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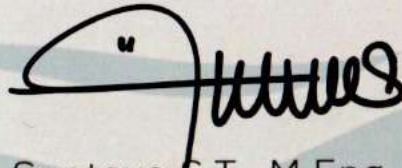
“The Influence of Additional Equipment to the Ultimate Strength of FPSO”

Presented

**In the 7th International Seminar on Ocean and Coastal Engineering,
Environmental and Natural Disaster Management**

ISOCEEN 2019

**Surabaya, Indonesia
6th and 7th of November**



Suntoyo S.T., M.Eng., Ph.D

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This document contains some comments of paper entitled : The Influence of Additional Equipment to the Ultimate Strength of FPSO

SECTION I: Comments per Section of Manuscript

General comment:	Good Paper
Introduction:	In the paper explained as follows: <i>In some years, about 70% of FPSO is produced from the conversion result.</i> Need to explain the data used.
Methodology:	Need to explain the finite element procedure used, the type of element, the assumptions used in the selection of element types It is necessary to validate the results of finite element modeling with exact solutions or experiments
Results:	It is necessary to add comparisons with the results of previous studies by other researchers
Discussion:	It needs to be explained more in every chart and figure available

SECTION II - Please rate the following: (1 = Excellent) (2 = Good) (3 = Fair) (4 = poor)

Bibliography/References:	ok2	
Contribution To The Field:	2	
Technical Quality:	ok2	
Clarity Of Presentation :	2	
Depth Of Research:	3	
Decision:	Accepted with minor revision.	

SECTION III - Recommendation: *(Kindly Mark With An X)*

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The influence of additional equipment to the ultimate strength of FPSO

Muhammad Zubair Muis Alie^{1,*}, Juswan¹, Wahyuddin Mustafa¹, Kevin Gabriel Pangalinan¹, and Nurul Inda Pratiwi²

¹Department of Ocean Engineering, 92171 Engineering Faculty, Universitas Hasanuddin, Makassar, Indonesia

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Abstract. Nowadays, there have many ship conversions for example from double hull tanker to Floating Production Storage Offloading (FPSO). The conversion is conducted due to the change function of ship. On the other hand, due to the change function of the ship, some items of ship such as equipment need to be evaluated because those related to the ultimate strength of ship. The objective of the present study is to analyse the influence of additional equipment to the ultimate strength of FPSO under vertical bending moment for hogging condition. The cross section of FPSO is taken by considering one-frame space. The numerical analysis is used and the Multi Point Constraint (MPC) is applied to both sides of the cross section and it is assumed to be remained plane. The shell element type is used for modelling the cross section. The material properties are set to be constant. As a simple calculation, the ship's cross section is analysed in intact. The additional equipment is calculated and included in the analysis to know the influence of it. The analysis of the ultimate strength including their influence of additional equipment is conducted under longitudinal bending. To compare the ultimate strength obtained by numerical analysis, the analytical solution is adopted. It is found that the ultimate strength gained by numerical analysis is almost identic with the analytical solution. The behaviour of the ship's cross section is also presented in term of stress distribution and deformation.

* Corresponding author: zubair.m@eng.unhas.ac.id

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The Influence of Additional Equipment to the Ultimate Strength of FPSO

Muhammad Zubair Muis Alie¹, Juswan¹, Wahyuddin Mustafa¹, Kevin Gabriel Pangalanan¹ and Nurul Inda Pratiwi²

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Keywords: FPSO, Cross Section, FEM, Ultimate Strength.

Abstract: Nowadays, there have many ship conversions, for example, from double hull tanker to Floating Production Storage Offloading (FPSO). The conversion is conducted due to the change function of the ship. On the other hand, due to the change function of the ship, some items of the ship such as equipment need to be evaluated because of those related to the ultimate strength of the ship. The objective of the present study is to analyze the influence of additional equipment to the ultimate strength of FPSO under vertical bending moment for hogging condition. The cross-section of FPSO is taken by considering one-frame space. The numerical method is used and the Multi-Point Constraint (MPC) is applied to both sides of the cross-section and it is assumed to have remained plane. The shell element type is used for modeling the cross-section. The material properties are set to be constant. As a simple calculation, the ship's cross-section is analyzed in intact. The additional equipment is calculated and included in the analysis to know the influence of it. The analysis of the ultimate strength, including their influence of additional equipment, is conducted under longitudinal bending. To compare the ultimate strength obtained by the numerical method, the analytical method is adopted. It is found that the ultimate strength gained by the numerical method is almost identical with the analytical method. The behavior of the ship's cross-section is also presented in term of stress distribution and deformation.

1 INTRODUCTION

The conversion of a ship is implemented in recent years. This phenomenon is mostly conducted for an offshore structure such as from Double Hull Tanker to Floating Storage Offloading (FSO) or Floating Production Storage Offloading (FPSO). The conversion aims to change the ship function and operation, including ship payload. In some years, about 70% of FPSO is produced from the conversion result. Time-consuming is shorter than a new design that is one of the reason. On the other hand, during conversion, some equipment must be added not only for the ship's function but also for the ship's construction. Due to this reason, the analysis of the ultimate strength of being converted considering the additional equipment must be taken into account.

The analysis of the ship's strength had been introduced by some papers like; The residual strength of an Aframax-class double hull oil tanker

damaged in the collision had been assessed by (Parunov, Rudan and Bužančić Primorac, 2017) considering the influence of the rotation of the neutral axis. The impact of nonlinear finite element method models on the ultimate bending moment for hull girder was studied by (Xu and Pan 2017). There were two analyses performed; those were implicit static analysis and explicit dynamic analysis. A structural reliability analysis model based on a Bayesian belief network was proposed by (Li and Tang, 2019) for the hull girder collapse risk after accidents. The Bayesian belief network was used to represent random states of variable risk events after accidents, as well as the dependencies between activities, and the structural reliability analysis was used to evaluate the failure probability hull girder for each possible accident conditions. The incidence of collision damage models on an oil tanker and bulk carrier reliability was investigated by (Campanile, Piscopo and Scamardella, 2018) considering the

IACS deterministic model against GOLADS/IMO database statistics for collision events, substantiating the probabilistic model. Safety of an oil tanker in intact condition was performed by (Campanile, Piscopo and Scamardella, 2017) to investigate the incidence of load combination methods on hull girder sagging/hogging time-variant failure probability. The simplified approach on the ultimate hull girder strength of asymmetrically damaged ships was conducted by (Muis Alie, 2018) considering the critical element under sagging condition. The effect of symmetrical and unsymmetrical configuration shapes on buckling and fatigue strength analysis of fixed offshore nonlinear finite element was conducted by (Muis Alie, 2016) and damages were modeled simply by removing the element on the damaged part. The ultimate hull girder strength considering section modulus under longitudinal bending was analyzed by (Muis Alie and Latumahina, 2018) and the cross-section of Ro-Ro ship was taken to be analyzed.

In the present study, the hull girder strength analysis caused by additional equipment on FPSO after being converted into FPSO under longitudinal bending is conducted. The ship's construction is not changed, but additional equipment after conversion is applied. For the simple calculation, the one-frame space of FPSO's cross-section is considered. The cross-section is assumed to have remained plane. The element type of shell 181 is used on the model. The initial imperfections, cracks, and residual welding stress are not taken into account. The ultimate strength obtained by the numerical method is compared with the analytical method and the behavior of the ship in term of stress distribution and deformation are also shown in this study.

2 FINITE ELEMENT MODEL

In the present study, the numerical method to analyze the hull girder strength due to load equipment of change function from FSO to FPSO before and after being converted is conducted. The ship has 172 m, 30 m and 18.4 m of length, breadth and depth of ship, respectively. The midship section consists of two kinds of longitudinal stiffeners those are flat-bar and angle-bar. The analysis of the ultimate strength is performed under hogging condition. There is also the inner hull in the cross-section. The element type is shell-181. The shell-181 element applied to all of the cross-section, as shown in figure 1.

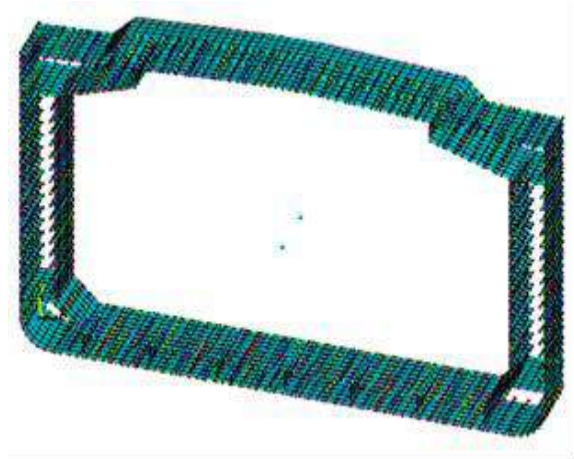


Figure 1: Finite element model

According to figure 1, there are two points located at the neutral axis. These two points are placed at both sides of the cross-section, and those are used to place the MPC (Multi-Point Constraint) for representing the behavior of the cross-section. The ultimate strength analysis, including the effect of change function from FSO to FPSO, is calculated using the numerical method under sagging condition. The rigidly linked corresponding with the boundary condition where MPC is applied to both sides of the cross-section, as shown in figure 2.

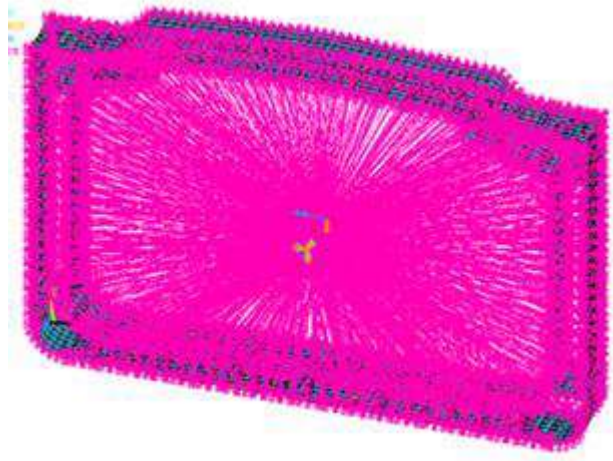


Figure 2: Rigid link and MPC

According to figure 2, the applied MPC is placed at the fore and aft side of the cross-section. This is done to connect the MPC called master node and the points of the all side's cross-section. The applied moment is placed at this MPC.

3 RESULT AND DISCUSSION

The behavior of the ultimate strength analysis is described in term of working stress distribution. Figure 3 shows the working stress of FSO under hogging condition. The deck and bottom part are under tension and compression since the hull cross-section is under hogging condition.

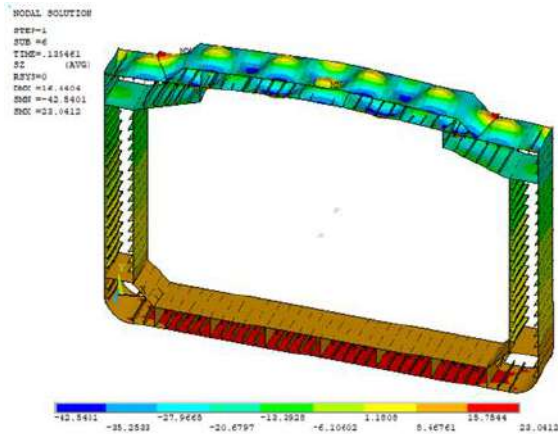


Figure 3: Working Stress of FSO in Hogging

Figure 4 shows the working stress of FPSO under hogging condition. It is observed that the characteristic of the working stress of FSO and FPSO is different from one another due to the additional equipment after being converted.

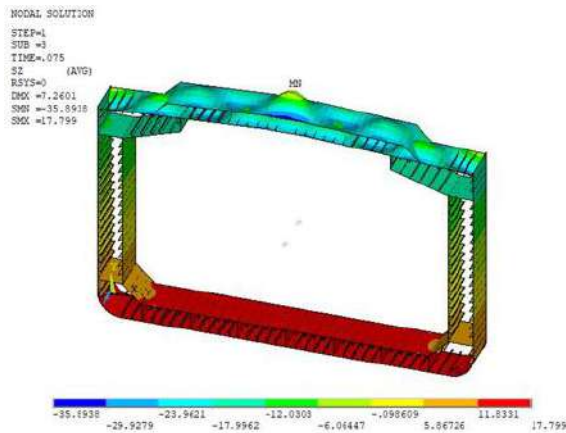


Figure 4: Working Stress of FPSO in Hogging

Figure 5 shows the moment-curvature relationship of FSO under hogging condition. The ultimate strength and collapse stages are represented by point A and B, respectively.

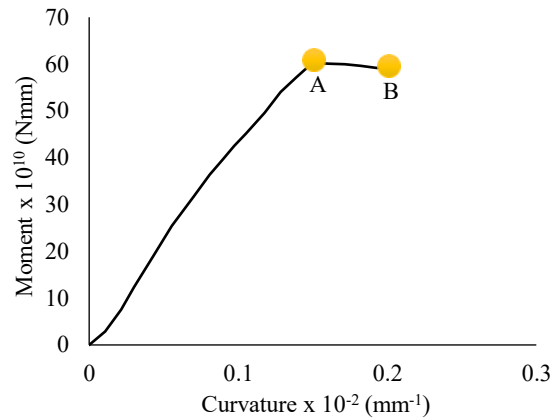


Figure 5: Moment-Curvature of FSO in Hogging

Deformation of the FPSO at the ultimate strength and collapse stage at point A and B under hogging condition are shown in figures 6 and 7, respectively. Failure takes place at some elements at the deck part where tension happen under hogging condition.

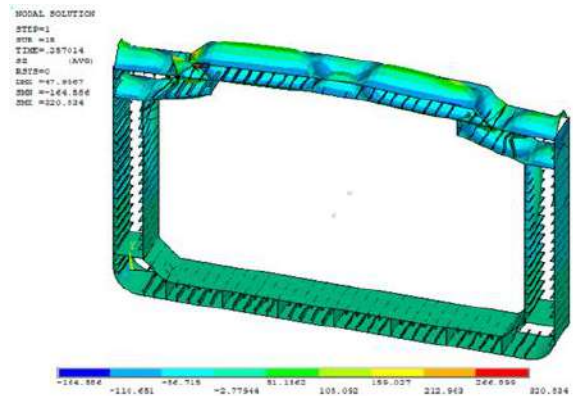


Figure 6: Deformation of FSO at Ultimate Strength

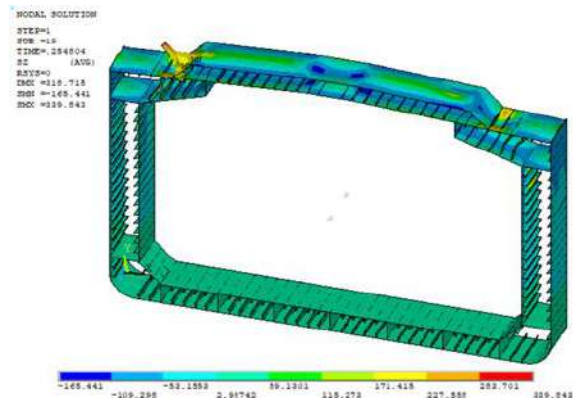


Figure 7: Deformation of FSO at Collapse

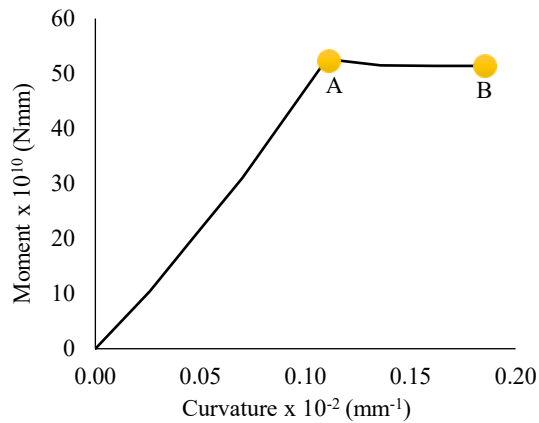


Figure 8: Moment-Curvature of FPSO in Hogging

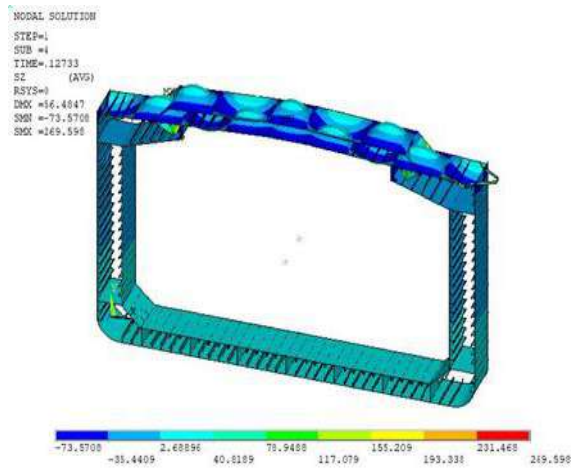


Figure 9: Deformation of FPSO at Ultimate Strength

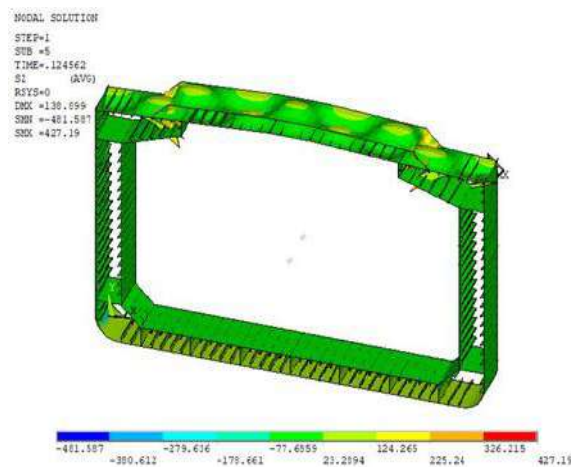


Figure 10: Deformation of FPSO at Collapse

Figure 8 shows the moment-curvature relationship of FPSO at point A and B in hogging conditions. Points A and B represent the ultimate strength and collapse regime, respectively. Figures 9 and 10 show the deformation at the ultimate strength and collapse region of FPSO under hogging condition.

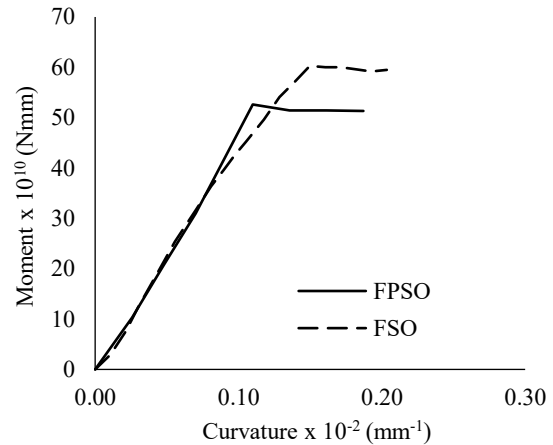


Figure 11: Moment-Curvature Relationship

Figure 11 expresses the comparison of the moment-curvature relationship after being converted from FSO to FPSO under hogging condition.

According to figure 11, the bending moment capacity of FSO is larger than FPSO. This is because the additional equipment is applied from FSO to FPSO so that the capacity bending moment after being converted is decreased.

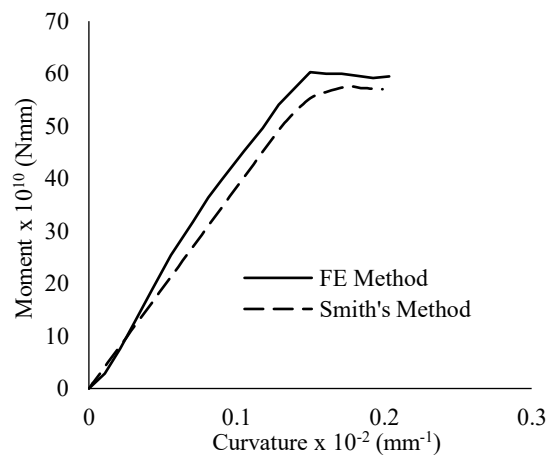


Figure 12: Comparison of Moment-Curvature Relationship

For the comparison purpose for the ultimate strength analysis, the analytical method is adopted. Figure 12 shows the comparison of the ultimate strength obtained by the numerical method and analytical method. The numerical method is represented by the solid line and the dashed line for the analytical method. It is found that the ultimate strength obtained by the numerical method is larger than the analytical method. This phenomenon happens due to the redistribution of the stress concentration takes place since the three-dimensional analysis is considered. The bending stiffness is also changed due to the additional equipment after being converted from FSO to FPSO.

4 CONCLUSIONS

The ultimate strength analysis has been conducted by using the numerical method under longitudinal bending in hogging condition. The following conclusions are; the additional equipment has a significant influence on the ultimate strength after being converted from FSO to FPSO. The bending stiffness is different due to additional equipment before conversion from FSO to FPSO. The ultimate strength obtained by the numerical method for FPSO is larger than FSO.

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Department of Ocean Engineering
Faculty of Marine Technology
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FOREWORD

Welcome to the 7th International Seminar on Ocean and Coastal Engineering, Environmental and Natural Disaster Management (ISOCEEN) 2019 was held in Surabaya, Indonesia, on 6 - 7 November 2019. The Organizing Committee acknowledged for their dedicated preparation over many years that led to successful seminar with broad participant. This event is held by cooperation among Institut Teknologi Sepuluh Nopember (ITS) especially Department of Ocean Engineering, Tohoku University, Japan, TU Delft, Netherlands, UNESCO-IHE Delft, Netherlands and HZ University of Applied Sciences, Netherlands. During the Plenary sharing Session with 2 keynote Speakers, 3 invited speakers, 42 regular talks contributions from over 120 participants from 6 countries (Indonesia, Japan, Netherlands, Lebanon, Portugal and Malaysia) were disseminated and gathered at Swiss-Belinn Hotel Manyar Surabaya, Indonesia. The 40 selected, peer reviewed papers contained in this Proceedings cover the topics including the field of Ocean, Offshore, Coastal engineering, Environmental and Disaster Management. This forum is more focused on creating and expanding professional network to foster the relationship between the University, industry, business and communities across the country.

Finally, on behalf of the organizing committee of the seminar, sincere appreciation is expressed to all authors contributing to our seminar. Special thanks are also due to all keynote speakers, invited talks and chairpersons for the efforts in preparing the manuscripts and managing the sessions, respectively.

On Behalf of the Organizing Committee ISOCEEN 2019
Department of Ocean Engineering, Faculty of Marine Technology
Institut Teknologi Sepuluh Nopember

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Mechanical Behaviour of Coir and Glass Fibre Reinforced Polymer Composites Material: A Literature Study

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Keywords: Coir Fibre Reinforced Polymer, Glass Fibre Reinforced Polymer, Vessel Hull Structure, Mechanical Properties.

Abstract: Glass fibre reinforced polymer (GFRP) has been commonly adopted in marine engineering for vessel hull as a high performance and durable material. Otherwise, the several efforts to replace the synthetic fibres with the natural fibres due to the increase of environmental awareness and oil resources depletion have been conducted. The natural fibres such as coir, jute, hemp, ramie and bamboo which are available in abundance, biodegradable and low density is the main reason to explore the application potential in shipbuilding sectors. In this paper, the mechanical behaviour of coir and glass fibre reinforced polymer composites material is reviewed. The surface modification of the natural fibre are also reviewed and it is found that the surface treatment is an alternative method to improve the natural fibre mechanical properties for successfully utilizing the natural fibre in marine and shipbuilding applications. From this review of both coir and glass as a reinforced fibre, it is concluded that a focus on the material properties of coir fibre combined with the glass fibre as a hybrid material is needed if the behaviour of vessel hull composite structures during sailing operation and other sources of external load is to be fully understood.

1 INTRODUCTION

Knowledge and methods for making composite compositions as raw materials of ship hull construction have been developed since 1970. At that time the lamina layer was made using fiberglass which was chopped and given wet resin using a manual coating technique known as a hand layered method. The design and application of composite materials on ships is able to provide better performance in terms of speed, transport/cargo capacity, and fuel consumption which is more economical when compared to ships made of steel or aluminium. These improvements can be achieved because the composite material offers a lighter construction weight. Otherwise the FRP hull surface texture is smooth. Therefore it might reduce the magnitude of the ship resistance.

Composite construction that is widely used in ship construction is glass fibre reinforced polymer / plastic (GFRP). In reinforced polymer/plastic fibres, fiberglass is a component which is used to support loads, otherwise the function of polymers/plastics is stabilizing, bonding and distributing loads on the

glass fibres, and provide water-resistant conditions in GFRP construction. Stiffness and rigidity of glass material fibres depends on the fibre material and the position of the fibre in the lamina layer. The ideal GFRP composite model is generally used as a sandwich construction consisting of a lightweight core which is enclosed in two rigid and strong layers (top and bottom layer) in the form of GFRP laminate. The adopted core material for hull construction can be PVC (polyvinyl chloride), polyurethane foam and balsa wood. This sandwich material offers a light weight construction but it has very good strength and rigidity.

Since 1990, the natural fibres have been introduced as an alternative reinforcing material in the composite laminate. The natural fibres can be used as a replacement of glass fibre or applied as an alternative fibre for core-layer material on the composite laminate board. Natural fibres such as hemp fibre-epoxy, flax fibre-polypropylene (PP) and china reed fibre-PP have been applied to the automotive industry, because it offers lower price and lighter density. Natural fibres also offer environmentally friendly properties through reducing

the dependency on non-renewable energy sources, low pollutant emissions, low green-house gas emissions, and the ability to be degraded naturally (biodegradability). Natural fibres have attracted scientists to be investigated for its capability to be implemented as alternative materials in the manufacturing and construction industries. The use of natural fibres, especially coconut fibre (coir fibre), as reinforced fibres of the core material of fibre reinforced plastic (FRP) composite material is an alternative method to achieved environmentally friendly vessel hull construction.

Based on its mechanical properties the strongest and most rigid FRP material is carbon fibre reinforced polymer. Several lamina configurations on FRP material can be carried out to obtain the desired mechanical properties. Figure 1 shows that FRP material does not show a clear yielding point compared to steel material. This characteristic of FRP materials can provide advantages and disadvantages in ship construction. Some other advantages of FRP material are corrosion resistance. The woven roving fibre composites have better rigidity and impact resistance. The addition of natural fibres to FRP material can influence the mechanical properties of FRP. Therefore, the use of natural fibres as additional reinforcing fibres in FRP material needs to be done in-depth studies so that they can be used in ship construction in receiving dynamic loads such as wave bending moments, green water loads, vibrations, collisions and explosions.

Although FRP composite strength is able to withstand dynamic loads, the mechanical characteristics of FRP are sensitive to strain rate and temperature. The tensile strength, modulus of the young and the shape of the stress-strain diagram of the FRP will change when subjected to loads that have different strain rates. Furthermore the addition of natural fibres in FRP material can also have an effect on its mechanical characteristics. Based on this, it is necessary to study to find out more about the effect of strain rate and temperature on FRP material accompanied by the addition of natural fibres as reinforcement (hybrid coir fibre and glass fibre) to the mechanical properties of the material.

2 MECHANICAL PROPERTIES OF GFRP

The GFRP composite material consists of glass fibers that function as reinforcement which is integrated with polymer resin (called matrix). Fillers also exist

apart from fibres and matrices used to increase interactions, reduce manufacturing costs and improve the mechanical properties of composites (Bakis, 2002; Binshan, 1995; Richardson, 1996). However, the nature of filler metal material has an insignificant relationship to the composite material properties, (Paciornik, 2003).

E-glass fibre is the most widely used type of fibre almost 95% of total glass fibre production, (Varma, 1984). The traveling glass is basic shapes with continuous filament bundles. Chopped strand mat (CSM) is a sheet that contains randomly chopped fibres, tied together adhesive. Figure 1 gives an example from failed GFRP coupons that show the roving and CSM micro.

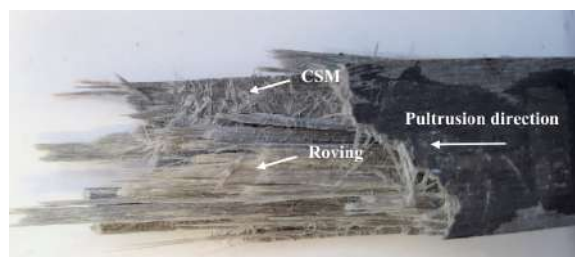


Figure 1: The failed GFRP composite material.

Polymer resins are non-fibrous parts of FRP that bind fibres together and are also known as matrices or binders. Polyester, vinyl ester and epoxy are the most widely used thermoset polymers, (Bank, 2006; Edwards, 1998), and the choice of polymer type depends on the mechanical properties required and the manufacturing methods available. Vinyl ester has been shown to have a higher rigidity than polyester or epoxy, regardless of the shape of the fibre, (Bank, 1989; El-Habak, 1991), but is more expensive. Therefore, polyester resin or polyester and vinyl ester mixtures are generally used in the production of GFRP composite profiles. However, in situations where there may be endurance issues, vinyl ester and epoxy can be the preferred choice.

Considering the orthotropic nature of the GFRP composite material, and its production, there are many factors that can influence its material properties. Several investigations since the 1990s have shown that the main factors are:

- fibre and resin type;
- fibre volume fraction (FVF);
- fibre orientation;
- sample geometry (including shape, length to width ratio);
- strain rate;
- interface properties; and
- temperature

These works include Naresh et al. (2018; 2017), Chelladhural et al (2018), Danie Roy et al (2018), Windyandari et al (2019), Zakki et al (2019a; 2019b), Bazli et al (2017), Hasan et al (2019), Rossini et al (2019), Zhang et al (2019), Reis et al (2018), and Kim et al (2015). In the works done, there were focused on the strength of the GFRP material which is applied to the civil structure and marine structure. However the published work on the material properties of GFRP combined with natural fibre is limited and often conflicting.

In the literature that concern on the influence of strain rate to the GFRP laminate composite, it is found that the strength is rise as the strain rate is increased regardless the resin type, and fibre type (woven roving/CSM), (El-Habak, 1991). Nevill et al (1971), and Binshan et al (1995), present that the material strength increase significantly increase at high strain rate (above 10^{-1}). Furthermore, Sierakowski (1997), has reviewed the strain rate (below 20 s^{-1}) effect of composite laminate that show a little increase in GFRP strength. The elastic modulus of GFRP laminates is also founded have similar tendency as the strength GFRP. It can be found that the elastic modulus of GFRP laminates is increasing when strain rate is increased, (Dorey, 1986; Hsiao, 1998). Regarding the reviewed literature, it can be seen that there has been several research to understand the influence of strain rate on the GFRP composite materials.

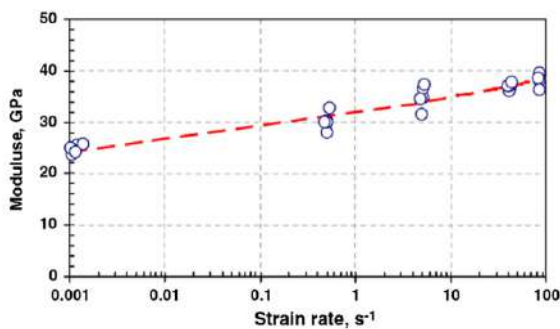


Figure 2: The longitudinal modulus of GFRP composites under compression, (Shokrieh, 2009).

In the case of longitudinal elastic modulus, the strain rate from 10^{-4} to 10^2 s^{-1} has increased the elastic modulus in the range of 20%-55%, (Rotem, 1971; Shokrieh, 2009; Tay, 1995). The similar increase also can be found on the smaller strain rate, (Armenakas, 1973; Amijima, 1980; Kawata, 1981; Bai, 1984; Li, 1995; Takeda, 1995; Yuanming, 1996; Huang, 2004). However, the other studies have shown a slightly different tendency on the influence strain rate on the elastic modulus. Hayes and Adams (1982), present

that the strain rates 60 s^{-1} to 80 s^{-1} have decreased the initial longitudinal modulus of 20%. Kumar et al (1986), shows that the dynamic modulus are 29% and 16% lower than the corresponding quasi static value on the strain rates 0.0002 s^{-1} and 265 s^{-1} , respectively. Ou et al (2015), present that the longitudinal modulus is decreased from strain 0 to 40 s^{-1} . However the elastic modulus is increased after 40 s^{-1} . The average value of all dynamic moduli can be shown in Fig. 2 and Fig. 3

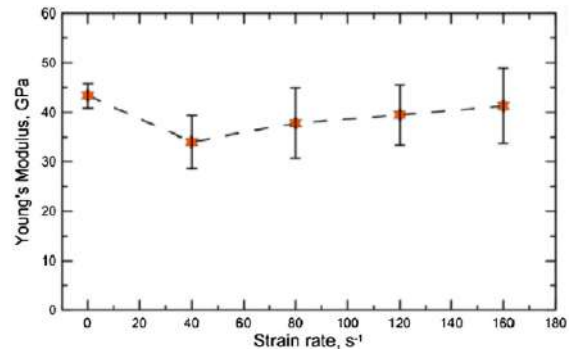


Figure 3: The longitudinal modulus of GFRP composites under tension, (Ou, 2015).

According to the literature review, the strain rate have influence the elastic modulus of GFRP material. It can be seen that the longitudinal elastic modulus have been reviewed more than the transverse modulus. Moreover, most researches generally deal with quasi static and high strain rate instead of the intermediate strain rate. Generally, it can be concluded that the increase of strain rate might cause an increase in the elastic modulus of GFRP laminate composite, especially for high strain rate. However the exception still can be found on such literature.

In the case of fibre orientation, the off axis fibre orientation have shown less sensitivity to the change of the strain rate compared to zero degree orientation. However, it cannot be found the research that was focused on the modulus of off axis of combined coir and glass FRP on the intermediate and high strain rate.

In the case of resin type and internal reinforcement structure, the several studies show that there is a significant influence on the elastic modulus of the GFRP laminate composite. The most common resin that was used as matrix is epoxy resin. The epoxy resin has higher stiffness characteristics that the other type of resin. The different sequence of fibre stacking also shows the different strain rate sensitivity to the elastic modulus.

Finally the reviewed researches usually provide an assumption that the GFRP composite have the same properties under compressive and tensile load.

The assumption should be validated on the experimental studies.

3 MECHANICAL PROPERTIES OF COIR FIBRE

Coir is natural hair-fibre that is obtained from the outer skin (endocarp), or husks of coconuts. Coir is also known as "Kokos" or "Coco". Coconuts are the fruit of *Cocos nucifera* tree, the tropical plant *Areaceae* (Palmae) family. *Cocos nucifera* have grown many especially in the coastal area in the tropical countries. The first coir factory for manufacturing coir products was found to be established at Alleppy, in Kerala in 1858, (Rajan, 2007). The word coir comes from "Kair" - Malayalam, a south Indian language (Tamil, *kayiru*) which means the umbilical cord. Coir fibres or coconuts are categorized in the group of hard structural fibres, and lignocellulose.

Coir fibre is physically and visually looked as coarse, stiff and reddish brown coloured fibres. It contains of smaller threads with diameter of 12 to 24 microns and the fibre length of 0.01 to 0.04 in. Coir fibre is composed of lignin, substance of plant, and cellulose. The advantage of coir fibre is the capability to stretch beyond its elastic limit with no rupture damage. It seems like having the power to overcome the permanent deformation. Coir fibres also have good resistance to microbial degradation and salt water. Therefore the coir fibre is suitable to be adopted as material of marine industry product.

Morphologically coir fibres are contains of micro fibrils that is organized the aggregates of cellulose molecules. The micro fibrils are embedded in a non-cellulosic polysaccharides and lignin matrix. The lignin is located between the non-saccharide parts of the cell wall as the unique polymer. The large lignification can be found on the middle lamella and primary cell, and the least is found on the secondary wall. The lignification function is to strengthen the cell wall through cementing the cellulose micro fibrils and protecting them from the chemical and physical deteriorations.

As a natural cellulosic fibre, the coir fibres firstly have an almost white colour. The increasingly lignified process will made the coir fibre colour become darker. The coir have red-brown colour in the dry husk of matured coconuts. Bismarck et al (2001), study the thermal stability of coir fibre that presented the two-step decomposition curve and the degradation onset between 190 °C and 230°C.

The preliminary study on the chemical changes of the lignocellulose of coir fibre was made by Menon and Pandalai (1936). The lignocellulosic coir fibre is biodegradable. Since the coir fibres have high lignin contents, coir has better durability compared to the other natural fibres. The advantages of the coir fibres are include:

- 100% natural fibre,
- Biodegradable material
- Obtained from renewable resources
- High water retention
- Biological deterioration resistant
- Good heat and noise insulator

The chemical composition of coir such as cellulose, cellulosan, lignin and hemicellulose is depended upon the coconut age, (Menon, 1936). It is different with the jute fibres that have uniform chemical composition at all stages of the growth process. The physical properties and the chemical composition of coir fibre can be seen in Table 1 and Table 2, respectively.

Table 1: Physical properties of coir fibre, (Bledzki, 1996; Chand, 1994).

Length [cm]	15 – 20
Diameter [μm]	100 – 450
Density [g/cm^3]	1.15
Tenacity [g/tex]	10.0
Tensile strength [MPa]	131 – 175
Young modulus [GPa]	4 – 6
Elongation at break [%]	15 – 40
Swelling in water [%]	6 – 8.5

Table 2: Chemical composition of coir (wt%), (Ugbolue, 1990; Varma, 1984; Varma, 1986).

Lignin	41 – 45
Cellulose	36 – 43
Pectin	3 – 4
Hemicellulose	0.15 – 0.25
Microfibrillar/spiral angle [deg]	41 -45
Moisture content	8.0

4 SURFACE MODIFICATION OF NATURAL FIBRES

As an alternative material for the replacement of synthetic fibres, natural fibres have any kind of advantages such as low density, low cost and biodegradable. However natural fibres also have some drawbacks which are include poor compatibility with different matrices, high moisture

absorption and swelling capability that may cause crack in brittle matrices. Since the drawbacks of natural fibres can decrease the laminate composite quality, therefore some techniques have been adopted to modify the surface of natural fibres, especially to reduce moisture absorption capability and to improve adhesion characteristics with polymer matrices. There are three kind of surface treatment for surface treatment of the natural fibres which are included: physical techniques, chemical techniques and biological treatments.

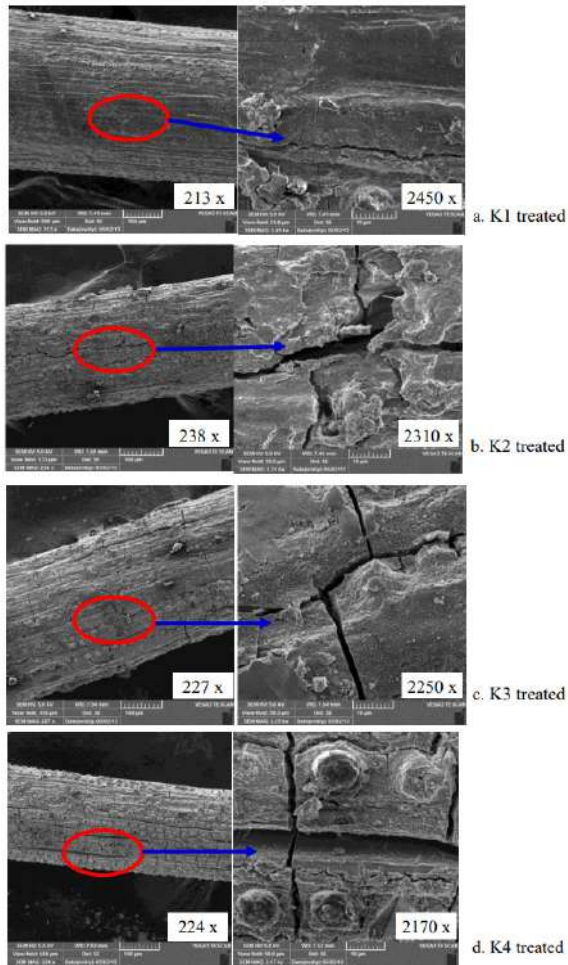


Figure 4: Surface morphology of coconut fibre with alkali treatment, (Muhammad, 2015).

A physical technique that is used for surface treatment of natural fibres is known as plasma treatment. The plasma treatment which has been used to modify the surface of various natural fibres can improve the mechanical properties significantly, (Oliveira, 2012; Shahidi, 2013). In addition, plasma treatment introduced functional group that form strong covalent bond with the matrix, therefore the

plasma treatment may produce the strong fibre/matrix bonding interface to improve the quality of laminate composite. Surface etching on the plasma treatment can improve the roughness of the fibres surface. Therefore the better mechanical interlocking between the fibres and matrices is obtained.

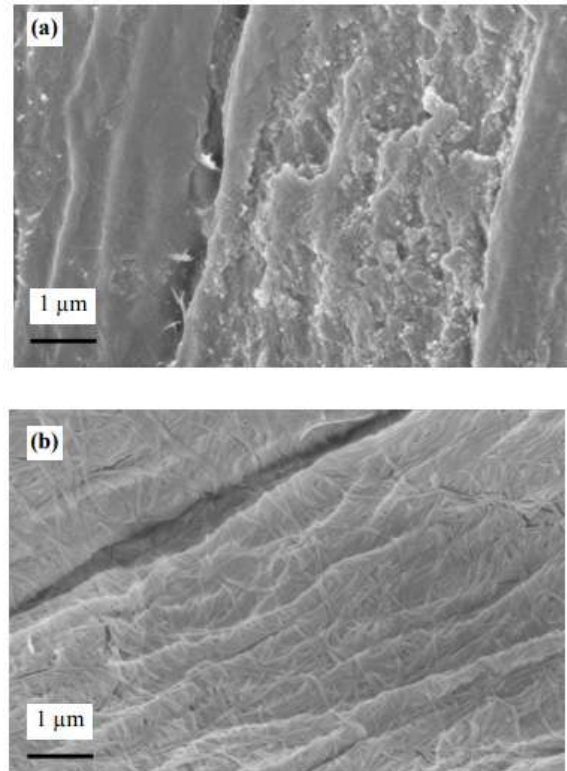


Figure 5: SEM micrographs of hemp fibre surfaces; (a) Natural hemp fibre; (b) Hemp fibre after bacterial cellulose modification, (Pommet, 2008).

Chemical treatments have been adopted for the surface treatment of natural fibres. The used of various chemical constituents such as alkali, silane, water repelling agents, peroxides, permanganates have been proved able to improve the mechanical properties significantly. The alkali treatments modified the crystalline structure of natural fibres through removing the weak components such as hemicelluloses and lignin of the fibre structure, (Xue, 2007). The water repelling agents can reduce the moisture absorption and natural fibres swelling. Furthermore the silane coupling agents can improve the fibres-matrices interfacial interaction through strong chemical bonding formation, (Xie, 2010). The illustration of the surface morphology of coir fibre with alkali treatment can be seen in Fig. 4, (Muhammad, 2015).

Finally, biological treatment is adopted to modify the natural fibre surface characteristics. The biological treatment is conducted through depositing cellulose nano fibrils on the surface of the sisal and hemp fibres, (Pommet, 2008). The cellulose nano fibrils were used as substrates for the fermentation process of bacterial cellulose. The significant improvement was made on the interfacial adhesion with polymeric matrices such as cellulose acetate butyrate and polylactic acid. The percentage of deposition of bacterial cellulose is about 5-6% on the natural fibre surface. The illustration of bacterial cellulose modification on the natural hemp fibres can be seen in Fig. 5.

5 CONCLUSIONS

In this paper, the existing literature on the material properties of GFRP and coir fibre has been reviewed. The coir fibre shows great opportunity to be applied in marine structures such as boat hull structures, due to the excellent durability and high water retention. As an alternative material, it is important that its behaviour under stress applied at different strain rates, different direction and combined with the GFRP as hybrid material is more fully understood. Although many researches have been found on the mechanical properties of GFRP laminates, there is yet limited studied on the dynamic properties of coir fibre and hybrid coir-glass fibre.

In comparison with other natural fibres, coir fibres as a high lignified material have better resistant to chemical and microbial attack. The advantage of coir fibre compare to synthetic fibres such as glass fibres, talc and mica are acceptable strength properties, low cost, low density, good thermal properties, non-abrasivity, enhanced energy recovery and biodegradable. Surface treatment of natural fibres that made bio-softened coir can be blended with the synthetic fibres as a hybrid material for producing the marine structures.

According to the advantage and the mechanical properties of coir fibre, it is indicated that coir fibres is able to be adopted as an alternative reinforced fibres in composites material. The surface treatment of natural fibres such as physical, chemical and biological treatment methods, and also the mechanical characteristics of the coir fibres and the hybrid material (coir-glass reinforced polymers) is the subject of interest for the future research in order to fully utilize the advantages of natural fibres in composite materials and to successfully utilize it in the marine industrial application.

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Analysis of Ocean Thermal Energy Conversion (OTEC) Potential using Closed-cycle System Simulation of 100 MW Capacity in Bali Sea

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Abstract: The Bali Sea is one of the body water that has OTEC potential because of its location in tropical region, thus it has high sea surface temperatures. This research was conducted to examine temperature differences to run the system and analyze the results of simulation of the closed-cycle OTEC system based on the simulation results of Uehara and Ikegami (1990) as a basic reference in the installation of plants and obtain net power produced by the cycle. The data used in this study is temperature in depth data per day during October 2008-June 2017 which downloaded from HYCOM website and vertical temperature data of CTD which is the result of P3GL survey on 21 May 2017 -3 June 2017. Data processing was done by calculating the differences in temperature of warm sea air on the surface and cold sea water at depth of 800 m by qualifying the minimum difference requirement of 20°C. The results of temperature data processing yield the difference in temperature minimum requirements of 20°C at all study area in the range of 21.895°C-24.7°C. The parameter values in the closed cycle OTEC system obtained are the warm sea water (TWS) and cold (TCS) temperatures of 28.4°C-30.36°C and 5.591°C-6.711°C; warm seawater pump power (PWS), cold seawater pump power (PCS), and working fluid pump power (PWF) of 10.9596-11.521 MW, 16.0596-16.621 MW, and 1.94-1.97 MW; and the heat transfer area in evaporator (AE), the heat transfer area in condenser (AC), and total of heat transfer area (AT) of 0.737-1.7478 x10⁵ m², 0.9685-1.614 x10⁵ m², and 1.7058-3.3435 x10⁵ m². Net power potential of OTEC in Bali Sea has range between 70-71 MW with maximum net power found at Point 2 of 71.041 MW with capacity of 100 MW and produce a cycle efficiency of 0.41323 or 41.323%.

1 INTRODUCTION

Sunlight reaches the earth's surface 35-100 m (Avery and Wu, 1994). The sea in the tropics absorbs the sun continuously all the time causing sea surface temperatures to vary to reach 27°C-29°C. The total area of the world's tropical oceans totaling 60 million km² produces energy equivalent to 250 trillion fuels per yield (Nihous, 2005). Seas in Indonesia with a total thermal potential of 2.5x10²³ Joules and an efficiency of 3%, can produce power of 240,000 MW (Prabowo, 2012).

Sea Heat Energy Conversion (OTEC) is one of the solutions in developing ocean energy by using temperatures between the sea and the deep sea in the

tropics to produce electrical energy with a minimum temperature difference of 20°C (Nihous, 2007). Energy generated from sea heat is very suitable to be applied in tropical regions such as Indonesia. If this can be done effectively and on a large scale, OTEC provides a renewable energy source that is needed to complement various energy problems (Syamsuddin, 2015).

Various studies and OTEC studies that have been carried out in Indonesia to date have been carried out by Sinuhaji (2015) in the North Bali Sea with the result of an increase in surface temperature between 28-31°C with a difference of 24°C; Negara and Koto (2016) in Karangkelong, North Sulawesi with a potential of 100 kW; and Syamsuddin (2015) at seven location points in Indonesia using World

Ocean Atlas 2009. Previous research on the potential of the OTEC in the Bali Sea is only theoretical and must be further discussed to obtain the expected results.

The goal of this research is to obtain minimum temperature difference requirement of 20°C at seven study points in the Bali Sea Waters, to obtain minimum temperature difference requirement of 20°C at seven study points in the Bali Sea Waters and to analyze the result of OTEC system simulation closed cycle with 100MW power capacity.

2 SEA WATER TEMPERATURE AND OTEC OVERVIEW

2.1 Vertical Distribution of Sea Water Temperature

Solar thermal energy is absorbed by the surface layer and penetrates the deeper sea. However, the conduction process occurs so slowly that only a small portion of the heat flowing in (Santos et al., 2012). The temperature will decrease dramatically at depths between 200-300 m to 1000 m. This decreasing depth layer is known as a thermocline that is thinner at low latitudes than at high latitudes. At a depth of 300-1000 m, seawater does not change in temperature and ranges from 0-3°C. It is influenced by cold temperatures originating from the mass of water from the poles then flowing into the equatorial region (Santos et al., 2012) equatorial region (Santos et al., 2012).

2.2 Ocean Thermal Energy Conversion (OTEC)

It is a powerplant by utilizing the temperature difference of seawater on the surface and the temperature of the seawater where the ocean, which covers two-thirds of the earth's surface area, receives heat from solar radiation. This thermal energy can be utilized by converting it into electrical energy with a technology called Ocean Thermal Energy Conversion (OTEC). A large amount of energy is absorbed by the oceans in the form of heat that comes from the sun's rays and magma located beneath the seabed (Masutani and Takahashi, 2001).

2.3 OTEC System

The OTEC system is divided into three types, namely open cycle, closed cycle, and hybrid cycle

(Aldale, 2017). The closed-cycle OTEC system is more widely studied than other systems based on various source literature. A closed-cycle requires a turbine that is smaller than an open cycle (open-cycle) and can increase the efficiency of the electrical energy produced by the generator (Masutani and Takahashi, 2001). This study will use a closed-cycle OTEC system.

2.4 OTEC Power Calculation

The power generated from the turbine generator in the OTEC system according to (Uehara and Ikegami, 1990) is

$$P_G = m_{WF} \eta_T \eta_G (h_1 - h_2) \quad (1)$$

Where

- P_G : turbine generator power(MW)
- m_{WF} : the mass flow rate of the working fluid(kg/s)
- η_T : turbine efficiency = 0.85
- η_G : generator efficiency
- $h_1 - h_2$: a decrease in adiabatic heat between the evaporator and the condenser

The net electrical power equation used is

$$P_N = P_G - (P_{WS} + P_{CS} + P_{WF}) \quad (2)$$

Where

- P_N : clean electric power (MW)
- P_G : turbine generator power (MW)
- P_{WS} : warm sea flow pump power (MW)
- P_{CS} : cold sea water pump power (MW)
- P_{WF} : working fluid pump power (MW)

3 METHOD

The method used in this research is a quantitative method. The first step is taking CTD temperature data and collecting HYCOM temperature data by using a data model downloaded from the website <http://ncss.hycom.Org/thredds/catalog.html> with the Net Common Data File (NetCDF) format which used daily temperature data for 9 years (October 2008-June 2017) with a resolution of 1/12° and a depth of 0-5500 m. The second step is the processing of CTD temperature data and HYCOM temperature data processing by using data verification using the Root Mean Square Error (RMSE) (Neill and Hashemi, 2018).

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n (s_i - o_i)^2} \quad (3)$$

Where

n : number of observation

O : observation value

S_i : predictive value

Formula parameters to determine the bias values by Neill and Hashemi (2018), as follows:

$$bias = \bar{S} - \bar{O} \quad (4)$$

3.1 Calculation of Sea Water Pump Power and Working Fluid

To calculate sea water pump power and working fluid is used the formulation by Uehara and Ikegami (1990) and it can be followed this step.

The power of warm seawater pump is calculated by the formulation as follows

$$P_{WS} = \frac{m_{WS} \Delta H_{WS} g}{\eta_{WSP}} \quad (5)$$

Cold sea water pump power is calculated with the following formulation:

$$P_{CS} = \frac{m_{CS} \Delta H_{CS} g}{\eta_{CSP}} \quad (6)$$

Power of the working fluid pump is calculated by the formulation as follows:

$$P_{WF} = \frac{m_{WF} \Delta H_{WF} g}{\eta_{WFP}} \quad (7)$$

Where

P_{WS} : warm sea flow pump power (MW)

P_{CS} : Cold sea water pump power (MW)

P_{WF} : working fluid pump power (MW)

m_{WS} : Mass flow rate of warm sea water (t/s)

m_{CS} : Cold sea water mass flow rate (t/s)

m_{WF} : Working fluid mass flow rate (t/s)

η_{WSP} : Warm water pump efficiency

η_{CSP} : Cold water pump efficiency

η_{WFP} : Efficiency of the working fluid pump

ΔH_{WS} : The total pressure difference in Warm water pipes

ΔH_{CS} : The total pressure difference in the Cold water pipe

ΔH_{WF} : The total pressure difference in the working fluid pipe

g : acceleration of gravity (m/s²)

3.2 Calculation of Sea Water Pump Power and Working Fluid

In this study, the A_T value was obtained from the interpolation between the temperature difference value (ΔT) from the temperature data processing and the A_T value using Uehara and Ikegami (1990).

To determine the total heat transfer area, the following equation can be used from Uehara and Ikegami (1990):

$$A_T = A_E + A_C \quad (8)$$

On the evaporator, the heat transfer area is calculated by the formula:

$$A_E = \frac{Q_E}{U_E (\Delta T_m)_E} \quad (9)$$

To find out the value of the heat transfer rate on the evaporator, can use the formula:

$$Q_E = m_{WF} (h_1 - h_4) \quad (10)$$

In the condenser, the heat transfer area is calculated by the formula:

$$A_C = \frac{Q_C}{U_C (\Delta T_m)_C} \quad (11)$$

To find out the value of the heat transfer rate in the condenser, can use the formula:

$$Q_C = m_{WF} (h_2 - h_3) \quad (12)$$

Where

A_T : The area of heat transfer in the evaporator (m²)

A_E : The area of heat transfer in the condenser (m²)

A_C : Total heat transfer area (m²)

Q_E : Heat transfer rate to the evaporator

Q_C : Heat transfer rate to the condensor

$(\Delta T_m)_E$: logarithmic mean temperature differences (LTMD) pada evaporator

$(\Delta T_m)_C$: logarithmic mean temperature differences (LTMD) pada kondensor

$(h_1 - h_4)$ and $(h_2 - h_3)$: the enthalpy value matches the Rankine cycle

3.3 Calculation of OTEC Net Power and Rankine Cycle Efficiency

The net power of OTEC (P_{NET}) can be calculated using equation (2). The efficiency of the Rankine cycle (η_{Ran}) can be calculated by the equation

$$\eta_{Ran} = \frac{P_G}{Q_E} \quad (13)$$

Where

η_{Rank} : Rankine cycle efficiency

P_G : Turbine generator power (MW)

Q_E : Heat transfer rate to the evaporator

4 RESULT AND DISCUSSION

4.1 Verification of HYCOM and CTD Temperature Data

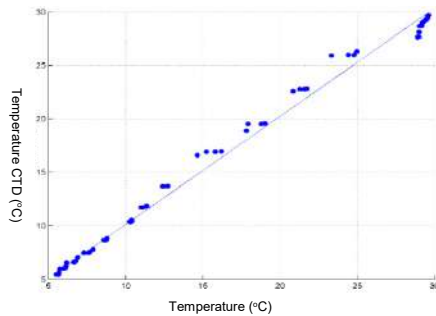


Figure 1: Distribution of Temperature Value to Depth between HYCOM Data and CTD Data (Source: Data Processing).

The error value obtained and the data distribution graph below, then HYCOM has good data and is considered capable of representing the temperature conditions of the CTD collection.

4.2 Vertical Temperature Distribution in the Bali Sea

The temperature of warm seawater at the surface (T_{ws}) and the temperature of cold seawater at depth (T_{cs}) are obtain to get the difference in temperature (ΔT) between sea level and depth.

In general, temperatures that tend to be uniform in the mixed layer (Figure 2-8) are caused by the turbulence mechanism by wind and waves and heat flux at sea level. Changes in temperature at the surface of the sea and layers are mixed due to the strength of the wind influenced by monsoons so that the temperature at the surface of the sea and the layers are mixed experiencing monthly variations. Also, according to Atmadipoera and Hasanah (2017), the temperature in the mixed layer shows seasonal variability. The cause of this variability is thought to be due to the influence of the mass movement of water in the Java Sea and the Flores Sea which partly entered the Lombok Strait.

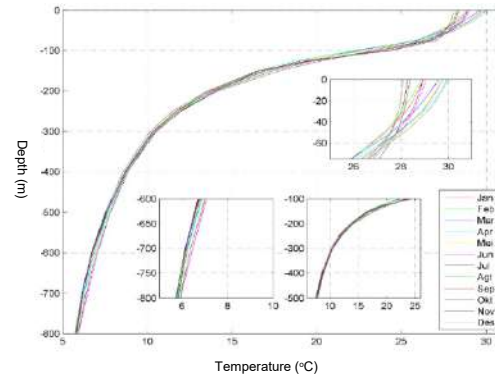


Figure 2: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 1 (Source: Data Processing).

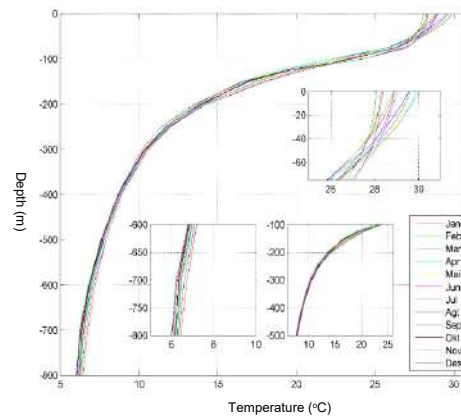


Figure 3: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 2 (Source: Data Processing).

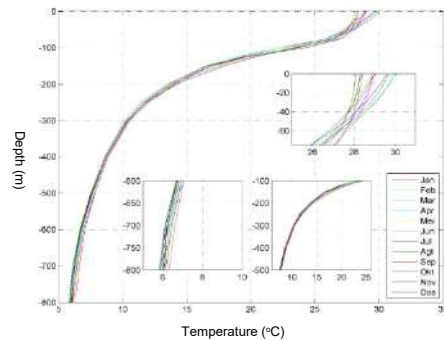


Figure 4: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 3 (Source: Data Processing).

4.3 Difference in Surface and Depth Temperature in the Bali Sea

From 2008 to 2017, the maximum temperature difference value occurs in April at all points as shown in Figure 12 with the average maximum temperature difference of 23,998°C. From April to

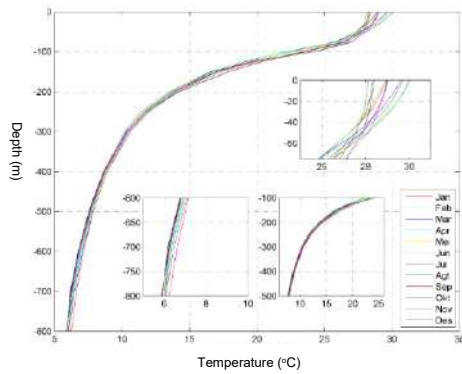


Figure 5: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 4 (Source: Data Processing).

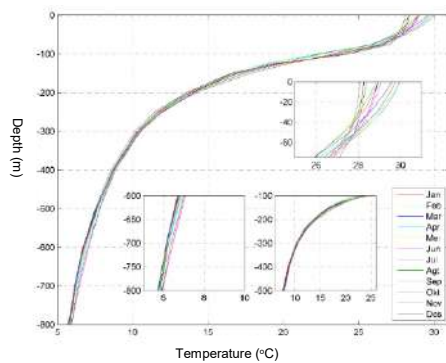


Figure 6: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 5 (Source: Data Processing).

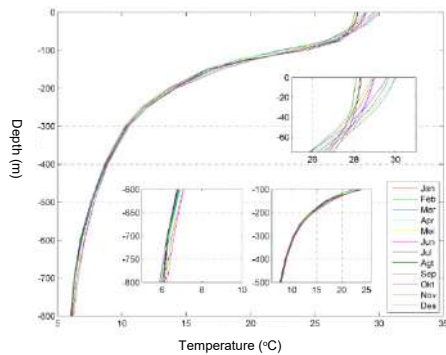


Figure 7: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 6 (Source: Data Processing).

August, the temperature difference decreased quite dramatically at all points with an average decrease of 1,975°C. The lowest temperature difference occurred in August, with an average of 22,023°C. Then, starting in August there was a significant increase of 1,933°C at all points until November with an average temperature difference of 23,956°C for the month.

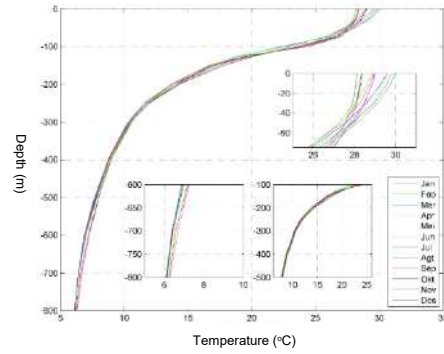


Figure 8: Temperature Vertical Distribution Plot of 2008 - 2017 at Point 7 (Source: Data Processing).

Bali Sea meets the potential OTEC requirements of more than 20°C during October 2008-2017 with a temperature difference ranging from 21.9°C to 24.5°C. However, this potential needs to be studied further to get the true potential by determining the net power value based on a closed-cycle system simulation of 100 MW.

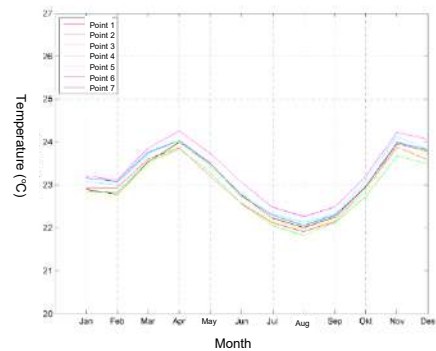


Figure 9: Average Monthly Temperature Difference in 2008-2017. (Source: Data Processing).

4.4 Closed Cycle System Simulation

4.4.1 Warm and Cold Water Temperature Profiles

This shows that the temperature of cold seawater is not affected by monsoons like warm seawater temperatures. The almost uniform temperature is caused by a large enough density in the deep sea so that the mass of seawater is denser. The high density is caused by layers in the deep sea not having a direct influence from the wind, the intensity of sunlight, precipitation and evaporation, and cloud cover such as the temperature of warm seawater.

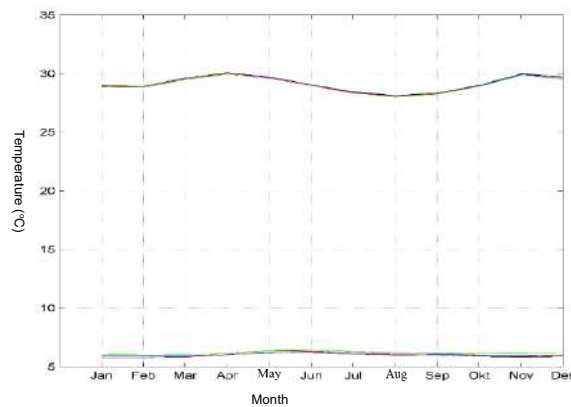


Figure 10: Variations in Surface Temperature Monthly (Tws) and Temperature Depth (Tcs) in 2008-2017 (Source: Data Processing).

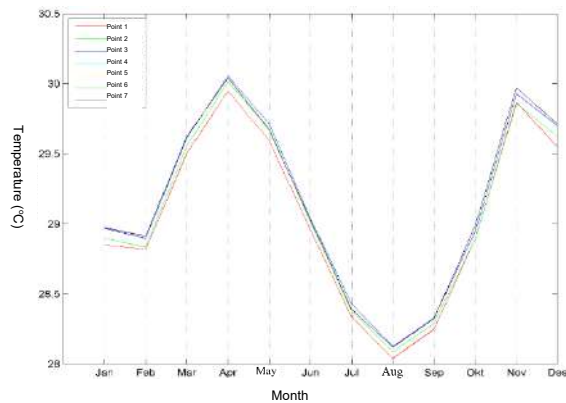


Figure 11: Monthly Variations of Warm Sea Water Temperature on Tws 2008-2017. (Source: Data Processing).

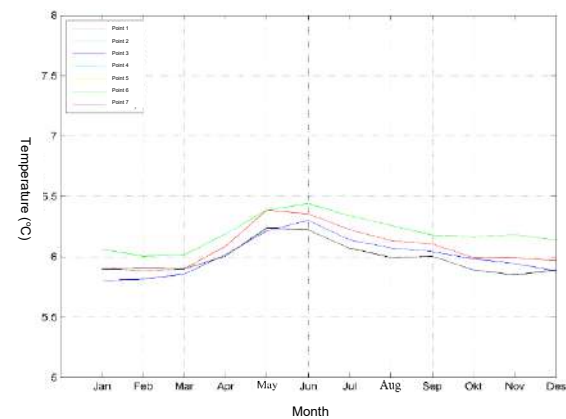


Figure 12: Monthly Variations in Cold Sea Water Temperature at Depths of 800 m (Tcs) in 2008-2017 (Source: Data Processing).

4.4.2 Pump Power for Warm Water, Cold Water and Working Fluid

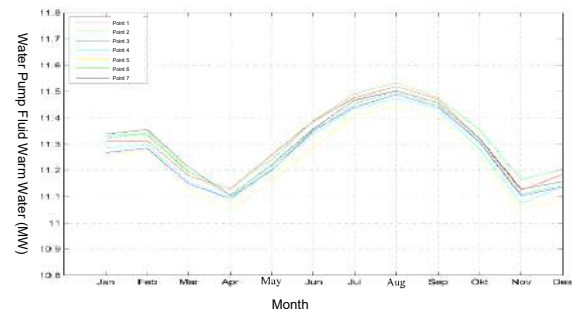


Figure 13: Monthly Warm Water Pump Power (Pws) in 2008-2017 (Source: Data Processing).

Warm seawater pump power reached the highest rate in August, which was an average of 11,495 MW. Meanwhile, the lowest warm seawater pump power occurred in April with an average value of 11.1 MW.

Coldwater pump power reached the highest rate in August, which was an average of 16,595 MW. Meanwhile, the lowest warm seawater pump power occurred in April with an average value of 16.2 MW. Coldwater pump power is stable from year to year with a range between 1.94-1.97 MW.

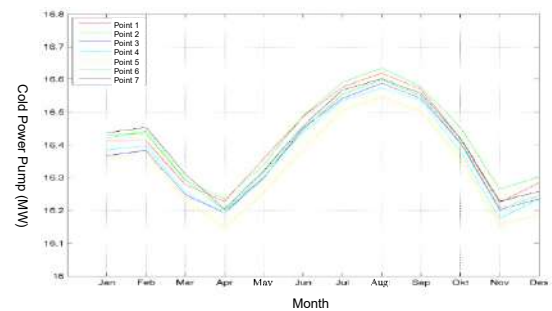


Figure 14: Monthly Cold Power Pump (Pcs) 2008-2017 (Source: Data Processing).

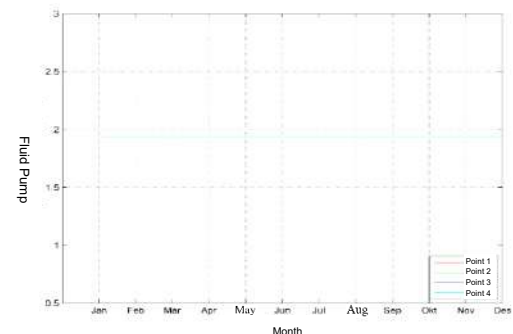


Figure 15: Power of Monthly Working Fluid Pump (Pwf) in 2008-2017 (Source: Data Processing).

4.4.3 Heat Transfer Area

The heat transfer area in the evaporator reached the highest number in August, which was an average of $1.70164 \times 10^5 \text{ m}^2$. Meanwhile, the lowest heat transfer area in the evaporator occurred in April with an average value of $0.99052 \times 10^5 \text{ m}^2$.

The area of heat transfer in the condenser reached the highest number in August, namely an average of $1,58466 \times 10^5 \text{ m}^2$ and the lowest in April with an average value of $1.130330 \times 10^5 \text{ m}^2$.

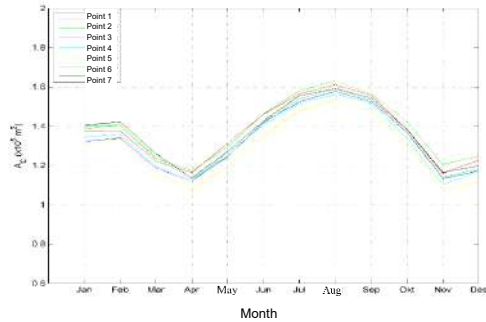


Figure 16: Area of Heat Transfer in Monthly Evaporators 2008-2017 (Source: Data Processing).

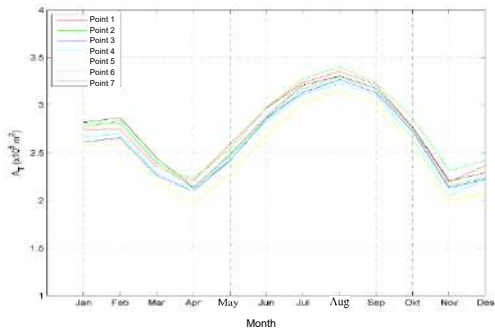


Figure 17: Heat Transfer in Monthly Condenser (A_c) 2008-2017 (Source: Data Processing).

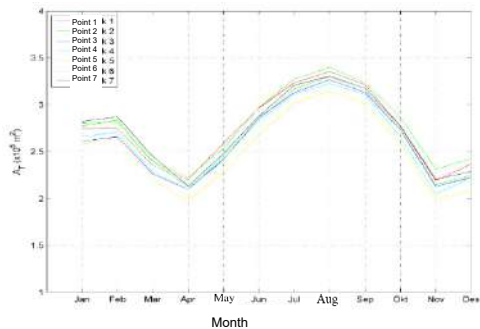


Figure 18: Total Heat Transfer Area (A_T) in 2008 -2017 (Source: Data Processing).

4.5 Calculation of OTEC Net Power and Cycle Efficiency

The results of the OTEC net power estimation results are listed in the table 1.

The highest net power for 9 years (Table 1.) was achieved in April with an average net power of 70,759 MW and the lowest occurred in August with an average net power of 69,969 MW. The maximum cycle efficiency is 0.4061 at Point 5. The highest cycle efficiency occurs in April with an average maximum value of 0.402, while the lowest cycle efficiency occurs in August with an average minimum value of 0.3704.

Table 1: OTEC Clean Power at Study Point 2008-2017.

Location	Temperature Difference ($^{\circ}\text{C}$)	Clean Power (MW)		
		Average	Min	Max
Point 1	22.969	70.348	69.922	70.711
Point 2	22.896	70.318	69.891	70.689
Point 3	23.140	70.416	69.981	70.775
Point 4	23.157	70.423	70.010	70.810
Point 5	23.327	70.491	70.064	70.863
Point 6	23.053	70.381	69.961	70.759
Point 7	22.990	70.356	69.956	70.750

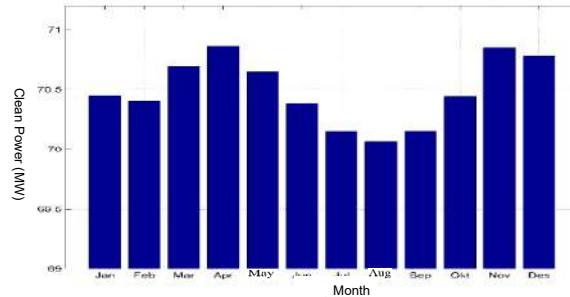


Figure 19: Monthly Net Power in 2008-2017 at the Potential Point (Source: Data Processing).

Table 2: Efficiency of the Rankine Cycle at the Study Point in 2008-2017.

Location	Clean Power Average (MW)	Efficiency		
		Average	Min	Max
Point 1	70.348	0.3855	0.3685	0.4000
Point 2	70.318	0.3843	0.3672	0.3992
Point 3	70.416	0.3882	0.3709	0.3026
Point 4	70.423	0.3885	0.3720	0.4040
Point 5	70.491	0.3912	0.3742	0.4061
Point 6	70.381	0.3868	0.3700	0.4020
Point 7	70.356	0.3858	0.3698	0.4016

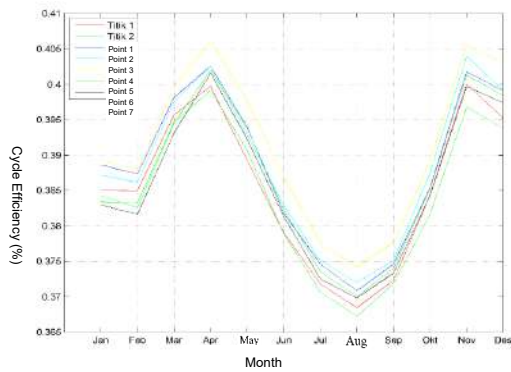


Figure 20: Efficiency of Monthly Cycles in 2008-2017 (Source: Data Processing).

4.6 Discussion of Closed Cycle OTEC System Simulation

The power of warm seawater pumps (P_{WS}) and cold seawater pump power (P_{CS}) decreases but is not very significant, while the power of the working fluid pump (P_{WF}) is stable against the increase in T_{WS} . Meanwhile, the net power (P_{NET}) increased from 70.06 MW to 70.86 MW. P_{NET} value is derived from the remaining power generated by the turbine due to be used to pump warm seawater, cold seawater, and working fluid.

The heat transfer area in the evaporator (A_E) decreases from $1.61633 \times 10^5 \text{ m}^2$ to $0.89701 \times 10^5 \text{ m}^2$. Likewise, the area of heat transfer in the condenser (A_C) which also decreased from $1.53015 \times 10^5 \text{ m}^2$ to $1.0705882 \times 10^5 \text{ m}^2$. This decrease certainly affects the value of the area of heat transfer on the condenser (A_T) which decreased from $3.14648 \times 10^5 \text{ m}^2$ to $1.967596 \times 10^5 \text{ m}^2$.

Table 3: Simulation Parameter Value Based on 100 MW Closed Cycle Simulation.

Location	Pump Power (MW)			Heat Transfer Area ($\times 10^5 \text{ m}^2$)			Net Power (MW)	Cycle Efficiency (%)
	P_{WS}	P_{CS}	P_{WF}	A_E	A_C	A_T		
Point 1	11.306	16.406	1.94	1.361	1.367	2.728	70.348	0.3855
Point 2	11.321	16.421	1.941	1.387	1.384	2.771	70.318	0.3843
Point 3	11.272	16.372	1.951	1.300	1.328	2.628	70.416	0.3882
Point 4	11.269	16.369	1.951	1.293	1.324	2.617	70.423	0.3885
Point 5	11.235	16.335	1.94	1.232	1.285	2.517	70.491	0.3912
Point 6	11.289	16.389	1.951	1.331	1.348	2.679	70.381	0.3868
Point 7	11.302	16.402	1.95	1.354	1.362	2.716	70.256	0.3858

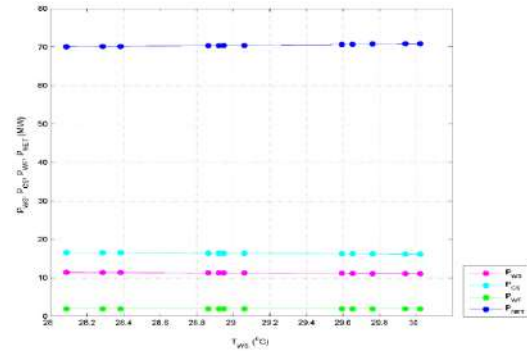


Figure 21: Relationship Between Pump Power (P_{WS} , P_{CS} , P_{WF}) and Clean Power (P_{NET}) (Source: Data Processing).

Uehara and Ikegami (1990) stated that the cost of the heat exchanger is one of the components that most cuts the cost of generation, around 25-50% of the total cost. Therefore, it is necessary to obtain the minimum objective function value by increasing the production of clean electric power and the heat exchanger needed to optimize the area of heat transfer can be suppressed.

The graph (Figure 24.) shows a linear relationship between net power and cycle efficiency. Increasing the value of net power will also increase the value of cycle efficiency. To produce a maximum electric power of 70.86 MW, the efficiency of the issued Rankine cycle reaches a maximum of 0.4061 or 40.61%. The efficiency of the Rankine cycle comes from how efficient the OTEC generator is in releasing power to pump warm seawater (P_{WS}), cold seawater (P_{CS}), and working fluid (P_{WF}). Thus, the greater the net power produced, the pump power produced must also be greater and this will increase the value of the Rankine cycle efficiency.

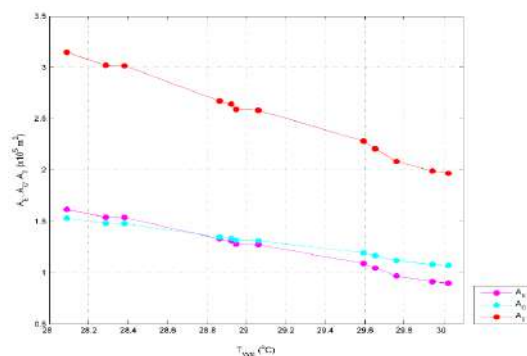


Figure 22: Relationship Between Heat Transfer Area in Evaporator and Condenser (A_E , A_C) and Total Heat Transfer Area (A_T) (Source: Data Processing).

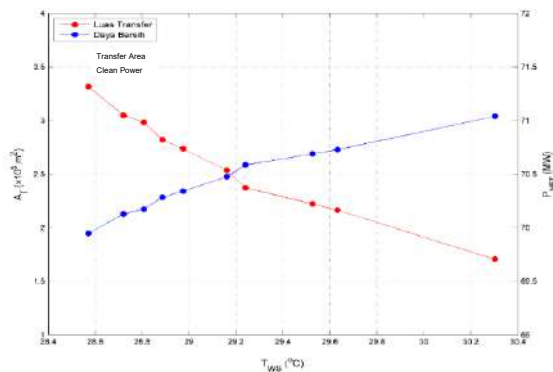


Figure 23: Relationship Between Total Heat Transfer Area (A_T) and Clean Electricity Power (P_{NET}) (Source: Data Processing).

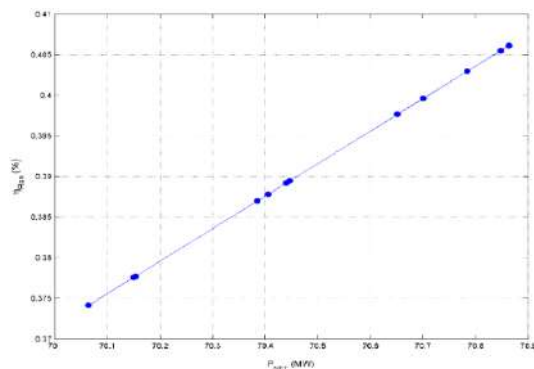


Figure 24. Relationship Between Clean Electric Power (P_{NET}) and Cycle Efficiency (η_{Ran}) (Source: Data Processing).

5 CONCLUSION

1. Minimum temperature difference requirement of 20°C at seven study points in the Bali Sea Waters was fulfilled during 2008-2017 with a range between 21.9°C - 25.3°C.
2. Potential net electric power OTEC in the Bali Sea has a range between 69.8-70.8 MW with a maximum net power found at Point 5 of 70.86 MW with a capacity of 100 MW with cycle efficiency of 0.4061 or 40.61%.
3. Based on the simulation of closed cycle OTEC, the components that describe and support the potential of OTEC are warm sea water temperature (T_{WS}) and cold water (T_{CS}) of 28°C-30°C and 5.7°C-6.4°C; warm sea water pump power (P_{WS}), cold sea water pump power (P_{CS}), and working fluid pump power (P_{WF}) of 11,048-11,543 MW, 16,148-16,634 MW, and 1.94-1.97 MW; heat transfer area on the evaporator (A_E), heat transfer area on the condenser (A_C), and the

total heat transfer area (A_T) of 0,897-1,772x105 m², 1,07059-1,62968x105 m², and 1,9676-3.401794x105 m².

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Numerical Modeling of Vertical Axis Hydro Turbine with Experimental Validation

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Keywords: Darrieus Turbine, CFD, Validation, Torque, Coefficient of Turbine Performance.

Abstract: Ocean current energy is an energy source that has great potential in Indonesia but is dominated by low current velocity. Many potential locations such as Riau, Boleng, and Mansuar strait only have the maximum current speed by 1.39 m/s, 1.5 m/s, and 1.79 m/s. Therefore, energy conversion technology is needed that is able to exploit the potential of low-speed current energy into electricity. Darrieus turbine is a type of vertical axis hydro turbine that is suitable to be developed in Indonesian waters because it can work at a lower current speed than the other types. To study the Darrieus turbine performance on a low-speed ocean current, numerical modeling based on computational fluid dynamics (CFD) which is validated with experimental results must be done. The numerical validation uses Darrieus turbine experimental data with diameter 0.91 m, chord blade 0.07 m and span 0.7 m. The validation process starts with defining boundary geometry dimension, meshing construction and turbulence model suitable. The simulation is carried out at the current speed of 1.5 m/s with TSR variations. The validation parameter is the torque value that correlates with the coefficient of turbine performance. As the result obtained validation rate with the average error of 7.3%.

1 INTRODUCTION

Ocean current energy is one of the marine energy sources that has great potential in Indonesia. Ocean currents are the movements of a mass of water caused by winds, density differences, tidal activity or long-wave movements (Daruwedho, 2016). The potential of ocean current energy in Indonesia is dominated by low current velocity. Some potential locations such as the Riau Strait, Boleng Strait, and Mansuar Strait only have a maximum current velocity of 1.39 m/s, 1.5 m/s, and 1.79 m/s (Mukhtasor, et. al., 2014). This figure is smaller than the current velocity at the location of the Sea Gen turbine placed, Northern Ireland, which is 2.5 m/s or at the location of the largest tidal current turbine namely Scotrenewables SR2000, Orkney, UK, where the current velocity is 3 m/s (MeyGen, 2019). Because Indonesia has a relatively low current velocity, research that focused on developing low-speed current turbines is important.

Darrieus turbine is a type of vertical axial tidal current turbine (VATCT) that is suitable to be developed in Indonesian waters because it is able to extract ocean currents from various directions and

works at lower current speeds compared to other turbine types (Hantoro, et.al., 2009, 2011, 2018; Rachmat, 2015; Kasharjanto, et.al., 2017). To do more advanced research about Darrieus turbine performance, numerical modeling based on computational fluid dynamics (CFD) was conducted.

CFD is a numerical program developed to quantitatively analyze fluid flow, heat transfer, pressure, energy, and chemical reactions. There are several procedures to do a simulation in CFD, such as creating a geometry model, mesh generation, definite solver, checking results, and presenting numerical output. The use of CFD methods in research must pass the validation process to ensure the numerical output are correct. According to Marsh, et. al. (2015), the validation process of the Darrieus turbine numerical modeling includes studies of boundary condition size, number of grids, and definite solver. Good numerical modeling is validated with experimental results. Therefore, this study will focus on numerical modeling procedures to obtain validated results with experimental results.

However, the use of a 3D model in a simulation requires a supercomputer facility and takes a long time for running simulation (Marsh, et. al., 2015).

According to Dai, et. al. (2009), shaft and strut parts are not needed to include in a numerical simulation of Darrieus turbine if only for analyzing a hydrodynamics performance. Without strut, the numerical modeling of a Darrieus turbine can be converted from a 3D model to a 2D model, so that with the 2D model, the simulation would become lighter and faster.

2 COMPUTATIONAL FLUID DYNAMICS OF TURBINE

2.1 RANS Governing Equations

The average motion equation for fluid flow is generally better known as Reynolds Averaged Navier-Stokes Equation. The RANS equation uses a turbulent model to model all large and small scale motion based on the average quantity of flow or model approach. The modeling approach results in a loss of all spectral effects in the time averaging process (Mulualem, 2003).

The unsteady RANS modeling approach solves the Reynolds averaged equations of mass and momentum conservation given in equation 1 and equation 2.

$$\frac{\partial \bar{U}_i}{\partial x_i} = 0 \quad (1)$$

$$\begin{aligned} \frac{\partial \bar{U}_i}{\partial t} + \frac{\partial}{\partial x_j} (\bar{U}_i \bar{U}_j) = & -\frac{1}{\rho} \int \left(\frac{\partial \bar{p}}{\partial x_i} + \delta_{i1} \frac{\partial (P)}{\partial x_1} \right) \\ & + \frac{\partial}{\partial x_j} \left(v \frac{\partial \bar{U}_i}{\partial x_j} - \overline{U'_i U'_j} \right) \\ & + \rho g_i + \bar{F}_s + \bar{F}_b \end{aligned} \quad (2)$$

where the bar $(\bar{\cdot})$ defines the time averaged components; $\bar{U}$ adalah is the Reynolds averaged velocity; $\bar{p}$ is the Reynolds averaged pressure; v is a kinematic viscosity; δ_{i1} is the Kronecker-delta, $\frac{\partial (P)}{\partial x_1}$ is driving force, a constant streamwise pressure gradient, and $\overline{U'_i U'_j}$ are the Reynolds stresses. The Reynolds stresses are defined as.

$$\overline{U'_i U'_j} = v \left(\frac{\partial \bar{U}_i}{\partial x_j} - \frac{\partial \bar{U}_j}{\partial x_i} \right) - \frac{2}{3} k \delta_{ij} \quad (3)$$

Turbulent kinetic energy and dissipation rate can be definite using equation 4 and equation 5.

Turbulent kinetic energy:

$$\frac{\partial k}{\partial t} + \frac{\partial \bar{U}_i k}{\partial x_i} = v_t S^2 - \epsilon + \frac{\partial}{\partial x_j} \left[\left(v + \frac{v_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} \right] \quad (4)$$

Dissipation rate:

$$\begin{aligned} \frac{\partial \epsilon}{\partial t} + \frac{\partial \bar{U}_i \epsilon}{\partial x_i} = & C_{\epsilon 1} \frac{\epsilon}{k} v_t S^2 - C_{\epsilon 2} \frac{\epsilon^2}{k} \\ & + \frac{\partial}{\partial x_j} \left[\left(v + \frac{v_t}{\sigma_\epsilon} \right) \frac{\partial \epsilon}{\partial x_j} \right] \end{aligned} \quad (5)$$

where S is the turbulent kinetic energy production, C_{μ} , σ_k , σ_ϵ , $C_{\epsilon 1}$, $C_{\epsilon 2}$ are model coefficients with their values 0.09, 1, 1.3, 1.44, and 1.92 (Sun, et.al., 2008; Salim, et.al., 2011).

