of measurement and transect

line is recorded using the reference datum DTGSM (Geodetic Vertical Datum of Peninsular Malaysia) for monitoring purposes by RTK surveying tool set shown in Figure 6. Survey works on beach profile was set within 3 days after spring tide cycle.



Figure 6: Beach profile measurement at both sites during lower water of spring tide cycle

Results and Discussion

NEXC Block Structure Stability Analysis

Table 3 showed the moment acting on coastal block for stability analysis on overturning check. The acceptable factor of safety for overturning is set minimum at 2.4. The force acting on coastal block for stability analysis on sliding check are summarised as shown in Table 4. The stability of coastal block is analysed using full scale data collected from site and secondary data where necessary. Hence the design of coastal block must be sufficiently stable against overturning and sliding failures. The calculation of wave forces on coastal block is according to EM 1110-2-1100 (Part VI), Goda formula for irregular waves (Goda, 1974; Tanimoto et al., 1976). Overturning failure is rotation of wall about its toe due to exceeding of moment caused due overturning forces to resisting forces. Eq. [1] show the factor of safety against overturning is given by:

$$F_0 = \frac{\sum M_R}{\sum M_0} > 1.5 \quad (1)$$

$\sum M_R$ = sum of resisting moment about toe
 $\sum M_0$ = sum of overturning moment about toe

The factor of safety sliding may be expressed by Eq. [2] below, which generally a minimum factor of safety of 1.5 is required:

$$FS = \frac{\sum R_V \tan \delta + k c_2 B}{\sum R_H} \quad (2)$$

where,

$\sum R_V$ = sum of the vertical resisting forces
 $\sum R_H$ = sum of the horizontal resisting forces
 δ = angle of friction between the soil and base slab
 k = the range between 1/3 to 2/3
 c = cohesion of soil
 R = resistance force
 B = bottom length of structure

Table 3: Overturning check on coastal block structure at point 0

Design	θ	Moment at 0		$\frac{\sum M+}{\sum M-}$	Remark
		$\sum M+$ (kNm)	$\sum M-$ (kNm)		
2	45°	18.1	7.58	2.4	Ok

Table 4: Sliding check on coastal block structure at point 0

Angle (°)	$\sum R_V$ (kN)	$\sum R_H$ (kN)	B (m)	FS	Remark
45	40.2	15.36	1.2	2.8	Ok

Physical Modelling Experiment Result and Analysis

The result of the physical modelling in laboratory is based on three main scenarios as mentioned earlier. Bigger wave height (H_s) caused wave breaking to occur more shoreward for all cases of water level (MLW, MSL & MHW). Bigger wave high has more impact due to bigger force, more energy, higher speed and wave run up. Larger wave also causes more impact to bed scour hence erosion will occur far inland. In these scenario, the erosion condition is dominant compare to accretion condition. However it is observed that the accretion of sediment occurred beyond breaking wave and bed scour area as shown in Figure 9. More accretion of sand sediment occurred in the cases of lower water level and calmer wave scenario. For scenario 3 simulation with High Water Level and Extreme Wave, severe scour occurred and sediment loss in front of the toe area of the NEXC Blocks Structure as shown in Figure 7.

It was found that in the model structure was able to withstand wave force and managed to reduce the impact of erosion especially at the back side of installation, as compared to Case 1 vacant beach scenario. However scour problem was prevalent at both Case 2 and Case 3 scenarios, in which scour depth was found deeper at vertical coastal wall as compared to NEXC Block simulation as Figure 10.

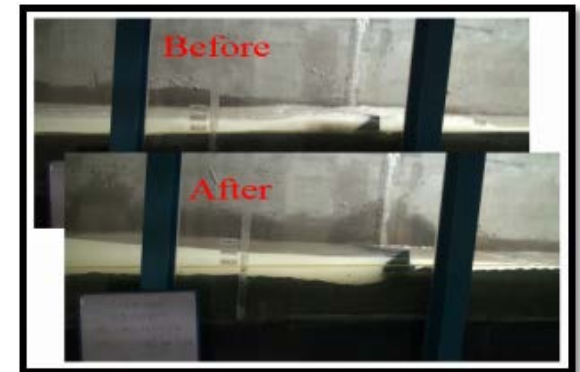


Figure 7: Testing Condition of Scenario 3 with HWL and HsE

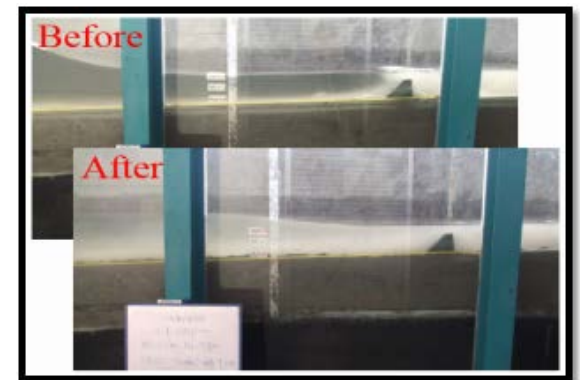


Figure 8: Testing Condition of Scenario 3 with HWL and HsC

Figure 8 showed that some accretion of sediments are noted during Calm Wave simulation at all water level cases. Accretion also occurred at the back side of the product installation especially during low water level. However, due to the limitation of no sediment supply added to this test model, sediment transport element was not taken into account in this experiment. In addition, the impact of beach erosion and scour is reduced by introducing additional toe protection such as crusher run acting as armor rock and artificial toe straw as presented in Figure 11.

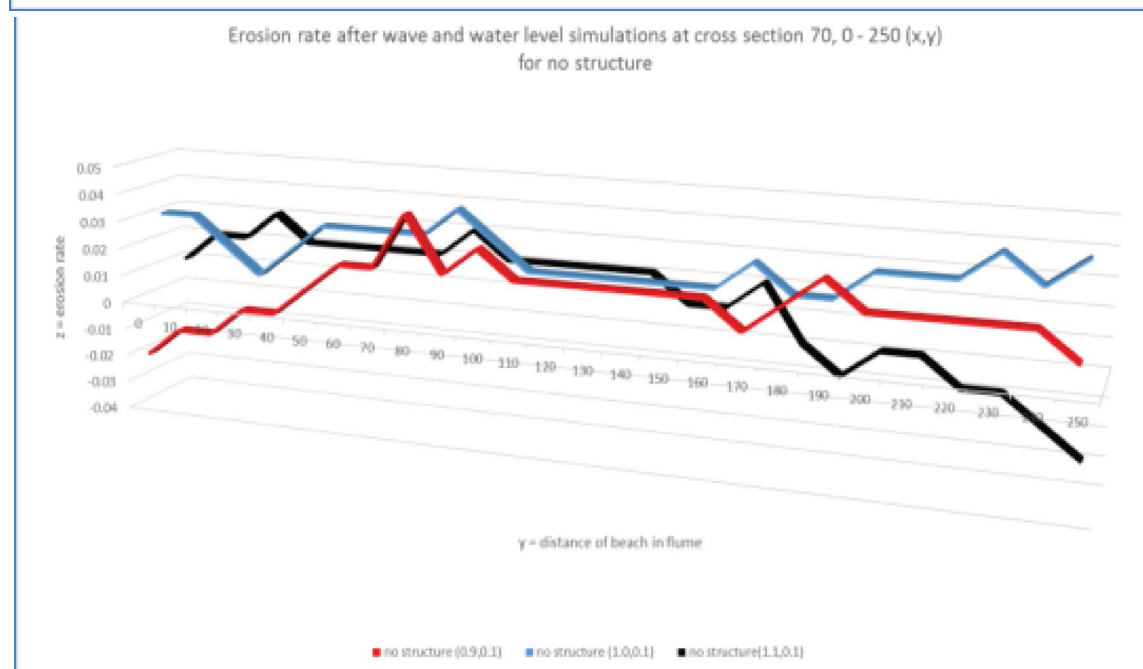
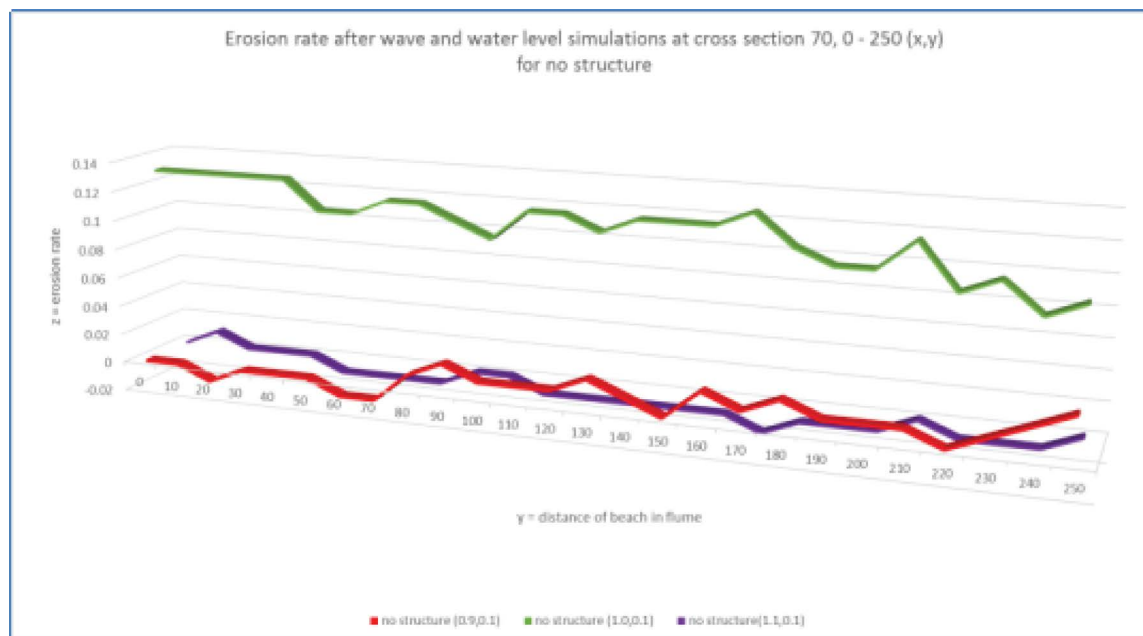


Figure 9: Beach profiles at vacant beach after simulations completed (control)

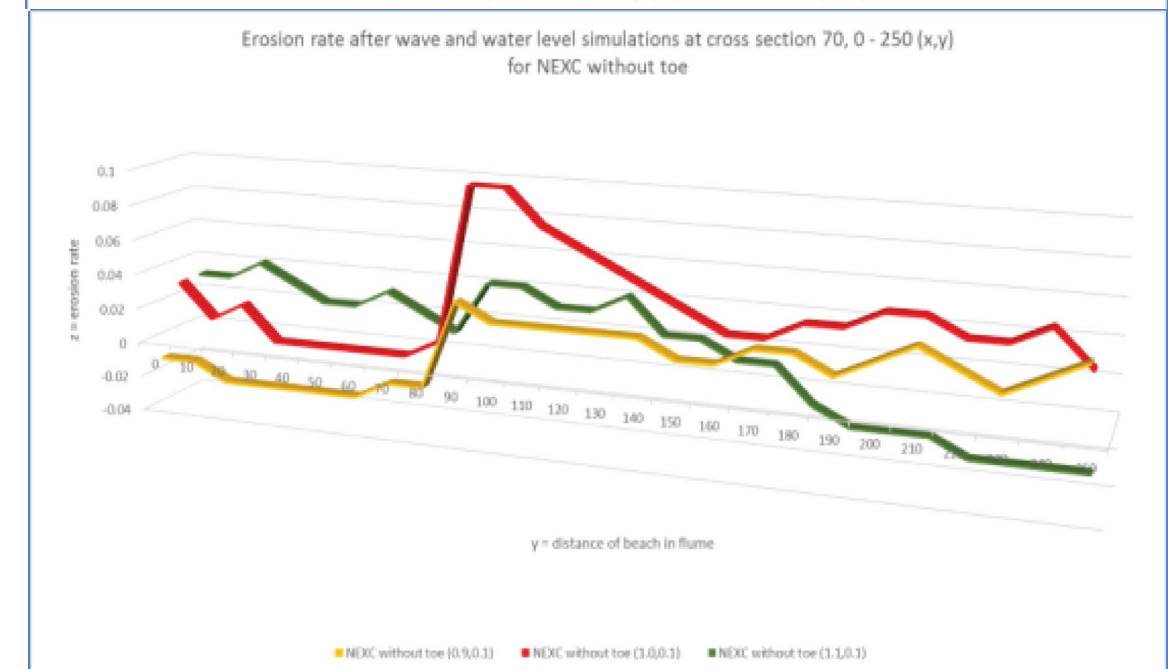
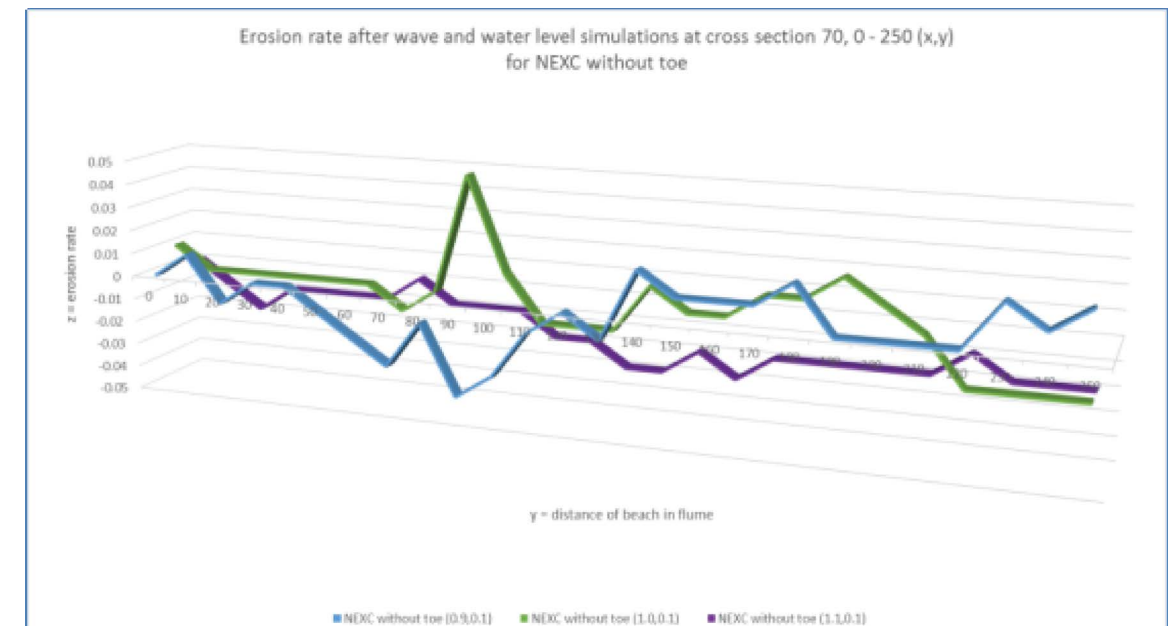