2.2 Turbine Performance Formulations

After getting the torque value from CFD result, then the coefficient of turbine performance (C_p) can be define using the formula:

$$C_p = \frac{\text{Turbine Power}}{\text{Power Potential}} \quad (6)$$

The turbine power (P_n) and the power potential (P_t) can be define using the formula:

$$P_n = \omega \cdot \tau \quad (7)$$

$$P_t = 0.5 \rho A U^3 \quad (8)$$

The torque value (τ) is the result of the numerical modeling while ω is initial condition input. The relationship between rotational speed (ω), tip speed ratio (TSR) and current speed (U) can be expressed as:

$$TSR = \frac{\omega r}{U} \quad (9)$$

3 TURBINE EXPERIMENTAL DATA

Experimenting of the Darrieus turbine with hydrofoil NACA 63 (4) 021 has been carried out in towing tank of Hydrodynamics Laboratory at the University of British Columbia by G.W. Rawlings (2008). The mechanism of the experiment is the Darrieus turbine towed by carriage and combined with the rotation of the electric motor connected by the gearbox to set the turbine rotation during testing.

3.1 Turbine Dimension and Characteristic

The turbine has diameter of 0.91 m, consists of 3 blades hydrofoil NACA 63(4)021 with chord 0.07 m and span 0.7 m. The turbine geometry and specifications can be seen in more detail in figure 1 and table 1.

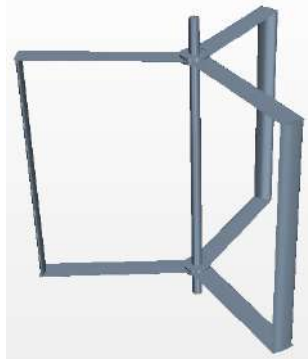


Figure 1: Darrieus turbine geometry (Rawlings, 2008).

Table 1: Principal model turbine parameters (Rawlings, 2008).

Parameter	Dimension and characteristic
Diamater turbine	0.91 m
Number of blades	3
Blade span	0.7 m
Blade profile	NACA 63(4)021
Chord length	0.07 m
Shaft diameter	0.05 m

The turbine geometry in figure 1 can be shown in the 2D sketch (top view) as shown in figure 2.

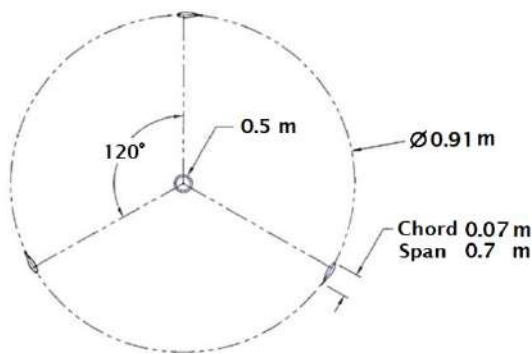


Figure 2: Turbine rotor sketch on top view (Rawlings, 2008).

3.2 Turbine Experimental Result

The turbine experimental data is reprocessed based on the turbine performance graph. The experimental

result is processed using the turbine performance formulations, as shown in point 2.2. Turbine experiment data can be seen in table 2.

Table 2: Turbine experimental result (Rawlings, 2008; reprocessed).

U (m/s)	TSR	ω (rad/s)	τ (N.m)	P_n (watt)	C_p
1.5	1.5	4.92	15.32	75.39	0.071
1.5	2.0	6.56	22.46	147.38	0.140
1.5	2.25	7.38	28.93	213.56	0.202
1.5	2.5	8.20	34.12	279.86	0.265
1.5	2.75	9.02	35.73	322.37	0.305
1.5	3.0	9.84	32.57	320.57	0.304
1.5	3.5	11.48	26.12	299.93	0.284

4 METHODOLOGY

The numerical simulation begins with creating turbine geometry in .iges format. Next, the turbine geometry is imported into the CFD program and then creates boundary condition geometry and region determination as domain modeling. For the next step, generate grid meshing and do some simulation settings such as selecting solver and input initial condition and then running the simulation.

For validating the numerical modeling output with the experimental result, firstly is to define the boundary geometry dimension, and then study grid meshing independency and compare the turbulence model. The simulation is carried out with varying TSR rate at constant current speed, 1.5 m/s.

Simulation result error is calculated using the formulation:

$$\% \text{ error} = \frac{|\text{experiment} - \text{CFD}|}{\text{experiment}} \times 100 \quad (10)$$

To get the results of numerical modeling at a certain degree of turbine rotation, the time step calculation is required. Meanwhile, to determine a total time for every single simulation, the physical time calculation is required.

The time step in this study was calculated for every 5° turbine revolute. While the physical time is determined 10 times of the turbine rotation at the current speed of 1.5 m/s.

Following is the example of calculating physical time and time step at the turbine rotational speed of 4.92 rad/s.

Physical Time (for 10 times of turbine rotation) can be calculated with the formulation:

$$\omega = \frac{\theta - \theta_0}{\text{Physical Time}} \quad (11)$$

where :

θ = degree for 1 rotation = 2π rad

θ_0 = degree for 0 rotation = 0

So that

$$\begin{aligned} \text{Physical Time} &= \frac{(2\pi - 0)\text{rad}}{4.92 \text{ rad/s}} \times 10 \\ &= 12.8 \text{ s} \end{aligned}$$

Time Step (for every 5° turbine revolute) can be calculated with the formulation:

$$\begin{aligned} \text{Time Step} &= \frac{\text{time for 1 simulation}}{\text{number of steps}} \quad (12) \\ &= \frac{1.28 \text{ s}}{\left(\frac{360^\circ}{5^\circ}\right)} \\ &= 0.018 \text{ s} \end{aligned}$$

The setting of the boundary condition domain in every numerical simulation can be seen in table 3.

Table 3: Boundary condition domain in the numerical modeling simulation.

Region	Boundary condition
Inlet	Uniform flow : 1.5 m/s
Outlet	Pressure : 0 Pa
Side Walls	Slip walls
Top	Symmetry wall
Bottom	Symmetry wall
Turbine	Wall : no slip

5 RESULT AND DISCUSSION

5.1 Defining Boundary Geometry Dimension

Numerical modeling uses 2D models so that the determination of boundary sizes is sufficient to be analyzed on the dimensions of length and width. Simulation of defining boundary dimensions using the same grid meshing composition, with input current speed of 1.5 m/s and TSR 3.5.

The numerical simulation results based on variations in boundary size can be seen in table 4 and table 5.

Table 4: Determination of boundary length.

Length variations	ω (rad/s)	C_p (experiment)	C_p (CFD)	Error
L 12D	11.48	0.284	0.286	0.8%
L 14D	11.48	0.284	0.293	2.3%
L 16D	11.48	0.284	0.285	0.4%
L 18D	11.48	0.284	0.287	1.01%

Table 5: Determination of boundary width.

Width variations	ω (rad/s)	C_p (experiment)	C_p (CFD)	Error
W 4D	11.48	0.284	0.45	59.3%
W 6D	11.48	0.284	0.34	19.4%
W 8D	11.48	0.284	0.29	0.4%
W 10D	11.48	0.284	0.28	2.8%

Based on the simulation results in table 4 and table 5, the dimension of boundary condition geometry is taken 12D in length and 8D in width, where D is the turbine diameter.

5.2 Grid Independence Analysis

The independence grid study is conducted to define the best grid composition for getting the best analyzing results. There are several considerations on the meshing process including the number of grids generated, time for meshing, time for simulating, the computer performance and the error value.

In the analysis of grid independence, the simulations are carried out with variation number of grid. Figure 3 shows grid meshing comparison between tenuous and precision density. For the next, to find out the numerical results based on the number of grid density, this study simulates Darrieus turbine on the current velocity of 1.5 m/s and TSR 3.0. The simulation results are presented in table 6.

The simulation result in table 6 shows that the numerical model with the smallest error value is produced when using meshing 5. However, consideration to the computer performance which difficult to generate meshing 9,207,251 cells and need longer time for simulation, so that it is decided to use meshing 4 with error value 3.3%. This error value is not much different from the error value of meshing 5. Moreover, meshing 4 only consists of 73,397 cells after being converted into the 2D model.

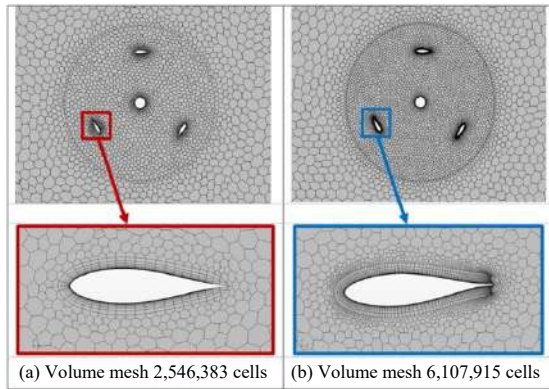


Figure 3: Comparison of meshing results when using tenuous grid (a) and precision grid (b).

Table 6: Numerical simulation results based on number grids variations.

Meshing variations	Grids number		Time for simulation (minutes)	Error
	Model 3D	Covert to 2D		
Meshing 1	1,387,775	66,410	38	20.4%
Meshing 2	2,546,383	68,619	45	18.5%
Meshing 3	3,944,757	71,394	75	15.2%
Meshing 4	6,107,915	73,397	96	3.3%
Meshing 5	9,207,251	78,295	135	0.8%

5.3 Turbulence Model Comparison

One of the factors that have a great effect on the result of the CFD analysis is the flow turbulence model. The numerical validation process in this study simulates several turbulence models to define what turbulence model produces the smallest error values. The turbulence models that compared in this study are $k-\epsilon$, $k-\omega$ SST, Spalart Allmaras and Reynolds Stress Turbulence.

Turbulence model comparison is performed at the current velocity of 1.5 m/s, TSR 2.5, and using the turbulence intensity of 0.05 (Marsh, 2015,2016). The simulation result of the turbulence model comparison can be seen in table 7.

The simulation result in table 7 shows that the smallest error is produced by $k-\omega$ SST turbulence model. This result has corresponded with many studies (Dai, 2009; Marsh, 2015; 2016; Castelli, 2010; Rahmawati, 2017; Nobile, 2011; Madi, et.al., 2019) who used the $k-\omega$ SST model in their ocean current turbine numerical model.

Table 7: The result of turbulence model comparison.

Turbulence Models	τ (experiment) (N.m)	τ (CFD) (N.m)	Error
$k-\epsilon$	34.12	38.11	11.7%
$k-\omega$ SST	34.12	36.67	7.5%
Spalart Allmaras	34.12	39.42	15.5%
Reynolds Stress Turbulence	34.12	26.62	22%

5.4 Numerical Validation Result

After defining boundary geometry dimensions, grid independency analysis, and selecting the turbulence model appropriate, then the next step is validation the numerical model of Darrieus turbine with the experimental result. The validation parameter is the torque that correlates with the performance coefficient.

5.4.1 Torque Validation

Torque simulation for the validation process is performed at the current speed of 2 m/s and TSR 2.5. The result of torque using numerical model CFD is shown in figure 4 and validation with the experimental result is shown in figure 5.

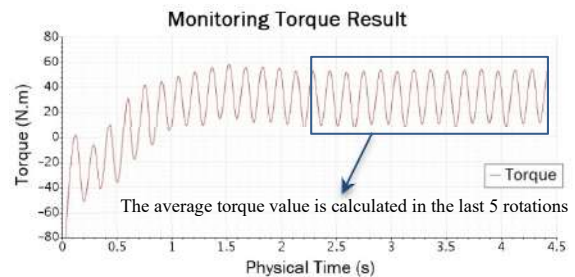


Figure 4: Torque result of numerical modeling at the current speed of 2 m/s and TSR 2.5.

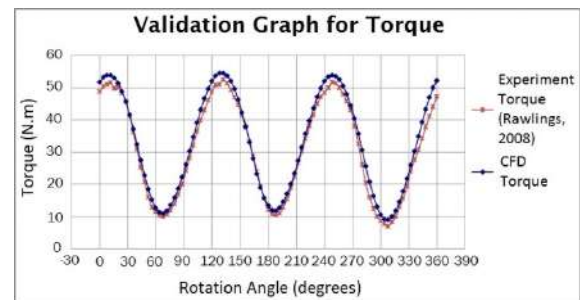


Figure 5: Torque validation at the current speed of 2 m/s and TSR 2.5 for 1 rotation.

One of the torque validation can be seen in figure 5. On the figure, both experimental torque and CFD torque are simulated at the current speed of 2 m/s and TSR 2.5. The validation torque on this condition has error value of 7.02%.

5.4.2 Coefficient of Performance (Cp) Validation

The coefficient of performance well known as turbine efficiency is the ratio between power produced by the turbine operating with the potential current energy absorbed by the turbine theoretically. Turbine efficiency and its validation can be calculated using several data such as turbine dimension, current velocity and torque result both from experimental testing and CFD analysis.

For example, turbine efficiency and its validation are calculated at the current speed of 1.5 m/s, TSR 1.5.

Using equation 8, the theoretical power potential of the turbine can be calculated

$$P_t = 0.5 \times 1000 \text{ kg/m}^3 \times (0.91 \text{ m} \times 0.7 \text{ m}) \times (1.5 \text{ m/s})^3 = 1055.64 \text{ watt}$$

Using equation 7, the turbine power from CFD analysis can be calculated.

$$\begin{aligned} P_{nCFD} &= 4.92 \text{ rad/s} \times 16.59 \text{ N.m} \\ &= 81.65 \text{ watt} \end{aligned}$$

The result of turbine efficiency from CFD analysis can be calculated using equation 6.

$$\begin{aligned} C_{pCFD} &= 81.65 \text{ watt} / 1055.64 \text{ watt} \\ &= 0.077 \end{aligned}$$

By using the experimental data in table 2, the validation of turbine efficiency can be calculated using equation 10.

$$\begin{aligned} \% \text{ error} &= ([0.071 - 0.077] / 0.07) \times 100 \\ &= 8.3 \% \end{aligned}$$

The validation results of CFD analysis and experimental data of turbine Darrieus can be seen in detail in table 8 and figure 6.

Table 8 shows the comparison between the CFD analysis and the experimental results. CFD analysis uses initial conditions which are equated with

Table 8: The validation of Darrieus turbine numerical CFD and experimental results.

U m/s	TSR	ω (rad/s)	P_t (watt)	Experimental result (Rawlings, 2008; reprocessed)			Numerical CFD			Error
				τ (N.m)	P_n (watt)	C_p	τ (N.m)	P_n (watt)	C_p	
1.5	1.5	4.92	1055.64	15.32	75.39	0.071	16.59	81.65	0.077	8.3%
1.5	2.0	6.56	1055.64	22.46	147.38	0.140	26.17	171.70	0.163	16.5%
1.5	2.25	7.38	1055.64	28.93	213.56	0.202	31.73	234.21	0.222	9.7%
1.5	2.5	8.20	1055.64	34.12	279.86	0.265	36.67	300.77	0.285	7.5%
1.5	2.75	9.02	1055.64	35.73	322.37	0.305	37.64	339.62	0.322	5.4%
1.5	3.0	9.84	1055.64	32.57	320.57	0.304	33.63	330.98	0.314	3.3%
1.5	3.5	11.48	1055.64	26.12	299.93	0.284	26.23	301.23	0.285	0.4%

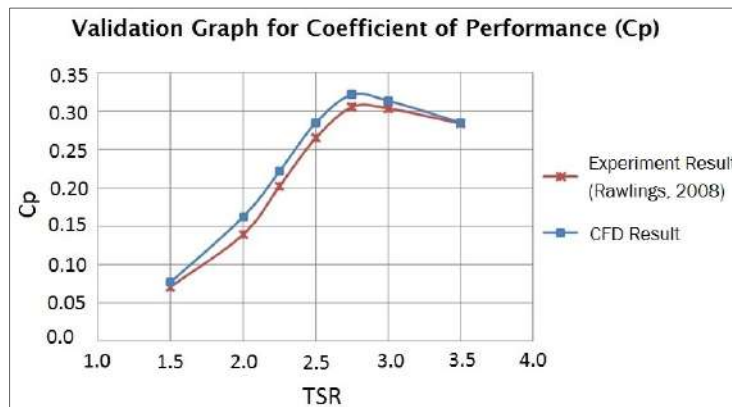


Figure 6: Darrieus turbine performance on numerical CFD and experimental results.

experimental data that are simulated at the current speed of 1.5 m/s with TSR variations between 1.5 to 3.5. Validation results are obtained numerical model error value at TSR 1.5 is 8.3%, TSR 2.0 is 16.5%, TSR 2.25 is 9.7%, TSR 2.5 is 7.5%, TSR 2.75 is 5.4%, TSR 3.0 is 3.3%, and at TSR 3.5 is 0.4%.

Figure 6 shows that the C_p of the CFD analysis results tend to be higher than the C_p of the experimental results because numerical analysis uses 2D models that ignore the influence of the turbine arm in the simulation. Basically the turbine arm will produce a drag force which can reduce turbine performance.

The numerical modeling results of the Darrieus turbine are presented in the flow direction vector that trough pass the turbine and flow velocity contour as shown in figures 7 and 8. In these figures, the Darrieus turbine is simulated at the free stream speed of 1.5 m/s with TSR 2.75 (figure 7) and TSR 3.5 (figure 8).

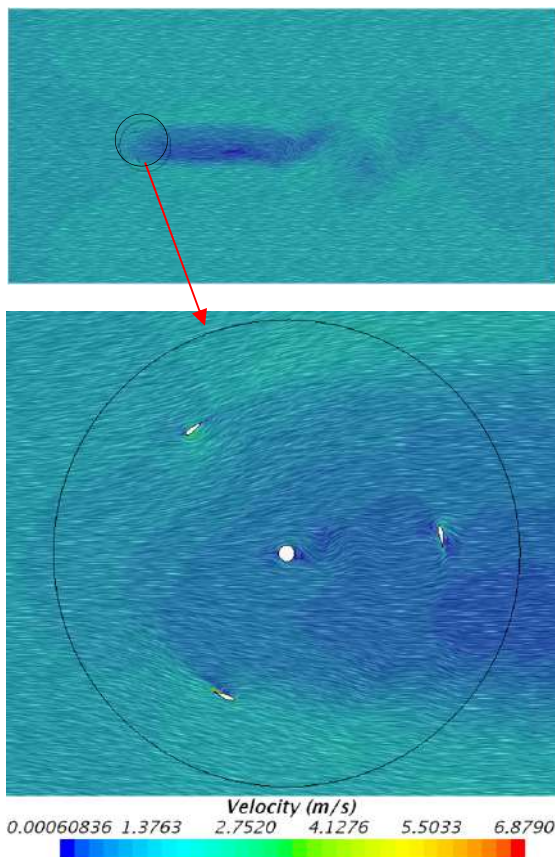


Figure 7: Flow velocity direction on the turbine, rotating at TSR 2.75.

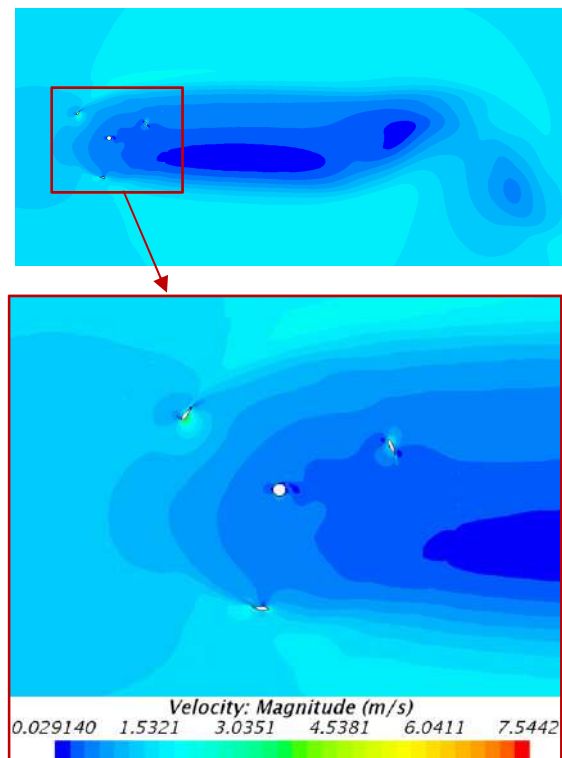


Figure 8: Flow velocity contour on the turbine, rotating at TSR 3.5.

6 CONCLUSION

Validation of the Darrieus turbine numerical model starts from defining the boundary geometry dimension, study the grid independency, and selecting the turbulence model appropriate. The numerical modeling error value at TSR 1.5 is 8.3%, TSR 2.0 is 16.5%, TSR 2.25 is 9.7%, TSR 2.5 is 7.5%, TSR 2.75 is 5.4%, TSR 3.0 is 3.3%, and at TSR 3.5 is 0.4%. The average error is 7.3% which is categorized as a good agreement result.

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Evaluation of Underwater Pipeline Design Criteria Due to Safety Requirement based Hydrodynamic and External Load

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Keywords: Pipeline, Free Span, Rock Berm.

Abstract: Several problems can be solved by piping engineers to protect pipelines so it can be resistant to environmental and man-made hazards. Factors that can damage the pipeline are usually shipwrecks, faults in dropping and pulling anchors, dredging activities, fishing activities, and the exploration of undersea. Burial pipelines are the solutions most often used by engineers for the protective pipeline. However, the burial of the pipeline cannot always be applied due to unfavorable seabed issues or if there are other factors. This paper will explain the results of research related to the pipeline by calculating the dimensions of rock berms to protect pipelines from external loads and analysis of the free span pipeline to check the feasibility of being exposed to environmental loads.

1 INTRODUCTION

The Oil and Gas Industry is an important sector in national development both in terms of meeting the needs of energy and industrial raw materials in the country as well as producing foreign exchange so that the management needs to be done as optimal as possible. The underwater pipeline is one of the most efficient long-distance transportation infrastructure for oil and gas for the transfer of oil and gas production both from exploration on land, near the coast and from the deep sea with effective and efficient methods. Failure in the pipeline system can be caused by various problems, such as free span (due to environmental loads), and anchor loads (external loads) from ships that are leaning in the Madura Strait area. A free span occurs due to vibration or commonly known as the Vortex-Induced Vibration (VIV) phenomenon that occurs in parts of the pipe that touches the seabed. Stability analysis of the pipeline from the environmental load (wave and current) is very important because it can determine feasibility design of the pipeline (length of the free span) that has been installed so that in the future preventive steps can be planned for the best. Pipeline protection from external loads is also important because if the pipeline is hit by anchor load it can cause damage such as buckling and pipe leakage so that it disrupts the oil and gas distribution process and causes environmental pollution.

2 RESEARCH DATA

In this study, the area to be analyzed is the Madura Strait, and the pipeline that will be designed for rock berm protection and free span analysis are the Block BD pipeline. In the analysis of concrete armor design (rock berm) design, an external load size calculation will be performed. External loads have a big role in damaging the pipeline system on the seabed, in this case, the movement anchor from the seabed. In determining the size of the anchor, an analysis of ship mobility that often crosses the study area will be carried out, namely the Madura Strait. The following are the data used in this study:

Table 1: Ship Sailing Data in the Madura Strait.

(source: PT. Pelindo III)

Ship Type	In 2012
Container Ship	2040
General Cargo Ship	2144
Bag Cargo Ship	558
Fuel Tank Ship	1264
Liq. Bulk Non Fuel Ship	447
Dry Bulk Ship	616
Barge	5908
Passenger Ship	1889

Table 2: Ship Capacity Data in the Madura Strait.

(source: PT. Pelindo III)

Ship Type	Max. Capacity
Passenger Ship	15.000 GT
Cargo Ship	5.000 DWT
Ferry Ship	10.000 DWT
Roro Ship	5.000 DWT
Tanker Ship	5.000 DWT

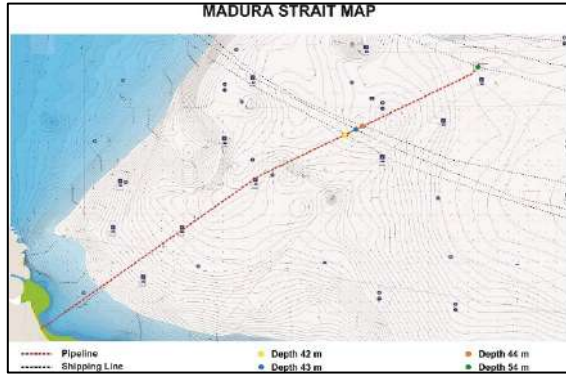


Figure 1: The waters map in the Madura Strait (source : navionics.com and maps.google.com).

In this study for wind data, current velocity, and wave data are at coordinates with longitude 113.066248 E and Latitude 7.567736 S over 5 years namely 2014-2019 with ground data using granules D50 = 0.03 m.

Table 3: Wind Data in the Madura Strait.

Date	Time (GMT +7)	Wind Dir (deg)	Wind Speed (knot)	High Wave Sign. (m)	Curr. Speed (cm/s)
01-12-2014	00.00	169.26	3.4	0.01	0.51
02-12-2014	01.00	173.02	3.56	0.01	0.62
03-12-2014	02.00	176.46	3.72	0.01	0.69
...	...	...	...	...	...
13-07-2019	17.00	144.27	2.08	0.01	4.28
14-07-2019	18.00	139.43	1.99	0.01	4.45
15-07-2019	19.00	134.2	1.93	0.01	4.61

Table 4: Pipeline Data.

Parameter	Value	Unit
Inner diameter of pipe	404.14 (16)	mm (inch)
Wall thickness of pipe	13 (0.5)	mm (inch)
Wall thickness of concrete	60 (2.4)	mm (inch)
Nominal diameter	0.45	m
Allowable free span	23.8 (78)	m (ft)
Length of pipe	53	km
Effective Mass	282.68	kg/m
Period	20	year

3 RESEARCH DATA PROCESSING

3.1 Free Span Calculation

Free span calculation is used to determine the maximum span length so that the stress that occurs in the free span does not exceed the yield stress of the pipe material. The flow of waves and currents that arise around the pipe, arises a vortex that results in pressure distribution. This vortex produces oscillations/vibrations in the pipe. If the frequency of this vortex approaches the natural frequency of the pipe, resonance occurs, and this causes fatigue in the pipe (Yong Bai, 1981).

3.2 Calculation of Critical Span Length

In Boyun Guo (2005), critical span length or pipe length without support where oscillations occur due to currents is a relationship between the natural frequency of the pipe span and the reduced velocity. The critical span length for cross-flow motion is

$$L_s = \sqrt{\frac{C_e U_r D}{2\pi}} \sqrt{\frac{EI}{M_e}} \quad (1)$$

In Boyun Guo (2005), the natural frequency of a pipe depends on the stiffness of the pipe, the condition of the end of the pipe span, the span length, and the effective mass of the pipe. The natural frequency equation of the pipe is as follows:

$$f_n = \frac{C_e}{2\pi} \sqrt{\frac{EI}{M_e L_s^4}} \quad (2)$$

f_n = frequency of pipe natural (Hz)

L_s = longspan (m)
 M_e = massa of effectivity pipe (kg/m)
 C_e = 9.87 pin-pin
 E = 2.07 E+11 (N/m²)
 I = 0.00018 (kg/m²)

The critical span lengths for in-flow motion are:

$$L_s = \sqrt{\frac{C_e f_n}{2\pi} \sqrt{\frac{EI}{M_e}}} \quad (3)$$

L_s = Span long of critical (m)
 C_e = Span end constants
 U_r = Reduced Velocitty (m/s)
 D = Diameter of outer pipe (m)
 M_e = Mass of effectivity pipe, (kg/m)

After the calculations we have:

Table 5: Critical length of span.

Parameter	Water Depth			
	42 m	43 m	44 m	54 m
U_r cross flow	5	4.7	4.8	5
U_r in flow	1.4	1.4	1.4	1.4
L_s cross flow (m)	34.57	33.52	33.87	34.57
f_n	0.48	0.51	0.5	0.48
L_s in line (m)	16.49	17	16.83	16.49

For most projects, the allowable span length is the critical span length calculated for in-line motion. However, when economic factors are taken into consideration, the length of the critical range calculated for the cross-flow movement can be chosen.

3.3 Free Span Due to Scouring

Spans in the pipeline can arise due to local scour of sea-floor sediments or where the pipe routes through the seabed are irregular. When the lower current passes through the pipe, separately vortices are formed from the top and bottom of the pipe. This causes fluctuations in hydrodynamic forces which can produce large oscillations or spans in the direction of cross-flow when the frequency of vortex shedding approaches the natural span of vibration. Pipe failures which can be caused by vortex movement can be prevented if the vortex shedding frequency is far enough from the natural frequency of the pipe stretch so that the dynamic oscillation of the pipe can be minimized. The frequency of vortex shedding can be written:

$$f_s = \frac{S V_{eff}}{D} \quad (4)$$

f_s = frekuensi vortex shedding
 S = Strouhal Number
 V_e = effective current speed (m/s)
 D = Diameter of Pipe (m)

Strouhal number is a function of Reynolds' number of current flow. The drag coefficient is also a function of Reynolds' number.

The relationship between the drag coefficient with the Strouhal number is:

$$S = \frac{0,21}{C_d^{0,75}}$$

For practical problems, usually, the Strouhal number is taken at 0.2.

After calculating the results obtained as follows:

Table 6: Free Span.

Parameter	Water Depth			
	42 m	43 m	44 m	54 m
S	0.207	0.201	0.195	0.183
Free Span	0.147	0.131	0.116	0.043

Mousselli (1981) states that the pipeline stretch has begun to oscillate when the shedding frequency is 1/3 of the natural frequency of the vibration of the pipe stretch. To design pipe vortex shedding frequency comparison is smaller than 0.7 times the natural frequency of the pipe stretch so that oscillation does not occur. So it can be written that oscillation does not appear if: $f_s \leq 0.7 f_n$. Based on calculations and limitations that $f_s \leq 0.7 f_n$, the pipe design in the BD block is feasible when viewed from the scouring analysis of the free span, namely:

Table 7: Analysis of freespan.

Water Depth	Parameter			Check
	f_s	f_n	$0.7 f_n$	
42 m	0.147	0.477	0.334	OK
43 m	0.131	0.507	0.355	OK
44 m	0.116	0.497	0.348	OK
54 m	0.043	0.477	0.334	OK

4 ANCHOR CALCULATION

4.1 Calculation of the Main Dimensions of the Ship

Anchor calculations are performed to determine dimensions, number of anchors needed, anchor

weight, and chain dimensions. In this study, the anchors of a 15000 GT Passenger Boat with 10000 DWT Ferry Ships will be compared with the following data:

Table 8: Data of Ship Comparison for Passenger Ship 15000 GT.

(source: equasis.com)

Ship Name	GT (m ³)	DWT (ton)	Lpp (m)	B (m)	T (m)	Vs (knot)
Nggapulu	14739	3175	146.5	23.4	6	15.6
Sinabung	14716	3485	146.5	23.7	6	16.3
Bukit siguntang	14643	3686	146.5	26.5	5.7	15.5
Ciremai	14581	3480	144.8	26.8	5.5	16
Dobonsolo	14581	3500	146.5	23.7	6	15.7
Doro londa	14685	3175	146.5	23	5.9	15.1
Kambuna	14501	3434	144.8	23.7	5.9	17.4
Kelud	14665	3537	146.5	23.7	5.8	17.3
Lambelu	14649	3685	136.03	23.7	5.5	15.9
Rinjani	14501	3434	144.8	23.7	5.7	17.1
Tidar	14501	3200	144	23.7	6.1	15.4
Umsini	14501	3434	141	22	6	12.5

Table 9: Data of Ship Comparison for Ferry Ship 10000 DWT.

(source: equasis.com)

Ship Name	GT (m ³)	DWT (ton)	Lpp (m)	B (m)	T (m)	Vs (knot)
Europalink	9653	46124	218.8	30.5	6.9	14.7
Finnlady	9653	45923	218.72	30.52	6.8	22.6
Finnmaid	9653	45923	218.77	30.5	6.7	21.3
Finnstar	9653	45923	218.77	30.5	6.7	21.6
Finnswan	9653	45923	218.8	30.5	6.9	14.9
La superba	9750	49257	211.5	30.4	7.3	22.7
La suprema	9720	49257	211.5	30.4	7.2	22.8
Skane	8787	42705	200.2	29.6	5.9	11.2
Spirit of britain	9500	47592	212	31.4	6	16.7
Spirit of france	9884	47592	212	31.4	6	14.8
Stena adventurer	9487	43532	211.56	29.88	5.5	19
Ulysses	9665	50938	209.08	31.84	6.3	18.7

From comparison ship data tables 3.5 and 3.6 can be made the relationship graph between GT and DWT, GT with Lpp, GT with B, GT with T, and GT with Vs for 15,000 GT Passenger Ship and also graph the relationship between DWT and GT, DWT with Lpp, DWT with B, DWT with T, and DWT with Vs for 10,000 DWT Ferry Ships to determine the size of the main dimensions of the ship to be measured anchored using linear regression equations.

After obtaining a graph from the comparison, from the linear regression above (each equation), the values of the main dimensions of the 15,000 GT Passenger Boat and the 10,000 DWT Ferry Ship are as follows:

Table 10: The Main dimensions of Passenger Ships 15000 GT.

Parameter	Value	Units
GT	15000	m ³
DWT	3429.5	ton
Lpp	148.58	m
B	25	m
T	6	m
Vs	15.68	knot

Table 11: The Main Dimensions of Ferry Ships 10000 DWT.

Parameter	Value	Units
DWT	10000	ton
GT	50156	m ³
Lpp	221.5	m
B	30.3	m
T	6.5	m
Vs	19.2	knot

4.2 Determination of Ship Coefficient

Based on Froude numbers, CB can be calculated with the formula Watson-Gilfilla, CM, and CWP can be searched by equations in the book "Parametric Ship Design" page 11. Furthermore, the length of LWL, LCB, ∇ , and Δ can be calculated, which are:

$$\text{Froude Number (Fn): } \frac{Vs}{\sqrt{g Lpp}} = 0.412 \quad (5)$$

$$\text{CB : Block Coefficient: } -4.22 + 27.8 \cdot \sqrt{(\text{Fn})} - 39.1 \cdot \text{Fn} + 46.4 \cdot \text{Fn} \quad (6)$$

$$\text{CM: Midship Coefficient: } 0.977 + 0.085 \cdot B - 0.6 \quad (7)$$

$$\text{CWP: Waterplane Coefficient: } 0.180 + 0.860 \cdot \text{CP} \quad (8)$$

$$\text{LCB: Longitudinal Center of Buoyancy: } 8.80 - 38.9 \cdot \text{Fn} \quad (9)$$

$$\text{CP: Prismatic Coefficient: } C_B/C_M \quad (10)$$

$$\nabla: \text{Volume Displacement: } L \cdot B \cdot T \cdot \text{CB} \quad (11)$$

$$\Delta: \text{Displacement: } \nabla \cdot \rho \quad (12)$$

Where, $\rho = 1.025 \text{ ton/m}^3$

Based on the explanation and formula above the results of the calculation of the coefficient of the ship based on the dimensions of the ship obtained are as follows:

Table 12: The dimensions of the Passenger Ship 15000 GT based on the coefficient of the ship.

Parameter	Value	Unit
Cb	0.75	-
Cm	0.99	-
Cp	0.76	-
Cwp	0.84	-
Lcb	7.19	behind of the midship
Δ	805.69	m ³
∇	825.83	ton

Table 13: The Dimensions of the Ferry Ship 10000 GT based on the coefficient of the ship.

Parameter	Value	Unit
Cb	0.76	-
Cm	0.99	-
Cp	0.77	-
Cwp	0.84	-
Lcb	7.23	behind of the midship
Δ	1075.29	m ³
∇	1102.17	ton

4.3 Anchor Dimensions and Weight

Based on BKI Vol. II of 2001 section 18-2, the Z number can be calculated using the following formula:

$$Z = D2/3 + 2.h.B + A/10 \quad (13)$$

Where,

D2/3: Represents the amount of water displaced (displacement) when the waterline is in summer in seawater which has ρ seawater 1.025 tons / m³

H: The effective height is measured from the line of loading water in summer to the highest end of the deck.

B: Ship Width

A: The area (m²) is the appearance of the hull profile, superstructure and houses which have a width greater than B / 4 which is above the loading line in summer including length L and above from height h.

LWL: LPP + (3% * LPP)

D2/3: (LWL x B x T x CB)^{2/3}

In calculating h, it is assumed to be the upper building and the deck is 2.4 m, so the upper building and house building

$$h : Fb + \sum h$$

$$: (H-T) + (\text{Number of Floors} \times \text{Floor Height})$$

$$A: \text{LWL} \times T$$

Calculate Equipment Number (Z) with the following equation:

$$Z: D2/3 + (2 \times h \times B) + A/10 \quad (14)$$

After calculating the results obtained are as follows:

Table 14: Calculation of Equipment Number (Z) of Passenger Ship 15000 GT.

Parameter	Value	Unit
Lwl	153	m
D2/3	664.76	-
H	14.06	-
A	911.23	-
Z	1456.1	-

Table 15: Calculation of Equipment Number (Z) of Ferry Ship 10000 DWT.

Parameter	Value	Unit
Lwl	228.14	m
D2/3	1048.2	-
H	15.43	-
A	1475.2	-
Z	2129.8	-

Based on the calculation, the Z value of the 15,000 GT Passenger Ship is 1456,104 while the Z value of the 10,000 DWT Ferry Ship is 2129.88. From the results of calculations prove that the 10,000 DWT Ferry has a Z value greater than the Z value of the 15,000 GT Passenger ship. Because the greater the Z value, the greater the anchor obtained from the BKI table, so in this study we used the Z value of the 10,000 DWT Ferry with a Z value of 2129.88.

Based on the table BKI Volume II 2006 section 18, then with a value of Z = 2129.79 obtained anchor data as follows.

- Number of anchor bower: 2 anchor
 - Anchor Bower Weight : 6450 kg
 - Anchor Chain
 - Length : 605 m
 - Diameter
 - D1 : 81 mm
 - D2 : 70 mm
 - D3 : 62 mm
 - Mooring Rope
 - Amount : 5 pieces
 - Length : 200 m
 - Broken Load : 425 kN
 - Pull Rope
 - Length : 240 m
 - Broken Load : 1260 kN
- Anchor weight = 6450 kg, then from the catalog obtained anchor dimensions that will be used on this ship are:
- A = 2920 mm
 - B = 2046 mm
 - C = 906 mm
 - D = 1885 mm
 - E = 1461 mm
 - ØF = 110 mm

4.4 Anchor Chain Determination

After getting the data from the anchor, the anchor chain is selected from the catalog, namely by:

- a. Total Length Selected: 605 m
 - b. Diameter of chain anchor selected: 81 mm
- Komposisi dan kontruksi dari rantai jangka meliputi:
1. Common link
 - 1). $1,00 d = 81 \text{ mm}$
 - 2). $6,00 d = 486 \text{ mm}$
 - 3). $3,60 d = 291.6 \text{ mm}$
 2. Enlarge Link
 - 1). $1,1 d = 89.1 \text{ mm}$
 - 2). $6,6 d = 534,6 \text{ mm}$
 - 3). $4,0 d = 324 \text{ mm}$
 3. End Link
 - 1). $1,2 d = 97,2 \text{ mm}$
 - 2). $6,75 d = 546,75 \text{ mm}$
 - 3). $4,0 d = 324 \text{ mm}$

Based on the calculation of the dimensions that have been obtained the following is an illustration of the anchor size obtained from the calculation results based on table Z BKI.

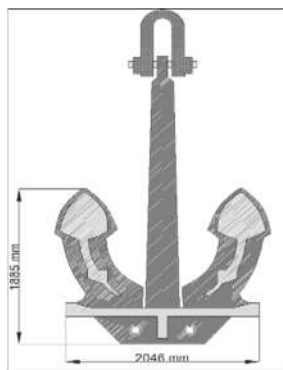


Figure 2: The dimension of anchor in the front look.

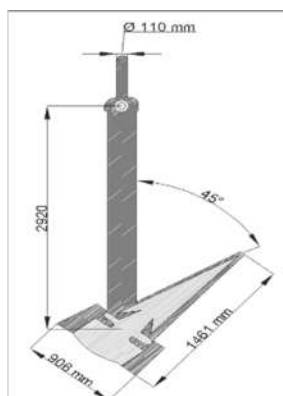


Figure 3: The dimension of anchor in the beside.

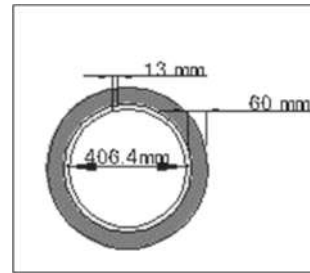


Figure 4: The dimension of pipe and layer in the concrete.

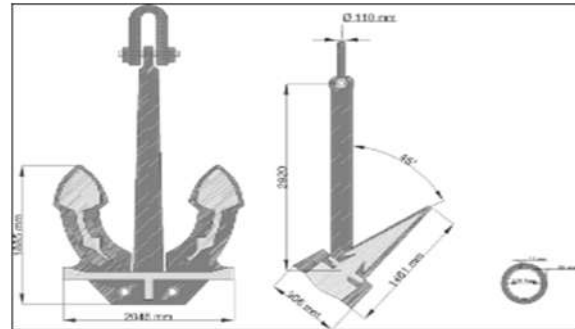


Figure 5: The comparison of Anchor and Pipe.

5 CALCULATION OF ROCK BERM

The use of berm rock is a common practice to protect pipes against collisions from fishing gear such as trawlers and trawlers. Rock berms must be able to withstand horizontal impact loads, which mainly depend on the following:

- The shape and mass of a trawler
- Trawling speed
- Direction of attraction
- Seabed conditions

5.1 Dimensions of Rock Berm

The rule of thumb for the design of suitable protection against anchor anchors has been derived from tests carried out for 20 years and is mainly used in connection with the following rock berm parameters:

- Protective stone size (D50)
- The thickness of the protective layer
- Filter layer thickness (if applicable)
- Minimum width of the rocky peak
- Minimum width of the berm rock base

Below is a visualization of the structure of berm rock based on the calculations:

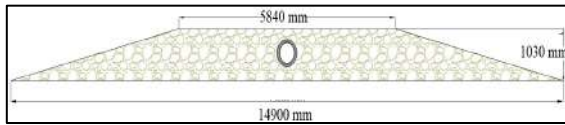


Figure 6: Rock berm illustration.

Based on calculations, the dimensions of the stone which can be sufficient against the drag anchor are as follows:

Table 16: Rock Berm Dimension.

Rock berm dimension	Value
D ₅₀ , armour	• 4 * 81 mm = 0.32 m
H _{min} , armour	• 1.46 m * sin(45) = 1.03 m • 3 * 0.32 m = 0.97 m
H _{min} , filter	• 1.5 * 0.32 m = 0.49m • 0.3 m
B _{min} , top	• 2 * 2.92m = 5.84m
B _{min} , bottom	• 5.84 m + 2*2.5*(1.33+0.48 m) = 14.9 m

6 CONCLUSION AND SUGGESTION

6.1 Conclusion

Based on the analysis, the free span parameters in the pipeline show that they are still feasible and do not require handling to overcome the free span. the pipeline design to free span with a limit of $fs \leq 0.7$ fn: (Refer to **Table 7**).

Furthermore, from the analysis of the anchor weight, the dimension of berm rock which is used as protection for pipeline on the bottom of the sea from anchor threats. Dimensions of rock berms are:

B_{top} : 5.84 m
B_{bottom} : 14.9 m
H_{armour} : 1.03 m

(Refer to **Table 16**).

6.2 Suggestion

The suggestion of this research is:

When doing free span calculations it would be better if done with 3D modeling, the author has not done 3D

modeling to provide a clearer visual appearance to the reader.

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Modeling of Total Suspended Solid based on Remote Sensing Reclamation Data of Teluk Lamong Port

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Abstract: In relation to socio-economic development many construction projects have been carried out in Indonesia to meet the requirements of urban, agricultural and industrial use. These projects include urban construction, for example, metro construction, groundwater pumping, and reclamation in coastal areas. Activities that have a significant influence on the environment are port operational activities. Teluk Lamong Harbor is the reclaimed land product. Based on this, a physical impact study of the Teluk Lamong Bay Multipurpose Terminal Development Port is needed to find out how much influence the impact of the reclamation has on marine environmental resources. Therefore a flow pattern modeling is done, Total Suspended Solid (TSS) modeling is based on remote sensing data to see the impact after the development of the Teluk Lamong Multipurpose Terminal Port. The objectives to be achieved in this study are: Knowing the modeling of current patterns at the Port of Multipurpose Terminal in Teluk Lamong 2020, Knowing the modeling of TSS concentrations at the Port of Multipurpose Port of Lamong Bay in 2020. Stages Method: 1. modeling the flow patterns using Mike 21 software before and after reclamation, 2. TSS concentration modeling using Mike 21 software based on remote sensing data using Google Earth Engine (GEE). Model of the current pattern at low tide the maximum water level at low tide is -1.0875 meters and -1.055 meters, the speed of the current under these conditions ranges between 0.01 m / s for both points. For the current tide pattern model with the maximum water level height at tide conditions are 0.7744 meters and 0.7640 meters, the current speed under these conditions ranges from 0.008 m / s for the yellow dot and 0.008-0.016 m / s for both blacks. (D50) sediment concentration of 0.0738 mm around the Port of Teluk Lamong with an average TSS value of 4.998 per area.

1 INTRODUCTION

Indonesia has a coastal region that stretches from Sabang to Marauke which is rich and a variety of natural resources that have been utilized by the community as one source of economic income. In addition to providing a variety of resources, Indonesia's coastal area has various other functions, such as transportation in the form of a port, industrial estate, agribusiness and agro-industry, recreation and tourism, residential areas, and waste disposal sites. One of the development ideas that is being discussed by many Indonesians is the concept of reclamation. Reclamation can be defined as an effort to improve, utilize, restore capacity, and improve the quality of land through the empowerment of various technologies and community empowerment focused on land that is naturally of low quality or as a result of human influence that makes the land less

productive. With changes in environmental conditions can make the coastal region undergo complex changes. The development of Teluk Lamong Multipurpose Port will have an impact on the surrounding environment due to changes in environmental conditions that adjust to environmental changes due to coastal reclamation. The construction of the Lamong Bay Multipurpose Port regarding reclamation has been explained and legalized as contained in Law Number 27 of 2007 article 34 junto Law 1 of 2014, concerning Management of Coastal Areas and Small Islands, explains that reclamation has the meaning of activities carried out by people in the context of increasing the benefits of land resources in terms of environmental and socio-economic aspects by means of confinement and drying of land. Coastal changes that occur due to abrasion and accretion, the main causes of abrasion and accretion are the action of

waves, wind and tides. The most influencing process is waves. When moving towards the beach, the waves undergo transformation which then generates currents near the coast. Currents moving along the coast move sediments, causing shoreline changes. The change in coastline is related to sediment transport that occurred at the port of Teluk Lamong. The rate of sedimentation increases, so siltation at the Port of Teluk Lamong will have an even faster impact on other problems. Therefore, knowledge about aquatic hydrodynamics is very important to understand in order to predict the distribution of sediment after reclamation. The results of the model that have been validated and have shown correlations or similarities with actual conditions in the field can be used to predict the dynamics of various processes that occur in the waters.

2 THEORETICAL

The ecological coastal area is a transitional area between terrestrial and marine ecosystems, which is towards the coastal area including land areas, both dry and submerged in water affected by marine processes, such as tides, sea winds, and sea water intrusion, while towards the sea area coastal areas include ocean waters that are influenced by natural processes such as sedimentation and freshwater flow, as well as those caused by human activities on land (Dahuri et al, 1996). According to Law No.27 / 2007 article 1 paragraph 2, coastal areas are transitional areas between terrestrial and marine ecosystems which are influenced by regulations on land and sea. Coastal waters are seas bordering the plains covering waters as far as 12 nautical miles measured from the coastline, waters connecting the coast and islands, estuaries, bays, shallow waters, swamps, brackish, and lagoons (Law No.27 / 2007 article 1). In coastal areas there are interrelated ecosystems. Coastal ecosystem is a unit that interacts between organisms and the environment and together carry out their respective functions in habitat (Odum, 1971). Coastal ecosystems are a set of biological (biotic) and non-biological (abiotic) components that are absolutely necessary for life and improve the quality of life (Bengen, 2004). Furthermore, biological and non-biological components are functionally related to each other and interact with each other to form a system. If there is a change in one of the existing systems, it will affect both the functional structure unity and balance (Bengen, 2002). One form of linkages between ecosystems in coastal areas is the movement of river water, runoff, runoff with various

materials contained (nutrients, sedimentation, and pollutants) which will all lead to coastal waters. In addition, this pattern of movement of water mass will also play a role in the movement of aquatic biota (plankton, fish, and shrimp) and pollutants from one location to another (Bengen, 2004). Ecosystems in the coastal and marine areas are natural and artificial. Natural ecosystems located in coastal areas, namely mangrove forests, coral reefs, seagrass, sand beaches, rocky beaches, and estuary waters.

2.1 Reclamation

Coastal reclamation is carried out taking into account the socio-economic conditions of the population, given the increasingly rapid growth rate, which causes the land for development increasingly narrow. Reclamation makes watery areas that are damaged or of less value become better and more useful. The new area is usually used for residential, industrial, business, and urban areas, ports, and tourist attractions. In the theory of urban planning, coastal reclamation is one step in city expansion. Usually the reclamation is carried out by the state or big city with the rate of growth and land needs increasing rapidly, but experiencing constraints of land limitations. This condition is no longer possible to expand to the mainland, so new land is needed. Another alternative is to form a vertical division by building a port as part of the distribution of goods and anchoring ships. Coastal reclamation is a subsystem of the coastal system.

2.2 Impact of Coastal Reclamation

Based on the Minister of Home Affairs Regulation (PERMENDAGRI) No. 1 of 2008 concerning coastal reclamation, the implementation of coastal reclamation must pay attention to environmental interests, ports, mangrove forest areas, fishermen, and other functions in the coastal area and the sustainability of the surrounding coastal ecosystem. Planning in reclamation activities should be aligned with the city spatial plan. The new city spatial planning must pay attention to the social and ecological carrying capacity of the city. Reclamation project activities around the coastal area require a scientific feasibility study through a technical study of how much environmental damage will be caused and then conveyed openly to the public. It is important to remember that reclamation is a form of human intervention in the balance of the natural environment of the coast. In a coastal ecosystem that has long been formed and arranged as it should, it will

lose its balance due to reclamation activities. The effect of these impacts is one of which affects the lives of surrounding communities. Many fishermen and workers in the fisheries sector will lose their livelihoods due to the decrease in the number of fish due to damage to the ecosystem due to runoff from sediments (Francisca, 2017).

2.3 Limitations of Environmental Carrying Capacity Due to Coastal Reclamation

According to Dahuri (2000) the main problems in the management of coastal areas are activities of pollution, overfishing, erosion, coastal sedimentation, extinction of biota types, and conflicts in the use of the region: due to the high environmental pressure caused by the population along with all the gait of life and development of the environment of the coastal area which has a limited ability to support the concept of the environment based on the idea that the environment has the maximum capacity to support the growth of organisms (Bengen, 2002). The environmental carrying capacity classification includes:

1. Ecological: The maximum level (both quantity and volume) of the use of a resource or ecosystem that can be accommodated by an area before ecological decline;
2. Physical: The maximum amount of utilization of a resource or ecosystem that can be absorbed by an area without causing physical quality degradation.
3. Social: The level of comfort and appreciation of users of a resource or ecosystem to an area due to the presence of other users at the same time.
4. Economy: The level of scale of effort in the utilization of a resource that provides maximum economic benefit on a sustainable basis.

According to Scones (1993) the carrying capacity of the environment is divided into two namely ecological carrying capacity and economic carrying capacity. Ecological carrying capacity is the maximum number of organisms on a land that can be supported without causing death due to density factors, as well as the occurrence of permanent environmental damage. While economic carrying capacity is the level of production of businesses that provide maximum profit and is determined by the business objectives economically.

2.4 Tidal

Knowledge about tides will be very important when

we plan to build a port by coastal reclamation. The ups and downs that occur in each region are not the same. In general, tides in various regions can be divided into 4 types:

1. A single daily tidal (diurnal tide)
In one day there is one tide and one tide with a tidal period is 24 hours 50 minutes.
2. Install double daily shrinkage (semi diurnal tide)
In one day there are two tides and two tides with almost the same height and tides occur regularly. The average tidal type is 12 hours 24 minutes.
3. Tidal mixture tends to a single daily (mixed tide prevalling diurnal)
In one day there is one tide and one tide, but sometimes temporarily there are two tides and two tides with a very different height and period.
4. Tidal mixture tends to double daily (mixed tide prevalling diurnal)
In one day there were two tides and two times low tide, but the height and period were different.

According to Haryono (2004), tides are the result of gravitational forces and centrifugal effects. Centrifugal effect is the movement or push towards the outside center of rotation of the earth. The influence of the moon's gravitational force is two times greater than the force of the sun in generating the tides, because the distance of the moon is closer than the distance of the sun to the earth. Within a month, the daily variation of the tidal range changes systematically to the lunar cycle. The tidal range also joins to the shape of the volume of the water and the shape of the ocean floor undergoing movement. Vertical water movements associated with the ups and downs of the tides, accompanied by horizontal water movements called tidal currents.

2.5 Wave

The main energy that forms the coastal system is waves. Ocean waves are a natural phenomenon in the form of raising and decreasing water slowly and can be found throughout the world. Waves in the sea often appear irregular and change frequently. This can be observed from the surface of the water which is caused by the direction of wave propagation that is very varied and irregular waveforms, especially if the waves are under the influence of the wind. According Pratikto (2000) said that the shape and propagation of waves that vary and irregularly greatly affect the characteristics of the waves that occur in these waters. In addition to changes in height, length and speed of waves also occur other phenomena such as silting, refraction, diffraction and reflection before the wave breaks. The process resulted in changes in wave

speed. Sea waves that move into coastal waters experience a high increase which makes the waves to increase. Furthermore, the speed of water particles at the peak of the wave approaches the speed of the wave, so that when the water particles are greater than the speed of the wave, the wave becomes unstable and breaks (Duxbury, et. Al., 1994).

2.6 Current

The current is a very broad movement of water, often occurring throughout the ocean. Waves that come towards the coast can cause nearshore current. Currents are also formed due to the wind blowing in a very long time interval, can also be caused by waves that form the beach obliquely. It can also be caused by waves coming towards the coastline. Currents can also carry suspended sediments or those found at the bottom of the sea. Circulation of ocean currents is divided into two categories, namely surface circulation and circulation in the sea (intermediate or deep circulation). Sea surface currents are generally driven by wind stress acting at sea level. Wind tends to push the layer of water on the surface of the sea in the direction of the movement of the wind. But because of the influence of the earth's rotation or the influence of the Coriolis force. The variation of currents generated by the wind against depth is explained theoretically by Ekman (1905).

2.7 Total Suspended Solid

TSS are suspended materials ($> 1\mu\text{m}$ diameter) that are held in a filter with a pore diameter of $0.45\mu\text{m}$. TSS consists of mud and fine sand and microorganisms. The main causes of TSS in waters are soil erosion or soil erosion carried into water bodies. TSS concentration if too high will inhibit the penetration of light into water and cause disruption of photosynthesis. The spread of TSS in coastal and estuary waters is influenced by several physical factors including wind, rainfall, waves, currents and tides (Effendi, 2000). Sastrawijaya (2000) states that TSS concentrations in waters generally consist of phytoplankton, zooplankton, human waste, animal waste, mud, crop and animal residues, and industrial waste. Materials that are suspended in natural waters are not toxic, will occur if excessive amounts can increase the value of turbidity which further inhibits the penetration of sunlight into pools of water (Effendi, 2000). Remote sensing technology has been widely applied to study water quality, one of which is TSS. The quality of waters has different light penetration in certain areas, can be determined by

multispectral techniques (Barret and Curtis, 1982). The higher TSS concentration will have a higher reflectance value.

3 METHOD

This research was conducted at the Lamip Bay Multipurpose Terminal Port, located in the border area between Surabaya City and Gresik Regency, which is adjacent to two ports owned by PT Pelabuhan Indonesia III, namely Gresik Port to the west and Tanjung Perak Main Port to the east. The selection of research sites is done purposively. Based on the results of reading the literature and information related to the existence of port development by means of coastal reclamation that can affect coastal resources, this location was chosen to be used as a research location. Based on the Port Development will affect the condition of coastal ecosystems and the livelihoods of coastal communities, it is necessary to study the impact of coastal resources on the development of Teluk Lamong Multipurpose Terminal Port from the aspect of physical impact. The research was carried out starting in the third semester, with the research location in Figure 1 and the survey point in Figure 2.



Figure 1: Research Sites.



Figure 2: Location of the Research River.

3.1 Systematics and Research Limits

This thesis research is carried out for one semester, while the research activities include:

1. Problem Formulation.
This stage as the identification of problems, determinants or focus of a study. The main problem of this research is the factors or impacts caused by coastal resources namely, the pattern of water currents, TSS in the construction of the Port of the Lamip Multipurpose terminal.
2. Literature Study.
This activity is carried out to gather information related to research in the form of theoretical concepts and relevant matters. Literature sources obtained from the internet, reports related institutions, scientific articles, journals, media, books, and other documents.
3. Data Processing.
Namely knowing the analysis of shoreline changes, modeling the pattern of water currents, and modeling TSS concentrations based on sensing data long before and after the construction of the Teluk Lamong Multipurpose Port terminal
4. Data Validation.
Data validation is an activity carried out to measure the extent of differences and errors.
5. Results and Discussion.
It is a modeling of waters flow patterns in 2012 and 2020 based on Navy Hydrographic and Oceanographic Center data (DISHIDROS) and the National Meteorology, Climatology and Geophysics Agency (BMKG), and TSS concentration modeling based on remote sensing data of the 2A Sentinel Image 2020 National Institute of Aviation and Space (BMKG), and modeling of TSS concentrations based on remote sensing data of the 2A Sentinel Image in 2020. LAPAN). The input from this data processing explains the correlation of the coastal condition of the Teluk Lamong Multipurpose Port terminal with the physical environmental conditions based on 2020.
6. Conclusion.
It is the answer of the research objective, which is to know the current pattern and sediment distribution.

3.2 Data Processing Stages

In the stages of managing the impact study data before and after the construction of the Teluk Lamong Multipurpose Port:

1. Analyze shoreline changes using DSAS software

based on Landsat 7 in 2012 and Landsat 8 in 2020 satellite image data.

2. Determine the focal point of TSS research using remote sensing aids, namely Sentinel 2A satellite imagery recording on January of 2020 then processed using Google Earth Engine at the LAPAN Agency.
3. Modeling the condition of water flow patterns in the Port of Multipurpose Bay in Lamong Bay based on wind, tidal and bathymetry data processed using Mike 21 software.
4. TSS modeling using Mike 21 software based on remote sensing data.

4 RESULTS

4.1 Total Suspended Solid Remote Sensing

TSS is suspended material that causes water turbidity consisting of mud, fine sand and microorganisms mainly caused by soil erosion or water-borne erosion (Effendi, 2003). TSS is one of the important factors to measure water quality based on physical aspects including the addition of solids both organic and inorganic material into the waters so as to increase turbidity which will further inhibit the penetration of sunlight into water bodies. The amount of TSS that is in the waters can reduce the availability of dissolved oxygen. So the high TSS can also directly disrupt aquatic biota. To identify the TSS of Teluk Lamong Port, TSS remote sensing was carried out using Sentinel 2A imagery in 2020 and Landsat 8 satellite imagery in 2017 and 2019. Remote sensing of the distribution of TSS at Teluk Lamong Port using Sentinel 2A satellite imagery in 2020 using Google Earth Engine Software software. In the work of remote sensing the first step taken is to make a mosaic. Data analisi *Total Suspended Solid* (TSS) Data processing from Sentinel 2A satellite imagery in 2020 uses Google earth engine software that can produce TSS remote sensing output. The channels in the Sentinel satellite image data processing are used to obtain reflectance values which are used to estimate TSS concentrations. The first stage, radiometric correction is done to eliminate errors in the sun's elevation angle and the sun-earth distance. The second conversion from the DN value to the radiant value. Third is the conversion of the radiant value to the reflectance value. Fourth conduct TSS analysis.

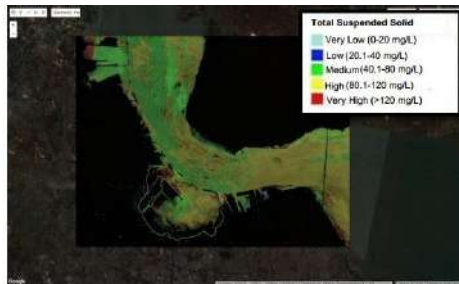


Figure 3: Distribution Total Suspended Solid.

4.2 Modeling of Teluk Lamong Harbor

4.2.1 Wind

One that influences the speed and direction of the current is the wind factor. In Mike 21 modeling, the use of wind data as input wind data forcing contained in the hydrodynamics module. In addition to influencing the pattern of current movement, wind can influence wave generation, so that in relation to sediment transport these three factors are intermittent. The wind data input used in March 2020 was adjusted for the timestep modeling simulation conducted. The following diagram of the rose wind shows the dominant direction coming from west to southeast.



Figure 4: March winds.

4.2.2 Depth

Data on seabed depth is a supporting data used in this study so it is necessary to know the seabed condition of the Gulf waters, using topex satellite imagery with file format in the form of SRTM in 2011 to create a year boundary depth of Teluk Lamong Port. The data is managed using Global Mapper 14 software which is done by generating contours with a contour interval of 0.1 meters. Furthermore, the data plot has been processed in Surfer 11 software to get a bathymetry contour map in the form of 2D maps. Then the data is meshed on a map plot in Arcgis 10.61 software used in the bathymetry plot in Mike 21 software. The input file is boundary of land and water.

Next is the display of the xyz input file modeling results in Mike 21 software.

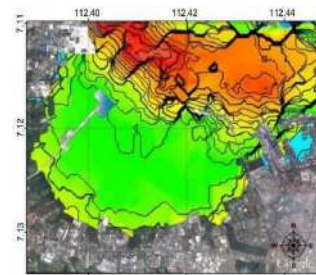


Figure 5: Depth.

4.2.3 Tides

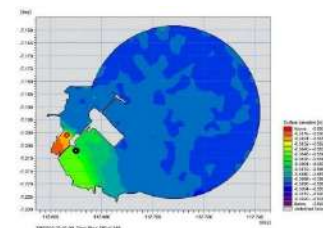


Figure 6: Low Tide Conditions.

For the model of current patterns at low tide in Figure 6 occurred on timestep 191 dated March 8, 2020 at 23:00 with the maximum water level at low tide conditions is -1.0875 meters at the red point and -1.055 meters at the black point, the speed of the current at this condition range between 0.01 m / s for both points.

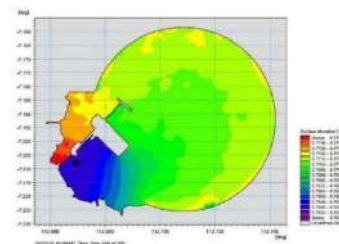


Figure 7: High Tide Conditions.

For the current tide pattern model in Figure 7 occurs on timestep 184 dated March 8, 2020 with the maximum water level at tide conditions is 0.7744 meters for the yellow point and 0.7640 meters for the black point, the current speed in this condition ranges from 0.008 m / s for the yellow dot and 0,008-0,016 m / s for the second black.

Tidal Validation. Tidal conditions are analyzed using the harmonic analysis method, this method has

a tidal pair hypothesis is the sum of several wave components that have a certain amplitude and frequency. This analysis is to obtain the amplitude and phase of the tidal components. Tidal recording and forecasting is carried out for 15 days in a row with observation points $7^{\circ} 12' 17''$ LS and $112^{\circ} 40' 46''$ East. The high observations that were started were analyzed using the Admiralty method and verified with the Root Mean Square Error (RMSE) and Cost Function (CF) methods. The following is the tidal data presented in Table 2 and Table 3. While graphs representing tidal data for 2012 and 2020 are presented in Figure 12.

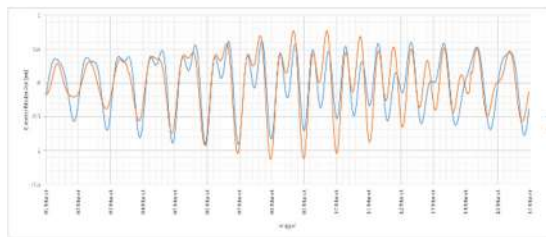


Figure 8: Tide Comparison Charts for 2012 and 2020.

Admiralty Analysis. Calculation using the admiralty method, which is a calculation to find the value of the amplitude (A) and phase difference (g_0) of the observation data for 15 or 29 pigs (observation days) and the mean sea level (S0) which has been corrected (Smoothing). The following are tidal components of the 2012 admiralty method analysis results and 2020.

Table 1: Tidal Component 2012.

Tidal Component Result										
	S0	M2	S2	N2	K1	O1	M4	MS4	K2	P1
A Cm	0.000	0.259	0.181	0.086	0.331	0.256	0.007	0.007	0.049	0.109
g°		282.07	179.70	30.07	104.52	67.42	147.06	282.48	179.70	104.52

The types of tides in the Madura Strait waters in 2012 according to Formzahl (F) numbers based on the tidal components in the table above are as follows.

$$\begin{aligned}
 F &= (K1+O1) / (M2+S2) \\
 &= (0.331+0.256) / (0.259+0.181) \\
 &= 1.336
 \end{aligned}$$

Table 2: Tidal Component 2020.

Tidal Component Result										
	S0	M2	S2	N2	K1	O1	M4	MS4	K2	P1
A Cm	0.000	0.253	0.180	0.105	0.381	0.239	0.002	0.003	0.049	0.126
g°		249.03	189.46	4.63	125.04	4.21	157.48	256.17	189.46	125.04

Tidal types in the waters of the Madura Strait in 2020 according to the Formzahl (F) number based on the tidal components in the table above are as follows.

$$\begin{aligned}
 F &= (K1+O1) / (M2+S2) \\
 &= (0.381+0.239) / (0.253+0.180) \\
 &= 1.433
 \end{aligned}$$

The explanation of the tidal components above is as follows:

- S0 is MSL
- M2 is a tidal component that is influenced by the moon's gravitational phenomenon with circular orbits and parallel to the earth's equator.
- S2 is a tidal component that is influenced by the phenomenon of solar gravity with circular orbits and the earth's equatorial alignment.
- N2 is a tidal component that is influenced by the phenomenon of changes in the distance of the moon to the earth due to elliptical paths.
- K1 is a tidal component that is influenced by the phenomenon of the declination of the moon and sun systems.
- O1 is a tidal component that is influenced by the phenomenon of moon declination.
- M4 is a tidal component that is affected by the phenomenon of twice the angular velocity of M2.
- MS4 is a tidal component that is influenced by the phenomenon of interaction between M2 and S2.
- K2 is a tidal component that is influenced by the phenomenon of changes in the distance of the moon to the earth due to elliptical paths.
- P1 is a tidal component that is influenced by the phenomenon of solar declination.

Table 3: Classification of Tidal Type.

Value	Tidal Type	Phenomenon
$0 < F < 0.25$	Coupled Daily	2x pairs a day with the same relative height
$0.25 < F < 1.5$	The mixture tends to double daily	2x a day with different height and different intervals
$1.5 < F < 3$	The mixture tends to single daily	1x or 2x pairs a day with different interval
$F > 3$	Single Daily	1x pair, when spring occurs 2x pairs a day

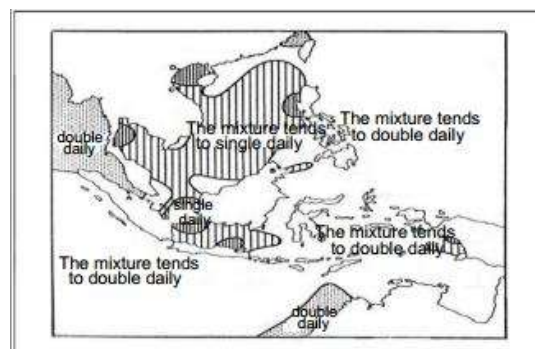


Figure 9: Distribution of Tidal Types in Indonesian Waters (Triatmodjo, 1999).

Formzahl's numbers are 1,336 and 1,433, according to Table 4.9, the numbers are classified as $0.25 < F < 1.5$ or a mixture of double daily skew. In Figure 2, the tidal type in the Madura Strait Waters indicated by a red circle is the Mixed Daily Condong Mix.

Data Verification. 2012 Tidal Analysis is used as a reference for verification of 2020 data. Quantitatively by calculating the amount of errors that occur from each data can be calculated by:

$$RMSE = \left(\frac{\sum (y_i - \hat{y}_i)^2}{n} \right)^{1/2}$$

dimana:

RMSE : Root Mean Square Error

y : data of 2012

$\hat{y}$: data of 2020

n: amount of data

Based on the results of analysis and verification of tides obtained RMSE value of 0.253 or a level of confidence of 74.7%.

Model verification is used to determine the accuracy of both wind data sources quantitatively by calculating the amount of error that occurs from each data. Wind speed verification uses a Cost Function (CF) statistical analysis. According to George et al. (2010), the calculation of the CF method can be done with the formula:

$$CF = \frac{1}{N} \sum_{n=1}^N \frac{|Dn - Mn|}{\sigma D}$$

dengan

$$\sigma D = \sqrt{\frac{1}{N} \sum_{n=1}^N (Dn - \bar{D})^2}$$

Where N is the amount of data; n is nth data; D is the 2012 data value; σD is the standard deviation; M is the 2020 data value; $\bar{D}$ is the 2020 average data and CF is the Cost Function. According to George et al. (2010), the criteria used are:

$CF < 1$ = Sangat Baik $1 < CF < 2$ = Baik

$2 < CF < 3$ = Less $CF > 3$ = Very Less

Based on the analysis, it is known that CF is obtained by 0.485 This means that verification of data for 2012 and 2020 is in the range of $CF < 1$ or very good.

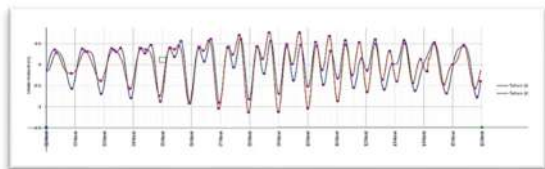


Figure 10: Tidal 2012 and 2020.

4.3 Total Suspended Solid

Sediments on the coast cannot be separated due to oceanographic factors that occur in the waters around the coast. Oceanographic factors that influence sediment type and sediment distribution are tides, wind, waves and currents. Sediment distribution modeling uses the parameters of currents generated by tides and wave effects. Sediment sample data based on the journal Sediment Distribution Pattern Analysis to Support Maintenance of Port Waters Depth Using 3D Hydrodynamic Modeling by Pratomo et al., 2017 with a density value of 1188 kg/m³, dry density 884 kg/m³ and sediment concentration of (D50) 0, 0738 mm. Sediment transportation is important to know the speed of sediment, especially supine sediment. For non cohesive sediments, such as sand, sedimentation velocity depends on the density of sediment and water mass, water viscosity, dimensions and shape of sediment particles (Triatmodjo, 2016). Also in calculating the rate of sedimentation hydrometer analysis is needed which aims to determine the grain size of the suspended sediment which intends to determine the speed of deposition of soil grains in water using Stoke's Law, with the formula;

$$V = \frac{1}{8} + \mu \frac{Dxg}{\eta} (\gamma S - \gamma w)$$

D : grain diameter (mm)

V : speed of settling soil grains (cm / s)

γS : grain weight (g / cm³)

γw : weight of water content (g / cm³)

μ : water thickness

Sediment transportation is the transfer of material from one place to another. This transfer is in the form of an increase (inflow) or reduction (outflow). (Achmad, 2011).

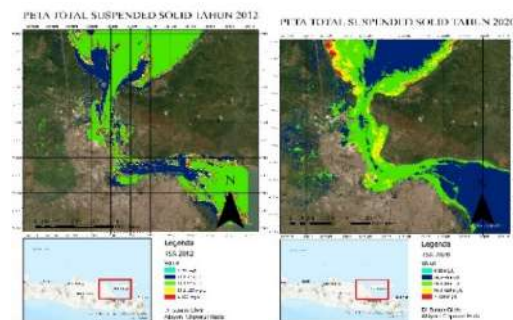


Figure 11: Remote Sensing of Total Suspended Solid.

Sources of sediment data based on the journal Study of the Impact of the Reclamation Plan in Lamong Bay, East Java Province on Tidal Flow

Patterns and Sediment Transports and data on the tide and tide flow velocity results of Mike 21 modeling based on BIG 2012 data Tidal current velocity of 0.2 m / s, tidal current velocity of 0.1 m/s and (D50) sediment concentration of 0.0738 mm.

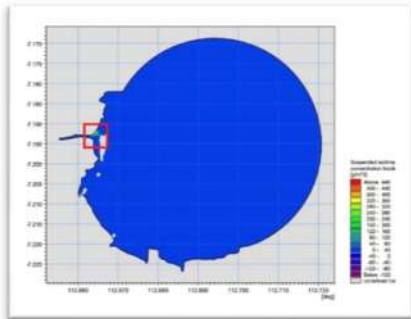


Figure 12: TSS of Low Tide 2012.

Based on observations of suspended sediment at low tide in Figure 12 on March 5, 2012 at 22:00 timestep 115. It can be seen that the value is quite high in the area around the Lamong River with values ranging from 320-360 g / m³.

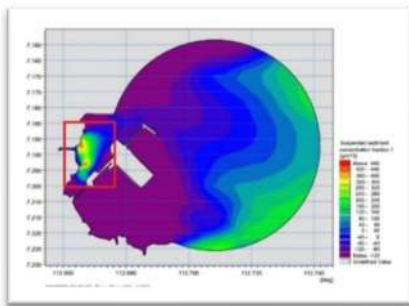


Figure 13: TSS of Low Tide 2020.

Based on observations of suspended sediment at low tide in Figure 13 on March 8, 2020, 23:00 timestep 191. There are high values in the area around the Lamong River with values ranging from 360-400 g / m³.

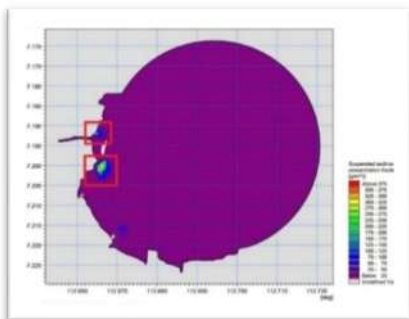


Figure 14: TSS of High Tide 2012.

Based on observations of suspended sediment at low tide in Figure 14 on March 7, 2012 at 17:00 timestep 161. It can be seen that the value is quite high in the area around the Lamong River at two observation points with values ranging from 325-350 g/m³.

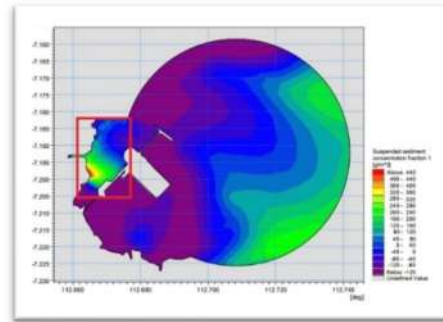


Figure 15: TSS of High Tide 2020.

Based on observations of suspended sediment at low tide in Figure 15 on March 8, 2020 at 16:00 timestep 184. There are high values in the area around the Lamong River at two observation points with values ranging from 440 g/m³. Changes in the bottom profile of the water can occur due to the sedimentation process or due to silting. Factors affecting the sedimentation or siltation process include the movement of currents, waves, and tides as well as new buildings created on the coast of Lamong Bay. In this study, sediment changes were observed based on changes in the bed base change (bed level change) located in the area within the Port of Teluk Lamong. Modeling observations were carried out by comparing changes in the bottom profile of the water after modeling simulations for 15 days with the object of the port research conditions before reclamation and after beach reclamation. Based on the results of the pomedelan, TSS conditions are greater in the Lamong River. Lamong River is part of the Bengawan Solo River Basin Unit which is managed by the Solo Bengawan River Basin. The Lamong River estuary is a tidal area, where at high tide the area is submerged in seawater, but at low tide this area becomes landlocked. As a result of this land tends to increase sedimentation processes so that the land area becomes wider. The Lamong River downstream flows on the alluvial land with a gentle slope, so sediment transport is dominated by fine fractions with relatively large amounts that can settle at the estuary (Sulistyaningsih, 2000). The following is data on river flow and river sediment concentrations which empties into Lamong Bay based on research by Alwafi Pujiraharjo (2013) with the title Study of the

Impact of the Reclamation Plan in Lamong Bay, East Java Province on Tidal Flow Patterns and Sediment Transportation. With the reclamation of the waters of the lamong bay, the narrowing of the flow of water around the lamong river and the silting of galang island, the velocity of the flow between the lamong river becomes large. This will result in greater sediment transport from rivers. Likewise, at low tide, outflows from lamong bay pass through the lamong river area. The currents are quite large at high tide but at low tide, the flow velocity in the lamong river and galang island areas becomes so small that the sediment transported will be easily deposited and gradually cause siltation.

5 CONCLUSION

The model of the current pattern at low tide in Figure 6. The maximum water level at low tide is -1.0875 meters at the red point and -1.055 meters at the black point, the current velocity at this condition ranges between 0.01 m / s for both points. For the current tide pattern model in Figure 7 occurs with the maximum water surface height at tide conditions is 0.7744 meters for the yellow point and 0.7640 meters for the black point, the current speed in this condition ranges from 0.008 m / s for the yellow point and 0.008-0.016 m / s for the second black. Tidal current velocity of 0.2 m / s, tidal current velocity of 0.1 m / s and (D50) sediment concentration of 0.0738 mm around the Port of Teluk Lamong with an average TSS value of 4.998 per area.

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Variation Number of Blades for Performance Enhancement for Vertical Axis Current Turbine in Low Water Velocity in Indonesia

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Keywords: Vertical Axis Current Turbine, Number of Blades, Performance Enhancement, Low Water Velocity.

Abstract: Asosiasi Energi Laut Indonesia (ASELI) has provided the results of the ratification of the energy potential of ocean currents in Indonesia amounting to 17,989 MW. This amount is very large considering that Indonesia is classified as in low water velocity. The vertical axis current turbine is proposed by many researchers because they have lower performance than the horizontal axis current turbine. So this study proposes research to improve the performance of the vertical axis current turbine for low water velocity by varying the number of blades. The type of blade NACA 634021 chose because has good performance for the vertical axis current turbine of Darrieus type. The model of the turbine is simulated by Computational Fluid Dynamics (CFD) with variations of the number of blades namely, 3, 4, and 5. The totals of the statistic element of meshing are 16,090, 167,020, and 174,375 at the number of blades 3, 4, and 5, respectively. The inlet velocity set at 1.5 m/s and tip speed ratio (TSR) are 1.5, 2, and 2.5. The final results of this study show that the blade numbers can improve the performance of the turbine model on all TSR ranges. The blades 4 and 5 are increased by 21.8% and 15% respectively from the blades 3 on TSR 2.5. The highest performance is obtained by the number of blades 4 on TSR 2.5 with the value of the power coefficient (C_p) 0.26.

1 INTRODUCTION

One of the technologies that are usually used for the current energy source is the turbine. The turbine is the main equipment in addition to the generator (Madi et al., 2019). Generally, the turbine that is used in the world operated in the higher current speed, such as in Columbia 1.5-2.5 m/s (Rawlings, 2008), Italy 2 m/s (Castelli et al., 2013), China 3-4 m/s (Jing, 2014), Korea 3 m/s (Quang Le et. al, 2014), and Australia 1.5-2 m/s (Marsh et al., 2015). Whereas several potential locations in Indonesia are classified in the low water velocity, they can only achieve a maximum current speed of 1.39, 1.5, and 1.79 m/s at Riau strait, Boleng strait and Mansuar strait, respectively (Mukhtasor et al., 2014; Satrio et al., 2018). So, the turbine which already exists in the world, cannot be applied in Indonesia.

Asosiasi Energi Laut Indonesia (ASELI) has provided the results of the ratification of the energy potential of ocean currents in Indonesia amounting to 17,989 Megawatt (Mukhtasor et al., 2014). The amount is very abundant considering that Indonesia is classified as in the low water velocity. Therefore,

the turbine is needed that can be applied in the low water velocity in Indonesia.

In general, the turbine consists of two based on its rotating axis namely, the vertical axis current turbine (VACT) and horizontal axis current turbine (HACT) (Khan et al., 2009; Hydrovolts, 2006 and Duvoy et al., 2012). The VACT can respond to any flow direction, so the results of efficiency will stable. Whereas the HACT only responds in one flow direction, so the results of efficiency will not stable (Kirke and Lazauskus, 2011; Zeiner, 2015; Bachant and Wosnik, 2015; Satrio et al., 2016). Therefore, this study case will focus on the design of VACT.

Particularly, the VACT consists of two namely, Darrieus type and Savonius type (Khan et al., 2009). The turbine of the Darrieus type is higher efficiency than the Savonius type. However, in practice, the efficiency is still under the HACT. It is a challenge for researchers in this research. This study case will try to improve efficiency the VACT of Darrieus type in the low water velocity, by varying the number of blades 3, 4, and 5 (Fig. 1). The blades are simulated by Computational Fluid Dynamics (CFD).

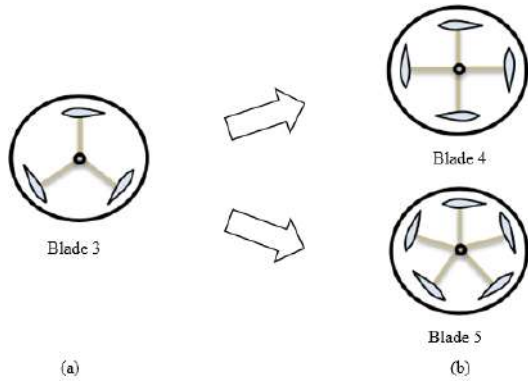


Figure 1: (a) Basic turbine, (b) variations turbine.

The type of blade NACA 634021 is chosen for this study because it is similar to the morphology of humpback whales flippers (Fish and Battle, 1995). The humpback whales are the most maneuverable of other species and have a symmetric body so that they will be stable and fast to catch prey (Johari et al., 2007). The blade profile of NACA 634021 in the VACT of Darrieus type has a good performance (Marsh et al., 2015). The profile of NACA 634021 can be shown in Fig. 2.

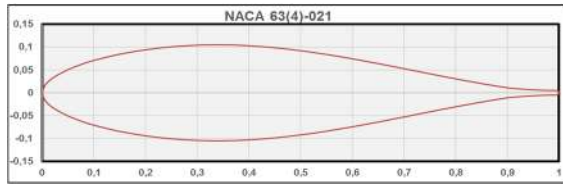


Figure 2: The profile of NACA 634021.

2 NUMERICAL SIMULATIONS

2.1 Turbine Geometry

Three types of turbine designs are simulated to evaluate the influence of variations of the number of blades. The turbine geometry (Table 1) is used based on the published 3D CFD model data by Marsh et al (2015). The turbine with three blades namely basic turbine (Fig. 1a) is designed and simulated for compared with the results of the 3D CFD model by

Table 1: Turbine Geometry.

Geometry	Dimensions
Blade type	NACA 634021
Chord length (C)	0,065 m
Number of blades (N)	3, 4 and 5
Diameter of turbine (D)	0,914 m
Height of turbine (H)	0,686 m

Marsh et al (2015). After that, the variations turbine (Fig. 1b) is designed and simulated to obtain the influence of the performance of the turbine.

2.2 Key Performance Parameters

The performance parameters of the turbine blades are investigated for this study namely, solidity (σ), torque (T), and power of the turbine (P_t).

The solidity of the turbine is defined as the ratio of rotor blade surface area to the frontal, swept area that the rotor passes through (Li, 2010), where,

$$\sigma = \frac{NC}{2\pi R} \quad (1)$$

where N is the number of blades, C is chord length (m), and R is the radius of the turbine (m).

The torque of the turbine represents the torque coefficient. The comparison of torque of the turbine and hydrodynamic subsystem is called by the torque coefficient (C_t), where,

$$C_t = \frac{T}{0,5 \rho A V^2 R} \quad (2)$$

where ρ is the density of the water (998.2 kg/m³), A is the turbine swept area (HxD), and V is current velocity (m/s).

The power of the turbine represents the power coefficient (C_p), where,

$$C_p = \frac{P_t}{P_a} \quad (3)$$

where P_t is the power mechanic of the turbine (watt) and P_a is the power kinetic of the water (watt).

The power of the turbine is obtained from the value of the torque and the rotational speed, shown in equation 4. Whereas the power kinetic is obtained of available in the water (equation 5).

$$P_t = T \omega \quad (4)$$

where ω is the rotational speed (rad/s) and T is the torque of the turbine (N.m).

$$P_a = 0,5 \rho A V^3 \quad (5)$$

2.3 Boundary Conditions

The dimension of domain boundary in this study is determined namely 10D x 20D (Fig. 3). The length and width of domain boundary 20D and 10D respectively were studied and recommended by Marsh et al (2015). The domain boundary consists of two main zones, namely, rotary, and stationary for using the meshing method. The position of the circular turbine is on the 5D from the inlet, 15D from the outlet, and in the middle of the symmetry wall.

The boundary conditions are simulated in the 2D CFD model with the use of free stream conditions. In this study case, the turbine without use arms and shaft. The inlet and outlet conditions are set in the velocity of 1.5 m/s and pressure 0 Pa, respectively. The walls are set as symmetry so that the fluid has the same distributions at the top and bottom of the walls.