Figure 10: Beach profile changes with NEXC Block protection



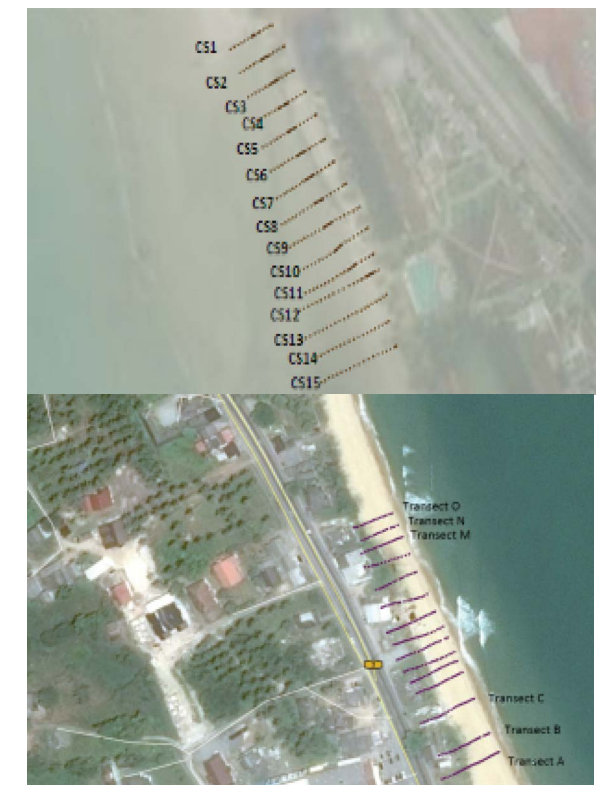
Figure 11: Beach profile changes with additional scour protection

Beach profile changes

Perpetual survey works for beach profile changes monitoring were carried out after the completion of installation works at both sites. In each site, 15 transect lines were created with an interval of approximately 10m interval as shown in Figure 12. The length of each interval line depends on the dry boundary to water mark limits. Interval lines at Port Dickson were named Cross Section 1 – Cross Section 15, whereas in Marang labelled as Transect A – Transect O. In this paper, the assessment of beach profile change was conducted based on selected one profile interval line for each site, which are Cross Section 6 and Transect E (Hisham, 2018).

Plotted graph of beach profile survey for Cross Section 6 at Site A, Port Dickson is shown in Figure 13. It is noted that the pattern of beach profile is almost similar for all cross sections. From the overall result, it is observed that beach profile undergoes a very dynamic changes within a short period of time due to the wave action and limited sediment transport processes as well as monsoon driven factor. This is due to the specific location of structure installation which is within mean higher sea level range. It was found that with product deployment, accretion occurred for short period of time (November 2015 to April 2016) in front and at the back of the NEXC Block especially during calm period as in presented in Figure 14. Sediment supply moving towards the shoreline from offshore sand bar during off monsoon season but erosion takes place during higher water level and higher wave events. Towards the end of the year 2016, as some series of king tides with large wave incidents along the coast of west coast of Peninsular Malaysia, severe erosion occurred during South West Monsoon period along Port

Dickson beach. The bulk of trapped sediments behind the NEXC Blocks were washed away during those period. Based on observation the cycles of natural sediment accretion or nourishment is still occurring until now but with little progress due to short of sediment supply and structural blockade which are a discharge drain at north and headland reclamation within the southern regions (A.H.M. Rashidi et.al, 2018).



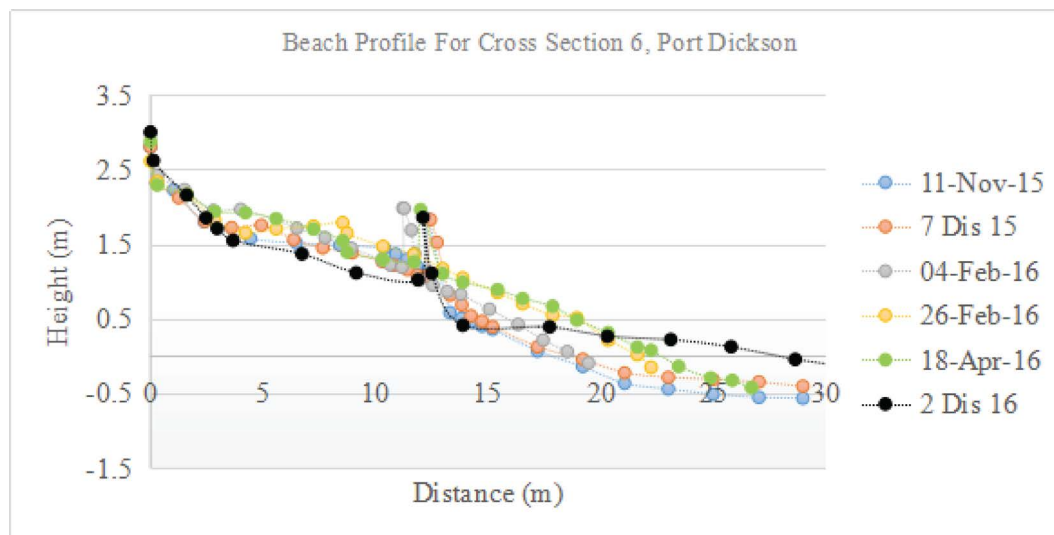


Figure 13: Beach profile for Cross Section 6 (CS6), Site A Port Dickson

The installation of NEXC Block has successfully protected the hotel main area from erosion impact. NEXC Block acted as wave breaker hence avoiding the impact of wave from reaching the paved road, swimming pool and outdoor restaurant compound. The influence of structure orientation in mechanism of natural sediment accumulation could be improved, as it was recorded that some accretion was profound

in specific scenarios. One more issue which should be highlighted in this study is the scour and base problems. The placement of scour protection in form of medium size armour rock in front and beneath of the structure is inadequate to overcome the impact of during extreme event. Hence for future development, artificial scour protection structure and footing system are recommended.



Figure 14: Condition of at Site A during extreme event (exposed toe) and calmer period (accretion of sediment)

Figure 15 presented the plotting of Transect Line E for site B, Marang, Terengganu. The changes in beach profile is mainly due to the sediment loss during North East Monsoon. During calmer period especially from Mei to July each year, the eroded beach will be recovered naturally. This impact as shown in Figure 16 where exposed armor rocks are prevalent during monsoon, in which are covered in sediment outside monsoon season. Some coastal vegetation have started to regrow, covering NEXC Block at the stable beach during those period.

It also should be noted that the dynamic changes due to NEXC Block at Site B is not as high as Site A, which is due to the specific location of structure installation. In Marang, NEXC Block is placed within Highest Astronomical Tide level hence the structure – wave – water – sediment interactions are minimized. Since the installation in 2015, NEXC Block has successfully provide ample protection to the surrounding village houses. The mechanism of sediment trap is also found successful hence maintaining existing ecosystem along the beach.

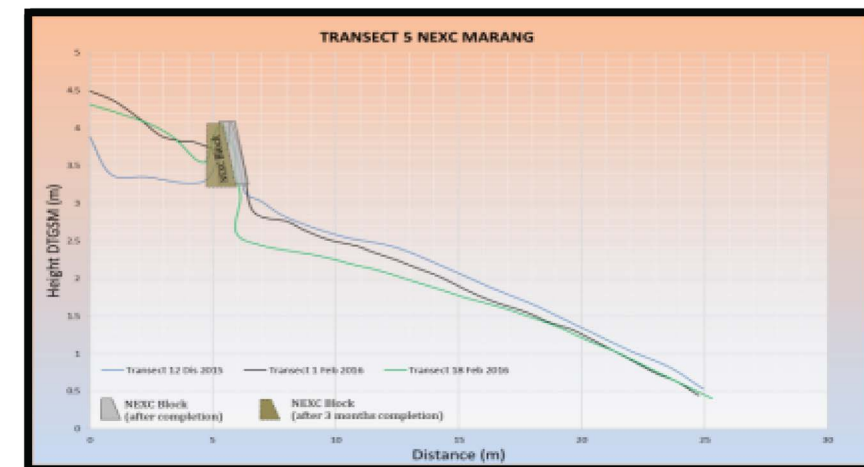


Figure 15: Plotting of beach profile changes for Transect Line E, Site B, Marang



Figure 16: Condition of Site B in calmer period vs during North East Monsoon

Conclusion

Based on the study which was conducted both in wave flume facility and on site analysis, some conclusions are presented. Higher water level led to erosion farther landward. Higher wave magnitude caused greater erosion level or scarp height. Rate of erosion is quicker and higher at a steeper slope and higher water levels conditions, until it reaches equilibrium. This provided more understanding of monsoonal effect towards the beach and the interactions of structure – wave – water – sediments in a controlled environment. The mechanism of erosion, accretion and scour was found comparatively almost similar at sites with in laboratory.

NEXC Block was found successfully in controlling erosion to occur only at the face of structure installation, the back area is fully protected. Other facilities and infrastructures both at hotel compound at Site A and village houses at Site B are well protected. However the mechanism of sediment trap for beach expansion should be improved. This include the improvement on artificial toe protection and base footing system.

At Site B, Marang Terengganu, erosion and scour occurred only during monsoon season. During calm period, sediment are returned naturally. Existing sediment at the back of structure is preserved. This scenario represented structure installation at beyond upper MHHW in wave flume. While at Site A, Port Dickson Negeri Sembilan, erosion and scour occurred during high water and high wave throughout the year especially during monsoons. During calmer period, patches of sediment return are recorded as the area not having enough sediment supply. Disturbance in sediment supply due to man-made structures is also a potential source of erosion within the specific beach area. Natural sediment accretion or beach

expansion was only happened during very calm period. One of the proposed methods to stabilize the shoreline is beach nourishment works.

Acknowledgements

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PERFORMANCE OF THE HYDRAULIC EFFICIENCY FOR DIFFERENT GRATE CONFIGURATION

Nuridah Sabtu¹, Sarah Alia Md Wakif¹

ABSTRACT

Gully grates are an important feature of the gully system since it permits the passage of water into the gully pot and hence the drainage system. The grates used in a gully system are an important factor in determining the overall efficiency of the system. A good drainage system is crucial in order to ensure road safety as well as reduces the occurrence of accident rates. It is observed that one of the factors that caused urban flooding in Malaysia is due to the poor drainage design in which the drainage system is insufficient to cope with the high amount of storm water runoff during continuous and heavy rainfall events. The need for this study arose to acknowledge the authorities about the importance of a proper/ resilient drainage design. The aim of this study is to determine the hydraulic efficiency for different grate configuration. A full-scale laboratory system was designed and built comprising of a testing platform that is connected to 2 smaller tanks on both left and right side of the platform to serve as an inlet/outlet tank. The results showed that Grate A has higher efficiency as compared to Grate B for the flat bed and 1/100 slope, but with similar efficiency at a higher slope.

Keywords: Urban flooding, Grate configuration, Hydraulic efficiency

Introduction

The occurrence of flooding event has brought a national outcry for the betterment of not just flood mitigation measures, defences, and awareness but also of flood prediction methods as well as flood maps. This consequently brought forth the idea of the importance of knowledge of storm frequency, gutter flow and inlet capacity and surcharge conditions for the purpose of a good design of surface drainage systems. Without an adequate regard for the surface drainage system, implementation of expansive hydraulic structures might not be able to function optimally because the intended flow did not get into the network (Gomez et. al., 2006).

According to BS 752, flooding is a condition where wastewater and/or surface water escapes from or cannot enter a drain system and either remains on the surface or enters buildings. Surcharge on the other hand is described as a condition in which waste water and/or surface water is held under pressure within a gravity drain or sewer system, but does not escape to the surface to cause flooding.

Extended surcharge conditions may eventually lead to a rise in the water level above surface where water either escapes from the sewer system or prevents surface water from entering the system (Schmitt et. al., 2004). The occurrence and possible effects of surface flooding depend much more on local constraints and surface characteristics such as street gradient, sidewalks, as well as kerb heights. These characteristics however are much more difficult to describe physically and the data are not available in practice. This urban runoff process is also described to be a two-phase phenomenon, incorporating a surface phase with an underground phase (Kidd and Helliwell, 1977; Gomez et. al.,

2011). Existing simulation models are not fully adequate to simulate the relevant hydraulic phenomena associated with surface flooding especially in representing the relationship between inlet capacity, inlet efficiency and surcharge conditions of different types of gully systems commonly used in practice. The shortfall in this knowledge could produce unreliable simulations from storm water management models (Russo et. al, 2005; Gomez et. al, 2011). This study was therefore proposed to address the lack of knowledge between the two phases of the urban runoff process and to give an insight of this complicated relationship with the hope that the outcome is of use to further develop the existing flood prediction models. Due to the expansive nature of the study that could be carried out, this study will focus only on one structure, which is the gully system.

An essential feature of a highway drainage system is that it should provide a route for storm water to flow along from the highway to a suitable discharge point known as the outfall. However, before reaching an outfall, storm water from the catchment is transferred to the sewer drainage network via gully system, which comprises of gully inlets/grates and gully pots. As have been previously mentioned, there is a lack of understanding on how inlets perform hence leading to a need for further research to improve on the shortfall in knowledge. The importance of this knowledge is that it ultimately determines and regulates the extent and the damage due to flooding on the surface network caused by sewer surcharge (Leandro, 2008).