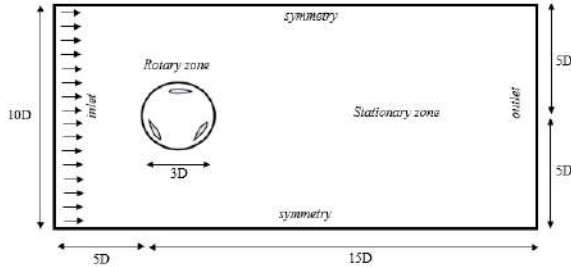


Figure 3: Domain boundary of the turbine model.

2.4 Meshing

The meshing is part of designing CFD simulations, a net strategy which is a process that determines the final results of design studies using CFD. The use, the number of elements in the meshing that is more and finer will produce a better the final results, and vice versa. However, the number of elements that are many and smooth requires a long time. So, it takes a meshing strategy with smooth elements and not at the same time. In this study, the statistics of meshing elements are used 16,090, 167,020, and 174,375 at the blades 3, 4, and 5, respectively. The number of elements based on the criteria for grid independence a study was conducted by Marsh et al (2017).

In this study, the structure of meshing uses the triangle method. The process of meshing structure in the 2D CFD model is arranged so that the rotary zone is made denser than the stationary zone and the blade zone is made denser than the rotary zone. The results of the meshing structure in this study can be shown in Fig. 4.

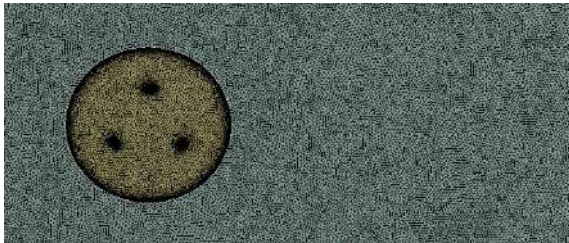


Figure 4: Meshing of domain boundary the basic turbine.

The mesh density is refined (Fig. 5) in the area of the blades, interior fluids, and interface zone by

specifying the relevance center, smoothing, span angle center, face sizing, and edge sizing to receive hydrodynamic flow (Marsh et al., 2015). Edge sizing in all areas the turbine blades are the same at 0.0004 m. And then face sizing in the area of the turbine and the interior fluids are set at 0.05 m and 0.054, respectively.

The Inflation layers are utilized to control cell heights near all the turbine blades wall to resolve the boundary layer flow (Marsh et al., 2015), shown in Fig. 6. According to recent studies, the inflation layer set at 20 layers and the global growth rate set at 1.2 (Satrio et al., 2018).

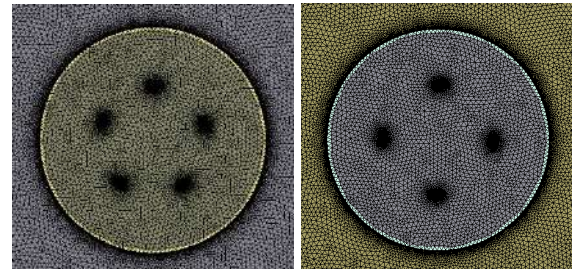


Figure 5: Refinement meshing of the variations turbine.

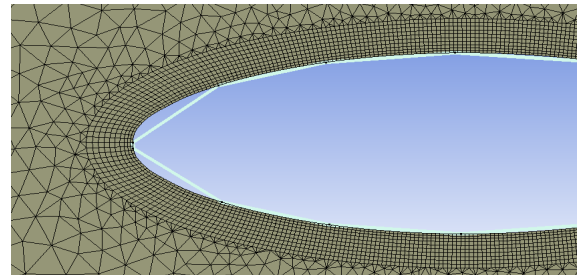


Figure 6: Inflation layers of the blade.

2.5 Solver Setup

In this study uses 6 core internal parallel processing for simulation the turbine blades with 2D CFD. The code is used to solved incompressible turbulence of Unsteady Reynolds Averaged Navier Stokes (U-RANS) equations. The $k-\omega$ SST turbulence model is utilized because it has a good accuracy model both free stream and boundary layer region (Marsh et al., 2016). Beside that often is used by researchers for vertical axis current turbines (Dai and Lam, 2009; Castelli et al., 2010; S Lain, 2010; Malipeddi and Chatterjee, 2012; Marsh et al, 2012, 2013 and 2014). The setting of the solution method at pressure velocity uses Semi-Implicit Method for Pressure Linked Equation (SIMPLE) for algorithm scheme and overall is set as second-order.

In this research simulation generally uses general transient type settings for an application to water fluid types. General transient type settings for an application to water fluid types. So that in the material setting choose the type of fluid that is water with a density of 998.2 kg/m^3 . Furthermore, at the cell zone condition step, to input the rotational speed at TSR 1.5, 2, and 2.5 are 4.92 rad/s, 6.56 rad/s, and 8.21 rad/s, respectively. The current speed data is set at the boundary condition-stage of 1.5 m/s.

After that, residual monitor for convergence is set at overall equation 10^{-4} and the maximum of iteration is 70. The input of calculation simulation is the number of time step (NTS) and time step size (TSS). In this study uses increment angle 0.9 degree and 6 rotation of turbine at all TSR. TSS represents the addition of angles each time the turbine rotates and NTS represents how many turbines rotated during the simulation (Satrio et al., 2018).

3 RESULTS AND DISCUSSION

3.1 Verification of 2D CFD Model

Verification this study of 2D CFD model simulation is carried out against was published data of the 3D CFD model by Marsh et al (2015). This study case compares the basic turbine 2D CFD with the 3D CFD model, shown in Fig. 7. The curve of C_p -TSR shows that the result of simulation 2D CFD has a similar trend with simulation 3D CFD. The result of C_p in this study shows that at TSR 1.5 represents low water velocity, different 16% with the 3D CFD model data. Differences in the value of C_p due to the use of different model dimensions and this study without uses of arm and shaft model design. Generally, at the 2D CFD model turbine by researchers without use arm and shaft.

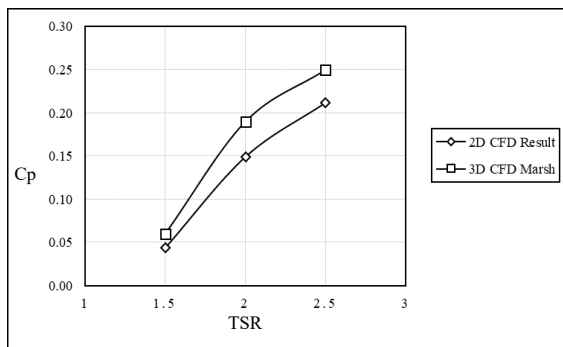


Figure 7: Verification 2D CFD result with the published 3D CFD data.

3.2 Effect of Blade Number on Torque

The first performance parameter in this study case is about the torque of the turbine. Output simulation with the 2D CFD model is data of torque coefficient (C_t) to represent the torque of the turbine. The curve of C_t -TSR in Fig. 8, shows the value of C_t and TSR at the basic turbine (blade 3) and the variations turbine (blades 4 and 5).

The value of C_t will determine the value of torque by using equation 2. The value of the torque at all the turbine blades is shown in Table 2.

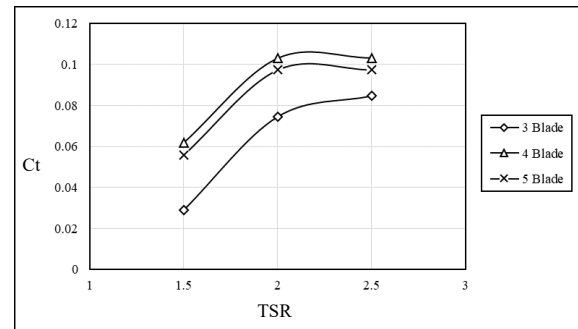


Figure 8: C_t -TSR curve at all the turbine blades.

Table 2: The value of torque (N.m).

TSR	T 3 blade	T 4 blade	T 5 blade
1.5	9.353	19.925	17.983
2	23.996	33.185	31.330
2.5	27.278	33.229	31.352

Based on Fig. 8 and Table 2 show that at all TSR, the blades 4 are higher torque than the blades 3 and 5. This study uses at TSR below 3 because it represents low water velocity. This study shows that at all low TSR ranges the blades 4 have the best performance with torque parameters. The torque of the blades 4 is increased by 21.8% from the blades 3, whereas the blades 5 is increased by 15% from the blades 3 or is decreased by 5.6% from the blades 4. So, this study shows that performance enhancement in the low water velocity is obtained by the turbine blades 4, with torque parameters.

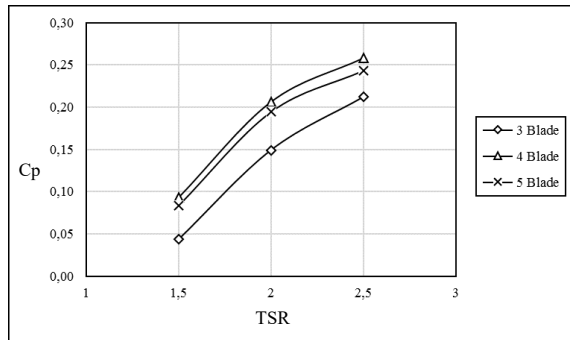
3.3 Effect of Blade Number on Solidity

The second performance parameter in this study case is about the solidity of the turbine. The value of turbine solidity is obtained by using equation 1. The more the number of blades, the greater the value of turbine solidity (σ) is shown in Table 3.

Table 3: The value of solidity.

The number of blades (N)	The value of solidity (σ)
3	0.07
4	0.09
5	0.11

Based on Table 3 show that the turbine blades 3, 4, and 5 have the value of solidity 0.07, 0.09, and 0.11, respectively. The effect of solidity on the turbine performance can be shown through a curve C_p -TSR, in Fig. 9. The power coefficient increase with the increase in turbine solidity from the turbine blades 3 ($\sigma = 0.07$). However, at the turbine solidity, 0.11 represents the turbine blades 5 decreases from blades 4. The authors predict that this is due to a loose lift force because of the turbine more solid. The highest C_p occurs at TSR 2.5 on blades 4 with a value of 0.26. However, the solidity 0.09 represents the turbine blades 4 improve performance to 21.8%. So, this study shows that performance enhancement in the low water velocity is obtained by the turbine blades 4, with solidity parameters ($\sigma = 0.09$).

Figure 9: C_p -TSR curve at all the turbine blades.

3.4 Effect of Blade Number on Power

The third performance parameter in this study case is about the power of the turbine. The value of power is determined by the rotational speed (ω) in rad/s using equation 4. The curve of P - ω in Figure 10 shows the effect of blade number on power (P) with the input of rotational speed namely, 4.92 rad/s, 6.56 rad/s, and 8.21 rad/s.

Based on Fig. 10 shows that the power of the turbine is influenced by rotational speed according to equation 4. The correlation between power output and rotational speed are comparable. The higher the rotational speed is the higher the power in watt at all blade numbers. The highest power occurs at TSR 2.5 on blades 4 with a value of 273 watts. The power of the blades 4 is increased by 21.8% from the blades

3, whereas the blades 5 is increased by 15% from the blades 3 or is decreased by 5.6% from the blades 4. However, the value of power at the turbine blades 4 to improve performance to 21.8% from blades 3. So, this study case shows that performance enhancement in the low water velocity is obtained by the turbine blades 4, with power parameters.

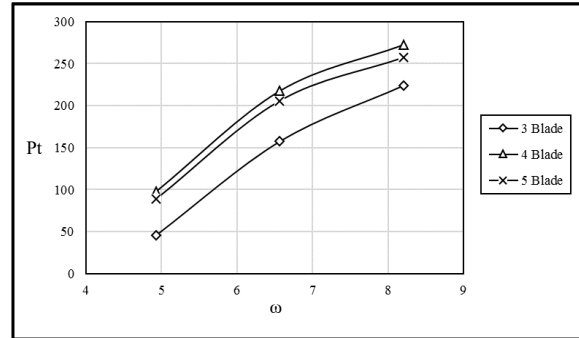


Figure 10: The results of the turbine power.

4 CONCLUSIONS

The simulation of variation of the number of blades 3, 4, and 5 successfully is done. The final results of this study case show that the number of blades can improve the performance of the turbine model on all low TSR ranges. The blades 4 and 5 are increased by 21.8% and 15% respectively from the blades 3 on the TSR 2.5. The highest performance is obtained by the number of blades 4 on TSR 2.5 with the value of C_p 0.26. So, this study case will choose the blades 4 for further research with the experimental method.

ACKNOWLEDGEMENTS

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Analysis of the Implementation of the Lamong Bay Reclamation based on the Implementation of UU No. 27 of 2007

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Keywords: Reclamation, Lamong Bay, UU No. 27 of 2007.

Abstract: Lamong Bay coast is a rich coastal area, this is because there are many fish habitats, marine biota habitats, mangroves, and other fisheries activities. But in 2010 there were reclamation activities by PT. Pelindo III which aims to increase the loading and unloading capacity of container terminals, waterfront city development, and multi-purpose terminals. By implementing UU No. 27 of 2007, the community service paper based on this research will explain the results of the study and evaluation in the implementation of the reclamation in Lamong Bay which will then be socialized to the community so that harmony is created between business permit holders and coastal communities of Lamong Bay.

1 INTRODUCTION

Lamong Bay is one of the strategic areas in Surabaya, East Java. In Lamong Bay, this also includes the Coastal area which has many fish habitats and marine biota. Around this area there are also many mangroves and fishermen. Through the Minister of Transportation's decision No. 4 of 1997 the Central Government issued a policy of granting permits to PT. Pelindo III to carry out beach reclamation or reclamation in the Tanjung Perak Port and Gresik Port working areas in the Lamong Bay area.

In Law No. 27 of 2007 Republic of Indonesia how to regulate activities carried out by people in order to increase the benefits of land resources in terms of environmental and socio-economic point of view by means of drainage, land drainage or drainage. Through PT. Pelindo III activities in the reclamation or reclamation of Teluk Lamong are carried out to increase the loading and unloading capacity and container terminals, waterfront city development, and building multi-purpose terminals. Of course, reclamation activities can be done easily. All aspects must meet the criteria established by the applicable laws and regulations. All types of risks and challenges must have been issued, both those that have been resolved technically and socially by the community.

Thus beyond the risks and challenges arising from the reclamation activities, it is necessary to conduct a study and evaluation in the implementation of the reclamation in the Lamong Bay area. Whether in the implementation of this reclamation activity is in line with the implementation of Law No. 27 of 2007. With the suitability of the procedures carried out starting from the planning process to completion, it is expected to be minimized as early as possible

- The activity of reclamation relates to the provincial Perda on Zonation based upon Act (UU) 26 Tahun 2007 and 27 Tahun 2007 junction UU 1 Tahun 2014
- Were there any Challenges and Obstacles in the Implementation toward Communities in the said areas.

2 THEORETICAL BASIC

2.1 Reclamation

Reclamation is an activity carried out by humans in the context of increasing the benefits of land resources in terms of environmental and socio-economic point of view by means of drainage, drainage or drainage. The reclamation limits set forth

* Author

in this report are reclamations carried out in coastal areas. The coastal area in question encompasses areas where ashore is still affected by marine processes and towards the sea as far as 12 miles. Activities that can be categorized as reclamation include all activities of adding dry land in coastal areas that result in changes in the shape of the morphology and coastal land use.

The reclamation system can be divided into four, namely the pile system, polder system, the combined system of pile and polder and drainage system. The explanation of each system is as follows:

1. Dumping System

Reclamation is done by hoarding coastal waters until the land surface is above the high sea level (high water level).

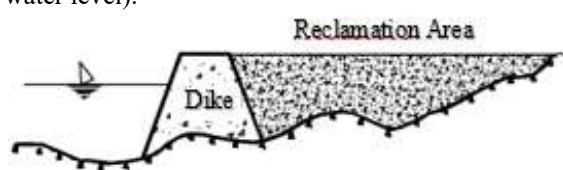


Figure 1: Reclamation of Dumping System.

The advantage of the embankment method is that the reclaimed land is always above sea level so that the facilities to be built on this land are relatively safe from excessive waves or standing water. While the weakness or disadvantage is the relatively large volume of landfill for land development.

2. Polder System

Reclamation is done by draining the waters to be reclaimed by pumping water that is in a watertight embankment to be discharged out of the reclaimed land area.

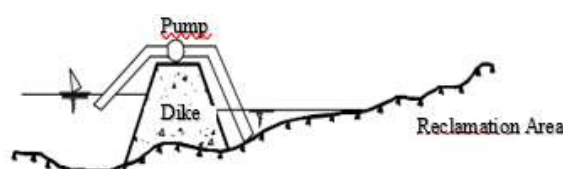


Figure 2: Reclamation of Polder System.

The advantage of reclamation with this system is the relatively small pile material requirements, but the loss is always threatened by standing water either from seawater or from rainwater. Thus the operational and maintenance costs for the drainage system require substantial funds.

3. Combination System between Polders and Piles

Reclamation of this system is a combination of the Polder System and the Dump System, that is after the land is obtained by the pumping method, then the land is buried to a certain height so that the difference in

elevation between the reclamation land and sea level is not large.

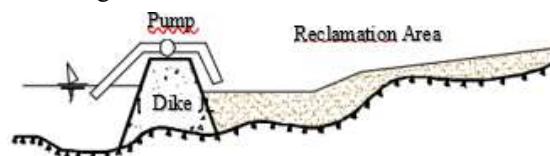


Figure 3: Reclamation of Combination System Polders and Piles.

4. Drainage System

Reclamation of this system is used for coastal areas that are flat and relatively low from the surrounding area but the land surface elevation is still higher than sea level elevation. This area can be a tidal swamp area or swamp area that is not affected by tides.

5. Aspects to be considered in reclamation

Reclamation, pre-reclamation, reclamation and post-reclamation activities must pay attention to several aspects, including physical aspects, ecological aspects, legal aspects, socio-economic aspects, and other supporting aspects.

2.2 UU Number 27 th 2007 concerning Management of Coastal Areas

Following the nature of the Unitary State of the Republic of Indonesia as a state of law, the development of a Management System for Coastal Areas and Small Islands as part of sustainable development with an environmental perspective must be given a clear, firm and comprehensive legal basis to ensure legal certainty. The legal basis was based on Pancasila and the 1945 Constitution of the Republic of Indonesia.

One example of exploitation that can have an impact on society is reclamation activities. In UU No. 27 of 2007, Reclamation of Coastal Areas and Small Islands is carried out to increase the benefits and/or value-added of Coastal Areas and Small Islands in terms of technical, environmental and socio-economic aspects. The implementation of the reclamation as intended is obliged to maintain and pay attention to:

- Sustainability of community life and livelihood.
- The balance between the interests of utilization and the importance of preserving the environmental functions of the Coast and Small Islands.
- Technical requirements for the collection, dredging and stockpiling of materials.

In the implementation and construction of a reclamation project, the scope of the Act consists mainly of five parts, namely planning, licensing, implementation, monitoring and evaluation, and transitional provisions.

2.2 Zoning Regulation

The zoning regulation is a tool for control that regulates the requirements for spatial use and the provisions for its control which are compiled for each block/designation zone (UU No. 26 of 2007 on Spatial Planning), where the block/zone designation as a reference is determined through a detailed plan spatial. This zoning regulation is better known as the popular zoning regulation, where the word zoning refers to the division of the city environment into zones of spatial use wherein each of these zones the control of spatial use is determined or different legal provisions apply (Barnet, 1982).

In the implementation of an area, development is often not in line with the spatial plan that has been prepared and makes both of them a conflicting product. The spatial plan that has been compiled will remain a document while the implementation of development continues to run based on market demand. control system is very necessary and must be prepared carefully in development planning. UU No. 26 of 2007 concerning Spatial Planning explained that the control is part of the spatial planning process that seeks to realize the spatial order. This activity is carried out to ensure that the process of spatial use is following the applicable spatial plan. The Reclamation implementation also should compliance to UU No 27 of 2007 juncto UU No 1 of 2014 in regards environmental and communities who live in the area.

In implementing the laws and regulations, the areas that are supposed to be used as development areas are misused by the community or local institutions. Therefore the zoning of the area that had been determined by the Government was reduced and finally, the addition of the Zoning Development Zone was determined.

Function and purpose of zoning control:

- Function to improve an activity that has lasted a long time but its existence is not following the existing spatial plan.
- Function to prevent development that is not following the references that have been prepared.

The two control functions are directed for two purposes, namely to direct and encourage development under existing legislation and vision and

mission rather than development itself. in connection with the implementation of spatial planning, functions and objectives of this control are carried out based on the spatial plan that has been prepared, where the spatial plan reflects the vision and mission of development to be achieved.

3 RESEARCH METHODOLOGY

- Site visit to conduct interviews with the community to find out whether there are environmental or social changes that occurred after the development of the Lamong Bay reclamation.
- Observation was carried out by visiting the Coastal Around Lamong Bay to collect data by giving interviews as well as questionnaires to the surrounding community as well as observing environmental conditions in the reclamation area.
- Observation data that have been obtained will be analyzed how reclamation involvement is carried out in the Lamong Bay area with the implementation of Law No. 27 of 2007.
- Presentation of the results of the analysis will be submitted to the surrounding community. This activity is carried out in the Village Hall or Pendopo District by providing information on the proper and correct reclamation procedures.

4 RESULTS OF COMMUNITY SERVICE

In 2010, Pelindo III built this terminal to improve port competitiveness. The construction of this project is one of the infrastructure developments included in the Master Plan for the Acceleration and Expansion of Indonesian Economic Development or commonly abbreviated as MP3EI because Tanjung Perak Harbor is currently overcapacity. So it takes another place as an new areas to solution. Furthermore, the Teluk Lamong Terminal is expected to be the gateway to the economy of the eastern Indonesia region and to become an alternative anchorage for logistic service users and businesses.

The construction of this terminal is planned with sufficient depth so that large ships can come to carry out loading and unloading activities. This terminal is specifically for dry bulk and container loads. But in practice, the operation of this terminal is also

earmarked by liquid bulk loads due to the lack of accommodation received.

The Teluk Lamong Terminal will go through several phases of development:

a. Phase I

It took place between 2010 - 2014. During this phase, the placement and construction of major infrastructure were carried out such as the jetty (domestic/international), stacking fields, and interchange land. Besides, procurement of loading and unloading equipment such as ship to shore (STS), ship unloader and automated stacking crane (ASC) is also carried out.

b. Phase II

This phase took place in 2014-2016. The development entered the process of increasing the number of loading and unloading equipment, expansion of the stockpile area, construction of a dry bulk dock area along with with a stacking field and the construction of a power plant area.

c. Phase III

Phase III will be carried out in 2016 – 2023. In this phase, additional distribution area construction and consolidation of container depots will be made using monorails. Besides, there are additional port facilities such as Container Crane, Ship Unloader, Automated Stacking Crane, etc.

d. Phase IV

Phase IV is planned to be carried out in 2023 – 2030. At this stage it is planned to expand the location of the pier and the stacking field and increase the port equipment facilities.

The following is the reclamation of the Lamong Bay reclamation layout (layout details are attached) and the condition of the Lamong Bay real estate when viewed from Google Earth.

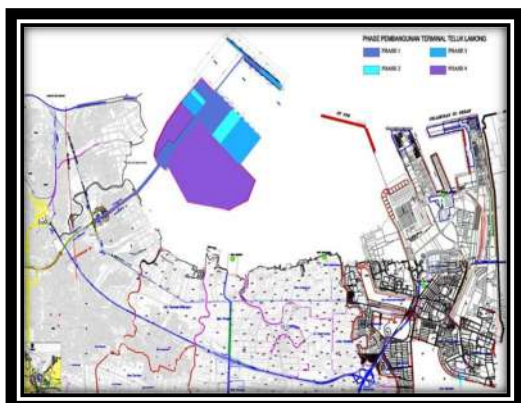


Figure 4: Stage Design Layout at Lamong Bay Terminal.



Figure 5: Real Condition at Lamong Bay Terminal.

The implementation of the Lamong Bay reclamation has been running for 9 years starting from 2010 which consists of several phases that have been described previously. At present the implementation of development has reached the 3rd stage and is planned to be finished until 2023. If it is seen from the use of the coastal area it seems that the reclamation phase that has been carried out from the first stage does not cause significant problems. The same thing applies in the second stage. This Han is the basis of that in Perda No. 1 Year 2018 Teluk Lamong Port is the Area of Utilization of Umun namely the Port. The following is the attachment to Perda No. 1 of 2018 which confirmed that the Lamong Bay was allowed as a port facility.

Table 1: Location of Lamong Bay Zoning Coordinates.

Location		Coordinate	
Districts	Object Name	Long (x)	Lat (y)
Kab. Gresik	Teluk Lamong Port	112.66938489900	-7.20450628100



Figure 6: Port Coordinates According to Perda Zoning No. 1 of 2018.

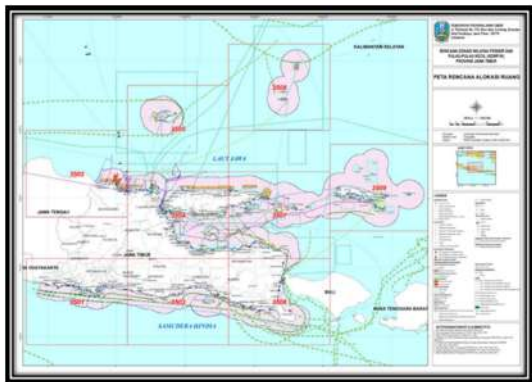


Figure 7: Zoning Plan for Coastal Areas and Small Islands RZWP-3K East Java Province 2018-2038.

Now, It can be understood that the implementation of the Lamong Bay reclamation in phase 1 (2010-2014) and phase 2 (2014-2016) has been completed, and phase 3 (2016-2023) which is still running until now can be categorized according to the zonation stipulated, namely Zone of Public Use Port Area. However, socialization in its implementation needs to be communicated to districts, sub-districts, and even the general public so that the meeting point and integration between agencies can be achieved

In carrying out the construction of Among Bay reclamation must be escorted both by the government or community intense. In this case, the party that has the authority in the implementation of the Lamong Bay Reclamation is the Provincial Government. The authority of the Lamong Bay reclamation development is not the property of the Government of the city of Surabaya or the Government of the city of Gresik as a city directly affected by the reclamation activity. But in Pergub No. 1 of 2018 regulates that coastal areas are assets of the province. (Note: UU No. 23 of 2014 concerning Regional Government). So that licensing to revocation of licenses is also the authority of the provincial government with various considerations from relevant agencies, namely:

- Provincial Government
- City / Regency Government
- BAPPEDA
- Transportation Agency
- Maritime and Fisheries Service
- Environmental Agency, etc.

The thing to know is that there are people who feel less fortunate in the implementation of the Lamong Bay Reclamation. Indeed reclamation can momentarily have an impact on the livelihoods of surrounding communities but that does not mean eliminating their livelihoods.

The main purpose of this project development is that in the future it will greatly affect the community, especially the nation's next generation. This development can help accommodation and distribution of goods to facilitate economic flows that occur. But what needs to be underlined is that the management of this activity must be right on target so that the main targets and objectives can be achieved as well as communication between related agencies.

In practice, the application of UU No. 27 of 2007 in conjunction with UU No. 1 of 2014 is the implementation of the integrated coastal area management concept. ICZM (Integrated Coastal Zone Management), which provides formulations/suggestions in management procedures that contain integration between the center and the regions, between provinces and districts and then between agencies. By prioritizing sitting together and maintaining communication between related agencies, it is hoped that harmony between agencies is maintained.

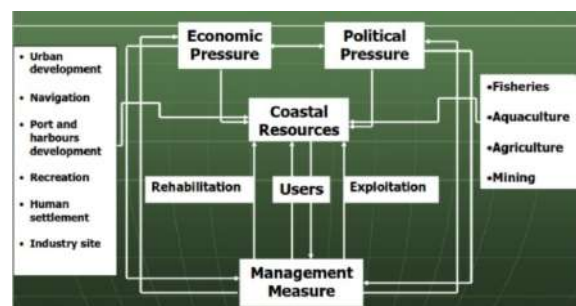


Figure 8: Integration and Integrated Management of Coastal and Marine Region.



Figure 9: Cohesiveness of ICZM (Integrated Coastal Zone Management).

For fishermen both fish and salt, the municipal government and Maritime and Fisheries Service, also continue to try to become facilitators in the sustainability of the communities in the coastal areas. Workable solutions include:

- Transfer of the concept of community livelihood in some areas towards coastal ecotourism.
- Transfer of fisheries and fishponds that have been displaced by reclamation activities.
- Absorption of community labor directly affected by the reclamation of Lamong Bay.

5 CONCLUSIONS

The conclusion of this community service are:

- Teluk Lamong has been carried out in 4 phases starting from 2010 and planned to be completed until 2030 under PELINDO III's needs as a means of distribution of containers and dry bulk loads. Implementation of UU No. 27 of 2007 is to issue zoning plans for coastal areas and small islands as outlined in UU No. 26 of 2007 through the PERDA of East Java Province No. 1 of 2018. the implementation of lamong bay reclamation is following the specified zoning, namely the Port General Use Area. However, socialization in the application needs to be communicated.
- In practice, the application of UU No. 27 of 2007 in conjunction with UU No. 1 of 2014 is the implementation of the integrated coastal area management concept. ICZM (Integrated Coastal Zone Management) provides formulations/suggestions in management procedures that contain integration between the center and the regions, between provinces and districts and then between agencies. by prioritizing sitting together and maintaining communication between relevant agencies. thus, harmony between agencies is expected to be maintained.
- It is hoped that communication between relevant stakeholders can make development sustainable. The main objective of the development of this project is that in the future it will have a big impact on the community, especially the nation's next generation. This project can help the accommodation and distribution of goods to facilitate economic flows that occur. But what needs to be underlined is that the management of these activities must be right on target so that the main targets and objectives can be achieved. The local government and related agencies will also continue to develop the potential that exists in the area around the Lamong Bay reclamation.

6 SUGGESTIONS

In the reclamation procedure which began in 2010 and the plan until 2030, it turns out that there are some residents who need assistance, one of which is the placement of workers, this can be overcome through increased communication between agencies so that it can provide positive results. Further studies are expected to be able to determine the potential of coastal areas in the reclamation area so as to overcome these problems.

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Study on the Influence of Tool Rotating Speed on the Weld Joint Strength of Friction Stir Welding Method

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Keywords: Aluminum 6061, Friction Stir Welding, RPM Speed, Tensile, Impact, Micrographic Test.

Abstract: Aluminum 6061 is a lightweight metal and has a corrosion resistance and good conductivity. In the marine industry, aluminum 6061 is widely used for ship construction. Friction Stir Welding (FSW) uses the principle of utilizing friction from a rotating work piece with another stationary work piece so that it is able to melt the stationary work piece and finally connect it together. The focused of this study is to determine the impact strength, tensile strength and micrographic structure of the butt joint of FSW Welding. The feed rate is determined as 10 mm / min with the variations of rpm speed is defined as 1640 rpm, 2620 rpm, and 3820 rpm. Tensile strength, impact strength and micrographic evaluation will be discussed to assess the effect of rotational speed on the strength of the welded joint produced by Friction Stir Welding.

1 INTRODUCTION

In the era of modern technology, humans are required to create an effective and efficient progress and development that can give benefits to the society in all of fields. One of the fields is the field of marine and shipbuilding industry. Technological developments in the shipbuilding industry are the use of various types of materials such as steel, aluminum, fiberglass, etc. All of the types of materials have advantages and disadvantages. One type of material that has good strength against corrosion is aluminum.

Aluminum is a metal that has relatively low strength and soft. Aluminum is a lightweight metal and has good corrosion resistance, good electrical conductivity and other properties. Generally aluminum is mixed with other metals to form aluminum alloys. Aluminum 6061 is a kind of aluminum alloy between magnesium and silicon that has good mechanical properties without reducing electrical conductivity. In the marine industry, aluminum is widely used for construction in piping and tank sections such as fresh water tanks or fuel tanks. Aluminum is a metal that has mechanical properties that are resistant to corrosion and relatively good electrical conductivity. This metal is widely used not only for household appliances, but

also for aircraft, automotive, marine and building construction materials, (Surdia et. al., 1999).

FSW (Friction Stir Welding) is a welding method that was discovered and developed by Wayne Thomas for aluminum and aluminum alloy work pieces in 1991 at TWI (The Welding Institute) in the United States (Nandan et. al., 2009). The working principle of FSW is to utilize the friction of a rotating work piece with another stationary work piece so that it is able to melt the stationary work piece and finally connect it together. The welding process with FSW occurs in solid conditions (Solid State Joining). The welding process with FSW occurs at the temperature of the solvus, so there is no decrease in strength due to over aging and the dissolution of coherent deposits. Since the welding temperature is not too high, the residual stresses and the distortion that are formed due to heat are also low (ASM, 2007). The mechanical characteristics of the weld joint in the FSW are determined by the parameters: welding speed, tool rotation, and tool pressure (Jayamaran et. al., 2009).

The application of FSW technique to support the manufacture of marine structures and shipbuilding also can be found in (Maggiolino, 2008; Feistauer, 2014; Farajkhah, 2016; Singh, 2019; Ramesh, 2020).

Previous studies have conducted a study on the effect of rotating tools on micro structures and mechanical properties of friction stir welding

connections on aluminum alloy 6061 (Wartono et. al., 2015) where in that study using tools rotation speed: 540 rpm, 910 rpm, and 1500 rpm. Tensile test results showed that the average ultimate strength for welding using 540 rpm tool speed was 139 MPa, for 910 rpm tool speed was 157 MPa and at 1500 rpm tool speed was 155 MPa. With these results it can be seen that the highest ultimate strength is to use the 910 rpm tool rotation. Furthermore, in case of the impact strength behaviour, the weld joint impact strength at 540 rpm has magnitude of 0.594 (J/mm²), for a speed of 910 rpm it has an impact strength of 0.624 (J/mm²), and at a speed of 1500 rpm has an impact strength of 0.573 (J/mm²).

In this study a similar study was made, however the rotational speed was significantly increased by: 1640 rpm, 2620 rpm, and 3820 rpm. The experimental studies was conducted to assess the strength of the connection consist of tensile test, impact test and micrographic test.

2 PREPARATION AND TESTING METHOD OF THE WELD JOINT

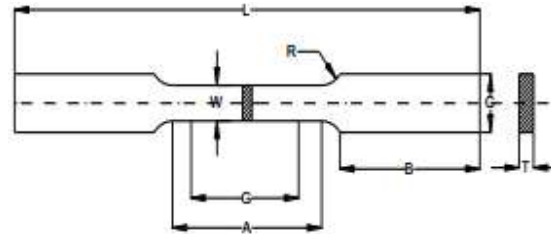
The Aluminum 6061 was produced by the local steel industry that was used in the ship production process with the thickness 10 mm. The material properties of the Aluminum 6061 can be seen in the Table 1.

Table 1: Mechanical properties of Aluminum 6061.

Aluminum 6061	
Poisson's Ratio	0,33
Modulus Of Elasticity	68,9 GPa
Density	2700 kg/m ³
Yield stress	276 MPa
Fatigue Strength	633 MPa
Failure Strain	0,39
Tensile Strength	324 MPa

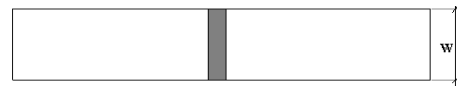
The specimens are made for the purposes of tensile testing, impact testing and micrographic testing. In making specimens for test experiments not used the standard ASTM E8 / E8M-09. Illustration of dimensions and shape of tensile test specimens can be seen in Fig. 1. In impact test experiments, specimen preparation is carried out using the ASTM E23 standard. Illustration of dimensions and shape of the impact test specimens can be seen in Fig 2. The existing welded joints on the specimens were made using the friction stir

welding method with the rotating pin tool with variations in the pin tool speed of 1640 rpm, 2620 rpm, and 3820 rpm. Illustration of pin tool dimensions can be seen in Fig. 3. The numbers of specimens in the tensile test and impact test are 3 specimens, while for the micrographic test are 1 specimen.



Gage length (G)	: 50.0 mm
Length of reduced section (A)	: 57 mm
Width (W)	: 12.5 mm
Thickness (T)	: 10 mm
Radius of fillet (R)	: 12.5 mm
Overall length (L)	: 200 mm
Width of grip section (C)	: 20 mm
Length of grip section (B)	: 50 mm

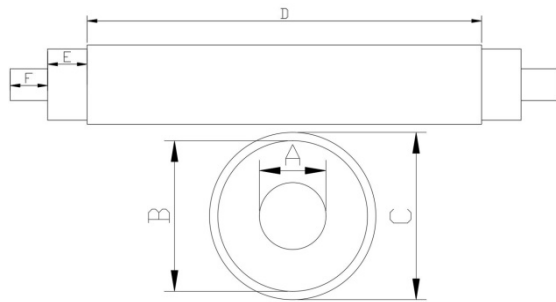
Figure 1: Tensile test specimen (ASTM E8/E8M-09).



Overall length (L)	: 55 mm
Width (W)	: 10 mm
Thickness (T)	: 10 mm
Notched Charpy	: 45°

Figure 2: Impact test specimen (ASTM E23).

The weld joint specimens were prepared by friction stir welding method. The universal milling machine was used to create the weld joint. Rotational speed of universal milling machine can be arranged with the maximum rotational speed of 4000 rpm. In this study the rotational speed was determined on 1640 rpm, 2620 rpm, and 3820 rpm. The universal milling machine has been equipped with an automatic moving table. Therefore the friction stir welding process can be accomplished with the steady travel speed. The travel speed of the rotating tool of the welding process was determined as 30 mm/min. The rotating pin tool has the main function to generate the heat and to mix the work piece material. Therefore the material of the rotating tool should have a higher melting point compare with the weld material. The K-110 KNL EXTRA steel was selected as the material of the rotating tool.



Material of Pin : KNL 110 Extra
Hardness : 61 HRC
Diameter of Pin (A) : 8 mm
Diameter of Probe (B) : 18 mm
Diameter of Tools (C) : 20 mm
Length of Tools (D) : 70 mm
Length of Probe (E) : 10 mm
Length of Pin (F) : 9.5 mm

Figure 3: Illustration of the rotating pin tool design.

3 RESULTS AND DISCUSSIONS

3.1 Tensile Strength Test Results

From the tests that have been done, the results obtained in Table 2, tensile testing show that the value of the maximum tensile strength resulting from the FSW (Friction Stir Welding) welding carried out in this research is 88.13 MPa at a speed of 1640 rpm (specimen 2), and the result of the smallest tensile strength is 16.80 MPa at a speed of 2620 rpm (specimen 3).

Table 2: Tensile stress of Aluminum 6061.

Specimen	Thick. (mm)	Width (mm)	Area (mm ²)	P Max (N)	σ Max (MPa)	σ aver. (MPa)
X1	10.09	12.76	128.75	10630	82.56	83.34
X2	10.08	12.72	128.22	11300	88.13	
X3	10.10	12.93	130.59	10360	79.33	
Y1	10.55	12.57	132.61	2750	20.74	19.27
Y2	10.58	12.30	130.13	2640	20.29	
Y3	10.35	12.94	133.93	2250	16.80	
Z1	10.26	12.82	131.53	7440	56.56	45.02
Z2	10.36	12.97	134.37	4970	36.99	
Z3	10.38	13.02	135.15	5610	41.51	

The average magnitude of the tensile strength of the aluminum 6061 weld joint using FSW (Friction Stir Welding) welding with variations in the tool rotation speed carried out in this research was 83.34 MPa at 1640 rpm, 45.02 MPa at 3820 rpm, and 19.27 MPa at 2620 rpm. Before the tensile test is carried out, a visual inspection of the welding surface is carried out first, and the speed of 2620

rpm has the worst weld results. There are worm holes after welding is the main reason that reduces the tensile strength test results on the specimen with a speed of 2620 rpm.

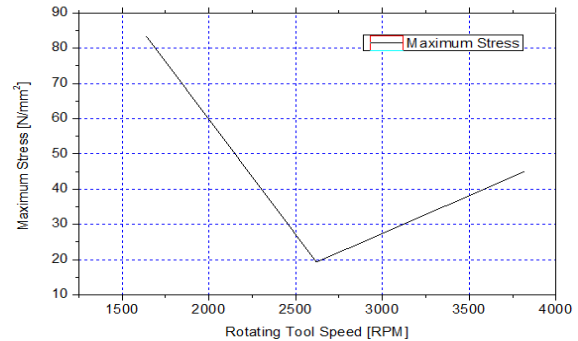


Figure 4: Tensile stress of Aluminum 6061 at the defined rotating tool speed.

Table 3: Tensile strain of Aluminum 6061.

Specimen	L0 (mm)	L1 (mm)	ΔL (mm)	Strain ϵ (%)	ϵ aver. (MPa)
X1	50	52.11	2.11	4.21	5.93
X2	50	53.86	3.86	7.71	
X3	50	52.93	2.93	5.87	
Y1	50	51.92	1.92	3.84	3.41
Y2	50	51.66	1.66	3.31	
Y3	50	51.53	1.53	3.07	
Z1	50	52.03	2.03	4.06	3.65
Z2	50	51.92	1.92	3.83	
Z3	50	51.53	1.53	3.06	

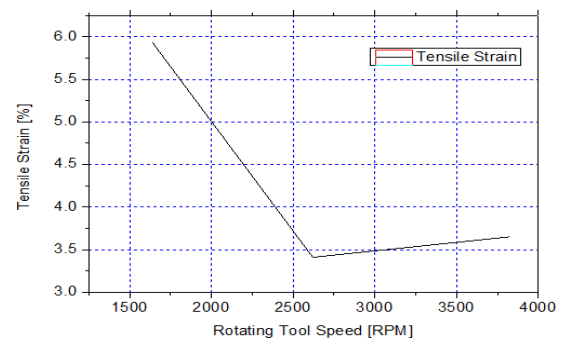


Figure 5: Tensile strain of Aluminum 6061 at the defined rotating tool speed.

Based on the conducted test results of tests in this study, the maximum tensile strain value resulting from friction stir welding joints which was occurred on aluminum 6061 is 7.71% at a speed of 1640 rpm specimen 2, and the smallest tensile strain yield is 3.07% at a speed of 2620 rpm specimen 3.

The average tensile strain value from highest to lowest resulting from the aluminum 6061 weld joints using FSW (Friction Stir Welding) welding with

variations in the tool rotation speed carried out in this research was 5.93% at 1640 rpm, 3.65% at 3820 rpm, and 3.41% at 2620 rpm.

Table 4: Modulus of elasticity of Aluminum 6061.

Specimen	σ Max (MPa)	Strain ϵ (%)	E (GPa)	E aver. (GPa)
X1	82.56	4.21	19.61	14.85
X2	88.13	7.71	11.42	
X3	79.33	5.87	13.52	
Y1	20.74	3.84	5.40	5.67
Y2	20.29	3.31	6.13	
Y3	16.80	3.07	5.47	
Z1	56.56	4.06	13.93	12.38
Z2	36.99	3.83	9.65	
Z3	41.51	3.06	13.57	

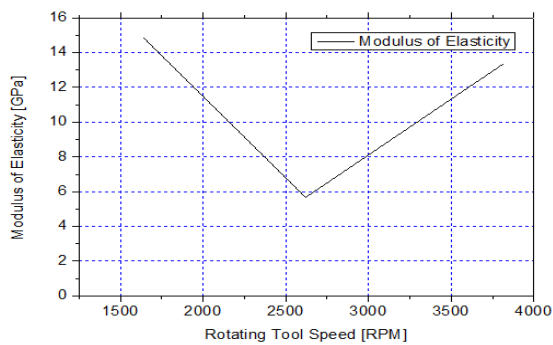


Figure 6: Modulus of elasticity of Aluminum 6061 at the defined rotating tool speed.

From the diagram of the test results that have been carried out, the highest elastic modulus value at the aluminium 6061 weld joint using FSW (Friction Stir Welding) welding is 19.61 GPa at a speed of 1640 rpm specimen 1. While the lowest elastic modulus value of the specimen of aluminum 6061 weld joint using FSW (Friction Stir Welding) at a speed of 2620 rpm specimen 1 has shown a modulus of elasticity of 5.4 GPa. From the highest to lowest, the average modulus of elasticity of the aluminum 6061 welding connection using FSW (Friction Stir Welding) welding is 14.85 GPa at 1640 rpm; 13.81 GPa at a speed of 3820 rpm; and 5.67 GPa at a speed of 2620 rpm.

3.2 Impact Strength Test Results

From the diagram of the results of the tests that have been done, the impact strength on the aluminum 6061 weld joint using FSW (Friction Stir Welding) welding is 0.27 J/mm² at 3820 rpm of specimen 1. While the lowest impact strength on the aluminum 6061 weld joint using FSW welding (Friction Stir Welding) is produced at a rotating speed of 2620

rpm specimen 2 with an elastic modulus value of 0.10 J/mm². The highest to lowest average impact value generated at the aluminum 6061 weld joint using FSW (Friction Stir Welding) welding is 0.23 J/mm² at 1640 rpm; 0.20 J/mm² at a rotating speed of 3820 rpm; and 0.13 J/mm² at a rotating speed of 2620 rpm.

Table 5: Impact strength of Aluminum 6061.

Specimen	Width (mm)	Height (mm)	Absorbed Energy (J)	Impact Strength (J/mm ²)	Impact Strgh. aver. (J/mm ²)
X1	10.16	8.88	22.18	0.25	0.23
X2	10.28	9.00	17.08	0.18	
X3	10.20	8.88	23.95	0.26	
Y1	10.21	9.29	10.78	0.11	0.13
Y2	10.16	8.77	8.65	0.10	
Y3	10.16	8.66	15.45	0.18	
Z1	10.23	9.24	25.75	0.27	0.20
Z2	10.26	9.31	16.18	0.17	
Z3	10.32	8.67	14.45	0.16	

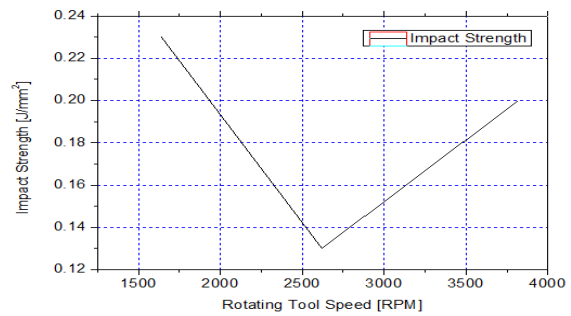


Figure 7: Impact strength of Aluminum 6061 at the defined rotating tool speed.

3.3 Micrographics Test Results

The specimens used for micrographic testing are impact specimens with a size of 55 mm × 10 mm × 10 mm as many as 3 specimens of FSW weld joint. The shape of the microstructure at the aluminum 6061 weld joint can be seen through the surface treatment which is carried out on the test specimen so that the micro structure of the aluminum weld joint can be clearly observed using a microscope. Firstly, the specimens are smoothly grinded using 4 sequences of sandpaper layers, namely sandpaper numbers 100, 200, 400, 600 and 1000. After being smooth, then the specimen is smeared with the autosol until glossy. After that, the surface of the specimen that has been smeared with autosol is then given a 50% NaOH etching solution so that when viewed using a microscope, the microstructure of the aluminium 6061 welded joint can be clearly seen. Furthermore, micrographic testing is performed to

see the microstructure shape of the aluminum 6061 weld joint using a microscope with a magnification of 200 times.

From micrographic testing on aluminum 6061 welding (Base Metal, HAZ, and Weld Joint) using FSW (Friction Stir Welding) welding, the following results were obtained, see Fig. 8 – Fig. 10:



[a] Weld Joint Region



[b] HAZ Region



[c] Base Metal Region

Figure 8: Microstructure of the aluminium 6061 weld joint at 1640 RPM.

From the micrographics test, the results of the aluminium 6061 weld joint using FSW (Friction Stir Welding) welding with variations in the rotational speed of the tool was presented in the form of particle size and shape at the regions of the HAZ (Heat Affected Zone), the base metal and the weld joint. In the base metal area has a small grain size, because the base metal region is not affected by the effects of heat due to welding. Otherwise, in the



[a] Weld Joint Region



[b] HAZ Region



[c] Base Metal Region

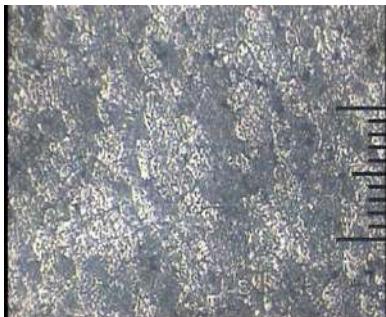
Figure 9: Microstructure of the aluminium 6061 weld joint at 2620 RPM.

HAZ region, particle granules change in the shape and grain size larger and coarser, and the density of particles is more tenuous when compared to the base metal region. This is because the HAZ is only affected by heat caused by friction when welding process occurred.

In the area of the steering zone changes in the shape and particle size looks rough and larger than the particle size in the HAZ and base metal. This is caused by the temperature and rotational movement of the tool when welding around the stir zone. In Fig. 8[a], Fig. 9[a] and Fig. 10[a], the weld joint region (stir zone) will have soft granules which are caused by recrystallization.



[a] Weld Joint Region



[b] HAZ Region



[c] Base Metal Region

Figure 10: Microstructure of the aluminium 6061 weld joint at 3820 RPM.

Based on the results of micrographic testing, the higher of the rotation speed, the grain size of the particles produced is greater, it is caused by several factors such as heat input, welding rate and magnitude of cooling rate. Therefore the weld joint at a speed of 1640 rpm has a good density level compared to the weld joint that is produced on the speed of 2620, and 3820 rpm. It can be seen on the results of the micrographics photo in the HAZ region.

4 CONCLUSIONS

The tensile test results of the weld joint with FSW (Friction Stir Welding) welding have an average stress of 83.34 MPa and an average strain of 5.93%

at a rotational speed of 1640 rpm. At a rotation speed of 2620 rpm it has an average stress of 19.27 MPa and an average strain of 3.41%. An average stress of 45.02 MPa and an average strain of 3.65% are held at the 3820 rpm. Prior to the tensile test, visual inspection of the welding surface should be carried out, and the speed of 2620 rpm has the worst weld results. There are hot cracks and worm holes after welding is the main reason that reduces the tensile strength test results on the specimen with a speed of 2620 rpm.

The impact test results on the results of welding joints with FSW welding have the average impact strength of 0.23 J at the rotating speed of 1640 rpm. The average impact of 0.13 J was appeared at 2620 rpm rotation speed, and at the rotating speed of 3820 rpm of the tool has the average impact strength of 0.20 J.

Micrographic structure test results which is obtained in the HAZ region, and weld joint (nugget zone) at each tool rotation has a grain size larger than the base metal, it is caused by the influence of the tool rotation speed. The larger of the tool rotation, the greater the grain size produced in the welding results.

ACKNOWLEDGEMENTS

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Size Optimization of Foil-shaped Center Bulb on Catamaran Hull Form to Reduce Resistance

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Keywords: Catamaran, Foil-shaped Center Bulb, Resistance.

Abstract: The interference of resistance between the two catamaran hulls is a very popular topic to discuss. Errors in designing hull form and determining the distance between the hull will result in a large amount of interference that causes the resistance of the catamaran increased. Some researchers try to reduce the resistance of catamaran due to interference such as changing the shape of the demihull, giving a bulbous bow, and adding a center bulb. The idea to use center bulb on catamarans is still relatively new and still requires a lot of studies. Therefore, in this study the elliptical center bulb will be replaced with a foil-shaped center bulb. The purpose of this study was to obtain the optimal length (X_1) and diameter (X_2) foil-shaped centrebulb on the NPL hull catamaran scale model with Fr 0.7 using Response Surface Method (RSM). The optimal size of foil-shaped center bulb is determined by the size that the most makes of ship wave resistance (R_w) to a minimum. The results of the regression equation in Order 1 are $R_w = -0.168X_1 + 0.378X_2 + 14.862$ and the results of the regression equation in Order 2 are $R_w = -0.020239X_1 + 0.061318X_2 + 0.201557(X_1)^2 + 0.261325(X_2)^2 + 0.015(X_1X_2) + 13.34806$. Optimal center bulb size is obtained at a length of 103.78 mm and a diameter of 26.30 mm with a wave resistance value of 13.23 N. The center bulb size can reduce wave resistance by 11.74% from the initial center bulb model and reduce 4.72% compared to not using a center bulb.

1 INTRODUCTION

Catamarans are double hull ships, which have many advantages over single-hull boats with the same displacement. Among these advantages is the catamaran has a broad deck so that designers are more flexible in arranging the ship's accommodation space. Besides, catamarans have excellent ship stability so that catamarans are safer and more comfortable for passengers (Seif & Amini, 2004; Zouridakis, 2005).

Because catamarans have two hulls, interference between the two hulls is a prevalent topic to be discussed on catamarans. Errors in designing the hull shape and determining the distance between the hulls will result in the amount of interference of the ship's resistance, which makes the resistance of the catamaran ship to increase by almost four times even though the displacement has increased two times compared to each demihull. (Samuel, et al., 2015).

Some researchers researched to reduce the resistance of catamarans due to interference. Iqbal and Samuel have researched to reduce the resistance of catamaran fishing vessels by modifying the shape

of the ship's demihull hull using the Luckenby method. The method changes the CSA form of the boat to create a new hull. This method succeeded in reducing the total resistance by 6.5% (Iqbal & Samuel, 2017). The method has been successfully used by Iqbal and Rindo to improve the quality of the seakeeping of catamarans (Iqbal & Rindo, 2015).

Samuel et al. have also used the bulbous bow on catamaran fishing vessels to reduce ship resistance. Ship resistance can either be reduced or increase depending on the type of bulbous bow used. In that case, the kind used to overcome resistance is the nabla type, where the resistance can reduce by 10% (Samuel, et al., 2018).

Other research conducted to reduce the resistance of catamaran vessels has been carried out using the concept of the center bulb, which is bulbous that is between the hulls of the catamaran (Saha, et al., 2005; Danisman, 2014). The center bulb is bulbous between the two hulls of the catamaran (in the middle). The purpose of this center bulb installation is to provide wave interference between two catamaran hulls. When the waves were interfered, it expects that the

waves will break and reduce the wave resistance. In addition to reducing ship resistance, the utilization of center bulb can also improve the quality of seakeeping of catamaran vessels (Bruzzzone, et al., 2008; Zotti, 2007; Aprjal, et al., 2018).

In Saha's research, the center bulb that used is large and placed in the front and rear positions, as in Figure 1. The results of the study provide recommendations that the wave resistance on the catamaran ship with the center bulb depends on size and position of center bulb against catamaran hulls and the interaction of the catamaran wave resistance (Saha, et al., 2005).

Zotti gives fins at center bulb to examine its effect on resistance and the motion of the ship (see Figure 2). The results of the study are center bulb with fin reducing ship resistance at $Fr > 0.3$ and increasing resistance at $Fr < 0.3$. RAO curve peaks from heaving and pitching center bulb with fins are lower than catamarans without center bulb. This method indicates center bulb with fins has a positive impact on seakeeping ships (Zotti, 2007).

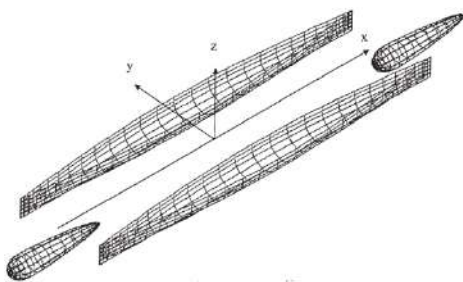


Figure 1: Concept of Center bulb on a catamaran (Saha, et al., 2005).

Danisman optimizes the position and geometry of the elliptical center bulb using Artificial Neural Networks (ANN). The research has succeeded in reducing wave resistance by 15% based on the results of numerical calculations and 13% based on the results of experimental tests in towing tanks (Danisman, 2014). The elliptical bulb center from Danisman has also been applied to catamaran fishing vessels by Samuel and can reduce resistance by 25.76% (Samuel, et al., 2018).

The idea to use center bulb on catamarans is still relatively new and still requires a lot of studies. Therefore, in this study, the elliptical center bulb form was replaced with a foil form. This form is expected to reduce ship resistance further because the tapered back shape of the foil will make the flow pattern streamlined and will not cause vortex shedding or repetition as when flow passes through a circular or ellipsoid shape.

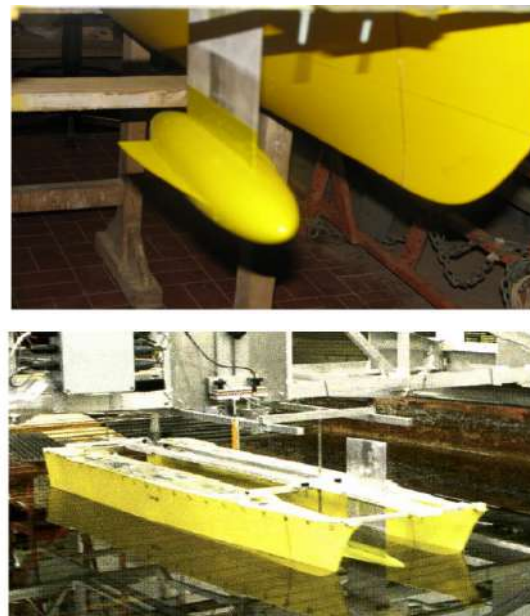


Figure 2: Concept of Center bulb on a catamaran (Zotti, 2007).

To producing maximum performance from the center bulb, the optimization method is used to get the optimal center bulb length and diameter. In this study, the optimization method used is Response Surface Methods (RSM). RSM is one of the most practical and economic optimization techniques that is widely used to evaluate variables from experiments that produce several responses (Bezerra, et al., 2008).

The advantage of this method is that Design of Experiments (DoE) based on RSM does not require a lot of trials/testing and does not require a lot of time compared to actual experimental studies (Ma, et al., 2015).

Some researchers have used this method to optimize several research variables. In the field of structure, Baroutaji optimizes the thickness, diameter, and width of the hollow cylindrical tube to produce the maximum capacity of Specific Energy-Absorbing (SEA) and minimize the Collapse Load (F) (Baroutaji, et al., 2015).

In the field of mechanical engineering, Awad et al., Used the RSM method to maximize brake power and thermal brake efficiency and minimize Brake Specific Fuel Consumption (BSFC) and emissions from NO_x, HC, and CO by optimizing three variables, which are fuel, engine speed, and throttle valve (Awad, et al., 2017).

In the field of naval architecture, RSM is very rarely used. Therefore, the research tries to apply the method in the field of naval architecture to minimize

the resistance of catamarans by optimizing variable length and diameter of the center bulb.

The purpose of this study was to obtain the optimal length ($X1$) and diameter ($X2$) foil-shaped centrebulb on the NPL hull catamaran scale model with Fr 0.7 using Response Surface Methods (RSM). The optimal size of foil-shaped center bulb is determined by the size that the most makes of ship wave resistance (R_w) to a minimum.

2 METHOD

In this study, the catamaran hull model used is the standard 4a hull model of NPL (Bailey, 1976). The principal dimension and experimental test results in towing tanks both as demihull and as catamarans are found in (Molland, et al., 2017). The shape of the body plan and 3D are shown in Figure 3. The principal dimension of the NPL ship model, when tested in towing tanks, are in Table 1. The results of the residual resistance in the experimental tests are shown in Table 2.

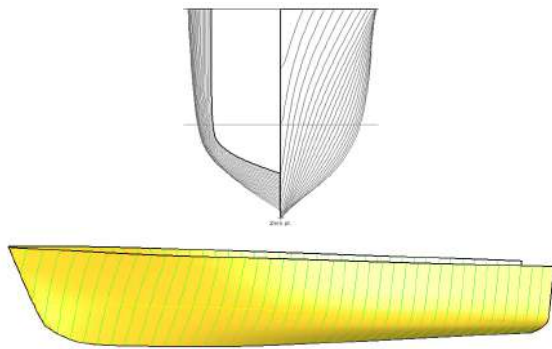


Figure 3. Body plan and 3D model of NPL hull.

Table 1: Principal Dimension of the NPL Hull Model.

Dimension	Scale Model
Lwl	1.60 m
B demihull	0.15 m
T	0.10 m
Cb	0.395
WSA	0.346 m ²
Displacement	0.0102 ton
S/L	0.3

Table 2: Residual Resistance of NPL Hull.

Fr	Mode 4a residuary resistance ($C_T - C_{FITC}$)				
	Monohull	$S/L = 0.2$	$S/L = 0.3$	$S/L = 0.4$	$S/L = 0.5$
0.200	1.909	2.327	2.564	2.495	2.719
0.250	2.465	3.148	3.315	2.937	3.484
0.300	3.273	3.954	4.283	4.396	3.875
0.350	3.585	5.073	4.576	4.064	4.173
0.400	4.100	4.874	5.871	5.900	5.109
0.450	5.305	8.111	7.953	7.220	6.299
0.500	5.526	8.365	7.150	6.650	6.140
0.550	5.086	7.138	5.990	5.692	5.615
0.600	4.431	5.878	5.090	4.880	4.981
0.650	3.924	4.815	4.392	4.269	4.387
0.700	3.477	4.047	3.949	3.834	3.911
0.750	3.128	3.556	3.594	3.512	3.570
0.800	2.904	3.224	3.187	3.252	3.296
0.850	2.706	2.923	2.966	3.054	3.070
0.900	2.544	2.729	2.839	2.881	2.873
0.950	2.398	2.550	2.657	2.767	2.707
1.000	2.272	2.433	2.437	2.687	2.558

In this study, the optimal size of elliptical center bulb geometry in Danisman was adopted to determining the initial size of the center bulb by comparing its geometry to the ship's geometry (Danisman, 2014). Table 3 shows the principal dimensions of the Danisman ship model. The size and calculation of the center bulb dimensions are listed in Table 4. The comparison is used to determine the geometry of the center of the foil-shaped bulb as shown in Figure 4.

Table 3: Principal Dimension of Danisman's Catamaran (Danisman, 2014).

Lwl	2,525 m
B	0,26 m
T	0,14 m
Cb	0,41

Table 4: Comparison of Center bulb Dimension.

Elipsoidal Center bulb (Danisman, 2014)	Ratio	Foil-Shaped Center bulb
Length (a) = 0,141 m	$0,141/2,525 = 0,0558$	Length (a) = 0,089 m ~ 0,090 m
Width (b) = 0,062 m	$0,062/0,26 = 0,2384$	Width (b) = 0,037 m;
Height (c) = 0,034 m	$0,034/0,14 = 0,2428$	Height (c) = 0,024 m Diameter (d) = $(b+c)/2 = 0,0305 \sim 0,040$ m

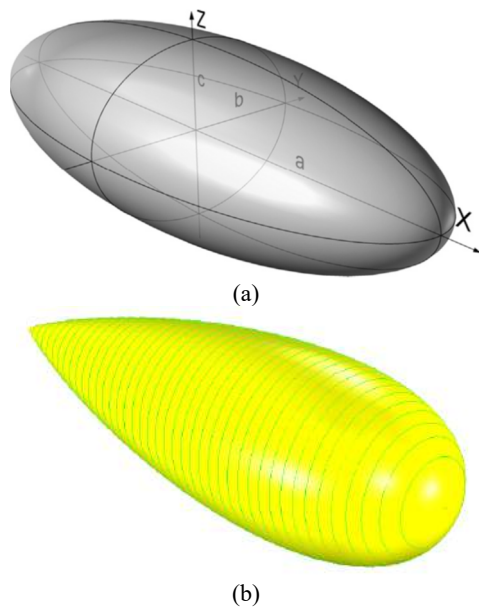


Figure 4: Elipsoidal Center bulb Geometry (Danisman, 2014) (a) Foil-Shaped Center bulb Geometry in this research (b).

The position of Center bulb is at 0.5 Lwl and 0.5 T. Ship resistance is simulated at Fr 0.7 (2.77 m/s). Catamarans that use the center bulb at Fr 0.7 have lower resistance than that do not use the center bulb (Danisman, 2014) and (Samuel, et al., 2018).

Ship resistance calculation uses CFD Software called Tdyn. This software can be downloaded and used free of charge but with a limited amount of meshing. To getting the full version, the password was required. It can be downloaded by registering first. The password is valid for a month since registration. Like other CFD software, the Tdyn analysis process is carried out in 3 stages: pre-processor, solver, and post-processor.

Before simulating ship resistance in CFD, the design of the experiment is determined by using the Central Composite Design (CCD). The first stage is Order I by making the minimum and maximum limits of length (X1) and diameter (X2) center bulb variations. Furthermore, the design of the second phase of the experiment is Steepest Descent. This stage is to find the minimum response value based on the coefficients X1 and X2 of the linear equation generated from Order I.

Next, the results of X1 and X2 from Steepest Descent become the initial model in Order II. Like Phase I, the minimum and maximum limits of variation in length (X1) and diameter (X2) of the center bulb of Steepest Descent are determined again for later to be tested in CFD simulation.

3 RESULTS AND DISCUSSION

3.1 CFD Validation

The validation step is conducted to ensure that the results of the CFD calculation have a small difference to the experimental results. Validated results of the CFD setup, such as mesh sizing (as shown in Table 5), number of steps, initial steps, and time increment, are used to the condition of CFD simulation for other models. These variables affect simulation results.

Table 5: Mesh Sizing.

Ship Surface	0.005
Free surface	0.05
Other Surface	0.1
Max Element	0.5
Transitional	0.5

Mesh size was obtained from several experiments with consideration of the 3 Dimension shape of the ship. The next stage is determining the CFD set up like number of steps, time increments, and initial steps. The final results are presented in Table 6 with the total resistance results, RT of 19.11 N consisting of wave resistance, RW of 13,855 N and viscous resistance, RV of 6.0564 N.

Table 6: Tdyn Setup.

Fr	0.7
No. Of Steps	900
Time Increment	0.08
Initial Steps	81
V (m/s)	2.77
RT (CFD)	19.11 N
RT (Molland)	18.89 N
Error	0.12 %

3.2 Response Surface Methods

3.2.1 Order 1

There are various Designs of Experiment (DoE) (Bezerra, et al., 2008). In this study, the DoE is used using the Center of Composite Design (CCD) with two factors/variables. The regression equation that produced in Order 1 is linear. The variables used are length of center bulb, L (X1) and diameter of center bulb, D (X2). The difference given is $\pm 5\%$. The CFD simulation conditions in Table 6 are used for the

calculation of wave resistance (RW) following the DoE contained in Table 7.

Table 7: Design of Experiment based on Central Composite Design and The Results of Wave Resistance, RW.

Model	Code		Parameter		Rw (N)
	X1	X2	L (mm)	D (mm)	
1	0	0	90	40	14.99
2	-1	-1	85.5	38	14.529
3	-1	1	85.5	42	15.466
4	1	-1	94.5	38	14.375
5	1	1	94.5	42	14.95

Through the regression method, a linear equation (found in Equation 1) is obtained from the experiments conducted in Table 7. This equation has the value $R^2 = 0.908977$, where the value can be considered valid enough to be used.

$$Rw = -0.168X1 + 0.378X2 + 14.862 \quad (1)$$

3.2.2 Steepest Descent

This process is carried out to determine the turning point of the tendency of wave resistance (Rw), which continues to fall and no longer matches the results of Equation 1. Based on the coefficients of each variable in the First Order model, the addition (Δ) of each coefficient for doing steepest descent was calculated by using the coefficient X1 as the basis as shown in Equations 2 and 3. Furthermore, the steepest descent process is shown in Table 8 and Figure 5.

$$\Delta X1 = \frac{-0.168}{-0.168} = 1 \quad (2)$$

$$\Delta X2 = \frac{0.378}{-0.168} = -2.26 \quad (3)$$

Table 8: Steepest Descent.

Step	Code		Parameter		Rw - CFD (N)
	X1	X2	L (mm)	D (mm)	
1 (Mod.1)	0	0	90	40	14.99
2 (Mod.6)	1.00	-2.26	94.50	35.49	14.19
3 (Mod.7)	2.00	-4.51	99.00	30.97	13.56
4 (Mod.8)	3.00	-6.77	103.50	26.46	13.35
5 (Mod.9)	4.00	-9.03	108.00	21.95	13.44

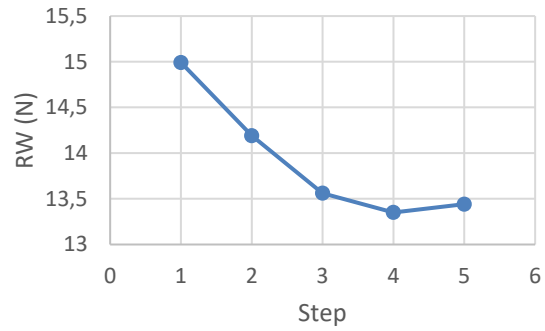


Figure 5: Steepest Descent.

Based on Figure 5, the lowest point is found in Step 4 or in Model 8. The wave resistance then increases in Model 9 so that the variables in Model 8 are used as the central point for experiments in Order 2.

3.2.3 Order 2

Experimental design in Order 2 produces non-linear equations for quadratic functions. For this reason, the number of experiments in DoE was added as in Table 9. The central points ($X1 = 0$ and $X2 = 0$) used were Model 8. The code used for the Second Order was set again as shown in Table 9. With reference to Table 9, DoE for Order 2 is shown in Table 10.

Table 9: Code for Orde 2.

	-1	0	1
X1, Length (mm)	98.33	103.50	108.68
X1, Diameter (mm)	26.46	26.46	27.78

Table 10: Design of Experiment in Order 2 and Wave Resistance, Rw.

Mod	X1	X2	X1 ²	X2 ²	X1*X2	Rw (N)
8	0	0	0	0	0	13.35
10	-1	-1	1	1	1	13.92
11	-1	1	1	1	-1	13.95
12	1	-1	1	1	-1	13.55
13	1	1	1	1	1	13.64
14	-1.414	0	1.9994	0	0	13.62
15	1.414	0	1.9994	0	0	13.98
16	0	-1.414	0	1.9994	0	13.79
17	0	1.414	0	1.9994	0	14.05

The regression equation from the experiments conducted in Table 10 is found in Equation 4. The

equation has the value $R^2 = 0.540206$. The 3D Surface of Eq. 4 is shown in Figure 6.

$$Rw = -0.020239X1 + 0.061318X2 + 0.201557(X1)^2 + 0.261325(X2)^2 + 0.015(X1X2) + 13.34806 \quad (4)$$

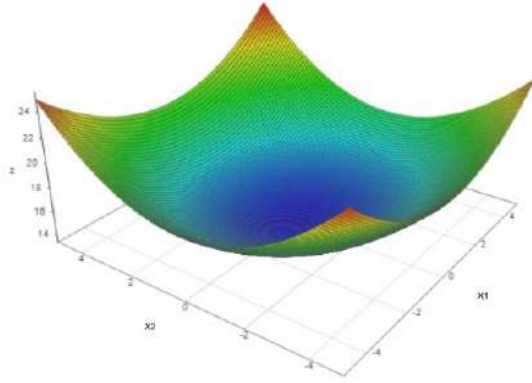


Figure 6: 3D Surface of Second Order Equation for wave resistance, RW.

3.2.4 Optimum Point

To find the minimum value of the Order 2 regression equation (Equation 4), then the first derivative of the equation must have a zero value according to Equations 5 and 6.

$$\frac{dRW}{dX1} = 0 \quad (5)$$

$$\frac{dRW}{dX2} = 0 \quad (6)$$

According to Equations 5 and 6, the optimum point is located at $X1 = 0.0546$ and $X2 = -0.1189$ are obtained. Based on the codification in Table 9, the optimum length and diameter are shown in Table 11. The center bulb length, which was 90 mm increased by 15.31% to 103.78 mm. While the center bulb diameter, which was 40 mm reduced by 34.25% to 26.30 mm. Visualization of the comparison of initial center bulb size with optimum center bulb size is shown in Figure 7.

Table 11: Optimum Size of Foil-Shaped Center bulb.

Model	Code		Parameter	
	X1	X2	L (mm)	D (mm)
18 (Optimum)	0.0546	-0.1189	103.78	26.30

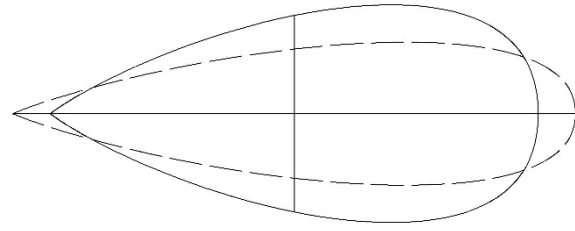


Figure 7: Model 1 (initial model) with solid lines and Model 18 (optimal model) with dashed lines.

3.3 Hydrodynamic Comparisons

The value of the wave resistance, RW from the optimum parameter based on CFD is 13.23 N, while based on Equation 4 is 13.34 N. The difference between the two is quite small at -0.83%.

The comparison of wave resistance between the optimum model (Model 18) and the initial model (Model 1) has been carried out. The optimum model reduces the wave resistance by 11.74% from 14.99 N to 13.23 N compared to the initial model, as presented in Figure 8. This optimization method successfully minimizes the wave resistance from initial center bulb models.

The next discussion is about the comparison of wave resistance using center bulb (initial and optimal models) to models that do not use center bulb. The value of wave resistance that does not use center bulb as stated in the validation section is 13.855 N.

As can be seen in Figure 8, the use of the initial center bulb model adds a wave resistance of 8.19%. This reason is that model has not been optimized. Center bulb configuration is also based only on center bulb geometry, not varying the position of center bulb placement. Besides, the determination of the

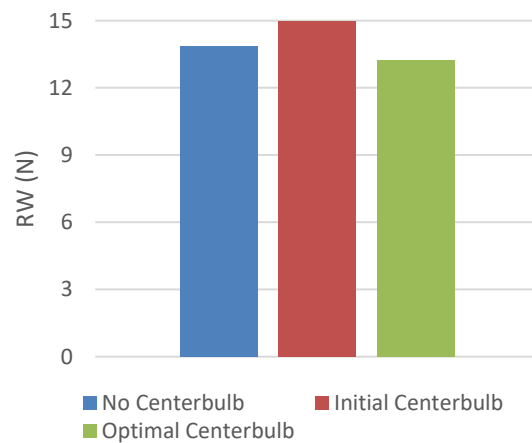


Figure 8: Comparison of Catamaran Without, Initial and Optimum Center bulb.

geometry of the initial model is obtained only by the ratio. So for the initial center bulb model, it is still far from the expectation to reduce wave resistance.

After the foil-shaped center bulb is optimized, the wave resistance decreases by 4.72%. When compared with the results of Danisman's research, this value is not significant enough. The reason is due to the optimization parameters still using two variables, namely length and diameter, whereas in Danisman's used 3 variables, namely height and width length of center bulb. Besides, the configuration of center bulb

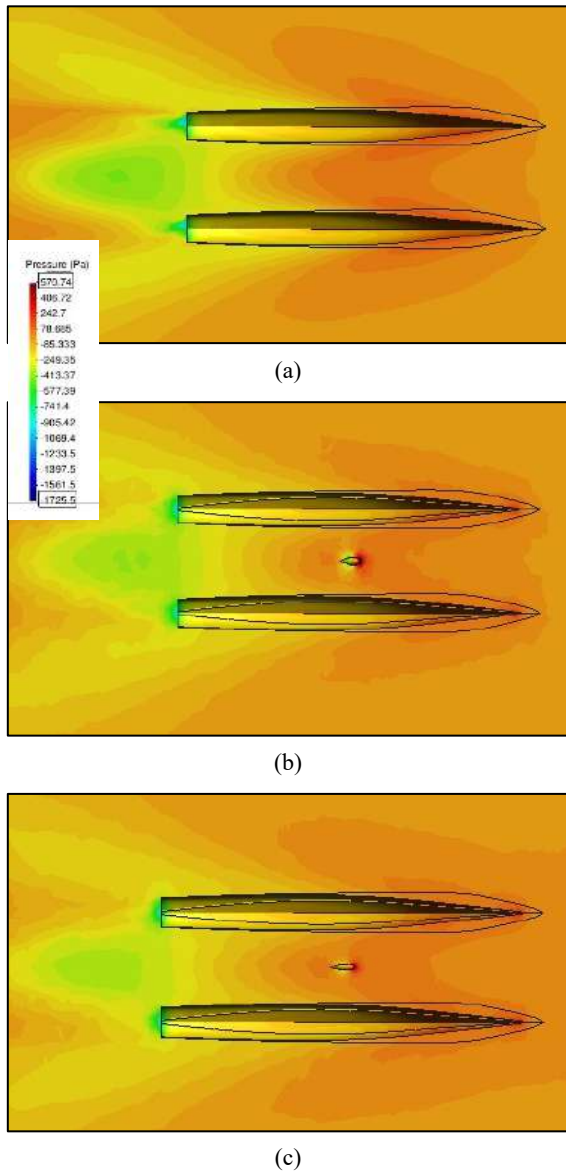


Figure 9: Comparison of Catamaran Pressure Contours that do not use Center bulb (a), Initial Center bulb (b) and Optimal Center bulb (c).

position was taken into account to produce a significant reduction in wave resistance.

Figure 9 is an illustration of the pressure contour between the two catamaran hulls seen based on the intersection of the z-axis in the center of the center bulb. It can be seen in Figure 9.a that overall, the pressure between the two hull has a higher pressure than that using center bulb (b and c). This phenomenon indicates that center bulb reduces pressure between the hull.

However, in Figure 9.b the pressure contour increases significantly only in front of the center bulb. The initial center bulb has wide shape making the water flow hit the center bulb. The flow speed will stop for a moment causing the pressure increases. In Figure 9.c. the optimal center bulb shape is slender than the initial model. When the flow hit the center bulb, the pressure did not increase significantly. For more details, see Figure 10.

The results of this study prove that the Response Surface Method (RSM) can be used in field Naval Architecture and successfully implemented as an optimization tool to find the lowest wave resistance based on the size of the center bulb. Overall, the Response Surface Method (RSM) makes it easy to find the optimal value of the shape size of the center bulb.

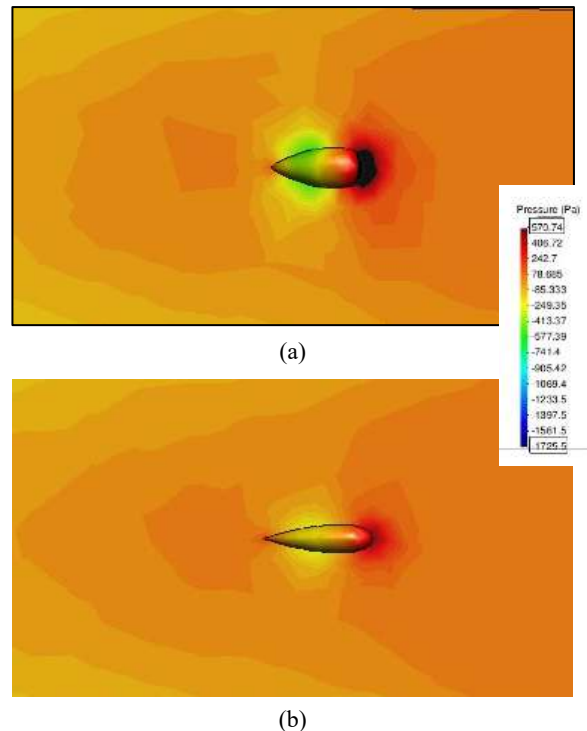


Figure 10: Comparison of Catamaran Pressure Contours Around Foil-Shaped Center bulb. (a) Initial model (b) Optimum model.

4 CONCLUSIONS

The optimal center bulb length is 103.78 mm or has 15.31% increase from the initial length of 90 mm. The optimal diameter size is obtained at 26.30 mm or reduced by 34.25% from the initial diameter of 40 mm. The optimal model reduces the wave resistance by 11.74% from the initial center bulb model and reduces by 4.72% compared to not using a center bulb. Overall, the Response Surface Method is another approach to determine the optimal size of foil-shaped center bulb in order to reduce wave resistance.

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Environmental Conditions in Karimunjawa to Support the Integrated Multi-Trophic Aquaculture (IMTA) Program

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Keywords: IMTA, Coral Reef, Aquatic Quality, GIS.

Abstract: Karimunjawa National Park is a nature conservation area managed by the zoning system. Of the 9 zones that have been determined, one of them is the Maritime Cultivation Zone which is intended to support the interests of aquaculture. Aquaculture based on Integrated Multi-Trophic Aquaculture (IMTA) is a development system that supports cultivation by integrating a variety of different species. This study aims to analyze the suitability of Karimunjawa waters based on the measurement of environmental parameters such as temperature, DO, Ph and current salinity to support IMTA in Karimunjawa waters and observe the coral reef ecosystem as a supporter of assessing the condition of the aquatic environment. Based on the results of measurements of environmental parameters, the results obtained are still relatively good quality based on water quality standards. The average water temperature is 29.7 °C, dissolved oxygen (DO) is 7 mg / l, pH is 8.1 and salinity is 32.7 ppt.

1 INTRODUCTION

Karimunjawa National Park is designated as a nature reserve area located in Jepara Regency, Central Java Province in Decree of the Minister of Forestry and Plantation No. 78/Kpts-II/1999 dated February 22, 1999, with an area of 111,625 hectares (BTNKJ, 2019). In Law No.5 of 1990 concerning Conservation of Natural Resources and their Ecosystems defines national parks as Nature Conservation Areas that have native ecosystems, managed with zoning systems that are utilized for research, science, education, supporting cultivation, tourism and recreation. Based on Decree of the Director General of PHKA No. SK 28/IV-SET/2012 concerning Zoning of Karimunjawa National Park, currently there are 9 (nine) zones within the Karimunjawa National Park area. The zoning of Karimunjawa National Park is fully presented in Figure 1. From 9 zones that have been determined, one of them is the Maritime Cultivation Zone, which is intended as a supporter of the interests of aquaculture such as seaweed cultivation, floating net cages with due regard to conservation aspects.

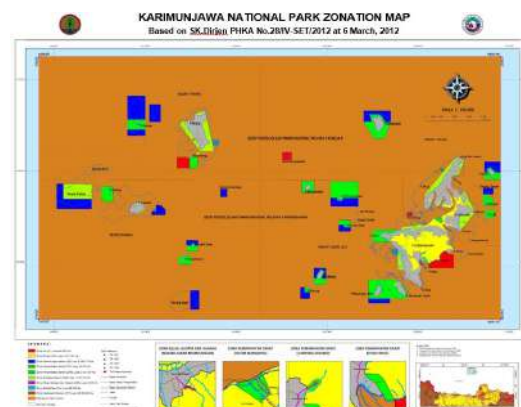


Figure 1: Zoning Map of Karimunjawa National Park.

Integrated aquaculture (integrated multi-trophic aquaculture / IMTA) is an innovation in the development of an aquaculture system that is applied to address the issue of the impact of aquaculture activities on the aquatic environment including sedimentation and enrichment of aquatic nutrients (Radiarta et al., 2014; 2015; Alexander et al., 2016). The IMTA system combines several different trophic-level species, between species that are fed (eg fish) and species that absorb inorganic material (eg seaweed) and species that absorb

organic matter (suspension and deposit feeders such as shellfish) Troell et al., 2009; Barrington et al., 2009).

The implementation of the IMTA system not only produces high economic value biomass through product diversification but can also reduce waste entering the waters. In the IMTA system, waste generated from the main commodities that enter the waters will be converted and utilized by species absorbing organic and inorganic materials. The selection of commodities for the IMTA system is very flexible, can be adjusted to commodities that have developed in an area be it fish, seaweed, or shellfish. The selected commodity is adjusted to its function in the ecosystem and is a commodity with high economic value. By choosing the right cultivation species, this IMTA system will reduce the organic and inorganic content of nitrogen, carbon, and phosphate; making this system a candidate in nutrient trading credits (Chopin et al., 2010; Yuniarsih et al., 2014).

Analysis of the suitability of the condition of the waters of Karimunjawa National Park with the measurement of environmental parameters such as temperature, salinity, DO, and pH, as well as observation of marine aquatic ecosystems such as coral reefs, needs to be done as a first step before the implementation of aquaculture that supports the IMTA system. The IMTA system which is considered to be able to support the conservation aspects is expected to be able to provide more value to the environmental sustainability and fishery products found in the waters of the Karimunjawa National Park.

2 STUDY AREA

The research location is in the waters of the Karimunjawa National Park where the focus of the location is on Karimunjawa Island and Kemujan Island to take samples of water parameters and observe the aquatic environment ecosystem. Sampling is distributed at 7 station points (Figure 2), including Station 1. Harbour, Station 2. Fishing Port, Station 3. Ujung Gelam Beach, Station 4. Mangrove and river mouth, Station 5. Legon Lele/Core Zone, Station 6. Merican Kemujan Island/Utilization Zone of Seaweed Cultivation.

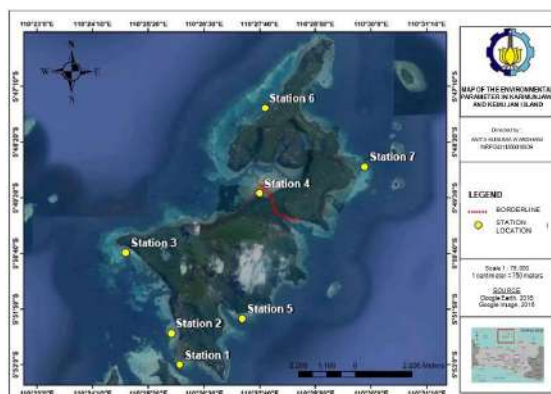


Figure 2: Map of Research Location.

3 METHODOLOGY

Parameter measurements are performed at 7 station points with 3 repetitions at each station point to get valid results. In this activity, fishing boats are used to go to 7 predetermined station points with the help of Global Positioning System (GPS) to obtain location accuracy (Figure 3).



Figure 3: Global Positioning System (GPS) as a Guide to 7 Station Points.



Figure 4: Water Quality Measurement using Refractometer for Salinity.