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Theoretical Information

Type of Gullies

Gullies can be classified according to its hydraulic operation, design or the position of the gully system itself. British Standard (BS 5911:2004) has identified two main types of gullies according to its design. The more common type is the trapped gully, as used in this study, which consists of a hollow cylinder concrete with a base with an outlet designed to form a water seal and a rodding eye – which is an outlet to facilitate the connection of a pipeline [Figure 2.1(a)]. The less common gully system [Figure 2.1(b)] – which is the untrapped gully, is similar to the trapped gully but lacks the water seal outlet.

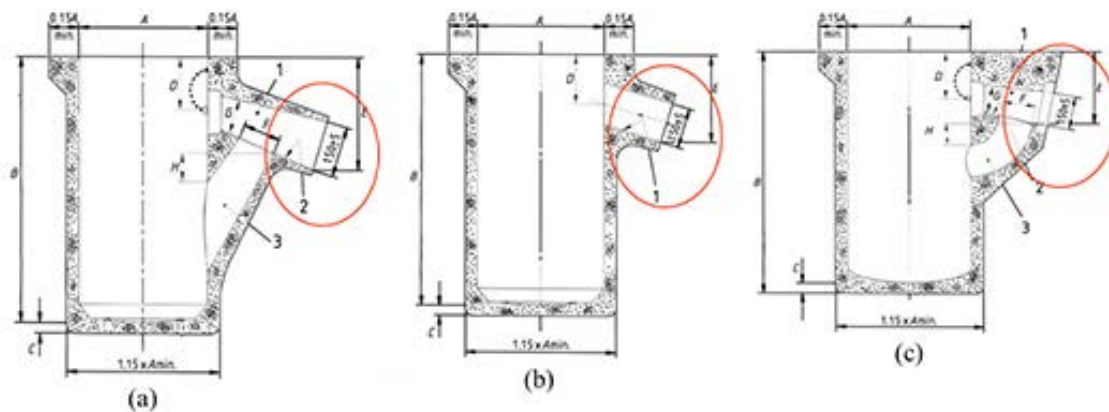


Figure 1: (a) Typical trapped gully with spigot; (b) Untrapped gully with spigot outlet; (c) Trapped gully with socket outlet (BS 5911:2004)

In a less significant way, gullies can also be classified according to the design of the final outlet. The gullies highlighted in Figure 1(a) and 1(b) show a similar outlet type, which is the spigot outlet, whereas Figure 1(c) highlights a gully with a socket outlet. In this study, the gully type shown in Figure 1(a) was used. The trapped gully with spigot outlet was chosen for the purpose of this study since it is the most commonly used and will be the best choice to represent the gully frequently found in practice.

Another classification made by Design Manual for Road and Bridges (DMRB) classifies gullies into 2 modes based on its hydraulic operation. Based on this classification, the design of gully spacing is made simpler. The gullies are classified as (HA 102/00):

(i) Intermediate gullies – gullies that permit some portion of the approaching flow to pass and are picked up by the next downstream gully.

(ii) Terminal gullies – gullies with no significant portion of flow to pass through it possibly due to the inexistence of another downstream gully or due to the interference to traffic/pedestrian if allowed.

Another form of gully is the sumpless gully and is different from the conventional gully because of the position of its outlet. The sumpless gully is a means of directing flows to the drainage system via discrete entry points without the construction of a sump for the purpose of trapping debris, sediments and oils. Sumpless gullies can be divided into two distinct types – chutes and pots. Both have similar characteristic in terms of their outlet position but they differ geometrically. The outlet, as can be seen in Figure 2, is situated at the base of the gully pot hence not allowing it to have the ability to retain

both oil and sediment. The base of the pot is designed in such a way that the flow of water is allowed to be channelled to the outlet directly minimising sediment build up at the edge of the pot base (HA 105/04). Chute type sumpless gully have similar plan area as its grates and frame but has a sloping face at the bottom end of the chute with a longitudinal outlet pipe incorporated into it. Pot type sumpless gully however, like its name has a cylindrical body with a rounded base shaped in such a way that all the flows are directed to the outlet. Chute type gullies are usually used on flat roads in low-lying areas where adequate pipeline gradients for drainage are difficult to obtain (HA 105/04). A more detailed description of sumpless gullies can be obtained from DMRB HA 105/04 – Sumpless gullies.

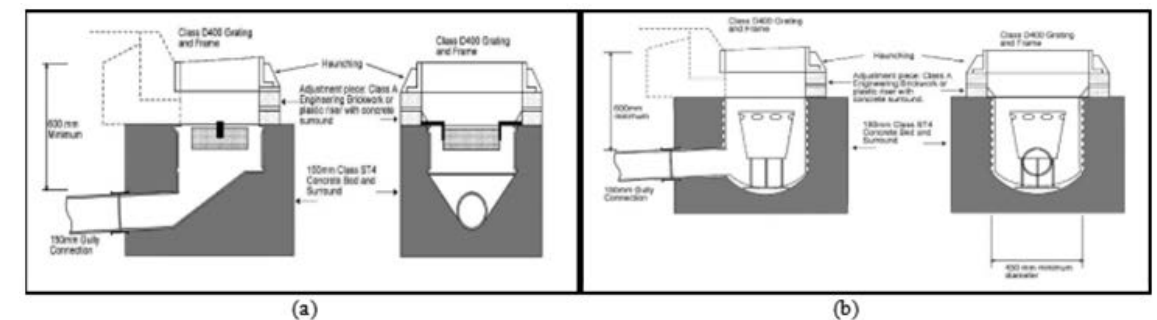


Figure 2: (a) Chute type sumpless gully; (b) Pot type sumpless gully (HA 105/04)

Storm water Inlets/ Gully Grating

Storm water inlets or also known as gully grates are an important part of an urban drainage system, since its primary role is to control the amount of water conveyed from the ground surface to the sewer network. The capacity of the inlets therefore determines the efficiency and reliability of the whole storm drainage system. Inlet capacity defines the largest quantity of gutter/overland flow that can be captured by the inlet. This definition assumes that the functionality of the inlet is not affected by

conditions in manhole, pipe or any other device, or by backwater effects and surcharging (Despotovic et. al., 2005). According to (Stephenson, 1981), there are two basic inlet configurations which are the vertical inlet or more commonly known as kerb inlet and horizontal screens or grated inlet. Kerb type inlets offers practical advantages to traffic but is less efficient hydraulically whereas horizontal screens are easily subjected to damage by heavy traffic.

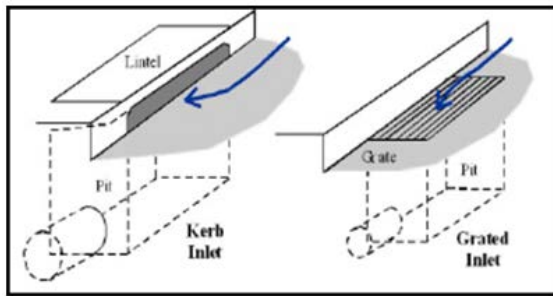


Figure 3: Kerb inlet and grated inlet (Pezzaniti et al., 2005)

Haestad and Durrans (2003), however classifies inlet openings into four general types, which are grate inlets, kerb inlets, combination inlets, and slotted drain inlets as shown in Figure 3. Grate inlets consists of an opening in a gutter or swale and is covered with one or more grates whereas kerb inlets have vertical openings in the kerbs and is covered by a slab. Combination inlet, as the name goes, combines both grate and kerb inlets configuration. Slotted drain on the other hand consists of a grate opening oriented along the longitudinal axis of a pipe and is manufactured integrally with the pipe itself. For the purpose of this study, only grate type inlets will be used. In terms of location, storm water can be classified into – in-sag or on-grade inlets. This classification is similar when compared to the classification based on the hydraulic modes of gully systems since in-sag inlets acts as a terminal gully which does not permit any significant amount of flow to pass through it whereas on-grade inlet acts as an intermediate gully which allows part of the flow that passes it to be captured by the downstream gully. Proper attention should be given to the conveyance of drainage water through sag inlets, because they frequently encounter water ponding which tends to increase the water spread over the pavement (Almedeij et al., 2006). There are many characteristics, which have to be taken into consideration while

designing a storm water inlet. Some of the factors include its hydraulic characteristics, loading class, hydraulic efficiency, orientation, dimensions, and road gradients as well as clogging factor.

Efficiency of Gully Grates

The efficiency of gully grates is a function of the approaching flow, the captured flow as well as the intercepted flow. In circumstances where the approaching flow is smaller than inlet capacity, it is often assumed that the designated inlets are able to capture the whole approaching flow. In circumstances where the approaching flow is greater than inlet capacity however, the inlet captures only a portion of the approaching flow, while the remaining flow passes over the inlet and is then captured by the downstream inlet. This relationship is as seen in Figure 4 and is used to describe the efficiency of gully grates.

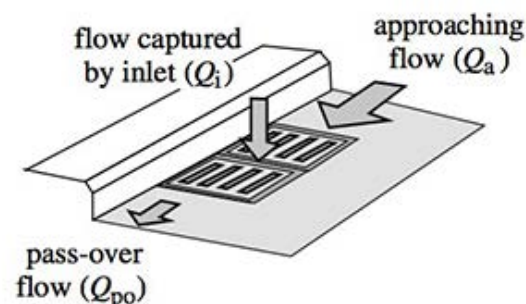


Figure 4: Definition of flow components (Despotovic et. al., 2005)

The efficiency of an inlet (η) is the ratio of the intercepted flow against the approaching flow towards the gully system. In simple terms, the efficiency of an inlet is given as (Haestad and Durrans, 2003; Despotovic et. al., 2005; Valentin and Russo, 2007):

$$\text{Efficiency } (\eta) = \frac{\text{Intercepted flow } (Q_i)}{\text{Approaching flow } (Q_a)} \quad (1)$$

Bypass flow (Q_b) or the discharge that has not been intercepted by the stormwater inlet can be defined as (Haestad and Durrans, 2003):

$$Q_b = Q - Q_{int} \quad (2)$$

There are also circumstances when the inlets are unable to capture the whole approaching flow although it is less than the capacity of the inlet. This perhaps is due to malfunction, which often reduces the capacity of the inlets. Malfunction of inlets which reduces partially or completely the capacity of gully grates are often caused by (Despotovic et. al., 2005):

- (i) inadequate position of the inlet in relation to the kerb or street other elements,
- (ii) deformation of the surrounding street pavement due to high temperatures and/or heavy traffic,
- (iii) clogging of the inlet grate openings by leaves or other debris,
- (iv) clogging of manholes, pipes or any other device downstream of the inlet.

Proposed Method

Laboratory Description

For the purpose of this experimental programme, a full-scale laboratory system was designed to mimic the hydraulic interaction between the above and below ground drainage system via gully inlets. This system was designed with a small catchment area. The laboratory system consists of a testing platform that is connected to a smaller tank on both left and right hand side of the platform to serve as an inlet/outlet tank depending on the system to be tested.

The testing platform is a rectangular platform 4.27m (L) x 1.83m (W) and drains a total area of 7.814 m². The dimension for the inlet and outlet tank itself is 0.61m (L) x 2.44m (W). Both of these tanks are each equipped with a sluice gate to allow control of the hydraulic depth on the testing platform. Both the outlet tank and the outflow from the gully system itself are each connected to a measuring tank, which allows manual measurements of flow rate to be taken. Each of the concrete measuring tank have the internal dimensions of 2m (L) x 1.25m (W) x 0.75m (D) and was an existing feature in the hydraulic laboratory (Unwin, 2008). Both of the measuring tanks are situated at the lower level of the hydraulic laboratory where the overall testing rig was built on.

Gully Grates

Gully grates according to HA102 are classified based on their hydraulic capacity and are divided to 5 different types which are P, Q, R, S and T with their hydraulic capacity decreasing respectively. BS EN 124 has also outlined that in order to ensure a reasonable level of hydraulic capacity, the total waterway area of the slots should not be less than 30% of the clear opening of the grates. The tested grates should also meet the minimum waterway area of 900cm² – commonly used in practice in the UK as highlighted in HA 104/02. The loading class selected is for Group 3 (BS EN 124) – C250 that is suitable to be installed in the area of kerbside channels of roads (Table 1). In the first phase of the study, grate with clear opening of 400mm x 432mm (HA 102 – R) was used followed by grate with 325mm x 437mm clear opening (HA 102 – S) (Figure 5).

Hardware & Software Control System

A set of control system has been assembled in order to give/send signals to the different equipment utilised in the laboratory set up. The control system consists of the following - National Instrument Compact FieldPoint system (NI-cFP) and LabVIEW RealTime (RT) as the hardware system, LabVIEW (version 7.1) and National Instruments Measurement and Automation Explorer (NI MAX) as the programmable controlling software.

Compact FieldPoint System

The control hardware system that was utilised in this experimental programme consists of a single Compact FieldPoint system that is embedded with Real-Time LabVIEW. A Compact FieldPoint system usually consists of one cFP-BP-x backplane, one cFP-20xx controller, one or more I/O modules, and or more connector blocks or

accessories. Each of this system can be accessed by an unlimited number of PCs and FieldPoint modules, forming a distributed computing system (NI, 2004). Compact FieldPoint was designed for industrial control applications that perform advanced embedded control and data logging as well as providing Ethernet connectivity. Due to the nature of some of its components, the Compact FieldPoint can also perform headless operations when appropriately assembled. Compact FieldPoint, was boasted to be the most rugged and reliable NI platform, and was designed for industrial and mobile environments tolerating high shock, high vibration, and even temperature extremes. FieldPoint is a lower-cost distributed I/O system with a variety of communication options besides Ethernet. It is designed to mount on DIN rails in static applications where the FieldPoint bank is connected to a PC for data collection, analysis, display, and storage.

Table 1: Hydraulic properties of the proposed grates

BS EN 124 loading class	Clear opening A x B (mm)	Over base C x D (mm)	Depth, E (mm)	Waterway area (cm ²)	Total mass (kg)	HA 102 reference
C250	325 x 437	475 x 524	75	933	29	S
C250	400 x 432	550 x 530	75	1128	33	R



Figure 5: (a) Grate with 325mm x 437mm clear opening (HA 102 reference – S); (b) Grate with 400mm x 432mm clear opening (HA 102 reference – R)

Figure 6 shows an example of a Compact FieldPoint system with multiple set of I/O module and connectors block. Highlighted are the components that make up the Compact FieldPoint system that is utilised in this experimental programme.