Figure 5: Water Quality Measurement using DO meter and pH meter.

Temperature measurement using a thermometer, salinity measurement using a refractometer (Figure 4), dissolved oxygen (DO) measurement using a DO meter and pH measurement using a pH meter (Figure 5).

The results of the measurement of environmental parameters are entered in the Surfer software to be processed into a map of the distribution of observed environmental parameters (Figure 7 - Figure 11).

Kriging interpolation is used in data processing to become a map of the distribution of environmental parameters.

Observation of the condition of coral reefs is also done as a consideration to assess quality based on quality standards. Observations were made using the GIS method and then carried out during a field check in the field with snorkelling (Figure 6).



Figure 6: Observation of the Condition of Coral Reefs.

4 RESULT AND DISCUSSION

4.1 Measurement of Water Parameters

Based on 7 predetermined station points, the results of measurements of water parameters include

temperature, salinity, DO, Ph, and currents in Table 1 to Table 7.

Table 1: Measurement of Water Parameters at Station 1.

Parameter	Repetition			Average
	1	2	3	
DO (mg/l)	6,9	6,7	6,7	6,76
Temperature (°)	29	29	29	29
pH	8,2	8,2	8,2	8,2
Salinity (ppt)	32	32	33	32,2

Table 2: Measurement of Water Parameters at Station 2.

Parameter	Repetition			Average
	1	2	3	
DO(mg/l)	6,4	6,5	6,5	6,46
Temperature (°)	30	30	29	29,6
pH	8,2	8,2	8,2	8,2
Salinity (ppt)	35	31	31	32,3

Table 3: Measurement of Water Parameters at Station 3.

Parameter	Repetition			Average
	1	2	3	
DO(mg/l)	7,2	7,3	7,6	7,4
Temperature (°)	29	29	29	29
pH	8,2	8,2	8,2	8,2
Salinity (ppt)	32	33	32	32

Table 4: Measurement of Water Parameters at Station 4.

Parameter	Repetition			Average
	1	2	3	
DO(mg/l)	6,9	6,7	7,2	6,9
Temperature (°)	30	30	30	30
pH	8,1	8,2	8,1	8,1
Salinity (ppt)	33	33	33	33

Table 5: Measurement of Water Parameters at Station 5.

Parameter	Repetition			Average
	1	2	3	
DO(mg/l)	7,2	7,2	7,7	7,4
Temperature (°)	30	30	30	30
pH	8,2	8,2	8,2	8,2
Salinity (ppt)	33	33	33	33

Table 6: Measurement of Water Parameters at Station 6.

Parameter	Repetition			Average
	1	2	3	
DO(mg/l)	6,9	7,2	7,7	7,3
Temperature (°)	30	30	30	30
pH	8,2	8,2	8,2	8,2
Salinity (ppt)	33	33	33	33

Table 7: Measurement of Water Parameters at Station 7.

Parameter	Repetition			Average
	1	2	3	
DO(mg/l)	7,0	7,0	7,0	7,0
Temperature (°)	30	30	30	30
pH	8,2	8,2	8,2	8,2
Salinity (ppt)	33	33	33	33

The value of the measurement of the water parameters is then processed in the Surfer software to determine the distribution pattern. The results are shown in Figure 7 through Figure 10.

From the data that has been obtained, it shows that the results of the measurement of the environmental parameters of the waters are still relatively good after compared to the quality standards (Table 10).

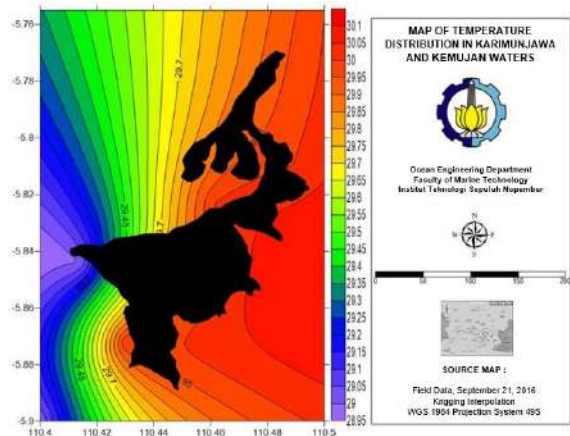


Figure 7: Water Temperatures Distribution on the Island of Karimunjawa and Kemujan.

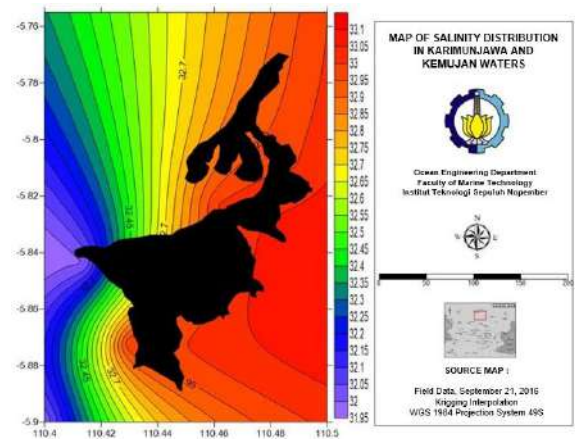


Figure 8: Salinity Distribution on the Island of Karimunjawa and Kemujan.

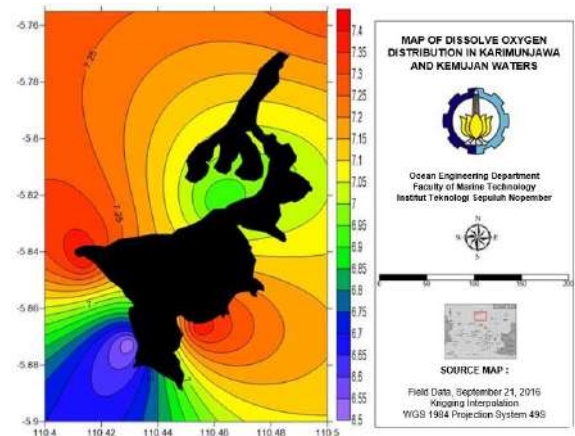


Figure 9: Dissolve Oxygen Distribution on the Island of Karimunjawa and Kemujan.

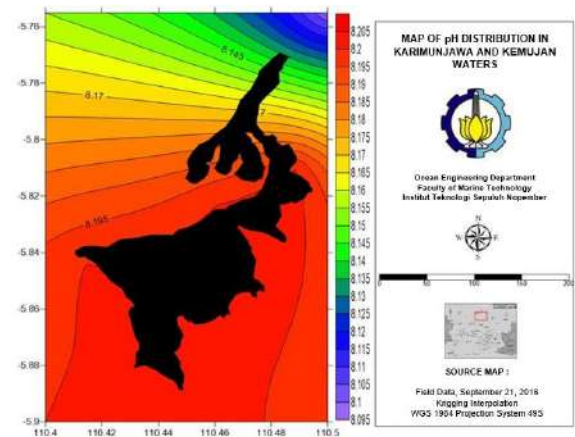


Figure 10: pH Distribution on the Island of Karimunjawa and Kemujan.

4.2 Sea Current Data Processing

Current data processing in the Surfer software shows the current velocity in the waters of the islands Karimunjawa and Kemujan ranging from 0.10 to 0.52 m / s (Table 8). Current data from OSCAR was taken in September, adjusting for the time when the ground check is roomy. In September there was the East to West transition season (September - November). Current patterns that occur in the waters around the Karimunjawa Islands are the effects of changes that are common in Indonesian waters (Ariyati et al, 2005). The map of the distribution of current speed can be seen in Figure 7.

Table 8: Current Speed on The Island of Karimunjawa and Kemujan.

Location	Result (m/s)
Station 1	0,248
Station 2	0,205
Station 3	0,113
Station 4	0,193
Station 5	0,268
Station 6	0,280
Station 7	0,514

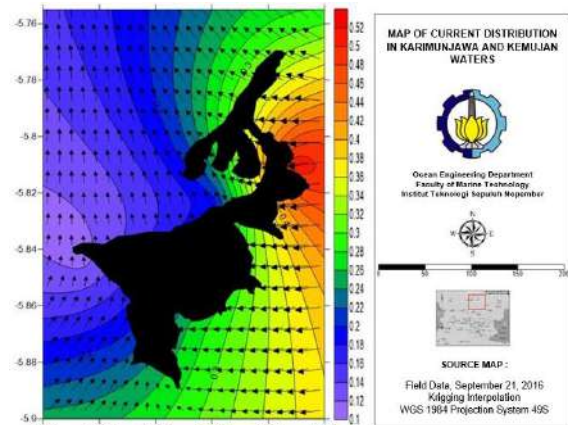


Figure 11: Current Speed Distribution on the Island of Karimunjawa and Kemujan.

4.3 Coral Reef Data Processing

As an additional parameter, the condition of the coral reef ecosystem in the waters of Karimunjawa Island and Kemujan Island was also observed. Initial observations were made using the GIS approach using Landsat 8. Satellite imagery data were taken

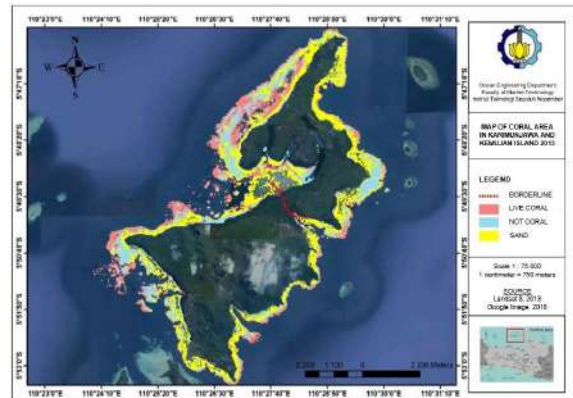


Figure 12: Map of Coral Area on the Island of Karimunjawa and Kemujan in 2013.

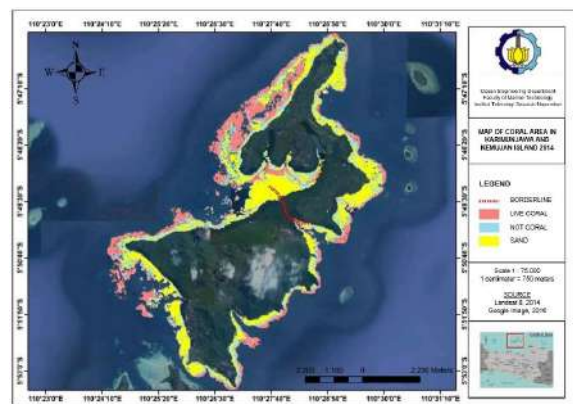


Figure 13: Map of Coral Area on the Island of Karimunjawa and Kemujan in 2014.

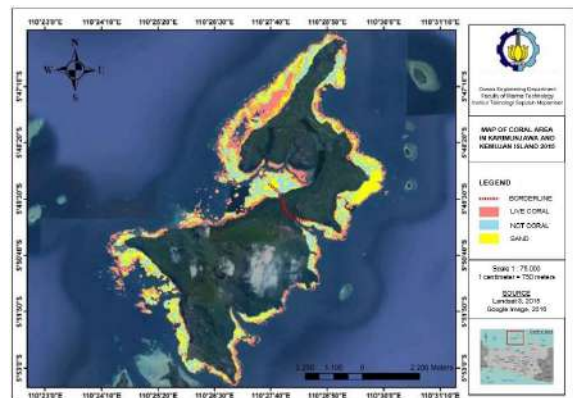


Figure 14: Map of Coral Area on the Island of Karimunjawa and Kemujan in 2015.

within 4 years, namely in 2013, 2014, 2015 and 2016 to see the changes and conditions. In this study, the 1981 Lyzenga algorithm was used to classify objects underwater. Detection of objects below sea level in Landsat data uses blue and red

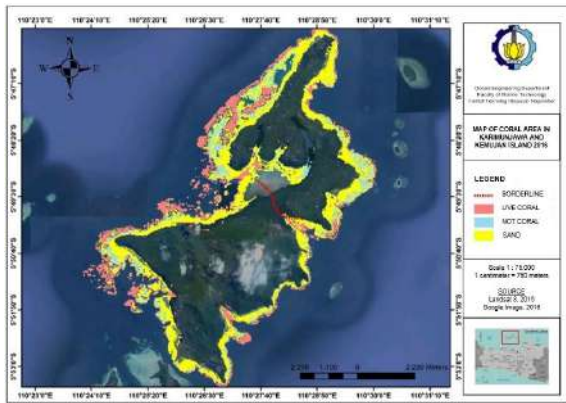


Figure 15: Map of Coral Area on the Island of Karimunjawa and Kemujan in 2016.

canal data. Kardono (1993) states that the blue and red canals are the best channels for penetration into the underwater column because in the blue and red canals there are wavelengths between 0.5 - 0.6 μm which are suitable for penetration into the water column. The following (Figure 12 to Figure 15) is a map of the extent of coral reefs from 2013 to 2016 on Karimunjawa Island and Kemujan Island.

Based on the results of the processing of coral reefs from 2000, 2013, 2014, 2015 and 2016 on Karimunjawa Island and Kemujan Island can be seen from Table 9 below:

Table 9: The Extent of Live Coral, Not Coral and Sand.

Class/ Year	2013	2014	2015	2016
Live Coral (Km ²)	5,864	6,891	6,254	5,914
Not Coral (Km ²)	6,437	6,456	6,070	3,886
Sand (Km ²)	8,884	8,430	6,258	10,802

Table 10: Comparison of Measurement Result of Water Parameter with Quality Standards.

	Parameter	St.1	St.2	St.3	St.4	St.5	St.6	St.7	Marine Biota Quality Standards	Coral Reef Quality Standards	Status
1	Temperature (°C)	29	30	29	30	30	30	30	± 2 natural variation	28-30	Good
2	DO (mg/l)	6,7	6,46	7,4	6,9	7,4	7,3	7,0		>5	Good
3	PH	8,2	8,2	8,2	8,1	8,2	8,2	8,2	6,5 - 8,5	7-8,5	Good
4	Salinity (‰)	32	33	32	33	33	33	33	18 32/ ± 10 natural variation	33-34	Good

From the above data, the results of the percentage estimates of the area obtained from the processing of Landsat satellite image data in 2013, 2014, 2015 and 2016. Classified into 3 classes, namely live coral, not coral and sand. From 2013 to 2014 the extent of coral reefs increased by 8.7%, from 5.864 Km² in 2013 to 6,891 Km² in 2014. From 2014 to 2015, the area of coral reefs decreased by 5.1% to 6,254 Km² in 2015. Then in 2016, there was also a decrease in the area of coral reefs to 5.914 Km² by 2.7%. Following Figure 16 is a graph of the classification results and the extent of Landsat satellite imagery data for 2013, 2014, 2015 to 2016.

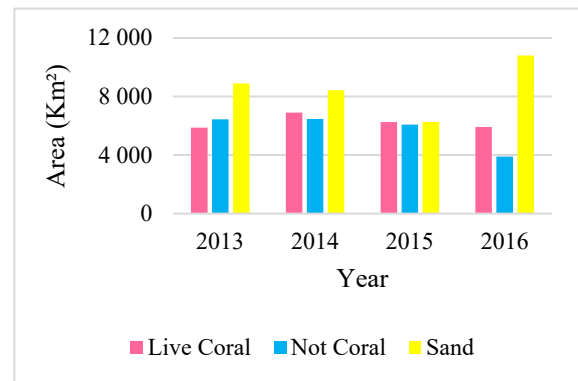


Figure 16: Results of Classification and Extents from Processing of Landsat Satellite Imagery.

Based on observations of the parameters of the quality of waters in Karimunjawa Island at seven points the location of sampling shows that the environmental conditions of the waters of Karimunjawa Island are in "Good" condition for aquaculture and marine biota, because no sample parameters were found with values that exceeded the quality standard as shown in Table 10.

5 CONCLUSIONS

Based on observations of the quality parameters of the waters of Karimunjawa Island at 7 points of the sampling station, it shows that the environmental conditions of the waters of Karimunjawa Island are in "Good" condition for the cultivation and ecosystem of coral reefs. Climate change and human activities are also thought to affect environmental conditions and changes in the size of coral reefs.

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The Influence of Additional Equipment to the Ultimate Strength of FPSO

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Keywords: FPSO, Cross Section, FEM, Ultimate Strength.

Abstract: Nowadays, there have many ship conversions, for example, from double hull tanker to Floating Production Storage Offloading (FPSO). The conversion is conducted due to the change function of the ship. On the other hand, due to the change function of the ship, some items of the ship, such as equipment, need to be evaluated because of those related to the ultimate strength of the ship. The objective of the present study is to analyze the influence of additional equipment to the ultimate strength of FPSO under vertical bending moment for hogging condition. The cross-section of FPSO is taken by considering one-frame space. The numerical method is used, and the Multi-Point Constraint (MPC) is applied to both sides of the cross-section, and it is assumed to have remained plane. The shell element type is used for modeling the cross-section. The material properties are set to be constant. As a simple calculation, the ship's cross-section is analyzed in intact. The additional equipment is calculated and included in the analysis to know the influence of it. The analysis of the ultimate strength, including their influence of additional equipment, is conducted under longitudinal bending. To compare the ultimate strength obtained by the numerical method, the analytical method is adopted. It is found that the ultimate strength gained by the numerical method is almost identical to the analytical method. The behavior of the ship's cross-section is also presented in terms of stress distribution and deformation.

1 INTRODUCTION

The conversion of a ship is implemented in recent years. This phenomenon is mostly conducted for an offshore structure such as from Double Hull Tanker to Floating Storage Offloading (FSO) or Floating Production Storage Offloading (FPSO). The conversion aims to change the ship function and operation, including ship payload. In some years, the conversion from tanker to FPSO is very significant. Time-consuming is shorter than a new design; that is one of the reasons. On the other hand, during conversion, some equipment must be added not only for the ship's function but also for the ship's construction. Due to this reason, the analysis of the ultimate strength of being converted considering the additional equipment must be taken into account.

The analysis of the ship's strength had been introduced by some papers like; The residual strength of an Aframax-class double hull oil tanker

damaged in the collision had been assessed by (Parunov, Rudan, and Bužančić Primorac, 2017) considering the influence of the rotation of the neutral axis. The impact of nonlinear finite element method models on the ultimate bending moment for hull girder was studied by (Xu and Pan 2017). There were two analyses performed; those were implicit static analysis and explicit dynamic analysis. A structural reliability analysis model based on a Bayesian belief network was proposed by (Li and Tang, 2019) for the hull girder collapse risk after accidents. The Bayesian belief network was used to represent random states of variable risk events after accidents, as well as the dependencies between activities, and the structural reliability analysis was used to evaluate the failure probability hull girder for each possible accident conditions. The incidence of collision damage models on an oil tanker and bulk carrier reliability was investigated by (Campanile, Piscopo and Scamardella, 2018) considering the

IACS deterministic model against GOLADS/IMO database statistics for collision events, substantiating the probabilistic model. Safety of an oil tanker in intact condition was performed by (Campanile, Piscopo, and Scamardella, 2017) to investigate the incidence of load combination methods on hull girder sagging/hogging time-variant failure probability. The simplified approach to the ultimate hull girder strength of asymmetrically damaged ships was conducted by (Muis Alie, 2018), considering the critical element under sagging condition. The effect of symmetrical and unsymmetrical configuration shapes on buckling and fatigue strength analysis of fixed offshore nonlinear finite element was conducted by (Muis Alie, 2016), and damages were modeled simply by removing the element on the damaged part. The progressive collapse analysis of the local element was conducted by (Muis Alie and Latumahina, 2019) and the cross-section of the Ro-Ro under longitudinal bending.

In the present study, the hull girder strength analysis caused by additional equipment on FSO after being converted into FPSO under longitudinal bending is conducted. The ship's construction is not changed, but additional equipment after conversion is applied. For the simple calculation, the one-frame space of FPSO's cross-section is considered. The cross-section is assumed to have remained plane. The element type of shell 181 is used on the model. The initial imperfections, cracks, and residual welding stress are not taken into account. The ultimate strength obtained by the numerical method is compared with the analytical method, and the behavior of the ship in terms of stress distribution and deformation are also shown in this study.

2 FINITE ELEMENT MODEL

In the present study, the numerical method to analyze the hull girder strength due to load equipment of change function from FSO to FPSO before and after being converted is conducted. The ship has 172 m, 30 m, and 18.4 m of length, breadth, and depth of ship, respectively. The midship section consists of two kinds of longitudinal stiffeners those are flat-bar and angle-bar. The analysis of the ultimate strength is performed under the hogging condition. There is also the inner hull in the cross-section. The element type is shell-181. The shell-181 element applied to all of the cross-sections, as shown in figure 1.



Figure 1: Finite element model.

According to figure 1, there are two points located at the neutral axis. These two points are placed at both sides of the cross-section, and those are used to place the MPC (Multi-Point Constraint) for representing the behavior of the cross-section. The ultimate strength analysis, including the effect of change function from FSO to FPSO, is calculated using the numerical method under sagging condition. The rigidly linked corresponding with the boundary condition where MPC is applied to both sides of the cross-section, as shown in figure 2.

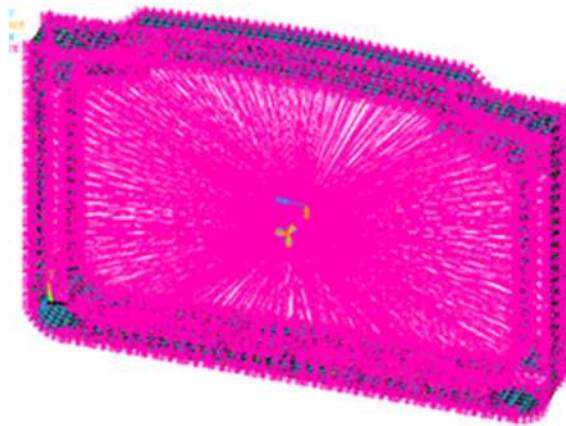


Figure 2: Rigid link and MPC.

According to figure 2, the applied MPC is placed at the fore and aft side of the cross-section. This is done to connect the MPC called master node and the points of the all side's cross-section. The applied moment is placed at this MPC.

3 RESULTS AND DISCUSSIONS

The behavior of the ultimate strength analysis is described in terms of working stress distribution. Figure 3 shows the working stress of FSO under the hogging condition. The deck and bottom part are under tension and compression since the hull cross-section is under hogging condition.

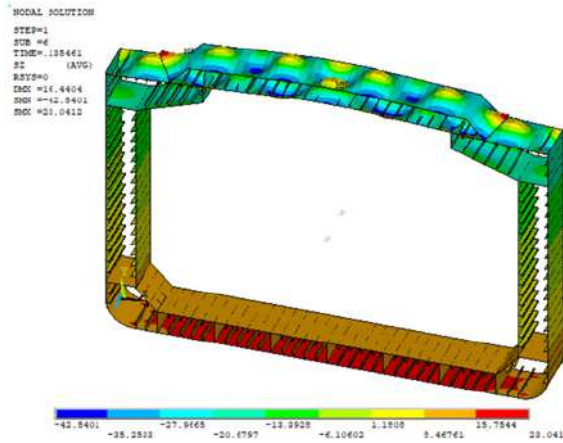


Figure 3: Working Stress of FSO in Hogging.

Figure 4 shows the working stress of FPSO under the hogging condition. It is observed that the characteristic of the working stress of FSO and FPSO is different from one another due to the additional equipment after being converted.

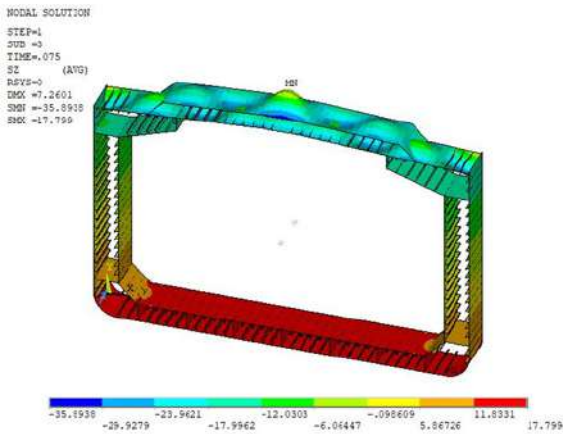


Figure 4: Working Stress of FPSO in Hogging.

Figure 5 shows the moment-curvature relationship of FSO under the hogging condition. The ultimate strength and collapse stages are represented by points A and B, respectively.

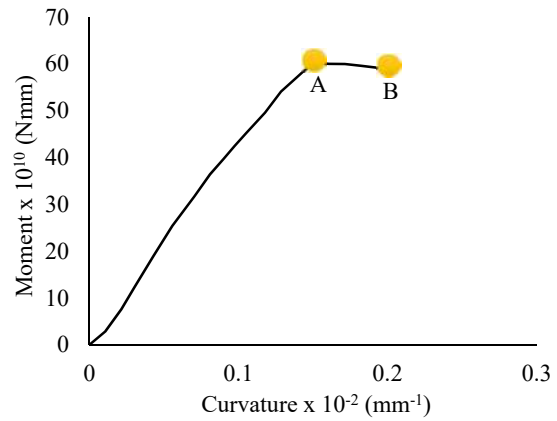


Figure 5: Moment-Curvature of FSO in Hogging.

Deformation of the FPSO at the ultimate strength and collapse stage at point A and B under hogging condition are shown in figures 6 and 7, respectively. Failure takes place at some elements at the deck part where tension happen under hogging condition.

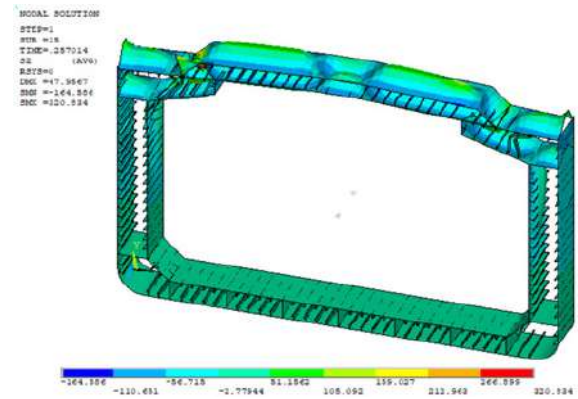


Figure 6: Deformation of FSO at Ultimate Strength.

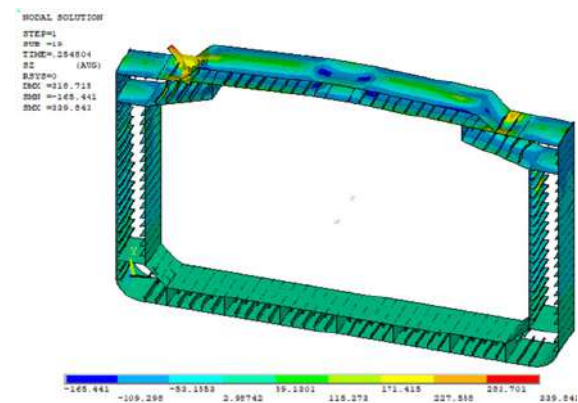


Figure 7: Deformation of FSO at Collapse.

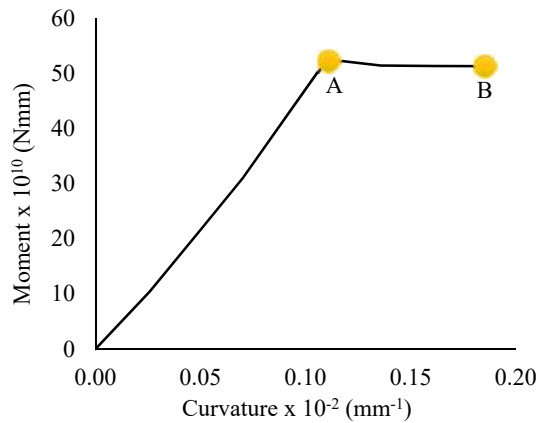


Figure 8: Moment-Curvature of FPSO in Hogging.

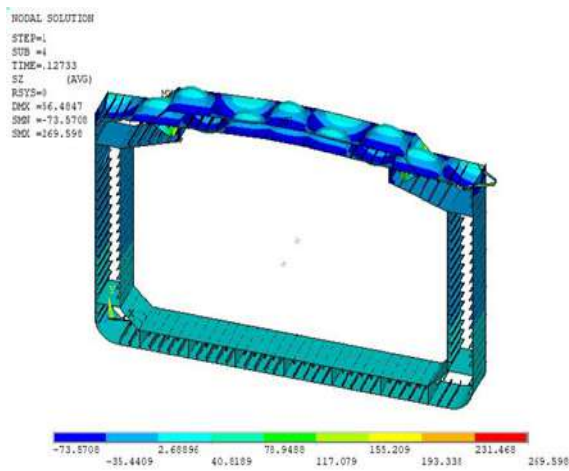


Figure 9: Deformation of FPSO at Ultimate Strength.

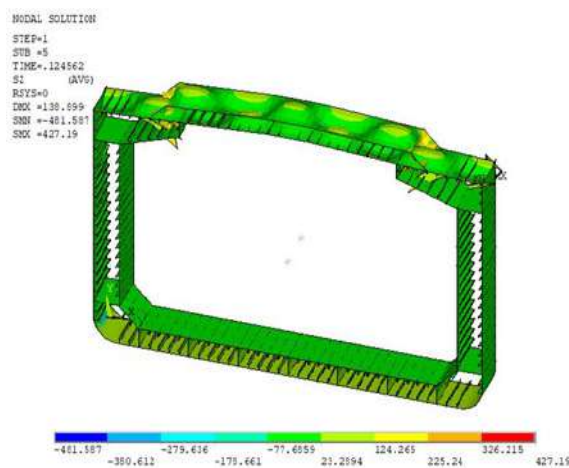


Figure 10: Deformation of FPSO at Collapse.

Figure 8 shows the moment-curvature relationship of FPSO at point A and B in hogging conditions. Points A and B represent the ultimate strength and collapse regime, respectively. Figures 9 and 10 show

the deformation at the ultimate strength and collapse region of FPSO under hogging condition.

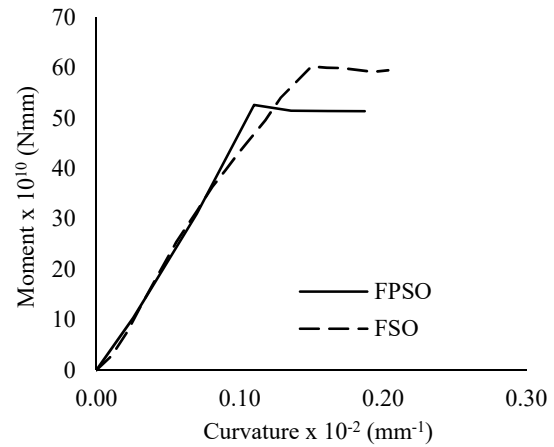


Figure 11: Moment-Curvature Relationship.

Figure 11 expresses the comparison of the moment-curvature relationship after being converted from FSO to FPSO under the hogging condition.

According to figure 11, the bending moment capacity of FSO is larger than FPSO. This is because the additional equipment is applied from FSO to FPSO so that the capacity bending moment after being converted is decreased.

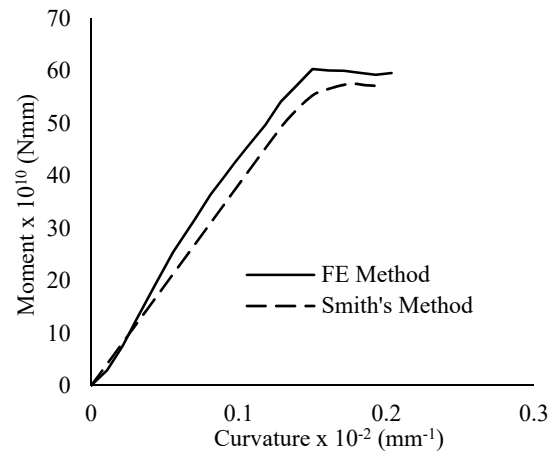


Figure 12: Comparison of Moment-Curvature Relationship.

For the comparison purpose for the ultimate strength analysis, the analytical method is adopted. Figure 12 shows the comparison of the ultimate strength obtained by the numerical method and analytical method. The numerical method is represented by the solid line and the dashed line for the analytical method. It is found that the ultimate strength obtained by the numerical method is larger than the analytical method. This phenomenon happens due to

the redistribution of the stress concentration takes place since the three-dimensional analysis is considered. The bending stiffness is also changed due to the additional equipment after being converted from FSO to FPSO.

4 CONCLUSIONS

The ultimate strength analysis has been conducted by using the numerical method under longitudinal bending in hogging condition. The following conclusions are; the additional equipment has a significant influence on the ultimate strength after being converted from FSO to FPSO. The bending stiffness is different due to additional equipment before conversion from FSO to FPSO. The ultimate strength obtained by the numerical method for FPSO is larger than FSO.

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Analysis of the Effect of Addition of Bilge Keel, Bulbous Bow, Skeg and Combination on Value of Ship Resistance and Seakeeping of Ship using CFD Method

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Keywords: Resistance, Seakeeping, Bulbous Bow, Skeg, Bilge Keel.

Abstract: The hull added by the Bulbous Bow, Skeg, and Bilge Keel can improve the ship's ability to operate. Based on several studies it is known that the best bulbous bow model is the V model, the best Skeg form is the Foil model and the most optimal Bilge Keel form is the hollow triangle shape, where the most optimal variation is to reduce the value of the resistance and ship seakeeping. To achieve these objectives, several stages are carried out, namely making a model, then analyzing the model using CFD software. The results of this study indicate that the best resistance value occurs in the ship model with Bilge keel with a value of 8.8 kN or a decrease of 13.72% of the ship without the addition of components. Then for the value of the Motion obtained Heaving value of 0.84774 m on the ship without the addition of components, for the best pitching value that is on the ship with the addition of Bulbous bow, Skeg and Bilge keel with a value of 0.080775 radians and for the best Rolling value is on the Ship with additions Bilge keel with a value of 0.000082404 rad.

1 PREFACE

The development of the shipping world is growing. This is related to the performance of the ship which is getting better over time. In the design of the ship there is a lot that must be considered including the resistance that occurs and the seakeeping of the ship. The resistance and speed of flow in the propeller of the ship will have a large influence on speed. In addition, high resistance also affects the power requirements to reach a certain speed. The greater the resistance of the ship, the greater the main engine power (main engine) needed to push the ship (Daud et al., 2019).

Beside of the resistance, the thing to note is the seakeeping. Seakeeping is the ability to defend itself as a result of waves that occur. We often call it Seakeeping. Seakeeping is one of the most important components to find out if the ship has the feasibility of making a cruise on the ocean waves. To get the seakeeping value can be analyzed using Towing Tank or computational or CFD methods. The vessels used in this study are mono hull vessels with modified component additions, namely bulbous bow, skeg and bilge keel.

Variations in the geometric shape of the skeg when using the bilgekeel model which has the smallest total resistance value are Foils with a value of 9.28 KN in the position of AP (stern) and model without using bilgekeel which has the smallest total resistance value ie Foils with a value of 9.31 KN in the AP position (Stern). The Foils model has the smallest resistance value because the laminar structure of the foils model can accelerate the flow of water and no flow is blocked when passing through the structure of the foils (Rishwanda., 2018).

While the shape of the bilge keel used is a hollow bilge with a triangular shape which has the smallest resistance value which can reduce the total resistance of the vessel by 20.626% at F_n 0.28 and 16.396% at F_n 0.36 (Avian et al., 2018).

This study will focus on the comparison of the values of resistance and Seakeeping from each variation with the CFD method. At present, CFD has become a numerical device that is very effective in analyzing fluid flow. ABS, as one of the classification bureaus, is making efforts to implement CFD technology to apply CFD technology to assess the strength of modern commercial vessels and high-speed naval vessels. Software used is Software Analysis of seakeeping.

From the explanation above the problem can be formulated in this study, namely the effect of adding components to the value of resistance and sea keeping and the comparison of the value of resistance and seakeeping of each variation.

2 METHODS

In this study, the authors have primary data on the main size of the KM Egon vessel:

LOA : 26.370 m
B : 5.500 m
T : 1,000 m
H : 2.622 m
V : 10 Knot

Wave Direction : 180 degree

For secondary data obtained from the literature (journals, books, and data obtained from previous studies).

The parameters of the study were focused on the effect of adding components to the hull on barriers and vessel seakeeping. The parameters used are as follows:

1. Permanent Parameters

The author uses primary data of the main size of the KM ship. Egon as a fixed parameter in this study.

2. Variable Parameters

- Ship without Component Additions
- Ship with the addition of bulbous bow type V
- The vessel with the addition of a Bilge Keel hollow triangular shape with a 50% LOA Length
- Ship with the addition of Skeg in the form of Foils
- Ship with the addition of Bulbous bow, bilge keel, and skeg

The process of making a model using Modeler. Calculation of resistance and seakeeping uses the Computational Fluid Dynamics (CFD) method, which is one branch of fluid mechanics that uses numerical methods and algorithms to solve and analyze problems related to fluid flow (Kim, 2011).

The Computational fluid dynamics method consists of three main elements, namely:

- *Pre Processor*
- *Solver Manager*
- *Post Processor*

Computational fluid dynamic cannot completely replace measurements experimentally, but the amount and cost of experiments performed can be reduced.

3 RESULTS AND DISCUSSION

3.1 Data Processing

This Design Modeling using Modeler.

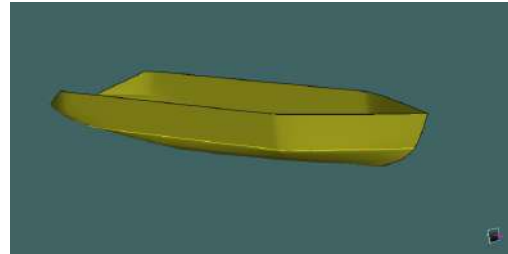


Figure 1: Model of Ship Without Additional Components.

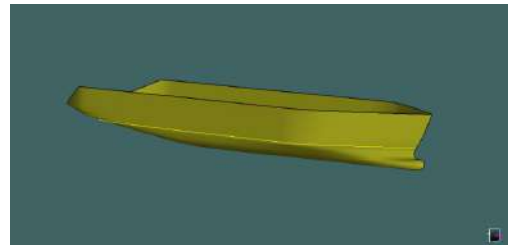


Figure 2: Model of Ship with Bulbous Bow Model V.

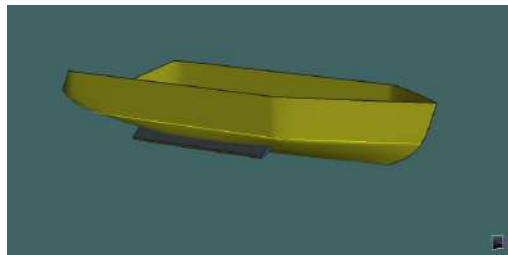


Figure 3: Model of Ship with Hollow Triangle Bilge Keel.

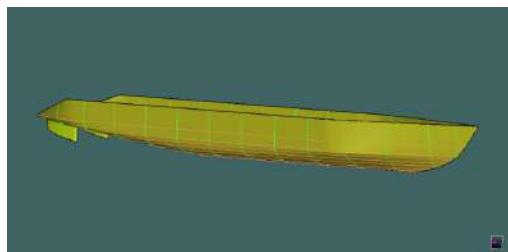


Figure 4: Model of Ship with Skeg Shape Foil.

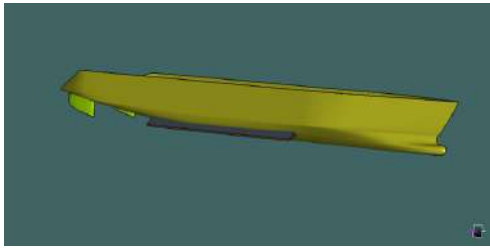


Figure 5: Model of ship with bulbous bow, Bilge keel, and Skeg.

After the model is ready, then the resistance values are tested and the motion is performed on the CFD software.

3.2 Value of Resistance

To get the resistance value, the author uses CFD software with the Hulltrot method, an efficiency of 85% and a speed of 10 knots. The test results of each variation can be seen in the following figure

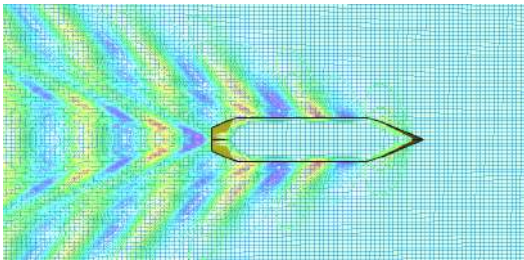


Figure 6: Ship Surface Image without Additional Components.

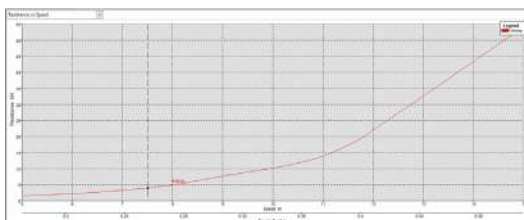


Figure 7: Resistance and Ship Speed Chart without Additional Components.

From testing on ships without additional components, the value of the total resistance was 10.2 kN at a speed of 10 knots.

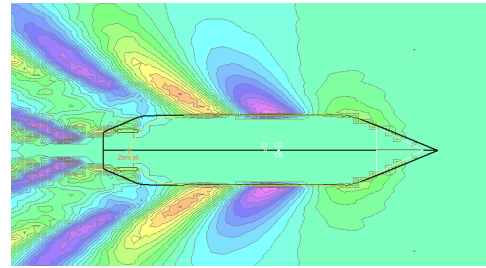


Figure 8: Ship Surface with Skeg.

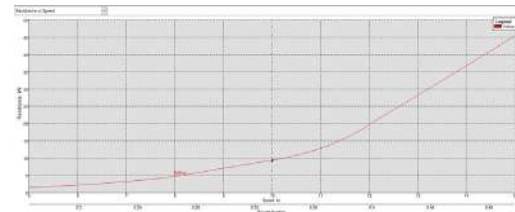


Figure 9: Resistance and Ship Speed Graph with Skeg.

From testing on a ship with Skeg, the total resistance value of 9.4 kN was obtained at a speed of 10 knots.

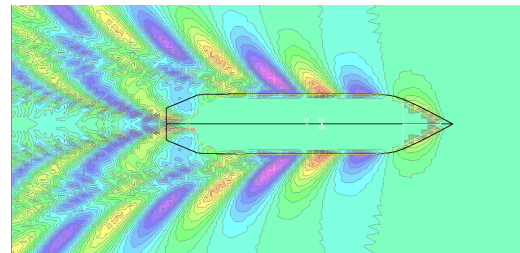


Figure 10: Surface Ship with Bulbous Bow.

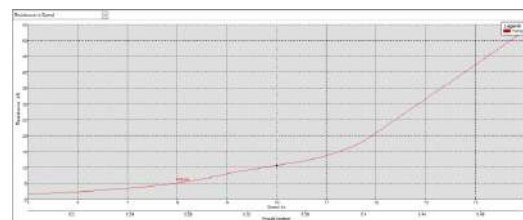


Figure 11: Resistance and Ship Speed Graph with Bulbous Bow.

From testing on a ship with a bulbous bow, it was found that the total resistance was 10.6 kN at a speed of 10 knots.

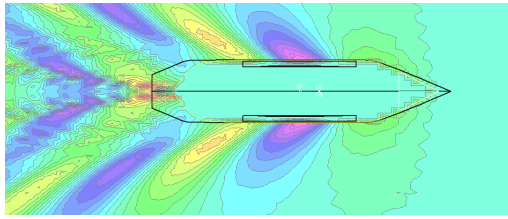


Figure 12: ShipSurface with Bilge Keel.

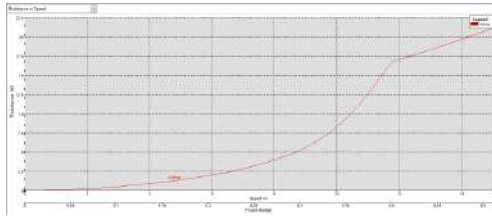


Figure 13: Resistance and Ship Speed Graph with Bilge Keel.

From testing on a ship with a bulbous bow, it was found that the total resistance was 8.8 kN at a speed of 10 knots.

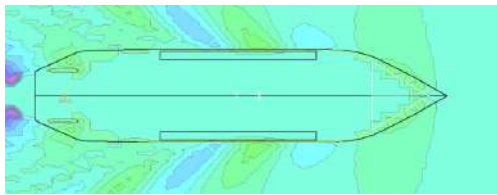


Figure 14: Ship Surface Combination.

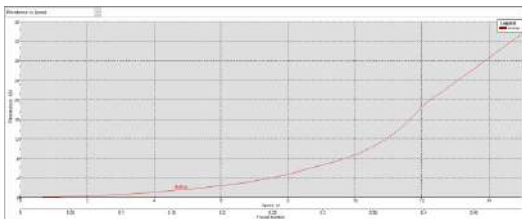


Figure 15: Resistance and Ship Speed Graph Combination.

From chart above can be summarized in the following diagram

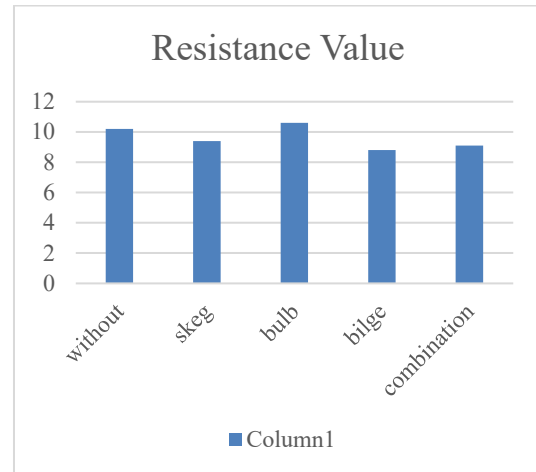


Figure 16: Resistance Value Graph for each variation.

Based on the graph above, it can be concluded that the most optimal resistance value of each predetermined variation is on the ship model with the addition of Bilge Keel with a value of 8.8 kN. When compared with the resistance value of the ship without adding components, the addition of bilge keel components can reduce the value of the resistance by.

3.3 Value of Seakeeping

The seakeeping sought in this study is the natural response of an object when hit by waves, including Heaving, Pitching and Rolling.

The waves used are regular waves with a wave frequency of 0.04 Hz, wave height of 1 m and the direction of the wave is 180 degrees.

The results of the ship seakeeping from each variation can be seen in the following Table and Diagram:

Table 1: Heaving value for each variation.

Model	Heaving Value (m)
Without	0.84774
Bulb	0.85998
Skeg	0.86012
Bilge	0.85881
Combination	0.86621

From Table 1 it is known that the smallest value of Heaving occurs in the model without the addition of components with a value of 0.84774 m. This can happen because the Heaving seakeeping displacement value greatly affects the value of the Heaving.

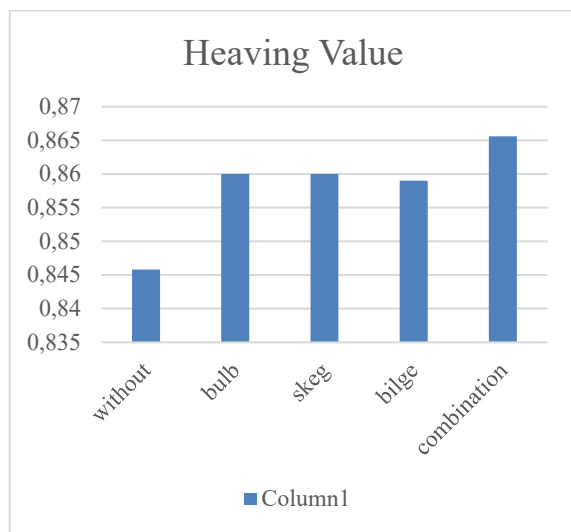


Figure 17: Heaving Value Diagram.

From Figure 17 above, it can be seen the difference in the value of heaving from each model variation where the effect of adding displacement actually adds to the heaving value of the ship

Table 2: Pitching value for each variation.

Model	Pitching Value (radian)
Without	0.081658
Bulb	0.080820
Skeg	0.081457
Bilge	0.081250
Combination	0.080755

From Table 2, it can be seen that the lowest pitching value is on the Ship with a combination of the addition of Skeg, Bulbous Bow and Bilge keel with a value of 0.080755 radians.

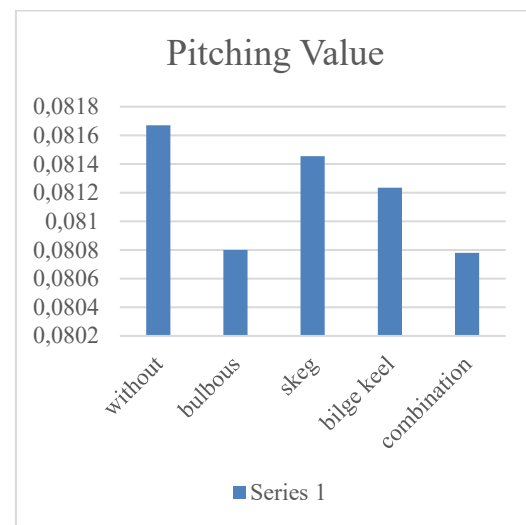


Figure 18: Pitching Value Diagram.

From Figure 18 it can be seen that the Pitching value is influenced by the addition of components in the longitudinal section, namely the bulbous bow even though the addition of the keel and skeg bilge also reduces the pitching value but is not significant.

Table 3: Rolling value for each variation.

Model	Rolling Value (m)
Without	0.0000157195
Bulb	0.0000122404
Skeg	0.0000102404
Bilge	0.0000082404
Combination	0.0000092404

From Table 3, the lowest rolling value can be seen on the Ship with Bilge Keel with a value of 0.0000082404 radians.

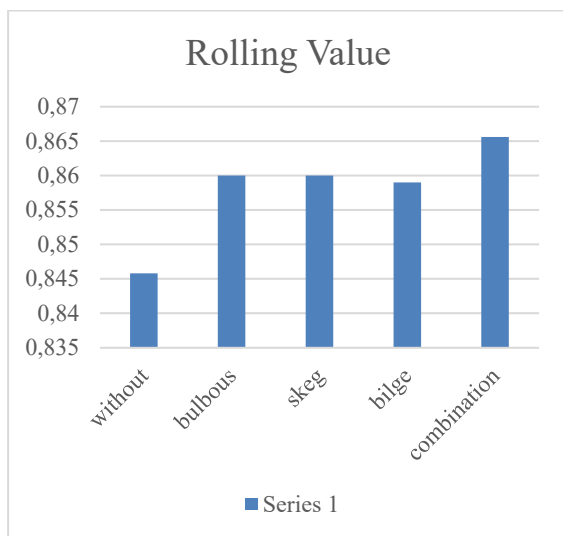


Figure 19: Rolling Value Diagram.

From Figure 19 it was affected by the addition of components in the transverse direction such as Bilge Keel and Skeg, but the addition of Bilge Keel was more significant in reducing the Rolling value.

4 CONCLUSION

Based on the experiments that have been done, it can be concluded as follows:

The value of total resistance is best for ships with the addition of triangular and hollow bilge keel which can reduce ship resistance by 13.72% with a total resistance value of 8.8 kN.

For the value of sports, each boat has a significant difference. The best value of Heaving is on a Ship without Component Addition with a value of 0.84774 m, for Pitching the best value is found on a Ship with a combination of adding a bulbous bow, skeg and keel bilge with a value of 0.080775 radians and in Rolling the best value is on the Ship Bilge Keel with a value of 0.000082404 radians.

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Experimental Study on the Effect of Mooring Angle on Mooring Tension of Porous Saw Type Floating Breakwater

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Keywords: Floating Breakwater, Porous, Mooring Tension, Model.

Abstract: The floating breakwater is commonly used as an alternative solution to conventional fixed breakwaters and can be used effectively in coastal areas in environments with relatively small waves, poor soil conditions, deep-sea depth, and shore with intense erosion rate. Since it is a floating structure, a mooring system is used to maintain its position. This study aims to determine the mooring tension that occurs on each mooring rope with a physical model test. This research was conducted on the type of pontoon and porous saw. In this study, it was found that on a porous saw type, the greatest maximum tension occurred at the mooring angle 30° (3.98 N) while the smallest maximum tension occurred at the mooring angle 60° (0.73 N). The pontoon type also gives the same result where the 30° mooring angle gives the highest maximum tension (4.91 N) while the 60° mooring gives the smallest maximum tension (1.27 N). Besides, the porous saw type of floating breakwaters can provide smaller mooring tension than the pontoon type. The porous saw type can reduce mooring tension by 10% up to 15% at an angle of 30° - 60° .

1 INTRODUCTION

In recent years, a lot of efforts have been made to manage and utilize the coastal areas of Indonesia. Such as industry, trade, transportation, residential, and the tourism sector. Thus, in line with population growth and the increase of socio-economic development activities, the value of coastal areas continues to grow. Coastal areas, despite having high economic value, are vulnerable to many threats. One of the serious threats faced by coastal areas is abrasion which can cause coastal area reduction, due to the large amount of wave energy that comes directly to the coastal area without the wave energy being absorbed beforehand.

Because it is considered important to avoid negative effects due to abrasion, it is very necessary to build and install a coastal protection structure that can reduce wave energy towards the coast area. One of the coastal protection structures that can protect the shore area from abrasion and erosion is the breakwater. A breakwater is a structure built to protect the area behind it from a wave attack. There are two types of breakwaters. Fixed breakwater and floating breakwater, that are built depending on the water depth and tides condition where the structure is

placed. Floating breakwater has several advantages than the fixed breakwater. Its efficiency is almost independent of tides and sea-level rise, its dependence on the environment is low, the impact of a floating breakwater to the environment is low, the cost of construction and decommissioning is low, the time required for installation is short, and it is possible to reset the module and/or to set the layout if there is a change in the future (Ruol et. al., 2012). However, since it is a floating structure, the anchor system is required which is placed at the bottom of the sea (Fousert, 2006). Figure 1 shows two types of floating breakwater schematically (Pianc, 1994).

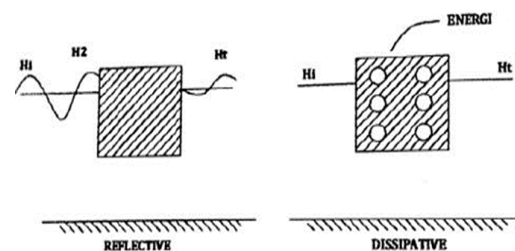


Figure 1: Two types of floating breakwater schematically.

There are so many types of floating breakwater, the most commonly used is the pontoon type which is connected and tethered to the seabed using a cable

or chain. A floating breakwater is a floating structure that requires a mooring system to keep the structure stay in place (McCartney, 1985). One of the aspects that must be considered when designing floating breakwater is the mooring system of the structure. Moorings, whether constructed of piles or mooring lines and anchors, must hold a breakwater in place and a careful assessment of mooring forces during design storm wave attacks must be made to ensure the survival of the breakwater. Waves loading generally dictates the design of floating breakwater and its mooring system (Tsinker and Gregory, 1995). In this study, a physical model test will be carried out to find the maximum tension value on several types of floating breakwater then examine by comparing type. Two types are being considered: porous saw type, and pontoon type. This test will be carried out by varying the water level as a simulation of tidal conditions. In the test, the mooring angle will also be varied by referring to previous research so that the optimum results are obtained.

2 MATERIALS AND METHODS

2.1 Research Scenario

The floating breakwater model is tied by a mooring rope with a mooring angle α at water level D and placed in wave characteristics under significant wave height H_s and period T (see Figure 2). These four environmental parameters are independent variables that will be varied. The effect monitored and recorded in this test is the mooring rope tension as the response variable.

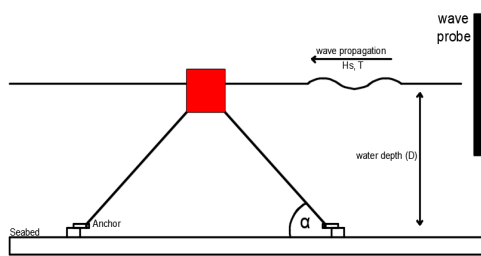


Figure 2: Layout sketch of floating breakwater physical model tests.

2.2 Floating Breakwater Design

The material used in the floating breakwater prototype is K300 concrete with a concrete density of 2300 kg/m³. The thickness used is 0.2 m. The porous saw type floating breakwater has the same basic

dimensions (length 10 m, height 3 m and width 3 m) and also use the same material as the pontoon type. There is a slight difference between the two models where the porous saw has a pore which is penetrated the structure from the front to the back. There is also a triangle shape lined up in the front of the structure.

In the porous saw type floating breakwater, pipes are used so that the front and rear sides of the breakwater are perforated. The pipe used has an outside diameter of 20 cm using High-Density Polyethylene (HDPE) material with a density of 960 kg/m³. The two breakwater types above float with a 2-meter draft. The pore in the porous saw type floating breakwater is take up to 5% area of the overall area in the side of the structure. The difference between these two models requires the thickness of the porous saw type floating breakwater to be 0.18 m or 0.02 m smaller to keep both drafts are the same. The reduction was caused by the loss of buoyant force due to a hole in the floating breakwater.

2.3 Test Modelling and Scaling

Tests are carried out in the laboratory so a model is needed from both the structure and the environmental conditions. in this test a 1:50 scale of length was used. Figure 3 and 4 show image of both type of floating breakwaters to be tested.

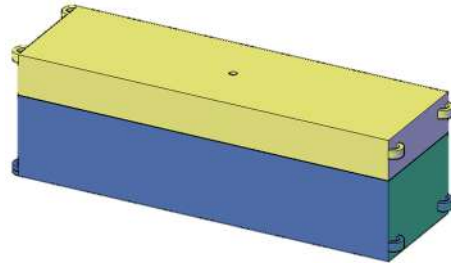


Figure 3: Floating breakwater type pontoon.

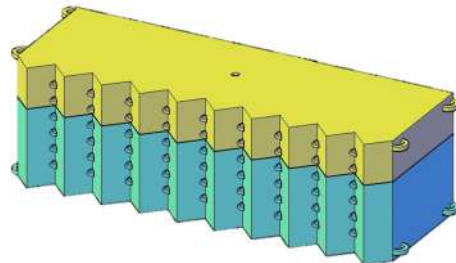


Figure 4: Floating breakwater type porous saw.

Table 1: The model scale of environmental condition tested (scale 1: 50 for D and Hs, 1 : $\sqrt{50}$ for T).

Parameter	Prototype	Model
Water level elevation (D)	21,5 m	41 cm
	21,5 m	43 cm
	22,5 m	45 cm
Wave height (Hs)	1,5 m	3 cm
	2 m	4 cm
	2,5 m	5 cm
Wave period (T)	7,8 s	1,1 s
	9,2 s	1,3 s
	11,3 s	1,6 s

The mooring angle (α) are at 30°, 45° and 60°.

2.4 Physical Model Tests Layout Setting

Several factors important for conducting the experiments include physical model design, the structure layout, and the measurement equipment used in Wave Flume Laboratory. These factors help researchers better understand the experiments. A Wave Probe is placed to record the wave height and wave period. In this test, an 35 cm artificial seabed was built at the bottom of the Wave Flume. This seabed used to adjust the designed water level scenario. Since the Wave Flume only works if the test was carried out at a water depth between 41 - 45 cm. Figure 5 depicts a sketch of the top and side view of the layout test.

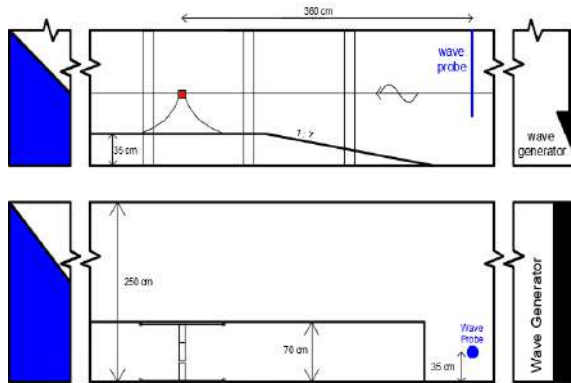


Figure 5: Model sketch layout of the test.

2.5 Equipment Calibration

The equipment to measure water elevation is Wave Probes, while Load Cells used to measure mooring tension. Both types of equipment are calibrated to see the equipment's condition whether it is still in good condition to be used in this test or not.

a. Wave Probe calibration

In general, the following steps required in the Wave Probe calibration process is explained below:

1. The Wave Probe is positioned where the tip of the Wave Probe touches the surface of the water. The voltage shown on the control computer will change slightly and the voltage will be recorded 3 times which will be averaged.
2. The Wave Probe then moved 20 cm deeper into the water. The voltage results are then recorded again 3 times and averaged. This process repeats until it reaches a depth of 80 cm from the tip of the probe. The illustration can be seen in Figure 6.

After the calibration process is carried out, the values obtained from each water depth are then arranged into a graph. Figure 7 shows the results of the Wave Probe calibration before and after the experiment. From the graph, it can be seen that there is no significant change and the value of the coefficient of determination is close to 1 (0.999) which indicates that the stability of the Wave Probe is still very good.

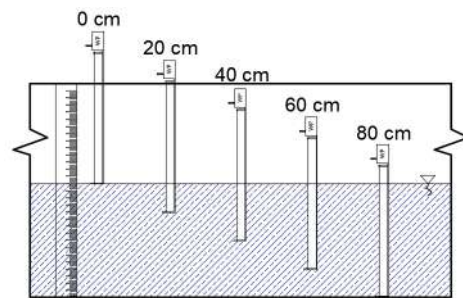


Figure 6: Wave Probe calibration process.

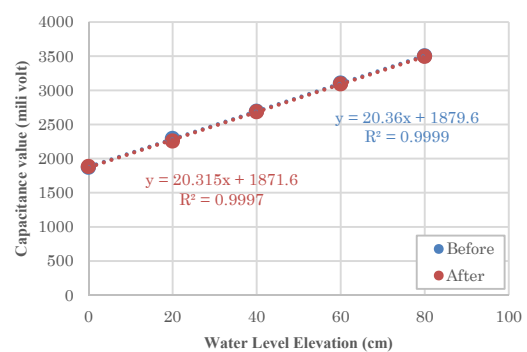


Figure 7: Wave Probe calibration graph.

b. Load Cell calibration

Four Load Cells with a capacity of 5 kg are being installed at the end of the bottom of the floating breakwater. Calibration is carried out by weighing

five different loads i.e. 1000 grams, 500 grams, 200 grams, 100 grams, and 50 grams. The load is weighed using a digital scale beforehand so the actual weight of each load used is already known. The recorded load show on the computer will be compared to the actual weight of each load. The Load Cell used in this experiment can be seen in Figure 8.



Figure 8: Load Cell with a capacity of 5 kg.

Calibration is done by changing the load on the Load Cell using a rope freely. Figure 9 shows the calibration results of each Load Cell used to validate the results of tension mooring recorded when the test takes place.

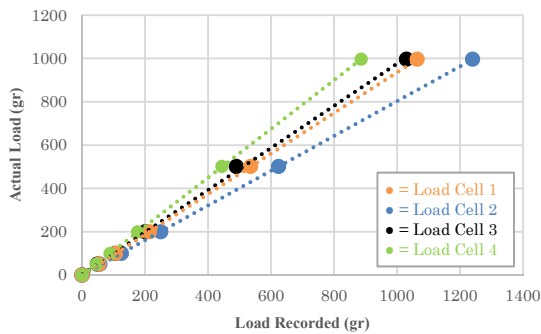


Figure 9: Load Cell calibration graph.

Figure 9 shows that each Load Cell has a different calibration value. Therefore, during data processing data, each Load Cell has a different regression equation. All regression equations (see Table 2) have the determination coefficient (R^2) really good (close to 1). These regression equations used as the correction factor for the load shown by the recorded computer.

Table 2: Load Cell calibration equation.

Load Cell	Calibration Equation	Determination Coefficient (R^2)
1	$y = 0,9369x - 0,4691$	1
2	$y = 0,8026x + 0,6541$	1
3	$y = 0,9728x + 3,6322$	0,9992
4	$y = 1,1252x + 0,2195$	1

3 RESULTS AND DISCUSSION

After all, preparations are completed, mooring tension data is collected every 2 minutes for each test, with an increment of 0.5 seconds. The recorded mooring tensions from each Load Cell recorded in four different columns then converted into kilograms (kg). The maximum tension was selected among data have been collected for analysis.

Preparation before analysis was conducted by grouping the data based on a combination between water level elevation and mooring angle. The total combination is 9 maximum tension with different H_s and T values. In this test the variables H_s and T are merged into one parameter, the parameter is known for wave steepness parameters (H_s/gT^2). Each group of data is then arranged into a graph that shows the relationship between mooring tension and wave steepness parameters, H_s/gT^2 . In addition, linear regression was also performed on each group of data to simplify the comparison between data groups.

3.1 The Effect of Mooring Angle

Based on the data recorded from the physical model test carried out with variations in the mooring angle, a various maximum mooring tension value was obtained indicating that there is the influence of the mooring angle on the mooring tension of the floating breakwater. Figure 10 shows the effect of the mooring angle used in this test on the maximum mooring angle on the floating breakwater type porous saw and pontoon at a water level elevation of 45 cm. It can be seen in Figure 10, the greatest maximum mooring tension occurs at the 30° mooring angle on both the floating breakwater type; porous saw, and pontoon. On a porous saw type, compared to an angle of 30° , an angle of 45° and 60° have a smaller maximum mooring tension value of 15% and 20% respectively. While the pontoon type, the angle of 45° and 60° have the smallest maximum mooring tension value of 34% and 45% respectively to the angle of 30° . At the water level elevation of 41 cm and 43 cm, the same phenomena happen where the greatest maximum mooring tension occurs at an angle of 30° and the smallest maximum mooring tension occurs at an angle of 60° .

Some researchers have examined how the effect of the mooring angle on mooring tension. Wei, et al. (2011), examined the effect of the mooring angle on the interaction between the floating breakwater and the wave, and one that was tested was the effect on the mooring force. From the test, it is known that there is an increasing mooring tension when the

mooring angle decreases from 90° to 0° especially when the mooring angle is below 30° . They added, even though the 60° angle gives the lowest mooring tension value, the motion of the structure is less stable. The results show that the sway, heave, and roll motions are high when the large angle is used, especially when the angle reaches 60° . The result of this study is not contradictory to the result obtained in Wei, et al. (2011).

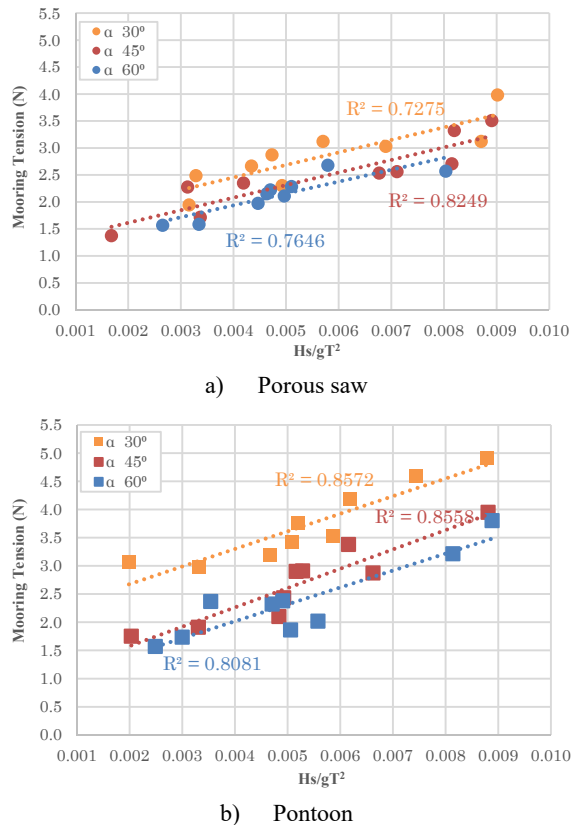


Figure 10: Mooring tension at water level 45 cm on floating breakwater type porous saw and pontoon.

3.2 Floating Breakwater Type Comparison

The testing of the floating breakwater mooring tension is carried out on two types of floating breakwater which are the porous saw type and the pontoon type so that there is a mooring tension difference experienced by the two types of floating breakwater tested. The comparison graph of the porous saw and pontoon type floating breakwater shows that a larger tension occurs on a pontoon type floating breakwater. Figure 11 shows a comparison of the two types of a floating breakwater at 45 cm water level elevation. It can be seen in figure 11 that

the pontoon type floating breakwater produces a greater mooring tension than the porous saw type. This proves that porosity addition can reduce the mooring tension value. The mooring tension reduction also varies from 10% up to 15%.

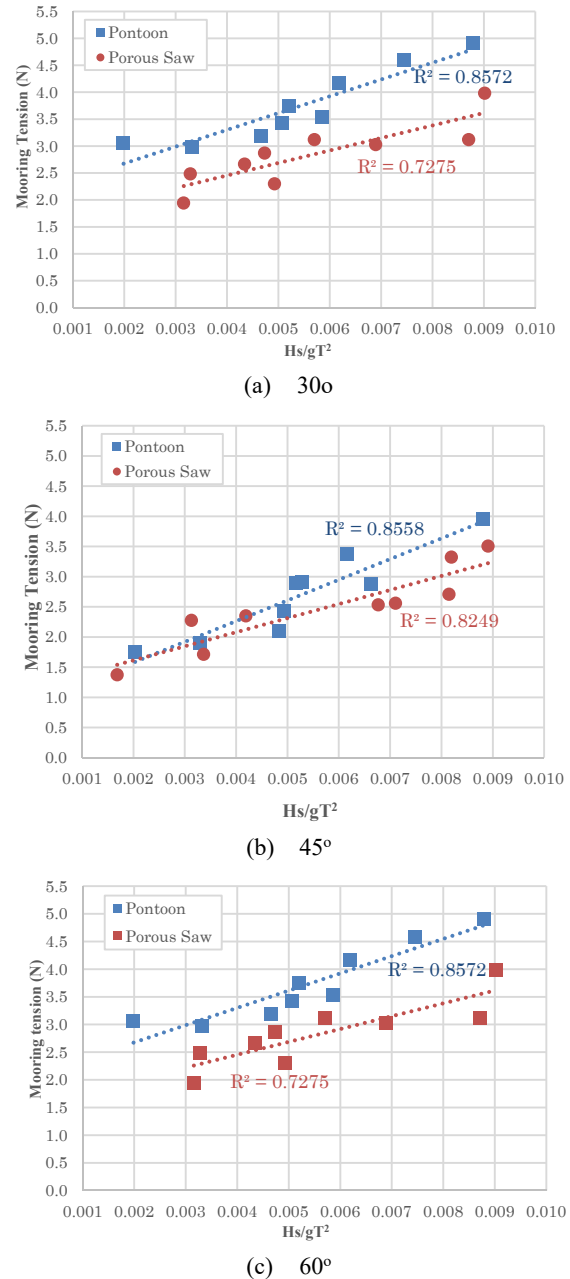


Figure 11: Comparison between both floating breakwater porous saw and pontoon types at various mooring angles.

There are some previous researchers also supported the results obtained in this study. One of them is Wilbur (1996), he carried out a study of perforated floating breakwaters and how they

affected various aspects of the floating breakwater. One of the aspects that were studied is the mooring tension and transmission coefficient. In terms of the transmission coefficients, it is found that solid floating breakwater has a smaller transmission coefficient value indicates that solid floating breakwater can reduce waves better. But in terms of the mooring tension produced in the mooring lines, both the mooring line on the seaward side and shoreward side produces a smaller mooring tension value on the perforated floating breakwater.

Wilbur (1996) says that when a wave hits the structure of a floating breakwater, some of the waves are reflected, some pass away, and some others enter through the pore that is in the structure and that, of course, reduces the impact received by the mooring system. Marks continued, if the floating breakwater used is a solid structure, the greater reflection will occur on the side of the floating breakwater wall with a high resultant force. In the case of a perforated floating breakwater, the wave force will be transmitted to the mooring rope and also some of the wave force will direct the floating breakwater to move (oscillating) so that it forms a new wave behind the structure. This is certainly one of the disadvantages of using a solid floating breakwater rather than the perforated floating breakwater. Research conducted by Wilbur was also proven in this study where the tension produced by a porous saw type floating breakwater is lower than that of a pontoon floating breakwater.

4 CONCLUSIONS

Based on this study, several conclusions can be drawn from the physical model tests of mooring tension on the porous saw type floating breakwater including:

1. Based on variations in the mooring angle and water level elevation, the following conclusions are obtained:
 - The greater the mooring angle, the smaller the mooring tension experienced by the mooring line.
 - Based on the variations of the mooring angle, 30°, 45°, and 60°, it can be seen that the greatest mooring tension occurs at an angle of 30° with mooring tension differences at 45° and 60° of 15% and 20% smaller than an angle of 30° respectively.

2. Floating breakwater type porous saw type gives a smaller mooring tension than the pontoon type. The porous saw type can reduce mooring tension by 10% up to 15% at an angle of 30° - 60°.

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Implementation of Lean Project Management in Offshore Pipeline Installation Project

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Keywords: Lean Project Management, Waste, Critical Chain, Fuzzy FMEA, Offshore Pipelines Installation.

Abstract: Project implementation is inseparable from the risks that can affect the project will go according to plan or not. Therefore, good project management is very important to avoid project problems such as delays. Project delays can be caused by unproductive activities and the elements involved. The un-productibility becomes something that does not have added value or in Lean Project Management known as Non-Value-Added Activities (waste). In the implementation of the Offshore Pipeline Installation Project undertaken by one of the oil and gas companies in Indonesia, there was a delay in the process, therefore a risk analysis of the causes of the delay is required with the Lean Project Management approach to identify waste by applying the Fishbone Analysis method and project risk analysis using the Fuzzy Failure Mode and Effect Analysis method that will prevent or detect the failure mode earlier. And to estimate the project, it is applied by using the Critical Chain Project Management (CCPM) method which will produce project optimization to meet the Lean Project Management criteria, namely eliminating invaluable activities in order to make the project more effective and efficient.

1 INTRODUCTION

The oil and gas industry have an important role in the mineral resources needed by the state for its people. With abundant oil and natural gas reserves in Indonesia, facilities are needed to support these natural resources. One of the infrastructures that are widely used for natural resources is using subsea pipelines. Therefore, this company carried out the offshore pipeline installation project in order to maintain the company's basic production and to obtain additional production. This project is basically to install a new pipeline and to replace the damaged pipeline.

In its implementation, the Offshore Pipeline installation project undertaken by this company was delayed, but in the end, the project could be completed on time. Seeing the various things that happened and the number of methods used in project planning, this paper will identify an offshore pipeline development project planning with the concept of Lean Project Management. This paper discusses the risks of pipeline project delay and optimize it. Many methods can be used to analyse the delay of a project, one of the more using fishbone diagram which functions as a method for analysing the causes of project delays that are divided into several causal

indicators, namely machine, material, and method workers (Khotimah, 2019). By using the fishbone method to analyse project waste and using the CCPM method to plan schedules to avoid Student Syndrome and Parkinson's Law Effects, it allows the writer to optimize and streamline project planning to be more effective and efficient.

2 METHODOLOGY

The flow work in this study conducted by stages based on the following steps. The first step is data and reference materials collection from textbooks, journals, previous research etc. During this stage a comprehension study of Lean Project Management, Critical Chain Project Management are also conducted for optimization project.

2.1 Data Collection

The flow and procedure of the research were carried out in the following stages. This stage includes efforts in understanding offshore pipeline studies, the use of the CCPM method and the collection of data needed to complete this paper. The data required include:

- Details of project activities based on existing schedule in accordance with project contract.
- Data analysis of the main problems of the project.

2.2 Identify Waste using Fishbone Diagrams

Identification of waste is done by using fishbone diagrams made from project's data analysis of the major problems and then discussed with expert judgment. Expert judgment is a major source of information that can provide vital input to project managers, who must ensure that projects are completed successfully, on time, and on budget (Szwed, 2016).

2.3 Managing Variation for Optimization of Project Scheduling using the CCPM Method

This scheduling begins by creating a network in accordance with project data that has been obtained. Then determine where the critical path is located. Critical Chain Project Management (CCPM) is a TOC tool used for planning and project management. It can be used both in one-project and multi-project structures where resources are being used in several projects simultaneously (Izmailov, 2016).

2.4 Risk Control

In this risk control precautions are taken to reduce the impact of the occurrence of risk variables that are in the red zone, while in the green zone risk control is not carried out because the impact can still be tolerated. Efforts to minimize this risk are carried out by implementing steps directed at the results of the assessment of data obtained from the risk analysis process. This is done by developing options and determining actions to increase opportunities and reduce threats to project objectives. Risk analysis have been used in many areas namely risk analysis for gas platform (Silvianita et al 2016).

3 RESULTS

3.1 Project Overview

Projects are one-time activities, with limited time and resources to achieve predetermined outcomes, for example products or production facilities (Soeharto, 1999). This project requires to maintain the

company's production base and obtain additional production. This project is basically to install new pipes and to replace pipes that are already damaged. The project is located in the working area of one of the oil and gas industries which is located about 80 - 200 km northeast of Jakarta. The replacement pipe that will be replaced is 8 "NPS Gas Line from Platform X to Platform Y.

Table 1: Pipe Description.

Year 2018				
No	Pipeline Name	Size (inch)	Length (Km)	Repair Strategy
1.	B – A	12"	1	Full
2.	E – C	10"	7.2	Full
3.	D – E	8"	7.2	Full

3.2 Data Collection

In this stage, data collection was obtained from the results of Major Problem reports that occurred in the project. Then the data is validated with interviews with an expert that related to the causes of waste in this study which is Offshore Pipeline Installation project.

3.3 Work Breakdown Structure

Work Breakdown Structure shows overall project activities that used as a basis for determining volume of work, duration of activity and also used as a scheduling guide. Below is the table of activities on Offshore Pipeline Installation project as follows:

Table 2: Work Breakdown Structure of Offshore Pipelines Installation Project.

Activity	Duration (Days)
Basic Engineering	210
<i>Project Management Team (PMT)</i>	129
Detail Engineering	113
Coated Pipe 10"	251
Pipe Bend	250
Crossing Material	274
Riser Clamp dan Riser Fabrication	49
Survey	8
Preparation	79
Pipeline Laying	8
Mobilization	26
Riser Installation	14
Instalasi Crossing	14
Precommissioning	8
Certification	33
Final Acceptance	22

3.4 Project System

A project that is experiencing delays is certainly due to factors that influence waste (Hapsari, 2014). A project that is experiencing delays is certainly due to factors that influence waste. Waste in the project is actually something that must be considered in the sustainability of the project. Project system have been carried out by understanding the project and discussions with the implementer to identify the waste that is likely to arise in project implementation.

3.5 Waste Identification using Fishbone Diagrams

Lean is an often-used adjective in business these days, but there's some confusion over its exact definition. In essence, the goal of Lean is to maximize value while minimizing waste (Eby, 2017). Identification of waste or known as Non-Value-Added activity aims to determine the waste that occurs in the project. The identification process is done by using fishbone diagrams and *If-Then* formulations so that the project implementer can take appropriate corrective and preventive actions for future projects where there is the potential for waste to occur so that excessive waste can be avoided. Based on fishbone diagrams below, it can be seen the types of waste that most often appear in Offshore Pipeline installation projects from 8 waste that have been defined by Womack and Jones (1996) are Waiting and Unnecessary Motion.

Waiting, which is a condition where project activities have been delayed so that the delayed activity can result in delays in project implementation.

Unnecessary Motion, which is the movement that is not necessary, where the movement of unproductive workers that should not need to be done in the implementation of the project.

From Figure 1, the Fishbone Diagram above can be seen as the causes of waste generation in the project. It contains 9 failure modes and 20 causes that have been classified based on factors causing waste. Those are Man, Material, Method, also Tools and Machine. The causes of waste then processed into *If-Then* formulation to further know possible actions that implementer can take to minimize waste with preventive or corrective steps.

3.6 Managing Variations

The variations contained in the project is considered as uncertainty, therefore the project executor needs to adjust the existing variations by estimating. In this

project, the variation made estimation is time. The aim is that the project executor can estimate the time needed during implementation.

Identification of waste in the project system project analysis found that the most influential waste in the project is waiting and unnecessary motion. By knowing the waste, then the scheduling improvement recommendations will be made with the Critical Chain Project Management (CCPM) as follows.

3.7 Critical Chain Project Management

All projects are carried out in a dynamic environment. This is an inherent characteristic of project activities which duration cannot be estimated precisely. Estimate Activity Durations is the process of estimating the number of work periods needed to complete individual activities (PMBOK, 2013). Among other factors, the accuracy of the estimation depends on the level of uncertainty. Variation problems must be addressed early in the development schedule. Safety time must be included in the project and must ensure its robustness. Time analysis have been used in many areas such as in graving dock project (Silvianita et al, 2018).

Table 3: Work Breakdown Structure with CCPM.

Pipeline Installation	
Summary Task	CCPM Duration (Days)
Basic Engineering	91
<i>Project Management Team (PMT)</i>	105
Coated Pipe 10"	126
Pipe Bend	125
Crossing Material	137
Riser Clamp dan Riser Fabrication	25
Survey	4
Preparation	40
Pipeline Laying	7
Mobilization	4
Riser Installation	13
Crossing Installation	7
Precommissioning	4
Certification	16
Final Acceptance	11

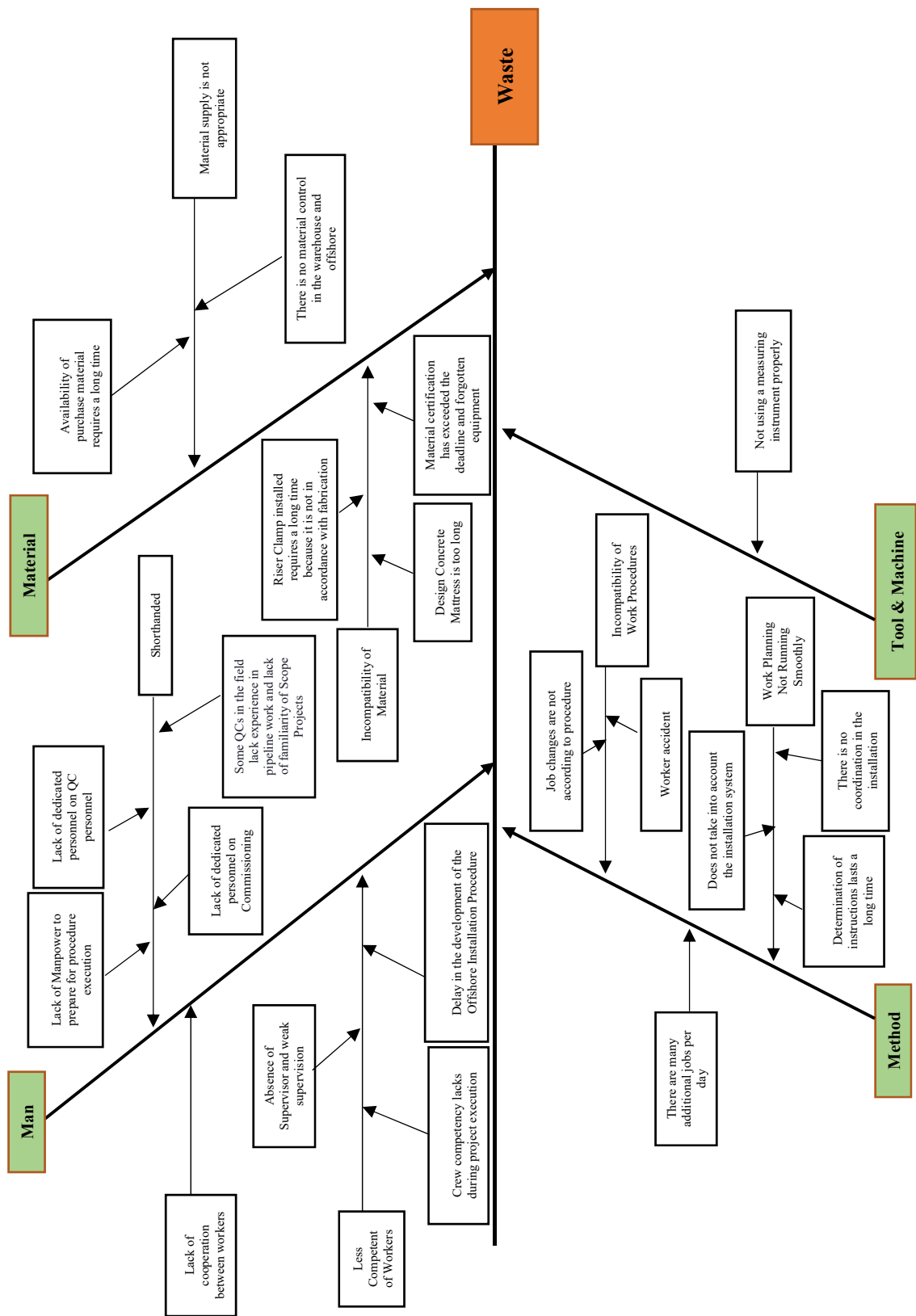


Figure 1: Fishbone Diagram of Waste.

However, the addition of time will increase the overall project duration, so that the lowest possible amount of safety time must be added to the schedule. CCPM controls several problems by including buffers in the project schedule (Leach, 2000). In this case, the CCPM adds a safety factor to the activity by solving several factors that impact human actions in the project environment. By applying the CCPM method to project scheduling, it is expected that problems that occur in the project, such as student syndrome and Parkinson's law effects can be avoided.

3.8 Network Planning

In the initial stage, the Work Breakdown Structure (WBS) is defined as a predecessor and successor between activities.

Table 4: Network Activities.

Activity	Predecessor	Successor
Basic Engineering	START	A3,B2,B3,C1
Project Management Team (PMT)	C1	D2
Detail Engineering	A1,C1	C3
Coated Pipe 10"	START	B4
Pipe Bend	A1	B4
Crossing Material	A1	C4
Riser Clamp dan Riser Fabrication	B1,B2	C5
Survey	A1	A2,A3,C2
Preparation	C1	C4
Pipeline Laying	A3	D1
Mobilization	B3,C2	C6,C7,D1
Riser Installation	B4	C6,C7
Instalasi Crossing	C4,C5	D1
Precommissioning	C4,C5	D1
Certification	C5,C6,C7	D2
Final Acceptance	A1,D1	END

3.9 Buffer Management Analysis

This analysis is used to monitor the schedule that already exists in Network Planning when the project is executed. The application of the CCPM method only requires supervision on the Project Buffer, in contrast to some other scheduling methods that must be monitored throughout the entire project activity. This Buffer Management is useful for maintaining the

reliability of the project schedule but does not change the critical trajectories that exist.

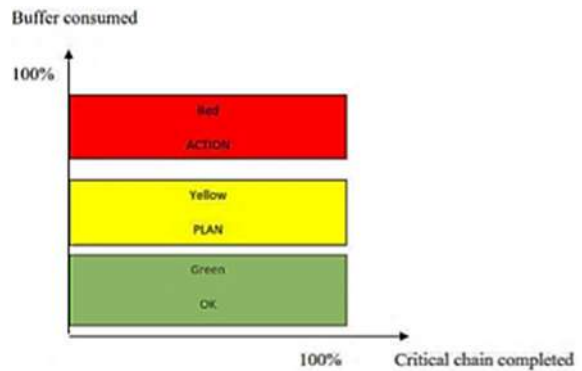


Figure 2: Buffer Distribution Area (Valikoniene, 2014).

Buffer Size =

$$2 \times \sqrt{\left(\frac{S_1 - A_1}{2}\right)^2 + \left(\frac{S_2 - A_2}{2}\right)^2 + \dots + \left(\frac{S_n - A_n}{2}\right)^2} \quad (\text{Eby, 2017})$$

Based on calculations and analysis, it has been found that the amount of project buffer is 82.4 days. From these results, will be divided equally based on buffer distribution area.

Table 5: Buffer Use.

Buffer Region	Range	Project Buffer (Days)	Used Duration (Days)
Green	0 % - 33 %	82	< 27,06
Yellow	34 % - 67 %	82	27,06 - 54,94
Red	68 % - 100 %	82	> 54,94

4 CONCLUSIONS

Based on the result of the analysis, it can be concluded that the delay of the offshore pipeline installation project was caused by 9 failure modes and 20 causes. And non-value-added activities or waste that arises in offshore pipeline installation projects based on 8 types of waste are waiting and inappropriate processing. Activities that do not added a value or waste that appears in offshore pipeline installation projects based on 7 types of waste are waiting and unnecessary motion. The application of the CCPM method, the acceleration of work time in

scheduling is 183 days and when the project buffer has been used for more than 59 days or has entered the red zone, the project implementer must take action.

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Risk Analysis on Offshore Pipeline Installation Delay: A Case Study for Mahakam Block, Indonesia

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Keywords: Fuzzy Fault Tree Analysis, Fuzzy Event Tree Analysis, Bowtie Analysis, Project Delay, Offshore Pipeline Installation.

Abstract: Pipeline has an important role in oil and gas industry to transfer oil or gas that have been discovered to production station, one of which is pipeline system belong to one of the companies located in Mahakam Block, East Borneo, Indonesia. Considering its important role, pipeline installation in not allowed for any delay and needs to operate as planned. However, the said offshore pipeline installation has experienced 21 days of delay, whereby this project started on December 5th 2018 and finished at February 3rd 2019. To analyse the delay of this project, this research uses two methods, namely fuzzy trapezoidal and bowtie analysis. The fuzzy fault tree analysis (FTA) diagram will generate the probability value of top event project delay of offshore pipeline installation. The fuzzy event tree analysis (ETA) diagram will generate how much fine that the contractor has to pay and the risk level from every factor that causes the delay. The results from bowtie analysis will determine preventive value which will be used as precaution and mitigation which will be used as reduction due to delays.

1 INTRODUCTION

In this modern era, oil and gas demands continue to rise, to meet consumer demands oil and gas companies have to increase their production performance. In this industry, pipeline has an important role to transfer crude oil that has been discovered from well to production station.

This research discuss the project of offshore pipeline installation belongs to one of the oil and gas company in Mahakam. This pipeline located in Mahakam Block, East Borneo, Indonesia. This pipeline is connecting two wells at a distance of 0.8km. which the installation shall be commenced from 5th December 2018 to 13th January 2019. during the execution, this project delayed for 21 days causing the project to be completed on 3rd February 2019 instead.

Delays on projects can cause losses to both the owner and the contractor. From ownerpoint of view, the construction delay eventually will impact on the progress of the production, in a way affecting the profit. From contractor point of view, extra cost will

incurred e.g. on equipment rent and fines (Muhamad, 2016).

To reduce the possibility of delays on offshore pipeline installation in the future, risk analysis can be used. Project delay for jacket structure and HRSG have been reviewed by (Silvianita et al 2017a, 2017b) and another project delay using Fault Tree Analysis (Silvianita et al 2014). This paper will examine the most dominant factor causing the delay on offshore pipeline installation project at the company. Pipeline specification data can be seen in Table 1.

Table 1: Pipeline Specification Data.

Description	Specification
Pipe Material	API 5L X 65
Outside Diameter	8.625 in
Line Pipe	12 m
Pipeline Wall Thickness	20.6 mm
Pipe Length	800m

2 LITERATURE STUDIES

Fault tree analysis (FTA) is a logic and graphical representation that explores the interrelationships between a potential critical events in a system and the reason for event (Mokhtari et al, 2011). FTA diagram consists of top event and basic events. Commonly, FTA method is used to find the probability of top event.

Meanwhile, Event tree analysis (ETA) is a method to detect and analyze the different events of pragmatic accidents possibilities with safety features following an initiating event (Raiyan et al, 2017). ETA diagram consists of initiating event and pivotal events, which normally used to analyze consequence that arise from failure or unwanted event.

To covers the loss event scenario uses a Bowtie diagram that consists of fault tree (FT) and Event Tree (ET) to identify the causes of top events or loss events and shows the consequences of unwanted events. The weakness of bowtie analysis is that the opinions used in the analysis have uncertainties and vaguenesses.

FTA and ETA considered probability value to determine cost of delaying fortotop event of FTA and initiating event of ETA respectively. But in the execution, these methods will produce unrealistic outcome as the probability values are solely depend on assumptions and lead to erroneous conclusion.

Fuzzy logic was introduced to overcome the ambiguity of human judgement as it can change the probability value to possibility number within the scale of 0-1. According to Aqlan et al (2014), fuzzy logic is used to find aggregates or the value of the word variable (linguistics) of the respondent, which are converted into a collection of numbers. The scale used in fuzzy logic is more flexible, making it easier to assess linguistic variables according to condition

Table 2: Fuzzy Likelihood of an Event (Zarei et al, 2019).

Grade	Likelihood	Membership Function
1	Very High (VH)	(0.8,1,1,1)
2	High-Very High (HVH)	(0.7,0.9,1,1)
3	High (H)	(0.6,0.8,0.8,1)
4	Fairly High (FH)	(0.5,0.65,0.65,0.8)
5	Medium (M)	(0.3,0.5,0.5,0.7)
6	Fairly Low (FL)	(0.2,0.35,0.35,0.5)
7	Low (L)	(0,0.2,0.2,0.4)
8	Low-Very Low (LVL)	(0,0,0.1,0.3)
9	Very Low (VL)	(0,0,0,0.2)

in the field (Shahriar, 2012). In this research, the use of linguistic variables and fuzzy numbers are as follow (Zarei et al, 2019).

This research using the trapezoidal fuzzy number to determine the possibility value from respondents assumptions based on interview. Trapezoidal fuzzy number can be defined as:

$$\mu_{\tilde{A}}(x) = \begin{cases} 0, & x < a_1 \\ \frac{x-a_1}{a_2-a_1}, & a_1 \leq x \leq a_2 \\ 1, & a_2 \leq x \leq a_3 \\ \frac{a_4-x}{a_4-a_3}, & a_3 \leq x \leq a_4 \\ 0, & x > a_4 \end{cases} \quad (1)$$

The formula can be defined as follows :

$$X^* = \frac{\int_{a_1}^{a_2} \frac{x-a_1}{x-a_2} x dx + \int_{a_2}^{a_3} x dx + \int_{a_3}^{a_4} \frac{a_4-x}{a_4-a_3} x dx}{\int_{a_1}^{a_2} \frac{x-a_1}{a_2-a_1} dx + \int_{a_2}^{a_3} dx + \int_{a_3}^{a_4} \frac{a_4-x}{a_4-a_3} dx} \quad (2)$$

$$X^* = \frac{1}{3} \times \frac{(a_4 + a_3)^2 - a_4 a_3 - (a_1 + a_2)^2 + a_1 a_2}{(a_4 + a_3 - a_1 - a_2)}$$

According to Clemen et al (1999) to change the linguistic variable to fuzzy number and combine the value of experts assumption to one fuzzy number, the formula can be defined as follows:

$$M_i = \sum_{j=1}^m W_j A_{ij}, j = 1, 2, \dots, n \quad (3)$$

Where :

A_{ij} = Linguistic expression of basic event by j

M = Number of basic events

n = Number of experts

W = Weighting factor of j and M

Representing the combined Fuzzy number of basic event i.

According to Lavasani et al (2012) the value of experts assumption can be determined by this following formula :

$$\text{Experts value} = PP + ET + EL + A \quad (4)$$

$$W_j = \frac{\text{value of expert}}{\sum_{i=1}^n \text{value of expert}} \quad (5)$$

Where:

PP = Professional Position

ET = Education Level

EL = Experience Time (year)

A = Age (year)

W_j = weighting factor of experts

Table 3 shows the value of experts (Ramzali et al, 2015).

Table 3: Value for Experts.

Constitution	Classification	Score	Constitution	Classification	Score
Professional Position	Senior academic	5	Education Level	PhD	5
	Junior academic	4		Master	4
	Engineer	3		Bachelor	3
	Technician	2		Higher National Diploma (HND)	2
	Worker	1		School Level	1
Experience Time (year)	≥ 30	5	Age (year)	≥ 50	4
	20-29	4		40-49	3
	10-19	3		30-39	2
	6-9	2		< 30	1
	≤ 5	1			

According to Onisawa (1998) to change the fuzzy possibility score (FPs) by summing the three fuzzy number parameters and the dividing by three. For the last step, change the FPS to fuzzy probability score (FPr) using the following formula:

$$FPr = \begin{cases} \frac{1}{10^k} & \text{if } FPs \neq 0 \\ 0 & \text{if } FPs = 0 \end{cases} \quad K = \left[\left(\frac{1 - FPs}{FPs} \right) \right]^{\frac{1}{3}} \times 2.301 \quad (6)$$

Where :

FPr = Fuzzy Probability Score

FPs = Fuzzy Posibilatas Score

K = Constant Numbers

The probability will be used to calculate the possibility on ETA fuzzy diagrams. To change the probability to possibility by using the following formula (Onisawa, 1988):

$$FPs = f(FPr) = \begin{cases} \frac{1}{1 + \left(k \times \log \left(\frac{1}{FPr} \right) \right)^3} & FPr \neq 0 \\ 0 & FPr = 0 \end{cases} \quad (7)$$

Where :

FPr = Fuzzy Probability Score

FPs = Fuzzy Posibilatas Score

$$k = \frac{1}{\log(1/(5 \times 10^{-3}))}$$

3 METHODOLOGY

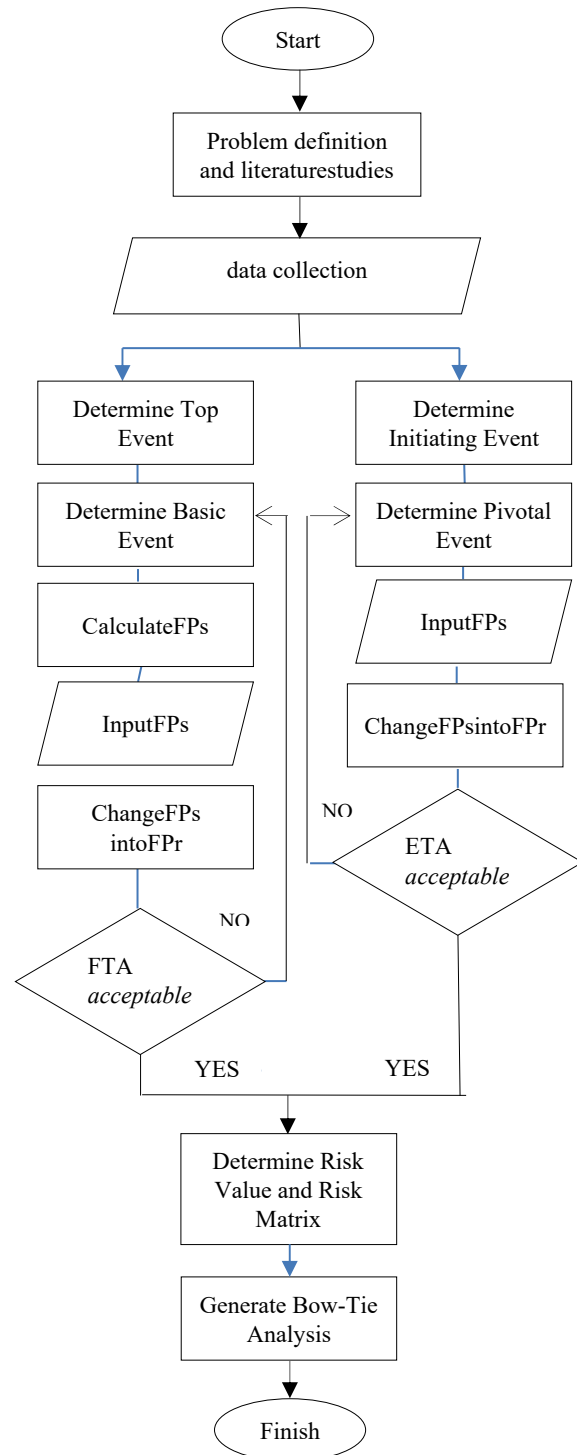


Figure 1: Flowchart.

The procedures of the research are as follow:

1. Formulations of Problems

The formulations of the problems are decided as the purpose or the goal of the research.

2. Literature Studies

3. Data Collection

The data used in this research are:

- Pipeline installation project master schedule data
- Actual pipeline installation project data schedule
- Contract data
- General data about the pipeline installation project
- Make a questionnaire to look for the probability of basic FTA events, pivotal ETA events, determination of risk matrices, barriers and escalation factors in a bow-tie diagram.

4. Data Analysis and Discussion

- Analysis of the results of the interview to determine the work items that are experiencing problems, the factors that cause delays in the project and the consequences caused by delays. At this stage a weighting is carried out for each respondent with equations 4 and 5
- Fuzzy FTA stage
- Fuzzy ETA stage
- Determine the Risk Matrix
- Bowtie Analysis

5. Conclusions and Suggestions

After all the results are obtained, conclusions are then drawn up containing the answers to the formulation of the existing problems and suggestions for further research, especially in the field of Risk Assessment.

4 RESULT AND DISCUSSION

4.1 Data Management Factors Delaying Offshore Pipeline Installation using Fault Tree Analysis and Fuzzy Logic

Fault tree analysis (FTA) is a method used to find the root causes of a top event or peak failure event. This research explained the factors that cause the delay in the offshore pipeline installation project.

The basic causes of delays and the probability of each basic event were illustrated through a fault tree diagram. The basic events can be seen in Table 4.

Table 4: Basic Event of FTA Diagram.

No.	Code	Name of Event
1	A111	Equipment / machinery not available
2	A112	Equipment / machinery is being used in another location
3	A121	Poor equipment / machinery maintenance
		
25	B11	Miscommunication between owner supervisor and contractor
26	B12	Workers do not understand the work items to do
27	B2	The process is not according to the initial plan

Meanwhile, the respondent's personal data can be seen in Table 5, and the respondents involved in the offshore pipeline installation were selected:

Table 5: Respondent Profile.

Expert	Category			
	Age	Experiences	Position	Education
1	34	12	Site Planner	S2
2	29	11	Engineer	S1
3	40	17	Supervisor	S1
4	26	5	Engineer	S1
5	29	6	Engineer	S1
6	32	8	Site Manager	S1
7	26	5	Engineer	S1
8	39	12	Supervisor	S2
9	28	5	Engineer	S1

Probability of each basic FTA was obtained through interviewing the respondents. the frequency of basic fuzzy FTA events was referred to the scale proposed by Zarei, et al. (2019), as listed in Table 6.

Table 6: Scale Fuzzy Likelihood of an Event.

Linguistic Variabel	Defenition	Membership Function
Very High (VH)	It happened every pipeline installation	(0.8,1,1,1)
High-Very High (HVH)	It happened in a span of 3 times the pipeline installation	(0.7,0.9,1,1)
High (H)	It happened in a span of 5 times the pipeline installation	(0.6,0.8,0.8,1)
Fairly High (FH)	It happened in a span of 15 times the pipeline installation	(0.5,0.65,0.65,0.8)
Medium (M)	It happened in a span of 25 times the pipeline installation	(0.3,0.5,0.5,0.7)
Fairly Low (FL)	It happened in a span of 50 times the pipeline installation	(0.2,0.35,0.35,0.5)
Low (L)	It happened in a span of 75 times the pipeline installation	(0,0.2,0.2,0.4)
Low-Very Low (LVL)	It happened in a span of 90 times the pipeline installation	(0,0,0.1,0.3)
Very Low (VL)	It happened in a span of 100 times the pipeline installation	(0,0,0,0.2)

4.1.1 Calculating Fuzzy Possibility (FPs)

From the results of interviews and questionnaires, a recapitulation of data for linguistic variable scales were compiled in Table 7.

Table 7: Questionnaire Results from Experts.

Activity	Category								
	1	2	3	4	5	6	7	8	9
A111	VL	VL	H	LVL	LVL	VL	L	M	L
A112	L	VL	HVH	L	FL	M	L	M	M
A121	L	LVL	VH	LVL	FL	L	LVL	M	LVL
.....									
B11	LVL	LVL	FL	LVL	M	VL	LVL	H	FL
B12	LVL	FH	FL	VL	L	L	VL	M	LVL
B2	LVL	FH	L	M	M	L	VL	M	FL

To determine the possibility of each basic event, the first step to identify the weight for each

respondent adopting equations 4 and 5, while the scores for respondents based on Table 3.

The example of value calculating of aexpert:

$$\begin{aligned}\text{Value of expert} &= PP + ET + EL + A \\ &= 3 + 3 + 4 + 2 \\ &= 12\end{aligned}$$

$$W_j = \frac{\text{weight of experts}}{\sum_{i=1}^n \text{weight of experts}}$$

$$W_j = \frac{12}{12+10+12+8+10+11+8+12+8}$$

$$W_j = \frac{12}{91}$$

$$W_j = 0,13$$

From the calculations above, we get the value of each respondent as written in Table 8.

Table 8: Value of Each Experts.

Respondent	Final Score
1	0.13
2	0.11
3	0.13
4	0.09
5	0.11
6	0.12
7	0.09
8	0.13
9	0.09

After knowing the weight of each respondent, then determined possibilities (FPs) of each basic event using equation 2 and 3.

We used Basic Event code A111 from the questionnaire results as an example to calculate the possibility (FPS).

Table 9: Questionnaire Results A111.

Experts									
1	2	3	4	5	6	7	8	9	
VL	VL	H	LVL	LVL	VL	L	M	L	

Table 9 shows the outcome of questionnaire survey. Then fuzzy numbers as in table 6 were converted to calculate the possibility (FPs). The results can be seen as below:

Expert 1 = (0, 0, 0, 0.2)

Expert 2 = (0, 0, 0, 0.2)

Expert 3 = (0.6, 0.8, 0.8, 1)

Expert 4 = (0, 0, 0.1, 0.3)

Expert 5 = (0, 0, 0.1, 0.3)

Expert 6 = (0, 0, 0, 0.2)

Expert 7 = (0, 0.2, 0.2, 0.4)

Expert 8 = (0.3, 0.5, 0.5, 0.7)

Expert 9 = (0, 0.2, 0.2, 0.4)

using equation 3, by multiplying the following fuzzy number with the weight of each respondent defined in Table 8. The following results were derived:

$$\begin{aligned}
 \text{Expert 1} &= (0, 0, 0, 0.2) \times 0.13 \\
 &= (0, 0, 0, 0.026) \\
 \text{Expert 2} &= (0, 0, 0, 0.2) \times 0.11 \\
 &= (0, 0, 0, 0.022) \\
 \text{Expert 3} &= (0.6, 0.8, 0.8, 1) \times 0.13 \\
 &= (0.078, 0.104, 0.104, 0.13) \\
 \text{Expert 4} &= (0, 0, 0.1, 0.3) \times 0.09 \\
 &= (0, 0, 0.009, 0.027) \\
 \text{Expert 5} &= (0, 0, 0.1, 0.3) \times 0.11 \\
 &= (0, 0, 0.011, 0.033) \\
 \text{Expert 6} &= (0, 0, 0, 0.2) \times 0.12 \\
 &= (0, 0, 0, 0.024) \\
 \text{Expert 7} &= (0, 0.2, 0.2, 0.4) \times 0.09 \\
 &= (0, 0.018, 0.018, 0.036) \\
 \text{Expert 8} &= (0.3, 0.5, 0.5, 0.7) \times 0.13 \\
 &= (0.039, 0.065, 0.065, 0.091) \\
 \text{Expert 9} &= (0, 0.2, 0.2, 0.4) \times 0.09 \\
 &= (0, 0.018, 0.018, 0.036)
 \end{aligned}$$

The fuzzy numbers is then summed, to obtain the ultimate fuzzy number, as follows:

$$\begin{aligned}
 M &= R1 + R2 + R3 + R4 + R5 + R6 + \\
 &\quad R7 + R8 + R9 \\
 &= (0 + 0 + 0.078 + 0 + 0 + 0 + 0 + 0.039 + 0 ; \\
 &\quad 0 + 0 + 0.104 + 0 + 0 + 0 + 0.018 + 0.065 \\
 &\quad + 0.018 ; 0 + 0 + 0.104 + 0.009 + 0.011 + 0 \\
 &\quad + 0.018 + 0.065 + 0.018 ; 0.026 + 0.022 + \\
 &\quad 0.130 + 0.027 + 0.033 + 0.024 + 0.036 + \\
 &\quad 0.091 + 0.036) \\
 &= (0.12 ; 0.21 ; 0.23 ; 0.43)
 \end{aligned}$$

Considering Eq (2), the Fuzzy Probability Score for event A111 can be obtained as follows:

$$\begin{aligned}
 \text{FPs} &= \frac{1}{3} \times \frac{(0.43+0.23)^2 - (0.43 \times 0.23) - (0.12+0.21)^2 + (0.12 \times 0.21)}{(0.43+0.21-0.12-0.21)} \\
 &= \frac{1}{3} \times \frac{0.25}{0.33} \\
 &= 0.25
 \end{aligned}$$

Similar procedure was repeated for the rest of the events and the results are as shown in Table 10.

Table 10: Calculation Results of Fuzzy Possibility (FPs).

No	Code	Fuzzy Number				FPs
		a	b	c	d	
1	A111	0.12	0.21	0.23	0.43	0.25
2	A112	0.22	0.39	0.40	0.57	0.39
3	A121	0.17	0.29	0.32	0.49	0.32
....	...					
25	B11	0.16	0.24	0.28	0.47	0.29
26	B12	0.12	0.23	0.25	0.44	0.27
27	B2	0.17	0.32	0.33	0.52	0.34

After finding the FPs of each basic event from the Fuzzy FTA Diagram, were then converted the possibility (FPs) to probability (FPr) using equation 6. The results of FPr are shown in Table 11.

Table 11: Calculation Results of Fuzzy Probability (FPr).

No.	Code	FPs	K	FPr
1	A111	0.25	3.30	0.0005
2	A112	0.39	2.66	0.0022
3	A121	0.32	2.,95	0.0011
....	...			
25	B11	0.29	3.09	0.0008
26	B12	0.27	3.23	0.0006
27	B2	0.,34	2.88	0.0013

The next step is calculating the minimum cut set by input the Fuzzy probability from each basic event to Top Event FTA software.

Calculations using the software will produce the Top Event Probability and Intermediate Event Probability values shown in Figure 2.

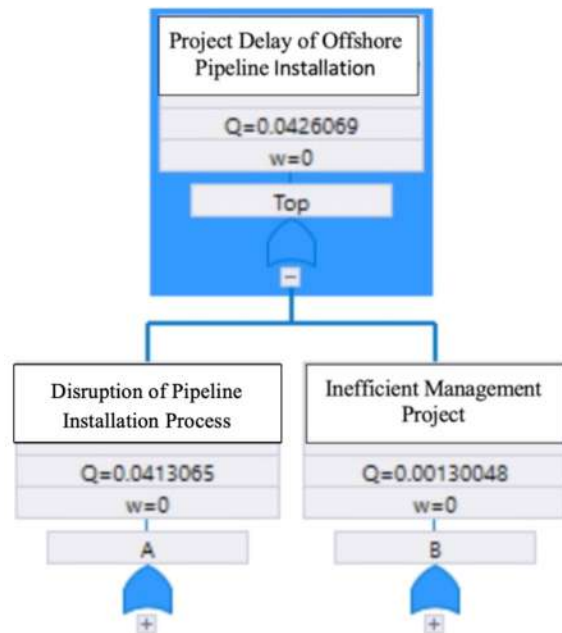


Figure 2: Minimum cut set results from Fuzzy FTA.

Figure 2 showed the minimum cut sets of fuzzy fault tree analysis (FFTA). The minimum cut set for project delay of offshore pipeline installation are due to disruption of pipeline installation process with a probability 0.04135065 and inefficient management project has a probability of 0.00130048. So the minimum total cut set for the top event is 0.0426069.

5 CONCLUSION

Based on discussion in the previous chapter, it can be conclude that the most dominant factor causing delays in offshore pipeline installation projects is Disruption of Pipeline Installation Process with a probability of 0.0413 and Inefficient Management Project with a probability of 0.0013.

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Project Planning of Offshore Pipelines Replacement: Study Case at Offshore North West Java

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Keywords: Six Sigma Analysis, Critical Path, Fault Tree Analysis, Event Tree Analysis, Top Event, Project Delay, Offshore Pipelines Installation.

Abstract: There are many factors that lead to project delay, for example hurricane, one of weather factors that make it impossible to do work or factors like lack of utilities and construction equipment, delays in purchasing material orders, lack of experienced and skilled human resources in the field, delay costs down to purchase activities on the project. In addition, there are delays in critical activities because they have a large impact on the duration of the project. Therefore, it is necessary to do an analysis by looking for activities that are critical to the project and factors that can cause delays in the new pipeline project because it can endanger the owners and contractors. This study will discuss three factors, i.e. (1) the determination of the critical path, (2) the determination of the root causes that can cause delays, and (3) the risk of over time, of new pipelines replacement project in North West Java, Indonesia. There are 4 indications of delay in this project, which are delay that occurred in June 2018, September 2018, and October 2018. The biggest deviation between the targeted value and the actual value occurred in October 2018, which was 5,05%. This study uses six sigma methods, critical path, fault tree analysis and event tree analysis. The results of fault tree analysis will get the probability value for the top event of the project delay. The results of the event tree analysis will get the risk level of each factor that causes delays. Six sigma results will get to the root of the problem, the value of sigma, and what corrective action is taken.

1 INTRODUCTION

Project implementation must be supported by good project management. The absence of project management will bring about change, unclear goals, non-challenging planning, high risk, poor project quality, more expensive project costs, and delays in improving the project. Project management can be subdivided such as project time management, project human resource management, and project risk management (PMBOK, 2008). The company plans to carry out the offshore pipeline repair and replacement project. This project covers the aspects of engineering, procurement, construction, and installation, called the EPCI project. Pipeline is located 80 - 200 km northeast of Jakarta. The core of this project is to replace pipes that are no longer suitable for use by installing new pipes for pipes that are already damaged. There are three pipelines

planned to be replaced, namely:

- (i) Pipe A with 12"OD and 1 km length
- (ii) Pipe B with 10" OD and 7.2 km length
- (iii) Pipe C with 8" OD and 7.2 km length

The construction project sector has a high level of responsibility and complexity. All construction projects can be categorized as complex projects. This is caused by a direct relationship between complexity and involves various interrelated parts that must be managed with regard to conditions of differentiation and interdependence, thus causing construction projects to have high risks (Baccarini, 1996). Based on the explanation in the paragraph above, this project is a complex project because it has three projects with different pipe sizes and lengths. Therefore, this paper will discuss project risks in the form of overtime (project delays) that can have an impact on project cost overruns. Project risk in the form of overtime and

excess costs must be avoided because it can endanger all parties such as the owner and contractor, for example from the owner will suffer losses because the object of the project cannot operate, so it does not generate profits. On the other hand, the contractor must be responsible for delays and the contractor's good name becomes bad.

Table 1: Indications of Delay in the Development of a New Pipe Replacement Project on Deteriorated Pipelines.

	June 2018	June 2018	Sept 2018	Oct 2018
<i>Planned Value</i>	0,06%	6,35%	40,11%	53,20%
<i>Earned Value</i>	0,00%	5,63%	39,71%	48,15%

Table 1 explains that there are indications of delays in the construction of new replacement pipe projects for damaged pipelines. Indications occurred in June 2018, September 2018, and October 2018. The largest deviation between the planned value and the value obtained occurred in October 2018 at 5.05%. Many factors can make a project experienced delay, for example weather factors that make it impossible to do work or factors of lack of utilities and construction equipment, delays in purchasing material orders, lack of experienced and skilled human resources in the field, delay costs down to purchase activities on the project, in addition, there are delays in critical activities because critical activities have a large impact on the duration of the project.

Therefore it is necessary to analyze the delay by looking for any activities that are critical to the project and factors that can cause overtime. To avoid overtime in this new pipeline project because it could endanger the owner and contractor as explained in the previous paragraph. In this study, we will discuss determining the critical trajectory, determining the root cause or factors that can cause delays, a large risk of delay, and what improvements can be made.

2 BASIC THEORY

2.1 Project Management

Project management is planning, organizing, leading, and controlling company resources to achieve short-term goals that have been determined. Project management has a basic function consisting of managing the scope of work, time, cost, and quality. The key to successful implementation of a project if

you can manage these aspects correctly (Samad, 2019).

2.2 Risk

Risk is a combination of the probability of an event and the impact of the event, the impact can be more than one for one event. However, risks are generally seen as negative things such as danger, loss, and other impacts. These impacts and losses is actually a form of uncertainty that should be understood and managed effectively by the project implementer (Szymanski, 2017).

2.3 Critical Path

Critical path is a project management technique that uses only one-time factor per activity. Critical path or critical path is the fastest path in project work, every project included in this path is not given a break / break for the process. Critical path is a path that consists of activities which if delay will result in delay completion of the project Heizer, 2014).

2.4 Six Sigma Analysis

The statistical concept that measures a process that is redelayd to the sigma number, where the closer to 6 the results will be as expected. Six Sigma is a management philosophy that focuses on removing defects by emphasizing understanding, measuring and improving processes (Brue, 2002). Six Sigma has 5 stages, as follows (Larasatie, 2019):

(i) Define

Define aims to identify the process to be improved and determine what resources are needed in project implementation.

(ii) Measure

Measure is an advanced stage of define where at this stage validation of a problem is done by calculating and obtaining numbers that can provide clues about the problem.

(iii) Analyze

At this stage, the cause of the deviation in a project will be investigated with a hypothesis which will then be tested.

(iv) Improve

At this stage, the main factors are ascertained and give new influences so that they get the desired results. The purpose of this stage is to implement a new system so that the project can run more optimally.

(v) Control

Maintain changes that have been made through

certain measuring devices so that it can be assessed whether the new system is optimally implemented in the project.

2.5 Fault Tree Analysis

Fault tree analysis is a method to identify and analyze all the factors that might cause a system failure and provide a basis for calculating the probability of a failure event. Unwanted events that are on top (top event) for all root causes that might occur at the bottom. The causative factors are deductively identified, logically arranged and represented using pictures in a tree diagram illustrating the causal factors and their logical relationship to the peak event (ISO, 2009).

2.6 Event Tree Analysis

Event tree analysis is a method used to evaluate processes and events that lead to the possibility of failure. This method is useful in analyzing the consequences arising from failures or unwanted events. The consequences of events are followed through a series of possibilities. By analyzing all possible outcomes, it is possible to determine the percentage of results that lead to both desired and undesirable results (Silvianita et al, 2017).

Event tree analysis is an analysis technique to identify and evaluate the sequence of events in a potential accident scenario after the initial event or initiating event. Event tree analysis uses a visual logic tree structure known as an event tree. The purpose of the event tree analysis is to determine whether the initiating event will develop into a serious accident or whether the event is sufficiently controlled by the safety system and procedures applied in the system design. Event tree analysis can produce many different results from one initial event, and this provides the ability to obtain probabilities for each outcome (Ericson, 2005). Project delay using ETA and Bow Tie Analysis for mooring change replacement can be seen in (Silvianita et al, 2018). and project delay for HRSG has been discussed (Silvianita et al, 2017b).

3 METHODOLOGY

The procedures of the research are as follow:

1. Formulations of Problems

The formulations of the problems are decided as the purpose or the goal of the research.

2. Literature Studies

To support the research itself, literature studies are needed to develop insight and analysis. In this case, the literature needed is as follows:

- a. A study about offshore pipeline projects.
- b. A study about project management and project delays.
- c. A study about critical path methods.
- d. A study about six sigma analysis.
- e. A study about fault tree analysis
- f. A study about event tree analysis.
- g. A study about risk matrix

3. Data Collection

Data collection is needed as material to support the research hypothesis. The data correlated with the evaluation and the current conditions. The data needed are as follows:

- a. Pipeline repair and replacement project report data
- b. Pipeline repair and replacement project master schedule data
- c. General data about the pipeline installation project
- d. Make a questionnaire to look for the probability of basic FTA events, pivotal ETA events, and determination of risk matrices.

4. Data Analysis and Discussion

The data that has been collected will be analyzed and discussed:

- a. Make network diagrams and find project critical activities using the critical path method.
- b. Identification uses 4 stages of six sigma analysis (define, measure, analyze, and improve).
- c. Determine the root causes that can cause the project to be delayed by using fault tree analysis.
- d. Determine risk by using event tree analysis.
- e. Determine the probability of project delay.

5. Results and Conclusions

The results of this study are used as a reference for decision making in planning subsea pipeline projects in the future, so as to advance the performance and smoothness of all underwater pipeline project activities. The flowchart of this paper are shown in Figure 1.

4 ANALYSIS RESULTS AND DISCUSSIONS

4.1 Data Collection

Data collection is the most important thing in project planning because without the complete data planning

the project cannot be planned properly. In this study, the following data were obtained from one of the oil and gas company. Data that are needed in this study include work data on project activities, duration of project activities and relationships between the project activities (predecessors).

Table 2: Project Activity, Duration, and Predecessor.

Activity ID No.	Activities	Duration (day)	Predecessor
1	<i>Project Start</i>	0	-
2	<i>Project Management & Service</i>	78	1FS+5
3	<i>Pipeline Report (Engineering)</i>	66	1FS+2
4	<i>Pipeline Drawing (Engineering)</i>	65	1FS+2; 3SS
5	<i>Material Take off (Engineering)</i>	45	1FS+2; 3SS
6	<i>Request for Quotation (Engineering)</i>	37	1FS+3; 3SS+1

32	<i>Post-Lay Survey (Installation)</i>	5	26FS
33	<i>Pre-Commissioning Pipeline A</i>	6	0FS; 31FS; 32FS
34	<i>Pre-Commissioning Pipeline C</i>	9	33FS
35	<i>Pre-Commissioning Pipeline B</i>	9	34SS
36	<i>Provisional Acceptance (PAC)</i>	0	35SS
37	<i>Punch List PAC Completion</i>	18	33FS
38	<i>Final Acceptance</i>	12	36FS; 37FS+20

4.2 Creating a Gantt Chart using Primavera P6 to Obtain a Network Diagram

Primavera P6 outputs are in the form of Gantt charts and network diagrams. The network diagram formed in Primavera P6 is a diagram of type AON (Activity on Node) where project activities are represented in nodes. However, Primavera P6 is not able to show the number of ES (Initial Start), EF (Initial Finish), LS (Delay Start), and LF (Delay Finish) manually.

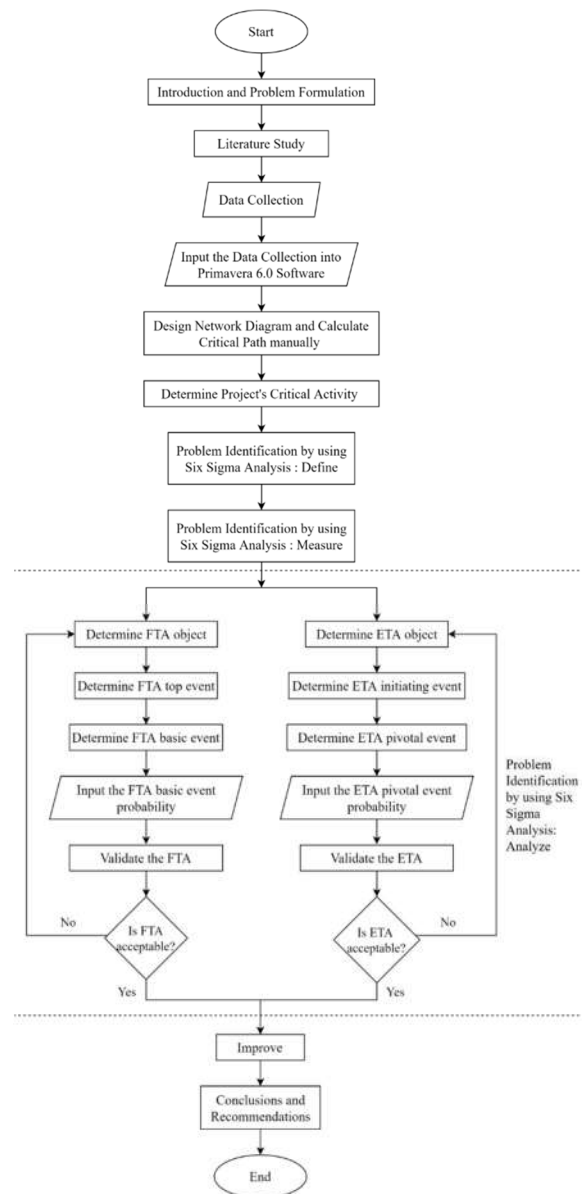


Figure 1: Flowchart.

4.3 Make Network Diagrams and Perform Critical Path Method Calculations Manually

This critical activity forms a critical path where if one or more project activities are delayed, it will cause the entire project to be delayed. Therefore, activities in the critical path need more supervision in the timeliness of completion. The activity nodes coloured in red indicate the critical activities and the direction of the red arrows indicate the critical paths.

4.4 Problem Identification using Six Sigma Analysis (Define)

At this stage a project charter will be made to review information and problems that will occur in the offshore pipeline replacement project of the company under study. Project charter can provide information about general information, project scope, resource requirements, critical time schedules, roles and responsibilities of the project team.

Table 3: Project Charter - Project Scope.

Project Scope
Situation / Problem / Opportunity
In this project there are indications of delays due to a large deviation in the project planning data.
Project Goals
Maintaining the company's oil and gas production as well as to obtain additional production by installing a new pipeline to replace the existing damaged pipe.

Table 4: Project Charter - Project Scope.

In Scope / Out of Scope
In Scope: • Installation of new pipes with a 12 "OD size and 1 km pipe length. • Pre construction survey. • Removal of the subsea obstacle that can hinder the installation of new pipes. • Survey of laying subsea pipelines with ROV. • Installing risers, bends, and spools. • Post lay survey. • Installation supports on pipelines identified as free spans. • Flooding, pigging, hydrotest, drying, purging and pre-commissioning of new pipes. • Installation of break out spool between riser and topside piping. • Disassemble existing OD 8 "riser sizes, including riser clamps and end caps installation. • Carry out work activities using a pressurized environment for installing riser clamps. • Perform bevel end repairs at the end of the pipe. • Welding procedure specifications using ASME IX code, API 1104 (PSL 2), AWS D1.1 and company specifications.

Out of Scope:

- There are no monetary problems at home or abroad during the project work.
- Project detail procedure has not changed and is in accordance with the contract during the project work.

Table 5: Project Charter - Project Scope.

Objectives
• Project on schedule. • Without problems or work accidents. • The project does not over cost. • Stages of work are carried out according to quality standards.
Project Assumptions
• The document engineering process ran smoothly. • The procurement process went smoothly. • The construction process is running smoothly. • The installation and checking process runs smoothly.
Risk and Dependencies
• Lack of resources. • The risk of a project being delayed due to concurrent workers.
Risk and Dependencies
• Increased project costs due to accuracy of cost estimates and poor scope. • Risk that the project will fail to produce results according to project specifications. • Mistakes in strategies such as choosing technology that cannot function properly. • Operational risks include risks from poor implementation and process problems such as procurement, production and distribution. • Risks associated with external hazards, including storms, floods and earthquakes.

4.5 Problem Identification using Six Sigma Analysis (Measure)

In calculating and identifying the problem, timeliness is defined as an element which directly redelayd to the expected time calculation. Project planning time must be according to actual progress time. Therefore, at this stage an analysis is carried out to determine the sigma value with a percentage deviation between the time of planning and the time of progress with the maximum value of 6. The sigma value is obtained from the conversion of DPMO (defects per million opportunities) value. The DMPO value is derived

from the difference of project time (in percentage), i.e. the deviation that occurs between the time of project planning and the time of actual progress.

For detailed calculation, the biggest deviation between monthly planning time and the actual time is taken. The result found that in October there was a time lapse of 5.05%. Defective Value per Million Opportunities (DPMO) is obtained from the formula:

$$\text{DPMO} = \text{Large Deviation} \times 1000000$$

In this calculation, the DPMO value is:

$$\text{DPMO} = 5.05\% \times 1000000 = 50500$$

DPMO calculation results show any deviations from the target that was decided. This DPMO number will then be used to get the sigma level by converting the DPMO number to the sigma number.

In the calculation of more specific sigma numbers, calculations can be performed using Microsoft Excel. The formula for calculating sigma number is:

$$\text{Sigma Number} = \text{NORMSINV} (1 - \text{Large Deviation} + 1.5)$$

By using pre-existing data, the sigma number is as follows:

$$\text{Sigma Number} = \text{NORMSINV} (1 - 5.05\%) + 1.5 = 3.140025$$

The sigma calculation results show a more specific result of 3.14 out of 6 (maximum) which can be increased again with improvements made. This result occurred in October where there were 3 procurement, construction and installation.

Table 6: The Basic Event and Intermediate Event of the FTA Procurement Phase Experienced Interference.

Intermediate Event Level 1	Intermediate Event Level 2	Basic Event
Procurement Stage Experiencing Interference	Problem in ordering material	Planning is not mature enough procurement
		Material provider negligence
		Communication and coordination are unclear
	Material arrived at the project site delay	Shipping goods from abroad
		Material order delay
		The fabrication process has been delayed
		Material delivery is delay
	Problems with payment	

Table 7: Basic Event and Intermediate Event FTA Construction Phase Experienced Disruption.

Intermediate Event Level 1	Intermediate Event Level 2	Basic Event
Construction Phase Has Disturbances	Problematic equipment section	Lack of equipment
		The equipment is damaged
		Equipment arrived delay
		Low ability of equipment
	The labor force has a problem	Lack of workforce
		Difficulties in labor mobilization
		Labor productivity is lacking
		Absent laborers
		Inexperienced workforce
		Problems arise between workers
	Design changes that resulted in the addition of work	
	Bad weather	
	Natural Disaster	
	An accident occurred during the construction process	Unexpected accident
		Lack of supervisor and control
		Communication and coordinator are unclear
		Problems arise between workers
		Lack of socialization and work safety training
	An error occurred during the construction process	Document/ picture not detail
		Inexperienced workforce
		Communication and coordination are unclear

4.6 Identifying Problems and Calculating Probability of Problems using Six Sigma Analysis (Analyze): Fault Tree Analysis and Event Tree Analysis

- Determine top FTA events and ETA initiating events:

The top event and initiating event that is being investigated is the delay in the offshore pipeline replacement project.

Table 8: The Basic Event and Intermediate Event of the FTA Installation Stage Are Interrupted.

Intermediate Event Level 1	Intermediate Event Level 2	Basic Event
Installation Phase Has Interference	Problem in equipment section	The equipment is damaged
		Low standard of equipment
	Bad weather	
	Natural disaster	
	An accident occurred during the installation process	Unexpected accident
		Lack of supervision and control
		Communication and coordination are unclear
		Problems arise between workers
	An error occurred during the installation process	Lack of socialization and work safety training
		Document/picture not detail
		Inexperienced workforce
		Communication and coordination are unclear

Table 9: Top Event Probability.

No	Basic Event	Probability
1	Procurement Stage Experiencing Interference	0,008360286
2	Construction Phase Has Disturbances	0,022554886
3	Installation Phase Has Interference	0,021781371
Top Event Probability (Total)		0,052696543

- Determine basic FTA events and ETA pivotal events:

Basic events and pivotal events are determined by conducting a literature review of delays in construction projects and subsea pipeline projects. The results of the basic event and pivotal event are obtained after validating to the respondent (client

side).

Opportunities for each basic event were obtained with a questionnaire from 7 respondents experienced in underwater piping projects. After that, do the calculations to get the probability of intermediate events and top event. The event tree diagram shows that the initiating event of this research is that the delay offshore pipelines replacement project has 6 probability events A, B, C, D, E, and F.

(a) Event A

Replacement of the underwater pipeline is completed on time because it runs smoothly and there is no problem in one of the pivotal events. The probability of Event A is $0.05270 \times 0.83 \times 0.77 \times 0.80 \times 0.69 \times 0.71 = 0.0132$

(b) Event B

Replacement of underwater pipelines is complete but can be delayed for 5-6 weeks due to shortages and conditions of available equipment. The probability of Event B is $0.05270 \times 0.83 \times 0.77 \times 0.80 \times 0.69 \times 0.29 = 0.0053$

(c) Event C

Replacement of underwater pipelines is complete but can be delayed for 3-4 months due to lack of manpower and inexperience. The probability of Event C is $0.05270 \times 0.83 \times 0.77 \times 0.80 \times 0.31 = 0.0085$.

(d) Event D

Replacement of underwater pipelines is complete but can be delayed for 4-5 months due to delays in procurement. The probability of Event D is $0.05270 \times 0.83 \times 0.77 \times 0.20 = 0.0067$

(e) Event E

Replacement of subsea pipelines is complete but can be delayed for 5-6 months due to lack of implementation and supervision of project management. The probability of Event E is $0.05270 \times 0.83 \times 0.23 = 0.0100$

(f) Event F

Replacement of subsea pipelines cannot be resolved due to cost issues during the project, considering costs are the main thing of the project's success. The probability of Event F is $0.05270 \times 0.17 = 0.0090$

The next step is to determine the risk categories of each event into the risk matrix. The probabilities of each output are matched into the Frequency Index (FI) table and the consequences of each output are matched into the Severity Index (SI) table. The FI and SI tables are referenced from DNV RP F107 (Pipeline Risk Assessment).

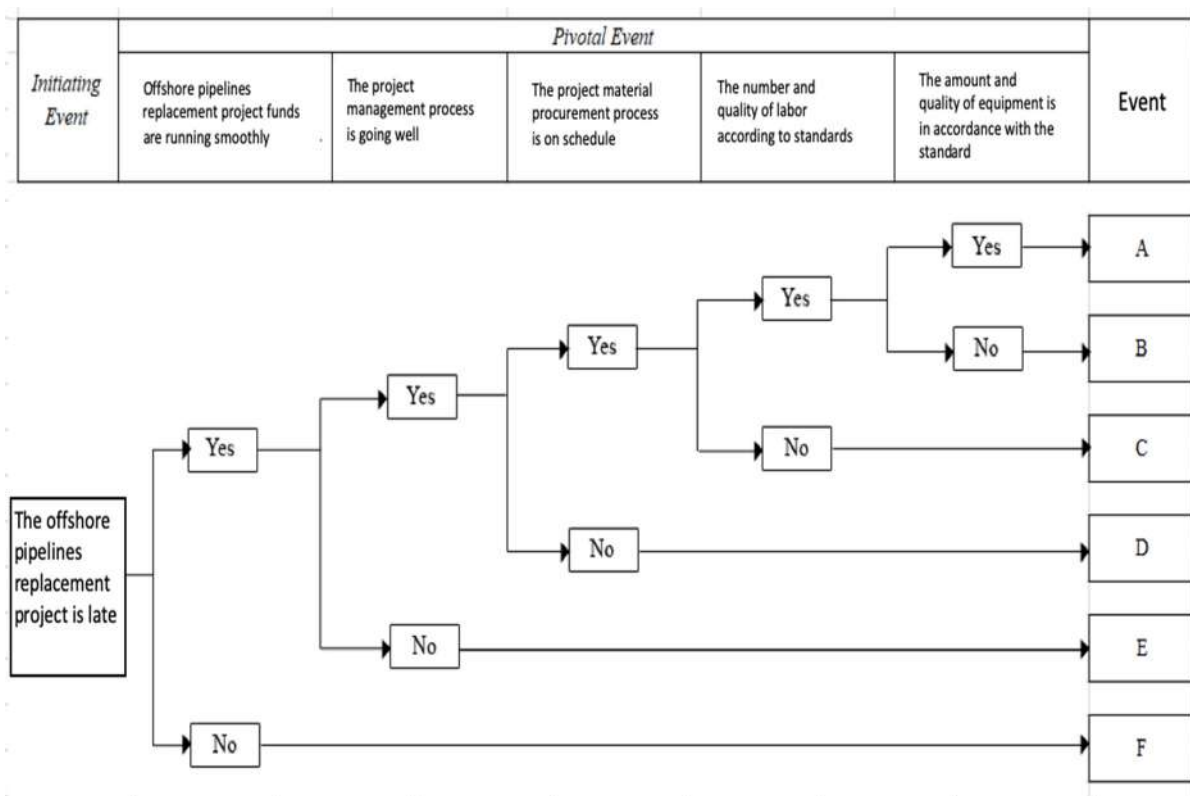


Figure 2: Event Tree Diagram.

Table 10 explains that event A is categorized as moderate risk while events B, C, D, E, and F are categorized as high risk due to high probability and high severity. In this result, there is no low risk result because the lowest probability of event is categorized as high probability (4) and very high (5).

Table 10: Risk Matrix.

		Severity				
		1	2	3	4	5
Frequency	5	A			E	F
	4			B	C, D	
	3					
	2					
	1					

High Risk
 Medium Risk
 Low Risk

4.7 Determination of Problems using Six Sigma Analysis (Improve)

It is known that in that month there are material delivery activities, the likelihood of this happening is that the material delivery is delayed in view of the large probability at the basic event. Delay material delivery has a probability of 0.007389714.

The step that can be taken is to complete all material financing because monitoring of funding in ordering materials is important because in the absence of funds the goods cannot be ordered or delivered on time. For the next project, it is expected that the procurement plan can be planned as well as possible so that there are no more errors in the delivery of materials and the project is running optimally.

5 CONCLUSIONS

Based on discussion in the previous chapter, these are the conclusion of the study:

1. The offshore pipelines replacement project has 19 critical activities created from 38 project

- activities.
2. Many factors cause project delay. These factors are divided into 3 intermediate event branches, namely the procurement, construction and installation stages.
 3. The offshore pipelines replacement project using Fault Tree Analysis (FTA) obtained the results of the top event probability on this project of 0.05270.
 4. Offshore pipelines replacement project using Event Tree Analysis (ETA) obtained 6 events with different probabilities and consequences.

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Time Acceleration Analysis of the Kijing Terminal Development Project in Mempawah, West Kalimantan

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Keywords: Precedance Diagramming Method (PDM), Monte Carlo Method, Time Cost Trade off, Project Duration Acceleration.

Abstract: The cargo port is a port that is planned specifically for the purpose of loading and unloading of goods and is equipped with a warehouse for storing goods and cranes to move cargo ship. With the construction of the port, the company needs a good project management design. The mismatch between the schedule design and the reality on the ground still creates problems. This is a challenge for a project manager who must carefully look at existing activities to avoid delays in the project. To overcome this problem, it is necessary to maximize performance by reducing the duration of each activity in the project that still includes safety time. This paper analyzes the acceleration of the duration of the Kijing terminal construction project in Mempawah West Kalimantan using the Precedance Diagramming Method (PDM) method, as well as the Monte Carlo method to find out the project's probability on time, then find the optimal duration with the Time Cost Trade Off (TCTO) method. From the results of the analysis with the PDM method obtained 12 activities that exist on the critical path. Based on the Monte Carlo simulation, the results show that the probability of the project to be completed on time is 55% or 45% chance of being late. The optimum duration and costs on the critical path are obtained at the addition of 1 hour of overtime. The total duration of the project, which was originally 595 days to 513 days with a difference of 82 days, with a total cost of Rp2,740,269,901,670 to Rp2,740,775,765,420 with a difference of Rp505,863,750 and also a cost slope of Rp 6,169,070 per day.

1 INTRODUCTION

The West Kalimantan region is one of the priorities in economic development announced by the Government of the Republic of Indonesia. This region has several potential natural resources such as palm oil, bauxite, rubber, wood, and other agricultural products. In addition, there are many investors (both local and foreign) who are interested in investing in the industrial sector in West Kalimantan.

This potential was realized by one of the oil and gas companies as a port service provider company is able to develop the potential of ports in Kijing Beach, Mempawah, West Kalimantan. The port development plan is also very likely, given the existing condition of the Pontianak Port which is the closest port to Kijing that has overcapacity and the limited depth of the Kapuas River channel ± 5 meters. It is hoped that the existence of the Kijing Terminal can later reduce the burden on existing ports in West Kalimantan, invite ocean going ships

to be able to dock at the Kijing Terminal and reduce overall logistics costs.

Kijing Terminal is located ± 80 km from the North of Pontianak Harbor where there is Temajo Island in front of Kijing Beach with a distance of ± 5 km that can be used to reduce wave energy. Kijing Terminal will be developed in 3 (three) stages, namely: Initial Phase, Phase I and Phase II. Kijing Terminal has offshore and onshore facilities, each of which has 4 (four) zones, namely: container zones, liquid bulk zones, dry bulk zones and multipurpose zones which are expected to accelerate economic growth in the West Kalimantan region and provide benefits to all parties involved. Based on the importance of doing a good scheduling and acceleration on the Kijing Terminal construction project, it is necessary to have a system that makes it easy to analyze the acceleration of the Kijing Terminal construction project in Pontianak Port, Mempawah, West Kalimantan.

Table 1: General Specification.

SPECIFICATION	
KIJING TERMINAL PROJECT	
INITIAL PHASE	
Length	5434 m
Total Length	5434 m
Depth	-12 sd 15
Area	32.8 Ha
Capacity (per year)	500.000 TEUs 500.000 tonnes
PHASE I	
Length	2441m
Total Length	4415 m
Depth	-12 sd 15
Area	49.2 Ha
Capacity (per year)	950.000 TEUs 23.840.000 tonnes
PHASE II	
Length	1747 m
Total Length	6162 m
Depth	-12 sd 15
Area	37.8 Ha
Capacity (per year)	1.950.00 Us

2 LITERATURE STUDIES

This section explains the theory used in this study. Project management for construction have been used by (Hendrickson, 2003). Project scheduling for gas transmission pipe by Silvianita et al (2018).

2.1 Presedence Diagram Method

Network planning analysis is to find out the critical path of the project (Demeulemeester, 2013).

The critical path is a series of activities that require the overall duration of the project and may not delay because it will affect the entire project. There are four constraints in the Presedence Diagram Method schedulling method, namely finish to finish, finish to start, start to finish, and start to start (Taha, 2007).

2.2 Monte Carlo Simulation

The Basics of Monte Carlo Simulation theory, one of which is the Random Variable. According to Bain and Engelhardt (1992) A random variable is a function defined in the sample space S that connects every possible outcome at e with a real number, namely $(e) = x$. If the set of possible outcomes of the

random variable X is a calculated set, $\{1, x_2, ..x_n\}$, or, $\{x_1, x_2, ..\}$, then X is called a discrete random variable. Random numbers to find the most possible cost (Most Likely Cos) = ML can be generated using the RAND function found in Microsoft Excel.

2.3 Time Cost Trade off

Time Cost Trade Off analysis method is a method to speed up time by making an exchange between time and cost. In this research, the time acceleration analysis used on the critical path of the project is the addition of overtime hours to workers in the field. The addition of overtime hours is carried out in several schemes, namely one hour, two hours, and three houts of additional overtime. Then the optimum additional overtime hours scheme is sought by considering crash cost, crash duration, and cost slope of each scheme.(Hullet & Nosbich, 2012).

3 METHODOLOGY

3.1 Problem Indentification

At the stage of formulating the problem, the problems that occur in the object of research will be obtained and become the focus of the research. The object of this research is multipurpose terminal jetty in Kuala Tanjung. Then, the researcher will determine the objectives that refer to the formulation of the problem that has been set.

3.2 Literature Study

Literatures and references are needed to determine the appropriate method in this research and to facilitate the writing of the steps in this paper.

3.3 Data Collection

Data of the Kijing Terminal project is needed for the completion of this paper. The data needed include: time schedule, draft budget, and manpower.

3.4 Critical Path Analysis using PDM Method

The PDM method was used to analyze the critical path in this study. The use of the PDM method in this study facilitates the description of network planning because of the overlapping activities on the project (Leach, 2000).

3.5 Probability Analysis using Monte Carlo Simulation

Monte Carlo Simulation is used to find out the probability of duration of project completion in this study. The results of using this method are knowing the smallest, largest, and also probability based on the actual contract (McCabe, 2003).

3.6 Acceleration Analysis using TCTO Method

This paper uses the TCTO method to accelerate the duration of the project. TCTO analysis is done by compressing activities that are on the critical path. The alternative used in the TCTO method in this study is to use a one-hour, two-hour, and three-hour overtime scheme (Kezner, 1995). TCTO has been used in many areas such as in loadout process (Silvianita et al., 2016).

3.7 Optimum Costs Determination

To determine the optimum duration of the project, cost analysis must be carried out. The optimum duration of the project is the duration of acceleration by issuing costs as effectively and efficiently as possible. If the amount of costs incurred to accelerate the project is too expensive, then another acceleration alternative will be sought. (PMI. 2013).

4 RESULT AND DISCUSSION

4.1 Project Time Schedule

Project time schedule data consists of the activity name, duration, start date, and completion date. (based on the project)

Table 2: Project Time Schedule.

No	Activity	Duration	Start	Finish
A Preparatory Work				
1	Site office	14 days	Mon 8/13/18	Sun 8/26/18
2	Mobilization & Demobilization	35 days	Mon 8/27/18	Sun 9/30/18
3	Stake out dan Positioning	47 days	Mon 10/8/18	Fri 11/23/18
B Work on Pier				
4	Concrete Pile Work Dia. 1000 mm Bottom Type C0	327 days	Mon 11/26/18	Fri 10/18/19

No	Activity	Duration	Start	Finish
1000 mm Bottom Type C0				
5	Concrete Pile Work Dia. 600 mm (Wave Screen)	145 days	Mon 2/25/19	Fri 7/19/19
6	Concrete Pile Work Dia. 800 mm (PMA)	327 days	Mon 11/26/18	Fri 10/18/19
7	Concrete Pier Work and PMA	271 days	Mon 2/25/19	Fri 11/22/19
8	PMA Support Buildings	91 days	Mon 11/25/19	Sun 2/23/20
34	Electrical Work	91 days	Sun 11/24/19	Sat 2/22/20
35	Testing and Commissioning	127 days	Sun 11/24/19	Sun 3/29/20

4.2 Activity's Constraint

Activity's constrain data is to find out which activities can be started earlier than other activities. In this study, the constraint and time schedule data obtained were inputted into Microsoft Project software to obtain the expected network planning.

Table 3: Activity's Constraint.

No	Activity	Duration	Predecessors	Successors
A Preparatory Work				
1	Site office	14 days	START	4
2	Mobilization & Demobilization	35 days	3	5
3	Stake out dan Positioning	47 days	4	7FS+2 days, 16FS+2 days, 18FS+2 days, 17FS+62 days, 22FS+142 days, 26SS+14 days
B Work on Pier				
4	Concrete Pile Work Dia. 1000 mm Bottom Type C0	327 days	5FS+2 days	8SS+90 days, 9SS,11FS+37 days, 14FS+37 days
5	Concrete Pile Work Dia. 600 mm (Wave Screen)	145 days	7SS+90 days	10SS
34	Electrical Work	91 days	43FF	42SS
35	Testing and Commissioning	127 days	44SS	FINISH

4.3 Critical Path Analysis

The critical path is a path that has a series of activities with the longest total amount of time and shows the fastest project completion time period. In addition to using software, the determination of the critical path was also done by manual calculation by calculating the earliest start, earliest finish, latest start, latest finish and float. Activities that have total float = 0 are activities included in the project's critical path. The formula of float is LS-ES or LF-EF.

Table 4: Critical Path Manual Calculation.

Activity	Activity ID	ES	EF	LS	LF	Float
Preparatory Work						
Site office	A1	1	582	8	575	7
Mobilization & Demobilization	A2	15	547	22	540	7
Stake out dan Positioning	A3	57	493	57	493	0
Work on Pier						
Concrete Pile Work Dia. 1000 mm Bottom Type C0	B1	106	164	135	135	29
Concrete Pile Work Dia. 600 mm (Wave Screen)	B2	197	255	225	227	28
Hoist Crane Work	H3	532	37	532	37	0
Electrical Work	H4	469	37	469	37	0
Testing and Commissioning	H5	469	1	469	1	0

4.4 Critical Path Determination

After determining the total float on the project, activities that are on the critical path can be identified. An activity is said to be critical if the Total Float value is equal to 0. The total float is obtained from LF minus EF or LS minus ES.

Table 5: Critical Path.

No	Activity	Activity ID	Float
1	Stake Out & Positioning	A3	0
2	Land Plant Work	E1	0
3	Rigid Concrete Work	E2	0
4	Rigid Work	F1	0
5	Paving Block Work	F2	0
6	Structure	G1	0
7	Building Support	G2	0
8	Mechanical Work	H1	0
9	Plumbing Work	H2	0
10	Hoist Crane Work	H3	0
11	Electrical Work	H4	0
12	Testing and Commissioning	H5	0

4.5 Project Completion Probability Analysis

The analysis carried out in this study is about the probabilities of the duration of project completion using the Monte Carlo method. There are several steps in using the Monte Carlo method (Hullet & Nosbich, 2012), including:

A. Optimistic Time and Pessimistic Time Data
Other data used for the Monte Carlo simulation are pessimistic time and optimistic time for each activity. This data was obtained from interviews with contractors and staff at one of the oil and gas companies. The following are the results of the interviews summarized in the table below.

Table 6: Optimistic Time and Pessimistic Time Data.

Activity	a	m	b
	Days		
Site office	7	14	20
Mobilization & Demobilization	26	35	42
Stake out dan Positioning	39	47	54
Work Pile Pile Dia. 1000 mm Bottom Type C0	316	327	334
Work Pile Dia 600 mm (Wave Screen)	137	145	154
Work Pile Pile Dia 800 mm (PMA)	316	327	337
Work PMA	262	271	283
Work Mechanical	146	154	162
Work Plumbing	146	154	162
Work Hoist Crane	23	28	45
Work Electrical	84	91	99
Testing And Commissioning	116	127	138

B. Calculate the Standard Deviation

Standard deviation is used as input normal distribution in generating random numbers. Calculation of standard deviation is done using the following formula:

$$\sigma = \frac{b-a}{6} \quad (1)$$

Where:

σ = standard deviation

a = optimistic time

b = pessimistic time

The following table is the result of calculating the value of the standard deviation and the new duration of the project.

Table 7: Calculation of the standard deviation and new duration.

Activity	sd	New Duration Days
Site office	0.8	14
Mobilization & Demobilization	1.3	35
Stake out dan Positioning	1.2	47
Work Pile Pile Dia. 1000 mm Bottom Type C0	1.8	323
Work Pile Pile Dia 600 mm (Wave Screen)	1.5	146
Work Pile Pile Dia 800 mm (PMA)	2.2	325
Work PMA	2.3	273
Work Mechanical	1.5	153
Work Plumbing	1.5	154
Work Hoist Crane	2.8	21
Work Electrical	1.2	92
Testing And Commissioning	2.3	129

C. Determine the Number of Iterations

It is carried out by means of gradual iteration, until there is little or no change in the outcome. In this study iteration is carried out in stages, namely from a range of 10 to 1000 times iteration. We can choose how many time we will do the iteration base on the graph of the standart deviation change and the graph of parameter avarage change (until stable). Following is a graph of the statistical parameters taken to see the difference in results from adding iterations.

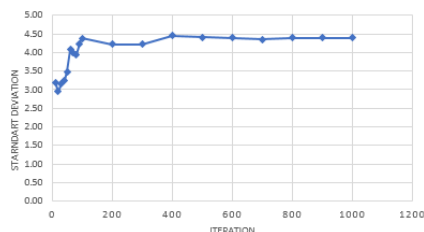


Figure 4: Graph of changes in the standard deviation of the number of iterations.

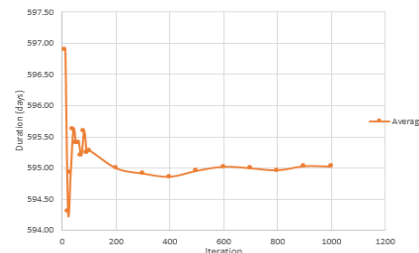


Figure 5: Graph of changes in the results of the parameters Average with the Number of Iterations.

From the graph above it can be concluded that by doing 1000 times the iteration of statistical parameters in the form of standard deviations, the median and the average are already in the stable results. Unstable results occur in iterations carried out between 10 to 500 times. After iterating 1000 times, the results obtained with statistical parameters such as in table 8.

Table 8: Statistical Parameters of Iteration Results.

No	Parameter	Quantity
1	Standart Deviation	4.17
2	Median	595
3	Kurtosis	0.2
4	Skewness	0.023
5	Average	595
6	Maximum	610
7	Minimum	610
8	Varian	17.4011
9	Mode	596

D. Calculate PDF and CDF

Calculating Probability Density Function (PDF) and Cumulative Distribution Function (CDF) is useful to find out the opportunities of project completion.A PDF is specifies the probability, CDF is a direct measure of probability.

The steps in calculating pdf and cfd are as follows:

Table 9: Monte Carlo Simulation Results in 1000 Times Iteration.

Duration	Cum	%cum	Prob	%prob
587	41	4%	41	4%
588	56	6%	15	2%
589	99	10%	43	4%
590	141	14%	42	4%
591	206	21%	65	7%
592	279	28%	73	7%
593	354	36%	75	8%
594	468	47%	114	11%
595	563	56%	95	10%
596	659	66%	96	10%
597	731	73%	72	7%

Table 9: Monte Carlo Simulation Results in 1000 Times Iteration (cont.).

Duration	Cum	%cum	Prob	%prob
598	796	80%	65	7%
599	855	86%	59	6%
600	909	91%	54	5%
601	936	94%	27	3%
602	964	97%	28	3%
603	979	98%	15	2%
604	985	99%	6	1%
605	992	99%	7	1%
606	997	100%	5	1%

Next is to plot the results of pdf and cdf in the table into graphical form, where the cdf results are drawn in scatter form, while the pdf results are in the form of a histogram as shown in Figure 6.

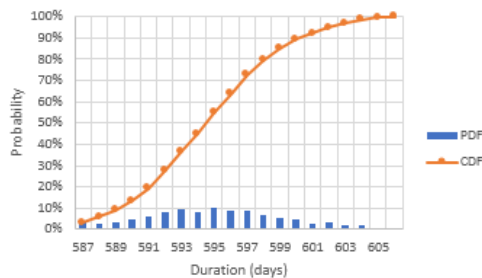


Figure 6: PDF and CDF Graph Results 1000 Times Iteration.

From the graph above shows the results of the Monte Carlo simulation in the form of PDF and CDF graphics, where CDF is the blue line, while the PDF is an orange-colored histogram. The x-axis in the graph above shows the duration of the project completion in days. The y-axis on the graph shows the probability. Percentile is the division of 100 data positions sorted in a distribution. From the graph above it is found that for percentiles / 50% probability, the project can be completed in 594 days or less. Whereas for 80% percentile / probability, the project can be completed in 598 days or less.

After calculating, it is found that completion of the project for less than 595 days has a 56% chance of success, or that the project 44% has a chance to be late. And a 100% chance of success of the project is estimated within 606 days.

4.6 Analysis using the Time Cost Trade off Method

Analysis of the Time Cost Trade Off method is an analysis by exchanging time and costs so that it can speed up the project completion time but result in an increase in costs. In this research, the project time is

accelerated in activities that are on the critical path by increasing the worker's overtime hours. In adding workers' melting hours there are government regulations that must be followed by the contractor.

A. Crash Duration Calculation

The following is an example of the calculation of the duration of the crash carried out in this project on Dia Concrete Pile Work. 1000 mm Bottom Type C0 for additional one hour of work:

Normal duration = 316 days

Job Weight = 5.108%

Productivity per hour = 0.00202057% per hour

Daily productivity crashes=0.01798307% per day

Overtime hours will result in a decrease in daily productivity experienced by workers from normal productivity. This is caused by several factors, such as worker fatigue, limited visibility, and cooler temperatures at night.

Table 10: Productivity Coefficient of Overtime Worker.

Extra time	Productivity Decrease Index	Decreased Work Performance (per hour)	Percentage of Productivity Decreased Job Performance (%)	Productivity Coefficient
a	B	c = a x b	d = c x 100	E = 100%-d
1	0,1	0,1	10	0,9
2	0,1	0,2	20	0,8
3	0,1	0,3	30	0,7

After getting the productivity coefficient, the next step is to calculate the crash duration of each activity. The following tables are the results of crash duration calculations for each activity.

$$\text{Crash Duration} = (\text{work weight}) / (\text{Daily productivity after crash}) \quad (2)$$

Table 11: Crash Duration of One Hour Overtime.

Activity	Duration	Quality	Crash Duration (Days)
	Days	%	1 Hour
Stake out dan Positioning	47	0.0580	42
Work Dump Soil	124	4.1640	111
Work Pile Rigid	28	2.0250	25
Work Rigid	35	0.8560	31
Work Paving Block	35	0.3790	31
Structure	91	2.4610	82
Work Building Support	126	2.6146	113
Work Mechanical	154	0.5201	138
Work Plumbing	154	0.3050	138
Work Hoist Crane	28	0.0290	25
Work Electrical	91	5.2886	82
Testing And Commissioning	127	0.1076	114

Table 12: Crash Duration of Two Hours Overtime.

Activity	Duration	Quality	Crash Duration (Days)
	Days	%	2Hour
Stake out & Positioning	47	0.0580	39
Work Dump Soil	124	4.1640	103
Work Pile Rigid	28	2.0250	23
Work Rigid	35	0.8560	29
Work Paving Block	35	0.3790	29
Structure	91	2.4610	76
Work Building Support	126	2.6146	105
Work Mechanical	154	0.5201	128
Work Plumbing	154	0.3050	128
Work Hoist Crane	28	0.0290	23
Work Electrical	91	5.2886	76
Testing And Commissioning	127	0.1076	106

Table 13: Crash Duration of Three Hours Overtime.

Activity	Duration	Quality	Crash Duration (Days)
	Days	%	3Hour
Stake out dan Positioning	47	0.0580	37
Work Dump Soil	124	4.1640	98
Work Pile Rigid	28	2.0250	22
Work Rigid	35	0.8560	28
Work Paving Block	35	0.3790	28
Structure	91	2.4610	72
Work Building Support	126	2.6146	100
Work Mechanical	154	0.5201	122
Work Plumbing	154	0.3050	122
Work Hoist Crane	28	0.0290	22
Work Electrical	91	5.2886	72
Testing And Commissioning	127	0.1076	101

B. Crash Cost Calculation

Addition to the cost of acceleration (crash cost) made on the crash program carried out at the direct cost (direct cost), which is done in addition to labor costs due to overtime. Indirect costs can be assumed to be the same as the budget plan (RAB) obtained. The wages for workers on the Kijing terminal construction project in Mempawah, West Kalimantan are as follows:

Salary per day (normal) = Rp140,000.00
 Hourly salary (normal) = Rp. 17,500.00
 Salary per day (1 hour crash) = Rp166,250.00
 Salary per day (2 hour crash) = Rp.201,250.00
 Salary per day (3 hour crash) = Rp236,250.00

After calculating workers' salaries by adding overtime hours, we can find the total cost of manpower due to the addition of overtime hours with the following calculation: Total crash cost = Subcontract & material costs + total manpower wages. Here is a table of the results of the

calculation of the crash cost for each activity on the critical path with each additional overtime hours.

Table 14, 15, and 16 show the crash cost calculation results for each activity on the critical path with each additional overtime.

C. Cost Slope Calculation

In accelerating using the time cost trade off method, it is necessary to find the lowest cost slope to find out the optimum overtime clock scheme. The following is a cost slope calculation performed on the acceleration of the critical path with the addition of overtime hours.

$$\text{Cost slope} = (\text{Crash Cost}-\text{Normal Cost}) / (\text{Normal Duration}-\text{Crash Duration}) \quad (3)$$

After calculating the cost slope for each scheme to add overtime hours, a cost slope of Rp 6,169,070 per day is obtained for the addition of 1hour overtime, Rp 6,822,825 per day for the addition of 2 hours overtime, and Rp 8,587,193 per day for addition of 3 hours overtime. So, the addition of 1 hour overtime is the most optimum scheme obtained in this study. A comparison can be seen in the table 17 and figure 7.

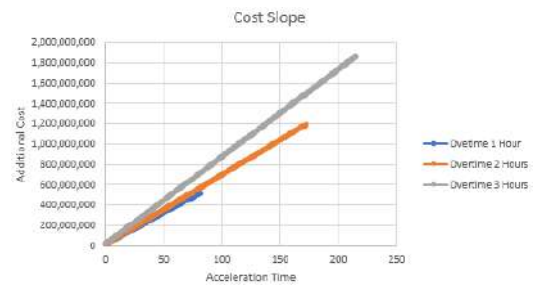


Figure 7: Cost Slope graph for each additional overtime hours.

5 CONCLUSION

From the results of the analysis of the acceleration of the duration of the Kijing sea terminal development project in Mempawah, West Kalimantan using the PDM method, Monte Carlo and the Time Cost Trade Off method, the following conclusions are obtained:

1. Activities that are on the critical path are activities that have a total float = 0. And activities that have a total float = 0 are as follows: Stake Out And Positioning (A3) - Landfill Work (E1) - Rigid Concrete Work (E2) - Rigid (F1) Work - Paving Block (F2) Work - Structure (G1) –
2. Supporting Buildings (G2) - Mechanical Work

- (H1) - Plumbing Work (H2) - Hoist Crane Work (H3) - Electrical Work (H4) - Testing and Commissioning (H5).
- The probability of the Kijing sea terminal development project in Mempawah, West Kalimantan being completed on time with a duration of 595 days based on the project contract is 55%, or the project has a 45% chance of being late. Based on the results of the Monte Carlo method, the Kijing sea terminal development project in Mempawah, West Kalimantan can be completed on time with a duration of 606 days.
 - The optimum duration and costs on the critical path are obtained by adding 1 hour of overtime. The total duration of the project which was originally 595 days to 513 days with a difference of 82 days, with a minimum total cost of Rp2,740,269,901,670 to Rp 2,740,775,765,420 with a difference of Rp505,863,750 and also The smallest cost slope of the other overtime hours scheme is IDR 6,169,070 per day.

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APPENDIX

Table 14: Crash Cost of One Hour Overtime.

Activity	Sub-cont&material cost	Man power	Manpower Total (Crash)	Total Crash Cost
	Rp		Rp	Rp
Stake out & Positioning	1.530.136.543	9	70.323.750	1.589.356.543
Work Dump Soil	113.740.278.706	21	432.915.000	114.104.838.706
Work Pile Rigid	55.470.865.509	5	23.275.000	55.490.465.509
Work Rigid	23.422.410.358	7	40.731.250	23.456.710.358
Work Paving Block	10.351.322.927	7	40.731.250	10.385.622.927
Structure	67.234.202.280	16	242.060.000	67.438.042.280
Work Building Support	71.329.576.849	18	377.055.000	71.647.096.849
Work Mechanical	13.691.583.759	26	665.665.000	14.252.143.759
Work Plumbing	7.797.263.200	26	665.665.000	8.357.823.200
Work Hoist Crane	775.078.271	5	23.275.000	794.678.271
Work Electrical	144.718.074.020	16	242.060.000	144.921.914.020
Testing And Commissioning	2.628.490.414	18	380.047.500	2.948.530.414

Table 15: Crash Cost of Two Hours Overtime.

Activity	Sub-cont&material cost	Manpower	Manpower Total (Crash)	Total Crash Cost
	Rp		Rp	Rp
Stake out & Positioning	1.530.136.543	9	85,128,750	1,615,265,293
Work Dump Soil	113.740.278.706	21	524,055,000	114,264,333,706
Work Pile Rigid	55.470.865.509	5	28,175,000	55,499,040,509
Work Rigid	23.422.410.358	7	49,306,250	23,471,716,608
Work Paving Block	10.351.322.927	7	49,306,250	10,400,629,177
Structure	67.234.202.280	16	293,020,000	67,527,222,280
Work Building Support	71.329.576.849	18	456,435,000	71,786,011,849
Work Mechanical	13.691.583.759	26	805,805,000	14,497,388,759
Work Plumbing	7.797.263.200	26	805,805,000	8,603,068,200
Work Hoist Crane	775.078.271	5	28,175,000	803,253,271
Work Electrical	144.718.074.020	16	293,020,000	145,011,094,020
Testing And Commissioning	2.628.490.414	18	460,057,500	3,088,547,914

Table 16: Crash Cost of Three Hours Overtime.

Activity	Sub-cont&material cost	Manpower	Manpower Total (Crash)	Total Crash Cost
	Rp		Rp	Rp
Stake out & Positioning	1.530.136.543	9	99,933,750	1,630,070,293
Work Dump Soil	113.740.278.706	21	615,195,000	114,355,473,706
Work Pile Rigid	55.470.865.509	5	33,075,000	55,503,940,509
Work Rigid	23.422.410.358	7	57,881,250	23,480,291,608
Work Paving Block	10.351.322.927	7	57,881,250	10,409,204,177
Structure	67.234.202.280	16	343,980,000	67,578,182,280
Work Building Support	71.329.576.849	18	535,815,000	71,865,391,849
Work Mechanical	13.691.583.759	26	945,945,000	14,637,528,759
Work Plumbing	7.797.263.200	26	945,945,000	8,743,208,200
Work Hoist Crane	775.078.271	5	33,075,000	808,153,271
Work Electrical	144.718.074.020	16	343,980,000	145,062,054,020
Testing And Commissioning	2.628.490.414	18	540,067,500	3,168,557,914

Table 17: Calculation of Cost Slope.

Activity	Sub-cont&material cost	Manpower	Manpower Total (Crash)	Total Crash Cost
	Rp		Rp	Rp
Stake out & Positioning	1.530.136.543	9	85,128,750	1,615,265,293
Work Dump Soil	113.740.278.706	21	524,055,000	114,264,333,706
Work Pile Rigid	55.470.865.509	5	28,175,000	55,499,040,509
Work Rigid	23.422.410.358	7	49,306,250	23,471,716,608
Work Paving Block	10.351.322.927	7	49,306,250	10,400,629,177
Structure	67.234.202.280	16	293,020,000	67,527,222,280
Work Building Support	71.329.576.849	18	456,435,000	71,786,011,849
Work Mechanical	13.691.583.759	26	805,805,000	14,497,388,759
Work Plumbing	7.797.263.200	26	805,805,000	8,603,068,200
Work Hoist Crane	775.078.271	5	28,175,000	803,253,271
Work Electrical	144.718.074.020	16	293,020,000	145,011,094,020
Testing And Commissioning	2.628.490.414	18	460,057,500	3,088,547,914

Hydrostatic Stiffness as Displacement Boundary Condition of Floating Cylindrical Structural Analysis in Waves

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Keywords: Hydrostatic Stiffness, Displacement Boundary Condition, Structural Analysis, Natural Period.

Abstract: Motion analysis is one of the mandatory aspects to predict the performance of a floating structure, as well as how its structural strength under certain wave load. However, in majority of floating body performance prediction, the calculation of motion and strength performance is done separately. Practically, engineers calculate the motion and hydrodynamics forces that work on the structure, then do separate calculations on the structure to predict structure's strength. These separate calculations often use assumptions that tend to be unrealistic, either over-constrained or under-constrained. This paper provides an alternative to the constraint problem by introducing hydrodynamic stiffness as boundary conditions, instead of using fixed or simply supported boundary conditions, spring boundary conditions are applied with hydrodynamic stiffness of floating body properties. It is expected that this model provides a more realistic constraint to the future analyses. The results achieved are very promising, where the boundary condition resulting a close natural frequency approximation compared with the analytical calculation. This configuration is hoped to be the baseline of more complex structure to be carried out in future research, in order to represent a more realistic structural displacement boundary condition.

1 BACKGROUND

Motion analysis is one of the mandatory aspects to predict the performance of a floating structure, as well as how its structural strength under certain wave load. However, in majority of floating body performance prediction, the calculation of motion and strength performance is done separately.

Practical engineering software package tends to disintegrate calculation of motion and hydrodynamics forces that work on the structure for used to assess the strength of particular floating body.

Traditionally, engineers consider the ship structure as fixed ends beam (Okumoto, et al., 2009) or simple beam (Molland, 2008).

Several researches on analytical level proposed the methods to incorporate 'sea springiness' of floating body during strength analysis. There are researches conducted to integrate Computational Fluid Dynamics (CFD) and Finite Element Analysis (FEA) via Fluid Structure Interaction (FSI) software packages. ANSYS, for instances, is one of the established software packages that used for this intention. In maritime application, various vessel forms has been used as object. For example,

composite ship structures (Ma & Mahfuz, 2012), horizontal cylinder (Raja, 2012) and ocean energy harvesting device (Agamloh, et al., 2008). Several open source software such as OpenFoam has also been used for the same intention. Wave-structure interaction method has been developed using OpenFoam (Chen, et al., 2014).

Still, the performed researches are still focused on the fluid interaction and tend to disregard the displacement boundary condition aspects. Majority of the those only consider the displacement boundary condition as buoyancy versus gravity only.

Recent studies provide the hydrostatic stiffness for linear hydroelasticity. The explicit formulation for the complete hydrostatic stiffness for flexible floating structures at rest in calm water is derived based on a consistent linearization of the external hydrostatic pressure and the internal structural stresses (Huang & Riggs, 2000). It is also found that the hydroelasticity formula deals with more terms, and, that under some assumptions, it is reduced to the known complete restoring stiffness (Senjanović, et al., 2011).

This paper introduces the practical hydrostatic stiffness to be used directly as displacement boundary condition of rigid floating body. Analytical

calculation is introduced as an approach to assess the application of ground spring to a floating cylinder. This method is hoped to be an applied practical guide to model a better and more realistic displacement boundary conditions for those who cannot afford the luxury of FSI study.

The proposed method has been initially developed by American Bureau of Shipping, depicted at ABS Floating Production Installation (ABS, 2014), only for ship shaped structures. Authors inspired by related theory explained at ABS FPI Part 5A, Chapter 3, Appendix 4, Point 17, to be used as basis of so-called analytical-practical approach of cylindrical floating structure's displacement boundary condition.

2 UNCOUPLED HYDRODYNAMIC MOTION

Based on classical theory as commonly known, the free-floating body has six degree of freedom in hydrodynamics motion, namely (Bhattacharya, 1978):

- 1 Surging = motion backward and forward in the direction of ship travels
- 2 Swaying = athwartship motion of ship
- 3 Heaving = motion vertically up and down
- 4 Rolling = angular motion about longitudinal axis (X axis). Traditionally, the angular motion alternating from portside to starboard and vice versa
- 5 Pitching = angular motion about the transverse axis (Y axis). Traditionally, the angular motion alternating from bow to stern and vice versa
- 6 Yawing = angular motion about the vertical axis (Z axis)

Above list of motion is illustrated by Figure 1 below

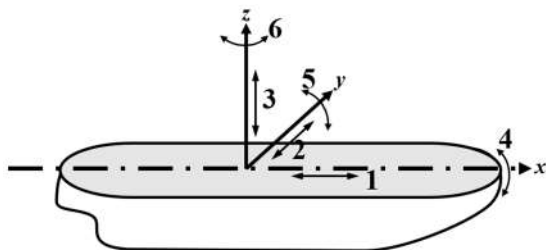


Figure 1: Six Degree of Freedom Hydrodynamic Motion.

2.1 Uncoupled Hydrodynamic Motion of Floating Cylinder

In this paper we limit the discussion only for cylindrical structure, which is a bi-symmetrical structure. Hence, the aforementioned hydrodynamic motion can be reduced due to similarities, into following motions:

1. Surging = swaying, with similar X and Y translation motion.
2. Pitching = rolling, with similar X and Y rotation motion.
3. Yawing, due to the bi-symmetrical structure, the Z rotation is considered negligible.

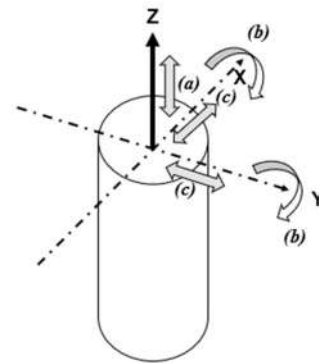


Figure 2: Floating Cylinder Motion.