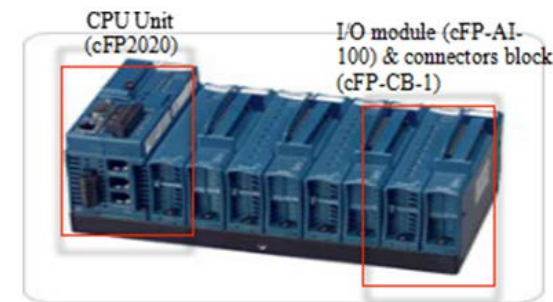


Figure 6: A Compact FieldPoint system (<http://www.ni.com/compactfieldpoint/>)

National Instruments Measurement and Automation Explorer (NI MAX)

National Instruments Measurement and Automation Explorer (MAX) is configuration software that is supplied with the Compact FieldPoint hardware. NI MAX is often used with NI application development programme such as LabVIEW to manage the low-level communications, hardware details, as well as simplifying programmatic access to the I/O channels. For the purpose of this experimental programme, the hardware components for the laboratory system is configured in NI MAX as a remote system which then allowed the author to programmatically design a set of control through Virtual Instruments (VI's) of the LabVIEW interface.

The NI MAX platform simplifies the use and integration of Compact FieldPoint systems with the utilised hardware. This is because it allows configuration of the entire system, including network parameters, module, and I/O settings, and named-channel items to be done in NI MAX itself. NI MAX also

detects the Compact FieldPoint as a remote system, which then allows for wireless configuration and tests of all the associated modules. NI MAX also allows interactive test of I/O modules and channels, viewing input data values, and setting of output values without writing any software code (NI, 2004). Figure 7 shows the item configuration window of the NIcFP-AI-100@6 as seen in NI MAX. This window allows the addition or removal of items/channels of the designated modules.

LabVIEW 7.1

The software monitoring system utilised in this experimental programme is the LabVIEW version 7.1. In order to communicate with the hardware, a simple user interface/front panel VI was designed. This VI has the control (knobs, dials, push buttons and etc.) as well as the indicator ability such as graphs, LEDs, and other output displays (LabVIEW manual). Codes are then added in the block diagram VI to control the objects in the front panel VI. Figure 8 shows a sample of the block diagram VI constructed to control the system. The block diagram VI shows a more detailed codes and signals used to control the front panel. In this case, the seven pressure transducers are being told to take measurements after a certain time delay (900s) at a designated time interval (1000ms) and results written to a LabVIEW measurement (.lvm) files. Measurements taken are also sent to the front panel and then displayed as graph and digital display.

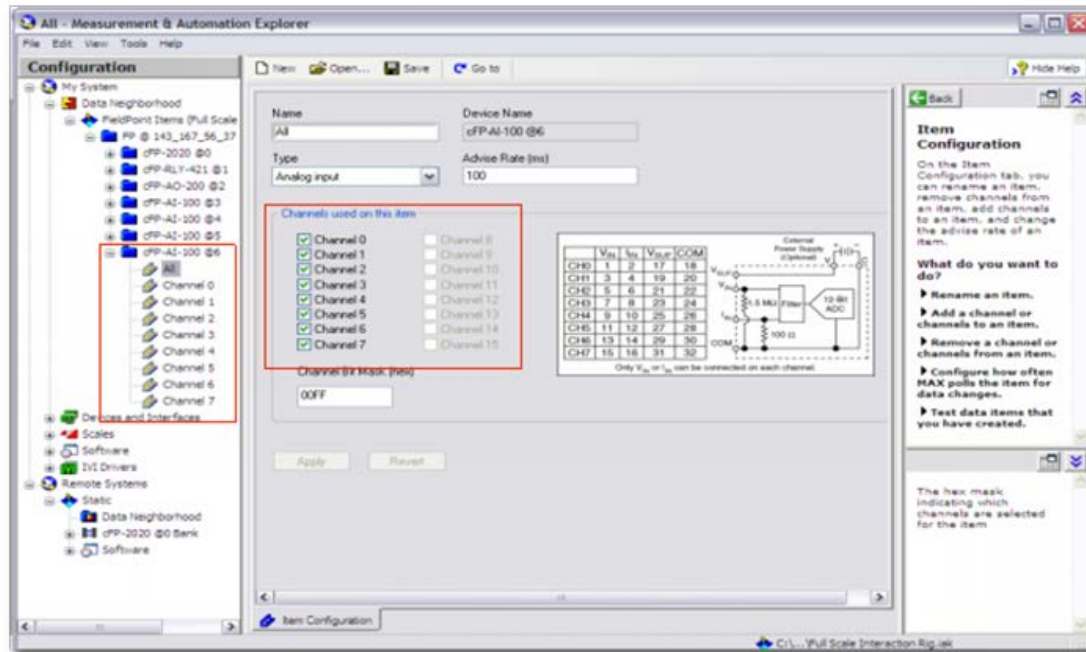


Figure 7: Configuration window of the NI cFP-AI-100@6

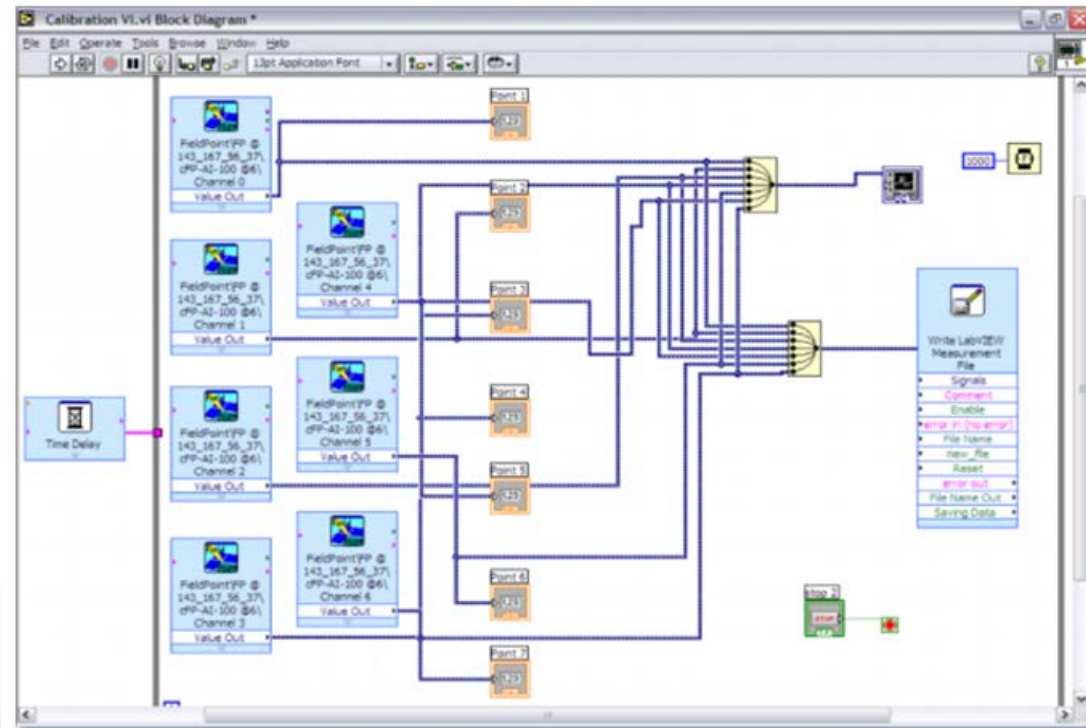


Figure 8: Block diagram VI

Results and Discussion

Efficiency of Intermediate tests

The efficiency of an inlet (η) is the ratio of the intercepted flow against the approaching flow towards the gully system. In simple terms, the efficiency of an inlet is given as (Haestad and Durrans, 2003; Despotovic et. al., 2005; Mustafa et. al., 2006; Valentin and Russo, 2007, Gomez et. al., 2011):