Figure 2 above explains the considered motions of floating cylinder. As it can be seen, letter (a) coded the heaving motion, while (b) coded the rolling motion, and finally we have (c) coded for swaying motion. (a) and (c) are the translational motions of the cylinder, with equation described below:

$$m\ddot{u} + c\dot{u} + ku = F_0 t \quad (1)$$

Where:

- $m\ddot{u}$ = translational inertial force
- $c\dot{u}$ = translational damping force
- ku = translational restoring force
- $F_0 t$ = translational excitation force

Inertial force for translational motion, is present when the cylinder is in oscillatory motion, consist of m (cylinder mass plus hydrodynamic added mass) multiplied by $\ddot{u}$, the motion acceleration for translational.

Damping force, is the force to resist the motion. This force consists of damping coefficient c and translational velocity, $\dot{u}$.

Restoring force is the spring force that brings back the cylinder into its equilibrium position.

Restoring force is composed of k , the hydrostatic stiffness of the motion, multiplied by u for translational motion, which is the translation of cylinder's Centre of Gravity (CoG)

Furthermore, (b) is the rotational motion of the cylinder, with equation described below:

$$m\ddot{\theta} + c\dot{\theta} + k\theta = F_0 t \quad (2)$$

Where:

$m\ddot{\theta}$ = rotational inertial moment

$c\dot{\theta}$ = rotational damping moment

$k\theta$ = rotational restoring moment

$F_0 t$ = rotational excitation moment

Inertial moment for translational motion, is present when the cylinder is in oscillatory motion, consist of m (cylinder mass plus hydrodynamic added mass) multiplied by $\ddot{\theta}$, the motion angular acceleration for rotation.

Damping moment, is the moment to resist the motion. This moment consists of damping coefficient c and rotational motion angular velocity, $\dot{\theta}$.

Restoring moment is the spring moment that brings back the cylinder into its equilibrium position. Restoring moment is composed of k , the hydrostatic stiffness of the motion, multiplied by θ for translational motion, which is the rotation of cylinder's Centre of Gravity (CoG)

This stiffness properties, both for translational and rotational motion, are used for the ground spring stiffness, to represent the actual condition when we perform the structural analysis of the cylinder.

2.2 Hydrostatic Stiffness of Heaving Motion

As stated at equation 1 above, the hydrostatic stiffness of heaving is used for vertical translational spring stiffness. The heaving stiffness is the waterplane area of cylinder multiplied by the water specific weight (Patel & Witz, 1991), consequently, the hydrostatic stiffness of cylinder heaving motion is as follows:

$$k = \gamma \pi r^2 \quad (3)$$

Where:

k = heave stiffness

γ = water specific weight

πr^2 = waterplane area of cylinder

The spring stiffness is attached at the bottom of the cylinder, to represent the restoring motion of heaving.

2.3 Hydrostatic Stiffness of Rolling Motion

At the same time, for rotational motion, the hydrostatic rolling stiffness is used. Rolling stiffness is the righting moment of the cylinder. The righting moment at any particular angle of inclination is expressed as:

$$k\theta = \Delta \overline{GZ} \quad (4)$$

For small angle of inclination (in radians):

$$k\theta \cong \Delta \overline{GM_T} \theta \quad (5)$$

Hence the hydrostatic stiffness of rolling motion:

$$k = \gamma \nabla \overline{GM_T} \quad (6)$$

Where:

k = rolling motion stiffness

γ = water specific gravity

∇ = water displacement

$\overline{GM_T}$ = metacentre height of cylinder

3 GROUND SPRING ELEMENT

The ground spring method has very long tradition to be included in dynamic analysis of structures. It is commonly used in seismic analysis to model the damping and stiffness of soil-pile interaction (Datta, 2010). Unlike in hydrodynamic analysis, there are a lot of established coefficient that model the spring and dashpot for a variety of foundation types and soil conditions (Gazetas, 1991). Figure 3 below shows the example of ground spring applied to a building (Datta, 2010).

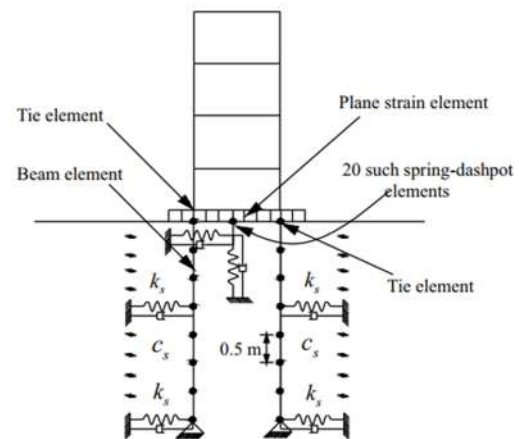


Figure 3: Example of Spring-Dashpot Equivalent Method to a Building (Datta, 2010).

The same concept applied to a floating cylinder with the spring stiffness from the hydrostatic properties of the body. In this paper, we only consider the stiffness properties of spring.

4 CASE STUDY: DISPLACEMENT BOUNDARY CONDITION OF A FLOATING CYLINDER

A simple cylindrical structure is presented to examine the usability of the ground spring applied to hydrodynamic case. Figure 4 below shows the proposed ground spring placement to create equivalent spring arrangement of hydrostatic stiffness.

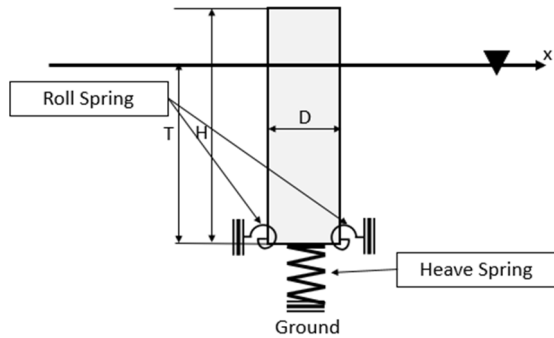


Figure 4: Equivalent Spring Arrangement.

4.1 Problem Setting

The proposed configurations above is treated as structural system with precalculated heave and roll spring as mentioned before at Chapter 2.2 and 2.3.

The heave and roll spring are placed at the bottom of the cylinder, assumed that the support is located at the bottom of the cylinder. This configuration is then compared with traditionally 'fixed' boundary condition at the bottom of the cylinder. In this paper, we only compare the 1st order natural period of heaving and rolling for:

1. Analytical hydrodynamic model
2. Rigid body-equivalent spring arrangement model
3. Rigid body-traditional fixed boundary condition arrangement model.

In analytical hydrodynamic model, the mass is calculated as addition of water displacement and added mass (Sarpkaya, 2010). Whereas the mass for rigid body arrangement, both for spring and boundary

condition model, are modelled as its real mass, instead of displacement and added mass. It is important to calculate the mass with aforementioned method, to check whether the 'dry' models, which represented by rigid body model, can imitate the natural frequency of 'wet' model, which represented by hydrodynamics model.

Heave and roll spring stiffness are calculated as mentioned in Chapter 2.2 and Chapter 2.3 respectively, and then placed at subsequent arrangement as depicted in Figure 4.

4.2 Cylinder Diameter, Height and Draught

The cylinder diameter (D) and height (T) are varied with value $0.1 \leq D/T \leq 1.0$. while the draught (T) is set as 0.8H.

4.3 Natural Period

The natural period of heaving and rolling will be calculated for the three configuration variations. The natural period for 'wet' arrangement is calculated as below for heaving motion:

$$T_n = 2\pi \sqrt{\frac{m + m_{AZ}}{k_z}} \text{ (s)} \quad (7)$$

Where:

- m = real mass
- m_{az} = added mass for heaving motion
- k_z = heaving motion stiffness

And for rolling motion:

$$T_n = 2\pi \sqrt{\frac{I + I_{ar}}{k_r}} \text{ (s)} \quad (8)$$

Where:

- I = real inertial rolling motion
- I_{ar} = added inertial rolling motion
- k_r = rolling motion stiffness

The natural period for heaving motion for 'dry' arrangement is calculated as below:

$$T_n = 2\pi \sqrt{\frac{m}{k_z}} \text{ (s)} \quad (9)$$

Where:

- m = real mass
- k_z = spring stiffness for heaving motion

The natural period for rolling motion for 'dry; arrangement is calculated as below:

$$T_n = 2\pi \sqrt{\frac{I}{k_r}} \text{ (s)} \quad (10)$$

Where:

I = real inertial rolling motion
 k_r = rolling motion stiffness

5 RESULT AND DISCUSSION

5.1 Hydrostatic Stiffness

Hydrostatic stiffness is calculated and used as spring stiffness and adequately inputted as spring stiffness at each motion. The natural period of each motion is then calculated and discussed as below.

5.2 Heaving Motion Natural Period

Figure 5 below shows the natural period characteristics for each boundary condition arrangement. First of all, the hydrodynamic natural period is calculated for each D/H, represented by triangle dots. The natural period increases with the increase of the D/H. Then the rigid body motion is calculated. Fixed boundary condition gives very low natural period, which is near to zero, and considered as unrealistic boundary condition due to the very wide gap between the hydrodynamic and this boundary condition.

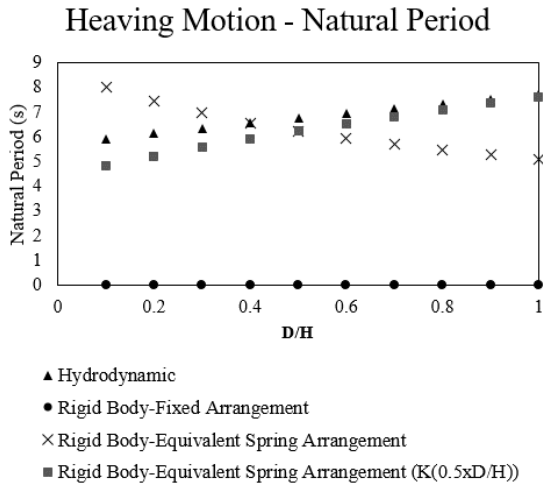


Figure 5: Heaving Motion Natural Period for Each Arrangement.

Equivalent spring motion natural period is calculated and presented at Figure 5 by the cross dots. The pattern of the equivalent spring natural period is rather different with the hydrodynamic one, but still in the same region. Maximum natural period for equivalent spring is at D/H=0.1, where the value is 8.034 s. Where the maximum natural period for hydrodynamics is D/H=1.0, the value is 7.683 s.

Adjustments is made to equivalent spring stiffness. In order to imitate the hydrodynamics properties, the spring stiffness is calculated by adding hydrostatic stiffness with the multiplication of the half ratio between diameter and height.

The value of the equivalent spring natural period is then matched with the hydrodynamics natural period, as explained by square dots at Figure 5. It turns out that by multiplying the hydrostatic stiffness with $0.5xD/T$, the natural character of spring arrangement is similar to the hydrodynamic characteristic.

Similar natural period can be achieved by arranging the spring as shown at Figure 4 for the heaving motion of cylinder, by applying below equation for the spring stiffness:

$$K_{ez} = \frac{2\gamma\pi r^3}{2H} \quad (11)$$

Where:

K_{ez} = heave equivalent spring stiffness
 γ = water specific weight
 r = cylinder diameter
 H = cylinder height

5.3 Rolling Motion Natural Period

Figure 6 below explains the natural period characteristics for every boundary condition arrangement. In the beginning, the hydrodynamic natural period is analysed for each D/H, symbolized by the triangular dots. Similar with heaving motion, the natural period increases with the increase of the D/H. The following result, which is rigid body motion, is calculated. Again, identical with heaving motion, fixed boundary condition gives very low natural period, which is near to zero. This boundary condition is considered as unrealistic due to the very wide gap between the hydrodynamic and fixed arrangement.

Equivalent spring motion natural period for rolling motion is calculated and presented at Figure 6 by the cross dots. The pattern of the equivalent spring natural period is similar with hydrodynamics motion but resulting rather higher period.

Maximum natural period for equivalent spring is at $D/H=1.0$, where the value is 6.344 s. Where the maximum natural period for hydrodynamics is $D/H=1.0$, the value is 2.006 s. It turns out that the difference between equivalent spring and hydrodynamic natural characteristic can be normalized by dividing the rolling natural period by the ratio between the diameter and the height.

The value of the equivalent spring natural period is then matched with the hydrodynamics natural period, as explained by square dots at Figure 6. It turns out that by dividing the rolling hydrostatic stiffness by D/T , the natural character of spring arrangement is similar to the hydrodynamic characteristic for rolling motion.

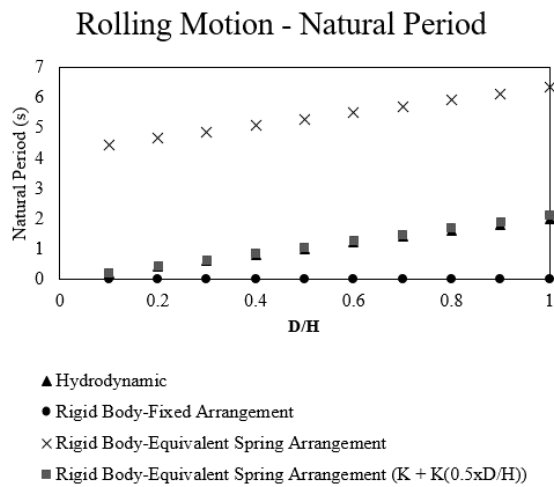


Figure 6: Rolling Motion Natural Period for Each Arrangement.

Similar natural period can be achieved by arranging the spring as shown at Figure 4 for the rolling motion of cylinder, by applying below equation for the spring stiffness:

$$K_{er} = \gamma \nabla \overline{GM}_T \left(1 + \frac{r}{H}\right) \quad (12)$$

Where:

- K_{er} = roll equivalent spring stiffness
- γ = water specific gravity
- ∇ = water displacement
- $\overline{GM}_T$ = metacenter height of cylinder
- r = cylinder radius
- H = cylinder height

6 CONCLUSION

After going with the explained procedures to create equivalent spring arrangement for heaving and rolling motion, especially to singular cylinder, we can draw conclusions as follow:

1. The value of the equivalent spring natural period matches with the hydrodynamics natural period by dividing the rolling hydrostatic stiffness by D/T the hydrodynamic characteristic for rolling motion.
2. For heaving motion, in order to imitate the hydrodynamics properties, the spring stiffness is calculated by adding hydrostatic stiffness with the multiplication of the half ratio between diameter and height.

7 FURTHER WORKS

The future works should refine the hydrostatic stiffness modelling by considering the ground spring height.

Further works also should develop more hydrostatic equivalent spring stiffness for more complex structure, e.g.: boxes, multiple cylinders, and ship shaped structures.

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Identification of Coastal Problem along the East Coast of Lampung Indonesia

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Keywords: Coastal Problem, East Coast, South Lampung.

Abstract: The dynamics of nature occurring in recent years, as well as climate change and the increasingly severe pressures of humans have had a noticeable impact on the coastal conditions. This research aims to present the results of identification of various damages that occurred along the East Coast of South Lampung Regency of Lampung Province with the method of field surveys through observation, measurement, study of supporting data and previous research. This study was conducted with several activities, namely field observation, interviews and documentation of conditions for damage to coastal areas. The final result of this identification is to describe: (a) the type of damage occurring; (b) Cause of damage; (c) The impact of any damages incurred; (d) Alternative countermeasures; and (e) The location coordinates are damaged. The results showed that the sedimentation process and damage to coastal structures occurred at the location of Ketapang Laut village. Sedimentation also occurs in the estuary of Ketapang Laut village of South Lampung to cause the presence of land arises, the sediment of river estuary and the coastline. Another problem is the decrease of mangrove forest area which is one caused by the opening of the area to be used as shrimp pond.

1 INTRODUCTION

The problem of coastal areas is a condition that inflict damage to coastal areas caused by natural processes or caused by human activities. The factors that cause damage to coastal areas can naturally come from the influence of the hydro-oceanographic processes occurring in the sea that can cause waves to wave, resulting in sedimentary transport, changes in current patterns, tidal variations, and climate change (Fadilah et al, 2013).

Some locations along the coast are damaged, especially in physical forms such as coastal line changes, either abrasion/erosion and accretion/sedimentation. East Coast of South Lampung Province is one of the coastal areas that is not separated from the general problems of the beach is erosion and sedimentation. The change of coastline is a process that lasts continuously through a variety of processes both erosion (abrasion) and additions (accretion) resulting from the movement of coastal sediments, longshore current, wave action and land use (Arief et. al., 2011).

Coastal erosion is a result of human activities and natural environment changes making the coastal

dynamic action (wave, current, wind) lose balance in the coastal process, and the long-term loss of sediments of coastal zone results in the destruction process of coastline retreat and beach erosion (Yin-can, 2017). Accretion or sedimentation is the siltation or addition of coastal land due to the deposition of sediment brought by sea water. The sedimentation process can take place naturally from the sedimentation process and freshwater flow, as well as those caused by land-based human activities such as deforestation and pollution. Moreover, accretion can also harm the coastal community, because in addition to influencing the stabilization of the coastline, accretion can also cause the traffic to the river estuary where the ship and the boat.

Coastal damage can be assessed objectively by using special criteria for coastal damage assessment. Based on the SE KEMENPUPR No. 08/SE/M/2010 it is written that the criteria of damage to the coast used is divided into 3 (three) kinds such as the criteria of coastal environmental damage, erosion criteria and building damage and sedimentation criteria.

This research aims to present the results of identifying various damages that occurred along the east coast of South Lampung Regency with the

method of surveying the survey through observation, measurement, study of supporting data and previous research.

2 METHODOLOGY

The data collected consists of primary data and secondary data. Primary Data was obtained using a survey method through field observation, deep interview with several respondents selected purposive and incidental at the location of coastal areas and damage documentation along the coastal areas. The data collection instruments used in this study are questionnaire, interview guidelines and cameras. Secondary Data is obtained using the literature study method.

The scope of the damage surveyed is all damages in the east Coast region of South Lampung Regency Lampung Province which can be observed and identified directly without requiring advanced analysis/analysis of laboratories (in this case the survey conducted excluding underwater ecosystems such as coral reefs and seagrass).

A large survey conducted to observe the damage occurring along the coast, especially the region in the coastal areas in South Lampung regency. The final result of this identification can describe: (a) the type of damage occurring in an area or coastal location; (b) Causes of damage in an area or coastal location; (c) The impact or consequences of any damage incurred in an area or coastal location; (d) Damage/rehabilitation alternative to damaged coastal areas; and (e) the location of details (location coordinates) of the coastal areas that have been damaged. To analyze the data already obtained used a descriptive analysis method.

2.1 Area of Study

This research was conducted along the east coast of South Lampung Regency, Lampung province. Live observation at the research site is done by going down the east Coast along ± 17 km in 1 (one) sub- district that is Ketapang which is one of the sub- district close to the access to the road of Bangun Rejo, shown on Figure 1.

2.2 Field Surveys

Collection of research data was conducted in May 2019 by walking through the beach using GPS, digital photos, and video cameras as well as conducting an interview method directly with the community in the



Figure 1: Area of study.

settlement along the coast. The results of this observation are also supported with secondary data in the form of the previous research, namely hydro-oceanographic data that includes wind data, wave data, sediment data, bathymetry data as well as various maps and other data needed as map topography, administration, land use, population data and so on that has been obtained from related agencies.

3 RESULTS

The process of sedimentation and damage to this coastal structure occurred at the location of Ketapang Laut village. The location of the sedimentation beach is one of the locations that belong to the East Coast region of Lampung province South Lampung and is located at the location of the coordinates $5^{\circ}44'26.05''$ S and $105^{\circ}47'48.21''$ E shown on Figure 2.

Based on information obtained from the previous study obtained information from the *Pekerjaan Proyek Pesisir* (1998) that the beach is experiencing sedimentation process and this process tends to be found many along the east Coast region of Lampung province. This is because there are many large river estuary along the coastal areas so that the source of sediment comes from the mud that is carried away from the upstream area and precipitates in the estuary.

In addition to the sedimentation problem the field survey results also showed that the sedimentation process that occurs constantly causes damage to



Figure 2: Sedimentation and coastal structure damages.

coastal structure. This type of coastal structure is revetment. Revetment is a coastal structure that is built on coastline and is used to protect the coast of Dai wave attacks and runoff waves (overtopping) to the ground (Triatmodjo, 2012). Revetment has a sloped side and can be made from a pile of rocks or bronzes, making it more flexible and adapts to the scours at the foot of the building. The revetment that functioned as a coastal protective structure was damaged because of the buildup of sediment in the concrete pile that formed the presence of new land. If it is reviewed based on the characteristic of the east coast of Lampung province that has the same characteristics as the north coast of Java (Sulaiman, 2018) that the coastal area is characterized by the low slope of the mud and swamp overgrown mangrove and has a very fine grain size, it is very possible once the dominant sedimentation process occurs in this location. The condition of the beach with muddy characteristics is also very related to the wave energy is very low, but the tide is greater. Based on the results of the research conducted by Tarigan et al. (2020) that the East Coast of South Lampung has a characteristic wave height which ranges from 0.36 m-2.57 m. East coast of South Lampung Regency that has a character sedimentary mud affects the sediment process that tends to undergo sedimentation process in the coastal area (Tarigan et al., 2020).

The process of in the village of Ketapang Laut South Lampung District, this led to the presence of land arising causing the increase of river estuary and the coastline. The ignition of the estuary that occurs can clog the stream, pollution, the fishing vessel traffic at low tide and on the other side when the tide can cause the widespread water over the bank.

People in the village of Ketapang Laut South Lampung District overcome the blockage of the river by doing simple dredging such as making a hole that



(a)



(b)

Figure 3: (a) and (b) Sedimentation.

functions to drain the flow of the river to the beach that shown on Figure 3(a) and Figure 3(b).

Based on the condition of the field obtained information that if not in the rainy weather conditions, the river discharge is relatively small so that the current speed is not able to erosion. The deposition to a clever consequence of the river estuary completely covered sedimentary deposits.

This location is very close to the residential area of residents and also the processing place of salty fish that have waste, under certain conditions some residents who live around the area is disturbed by a puddle that sometimes carries a waste of salty fish processing that creates unpleasant odor so that the citizens will be able to join the place in the mouth-mouth of the river, especially the stream is small.

To solve the problem, it is necessary to do the treatment to take the entry of sediment to the estuary of the river. The bank's estuary management strategy can be determined where the sedimentation of the

river mouth is the simplest and least inexpensive alternative handling (Triatmodjo, 1999). Dredging can be done at the beginning of the rainy season, either by mutual assistance by local residents or by using the help of heavy equipment if possible in the way of inspection. To get the maximum flow, a large volume of dredging is required. The difficulty of this alternative is the re-enlarger groove that is completed scraped, especially when the wave with greater energy and also if the interval between the completion of dredging with the advent of the first flood is still long enough, so that the groove that has been dredge will be closed back by deposits. Triatmodjo (1999) states that when dredging is done by mutual assistance, the population can choose the thinnest and most narrow deposits to be scraped on downstream turns. The purpose is the River estuary remains turned, so the discharge capacity of the addressing is reduced.

The decrease of mangrove forest land cover that occurred in the coastal area East coast of South Lampung district is at the coordinates of $5^{\circ}39'48.76''$ S – $105^{\circ}48'41.48''$ E. Based on the results of field surveys and interviews with local residents, one of the causes of damage to the mangrove forest is caused by the opening of the area to serve shrimp ponds. The field conditions taken in May 2019 are shown shown on Figure 4.

Based on the information quoted from previous research according to resource Atlas of Lampung coastal area (Wiryawan, 1999) coastal area along the coast of East Lampung except Way Kambas National Park occurs the change of land that initially swamp and mangrove forest into rice farming land and shrimp pond. The existence of mangrove forest covering about 81 percent of the beach in Lampung, can provide many benefits, both physically, biologically and economically. But excessive utilization especially on economic utilization can cause damage to ecosystems. However, if the land cover area of mangrove forest will be held continuously, it can cause impact on coastal area and beach.

Some of the impacts that can occur are coastal erosion and river borders, sedimentation, environmental pollution, decline in ecological function that will affect the economic aspect especially for fishermen who are in the East Coast district of South Lampung Regency and will cause sea water intrusion. Policy recommendation to improve the mangrove ecosystem and reduce the impact is to rehabilitate the mangrove ecosystem by actively involving the community.



(a)



(b)

Figure 4: (a) Mangrove forest area; (b) land conversion.

4 CONCLUSIONS

East Coast of South Lampung regency, Lampung Province showed that there are coastal damage that occurred at some point of location and need to get handling to do ecosistence improvement of coastal area and integrated beach. The location that is experiencing the sedimentation process is a location that affects increasing the economy of the local population working as fishermen and entrepreneurial fish processing.

East Coast of South Lampung regency, Lampung province problem is caused by natural and anthropogenic factors. Natural factors in the form of hydro-oceanography can damage ecosystems and buildings on the coastal borders. Factor anthropogenic by community activities that do not comply with local regulations and rules applicable.

Advanced research and studies need to be done to support the results of the Identification of Coastal

Problem along the East Coast of Lampung Indonesia by conducting a coastal protection analysis in accordance with the principles of integrated coastal management.

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Transmission Coefficient Analysis Floating Breakwater using Computational Fluid Dynamics (CFD)

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Keywords: Computational Fluid Dynamics, Floating Breakwater, Porous Breakwater, Transmission Coefficient.

Abstract: Breakwater is one of the coastal structures that was built with the aim to protect the coastal area against waves. This structure is generally designed to protect ships and facilities at the Port. Breakwaters that are often applied are conventional breakwaters such as the rubble mound type. Climate change that causes tidal variations and sea level height which tends to change at any time, as well as unsuitable soil conditions to receive large structural loads will be costly and more economical if using floating breakwater. Over time, the demand for floating breakwater development increased so that scientists and engineers did a lot of research development both in physical experiments and numerical models. In this study numerical simulations will be conducted by exploring the porous shape of floating breakwater from the Christensen experiment. The aim is to get the most optimal transmission coefficient. Numerical simulations using the Computational Fluid Dynamics (CFD) method with the help of Flow 3D Software. Validation is done first between numerical tests and experiments to get a valid approach before exploring the development of shape.

1 INTRODUCTION

Breakwater is one of the coastal structures that was designed to protect ships, marine ecosystems, port facilities, and for coastal protection against waves. Conventional breakwater is generally used in shallow water and the geometry is bottom founded. Unconventional breakwater is known as floating breakwater for the deeper the water. However, there are several advantages using a Floating Breakwater (FB). For example, more environmentally friendly to pollution and sedimentation problems because it does not inhibit water circulation (Dai et. al, 2018), a little effort to move another location (Christensen et. al, 2018). When sea water level (SWL) rises due to tides or climate change, FB can adjust so that is more economical than bottom founded. Further, floating breakwater may be the only solution for high load structures to poor soil conditions.

Floating breakwater (FB) was the first applied in 1811 to protect marinas and ships against waves at the port of Plymouth, England (Hales, 1981). Starting in 20th century, the demand of floating breakwater increased to anticipate infrastructures development of ocean space. So, Scientist and Engineers did a great developing FB research either experimentally,

numerically, or combination of both. Most research is usually done on various geometric shapes, configurations, or bilge keels of FB on the wave characteristics.

Experimentally and numerically of floating breakwaters have been studied. For instance, (Christensen et al., 2018) conducted physical experiments and numerical modeling to evaluate transmission, reflection, and performances of Regular Pontoon (RG), WP, and WP-P100. The study showed that wing plates (WP) reduced the motions of floating breakwater, while wing plates and porous media (WP-P100) reduced the reflection and transmission most effectively. (Ji Chunyan et al., 2018) conducted experimental study for a dual rectangular pontoon floating breakwater with two treatments (single-row and double-row). The result indicated that double-row floating breakwater significantly reduced transmission especially for short-period wave than single-row floating breakwater. (Cho, 2011) investigated transmission of incident waves interacting with vertical porous side plates using matched eigenfunction expansion method (MEEM). (Wang and Sun, 2010) conducted experimental study of a porous floating breakwater with fabricated large numbers of diamond-shaped blocks to reduce

transmitted wave height and mooring force. (Dong et al., 2008) conducted two-dimensional physics model test on three types of structures: the single box, the double box, and the board net floating breakwaters. The result showed that transmission coefficient depend on the width of board, and the length of the mooring chain lying on the sea bottom. (Rahman et al., 2006) conducted numerical study of single pontoon floating breakwater to estimate the nonlinear dynamics using the volume of fluid (VOF) method.

This paper presents numerical study or simulation to analysis transmission coefficients using computational fluid dynamics (CFD) method. The basic geometry is regular ponton floating breakwater that modified in several notched shape on the side of structure. Data parameters and numerical model verifications based on the result of experiments that conducted by (Christensen et al., 2018).

2 NUMERICAL SIMULATIONS

2.1 Floating Breakwater Geometry

Floating breakwater geometry is based on Regular Pontoon (RG) experiment test conducted by (Christensen et al., 2018). The experiment conducted two-dimensional physical model test where cross-sections were tested and analysed in wave flume. The data parameters are shown in Table 1 and illustrated in Figure 1. Furthermore, the experimental model will be developed on porous shape on side plates with numerical simulations using computational fluid dynamics (CFD) to optimization wave transmission. The developed shapes can be seen in Figure 2.

Table 1: Regular Pontoon Dimensions.

No	Geometry	Dimensions [m]
1	Length (L)	0.58
2	Width (W)	0.46
3	Draft (D)	0.31
4	Height (H)	0.39

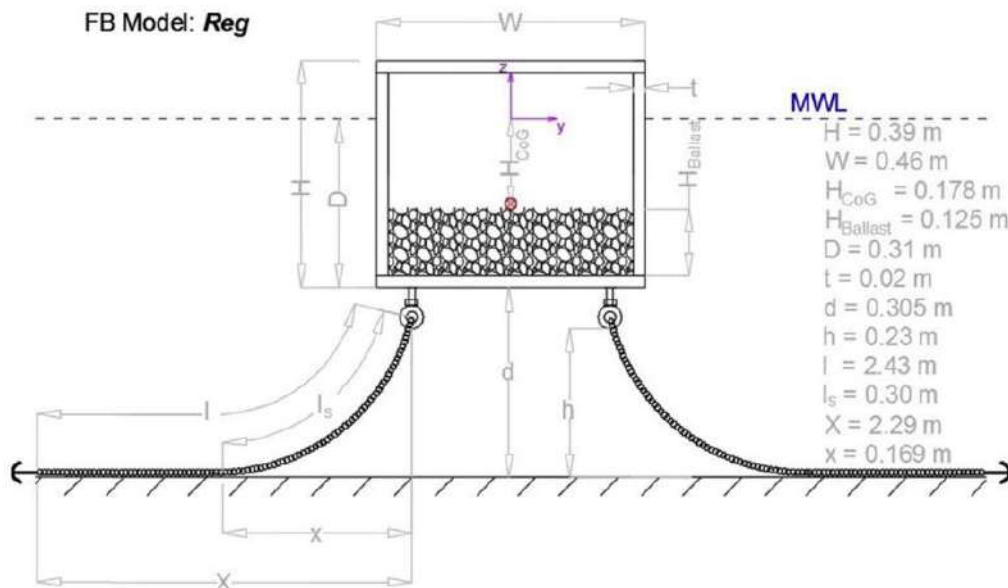


Figure 1: Basic Cross-section Regular Pontoon FB.

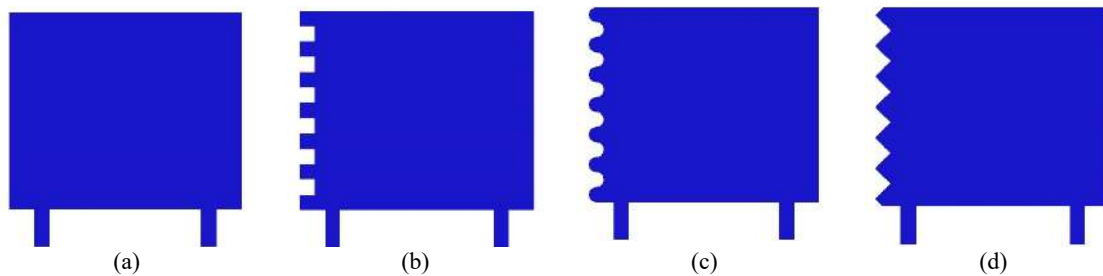


Figure 2: Developed Model Floating Breakwater (Notched Shape).

2.2 Data Parameters

The data were composed of wave parameters that are scaled. The scale is assumed to use Froude scaling by 1:65. Froude scaling was considered valid as long as viscous effects are negligible. The 2nd Stokes wave was used on this simulation. Table 2 shows conditions for numerical simulations.

2.3 The Wave Flume

The flume is 28 m long, 0.6 m wide, 0.8 m high, and 0.615 m initial surface elevation. The flume was filled with water with density of 1000 kg/m³. The flume was equipped with wave maker at one and wave absorber at the other end. Where, the floating breakwaters were placed almost cover the entire width of flume to reduce the effect of sidewalls. See, Fig. 3 for illustration.

2.4 Wave Measurements and Analysis

The objective of this study was to obtain the transmission coefficient (C_T), the ratio of transmitted wave height (H_T) to incident wave height (H_i). See equation 1 below:

$$C_T = H_T / H_i \quad (1)$$

Measurement of transmitted and incident wave heights were represented by surface elevation using wave gauges. 4 wave gauges were placed on the incident side, while 3 wave gauges were placed on the lee side. See figure 4.

2.5 Mooring System

The mooring lines were installed only to keep floating breakwater on position. The mooring system have an effect on the performances of stability and the mooring forces, but in this study no analysis of both. There are four mooring lines on each corner of floating breakwater, two on each side of the cross-section. The submerged weight of the mooring line was $w = 0.589$ N/m. The mooring line made from polyethylene (PE) with density of 880 kg/m³ and diameter of 0.01 m. The dimension of mooring line was illustrated on figure 1.

Table 2: Parameters Wave Data.

No.	Wave Length	Wave Period	Frequency	Depth Ratio	Wave Height
	L[m]	T[s]	f[Hz]	h/L[]	H _(2%)
1	1.174	0.868	1.152	0.524	0.023
2	1.883	1.116	0.896	0.327	0.036
3	2.252	1.240	0.806	0.273	0.041
5	3.333	1.613	0.620	0.185	0.052
6	4.024	1.861	0.538	0.153	0.055

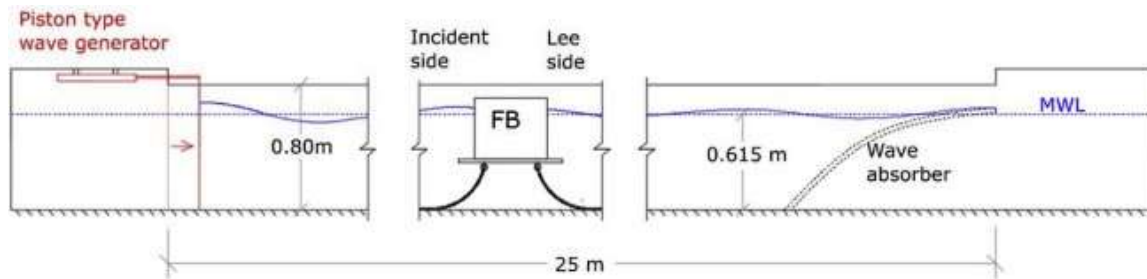


Figure 3: Sketch of the wave flume.

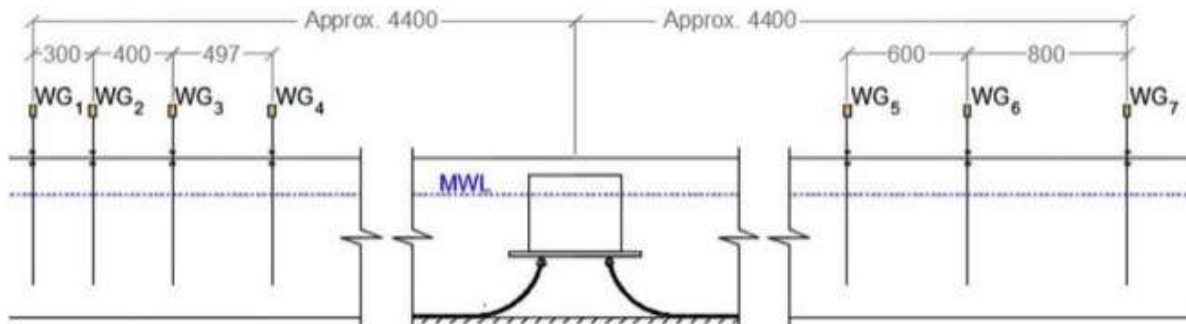


Figure 4: Position Wave Gauges in Unit [mm].

2.6 Boundary Conditions

The purpose of boundary conditions was to determine model conditions that represented experiment conditions. See figure 5. Boundary conditions used in the model are as follow:

- (1) Free surface (Wave): A wave boundary condition was defined at left (X Min). A surface wave entered the computational domain and propagated in the direction normal to the boundary. The wave was set 2nd stokes as wave generator represent the physical wave conditions at the boundary.
- (2) Outflow: The outflow boundary condition was defined at right (X Max). It allowed users to numerically investigate the effects of wave interactions with structures. The capability permitted a reduction in the extent of the computing mesh needed for accurate computations. A wave-absorbing layer used to reduce reflection of periodic wave at an open boundary.
- (3) Symmetry: The symmetry condition was defined at front (Y Min), behind (Y Max), below (Z Min), and up (Z Max). No-slip conditions were imposed using the wall shear-stress options described in the Prandtl Mixing Length model. A symmetry condition can be specified as free-slip conditions that have a non-zero wall shear-stress.

2.7 Meshing

Mesh block was used to determine the area that modelled. The smaller mesh will be more detailed, but the output files will be larger and simulations run longer. Floating breakwater modelling used one mesh block with a meshing size of 0.04 m at total length (X-axis) is 28 m, total width (Y-axis) is 0.6 m, and total height (Z-axis) is 0.8 m.

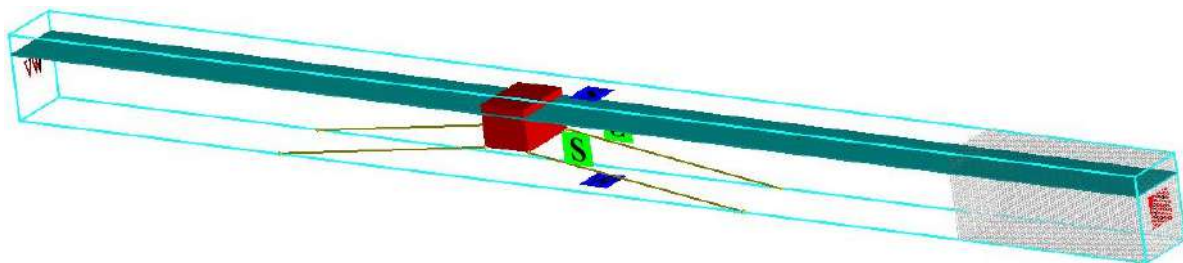


Figure 5: Boundary Conditions Model.

3 RESULTS AND DISCUSSION

3.1 Validation

Validation is done by comparing the results of transmission coefficient of the (Christensen, 2018) study with the numerical study of the Computational Fluid Dynamics (CFD) aided by Flow 3D Software. Christensen has conducted study on three shapes of floating breakwater, but only one shape of floating breakwater validated in this study is Regular Pontoon (RG). The purpose of validation is to find out whether the numerical study conducted is in accordance with the experimental test. Validation is done in a simpler shape (RG). If validation are represented, then the development of the model can be done to more optimal shapes.

In this study, validation is based on comparison of the transmission coefficient of numerical test simulation and experimental test. The error rate refers to Mean Absolute Percentage Error (MAPE) theory. MAPE theory can be seen on table 3. As for validation results can be seen in figure 6. Based on MAPE theory, this study can be said to be validated and can be developed more optimal shapes.

Table 3: MAPE Theory.

No	MAPE Value	Prediction
1	$MAPE \leq 10\%$	High
2	$10\% < MAPE \leq 20\%$	Good
3	$20\% < MAPE \leq 50\%$	Reasonable
4	$MAPE > 50\%$	Low

3.2 Transmission Coefficient Regular Pontoon (RG)

Regular Pontoon is one type of floating breakwater that was examined by (Christensen, 2018). In this section, the results of transmission coefficient of numerical simulation results can be seen in table 4. The transmission coefficient is obtained from the analysis of the output surface elevation results in

Flow 3D software. Surface elevation data used to analyze the transmission coefficient are obtained from wave gauge 1 (in front of the structure) and wave gauge 5 (behind the structure). Surface elevation data is converted into wave height using Wave Analysis (WAVAN) software. Then, to get the transmission coefficient the calculation is according to formula (1). The results of regular pontoon coefficient transmission are shown in table 4.

3.3 Transmission Coefficient Development Notched Shape

It is still planned to analyze the transmission coefficient of porous shapes floating breakwater. There are 3 exploratory shapes of porous floating breakwater, see figure 2. In this analysis, it used Flow

3D software such as what was done in getting the transmission coefficient of without porous shape floating breakwater (RG).

4 CONCLUSIONS

Four porous shapes of floating breakwater will be examined in experimentally and numerically. But, still planned. The porous shape is placed in lee side and back side of the structure. The basic shape of the structure is regular box. This experiment was conducted by (Christensen, 2018) in a wave flume in the hydraulic laboratory at the Technical University of Denmark. As well as numerical simulations carried out by the authors by taking Christensen's experimental data. Numerical analysis using

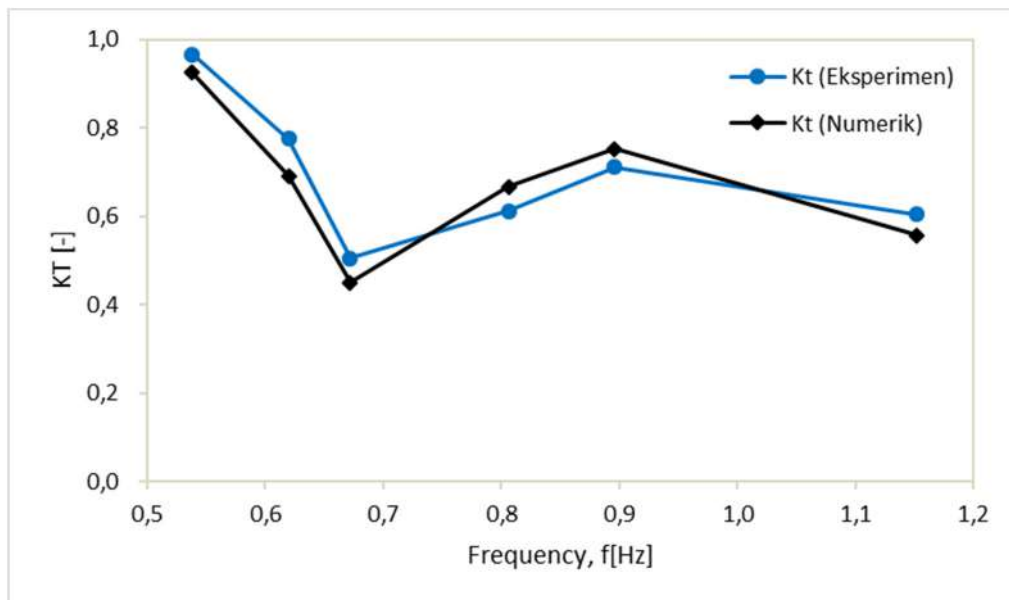


Figure 6: Validation Results.

Table 4: Value of Validation Results.

No	Data Input		H_I	H_T	C_T (experiment)	C_T (numeric)	Error (%)
	H [m]	T [s]					
1	0,0230	0,8680	0,0353	0,0197	0,6046	0,5581	8,3370
2	0,0360	1,1160	0,0329	0,0248	0,7114	0,7538	5,6248
3	0,0410	1,2400	0,0491	0,0328	0,6125	0,6680	8,3117
4	0,0490	1,4880	0,0521	0,0235	0,5054	0,4511	12,0483
5	0,0520	1,6130	0,0589	0,0408	0,7756	0,6927	11,9677
6	0,0550	1,8610	0,0498	0,0461	0,9673	0,9257	4,4936
Mean Absolute Percentage Error (MAPE)							8,4638

Computational Fluid Dynamics (CFD) method with the help of Flow 3D software. The aim is to optimize the transmission coefficient by exploring the porous shape, due to limited experiments. Previously, validation was done between numerical and experimental.

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Spatial Distribution of Pollutant Distribution in Jakarta Bay from Hydrodynamic-2D Transport Coupled Model

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Keywords: Pollutant, Jakarta Bay, Hydrodynamic, 2D Transport Coupled Model.

Abstract: Being a center of marine activities such as shipping and fishing which are accompanied by massive development and industrial, Jakarta Bay is susceptible to the environmental degradation from the pollutants. Observation was conducted in Jakarta Bay during the southeast monsoon (Juli-Agustus) 2015 to collect physical properties data such as sea surface temperature and heavy metal component. Pollutants distribution such as Pb, Ni, Zn, Polychlorinated biphenyl (PCB), Polycyclic aromatic hydrocarbons (PAH) and Dichlorodiphenyl- trocholoroethane (DDT) are simulated using Horizontal 2D hydrodynamic model from Princeton Ocean Model (POM) with the Arakawa-C grid system. The model domain consists of 400 x 260 grid and 110x110m grid resolution. With the high accuracy rate (92%) for the tidal validation and mean sea level residual 0.074 m, the simulation shows that pollutant distribution is mainly influenced by ocean current condition. Observed eddies in the simulation, which are triggered by significant difference of bathymetry and shear current velocity, indicate that the pollutants from Jakarta Bay are not easily transported by tidal current into the Java Sea.

1 INTRODUCTION

Jakarta Bay is one of the most heavily used semi-enclosed coastal waters in the world. Along with the massive development and industrial activities on the coast, water pollution caused by the dumping of industrial and domestic wastewater had been a serious ecological problem for coral reef (Kunzmann et al., 2018) as well as fisheries (Irianto et al., 2017). Research of metal concentration in Sediments in Jakarta Bay by Arifin (2008) revealed that its concentration experiences a rapid increased in the last 20 year.

Water quality monitoring by **BPLHD-DKI** in 2013 found that heavy metals concentration in Jakarta Bay such as Pb, Ni and Zn has exceeded the specified quality standard threshold by **MenLH**. Furthermore, Susanna and Yanagi (2002) argued that the sources of pollutants are coming from major river that flows into the Jakarta Bay.. As consequence, the study to get information of pollutant distribution are very essential to obtain the better understanding about water quality in the Jakarta Bay.

Several studies have been done to assess the pollution in Jakarta Bay including methods that involving biota (Soemodihardjo and Kastoro, 1977; Rees et al., 1999), sediment geochemistry (Sindern et al., 2016) as well as using a numerical ocean model simulation (Koropitan et al., 2009; van der Wulp et al., 2016).

The lack of pollutant dynamics study in the Jakarta Bay motivated the investigation that is presented in this paper. This paper aims to simulate the spatial pattern of pollutant distribution using hydrodynamic and Horizontal 2D transport coupled model. This study could give the useful information about heavy metal concentration in all regions of Jakarta Bay.

2 METHODS

2.1 Model Design

The coupled Hydrodynamic-Horizontal 2D transport model are simulated in forty-five (45) days from 1 July until 15 August 2015 with the time step of 2

second. The domain of this study lies on 106.633°E – 107.033°E and 5.868°S – 6.118°S (see Figure 1).

To simulate the pollutant distribution, horizontal 2D hydrodynamic model from Princeton Ocean Model (POM) is employed with the additional numerical solution from 2D transport equation. The model is built by Arakawa-C grid system, consisting of 400×260 grid with the resolution $110 \times 110 \text{ m}$.

The type of pollutant in this research are Pb, Ni, Zn, Polychlorinated biphenyl (PCB), polycyclic aromatic hydrocarbons (PAH) and Dichlorodiphenyl-trichloroethane (DDT) with the consideration of advection and diffusion of these pollutants.

2.2 Model Input

The several primary data such as bathymetry, coastline, tide, sea temperature, salinity, river debit rate are used as model input. While the temperature and salinity are measured from observation with thirty two (32) measurement station (see figure 1), the bathymetry is obtained from Indonesian Navy (DISHIDROS TNI AL). It is interpolated spatially

into the specified grid size of this study (see figure 1). Then, the temperature and salinity data are employed to gain the density reference value in the Bay. The tidal data for the main force in west, north and east part of domain come from Tide Model Driver (TMD) (see figure 1). Besides, the debit rate from five major rivers and wind velocity are also used as the main force. Referring the Courant-Friedrichs-Lewy (CFL), the main forcing data are interpolated into 2 second.

For the initial value of the model (see figure 2), this study assumes that the sea is calm ($u = 0$, $v = 0$, and $\eta = 0$) and the initial value for pollutant concentration in the domain model is spatial interpolation with the Kriging method from field measurement, conducted by Research Center for Oceanography, Indonesian Institute of Sciences (LIPI) on July-August 2015.

The results of the distribution of pollutant distribution in Jakarta Bay are verified using data from field measurements including tidal elevation and sea current velocity. The information regarding the data for model validation is written in Table 1.

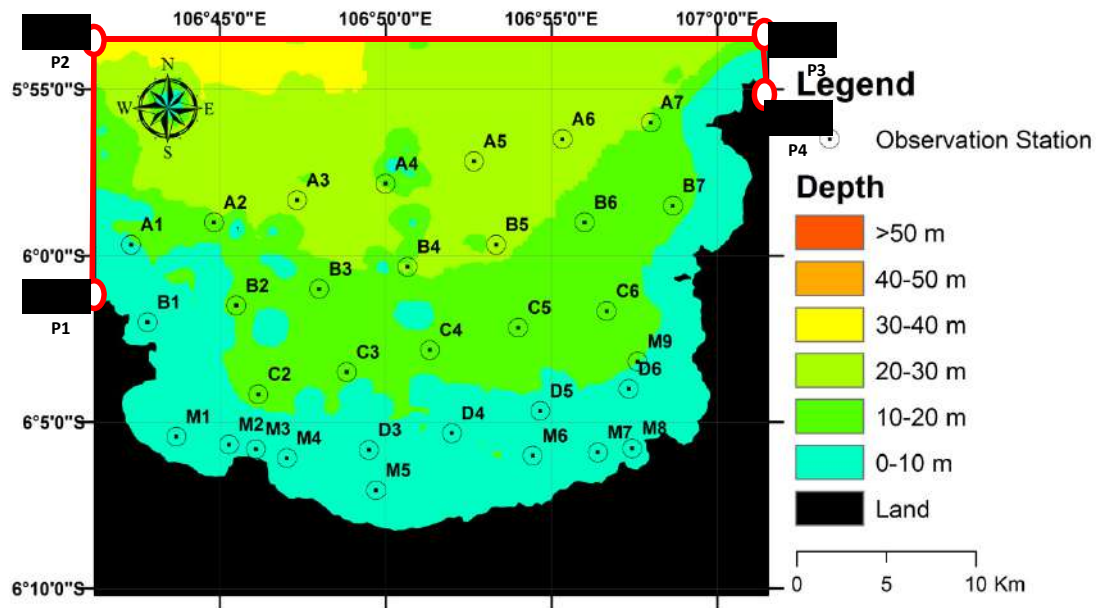


Figure 1: Bathymetry map with the thirty two (32) measurement stations with the open boundary of domain (red line) and spot for tidal data (red circle – P1, P2, P3 and P4).

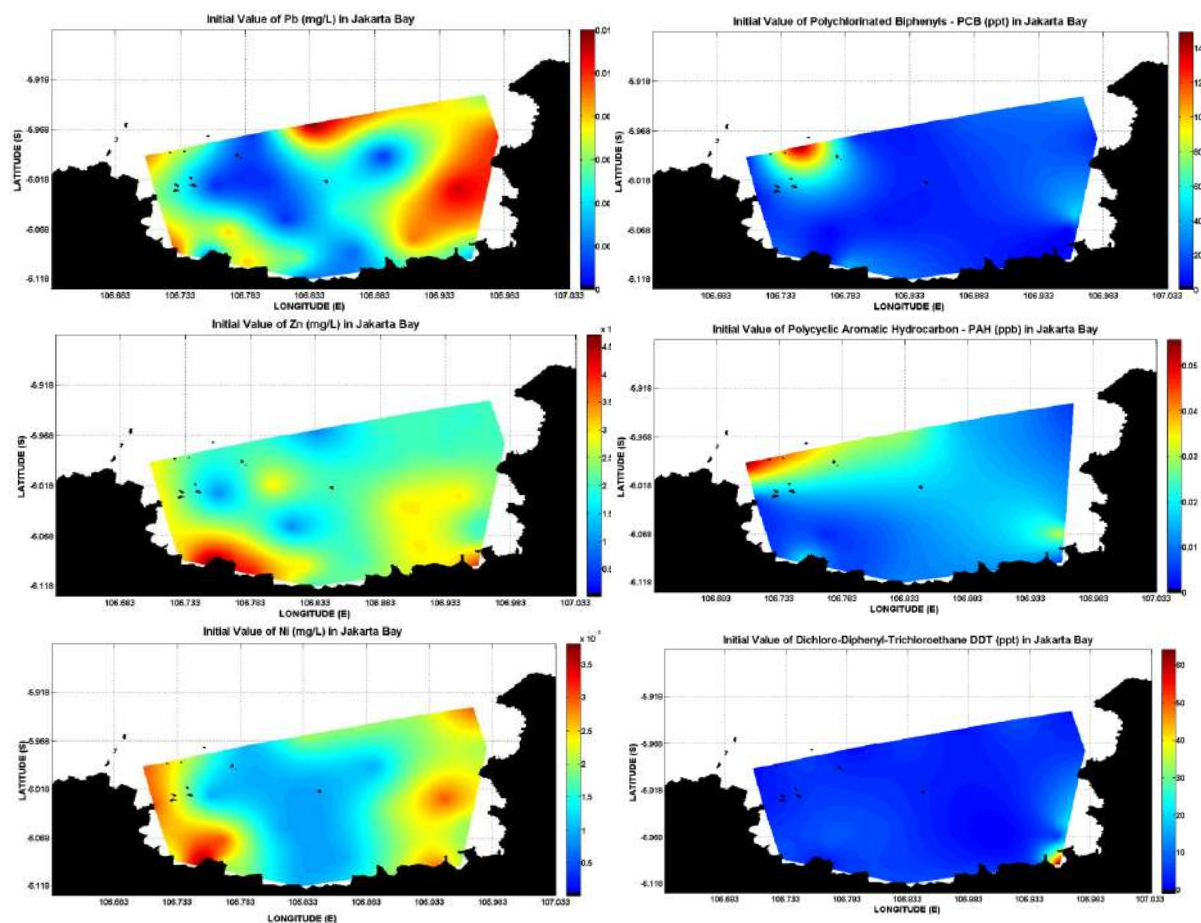


Figure 2: The initial value of heavy metal concentration; Pb (mg/L), Zn (mg/L), Ni (mg/L) (left panel) and organic pollutant; PCB (ppt), PAH (ppb), DDT (ppt) (left panel) based on the field measurement July-August 2015.

Table 1: Data for Model Validation.

No	Data	Source	Location	Period (dd/mm/yy)	Interval
1.	Tidal Elevation	Agency of Geospatial Information (BIG)	Kolinlamil station, Jakarta	01/07/15 – 31/08/15	5 minute
2.	Current Velocity	Observation	a. St.B2 (-6.025°S and 106.758°E)	29/07/15 - 30/07/15	10 minute
			b. St.B6 (-5.975°S and 106.978°E)	30/07/15 - 31/07/15	10 minute

2.3 Model Equation

The governing equation for hydrodynamic and transport coupled model consists of continuity equation (1.1), momentum equation in x-y axis (1.2a and 1.2b) (Mellor, 2004) and transport equation in x-y axis (1.6a and 1.6b) (Putri and Mihardja, 1999). On this equation, there is assumption and simplification which are only consider to advection and diffusion,

the source and sink of heavy metals from chemical reaction and other factors are not taken into account. The transport simulation on this study is conducted only based on the initial value from observation. In the absence of other supporting data, the input pollutants during the simulation period are not reviewed.

Then, this research produces a distribution pattern and pollutant flushing processes in the Jakarta Bay.

$$\frac{\partial \eta}{\partial t} + \frac{\partial \bar{U}D}{\partial x} + \frac{\partial \bar{V}D}{\partial y} = 0 \quad (1)$$

$$\begin{aligned} \frac{\partial \bar{U}D}{\partial t} + \frac{\partial \bar{U}^2 D}{\partial x} + \frac{\partial \bar{U}\bar{V}D}{\partial y} - \bar{F}_x - f\bar{V}D + gD \frac{\partial \eta}{\partial x} = -\langle wu(0) \rangle + \langle wu(-1) \rangle + \\ G_x - \frac{gD}{\rho_0} \int_{-1}^0 \int_{-1}^0 \left[D \frac{\partial \rho'}{\partial x} - \frac{\partial D}{\partial x} \sigma' \frac{\partial \rho'}{\partial \sigma} \right] d\sigma' d\sigma \end{aligned} \quad (2a)$$

$$\begin{aligned} \frac{\partial \bar{V}D}{\partial t} + \frac{\partial \bar{U}\bar{V}D}{\partial x} + \frac{\partial \bar{V}^2 D}{\partial y} - \bar{F}_y + f\bar{U}D + gD \frac{\partial \eta}{\partial y} = -\langle wv(0) \rangle + \langle wv(-1) \rangle + \\ G_y - \frac{gD}{\rho_0} \int_{-1}^0 \int_{-1}^0 \left[D \frac{\partial \rho'}{\partial y} - \frac{\partial D}{\partial y} \sigma' \frac{\partial \rho'}{\partial \sigma} \right] d\sigma' d\sigma \end{aligned} \quad (2b)$$

where,

$$\bar{U} = \int_{-1}^0 U d\sigma \quad \text{dan} \quad \bar{V} = \int_{-1}^0 V d\sigma \quad (3)$$

$$\bar{F}_x = \frac{\partial}{\partial x} \left[H2\bar{A}_M \frac{\partial \bar{U}}{\partial x} \right] + \frac{\partial}{\partial y} \left[H\bar{A}_M \left(\frac{\partial \bar{U}}{\partial y} + \frac{\partial \bar{V}}{\partial x} \right) \right] \quad (4a)$$

$$\bar{F}_y = \frac{\partial}{\partial y} \left[H2\bar{A}_M \frac{\partial \bar{V}}{\partial y} \right] + \frac{\partial}{\partial x} \left[H\bar{A}_M \left(\frac{\partial \bar{U}}{\partial y} + \frac{\partial \bar{V}}{\partial x} \right) \right] \quad (4b)$$

$$G_x = \frac{\partial \bar{U}^2 D}{\partial x} + \frac{\partial \bar{U}\bar{V}D}{\partial y} - \bar{F}_x - \frac{\partial \bar{U}^2 D}{\partial x} - \frac{\partial \bar{U}\bar{V}D}{\partial y} + \bar{F}_x \quad (5a)$$

$$G_y = \frac{\partial \bar{U}\bar{V}D}{\partial x} + \frac{\partial \bar{V}^2 D}{\partial y} - \bar{F}_y - \frac{\partial \bar{U}\bar{V}D}{\partial x} - \frac{\partial \bar{V}^2 D}{\partial y} + \bar{F}_y \quad (5b)$$

Generally, 2D transport equation can be written as (Putri dan Mihardja, 1999):

$$\frac{\partial C}{\partial t} + u \frac{\partial C}{\partial x} + v \frac{\partial C}{\partial y} = K_x \frac{\partial^2 C}{\partial x^2} + K_y \frac{\partial^2 C}{\partial y^2} + S + R \quad (6)$$

where C is , S and R are for source and reaction respectively. On this study, we ignore the S and R due to the absence of supporting data. Then, Kx and Ky are the horizontal diffusion coefficient in x-y which is different for each parameter. We use $9.45 \times 10^{-6} \text{ cm}^2/\text{s}$ [Pb^{2+}], $6.79 \times 10^{-6} \text{ cm}^2/\text{s}$ [Ni^{2+}], $7.15 \times 10^{-6} \text{ cm}^2/\text{s}$ [Zn^{2+}] (Li and Gregory, 1974), $3.16 \times 10^{-11} \text{ m}^2/\text{s}$ [PCB], $2.51 \times 10^{-11} \text{ m}^2/\text{s}$ [PAH] (Rusina et al., 2010) and $6.86 \times 10^{-12} \text{ m}^2/\text{s}$ [DDT] (Dueri et al., 2005).

3 RESULTS AND DISCUSSION

3.1 Model Validation

The model results are validated using two main parameters; sea level elevation and ocean current.

The sea level elevation is verified by observation data, which are obtained from the Agency of Geospatial Information (BIG) located at Kolinlamil station, Jakarta. The validation is carried out for 15 days on 10 July 2015 – 25 July 2015 (see Figure 3) showing the high accuracy of 92% with an average residual sea level elevation of 0.074 m.

The observational ocean current data from 24 hours measurement at B2 and B6 stations are also utilized to verify the model result. The measurement was conducted at 5 m depth in 28-29 July [station B2] and 29-30 July [station B6] with the interval of 10 minute. The results in station B2 showed that the accuracy of the zonal and meridional current are 54% and 55%, respectively. Station B6 has the accuracy rate of 50% and 73% easting and northing current correspondingly. Some components show an accuracy value which is not accordance with the field data. That conditions are potentially due to the lack of matching drag coefficient values for surface wind speed. Based on the wind speed during the simulation period, the wind is large enough to generate ocean currents. Therefore, the model results are underestimated during that condition.

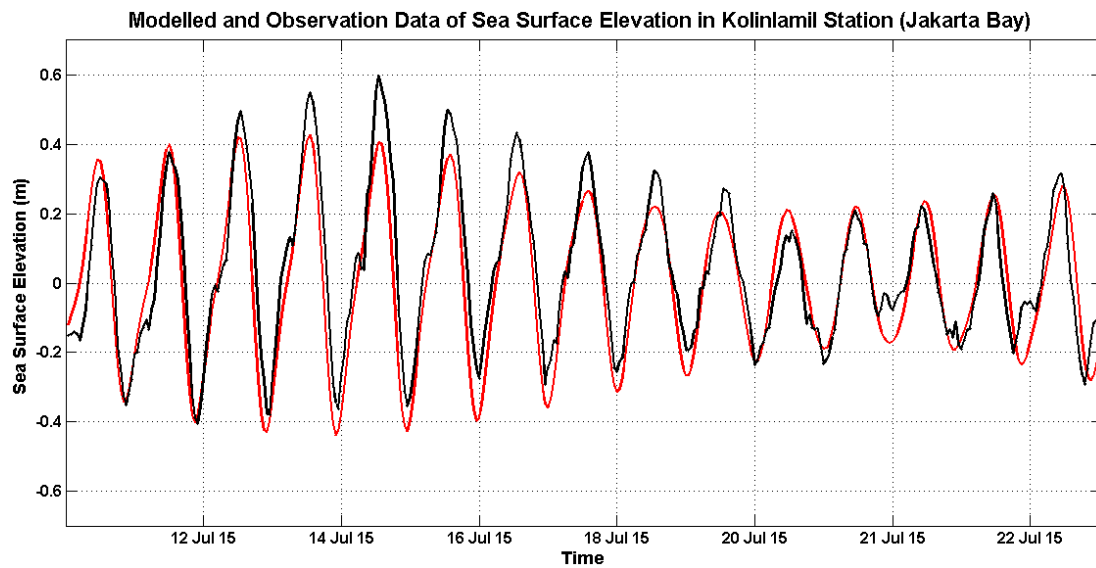


Figure 3: Model Validation between model result (red line) and observation (black line) on 10 – 25 July 2015, located on Kolinlamil Station, Jakarta.

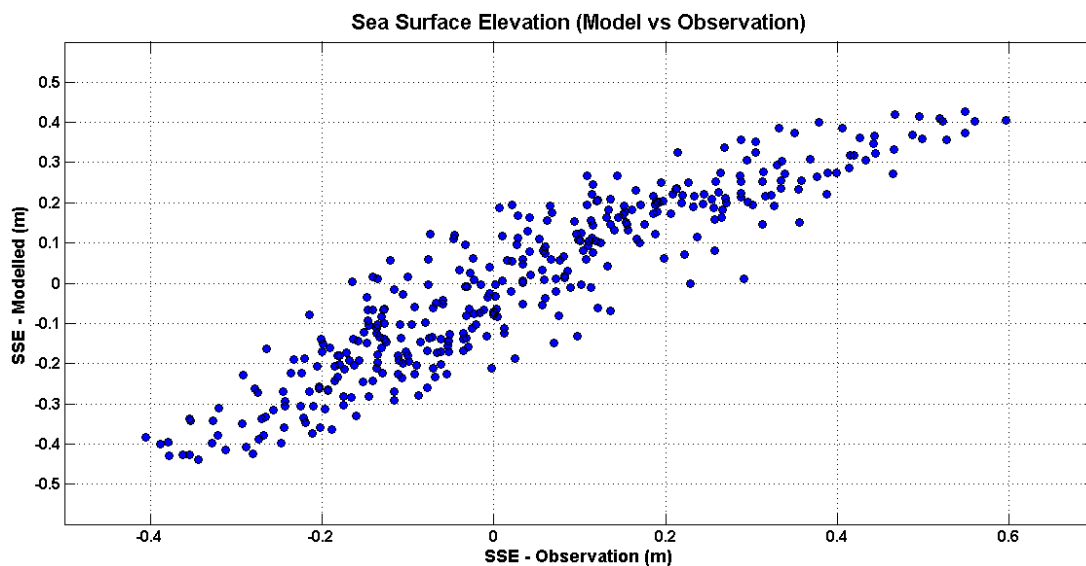


Figure 4: Model Validation between model result (red line) and observation (black line) on 10 – 25 July 2015, located on Kolinlamil Station, Jakarta.

3.2 Jakarta Bay Dynamics

Generally, the type of tide in the Jakarta Bay is mix tide with the flood and ebb tide occur just once in a day. Furthermore, ocean current patterns in the Jakarta Bay vary, depending on tides and monsoons. In some cases, where the wind blows so strong in the southeast monsoon, the ocean currents move north-eastward. During the ebb tide phase, the current flows into the open sea while there are southward currents in the eastern part of the Bay (Fig 5; upper panel),

while the ocean current dominantly moves into the mainland when the flood tide (Fig 5; lower panel). These results are consistent with Putri and Mihardja (1999) and Sanusi et al (2005). The magnitude of ocean current varies from 0 until 0.20 m/s. These conditions also indicate that the pollutant distribution is more influenced by ocean current than the diffusion rate of each pollutant.

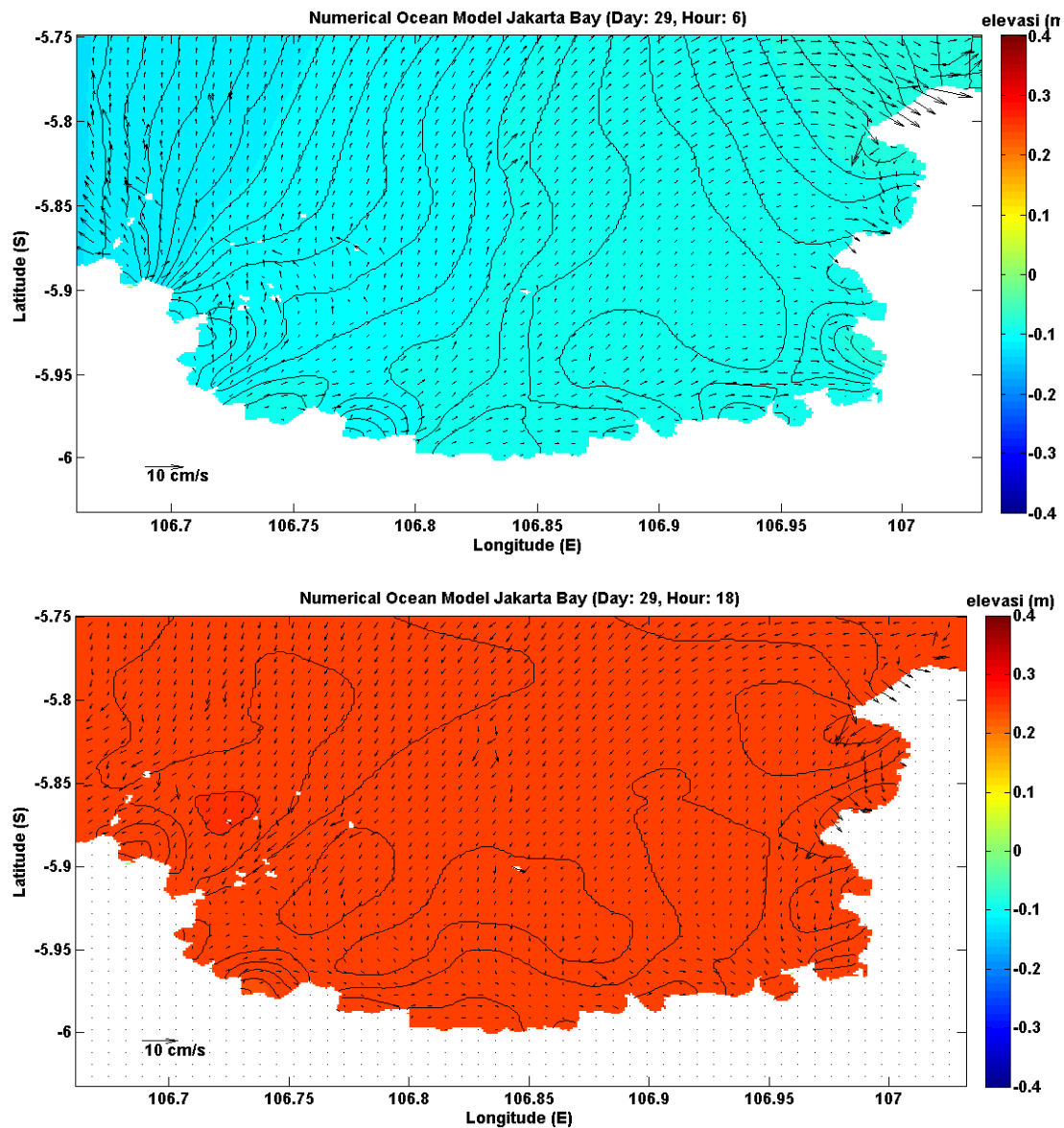


Figure 5: Model Validation between model result (red line) and observation (black line) on 10 – 25 July 2015, located on Kolinlamil Station, Jakarta.

3.3 Pollutant Distribution

Overall, the model results show the distribution of pollutant which is mainly influenced by the ocean current pattern (Figure 6 – 11). The variation of bathymetry and the shape of the coastline in Jakarta Bay contributed to eddies current in the several places along the Jakarta Bay. For the example, the observed eddy current in the eastern part of Bay is triggered by shear current velocity. In some areas, the eddy current are also found due to the significant difference of bathymetry. The formation of eddies causes the

particle movement that rotate around the centre of eddies. As a result, the pollutants from Jakarta Bay are not easily transported by tidal current into the Java Sea. In addition, the monsoonal cycle also has an important role to transport the pollutant by moving the ocean current. It can be indicated from the simulation result. The pollutant distribution has the dominant direction into the west while the southeast monsoon triggers the ocean current to move southwestward.

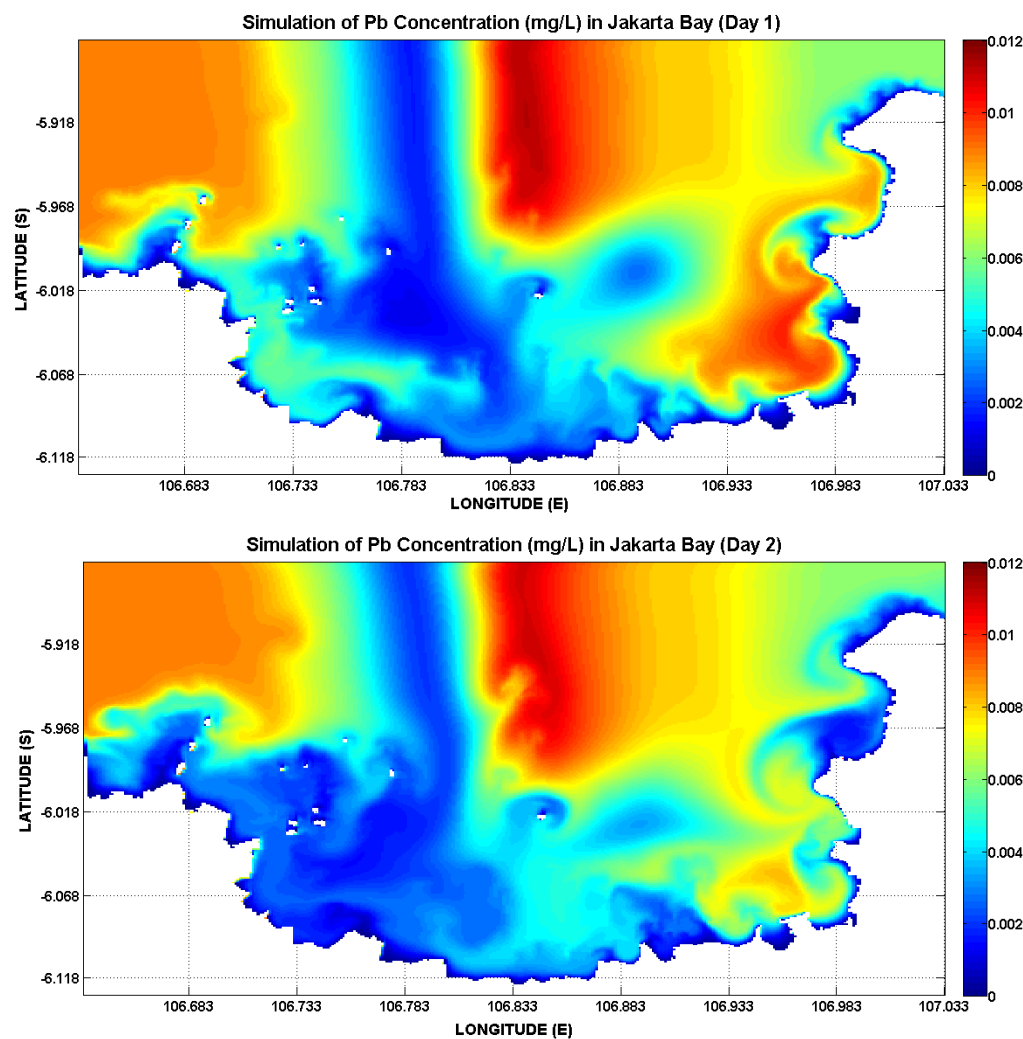


Figure 6: Simulation result of Pb concentration in Jakarta Bay.

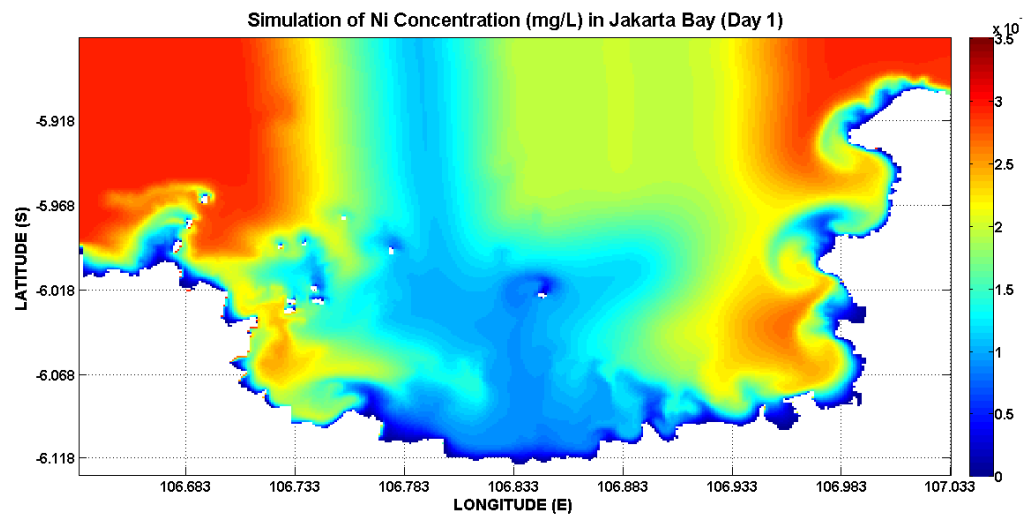


Figure 7: Simulation result of Ni concentration in Jakarta Bay.

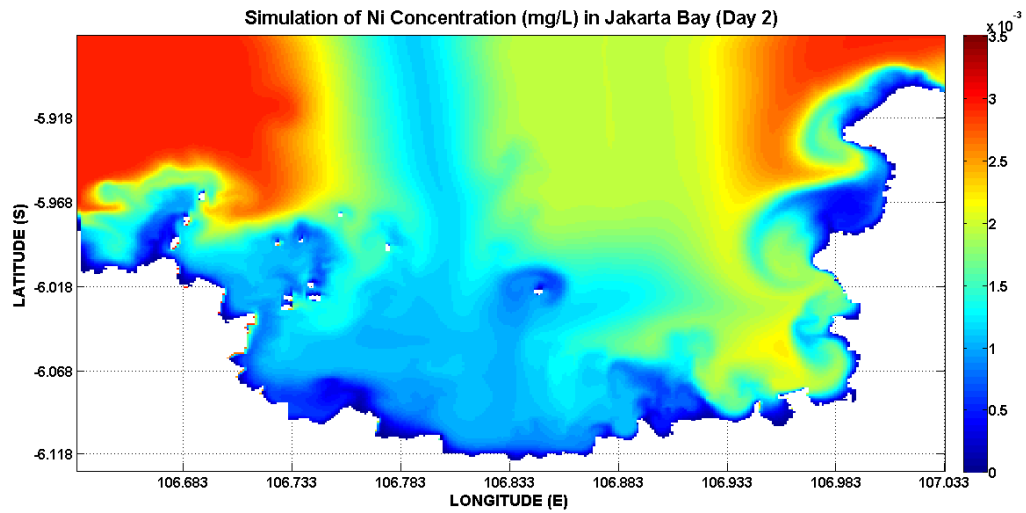


Figure 7: Simulation result of Ni concentration in Jakarta Bay (cont.).

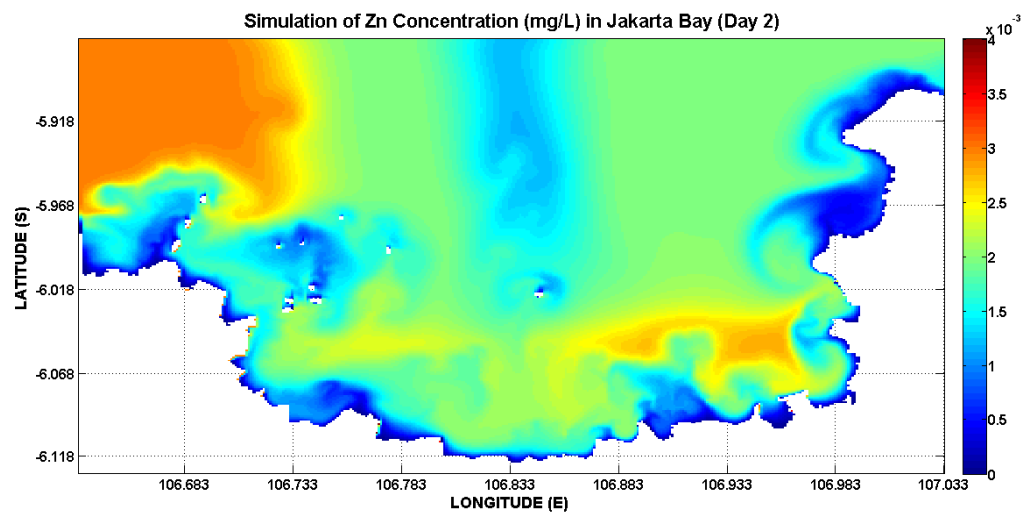
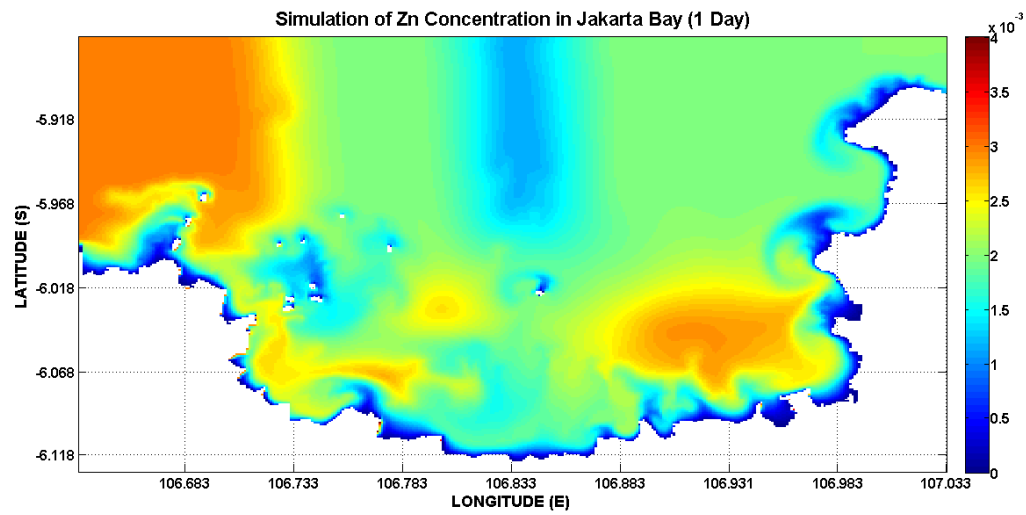


Figure 8: Simulation result of Zn concentration in Jakarta Bay.

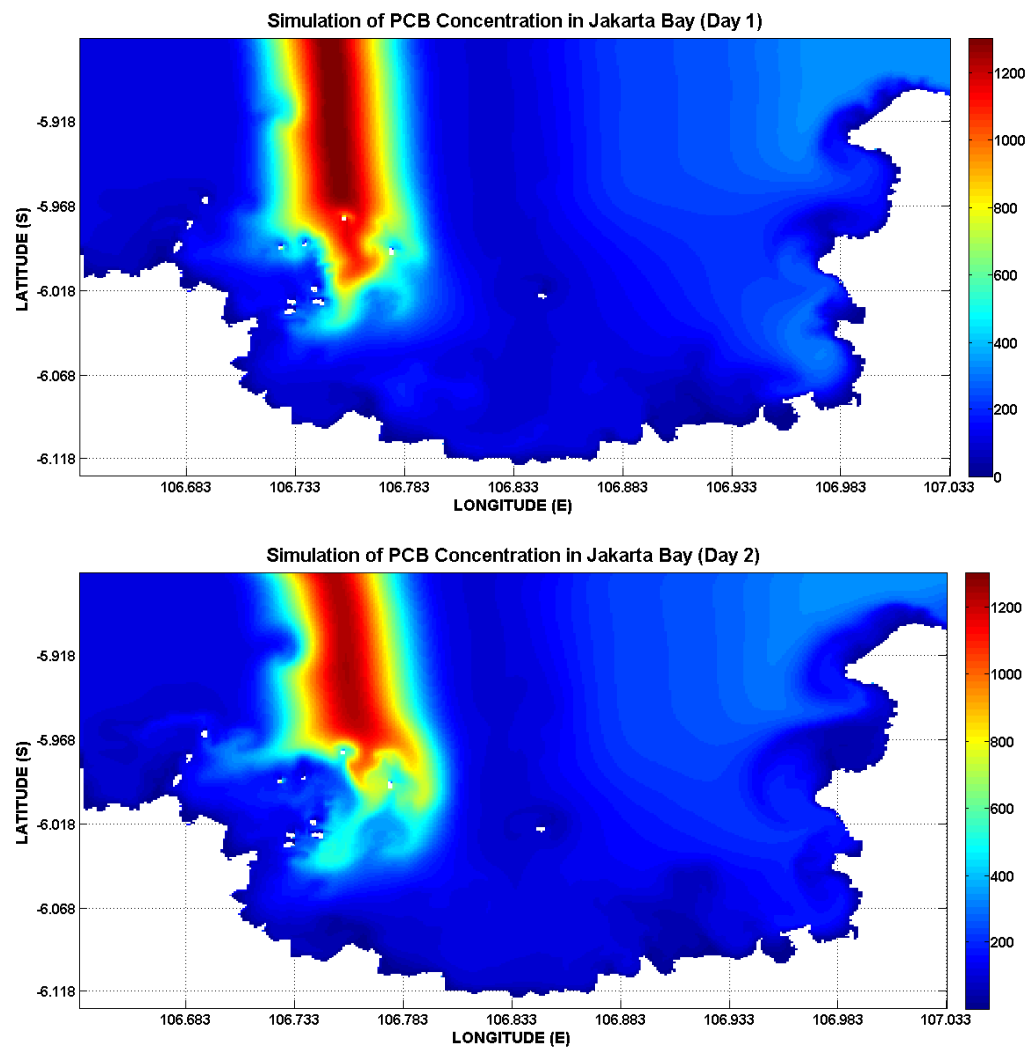


Figure 9: Simulation result of PCB concentration in Jakarta Bay.

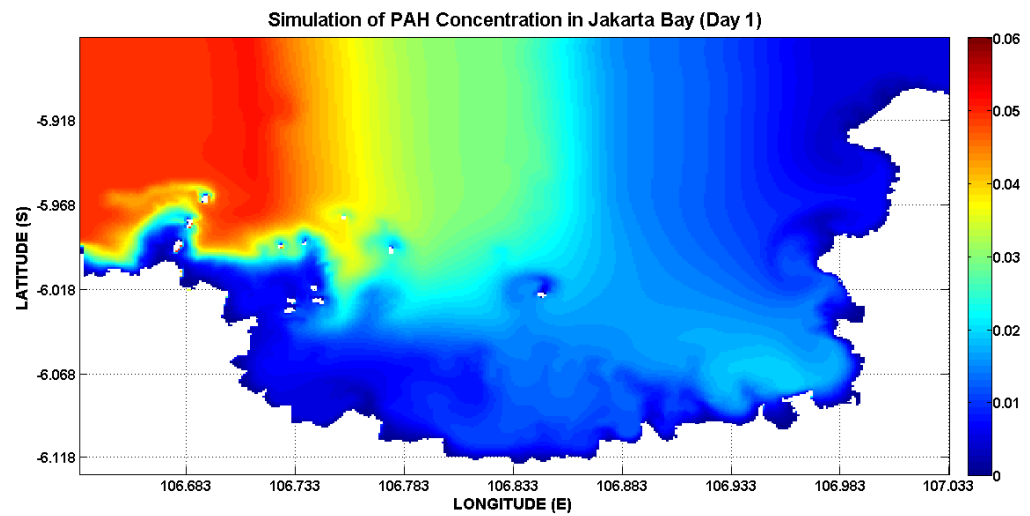


Figure 10: Simulation result of PAH concentration in Jakarta Bay.

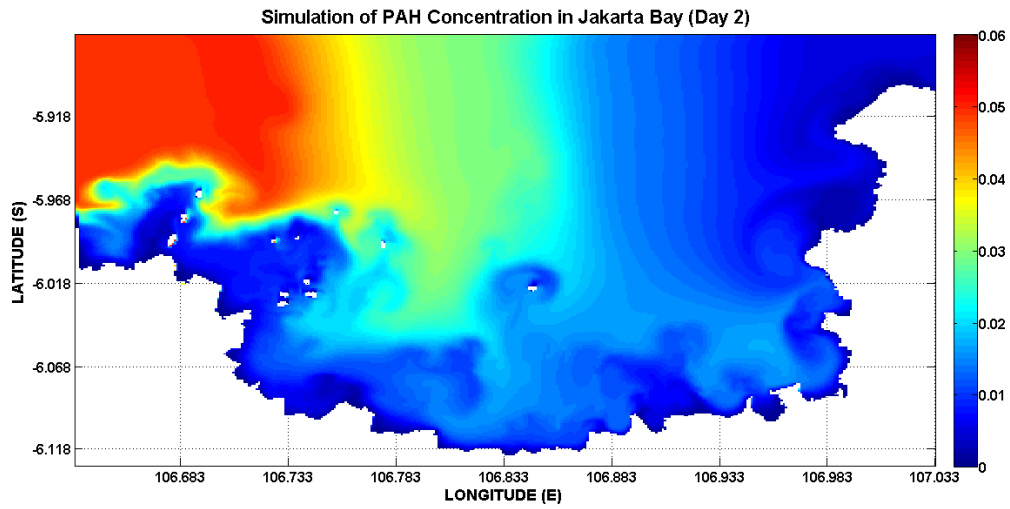


Figure 10: Simulation result of PAH concentration in Jakarta Bay (cont.).

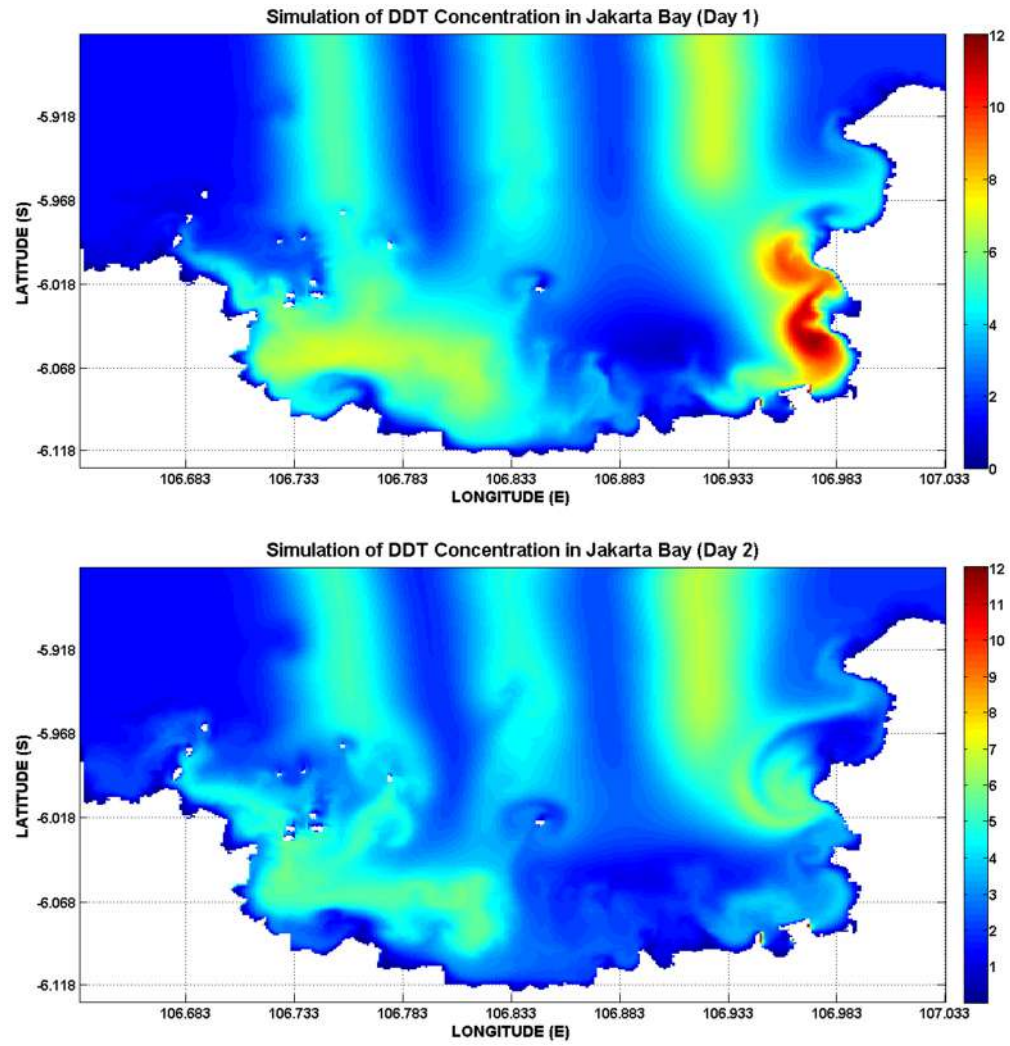


Figure 11: Simulation result of DDT concentration in Jakarta Bay.

4 CONCLUSION

From the simulation of pollutants distribution using coupled hydrodynamics-2D transport model, we conclude that the water movement in Jakarta mainly driven by tidal condition, debit of river and wind velocity.

Those ocean-atmospheric factors contributed to the spatial variation of the pollutant in the Jakarta Bay. The pollutants from Jakarta Bay are not easily transported by tidal current into the Java Sea due the formation of eddy currents.

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Dealing with the Port Impacts to Ensure the Sustainability of Port-cities Development by Employing Integrated Policy: A Case Study about Land Use in Semarang City

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Keywords: Port Impacts, Port-city, Sustainable Development, Integrated Policy-Making, Land Use, Semarang.

Abstract: The role of port-cities has increased along with increase in the function of ports as the backbone of global economy in distributing logistics throughout the world. Port development has positive as well as negative impacts on the growth of port-city. There are positive impacts in economic and social aspects, while negative impacts affect on aspects of land use, transport and environment. Semarang is a major port-city in Indonesia that takes advantage of positive impacts and deals with negative impacts. The aim of this research is defining the role of integrated policy-making in responding to port impacts, to achieve sustainable port-city development in Semarang. The design of this research is a case study using qualitative data analysis with the use of Atlas.ti-8 software for the coding process. Fieldwork had been conducted in 2018 for 2.5 months by observation and semi-structured in-depth interviews. This paper focuses on land use policy-making process in responding to port impacts on the city. The results indicate inter-territorial integration in complexity of authorities, horizontal integration through formation of cross-sector ad hoc team, vertical integration through provincial- national review, civil society involvement through public consultation and land use policy responses to port impacts.

1 INTRODUCTION

Port cities have a crucial role in economic development by facilitating national and international trade. In the era of globalization, the role of port-cities has increased along with increase in the crucial function of ports as the backbone of the global economy in distributing logistics throughout the world. Lately there have been many conveniences for various business sectors to gain opportunities in order to reach international markets so that it will increase trade among countries that are based on the sea logistics system. Moreover, improved technologies that support trade such as e-commerce also increase the volume of international trade for goods that must be sent physically using sea transportation (Terzi, 2011). The port is the centre of a sea transportation network that facilitates

trade among countries and supports international supply chains with volume of goods distribution reaching nearly 80 percent of international trade with an increasing trend during 2015-2018 (UNCTAD, 2016, UNCTAD, 2018).

Port-cities accommodate two main functions that occur in the coastal areas such as trade and port activities. Other functions of the use of coastal areas such as for housing, industry, recreation are also the responsibility of the port-cities. To ensure sustainability, the cities have to be managed thoroughly and integrated (Clark, 1994).

Port and city have a close relationship of origin, from a historical point of view. Some trading activities in a coastal area form a simple trade node as a natural entrance to facilitate loading / unloading and shipping of goods using ships, which then developed into a port. During the growth, these port activities eventually had positive impacts on

economic growth and social development of the surrounding port area, which was then able to generate a small town around the port. In its development, this small city grew into a major port-city. Then, as negative impacts from the growing port activities, city has to deal with various problems such as land use issues, traffic congestion and environmental degradation (OECD, 2014).

Semarang, as a major port-city in Indonesia, benefits from positive impacts but has to bear the brunt negative impacts from port activities. Semarang City was selected as a case study because Semarang, as the capital of the province of Central Java, has a strategic location. This city is located on the northern coast of the island of Java in the middle of the province so it is easily accessible from all directions. Semarang City plays a crucial role in the growth of the province through logistics distribution sector as the port is the main entrance for distribution of goods to and from the province of Central Java. Semarang port has the potential to be developed because it has a large hinterland area consisting of 35 regencies/cities, so it should be planned and managed in an integrated approach to be sustainable.

Semarang has a long history as a port-city. The history of Semarang begins with the emergence of trading activities at the edge of the Semarang river (called Kali Semarang). These trading activities form trade nodes that continued to develop into a major port-city. Subsequently Semarang became the provincial capital of Java during the colonial period, in 1775. The role of Semarang as a port-city began to escalate when the Dutch colonial authority determined Semarang as the centre of the Northeast Coast of Java in the 18th century (Knaap, 2015).

The research is concerned about the impact of implementation of integrated policies in the field of land use, transportation and environment that must be taken into consideration by the main stakeholders of Semarang Port-city. It is imperative that both pros and cons of port development be considered to ensure sustainable development. The key stakeholders are the Government of Semarang City as the local government, Harbourmaster and Port Authority (KSOP) as the authority of the port and Pelindo III as the port operator in the Port of Semarang.

This paper was written, as a part of the research, which will focus on the implementation of integrated policies in land use aspects, particularly response to port impacts, within the framework of the integrated policy towards sustainable development in Semarang Port-city. The paper begins with an

elaboration of the conceptual frameworks of integrated policies in ports, port-cities and sustainable development in section 2. Then sections 3 and 4 will discuss methodology and findings respectively, which is followed by conclusions in section 5.

2 CONCEPTUAL FRAMEWORKS

Conceptual frameworks are the model that describes relationships about several relevant theoretical building blocks, particularly those related to port development, port impacts on port cities, and sustainable development as well as integrated policy. The implementation of integrated policies in port-cities is an emphasis in the research, especially in relation to policies in the field of land use, transportation and the environment in responding to the positive and negative impacts of port development. Integrated policy acts as a catalyst that is expected to assist and accelerate port-city to achieve sustainable development. The research on the application of integrated policies in port-cities, mainly in the fields of land use, transportation and environment in responding to the positive and negative impacts of ports is expected to be a new contribution in policy-making process in port-cities in general. Port-cities in Indonesia, in particular, are expected to be able to implement this conceptual framework model in policy-making, specifically in the fields of land use, transportation and the environment.

In addition, this conceptual framework model was created to guide the research in answering the research question such as: how integrated policy-making can contribute to the sustainable development of port and port-city relationship, in the case of Semarang Port-city. Moreover, this model also seeks to answer the following sub-research questions:

- 1) How can be understood about the relationship between port and port-city?
- 2) How can the role of sustainable development in the port and port-city relationship be understood and operationalized, in the case of Semarang Port-city?
- 3) What is the role of integrated policy-making in achieving sustainable development in Semarang Port-city?

2.1 Relationship between Port and City

The conceptual frameworks start from relationship between port and its city. This sub section is a response to the first sub research question namely relationship between port and city. Alderton (2008), Talley (2009) and, Stopford (Stopford, 2009) describe port in the similar tone that port is an interface between ship and sea which is equipped with facilities for ship berths as well as maintenance, and equipped with several equipment for moving goods from ship to land and vice versa. In addition, there are terminal facilities for the movement of goods and people.

Most ports are located around cities to facilitate access to sellers, buyers, factories and manufacturers (Merk, 2018), so that there is a close historical relationship between ports and cities. In general, many cities flourish and thrive from ports. Many cities begin as inter-regional trade nodes that use ports as natural gateways and use the sea as interconnected regions. Further development of these trades encourage small towns to grow and develop into cities that further grow into major cities. That is the origin of port-cities.

During its development, relationship between ports and cities are mutually influenced. The development of ports will affect the growth of port-cities, and the growth of port-cities will affect ports development. The conceptual frameworks of this research only emphasize on port development impacts on the port-cities growth.

2.2 Impacts of Port Development to City

In general, the activities of ports development have two major influences on port-cities growth. This sub section is still a response to the first sub research question namely relationship between port and city.

The first impacts are considered as positive impacts, which subsequently transformed into port-related benefits of port-cities. These advantages consist of several dimensions in terms of economic and social development. These positive impacts tend to be spread out to surrounding cities. It means that the cities around the port-cities can also benefit from these positive impacts. Ports may contribute crucial advantages in economic development in coastal areas through job creations, generate tax (Miller, 2017), generate more revenue (Bichou and Gray, 2005), both directly and indirectly (Ferrari; Percoco, et al., 2010), as well as stimulating new business opportunities (Talley, 2009). Ports also deliver

significant contribution for economic development in hinterland areas because ports connect sea and land by using ships for transportation (Dwarakish and Salim, 2015). These activities generate first contribution of ports to cities namely cargo handling business. It is one of the earliest economic activities that acts as a driver to local jobs creation, tax revenues and other indirect economic activities (Xiao and Lam, 2017). The impact on social aspect could change local society life due to relocation of villages for port expansion, change in lifestyle, formation of slum areas, effect on tradition and culture and disturbance in the beach recreation areas. (OECD, 2014). Moreover, relocation and resettlement of local society may elicit some changes in population composition regarding racial issues between minority-majority groups and distribution of population related with age composition (Xiao and Lam, 2017).

The second major impact is considered as negative impacts of ports which consequently converted into port-related problems of port-cities. Those disadvantages affect several aspects such as land use, transport, and environment. Usually, these negative impacts tend to be localized inside the port-cities. The typical requirement of modern ports is the need for a very wide space so ports cover relatively large area of land in the cities. Usually, high density of work and effects of agglomeration are believed as the reasons of the growth of urban economy and this effect only occur in the port areas. The impacts of ports in term of urban economic development influence the surrounding ports areas only, (OECD, 2014) so that it makes land use in the area around the port become increasingly dense and complicated. Then port activities multiply traffic density that leads to traffic congestion particularly caused by traffic of hinterland to and from ports areas (OECD, 2014). Environmental degradation around the port is due to port activities such as air pollution sourced from ships engines, cargo-handling and port equipment, engines of railroad locomotives and trucks (Talley, 2009). Oil spills are the significant cause of water pollution at ports, even from normal or accidental activities. Most of the oil spills come from vessel movement, loading/unloading of oil tankers, leakages and the operation of ballast water (OECD, 2014). Moreover, untreated vessels ballast water, waste disposal of vessels, antifouling paints in vessels and dredging activities are the cause of water pollution in the port area (Talley, 2009). Other source of environmental pollution is soil contamination from oil spills and chemicals on the ground (Miola; Paccagnan, et al., 2009). The

disturbance created by vessels and port equipment during cargo handling. Trains, trucks, and other maintenances activities are the other sources of noise pollution in port areas. This continuous noise may cause sleep disturbance (OECD, 2014).

2.3 Sustainable Development of Port-City

Sustainable development term was introduced by the United Nations World Commission on Environment and Development in its report in 1987 namely “*Our Common Future*” (United Nations, 1987). This sub section is a response to the second sub research question namely the role of sustainable development in relationship between port and city. Diesendorf (2000) reveals that sustainability is for the long term and establishes moral value of fairness and balance between current and upcoming generations. In addition, United Nations (United Nations, 1997) implements the definition of sustainable development and its three dimensions in the description of development in the Agenda for Development as follows: “Development is one of the main priorities of the United Nations. Development is a multidimensional undertaking to achieve a higher quality of life for all people. Economic development, social development and environmental protection are interdependent and mutually reinforcing components of sustainable development.” In 2015, UN established the “2030 Agenda for Sustainable Development” that includes “17 Sustainable Development Goals” (United Nations, 2015).

In dealing with several impacts of ports, either positive or negative, port-cities need to employ sustainable development concept to maintain sustainability of their roles in the globalization period as the frontline. This conceptual framework will focus on typical aspects of sustainable development such as economic and social as well as environmental aspects. All these dimensions will deal with port-related benefits and port-related problems of port-cities comprehensively.

2.4 Integrated Policy in Sustainable of Port-city Development

Several port-related benefits and port-related problems of port-cities will be addressed by implementation of integrated policy to achieve sustainable development (see Figure 1). This sub section is a response to the third sub research question namely the role of integrated policy-

making in achieving sustainable development in port-city. An integrated policy is crucial for port-cities in dealing with the interaction between ports and port-cities, since they are closely interconnected and support each other so that port-cities have to ensure the implementation of policy integration among economic, social and environment aspects which are important for sustainable development (UN-ESCAP, 2015). These sustainable developments of port-cities will sustain the leading roles of port-cities in the globalization age as integrated parts of worldwide supply chain system of logistic (OECD and ITF, 2009, Merk and Notteboom, 2015). Integration is the key of sustainable development. Geerlings and Stead (Geerlings and Stead, 2003) emphasize that the integration is significant for sustainable development, particularly in land use fields and transport, as well as environmental aspect. It is a valuable approach of development of cross-sectors by crossing border and even across generations. Every decision should concern the prospective influences on the environment, the society and economical effect in the future (Strange and Bayley, 2008).

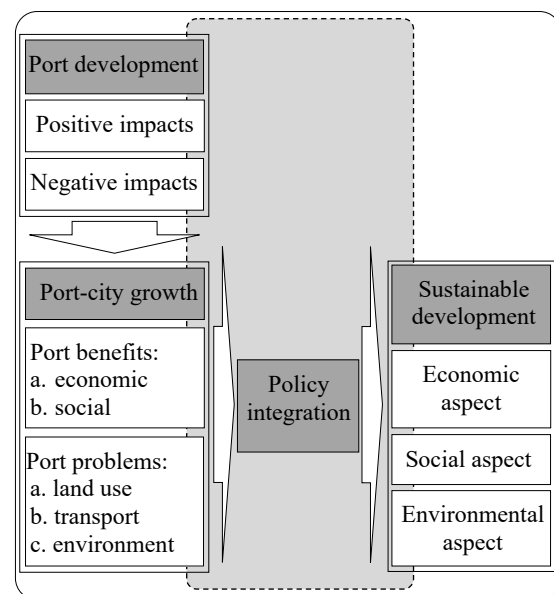


Figure 1: The model of conceptual frameworks for integrated policy in sustainable of port-cities development.

Policy integration sequence (Geerlings and Stead, 2003, Strange and Bayley, 2008),(Meijers, 2004) can be described as follows: co-operation which involves dialogue and information, each organization works concurrently to realize their-own targets so that output is efficient sectoral policy.

Then coordination (similar policy term with consistency and coherence) includes co-operation as well as openness to prevent conflicts. Mutual decisions may be not the same as their original outcomes. Policy coordination is more far-attaining than co-operation. The output is adjusting sectoral policies that are mutually enforcing and consistent. Lastly, integration or joined-up policy, which involves co-operation elements such as dialogue, discussion and information; includes co-ordination elements such as clear and transparent concept to prevent conflicts; and contains joint working, as efforts to build synergies among policies and use similar objectives to formulate policy. Policy integration is better than co-ordination with one joint policy as the new output related to sectors.

Furthermore policy integration can be distinguished (Geerlings and Stead, 2003) as follows: vertical integration (among various levels of government), horizontal integration (among various sectors or professions inside an organization), inter-territorial integration (among different authority with common interests such as resources and infrastructure), and intra-sectoral integration (combining among various sections or professions in a department and among fields of environment).

In conclusion, as illustrated in figure 1, the role of policy integration is as the catalyst to assist and accelerate port-city in policy-making proses particularly in the fields of land use, transportation and environment in responding to the positive and negative impacts of port to the city in order to reach sustainable development. The implementation of integrated policies in policy-making process of port-city, mainly in the fields of land use, transportation and environment in responding to positive and negative impacts of ports is expected to be a new contribution in policy-making proses in port-cities in general.

3 METHODOLOGY

The research design is a case study. Semarang is selected as a case study because this city is one of the three big port-cities on the Java Island in Indonesia, which has a history as a port-city. The data collection stage had been carried out in order to get appropriate and sufficient data, as a basis for observing and understanding the phenomena in the real-world context in depth and specifically, in order to analyse the policy-making process, particularly in the field of land use, transportation and environment

in Semarang Port-city. This study employs qualitative data collection, both for secondary data sources and primary data sources.

The collection of secondary data provides the preliminary overview about the profile of each stakeholder, the government policies in the three fields mentioned above and other related regulations before carrying out the fieldwork. Moreover, the secondary data can also enrich the process of making questions for semi-structured in-depth interviews at the primary data collection stage. Secondary data were obtained through contact persons in each city government agency in the form of data archives such as reports, minute of meetings and academic script during the policy-making process. Furthermore, planning and policy documents as well as related regulations, within the scope of the city government, provincial government and even the central government were obtained through internet-based search.

The collection of primary data delivers qualitative data, which is the main source of this research, in the form of semi-structured in-depth interviews and direct observation that are recorded by using digital recorder and by making several notes. The fieldwork was conducted in early October until mid-December 2018 in Semarang Port-city. Direct observation during fieldwork provides visual conformation of the phenomena, which takes place naturally. Then, this can enrich the interview process with the triangulation approach. Semi-structured in-depth interviews were conducted using a list of open-ended questions as a guide. The list is an implementation of the conceptual frameworks, which is a derivation of related theory building blocks. In general, this list contains questions anticipated to answer the main research question, namely how integrated policy-making can contribute to the sustainable development of port and port-city relationship, in the case of Semarang port-city. Moreover, those questions are also supposed to answer the sub-research questions as follows:

- 1) How can be understood about the relationship between port and port-city?
- 2) How can the role of sustainable development in the port and port-city relationship be understood and operationalized, in the case of Semarang Port-city?
- 3) What is the role of integrated policy-making in achieving sustainable development in Semarang Port-city?

Interviews were conducted with the official representative of each of the relevant stakeholders.

The main stakeholder is Semarang City Government as the holder of the local government authority such as the Mayor, Regional Development Planning Agency, City Spatial Planning Office, Transportation Office, Public Works Office, as well as Environmental Office. Other chief stakeholders are the Office of Harbourmaster and Port Authority (KSOP) as the port authority holder and Pelindo III as the port operator holder. In addition, interviews were also conducted with other stakeholders involved in the policy-making process, such as Semarang City Regional People's Representative Council (DPRD II/City Council), academics, experts, professionals, business associations, professional associations, non-governmental organizations, communities, journalists, heads of sub-districts in the port area, and heads of RWs/RTs (the smallest informal unit of society), informal leaders also residents around the port area. The respondents are expected to have sufficient capacity, knowledge and authority regarding the policy-making process in the field of land use, transportation and environment in Semarang Port-city.

In order to meet the appropriate persons, formal procedures were carried out to contact each representative of above mentioned relevant stakeholders. Each of these stakeholders or agencies would then give recommendations about the appropriate persons to contact who are directly involved in policy-making process or who currently have the authority in policy-making. The respondents were generally top level and middle level officers who were at least the first level officers, and several retired top and middle level officers who were directly involved in the policy-making process. The appropriate respondent representative of each of the above stakeholders were located using snowball sampling technique by asking for recommendations from active officers, so that all representative of each stakeholder are represented by at least one qualified respondent.

All data were processed by using Qualitative Data Analysis method. All recorded interviews were transcribed from voice data to written data. Furthermore, coding process was carried out using Atlas.ti-8 software by incorporating all interview transcript data, interview notes and secondary data such as policy documents, regulations and minute of meetings. The coding process, by using a deductive approach, began with organizing indicators obtained from the conceptual framework. The indicators serve as a preliminary guideline in the coding process. Data were grouped according to their similarity to

each other, then the groups were given names as new sub-indicators that still have relevance to their existing indicators. Furthermore, when the similarities of the groups did not have relevance to the existing indicators or sub-indicators as contained in the conceptual framework, new indicators and sub-indicators would emerge. After that, all the results of coding process were arranged in the matrix form so that the composition and relationship among the indicators and sub-indicators are clearly visible.

Based on the results of the coding process, the interview data from various respondents with similar indicators or sub-indicators were compared and analysed with the documents from secondary data such as policy documents, regulations and minute of meetings. Elaboration of this analysis process to ensure the relationship and consistency between indicators and sub-indicators contained in the conceptual framework in answering the main research question along with sub-research questions.

4 FINDINGS

4.1 The Challenge of Inter-territorial Integration in the Complexity of Authorities

There is a complexity of authorities in the relationship between the port and the city among the main stakeholders in Semarang City. In this port-city, there are three main stakeholders namely the city government, the port authority and the port operator. These main stakeholders have significantly affected the relationship between the port and the city. In general, the relationships among the key stakeholders in the Semarang Port-city can be described in figure 2.

The first main stakeholder is Semarang City Government, as the authority holder of the local government of the city. The City Government is equipped with agencies and offices that generally represent each sector. This research only focuses on agencies from the Semarang City Government which are related to policies in the fields of land use, transportation and environment. Agency and offices that have the primary role are the Mayor, Regional Development Planning Agency, City Spatial Planning Office, Transportation Office and Environment Office. The city government has authority over the area of Semarang City in the form of regional autonomy, while the mayor is elected directly by the citizens of the city through regional

elections held every five years. Regional Development Planning Agency is a body that has a coordinative nature among the sectors, while City Spatial Planning Office, Transportation Office and Environmental Office are the bodies that deal with the related sectors. All of these agency and offices are responsible to the Mayor.

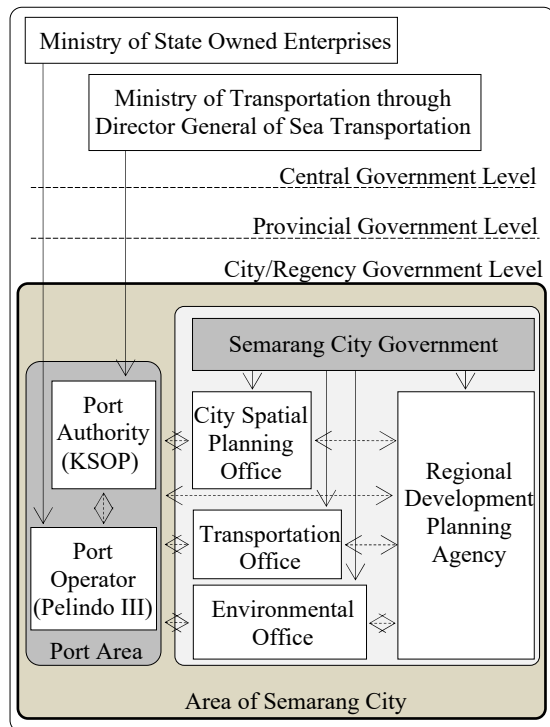


Figure 2: The complexity of authorities among the main stakeholders in Semarang Port-city.

The second main stakeholder is the Office of the Harbourmaster and Port Authority (KSOP), as the authority holder of the port in Semarang City. KSOP is the office that is responsible to the Ministry of Transportation through the Director General of Sea Transportation, so structurally this office is part of the Central Government. KSOP has authority over the port area in Semarang City.

The third main stakeholder is Pelindo III, as a port operator in the port of Tanjung Emas in Semarang City. Pelindo III is a state-owned enterprise that is responsible to the Ministry of State-Owned Enterprises, so that this company is organically part of the Central Government. As a business entity, Pelindo III is a port operator in the form of a Port Business Entity (BUP). In addition to running its business, Pelindo III has Land Management Rights (HPL) granted by the Central Government to the Tanjung Emas Port Working Area, in Semarang City.

The complexity of authorities among the main stakeholders occurs because there are differences in command lines and responsibilities of each of the main stakeholders (see figure 2). In general, in daily operational activities, both KSOP and Pelindo III have to comply with all applicable policies and regulations, both at the central, provincial and regional levels. Pelindo III, as a port operator is fully responsible to the Ministry of State-Owned Enterprises in terms of corporate and financial responsibilities, but Pelindo III has to also comply with all regulations set by the Ministry of Transportation through the KSOP as port authority in terms of shipping safety. Whereas as a stakeholder who has an interest in Semarang City, Pelindo III has to also comply with applicable local regulations at the city or provincial level. KSOP, as the port authority, is directly responsible to the Ministry of Transportation. Besides that, KSOP and Pelindo III, as a port operator, work together in maintaining the safety of shipping and complying with local regulations set by the Mayor or the Governor.

Semarang City Government as the owner of authority in Semarang City area must also comply with all regulations set by both the central government and the provincial government. As an autonomous region, Semarang City Government has the right to issue Regional Regulations, together with Semarang City Regional People's Representative Council (DPRD II / City Council). Policy in the field of land use is regulated in the Semarang City Regional Regulation on Regional Spatial Planning (RTRW). Since a while ago, the port area has been considered as the area of the central government that seems to have special autonomy, so that Semarang City Government cannot regulate land use within the port area.

The complexity of authorities among main stakeholders is a major challenge in implementing integrated policy in Semarang Port-city particularly in the fields of land use, transportation and environment. This is because port is an area that has major impacts on the city, not only positive impacts on economic and social development but also negative impacts on land use, transportation and environmental aspects. This paper only focuses on aspects of land use. In land use zoning, as stipulated in Semarang City Regional Spatial Plan, land use within the port area is only set as a transportation facility, in general, without further description related to land use in detail of the zoning of the transportation facility. This is because Semarang City Government does not have authority to regulate

within the port area, only KSOP, as the port authority holder, has the right to regulate zoning within the port area.

Port, in carrying out its activities, has to make a Port Master Plan for future development. In the process of making the Port Master Plan, the KSOP has to obtain approval and recommendations from the Mayor. This is actually a good opportunity as a starting point for the City Government to be involved in land use planning within the port area, together with the port authority, so that the negative impacts of the port from land use aspects can be anticipated such as slum dwelling around the port area and mixed land use for zoning as warehousing and residential areas. However, this is not easy to execute because there is no mandatory for the Port Authority to implement all the recommendations of the Mayor. The recommendations are implemented within the authority of the KSOP as the port authority holder. When the Port Master Plan is finished and determined by a Ministerial Decree, the Port Master Plan must be fully accommodated in the city land use plan in the future because it already has the legal force in the form of a ministerial decree.

Inter-territorial integration, which is an integration between different authorities with common interests such as resources and infrastructure, cannot be executed optimally among Semarang City Government, KSOP and Pelindo III. The main reason is that each stakeholder has different vertical responsibilities and has the respective authority over their respective territories. However, at least there have been efforts to carry out inter-territorial integration, even within the limits of their respective authority. There have been efforts from the port authority to request approval and recommendations from the Mayor in the process of making the Port Master Plan. Likewise, Semarang City Government employs cooperation and coordination with KSOP and Pelindo III by adopting the Port Master Plan entirely into Semarang City Regional Spatial Plan. This decision was taken with the consideration that the master plan regulates within the port area that the city government does not have the authority to regulate the land use within port area. Communication among competent related parties has begun even though it still uses formal communication and time consuming in the process of coordination and integration because each party has to consolidate internally according to their respective authorities and responsibilities.

4.2 Horizontal Integration through the Formation of Cross-sector Ad-hoc Team

The land use policy in Semarang City is contained in the Semarang City Regional Spatial Plan which is determined based on the Semarang City Regional Regulation number 14 of 2011 regarding Semarang City Regional Spatial Plan for 2011-2031.

In the policy-making process, City Government forms a technical team which is an ad hoc cross-sector team with the Regional Development Planning Agency as the leading institution because this body has a horizontal coordinating function among the sectors under the city government. This technical team works for a duration of one fiscal year with the main task of policies-making in the field of spatial planning. This land use policy concerns almost all sectors in Semarang City that requires space as the place to carry out its activities. The technical team essentially requires bureaucrats as member, who have proficiency, capability and in-depth knowledge related to policies in their respective sectors, so that they can represent the policies of each sector they represent. With the team members who meet the requirements, the discussions are expected to be valuable to yield prompt and qualified decisions. The members of this technical team consist of sectors whose activities are closely related to spatial requirements, such as the Regional Development Planning Agency as the coordinator, the City Spatial Planning Office, the Transportation Office, the Environmental Office, the Agriculture Office, the Housing and Settlement Area Office, and Public Works Office, as well as Land Office. The technical team is in charge of drafting concepts with the assistance of a consultant team, which has been selected through an auction process with the task of assisting the process of collecting data and providing updates on regulations related to spatial planning.

The horizontal integration process begins when the technical team works intensively in formulating policies which are realized through intensive weekly meetings. There are two types of meetings to build two-way communication such as plenary meeting and certain sectors meeting. Plenary meeting tend to use formal communication while certain sectors meeting mostly use informal communication apart from formal communication. At this stage, horizontal and intra-sectoral integration occurs among members of the Technical Team, who represent various city government sectors.

Sectoral egos are phenomena that arise when discussing policy formulation related to sector priorities, for instance when discussing zoning for housing. The agriculture office is insistent and adamant to maintain total amount of sustainable rice fields that has to be maintained in Semarang City, as a mandate from the Ministry of Agriculture through the provincial government. In another perspective, the Housing and Settlement Area Office proposes to increase the area for housing because the need for new residential areas always increases every year. This dynamic ends with the emergence of a common understanding that the purpose of the spatial planning policy is for the common goal, namely the development of Semarang City as a whole. However, it is cross-sectoral, so that it will not be able to please all sectors. Fortunately, after calculating in more detail, the need for additional areas for housing can be met while maintaining the amount of land area for sustainable rice fields.

4.3 Civil Society Involvement through Public Consultation

After the policy-making process in the land use sector is complete, the next step is to conduct public consultations. This process is carried out with the aim of socializing with the community and at the same time to collect community aspirations. This consultation process is mandatory by the regulations for a minimum of two times. The public consultations that have been carried out more than twice uses two approaches. The first approach is territorial based, namely by holding meetings in each sub-district attended by village heads, informal leaders, communities and non-governmental organizations. The second approach uses the focus group discussion method on certain segments of society such as academics, experts, professionals, business associations, and professional associations.

In the first approach, which is regional-based, the public consultations tend to be a formality, because it is held in a short time without giving much time for the communities to learn more about the draft material on land use policy. Usually the material given to participants at the beginning of the event is in the form of an executive summary, so there is not enough time for participants to study deeply. In addition, generally people still do not have sufficient knowledge and capacity in understanding spatial planning so that their questions and aspirations tend to be normative, qualitative and pragmatic, for instance the questions are related to micro problems such as clogged drains and damaged road within a

residential complex. Spatial planning delivers beyond than small matters.

In the second approach, which is based on certain segments of society, the public consultations tend to be of higher quality because most of the participants have the capacity to understand spatial policy and many have interest in commercial use of space.

4.4 Vertical Integration through Provincial and National Review

After completing the technocracy process in the technical team, the next process is political process. This political process involves Semarang City Regional People's Representative Council (DPRD II) or City Council in discussing Draft of Regional Regulation concerning Semarang City Regional Spatial Plan. At this stage, there is lot of input from the members of the city council in voicing the aspirations of their constituents, related to various issues such as economic, social and environmental issues, including the emergence of aspirations from the business sector. The dynamics during this discussion process are the involvement of the public, which is indirectly represented to the members of the city council involved. When the political process with the city council was completed, then the draft was approved by the city council in a plenary session. Next step is the application for the Governor's approval letter.

In this stage, vertical integration appears when there is a review of Semarang City Regional Spatial Plan from a policy perspective at the provincial level, such as in the form of maintaining the amount of land for sustainable paddy fields. This vertical integration process tends to use formal communication and is time consuming. After the provincial policy of the province is accommodated, then the governor's approval letter is issued. Then the vertical process needs to request for approval from the Ministry of Public Works to be aligned with various national policies. Then, as the last vertical integration process, the approval of the Ministry of Home Affairs was requested before being stipulated as a Semarang City Regional Regulation Number 14 of 2011 concerning Semarang City Regional Spatial Planning for 2011-2031.

4.5 Policy Responses in the Field of Land Use to Port Impacts

The city government has not yet made a specific study or assessment on the response to the positive and negative impacts of the port on the development

of Semarang City comprehensively. Semarang City Government also has not yet specifically counted the responses to positive and negative impacts of port on city development, as the main consideration in the policy-making particularly in the field of land use in Semarang Port-city.

However, there have been efforts from Semarang City Government to pay attention to the influence of port on the city development, although partially. This can be seen in the change in land use zoning in the area around the port. The existing condition is zoning for residential areas, while the zoning plan for the area around the port is mostly used for warehousing and mixed functions consisting of residential areas, trade and services.

5 CONCLUSIONS

In general, the main stakeholders in the port-city such as Semarang City Government, the Office of the Harbourmaster and the Port Authority (KSOP) as well as Pelindo III, realize that in carrying out their duties, they have to cooperate, coordinate, and even integrate with each other, who often differ in authority and perspective. Semarang City, which is one of the major port cities in Indonesia, faces several challenges in carrying out the process of integration in the policy-making, particularly in the field of land use. These main stakeholders have made a number of collaborative, coordinated and integrated efforts in the policy-making process, within their respective authorities, particularly in the field of land use to achieve a common goal, namely for the future development of Semarang City.

The complexity of authorities is a significant challenge in conducting inter-territorial integration where each major stakeholder cannot break through the differences in authority and structure of responsibilities in policy-making in the field of land use. So what has been done is optimally integrating within each authority. In making the Port Master Plan, the Harbourmaster and the Port Authority (KSOP) requests approval and recommendations from the Mayor, which is mandatory based on regulations. On the other hand, in the process of making Semarang City Regional Spatial Plan, the technical team also made an integrated effort by discussing and accommodating the Port Master Plan into Semarang City Regional Spatial Plan on the land that was intended specifically for the port. Furthermore, the city government made some adjustments in response to the presence of port in the area, such as changing the designation around the

port area for warehousing, trading and service functions. In this inter-territorial integration, among related stakeholders tend to use formal communication approach. This process is time-consuming because each party must adjust according to their respective regulations and authorities.

Horizontal integration occurs among several bodies under the authority of Semarang City Government. The formation of a technical team, which is an ad hoc cross-sectoral team, is a strategy to reduce sectoral egos that emerge from each sector. This sectoral ego arises because each of these sectors must meet the provisions of the relevant Technical Ministry. In general, the problems of sectoral egos can be overcome by the technical team. Each sector understands the main shared goals under the authority of Semarang City Government. This integration process uses two kinds of communication approaches such as formal communication and informal communication. However, the informal approach for communication is more often used. Each member of the technical team already knew each other well. Interpersonal interactions among the members of technical team are caused by several things, among others, due to the mutation process between agencies and offices in the scope of Semarang City Government on a regular basis. This mutation process could be due to promotion or rotation process. With this mutation process, people can get to know each other personally. In addition, because most of the agencies and offices are located in the same office complex, the process of social interaction leads them to get to know each other well. Getting to know each other personally and egalitarian attitude among the members of the technical team makes this technical team solid and makes it easier and faster in solving problems that arise between sectors, including the emergence of sectoral egos. This horizontal integration process is an effort of city government to carry out a bottom-up approach in policy-making by accommodating the aspirations of each sector within the city government based on applicable regulations, including involving the participation of civil society by conducting public consultations during the policy-making process.

Vertical integration occurs when the review processes are carried out, by both the provincial government and central government, to ensure that programs from the province and the central government are accommodated in the Semarang City Regional Spatial Plan. This integration process uses a formal communication approach. This process is time consuming because each level of government

has to ensure that their respective policies are accommodated. This vertical integration process is an attempt by the government to carry out a top-down approach in policy-making, because when provincial and central government policies are not accommodated by the city government, Semarang City Regional Spatial Plan cannot be ratified.

The involvement of civil society is possible in the policy-making process through public consultation, because this is mandatory by the regulations, at least two times. The public consultations were carried out more than twice during the land use policy-making process. Although in practice, the aspirations of civil society could not all be accommodated because of differences in the capabilities of the communities in understanding spatial planning. In addition, regulations regarding spatial policy-making are also a tool used to filter all public aspirations. This public consultation process is an attempt by city government to involve civil society participation in policy-making process, even though the accommodations of these aspirations still have to refer to the relevant regulations.

In the policy-making process in the field of land use, the city government has not yet considered a specific study or assessment on the responses of the city government to the positive and negative impacts of the port on city development in the future. However, there have been efforts to respond to the existence of port by changing zoning of land use in the area around the port, from residential areas to zoning for warehousing and mixed functions consisting of residential areas, trade and services. With this zoning change, it will support the port's operational activities and at the same time be able to utilize the positive impacts of the port, particularly in the economic development of Semarang City.

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Wind and Significant Wave Height at Indonesian Seas based on ERA5 Reanalysis Data from 2008 to 2018

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Keywords: Significant Wave Height, Wind, ERA5 Reanalysis Data.

Abstract: Wind and waves are important elements in coastal engineering and oceanography. Research on variations of in-situ winds and wave's measurements is still very limited in Indonesia. Therefore, wind speed and significant wave height data from ERA5 reanalysis data, provided by the European Centre for Medium-Range Weather Forecast (ECMWF), and is used in this study. ERA5 data that has a spatial resolution of $0.25^\circ \times 0.25^\circ$ is used to analyze correlation between wind speed and significant wave height at Indonesian seas in 2008 to 2018. The results show that wind speed and significant wave height in the closed water (inter-island in Indonesian archipelago) has a higher correlation compared to the Indonesian open sea that facing the Indian Ocean or Pacific Ocean.

1 INTRODUCTION

Wave height at sea level is very dependent on sea wind conditions. Understanding of tides and waves of sea water is a major requirement in the management of coastal areas (Yulius et al, 2017, Khoiri, 2015). Information about the sea wave itself is also very much needed in various maritime interests and activities. There are so many maritime studies that require this information, for example in research on breakwater structures (Torum et al, 2012). Significant wave height data is used to determine the speed of sediment transport on sandy beaches (Van Rijn, 2014). Significant wave height data is also needed in planning offshore buildings, for example in research on water breaking on ships (Dehghani et al, 2017) or effect of global warming on wave climate (Zikra et al, 2015).

Significant wave height is one of the important variables in disaster mitigation and coastal vulnerability using the CVI (Coastal Vulnerability Index) method (Serafim et al, 2019). This is used to estimate the vulnerability of coastal areas to the impacts of disasters that might occur. In the field of shipping, the height of sea waves is one of the important factors that are taken into account in

determining the recommendations of ship shipping safety (Lutfiana and Tirono, 2013).

Analysis of significant wave height needs to be done because of the importance of information about significant wave heights needed for the benefit of research and maritime activities. One of the data that can be used in a significant wave height analysis is satellite data. Satellite data from the European Center for Medium Range Weather Forecast (ECMWF) will be used in this final project. ECMWF is considered to have reliable atmospheric data for research and evaluation (Hwang et al, 2019). ERA5, the fifth generation of ECMWF Reanalysis, is one of the global atmospheric reanalysis data products provided by ECMWF. ERA5 is an ECMWF product made to replace its predecessor, ERA-Interim. ERA5 data has a higher resolution than ERA-Interim (Hersbach and Dee, 2016). ERA 5 has a grid size of $0.25^\circ \times 0.25^\circ$, while the ERA-Interim has a grid resolution of $0.75^\circ \times 0.75^\circ$. Olauson (2018) concluded that ERA5 has a lower error value and has more advantages than MERRA-2 (Olauson, 2018). Because of the various advantages of ERA5, the ERA5 data will be used in this research to analysis the variation of significant wave height (H_s) and wind speed in Indonesian waters.

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2 DATA

The wind and wave data used in this study is obtained from ECMWF ERA5 (<https://www.ecmwf.int>). Data obtained is hourly data in Indonesia seas during 2008 to 2018. The location points reviewed are:

1. Western Sumatera, Indian Ocean ($3,75^{\circ}$ S and $99,75^{\circ}$ E)
2. Sunda Strait (6° S and $105,75^{\circ}$ E)
3. Southern Java (8° S and 110° E)
4. Malaka Strait ($3,75^{\circ}$ N and $99,75^{\circ}$ E)
5. Java Sea ($6,75^{\circ}$ S and $112,5^{\circ}$ E)
6. Flores Sea ($6,75^{\circ}$ S and $121,5^{\circ}$ E)
7. Makassar Strait ($1,5^{\circ}$ S and $117,5^{\circ}$ E)
8. Halmahera Sea ($1,5^{\circ}$ N and $129,25^{\circ}$ E)
9. Northern Papua ($0,25^{\circ}$ S and 135° E)
10. Jayapura ($2,25^{\circ}$ S and $140,5^{\circ}$ E)



Figure 1: Reviewed location points in Indonesian Seas.

These locations are divided into 3 zones, namely open water zone facing the Indian Ocean (locations 1, 2 and 3), sheltered water zone (locations 4, 5, 6, and 7), and open water zone facing the Pacific Ocean (locations 8, 9 and 10).

3 RESULT AND DISCUSSION

3.1 Variation of Wind Speed

The trend of wind speed on monthly average during 2008-2018 can be seen in Table 1. In Southern Sumatera, the statistically significant trend is only appeared in September's with monthly maximum wind speed of $0.193 \text{ m.s}^{-1}/\text{year}$. In Sunda Strait, the monthly maximum wind speed has a significant increasing trend of $0.176 \text{ m.s}^{-1}/\text{year}$. Monthly maximum wind speed and mean wind speed in Southern Java have a significant increasing trend in

November and December. The trend of monthly maximum wind speed in Southern Java has a significant increasing

The trend of monthly maximum wind speed in November is $0.279 \text{ m.s}^{-1}/\text{year}$ and in December is $0.364 \text{ m.s}^{-1}/\text{year}$. The monthly mean wind speed trend in November is $0.104 \text{ m.s}^{-1}/\text{year}$ and in December is $0.126 \text{ m.s}^{-1}/\text{year}$. Monthly maximum wind speed in Malaka Strait has a value of increasing trend at $0.191 \text{ m.s}^{-1}/\text{year}$ in February, $0.201 \text{ m.s}^{-1}/\text{year}$ in April, $0.208 \text{ m.s}^{-1}/\text{year}$ in November, and $0.255 \text{ m.s}^{-1}/\text{year}$ in December. Whereas the monthly mean wind speed in Malaka Strait has a significant increasing trend in January ($0.147 \text{ m.s}^{-1}/\text{year}$), February ($0.066 \text{ m.s}^{-1}/\text{year}$), May ($0.051 \text{ m.s}^{-1}/\text{year}$), June ($0.046 \text{ m.s}^{-1}/\text{year}$), September ($0.174 \text{ m.s}^{-1}/\text{year}$), October ($0.147 \text{ m.s}^{-1}/\text{year}$), and December ($0.112 \text{ m.s}^{-1}/\text{year}$). The calculation of monthly mean wind speed trend in Java Sea shows that there is no significant trend every month throughout the period of 2008-2018. Meanwhile the monthly maximum wind speed trend value in Java Sea in June has a significant increasing trend of $0.095 \text{ m.s}^{-1}/\text{year}$. The calculation of maximum wind speed and monthly mean trend in Flores Sea shows that there is no significant trend every month throughout 2008-2018. The monthly average wind speed in Makassar Strait in December had a significant increasing trend of $12.52 \text{ m.s}^{-1}/\text{year}$. The trend was also significantly increase by $0.125 \text{ m.s}^{-1}/\text{year}$ in March, in contrast there was no significant monthly maximum wind speed trend value in Makassar Strait throughout the years of 2008-2018. Moreover, the monthly maximum wind speed in Halmahera Sea has a significant decreasing trend in May ($-0.375 \text{ m.s}^{-1}/\text{year}$) and a significant increasing trend ($0.172 \text{ m.s}^{-1}/\text{year}$) in October. The monthly mean wind speed in Halmahera Sea has a significant decreasing trend in May ($-0.07 \text{ m.s}^{-1}/\text{year}$) and a significant increasing trend in March and April of $0.117 \text{ m.s}^{-1}/\text{year}$ and $0.039 \text{ m.s}^{-1}/\text{year}$.

Furthermore, monthly maximum wind speed in Southern Papua does not have a significant trend throughout 2008-2018. The monthly mean wind speed in Utara Papua has a significant increasing trend in February by $0.099 \text{ m.s}^{-1}/\text{year}$, $0.081 \text{ m.s}^{-1}/\text{year}$ in April, $0.091 \text{ m.s}^{-1}/\text{year}$ in November, and $0.113 \text{ m.s}^{-1}/\text{year}$ in December. Monthly maximum wind speed in Jayapura in October has a decreasing trend of $-0.115 \text{ m.s}^{-1}/\text{year}$. The mean monthly wind speed in Jayapura has an increasing trend in February ($0.136 \text{ m.s}^{-1}/\text{year}$), November ($0.079 \text{ m.s}^{-1}/\text{year}$), and December ($0.093 \text{ m.s}^{-1}/\text{year}$). Wind rose at 10 reviewed location points during 2008-2018 can be seen in Figure 3.

Table 1: Monthly mean wind speed trend 2008-2018.

Month	Monthly Mean Wind Speed Trend (m.s ⁻¹ /year)									
	1	2	3	4	5	6	7	8	9	10
January	0.095	0.029	-0.024	0.147	-0.074	-0.104	0.057	-0.003	-0.043	-0.068
February	-0.111	-0.060	-0.016	0.066	-0.081	-0.100	0.035	0.052	0.099	0.136
March	-0.063	-0.058	-0.018	0.004	0.042	0.121	0.074	0.117	0.071	0.081
April	0.018	-0.013	0.002	0.035	0.022	-0.063	0.009	0.039	0.081	0.056
May	-0.060	-0.015	0.046	0.051	0.059	0.032	-0.019	-0.070	-0.007	0.017
June	-0.041	0.004	0.019	0.046	0.007	-0.007	-0.029	-0.027	-0.013	0.029
July	-0.029	0.005	-0.015	0.073	0.050	-0.004	0.028	0.073	-0.015	-0.068
August	0.014	0.034	0.003	0.085	0.023	-0.006	0.097	0.120	-0.019	-0.033
September	0.084	-0.019	-0.025	0.174	0.061	0.052	-0.011	0.090	-0.026	-0.018
October	0.028	0.009	-0.001	0.147	0.021	-0.015	0.043	0.068	-0.018	-0.040
November	0.084	0.025	0.104	0.080	0.070	0.047	0.083	0.061	0.091	0.079
December	0.037	0.057	0.126	0.112	0.100	-0.007	0.125	0.060	0.113	0.093

Table 2: Monthly mean significant wave height trend 2008-2018.

Month	Monthly Mean Significant Wave Height Trend (m/year)									
	1	2	3	4	5	6	7	8	9	10
January	0.014	0.012	0.000	0.002	-0.013	-0.018	0.006	0.001	-0.012	-0.009
February	-0.008	-0.017	-0.005	0.007	-0.015	-0.024	0.007	0.013	0.014	0.015
March	0.000	-0.010	0.004	0.001	0.008	0.021	0.007	-0.004	-0.004	-0.001
April	-0.002	-0.001	0.006	0.003	0.003	-0.008	0.002	0.006	-0.005	0.006
May	0.004	-0.002	0.005	-0.004	0.012	0.010	0.000	-0.004	-0.010	-0.003
June	0.013	0.005	0.007	0.003	0.000	0.007	-0.003	-0.008	0.003	-0.004
July	0.019	0.006	0.024	-0.001	0.006	0.000	0.001	0.010	-0.001	0.001
August	0.019	0.004	0.011	-0.001	0.001	-0.003	0.012	0.013	0.001	-0.002
September	0.005	-0.001	-0.006	0.003	0.011	0.007	-0.001	0.014	0.011	0.002
October	0.006	0.002	-0.003	0.002	0.003	-0.001	0.007	0.018	0.053	0.010
November	0.009	-0.001	0.009	0.002	-0.002	-0.003	-0.001	0.007	0.003	0.010
December	0.014	0.014	0.016	0.010	0.016	0.008	0.008	0.001	0.013	0.005

3.2 Variation of Significant Wave Height

Significant monthly mean wave height in Western Sumatera has a significant increasing trend in August at 0.019 m/year, while monthly maximum significant wave height in Southern Sumatera has an increasing

significant trend in July by 0.081 m/year and 0.062 m/year in August. The calculation of the maximum monthly significant wave height trend in Sunda Strait shows that there is no significant trend every month throughout the years 2008-2018, while the monthly significant wave height trend value in Sunda Strait, July, has a significant increasing trend of 0.006 m/year.

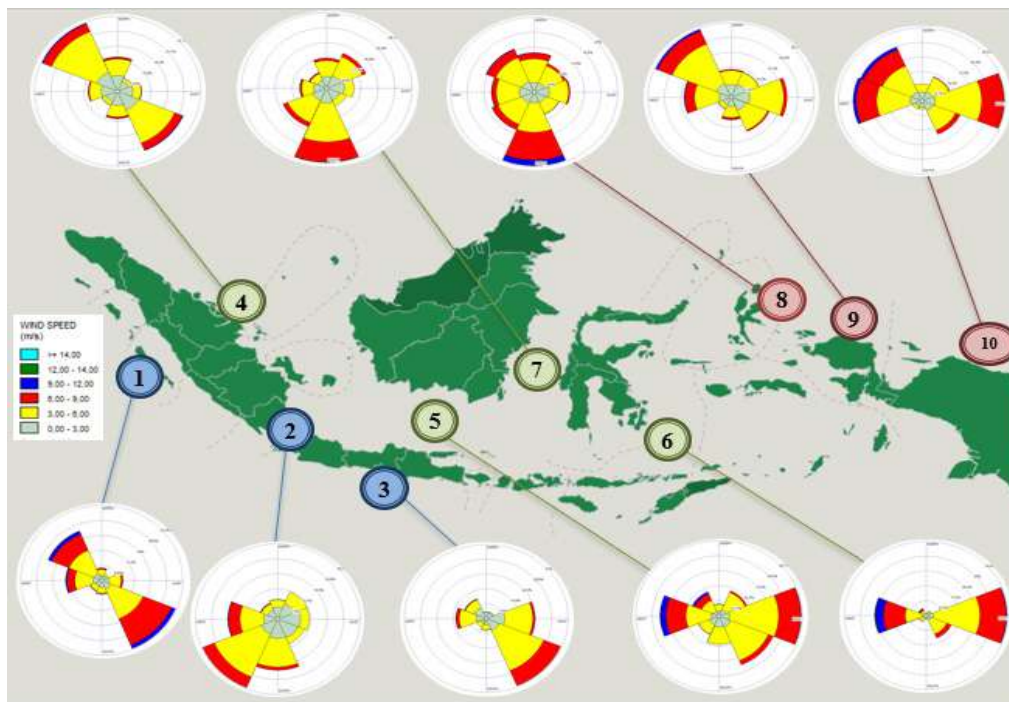


Figure 3: Wind rose at 10 reviewed location points during 2008-2018.

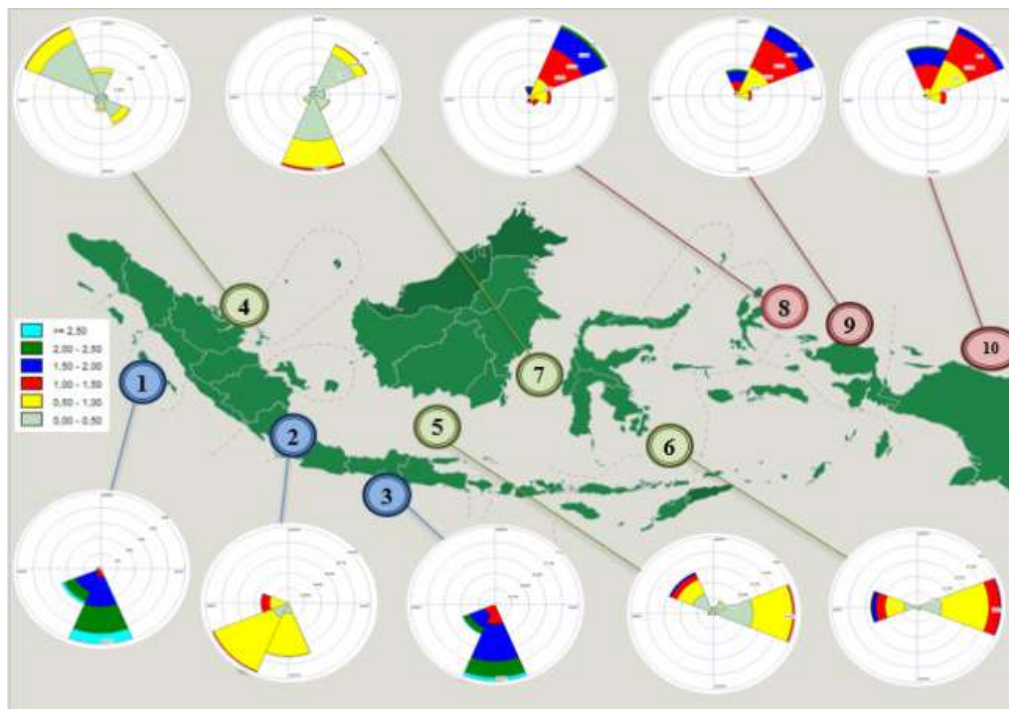


Figure 4: Wave rose at 10 reviewed location points during 2008-2018.

Monthly maximum significant wave height in Southern Java has a significant increasing trend of 0.05 m/year in May, 0.05 m/year in August, and 0.068 m/year in December.

Furthermore, the monthly mean significant wave height in Southern Java has a significant increasing trend in July of 0.024 m/year and December of 0.016 m/year. In February, there was a significant

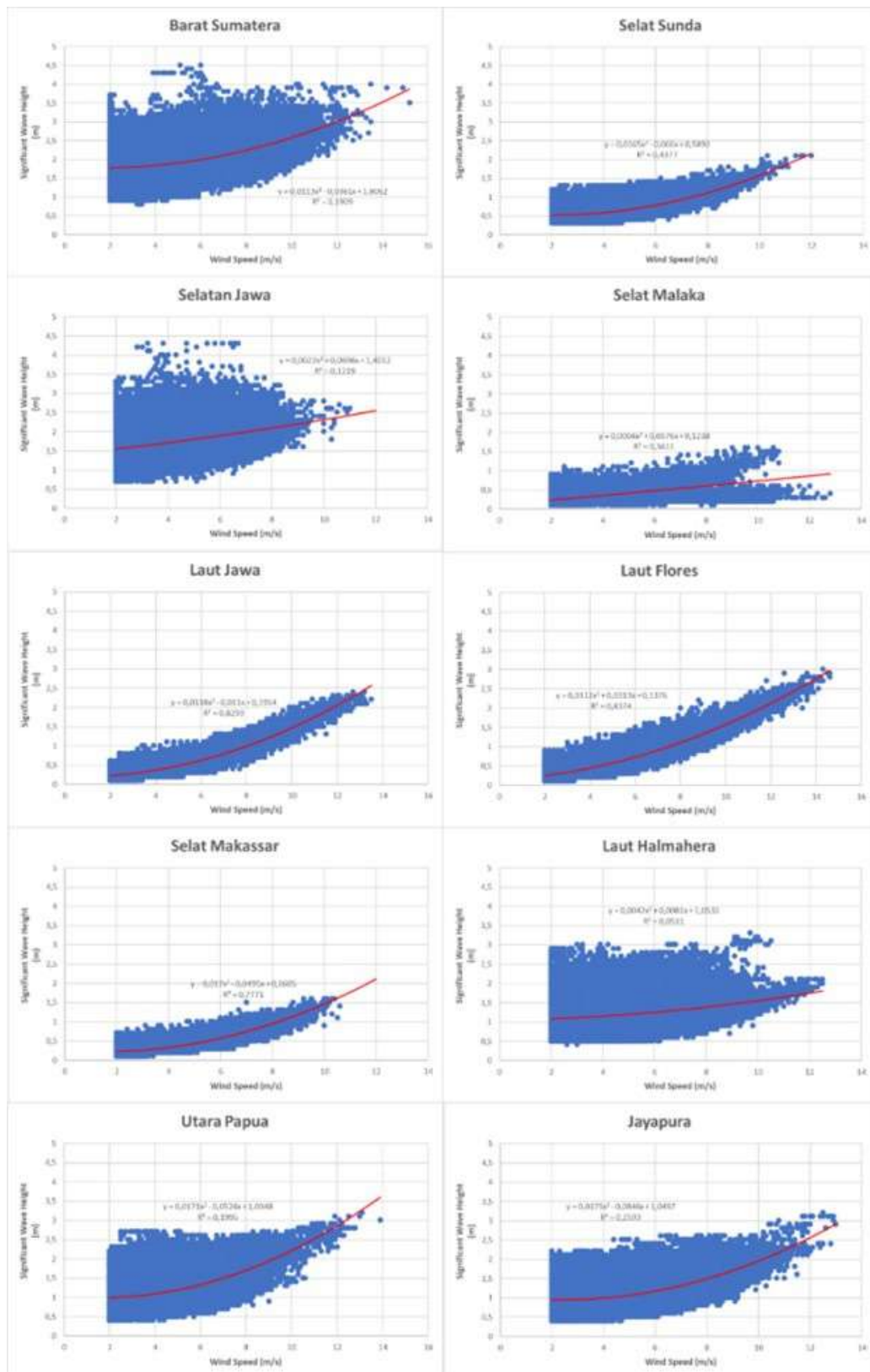


Figure 5: Regression graph of wind speed and significant wave height at Indonesian Seas.

increasing trend in Malaka Strait both at monthly maximum and mean significant wave height. For the significant monthly mean wave height, the significant trend is calculated by 0.007 m/year, while the maximum monthly mean wave height has a significant trend of 0.025 m/year. The calculation the monthly maximum significant wave height trend in Java Sea shows that there is no significant trend every month throughout the years 2008-2018. The trend value of monthly mean significant wave height in Java Sea has a significant increasing trend of 0.012 m/year in May. Monthly maximum significant wave height in Makassar Strait has a significant increasing trend of 0.028 m/year in August. Monthly mean significant wave height in Makassar Strait has a significant increasing trend by 0.008 m/year in December. The calculation of the monthly significant wave height trend in Northern Papua shows that there is no significant trend that occurs every month throughout 2008-2018. The trend of the maximum monthly significant wave height in Northern Papua in December has a significant decreasing trend of -0.055 m/year. There is no significant trend of monthly significant wave height in Jayapura during 2008-2018. Monthly maximum significant wave height in Jayapura has a significant decreasing trend in June and August, each at -0.054 m/year and at -0.03 m/year. The calculation of the monthly maximum and the monthly mean significant wave height trend in Flores Sea and Halmahera Sea shows that the trend that exists every month throughout the years 2008-2018 is not statistically significant. The calculation of monthly

mean significant wave height trend in 11 years can be seen in Table 2. Wave rose from 10 reviewed location points during 2008-2018 can be seen in Figure 4.

3.3 Relationship of Wind Speed and Wave Height

The relationship of wind speed and significant wave height in Indonesia during 2008-2018 at each reviewed location point has a positive correlation. It can be seen from Table 3 that the sheltered water of Indonesian archipelago, namely Malaka Strait, Java Sea, Flores Sea, and Makassar Strait, have a determination coefficient above 0.3. Even more, Java Sea and Flores Sea each has a determination coefficient 0.75. On the other hand, in waters facing the ocean, Western Sumatera, Southern Java, Halmahera Sea, Northern Papua, and Jayapura, the determination coefficient is below 0.3. This means that the relationship of significant wave height and wind speed in those waters is not as high as the relationship in the sheltered waters. Sunda Strait which is categorized as an open water in this paper turns out to have characteristics that tend to follow the characteristics of sheltered waters which have a quite high correlation between wind speed and significant wave height with determination coefficient of 0.438.

Regression chart of the correlation between wind speed and significant wave height in all reviewed location points can be seen in Figure 5. The regression method used is quadratic regression or polynomial order 2.

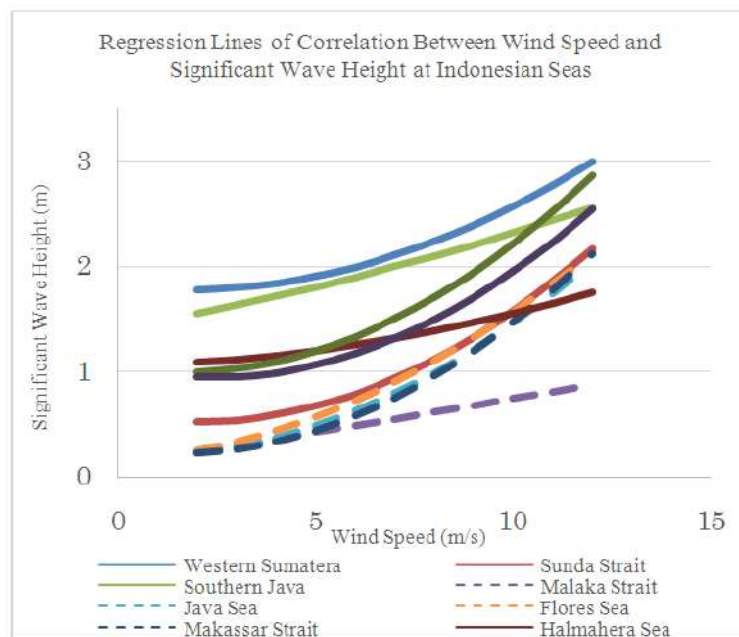


Figure 6: Regression line of wind speed and significant wave height correlation at 10 reviewed location points in 11 years.

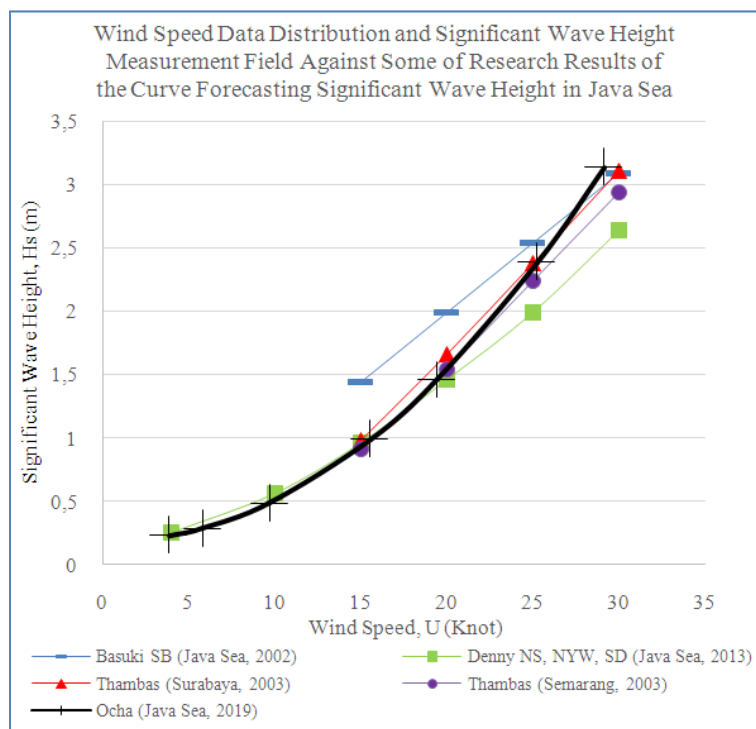


Figure 7: Wind speed data distribution and significant wave height of ERA5 data compared to some studies regarding wave forecasting in Java Sea.

It can be seen in Figure 6 that the regression lines of sheltered waters (Malaka Strait, Java Sea, Flores Sea, and Makassar Strait) are below the open water regression line, which means that the significant wave height in sheltered waters is lower than in open waters. The regression lines of sheltered waters are also more curved than open waters regression lines. It means that the correlation between wind speed and significant wave height in sheltered waters is higher than the correlation of wind speed and significant wave height in open waters. The graph only compares the wind speed and significant wave height without taking into account of the direction of the wind and wave itself. Therefore, the graph cannot be used as a reference in determining the significant wave height based on wind speed or vice versa.

Figure 7 shows the distribution data of wind speed and significant wave height from ERA5 compared to direct measurement field data on some studies of wave height forecasting in the Java Sea from Thambas and Yuwono (2003), Denny et al (2017), Basuki (2000). As seen in Figure 7, the wind speed data range is limited between 5 knots to 30 knots. Figure 7 indicated that ERA5 data has similar trend as other studies in the Java Sea.

4 CONCLUSIONS

Conclusions that can be obtained in this paper include:

1. Monthly mean wind speed at Indonesian Seas is about 2 m/s to 9 m/s. Monthly maximum and monthly mean wind speed at each reviewed location points have statistically significant trend, except at Flores Sea.
2. Monthly mean significant wave height at Indonesian Seas is about 0,2 m to 2,8 m. Monthly maximum and monthly mean significant wave height at each reviewed location points have statistically significant trend, except at Flores Sea and Halmahera Sea.

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Experimental Study of the Effect of Waves on SPAR Responses with and without Heave Plate in Intact and Damaged Mooring Systems

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Abstract: This research discusses the effects of heave plate utilization against dynamic responses of SPAR due to random waves. The analysis has been done by numerical and experimental methods. The experimental method was held in Maneuvering Ocean Engineering Basin of Balai Teknologi Hidrodinamika (BTH) – BPPT. The model scale of SPAR was constructed based on the classic SPAR prototype with taut mooring systems using four identical moorings in a 1:125 scale factor. Both experimental and numerical studies were conducted in time domain analysis with heave plate utilization in the keel of SPAR as the main variable. The diameter of the heave plate has a 1.5 ratio to the outer diameter of SPAR. Each analysis was conducted in intact and damaged mooring systems. The analyzed variables of dynamic responses are the surge, heave, pitch, and maximum offset of the SPAR. Results of both the experimental and numerical studies were then be compared, showing that the heave plate does not affect surge and pitch responses significantly. On the other hand, it significantly and consistently reduces the SPAR heave responses in every condition of analysis in both the numerical and experimental methods. The most significant reduction of SPAR heave response occurred when the mooring systems were in damaged condition, i.e., 33.29% and 27.84% heave reduction in the experimental and numerical method, respectively. The study also shows that the heave plate reduces SPAR maximum offset up to 31.69% in experimental analysis and 11.22% in numerical analysis.

1 INTRODUCTION

The necessity of hydrocarbon fuel which keep growing each year, demands hydrocarbon exploration in deep water to fulfil the hydrocarbon shortage. Therefore, the needs of structural and technology advancement for deep-water exploration cannot be neglected any further (Soeb et al, 2017). Between all development in deep-water exploration technology, SPAR floating platform has been the most optimal, efficient, and economic solution to be used as deep-water drilling and production facility (Glanville et al, 1991) (Horton and Halkyard, 1992).

SPAR floating platform has also been developed even further as a floater of floating offshore wind turbine (FOWT), which economically efficient choice in water depth above 50 meters (Jonkman and Matha, 2011).

The motion responses of SPAR is relatively low due to its very deep draught. This advantage, enhances the security of rigid risers below SPAR in deep-water production facility (Tao, Lim and

Thiagarajan, 2004). SPAR is also easier to be moved to another location, and also does not affected by water depth or earthquake (Soeb et al, 2017). Some of SPAR excellences are:

- Can be operated in deep-water up to 3000 m water depth, as drilling or production facility.
- It has absolute stability because of its center of buoyancy, which always above the center of gravity.
- Can be utilized as mobile drilling rig.
- It has better sea keeping characteristics compared to the other mobile drilling unit.
- Simplify the installation and operation of mooring and cable systems.
- Risers or other drilling units are protected inside its hull.

These advantages, makes SPAR more likely to be chose and more superior than the other alternative in utilization as drilling or production facility in deep-water (Jain and Agarwal, 2003).

The cylindrical hull of SPAR, which has a massive draft, provides wave load reduction to the

SPAR system due to its massive displacement which produces damping effect (Jain and Agarwal, 2003). SPAR also requires mooring systems to ensure its position still and stable (station – keeping). The dynamic responses of SPAR significantly affects the mooring lines tension. Yet, the type and characteristics of the mooring systems also govern the dynamic responses of SPAR (Seebai and Sundaravadivelu, 2009). Between all of dynamic loads experienced by SPAR, wave load has the most impact to SPAR dynamic responses. It is because the more closer the natural period of the structure to the wave period, the bigger dynamic responses produced (Djatkiko, 2012). Therefore, it is important to inspect the correlation of natural period of the structure and its environment in design consideration.

A collision between dynamic loads of waves and SPAR will produces dynamic responses in six degree of freedom. Those are surge, sway, heave, roll, pitch, and yaw. These dynamic responses, which hazardous to risers integrity, need to be minimized (Tao, Lim and Thiagarajan, 2004). Heave response also has been found harmful in small SPAR platform (Fischer and Gopalkhrisnan, 1998). Some solutions to reduce the heave responses of SPAR has been offered by previous studies, which are: increase the damping of the system, dissociate the natural period of the structure further from the wave period, and reduce the wave excitation forces acting on the structure (Haslum and Faltinsen, 1999). The utilization of heave plate at the SPAR keel will provide a significant increase in the damping of the system, which will also reduce the heave response of the structure (Tao, Lim and Thiagarajan, 2004). Further research results shows that the diameter of the heave plate, affects the increase of added mass, which will affects the damping of the system and the motion responses of the structure (Sudhakar and Nallayarasu, 2014). The utilization of double heave plates in the keel and the hull of classic SPAR also significantly affects the heave response of the SPAR. The diameter of the heave plates, and the distance between the heave plates, are the main variable, which contributes to the change in SPAR viscous damping (Subbulakshmi et al, 2015).

This research, discusses about the effects of the heave plate utilization in the keel of SPAR to its dynamic responses against irregular waves in intact and damaged mooring systems. The research has been done in numerical and experimental method, using classic SPAR model, which has been studied by Ivandito Herdayanditya in his research and has 1:125 scale factor, with heave plate utilization as the main modification. The numerical study has been done

using Orcflex 9.2a, and the experimental study was held in Maneuvering and Ocean Engineering Basin (MOB) of Balai Teknologi Hidrodinamika (BTH) – Badan Pengkajian dan Penerapan Teknologi (BPPT).

2 LITERATURE REVIEW

The research about non-linear response of SPAR platform due to wave, and current load in ultra-deep water and how water depth affects its responses has been studied in detail (Soeb et al, 2017). Chitrapu, et al. had also researched about non-linear responses of SPAR in varies of environment using time domain simulation (Chitrapu, Saha and Salpekar, 1998). Jain and Agarwal also accomplished a dynamic analysis of SPAR using time domain simulation, which concludes that the responses of SPAR due to waves and currents need to be restricted, since SPAR platform usually used as production and drilling facility (Jain and Agarwal, 2003). Tao, et al. also studied the correlation between heave response in classic SPAR and its viscous damping (Tao, Lim and Thiagarajan, 2004). Fischer and Gopalkhrisnan numerically and experimentally analyzed the characteristics of SPAR heave response, and represented the importance of heave response consideration in SPAR (Fischer and Gopalkhrisnan, 1998). Haslum and Faltinsen offered some solutions to reduce the heave responses of SPAR (Haslum and Faltinsen, 1999), which are:

- Increase the total damping of the system.
- Dissociate the natural period of the structure further from the wave period.
- And significantly reduce the wave load excitation forces.

Tao, et al. research, shows that the heave response of SPAR platform may be reduced by heave plate utilization around its hull, which will dramatically increase the damping of the structure (Tao, Lim and Thiagarajan, 2004). Yet, Haslum and Faltinsen mentioned that after using heave plate, the heave response of SPAR still in a critical state (Haslum and Faltinsen, 1999). Aside from heave plate utilization, an additional damping system of a SPAR may be achieved by installing helical strakes around its hull or increasing its draught. Sudhakar and Nallayarasu studied even further about the effects of heave plate utilization and its diameter to the SPAR responses, and found the optimal heave-plate diameter ratio to SPAR diameter (Sudhakar and Nallayarasu, 2014). Subbulakshmi, et al. also studied the effects of double heave plates utilization to the heave response

reduction of SPAR, and found the optimal diameter ratio of the heave plates and the optimal distance between them to reduce the heave response (Subbulakshmi et al, 2015).

3 OBJECTIVES AND SCOPE OF STUDY

The objective of this research is to comprehend how far the heave plate utilization in the SPAR keel affects the stochastic parameter of its dynamic responses. The dynamic responses which to be analysed are surge, heave, pitch, and maximum offset. The scope of study and boundaries of this research are as follows.

- Experimental and numerical study only considered one wave characteristic with 0° wave heading.
- The type of mooring systems is taut mooring system, with four identical mooring lines.
- The experimental and numerical study was conducted in the condition of intact and damaged mooring system.
- The influence of low frequency wave was not included in the consideration of the analysis.
- The only mooring line tension considered in this study is the pre-tension of mooring lines, which is similar for each mooring lines.
- Numerical study was conducted in Orcaflex 9.2a as a comparison of the experimental study.

4 METHODOLOGY

The flow and procedure of this research was conducted in stages as follows.

- The literature reviews was performed by referring to the previous study. The data collection of the laboratory, scale model, and any other experimental and numerical aspects was also conducted in this step.
- The determination of design criteria like scale factor, acceptance criteria, and so on.
- Modeling of the scale model, which comprised: the design aspects of the scale model, instrument preparation, trials, processing and analyzing the experimental data result.
- Numerical modeling of the structure, which consists: modeling stage in the software, numerical model validation, and numerical trials.
- Analyzing and comparing the results of the

experimental and numerical study followed by conclusions.



Figure 1: Scale model the SPAR with heave plate.

5 RESULTS AND DISCUSSION

5.1 Experimental Modeling

The experimental modeling consists two main section, which are scale model modeling, and mooring system modeling. The scale model of the SPAR hull was made of PVC pipe with 14 cm diameter. The heave plate was made of acrylic, while the topside was made of plywood as can be seen in Figure 1. The ballasting system of the SPAR was installed inside of the hull, which was made of six cylindrical steel with a shaft in the middle piercing each cylindrical steel. These cylindrical steel's position can be optimized inside the hull to achieve the desired stability equilibrium in water, its weight distribution parameter like keel to gravity (KG) and radius of gyration was obtained by a series of calibration. The ballasting system of the scale model was divided into two blocks of cylindrical steel. The first block consists four cylindrical steel which located in the keel of the SPAR, and the other blocks consists two cylindrical steel which located at 91.5 cm above the keel. The mooring system model was made of two section. The first section was rubber, and the second section was steel wire sling. The stiffness of the mooring lines, which need to be considered, was obtained by a series of calibration and calculation using Hooke's law equation. The data results from experimental modeling can be seen at table 1. The mooring system of the SPAR model was installed in

a configuration like Figure 2, 5.5, and 5.6. While the ballast of the SPAR and its configuration can be seen in figure 5.1.



Figure 2: Cylindrical steel as ballast (left) and its configuration inside the SPAR hull (right).

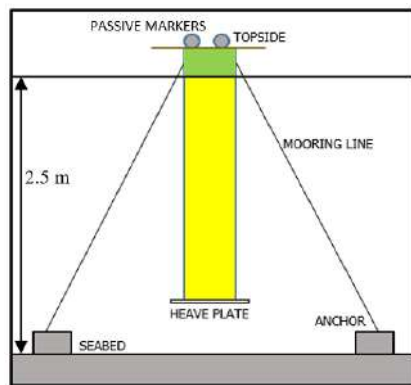


Figure 3: Side view of mooring system configuration.

Table 1: SPAR data and properties.

SPAR PROPERTIES (SCALE 1:125)				
Parameters		Scale Model Dim.	Scale Factor	Full Scale Dim.
Hull diameter (m)		0.140	λ	17.500
Keel diameter (m)		0.150	λ	18.750
Draft without heave plate (m)	Free floating	1.029	λ	128.681
	Moored	1.111	λ	138.896
Draft with heave plate (m)	Free floating	1.034	λ	129.305
	Moored	1.116	λ	139.520
Hull height (m)		1.245	λ	155.625
Keel cylinder height (m)		0.053	λ	6.625
Fairlead height of SPAR without from keel (m)		1.165	λ	145.625
Fairlead height with heave plate from keel (m)		1.170	λ	146.250

Mass of SPAR without heave plate (ton)	0.016	λ^3	31949.219
Mass of SPAR with heave plate (ton)	0.017	λ^3	32295.918
Keel to Gravity of SPAR without heave plate, KG (m)	0.457	λ	57.125
Keel to Gravity of SPAR with heave plate, KG (m)	0.452	λ	56.508
Roll radius of gyration, Rxx (m)	0.550	λ	68.750
Pitch radius of gyration, Ryy (m)	0.550	λ	68.750
HEAVE PLATE PROPERTIES (SCALE 1:125)			
Parameters	Scale Model Dim.	Scale Factor	Full Scale Dim.
Heave plate diameter (m)	0.210	λ	26.250
Heave plate thickness (m)	0.005	λ	0.625
Mass of heave plate (ton)	0.00015	λ^3	283.203
SELECTED MOORING LINES PROPERTIES (SCALE 1:125)			
Parameters	Scale Model Dim.	Scale Factor	Full Scale Dim.
Pre-tension (KN)	0.003	$\lambda^3 \epsilon$	6835.771
Rubber			
Axial stiffness, EA (MN)	0.0000196	$\lambda^3 \epsilon$	39.278
Length (m)	0.2	λ	25.000
Steel wire sling			
Axial stiffness, EA (MN)	0.00353	$\lambda^3 \epsilon$	7075.487
Length (m)	2.040	λ	255.000

5.2 Numerical Modeling

The numerical model was built based on experimental data structure in full scale. The numerical modeling also consists two main modeling section, which are structural modeling and mooring system modeling. First of all, numerical model of the SPAR was built using 3D diffraction theory in MOSES at its operation draught. The structure was modelled while using heave plate and not. Modeling in MOSES was intended to obtain the hydrodynamic and hydrostatic parameter of SPAR in a particular draught, such as damping matrix, added mass matrix, wave load RAO, and so on. After the validation of the numerical model, the numerical model of SPAR has also been analysed in Orcaflex 9.2a to simulate the random waves while SPAR in moored condition, using the

hydrodynamic and hydrostatic data obtained from MOSES as an input in Orcaflex 9.2a. The result of numerical modeling stages can be seen in Figure 4 and Figure 5.

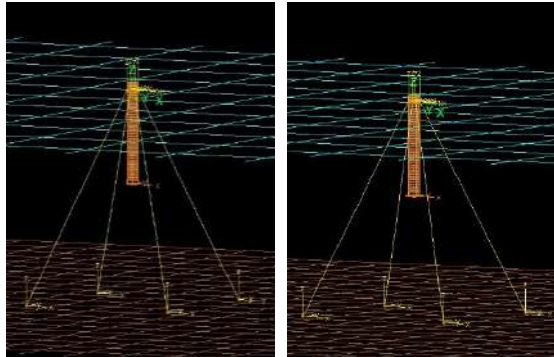


Figure 4: Numerical moored model of SPAR in Orcaflex 9.2a (Full scale) without heave plate (left), and with heave plate (right).



Figure 5: Numerical model of SPAR in MOSES (Full scale) without heave plate (left), and with heave plate (right).

5.3 Experimental Analysis

The experimental analysis was conducted in Maneuvering and Ocean Engineering Basin (MOB) of Balai Teknologi Hidrodinamika (BTH) – BPPT. SPAR and its mooring systems was attached together in the MOB based on the planned configuration. The experiment was conducted in four trial conditions, which are:

1. SPAR without heave plate in intact mooring system.
2. SPAR with heave plate in intact mooring system.
3. SPAR without heave plate in damaged mooring systems.
4. SPAR with heave plate in damaged mooring systems.

Each trial was conducted in time domain method with 6 minutes duration, which equivalent as 67 minutes in full scale referring to Froude's model scaling law. Qualysis motion capture camera, which placed under the moving bridge above the MOB, was

used to record the translational and rotational responses of SPAR. During the simulation, Qualysis recorded the whole SPAR's movement based on the coordinate of passive marker position shift, which located on the topside of the SPAR. The output from Qualysis was time history responses in six degree of freedom. Whereas the elevation of the trial's waves was measured using wave probe, which placed under the moving bridge, in front of the SPAR. The trial's wave characteristics can be seen in Table 2. The configuration of intact and damaged mooring systems used in the trials can be seen in Figure 6 and Figure 7. While the documentation of the moored model in MOB can be seen in Figure 8.

Table 2: Trial's wave characteristics.

Parameters	Values	Units
Full Scale		
Wave spectrum theory	JONSWAP	-
Hs	6.35	m
Tp	14.5	s
γ	3.3	-
Wave heading	0	Degree
Model (Scale 1:125)		
Wave spectrum theory	JONSWAP	-
Hs	0.0508	m
Tp	1.297	s
γ	3.3	-
Wave heading	0	Degree

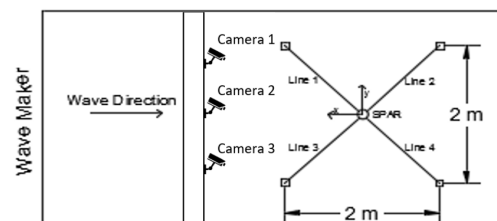


Figure 6: Intact mooring system configuration.

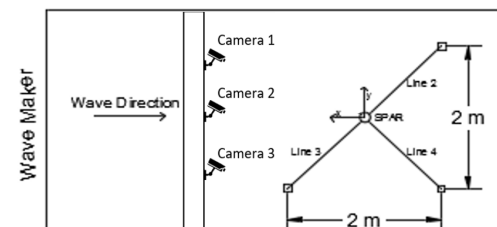


Figure 7: Damaged mooring system configuration.