$$\text{Efficiency}(\eta) = \frac{Q_{im}}{Q_a} \quad (3)$$

There is also another method to describe the capture of the flow by inlets – by using the capture efficiency (η), which is defined as the ratio of the flow captured by the grating, Q_i to the total approaching flow, Q_a (Mustafa et. al., 2006). Presented in Figure 9 is the relationship between the head and the efficiency of the grates (in %) as a function of the flowrate (l/s) for both Grate A and Grate B. The heads are

denoted simply with points and correspond to the y-axis whereas efficiency is denoted as points with dashed lines, which corresponds to the z-axis. It can be seen that as the head increases the efficiency of the grates decreases. Figure 10 shows the comparison between Grate A and Grate B in terms of efficiency. It can be seen that the difference in efficiency between Grate A and Grate B decreases as the slope increases. It was shown that the head discharge relationship for Grate A resulted in a lower head for the same flowrate when compared to grate B and hence these results highlight consistency in that, as expected, Grate A is more efficient in capturing flows when compared to Grate B for the flat bed as well as the 1/100 slope. However, it can be said that the efficiency of Grate A and B are similar at higher grade – in this case, the 1/30 slope where both sets of results display a very good agreement of efficiency values with each other.

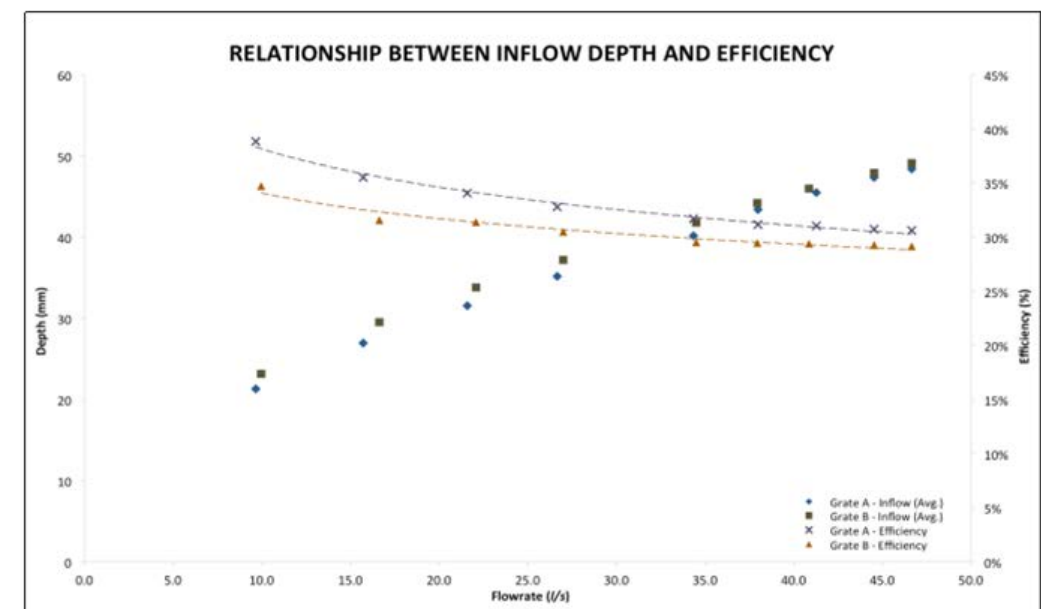


Figure 9: Comparison of flow depth and efficiency for Grate A and B with a flat bed

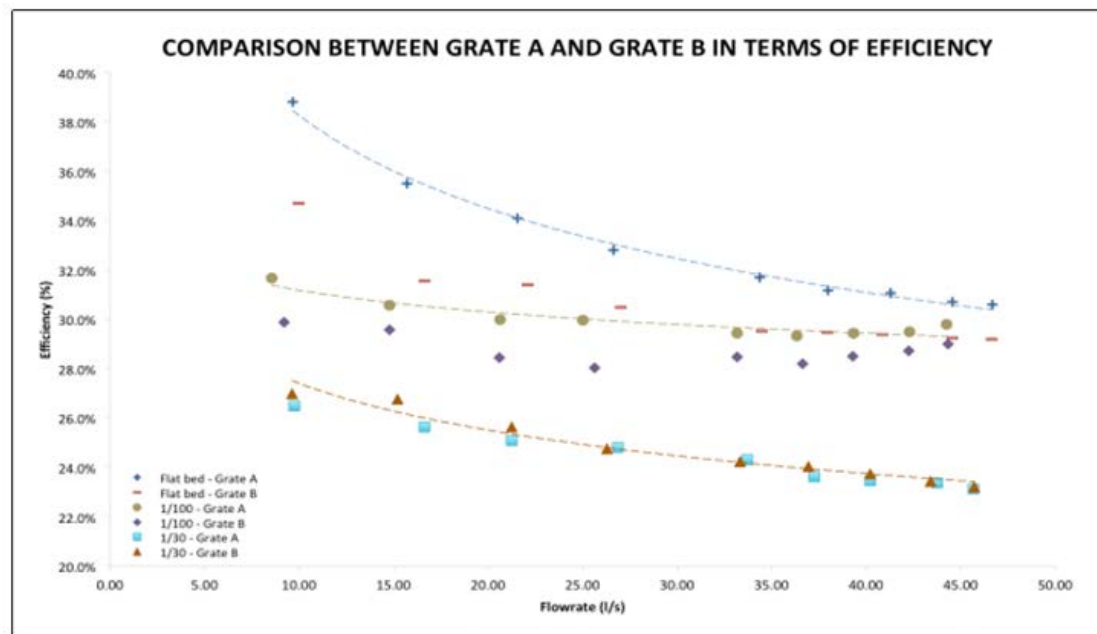


Figure 10: The comparison between Grate A and B in terms of slope and efficiency

Conclusion

The relationship between flow depth and flowrate was a function of the geometry of each individual grate but similar trends in the head discharge relationship for each grate were observed. The head discharge relationship for Grate A, in general, resulted in a lower head for the same flowrate when compared to grate B and hence concluding that Grate A is more efficient in capturing flows when compared to Grate B for the flat bed and 1/100 slope, but with similar efficiency at a higher slope. In terms of gully depth, the depth increases as the bed slope increases. However, the intercepted flow decreases as the slope increases. It showed that the width of the channel had little impact on the head discharge relationship at higher bed slope which is 1 in 30.

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