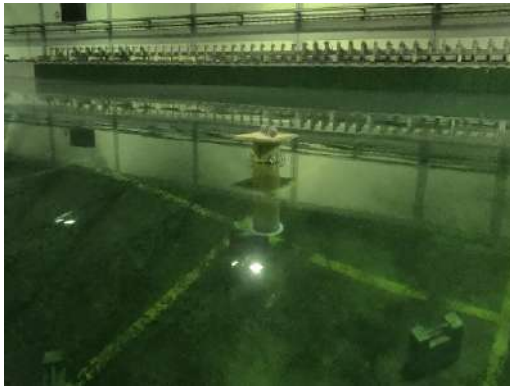


Figure 8: Moored scale model in MOB.

5.4 Numerical Analysis

Firstly, the numerical analysis was conducted in MOSES in frequency domain while SPAR in free-floating state using the operation draught. This analysis was intended to obtain the hydrostatic and hydrodynamic data of SPAR while in operation draught. Then, these data obtained by MOSES such as damping matrix, added mass matrix, and wave load RAO, will be used in Orcaflex 9.2a as an input, then further analyzed in moored condition. All trial set conducted in MOSES can be seen in Table 3.

Table 3: Trial set conducted in MOSES.

Condition	Analysis code	Operation draught
Without heave plate	1	Intact, 138.896 m
	2	Damaged, 136.845 m (based on static analysis in Orcaflex 9.2a)
With heave plate	3	Intact, 138.896 m
	4	Damaged, 137.473 m (based on static analysis in Orcaflex 9.2a)

In Orcaflex 9.2a, analysis was conducted using time domain simulation, in intact and damaged mooring system. Every aspects of simulation such as the duration of the simulation, wave heading, wave characteristics, was based on experimental data in full scale and adjusted as similar as possible. The water depth data in numerical analysis also based on the depth of MOB that scaled into full scale, which is 312.5 meters. The intact and damaged mooring system configuration in Orcaflex 9.2a is the same as experimental configurations.

5.5 Data Results Processing and Discussion

There are several steps conducted to obtain the stochastic parameter of the SPAR responses. The time history responses both from numerical and experimental analysis need to be converted into spectral density response in full scale condition for each dynamic responses. Then further analysis had been done in surge, heave, and pitch responses to obtain their stochastic parameters. Whilst the maximum offset can be obtained directly by plotting the surge and sway responses together, then measure the furthest movement of SPAR during simulation from the initial coordinate. The time history responses obtained from experimental analysis was scaled into full scale using Froude's law. The example of time history responses in full scale can be seen in Figure 9.

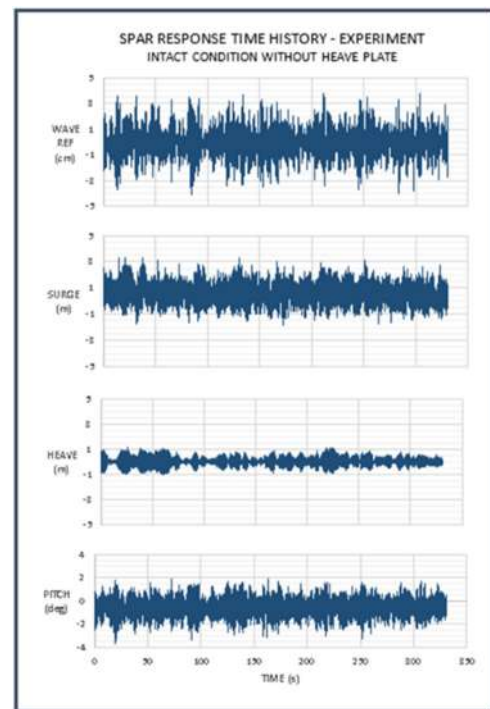


Figure 9: Time history responses output example.

Random waves and responses time history have similar characteristics, so both of them can be processed with the same method. First, the time history data need to be prepared by dividing each wave record into some segment. Every segment has a same point of measurement with the same interval length, which can be the point of zero-up crossing period, zero-down crossing period, or peak period. After that, incremental frequency or usually called

Nyquist frequency can be determined. The Nyquist frequency was used as the interval of the frequency in the converted time history responses, which was converted into a frequency domain record graphic. When the time history data ready, the conversion process was conducted in MATLAB using its Fast Fourier Transform (FFT) feature. The results, which was a record of amplitude of waves or responses in each frequency, further converted into an ordinate of spectral density curve using equation (1) (Djatkiko, 2012). The example of converted time history responses in time domain into frequency domain can be seen in Figure 10. And the example of the conversion results into the spectral density curves can be seen in Figure 11 until Figure 13.

$$S_{\zeta}(\omega)d\omega = \zeta_{n0}^2/2\delta\omega \quad (1)$$

Where,
 $S_{\zeta}(\omega)$ = Spectral density ordinate
 (m²/(rad/s))
 $\delta\omega$ = Nyquist frequency (rad/s)
 ζ_{n0} = Wave or responses amplitude (m)

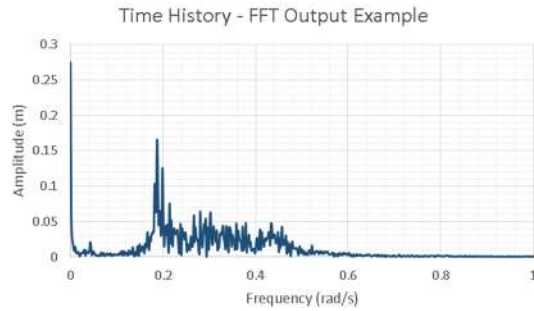


Figure 10: Example of time history FFT output (full scale).

After every component of amplitude in each recorded frequency has been converted into an ordinate of spectral density curve, each set of it was plotted into a diagram with Nyquist frequency as the abscissa. Then, the stochastic value of each diagram can be calculated based on the variants of the wave or responses elevations, which is equivalent with the area under the spectral density curve. The second, and fourth moment of spectral area can also be calculated using equation (2).

$$m_n = \int_0^{\infty} \omega^n S_{\zeta}(\omega) d\omega \quad (2)$$

Where m_0 is the variant of wave elevations or response amplitudes. m_1 , m_2 , and m_4 are the first, second, and fourth moment of spectral area. By knowing the value of these parameters, the stochastic values of each responses can be calculated. In

accordance with the scope and boundaries stated before, the contribution of the low frequency components are neglected. Therefore, the stochastic value calculations are started from the wave frequency of 0.26 rad/s, which is the lower limit frequency in wave energy spectrum density.

The stochastic responses calculated in this research are: significant responses, mean of 1/10 highest responses, mean responses, the most probable extreme responses, and the most probable extreme responses with a probability of 99% confidence not exceeded. Those stochastic responses can be calculated using equations as follows.

$$\zeta_{n_s} = 2\sqrt{m_0} \quad (3)$$

$$\zeta_{n_{av}} = 2.5/2\sqrt{m_0} \quad (4)$$

$$\zeta_{n_{1/10}} = 5.08/2\sqrt{m_0} \quad (5)$$

$$\zeta^- = \sqrt{m_0} \times \sqrt{2 \ln \left(\frac{60^2 T}{2\pi} \sqrt{\frac{m_2}{m_0}} \right)} \quad (6)$$

$$\zeta_{\alpha}^- = \sqrt{m_0} \times \sqrt{2 \ln \left(\frac{60^2 T}{2\pi \alpha} \sqrt{\frac{m_2}{m_0}} \right)} \quad (7)$$

Where,

ζ_{n_s} = Significant responses (m)

$\zeta_{n_{av}}$ = Mean responses (m)

$\zeta_{n_{1/10}}$ = Mean of 1/10 highest responses (m)

ζ^- = The most probable extreme responses (m)

ζ_{α}^- = The most probable extreme responses with a probability of 99% confidence not exceeded (m)

T = Duration of waves-making (s)

α = confidence number, 0.01 for 99% confidence not exceeded (m)

According to the results, the response changes between before and after heave plate utilization in each type of stochastic responses are all the same in percentage. Therefore, to simplify the analysis in seeing the changes that occurs, the significant responses was chosen as a representation of the SPAR responses in each motion, which was compared between each condition.

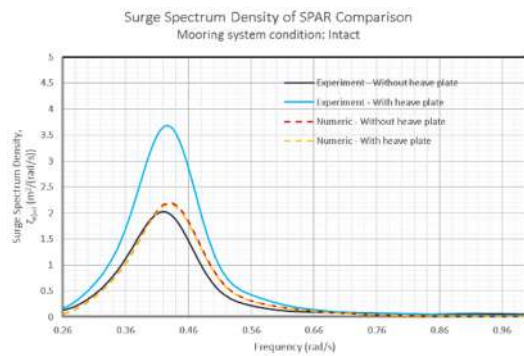


Figure 11: Example of surge responses spectrum density.

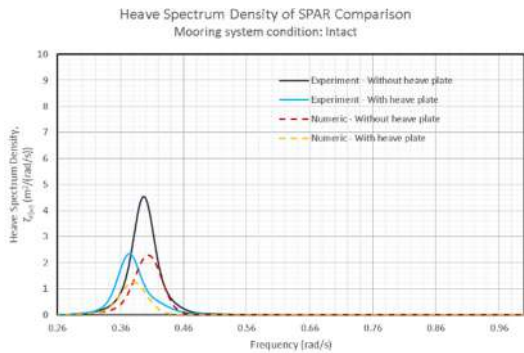


Figure 12: Example of heave responses spectrum density.

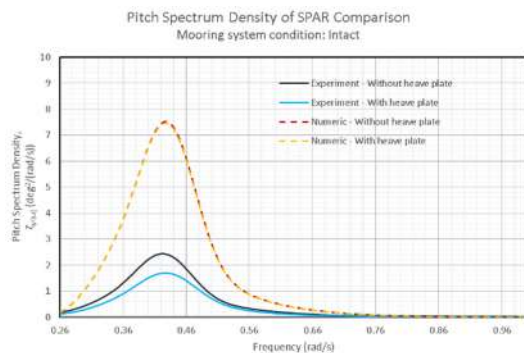


Figure 13: Example of pitch responses spectrum density.

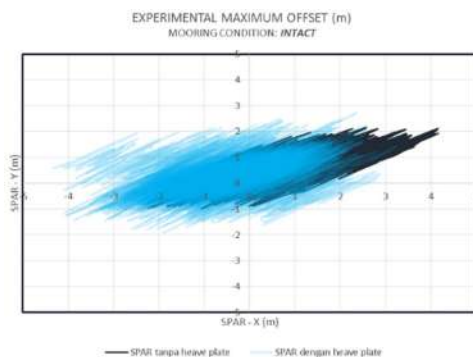


Figure 14: Offset of SPAR graphic example.

All responses assessed from experiment and numeric method was plotted together in both intact and damaged condition to observe the responses reduction due to heave plate utilization more clearly. As discussed above, the area below the spectral density graph is the response's energy which equals to the response's value. Thus, from those comparative graphics, it can be concluded that there are some cases of response amplifications and reductions after heave plate utilizations. All the results are presented in Table 4 to Table 7 below.

5.5.1 Surge Responses

All of the calculation's results of surge significant response's changes in each conditions are presented in Table 4 below.

Table 4: Significant surge of SPAR.

Significant surge responses (m) – Experimental			
Condition	Without heave plate	With heave plate	Reduction
<i>Intact</i>	1.64	1.74	-6.15%
<i>Damaged</i>	1.83	1.82	0.60%
Significant surge responses (m) – Numerical			
Condition	Without heave plate	With heave plate	Reduction
<i>Intact</i>	1.16	1.15	0.84%
<i>Damaged</i>	1.06	1.05	0.86%

5.5.2 Heave Responses

All of the calculation's results of heave significant response's changes in each conditions are presented in Table 5 below.

Table 5: Significant heave of SPAR.

Significant heave responses (m) – Experimental			
Condition	Without heave plate	With heave plate	Reduction
<i>Intact</i>	0.97	0.73	24.16%
<i>Damaged</i>	1.06	0.71	33.29%
Significant heave responses (m) – Numerical			
Condition	Without heave plate	With heave plate	Reduction
<i>Intact</i>	0.73	0.54	26.39%
<i>Damaged</i>	1.12	0.71	27.84%

5.5.3 Pitch Responses

All of the calculation's results of pitch significant response's changes in each conditions are presented in Table 6 below.

Table 6: Significant pitch of SPAR.

Significant pitch responses (m) – Experimental			
Condition	Without heave plate	With heave plate	Reduction
<i>Intact</i>	1.46	1.20	17.65%
<i>Damaged</i>	1.34	1.40	-4.56%
Significant pitch responses (m) – Numerical			
Condition	Without heave plate	With heave plate	Reduction
<i>Intact</i>	2.14	2.13	0.44%
<i>Damaged</i>	2.00	1.99	0.44%

5.5.4 Maximum Offsets

All of the calculation's results of maximum offsets changes in each conditions are presented in Table 7 below.

Table 7: Maximum offsets of SPAR.

Maximum offsets of SPAR (m)			
Intact mooring system			
Method	Without heave plate	With heave plate	Reduction
Experiment	4.65	4.36	6.22%
Numeric	2.46	2.23	9.61%
Damaged mooring system			
Method	Without heave plate	With heave plate	Reduction
Experiment	5.02	3.43	31.69%
Numeric	2.37	2.11	11.22%

As stated before, the maximum offsets value was obtained by plotting the surge and sway responses in each condition during simulation together, then the furthest SPAR movement from the initial condition was calculated as the maximum offset. The plotted graphic example can be seen in Figure 14.

6 CONCLUSIONS

The findings of the study could be revealed as follows:

- Heave plate utilization does not constantly and significantly reduce surge responses. It is indicated by the biggest reduction of surge responses that occurs during simulation was only 0.86%, which happened in numerical analysis. There were some inconsistency happened in experimental method, which the surge responses was amplified by 6.15% in intact mooring system. This inconsistency potentially caused by the couple response between surge and pitch. On the

contrary, the heave plate utilization does reduce the heave responses of SPAR, consistently and significantly. It is indicated by the significant reduction that occurs in every condition of experimental and numerical study. The biggest reduction happened in damaged mooring system during experimental study, which was 33.29%. As for pitch, the heave plate utilization numerically does not affects the responses significantly, it is indicated by the reduction are only 0.44% in each condition. Yet, in the experimental study the pitch responses inconsistently changes. Where in intact mooring system it was reduced up to 17.65%, and while in damaged mooring system it was amplified by 4.56%. In some conditions, SPAR model satisfies the requirements of Matthieu instability to occurs, this phenomenon potentially become the cause of the inconsistency in pitch responses.

- Heave plate utilization does reduce SPAR maximum offsets consistently in every conditions. The biggest reduction happened in damaged mooring system of experimental analysis, which was 31.69%.
- The comparison between two methods has found that in surge responses, inconsistency happened while in intact mooring system. In the numerical analysis, the surge responses are slightly reduced, yet in experiment it was amplified. In heave responses the results of the two method shows a convenient agreement, where it has been reduced consistently and significantly in both methods. As for pitch, inconsistency happened while in damaged mooring system, where experimentally it was a bit amplified, but numerically it was slightly reduced.

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Ferry Cruise Security Prediction based on West Season Wind and Wave Characteristics in the Kalianget-Kangean Cruise Route, Madura

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Keywords: Wind Speed, Wave Height, Java Sea, West Monsoon, Kalianget-Kangean, Cruise Security.

Abstract: In the Indonesian archipelago the West Season is a wet period when bad weather usually takes place. Bad weather at sea is often marked by strong winds and high waves making it dangerous for inter-island cruise. One of the busy cruise routes is the Kalianget-Kangean cruise route which is part of the Java Sea where the musonal process, especially the West Season, is ideal. This study aims to analyze the characteristics of wind speed and wave height for 10 years (2008-2017) and weekly patterns for three months (December 2017-February 2018) representing the West Season in the waters of the Kalianget-Kangean ferry cruise route, as well as predicting security status the voyage. The data used was obtained from Surabaya Maritime BMKG which was then processed to obtain descriptive statistics and visualization of rose charts and maps of wind and wave patterns at the study site. The results show that the wind and wave characteristics of 2008-2017 formed two peaks that marked the West and East monsoons, as well as two valleys during the transition period between them. Wind speed and wave height of ten years are generally below the danger threshold, ie <10 knots and <2 m, respectively. Exceptions occur in the West Season, especially in January which is the peak of the West Season. When the wind and waves have strengthened with a steady blowing direction. The maximum wind speed reaches 29 knots and the maximum wave height reaches 6.7 m. Weekly wind and wave conditions from December 2017 to February 2018 are relatively safe, to be sailed. Extreme conditions that are dangerous for cruise occur on January 23-29 2018 when the maximum wind speed and maximum wave height reach 25 knots and 3.8 m, respectively. The Kalianget-Kangean cruise channel is relatively safe except in the time period of the Western Season in December, January, February when wind speeds exceed 21 knots and wave height of 2.5 m.

1 INTRODUCTION

Indonesia's geographical condition which consists of thousands of islands makes the sea as a connecting factor that cannot be ignored and therefore sea transportation, especially inter-island crossings, becomes important in the utilization of marine resources that drive the wheels of the archipelago's maritime development. Things that must be considered in crossing between islands or between countries are the weather-oceanography factors in the form of wind, current and wave conditions. The physical condition of the surface sea needs to be assessed to provide a safe and comfortable cruise database (Aji 2015), especially in busy regular channels. Especially when remembered due to the

increasingly intensive global climate change today, the weather in Indonesia has become relatively erratic. Rainy season with above normal rainfall; the emergence of strong winds that have the potential to become whirlwinds, accompanied by heavy rain and lightning; and high sea waves are the negative impact of increasingly volatile weather. One area that was negatively affected was Sumenep Regency. Since January 2008 there has been a rainy season in the district on the eastern tip of Madura Island. Strong winds of up to 30 knots were recorded in December 2017 to January-February 2018. Extreme weather caused cruise activities from Kalianget port to Kangean Island to become paralyzed in early January 2011. The delay in cruise was even one week due to high sea waves that could reach up to 6 m in Sumenep waters. As part of the Java Sea

Oceanic region, Madura waters are on an ideal path for ongoing monsoonal processes that affect weather conditions. Monsoonally, the west season becomes the period with the strongest winds during the season period. This research was conducted with the aim of: analyzing the characteristics of wind and waves for 10 years (2008-2017), analyzing weekly wind and wave patterns for 3 months (December 2017-February 2018) as a representation of the West Season, and predicting the status of cruise safety in the Kalianget channel- Bawean according to the characteristics of the wind and waves.

2 RESEARCH METHODS

The study was conducted in August 2018 until July 2019 with the study area located in the Kangean Islands, Madura, which became the Kalianget-Kangean cruise channel. Geographically, the

research sites that are part of the Java Sea are located at $6^{\circ} 82' 49''$ - $7^{\circ} 18' 30''$ "LS and $113^{\circ} 95' 17''$ - $115^{\circ} 30' 34''$ "BT (Figure 1).

Materials needed in this research are secondary oceanographic data obtained from the Meteorology Climatology and Geophysics Agency (BMKG) Maritime Surabaya which consists of wind, and waves as shown in Table 1. The data is then processed with various software to create a map of the research position, visual display of descriptive statistics to the making of vectorial maps of wind and waves (Table 2).

Wind and current data is downloaded from the Windwaves-05 software. After down-loading the data, it is then processed with MS Excel to obtain a visual display of descriptive statistics. Wind and wave rose diagrams are obtained using WRPlot®, while wind and wave map layouts are obtained using GIS.

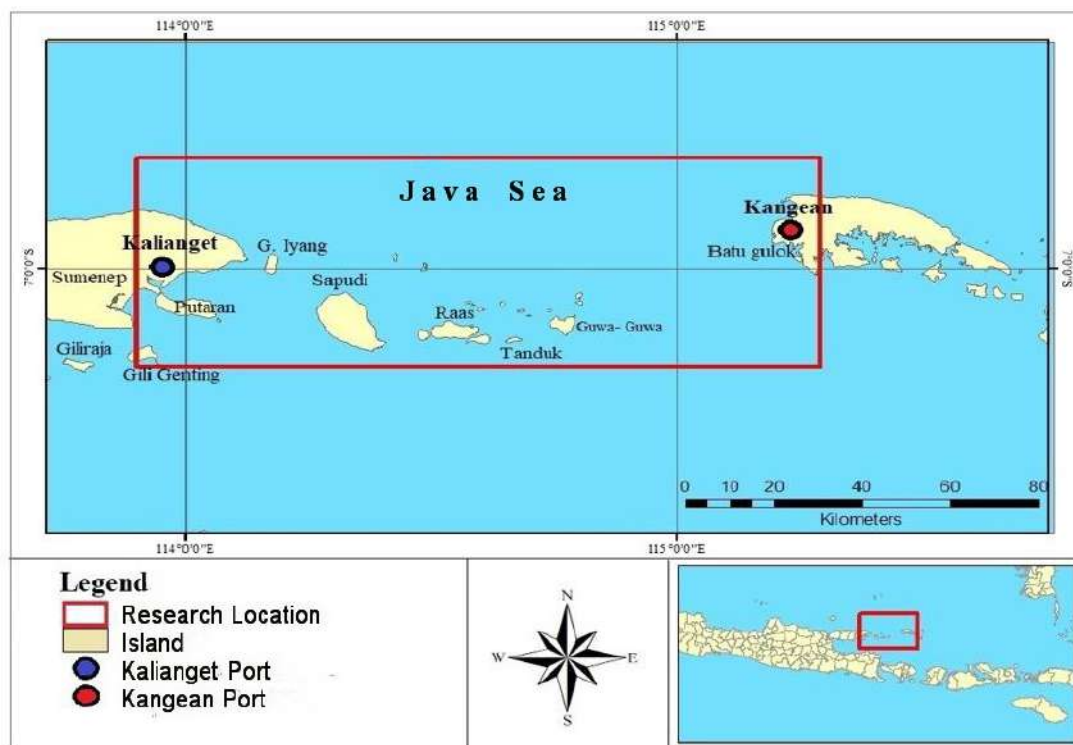


Figure 1: Research location in the Kalianget-Kangean cruise channel.

Table 1: Weather-oceanographic data of research materials.

Type	Period	Source
Wind	January 2008-December 2017	BMKG Maritime Surabaya
	December-February 2018	
Wave	January 2008-December 2017	BMKG Maritime Surabaya
	December-February 2018	

Table 2: Software and usability as a research tool.

Software	Function
Google Earth	Make research position
Matlab	Processing data, making charts and rose diagrams
GMT	Make outputs of wind and wave maps
Microsoft excel	Data processing and making charts of descriptive statistical
WRPlot	Processing of wind and wave roses
Windwave-05	Marine meteorological information

Table 3: Wind and wave risk matrix for cruise safety.

Level of Risks	Ferry Cruise	
Very Safe (VS)	Wind	<11 knot
	Wave	<1.25 m
Safe (S)	Wind	11-15 knot
	Wave	1.25-2.0 m
Less Dangerous (LD)	Wind	15-21 knot
	Wave	2.0-2.5 m
Dangerous (D)	Wind	>21 knot
	Wave	>2.5 m

The safety of ferry cruise on the Kalianget-Bawean channel is carried out by analyzing the weekly wind and wave data for December, January and February 2018 after being processed by Matlab. The data is then processed by MS Excel to create a visual display. Furthermore, weekly wind and wave data is processed using GMT to create a map layout, so it can be used as a prediction of the safety of the Kalianget-Kangean cruise.

Weekly wind and wave data for December, January and February 2018 were obtained from the Surabaya Maritime Maritime BMKG. The data is taken every 6 hours every day, ie at 00:00, 06:00, 12:00, 18:00 and back again at 00:00 the next day. It is assumed that the data collection period has represented observational data for 24 hours throughout the western season of December 2017 to February 2018. The data is then processed with Matlab and MS Excel to obtain descriptive visual statistics, then completed with a layout map of the results processed by GMT.

Wind speed and wave height results of the subsequent data analysis are compared with the wind and wave risk matrix for cruise safety (Table 3) issued by BMKG which for ferries has the following conditions.

3 RESULTS AND DISCUSSION

3.1 Wind Characteristics 2008-2017

In general, the results of the analysis of 10-year wind patterns (2008-2017), as shown in Figure 2, show the fluctuations of two peaks and two valleys of average wind speed. Peak fluctuations that indicate an increase in wind speed occur during the main season: west and east; while the two valleys take place during the transition period between them. The average seasonal wind speed shows slightly higher winds in the East Season (June-September) than the West Season (December-March).

The average wind speed <15 knots makes the cruise status throughout the season the cruise status is safe. However, when viewed from the maximum wind speed, vigilance must be prioritized, given that the West Season's wind speed is nearly 30 knots in January and is maintained until February, before experiencing a sharp speed decline to below 20 knots in March, and continues to decline in April-May. Starting the East Season, the wind speed increased again in May to reach the peak in August when the wind reached >21 knots, which means "dangerous" status for the ferry. After reaching the peak speed, the wind weakens again

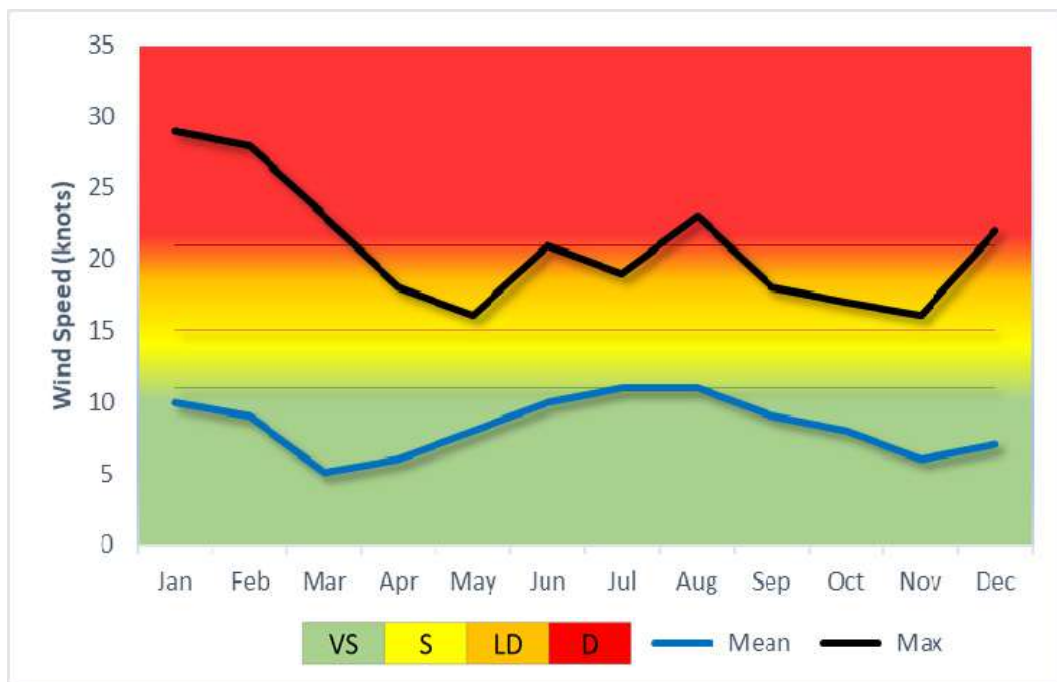


Figure 2: Monthly fluctuations in wind speeds during 2008-2017 confirm the high wind speeds in the West and East monsoons, as well as the weakening wind speeds during the transition periods.

to touch its nadir at the end of the Transition Season (November), before returning to rise in December which starts the Western Season.

Analysis of wind patterns during 2008-2017 in the West Season which is the season with the strongest wind speed throughout the season period shows the steady wind blowing from the West. In December, which began the West Season the west wind was seen to dominate nearly 40% of the frequency of events, followed by the northwest wind of around 20%. The rest with a percentage of <10% the wind moves from all directions. During December the dominance of the wind blows at speeds of 4-8 knots by 38%, followed by winds of 2-4 knots (33.4%). The maximum wind speed was recorded at 20-24 knots despite a very small frequency (0.5%) (Figure 3: Dec). A month later the West Season reaches its peak. This is indicated by the strengthening of the stability of the western wind, as shown in Figure 3: Jan, which now reaches ~65% with a maximum speed reaching 28-32 knots (0.4%). During January the dominant wind blows at speeds of 8-12 knots (23.4%) and 5-8 knots (22.3%). After reaching the peak, the West Season again weakened in February. In that month, although the dominance of the wind still blows from the west, its dominance is now <50% with a top speed of 24-26 knots (1.7%) and a dominance of 4-8 knots (29.4%) (Figure 3: Feb).

3.2 Wave Characteristics of 2008-2017

With a pattern similar to the wind pattern, the wave pattern during 2008-2017 shows two peaks that mark the wave height in the West Season and East Season; and two valleys showing low waves in the transition periods at the beginning and end of the year (Figure 4). The average wave height record for the whole year is generally below 2 m (except in July which reaches 2 m) so that from a perspective of average wave height, cruise along the Kalianget-Kangean channel is safe. However, the maximum record height of waves throughout the year confirms the very high waves in the West Season which peaked in January when the highest waves reached 6.7 m. In February the wave height was slightly weakened with the highest achievement of 5 m. Before reaching the peak in January, the maximum recording height of waves in December only reached 3.90 m. Thus throughout the West Season the maximum wave attainment becomes a prohibition for the delivery of the Kalianget-Kangean route since the waves can reach >2.5 m.

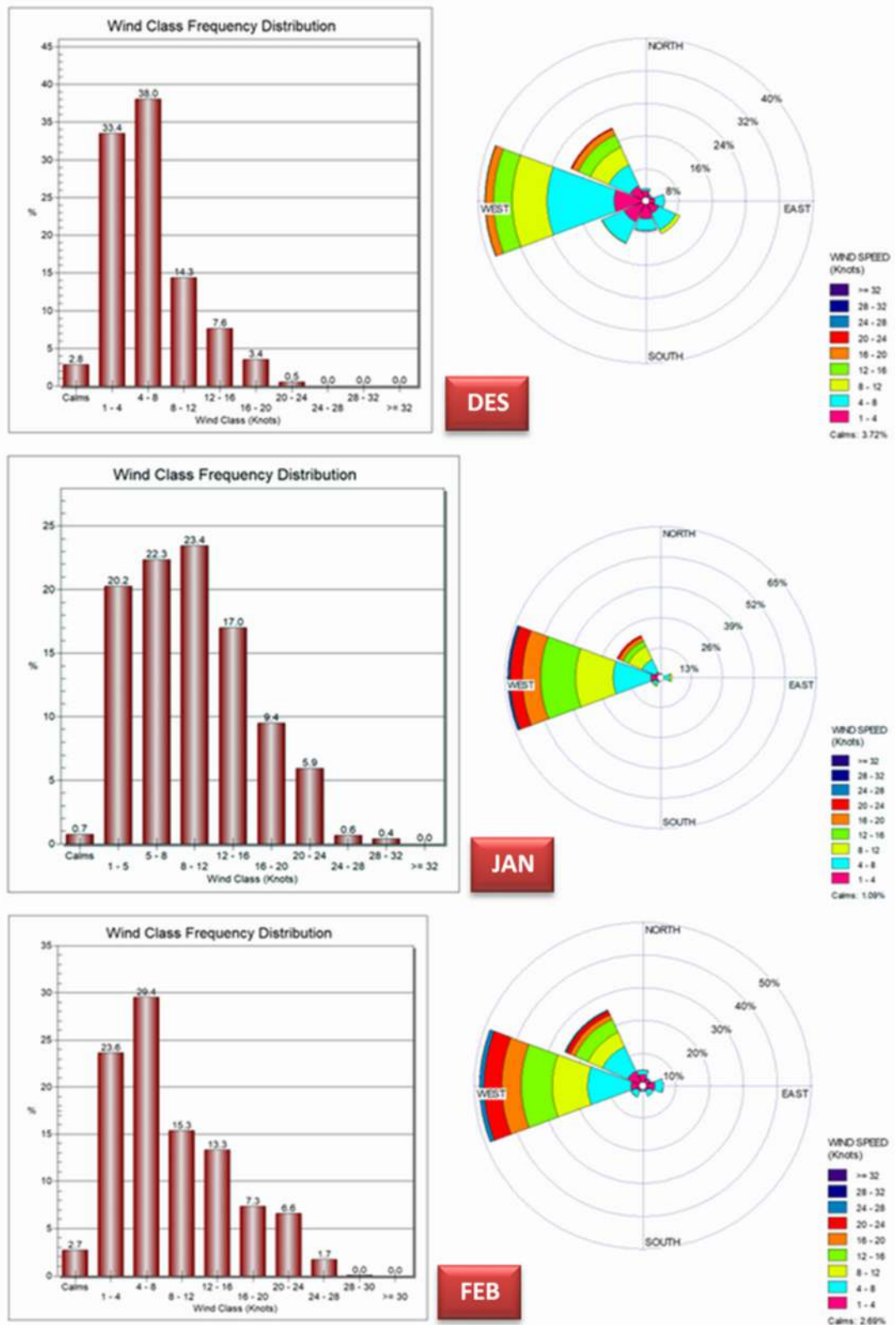


Figure 3: Frequency distribution and wind speed patterns during December-February representing the West Season 2008-2017 which shows the steady wind from the west.

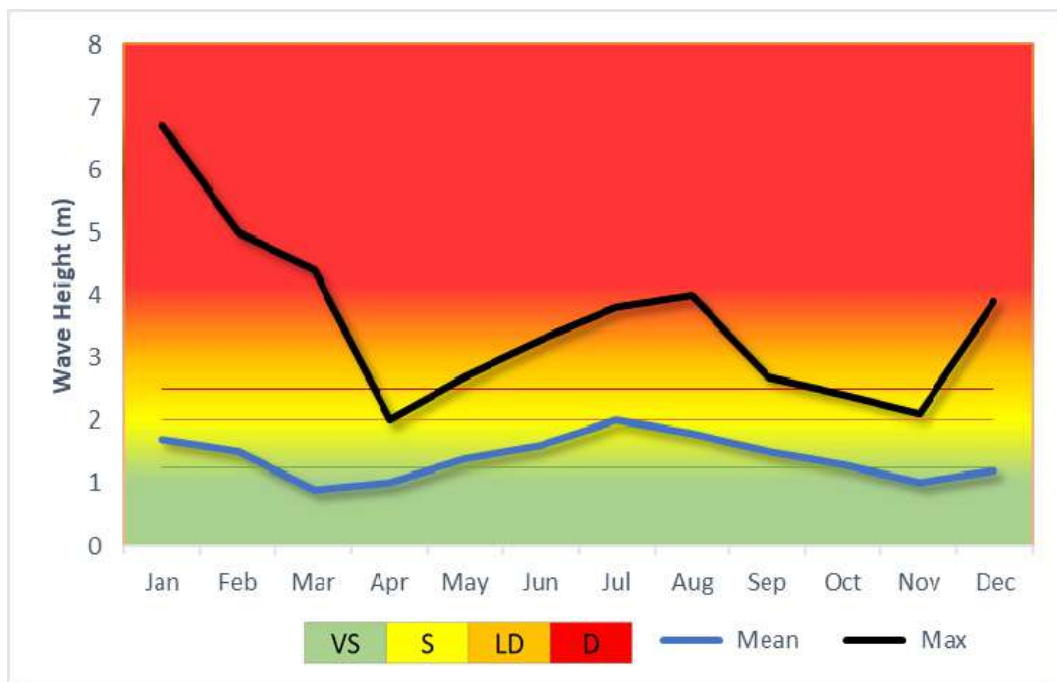


Figure 4: Monthly fluctuation in wave height during 2007-2018 confirms the increased wave height in the West and East Seasons interspersed with low waves in the early and late year transition seasons.

The transition season in the beginning of the year in March is marked by the lower wave height until it reaches nadir 2.0 m in April. Entering the Eastern Season, wave height during May again increased and reached its peak in August when the maximum wave height reached 4 m. Then the waves got lower until they reached 2.1 m in November.

More than 55% of the waves that occurred in December were 1-2 m high, with the highest waves of 3-4 m 2.7% (Figure 5). Waves at the end of this year predominantly move from the south (<50%) and northwest (>30%) (Figure 5: Dec). A month later, although waves as high as 1-2 m still dominate, the frequency is slightly reduced (47.5%). The decrease in the dominant wave height is followed by an increase in the highest wave attainment of 5-6 m (0.4%) to 6-7 m (0.3%). The direction of wave arrival is now recorded to be from the northwest (~45%). The south wave although it is still recorded a lot, but the frequency is now only <25%, almost the same as the frequency of western waves. Such conditions mark the maturity of the wave of the West Season which reaches its peak in January (Figure 5: Jan). After reaching the peak, as shown in Figure 5: Feb, wave height starts to look weaker even though data records still confirm the dominance of northwest waves (<40%). The maximum height of the waves has decreased to 4-5

m (3.9%); so is the wave region of 1-2 m which now reaches 40.7%.

Wind and wave patterns in the Kalianget-Kangean waters show cursory mismatches. The wind patterns in December-February are relatively strong, blowing from the west (~ 40-65%) and the northwest (~ 20-30%) which strongly represent the monsoon season. Meanwhile the waves are spreading in more varied directions. In December there was a dominance of the direction of the waves from the south (~ 45%) and northwest (~ 30%). This south wave is a wave with a small height (<2 m), while a wave from the northwest has a height of up to 4 m. Two months later the wave data recording was dominated by northwest (~ 34-40%) and west (~ 20-25%) which better represented the seasonality. However small waves are also found to move from the south (~ 25-30%).

The distortion of the direction of the incident wave, as stated by WMO (1998), can be caused by the shoaling effect of bathymetry. Such bathymetric variations are found in the coastal waters of small islands which are widely spread in the study location. The result is the discovery of southward dominance, but only at the beginning of the formation of the west season in December, which is characterized by relatively small waves.

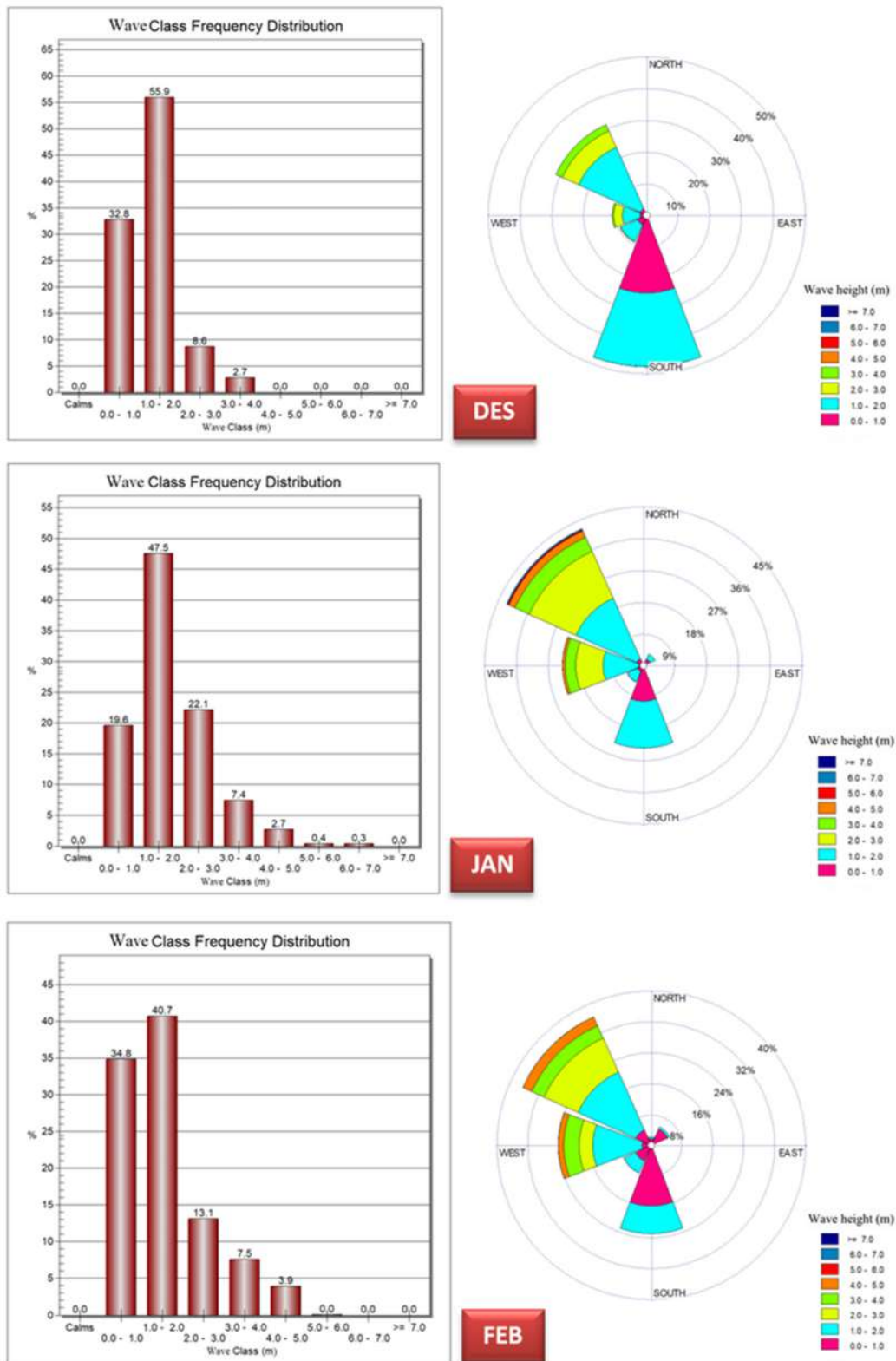


Figure 5: Frequency distribution and wave height patterns during December-February representing the West Season for the period 2008-2017. In general the dominance of waves moves from the northwest.

Along with the maturation of the season in January-February, the compatibility of the wind-wave pattern is relatively maintained with the dominance of the direction coming in the northwest-northwest scope. Stronger winds blowing from the west dominantly does not produce the direction of coming waves also moves from the west, but slightly veers to the northwest. This condition is in accordance with the oceanographic condition of the study site which forms a "gap" for wave motion on the southeast side.

3.3 Wind Characteristics from December 2017 to January-February 2018

Analysis of weekly wind patterns at the peak of the West Season during December 2017 to February 26, 2018 confirms the occurrence of two peaks and two valleys of wind speed as shown in Figure 6. The average velocity fluctuates with a pattern similar to the maximum wind speed. The average speed reaches the peak of its first fluctuation on December 19-25, 13 knots and the maximum reach is 22 knots. With speeds exceeding 21 knots, this time period becomes dangerous for cruise the Kalianget-Kangean route.

After reaching the first peak of speed fluctuations, the wind blows more slowly until the average and maximum speed touches nadir of 4 m and 7 m respectively on January 2-8 2018. Such conditions make the Kalianget-Kangean waters

relatively safe for cruise. Furthermore, the wind strengthened again and reached a maximum speed of up to 25 knots so that it was dangerous to sail on January 23-29 2018. The maximum wind speed above 20 knots was recorded until about two weeks later. After that the wind speed continues to weaken to 11 knots so that the waters are safe for cruise activities.

Illustration of various wind conditions and the risk level of the Kalianget-Kangean cruise route is carried out by taking an example of wind patterns that occur on January 4 at 12.00, January 15 at 18.00, January 17 at 17.00 and January 26, 2018 at 03.00 each shown by Paga Figure 7A to Figure 7D. On January 4, 2018 at 12.00 with a maximum wind speed of 6-8 knots, Kalianget waters and surrounding areas to the waters of Kangean Island and surrounding areas are in a very safe condition to be sailed (Figure 7A). On January 15 at 18:00, although the maximum wind speed has increased to reach 10-15 knots, based on the risk level of the Kalianget-Kangean cruise waters it is still very safe to navigate (Figure 7B). Two days later at 17:00 as shown in Figure 7C, the Kalianget-Kangean cruise route is at a rather dangerous level to be sailed given the maximum wind 15-20 knots in the waters of Sapudi Island, Raas Island and Tanduk Island to the waters around Kangean. On January 26, 2018 at 03.00, all cruise routes from Kalianget waters to Kangean waters were classified as dangerous to. navigate given the maximum wind speed at that time reached 20-25 knots (Figure 7D).

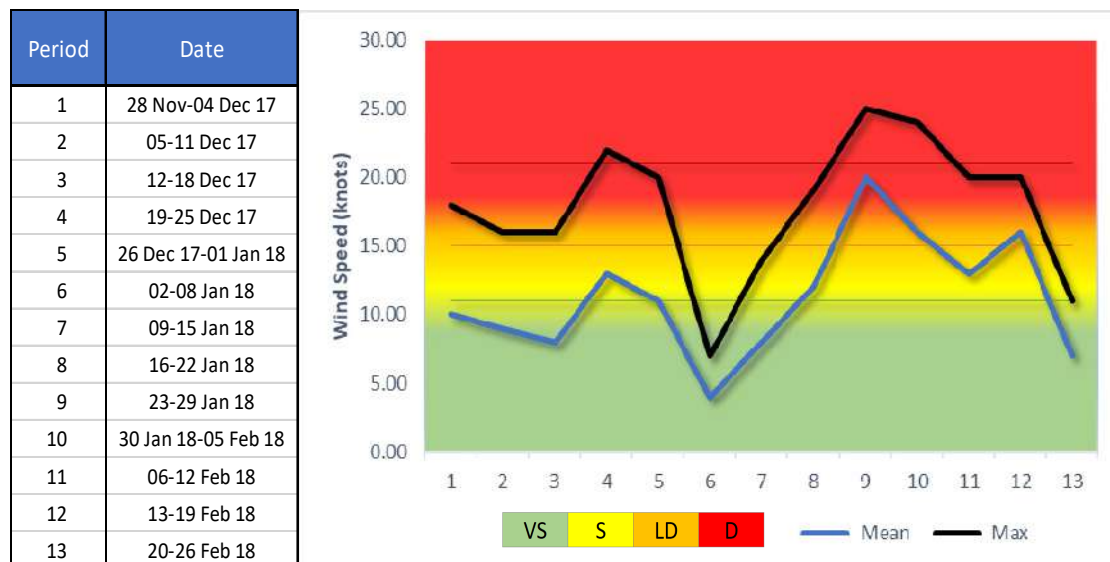


Figure 6: Weekly fluctuations in wind speed during November 2017-February 2018 which represent the peak of the West Season.

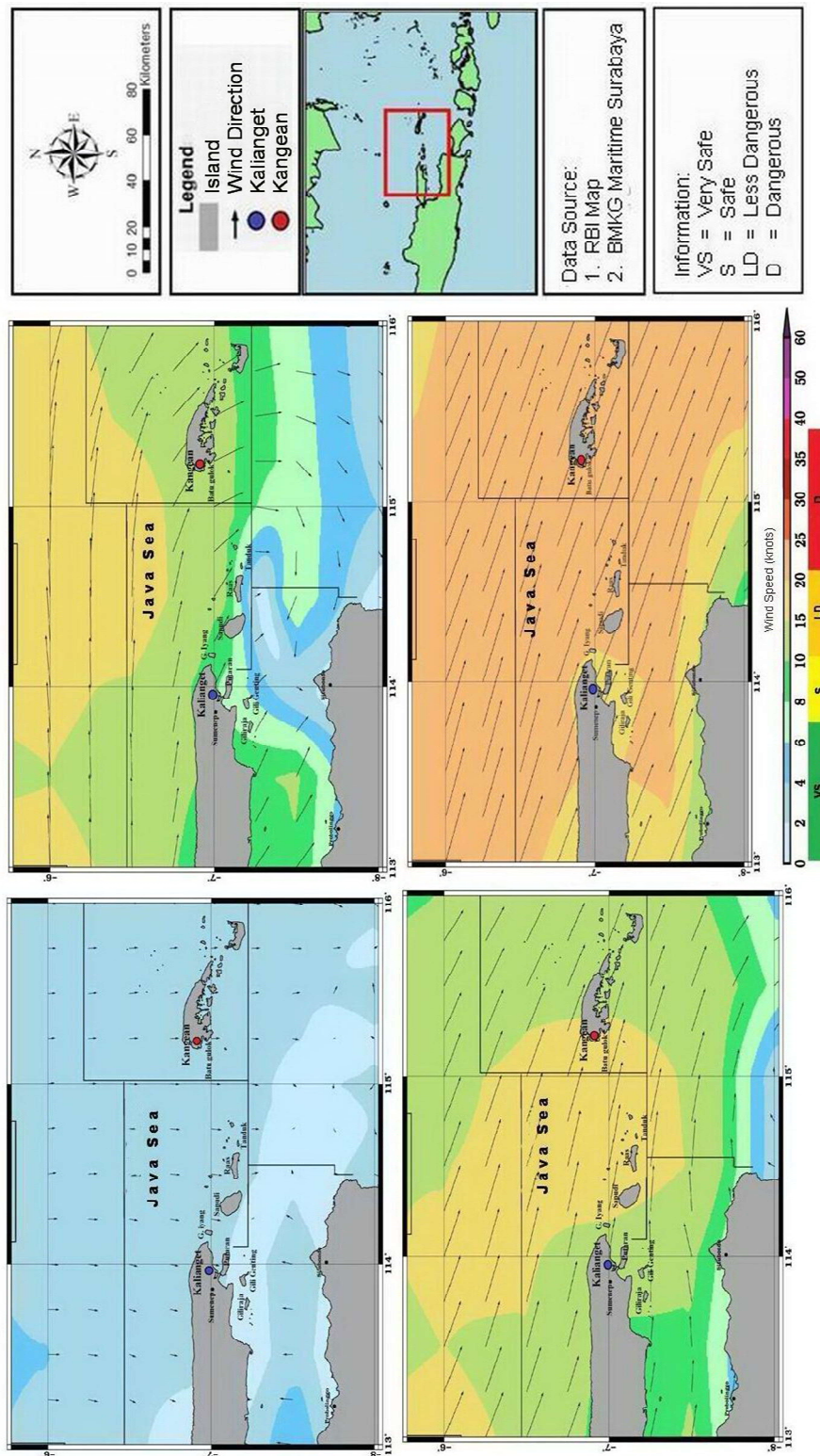


Figure 7: Maximum wind speed patterns in the waters of the Kalianget-Kangean cruise route on: 4 January 2018 at 12.00 (A), 15 January 2018 at 18.00 (B), 17 January 2018 at 17.00 (C), and 26 January 2018 at 03.00 (D).

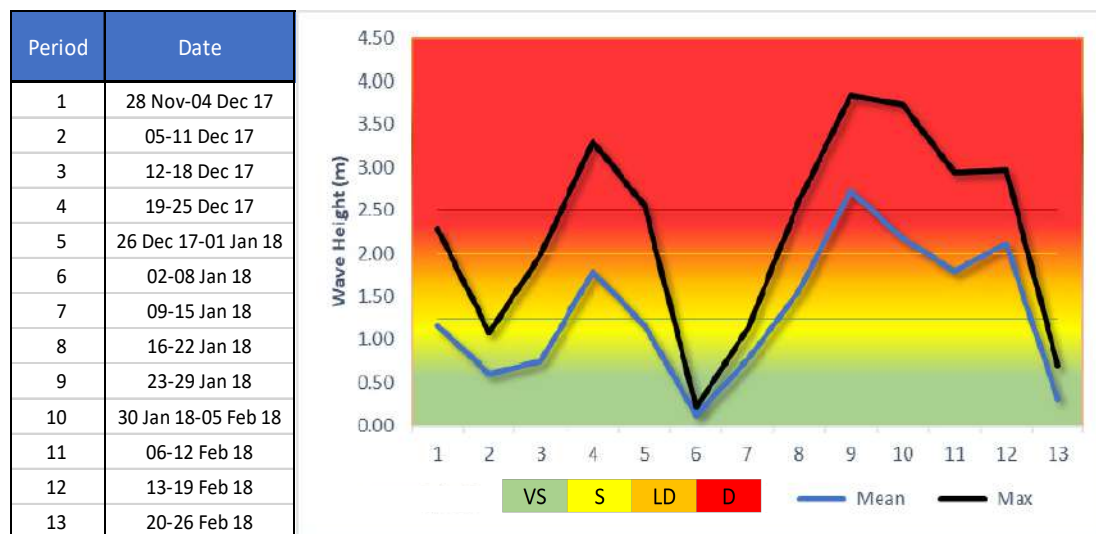


Figure 8: Weekly wave height fluctuations during November 2017-February 2018 which represent the peak of the West Season.

3.4 Wave Characteristics from December 2017 to January-February 2018

Like wind speed, weekly wave height fluctuations during the 2018 West Season, as shown in Figure 8, also shows two peaks and two valley wave heights. After reaching nadir on December 6-11 when the average and maximum wave heights reached 0.61 m and 1.09 m, the wave height increased to a maximum height exceeding 2.5 m which was dangerous for cruise. The maximum wave height of 3.3 m was recorded on December 19-25, 2017. During this period the average wave height was still in a safe condition for cruise with an achievement of 1.8 m (<2 m).

The next wave height decreases until it reaches the average and the maximum height is below 1.5 m on January 2-8 2018. Furthermore, the waves are getting higher until the average and maximum height respectively reach 2.73 m and 3.84 m on January 23-29 2018 which is dangerous for cruise. The maximum wave height above 2.50 m which was dangerous for the voyage was still maintained until the period of 13-19 February 2018. After that period the waves again weakened to an average height and the maximum reached 0.31 m and 0.70 m respectively on February 20-26 2018.

Illustration of risk levels with varying wave conditions on the Kalianget-Kangean route voyage is carried out by taking an example of wave patterns that occur on 2 January 2018 at 12.00, 15 January 2018 at 21.00, 16 January 2018 at 21.00 and 25

January 2018 at 21.00 which are respectively shown Figure 9A through Figure 9D.

On January 2, 2018 at 12.00 the waters of Kalianget and the Kangean archipelago and its surroundings were in a very safe condition to be navigated with a wave height of ~ 0.5 m (Figure 9A). On January 15, 2018 at 21:00 a recording of wave height shows safe conditions for cruise from Kalianget to Sapudi Island with wave height <2 m. After the waters of Sapudi Island to Kangean waters the wave height shows an increase to reach a rather dangerous level with a height of 2-2.5 m, even in the northern waters of Kangean Island the wave height reaches > 2.5 m and is dangerous for cruise (Figure 9B). The recorded waves weakened at 21.00 a day later as shown in Figure 9C. From Kalianget waters, wave conditions are very safe to navigate to the waters of Raas Island. Although in the waters around Tanduk Island it is still safe to sail, but further east approaching Kangean Island the voyage becomes a bit dangerous. Around the port of Kangean the wave drops to <2 m so it is classified as safe for cruise. On January 25, 2018 at 21:00. Wave conditions from Kalianget harbor to Sapudi Island waters are classified as very safe to navigate with wave height <2 m (Figure 9D). Towards East Timor, approaching the waters of Raas Island, the wave height reaches 2.5 m to be rather dangerous to navigate, further east in the sea waters before the Kangean waters, wave height > 2.5 m and can even reach 5 m so that it is dangerous to navigate. Towards the port of Kangean the waves weakened again until they reached <2 m.

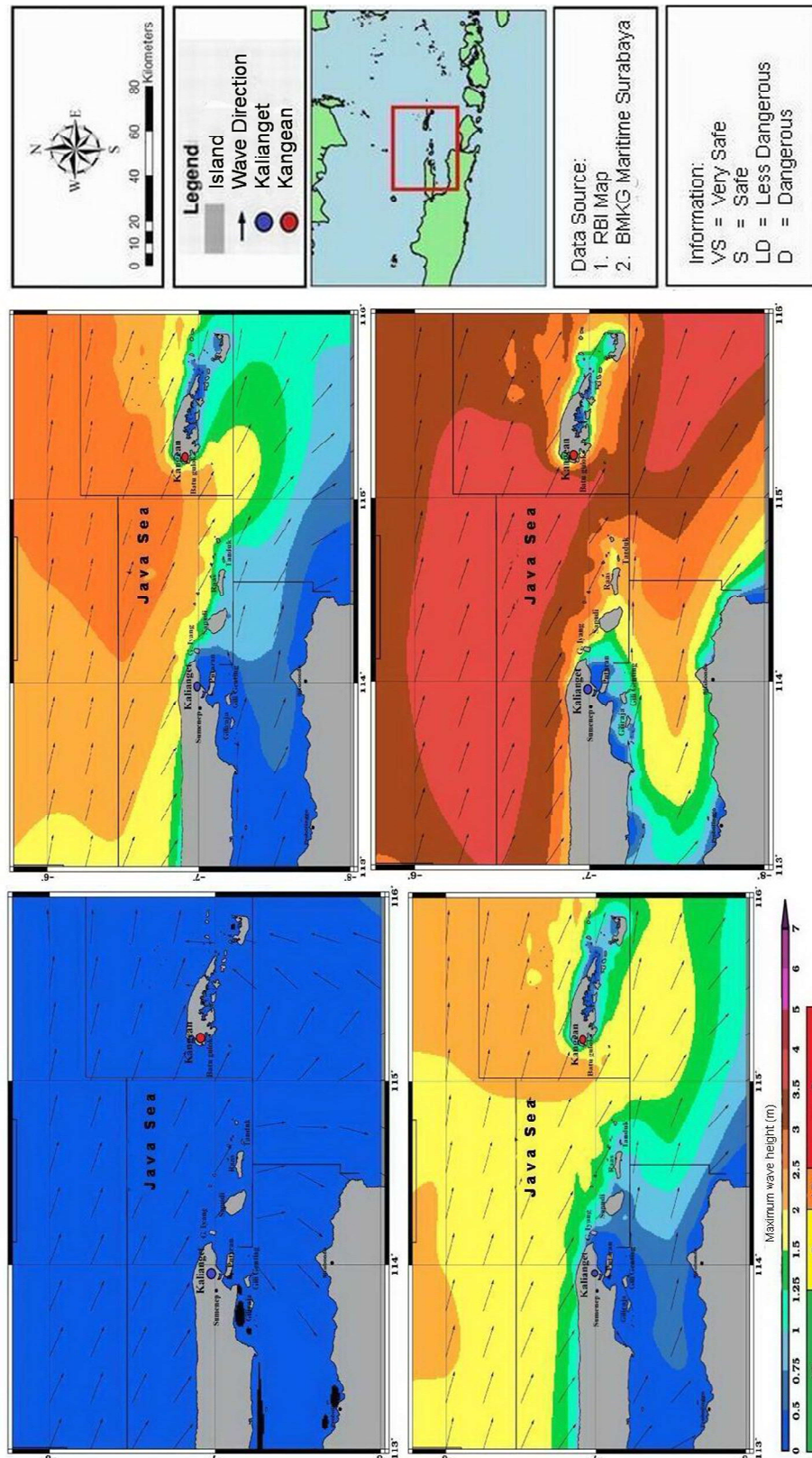


Figure 9: The maximum wave height pattern in the Kalianget-Kangean cruise route on: 2 January 2018 at 12.00 (A), 15 January 2018 at 21.00 (B), 16 January 2018 at 21.00 (C), and 25 January 2018 at 21.00 (D).

Wind and wave conditions in the waters between Kalianget and Kangean are steady moving from northwest-west. This condition reflects the strong influence of the (Western) season, in the study sites that are part of the Java Sea. The West monsoon is characterized not only by the strong blowing in the northwest-west direction, but also by strong winds and high waves compared to the period of the monsoon season when the wind is relatively weak and becomes unstable. This result corroborates the statement of Manu *et al.* (2002) about the important role of wind in the formation of surface layers, including currents and waves. Strong and steady winds will strengthen the waves and increase their height. The Northeast monsoon (West Season in Indonesia) marks the wet period that brought wind from the Asian Continent to the Australian Continent (Tomascik *et al.* 1997 and Nontji 1993). The results also showed a tendency to increase wave height in the waters between Tanduk Island and Kangean Island. This is made possible by the effect of the bathymetry depth and narrowing on the north-south side of the study waters. The silting effects in the waters around Kalianget and Kangean seaports are debilitating which makes wave conditions relatively safe for nearby cruise.

4 CONCLUSIONS

The character of the wind and waves of the 2008-2017 period generally forms two peaks that mark the main season (West Season and East Season) and two valleys that mark the transition periods. In general wind speed and wave height are below the danger threshold (<10 knots and <2 m). Exceptions occur in the West Season, especially in January which is the peak of the West Season when winds and waves have strengthened with a steady blow. The maximum wind speed reaches 29 knots and the maximum wave height reaches 6.7 m.

Weekly wind and wave conditions during December 2017 to February 2018 are relatively safe, to be sailed. Extreme conditions occurred on January 23-29 2018 when the maximum wind speed and maximum wave height reached 25 knots and 3.8 m which were dangerous for cruise.

The Kalianget-Kangean cruise channel is relatively safe except in the time span of the West Season during December, January, February when the wind speed exceeds 21 knots and the wave height is 2.5 m.

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Numerical Study of Bilge Keel Length Variations of Floating Breakwater to Optimize Transmission Coefficient

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Abstract: Waves and winds that move dynamically often cause damage on the coast so a protective beach building is needed. Breakwater is a coastal protection structure to destroy the incoming wave energy before reaching the coast. One type of breakwater is floating breakwater, this structure has an advantage compared to the fixed breakwater. Research on floating breakwater has been developed with the main goal of being the most efficient structure and can absorb waves well. In this research, floating breakwater simulation modeling with variations in bilge keel length was conducted. Validation is done by comparing the results of (GH Dong, 2008) with numerical results of the breakwater pontoon type. There are four variations of bilge keel length, namely 0 m, 0.6 m, 0.9 m, and 1.2 m. So, four variations can be concluded that the longer the bilge keel, the better absorb the waves.

1 INTRODUCTION

Coastal areas need to be considered for safety against erosion and wave surges. One of the structures to protect the coast from waves is a breakwater. There are two types of breakwater, bottom-founded breakwater, and floating breakwater. Floating breakwater has the advantage of bottom-founded breakwater: (1) The time of construction is shorter because it has been done in fabrication, (2) floating breakwater can be easily moved, reassembled with different layouts, and can be moved to different locations, (3) floating breakwater is suitable for muddy soil, (4) floating breakwater is more environmentally friendly because it does not cause pollution and sedimentation.

Floating breakwater (FB) research began around one century ago, many studies and model tests were carried out to develop floating breakwater. As technology develops, research on floating breakwater is rapidly increasing. There have been many studies exploring floating breakwater where exploration will continue to be developed to obtain the most optimal results.

Experimentally and numerically of floating breakwaters have been studied. According to G.H Dong (2008), floating breakwater can be used

effectively in coastal areas with relatively mild wave conditions. His experiment was to find a simple and relatively inexpensive type, by studying 3 types of structures including the shape of a single box, double box, and board net. (Wang and Sun, 2010) conducted a study of porous breakwater where the structure was fabricated with large numbers of diamond-shapes blocks arranged to reduce transmitted wave height and the mooring force. (Drimer et al., 1992) conducted a study of the simplification of a floating breakwater design where width and wavelength are the greater than the gap between the breakwater position and sea bed. (William and Abul-Azm, 1997) conducted a study of the hydrodynamic characteristics of a dual pontoon floating breakwater consisting of a rectangular floating cylinder connected by a rigid deck. (Liang et al., 2004) conducted research on the reflection and wave transmission of floating breakwater spar bouys as a well mooring forces.

This paper presents numerical simulation to analysis transmission coefficients using computational fluid dynamics (CFD) method. The basic geometry is pontoon floating breakwater that modified in several length variations of bilge keel on the bottom of structure. Data parameters and numerical model verifications based on the result of experiments that conducted by (G.H. Dong., 2008).

2 NUMERICAL SIMULATIONS

2.1 Dimension of Floating Breakwater

Floating breakwater dimension is obtained from international journal data entitled “Experiments on wave transmission coefficients of floating breakwaters” by G.H Dong (2008). See figure 1 and figure 2 for cross section of the prototype of floating breakwater by GH Dong. The data will be used to compare the G.H Dong experimental test with a numerical test. The data that will be used in numerical test modelling are as follows, see table 1.

2.2 Data Parameters

Research conducted by (G H Dong, 2008) uses environmental data (wave height and wave period data). The test uses a model scale, so the environmental data used data that has been scaled too. The data are scaled by assuming Froude scaling to be valid. For detail, see table 2.

Table 1: Pontoon Dimensions.

No	Dimension	Prototype [m]	Scale	Model [m]
1	Length (L)	20	1:40	0.50
2	Width (B)	6	1:40	0.15
3	Height (H)	4.8	1:40	0.12

Table 2: The Wave Parameter Data (Froude Scaling).

No.	Dimension	Prototype	Scale	Model
1.	Wave Height	2.5 m	1:40	0.0625m
2	Wave Period	6 s	$1:\sqrt{40}$	0.949 s
		7 s	$1:\sqrt{40}$	1.107 s
		8 s	$1:\sqrt{40}$	1.265 s
		9 s	$1:\sqrt{40}$	1.423 s
		10 s	$1:\sqrt{40}$	1.581 s

2.3 The Wave Flume

Wave flume that is modeled is in the State Key Laboratory of Coastal and Offshore Engineering, Dalian University of Technology, China. The flume is 20 m long, 3 m wide, 1 m high, and 0.5 m water depth. The flume is filled water with density of 1000 kg/m³. The flume is equipped with a hydraulically driven, piston-type irregular wave generator at one end and a wave absorber at the other. The floating breakwater model is located at 20 meters from the wave maker. The waves come from the left toward the right across the structure. The number of wave probes used is two, probe 1 is located 3 meters in front of the structure and probe 2 is located at 3 meters behind the structure. See, Fig. 3 for illustration.

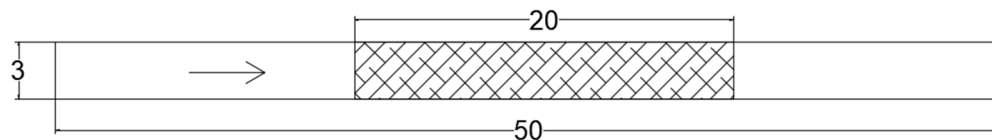


Figure 1: Basic Cross-section Pontoon FB (Top View).

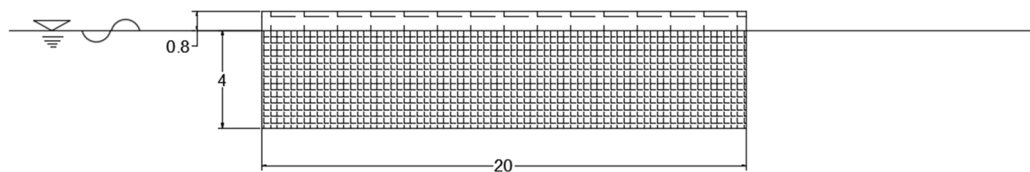


Figure 2: Basic Cross-section Pontoon FB (Side View).

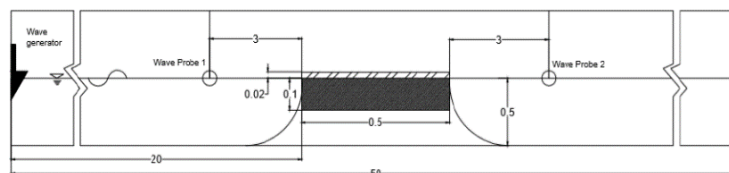


Figure 3: Sketch of Wave Flume (Side View).

2.4 Developed Floating Breakwater

Developed floating breakwater is intended to determine shapes of floating breakwater which is effective in reducing wave energy. Developed shape of floating breakwater refers to the single-box floating breakwater by GH Dong. Therefore, this research will be changed variations of bilge keel length. The developed floating breakwater can be seen on figure 4.

2.5 Wave Measurements and Analysis

The objective of this study was to obtain the transmission coefficient (C_T), the ratio of transmitted wave height (H_T) to incident wave height (H_i). See equation 1 below:

$$C_T = H_T / H_i \quad (1)$$

Measurement of transmitted and incident wave heights were represented by surface elevation using wave gauges.

2.6 Mooring System

The structure is connected to the sea bed by mooring chains. There are six mooring chains on each side of floating breakwater. Each mooring chain is 0.1 m in diameter, 60 m in length, and 230 kg/m in unit mass.

2.7 Boundary Conditions

The purpose of boundary conditions is to determine model conditions that represented experiment conditions. See figure 5. Boundary conditions used in the model are as follow:

- (1) Free surface (Wave): A wave boundary condition was defined at left (X Min). A surface wave entered the computational domain and propagated in the direction normal to the boundary. The wave was set 2nd stokes as wave generator represent the physical wave conditions at the boundary.

- (2) Outflow: The outflow boundary condition was defined at right (X Max). It allowed users to numerically investigate the effects of wave interactions with structures. The capability permitted a reduction in the extent of the computing mesh needed for accurate computations. A wave-absorbing layer used to reduce reflection of periodic wave at an open boundary.
- (3) Symmetry: The symmetry condition was defined at front (Y Min), behind (Y Max), below (Z Min), and up (Z Max). No-slip conditions were imposed using the wall shear-stress options described in the Prandtl Mixing Length model. A symmetry condition can be specified as free-slip conditions that have a non-zero wall shear-stress.

2.8 Meshing

Mesh block is used to determine the area that modelled. The smaller mesh will be more detailed, but the output files will be larger and simulations run longer. Floating breakwater modelling used two mesh block, (1) Block A with a meshing size of 0.06 m at total length (X-axis) is 20 m, total width (Y-axis) is 3 m, and total height (Z-axis) is 1 m, (2) Block B with a meshing size of 0.01 at length (X-axis) is 19.95-20.19, width (Y-axis) is 1.2-1.8, and height (Z-axis) is 0.4-0.6. See figure 6 for the illustration.

3 RESULTS AND DISCUSSION

3.1 Validation

Validation is done by comparing the experiment result with the modelling result. The comparison uses a transmission coefficient which is illustrated in one graph (H/L Vs K_t). Validated if the modeling results show the similarity of trend curve with experimental results. If not, redesign the geometry model and do the modeling again until the results really match.

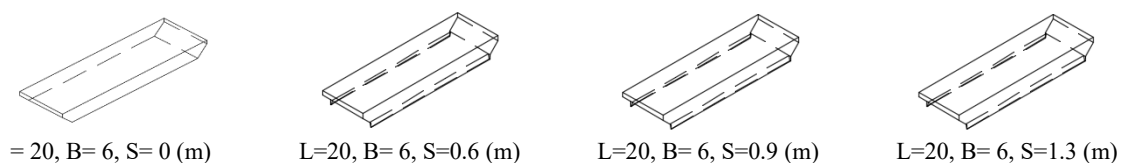


Figure 4: Developed of Floating Breakwater (S = Bilge Keel Length) in meter.

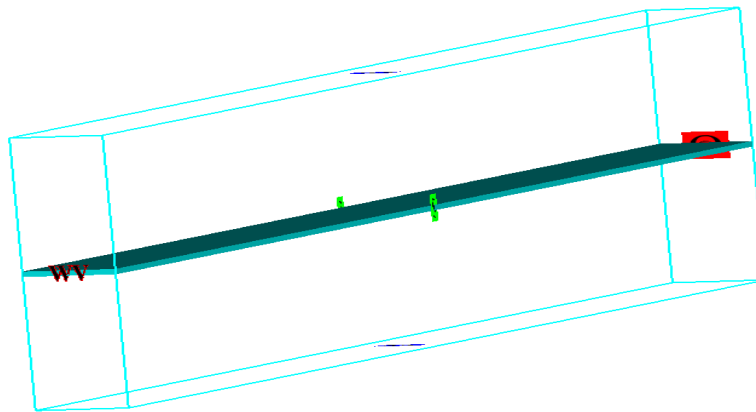


Figure 5: Boundary Conditions Model.

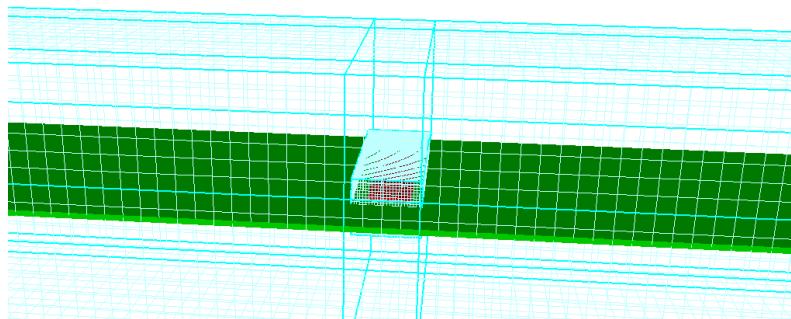


Figure 6: Mesh-size Model.

Here, the results of modeling validation with the experiment are shown on tabel 3. More detail see figure 7.

Table 3: The Validation of Transmission Coefficient.

H	T	H/L	K _T		Error
			Numeric	Experiment	
0.949	0.0452		0.325	0.356	7%
1.107	0.0345		0.661	0.654	2%
0.0625	1.265	0.0282	0.908	0.844	8%
1.425	0.0238		0.960	0.873	10%
1.581	0.0206		0.891	0.815	10%

The graph on figure 7 explain that the transmission coefficient in numerical modeling and experiment test has almost the same similarity and valid based on Mean Absolute Percentage Error (MAPE) theory. However, there are still difference in value of transmission coefficient which may be caused by differences in recording wave gauges or meshing size.

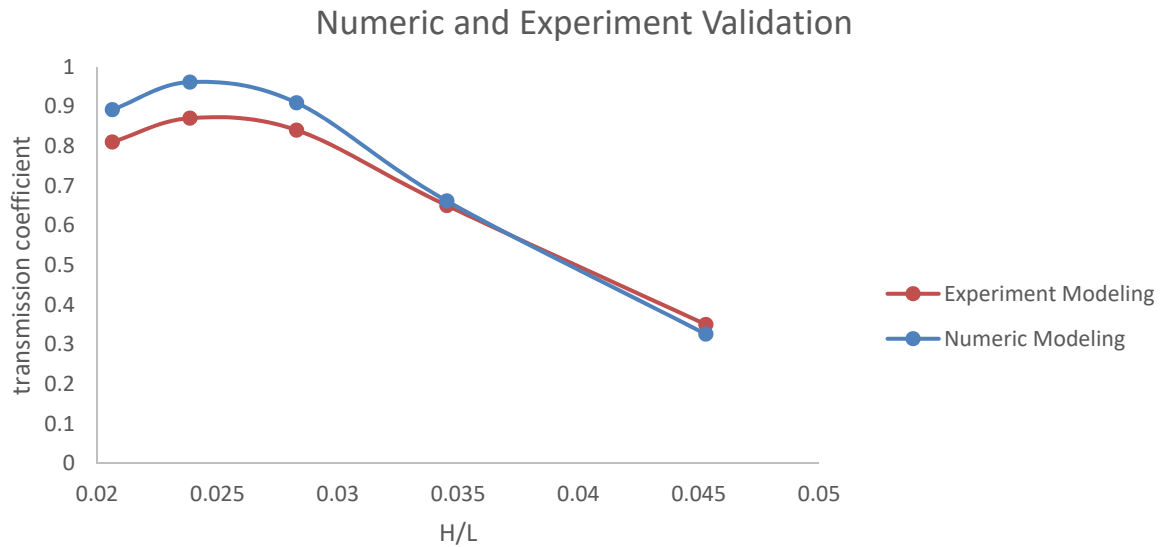
3.2 Transmission Coefficient Result

3.2.1 Model 1 (Without Bilge Keel)

Data from modeling of floating breakwater in Model 1 (without bilge keel) were obtained from recording of waves elevation on wave gauges in the form of time series data. The data is then processed using statistics Wave Analysis (WAVAN). The following is transmission coefficient result in Model 1, see table 4.

3.2.2 Model 2 (0.6 m Length of Bilge Keel)

Data from modeling of floating breakwater in Model 2 (0.6 m length of bilge keel) were obtained from recording of waves elevation on wave gauges in the from of time series data. The data is then processed using statistics Wave Analysis (WAVAN). The following is transmission coefficient result in Model 2, see tabel 5.

Figure 7: Validation graphic (K_T Vs H/L).

3.2.3 Model 3 (0.9 m Length of Bilge Keel)

Data from modeling of floating breakwater in Model 3 (0.9 m length of bilge keel) were obtained from recording of waves elevation on wave gauges in the from of time series data. The data is then processed using statistics Wave Analysis (WAVAN). The following is transmission coefficient result in Model 3, see tabel 6.

3.2.4 Model 4 (1.3 m Length of Bilge Keel)

Data from modeling of floating breakwater in Model 4 (1.3 m length of bilge keel) were obtained from recording of waves elevation on wave gauges in the from of time series data. The data is then processed using statistics Wave Analysis (WAVAN). The following is transmission coefficient result in Model 4, see tabel 7.

3.3 Comparison Model Result

In order to know the results that are easily understood, numerical modellings of Model 1 up to Model 4 are compared in one graph as shown in figure 8. Figure 8 shows the correlation of transmission coefficient (K_T) with wave steepness (H/gT^2) at depth of 0.5 meters and variations in wave height (H) and wave period (T). Transmission coefficient in model 1 ranges from 0.233-0.390, model 2 ranges between 0.221-0.377, model 3 ranges between 0.210-0.370, and model 4 ranges between 0.205-0.365. The higher the wave height the smaller the wave period is the steeper the wave steepness. Figure 8 shows that each model has

a transmission coefficient that continues to decrease with increasing wave steepness.

Table 4: Graphic of K_T Vs H/gT^2 on Model 1.

No	H_i	T_i	H/gT^2	K_T
1	0.0240	0.800	0.004	0.390
2	0.0380	0.684	0.009	0.298
3	0.0550	0.658	0.012	0.281
4	0.0550	0.575	0.016	0.233

Table 5: Graphic of K_T Vs H/gT^2 on Model 2.

No	H_i	T_i	H/gT^2	K_T
1	0.0240	0.800	0.004	0.377
2	0.0380	0.684	0.009	0.287
3	0.0550	0.658	0.012	0.270
4	0.0550	0.575	0.016	0.221

Table 6: Graphic of K_T Vs H/gT^2 on Model 3.

No	H_i	T_i	H/gT^2	K_T
1	0.0240	0.800	0.004	0.370
2	0.0380	0.684	0.009	0.278
3	0.0550	0.658	0.012	0.260
4	0.0550	0.575	0.016	0.210

Table 7: Graphic of K_T Vs H/gT^2 on Model 4.

No	H_i	T_i	H/gT^2	K_T
1	0.0240	0.800	0.004	0.365
2	0.0380	0.684	0.009	0.265
3	0.0550	0.658	0.012	0.250
4	0.0550	0.575	0.016	0.205

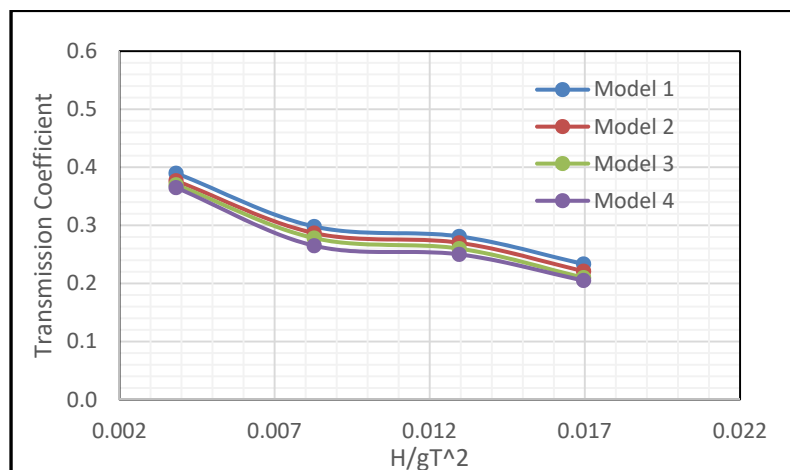


Figure 8: Comparison of Four Models Results (K_T Vs H/gT^2).

4 CONCLUSIONS

In this study, a two-dimensional numerical simulation was analysed for floating breakwater. Previously, the results were validated by experiments that had been carried out by G.H Dong. After that, the development of floating breakwater shapes are analysed by numerical simulation using Computational Fluid Dynamics (CFD) method. There are four development shapes of floating breakwater analysed that are based on the length of bilge keel. The results are that longer the bilge keel, the more efficient the transmission coefficient can reduce wave energy.

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Thermal Front in the North of Java Sea, Indonesia

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Keywords: Thermal Front, IOD Phase, North Java Seas, Single Image Detection, SST.

Abstract: Indian Ocean Dipole (IOD) is a phenomenon in the Indian Ocean caused by differences in the anomaly of Sea Surface Temperature (SST) between the West Coast of Sumatra and the East Coast of Africa. The phase difference of the formation of positive IOD (+) and negative IOD (-). The IOD phase can be known by using the Dipole Mode Index (DMI). This research aims to identify the effect of IOD phase on SST and distribution thermal front. Identification of distribution using the Pearson method, and analysis of the distribution thermal front using the Single Image Detection (SIED) method. Area research in North Java Seas during wet season 2013-2015. IOD phase with normal condition more often occur than IOD phase with weakly condition. Strong correlation occurred with negative direction, and moderate correlation occurred with positive direction. Meanwhile, correlation IOD with rainfall not significant or uncorrelation. During five years the heat temperature was concentrated in the coastal area. IOD with normal condition has a larger thermal front area 1976 km², while IOD with weakly condition has a 624 km² thermal front area. Thermal front distribution more often occur on north area of Central Java with a longwise and widening distribution.

1 INTRODUCTION

Indonesia, which is in the tropics, receives the most amount of solar radiation and is influenced by various atmospheric phenomena, making this region vulnerable to variability and climate change. The climate in Indonesia will not always run normally every year, there is a time when there is a decrease in rainfall but at another time there is high rainfall. In general, the cause of rainfall in Indonesia is influenced by several phenomena including ENSO or commonly called El Nino and Indian Ocean Dipole (IOD). The phenomenon of IOD (Indian Ocean Dipole) can occur where there is a difference in sea surface temperature between the western tropical Indian Ocean or the east African coast and the eastern tropical Indian Ocean or the West Coast of Sumatra (Yamagata et al, 2000 in Fadholi, 2013). Sea surface temperature (SST) is one of the oceanographic parameters that characterize the mass of water in the ocean and is related to the state of the seawater layer below, so that it can be used in analyzing phenomena that occur at sea such as currents, upwelling and fronts (confluence of two water masses different). Thermal front is one of the

oceanographic phenomena that can be identified by looking at the pattern of SST distribution (Inayah, 2015).

The potential of fish in the Java Sea is very large, but the potential utilization of fish in the Java Sea has already exceeded the limit or has reached 95 percent of the total available marine resources, it is one of the factors causing overfishing. Based on the description above, the understanding of the thermal front that is suspected to be an area that is liked by fish is important to be investigated. Thermal front as a local phenomenon cannot be separated from the influence of the adjacent regional oceanographic phenomena, in this case the IOD phenomenon that takes place in the Indian Ocean on the axis of East Africa and West Sumatra which is close to the research location, namely the Java Sea.

The research conducted focuses on the distribution of thermal fronts on IOD using SST parameters and rainfall in the east monsoon. SST and sea rainfall data used were obtained from imagery, and rainfall data was taken from representative samples in each region. Factors such as wind, surface currents, exposure time are ignored in this study.

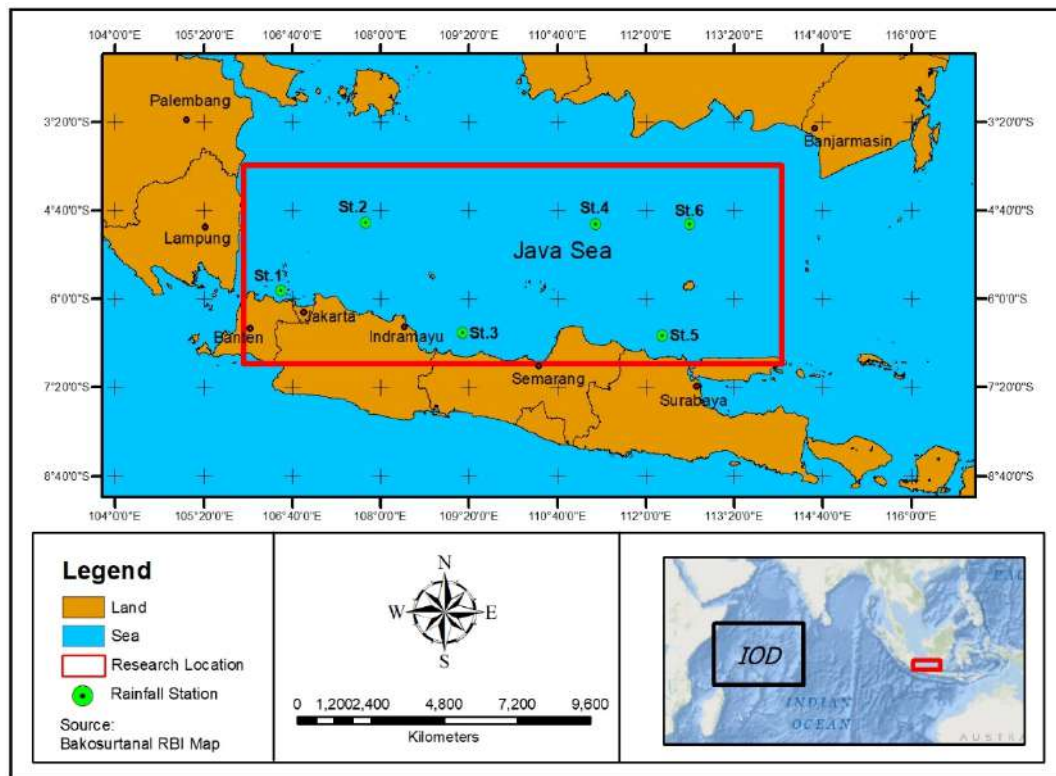


Figure 1: Research Location.

2 RESEARCH METHODS

This research was conducted in August 2018-June 2019 with the use of data for 5 years (2013-2017), with the area in the waters of the Java Sea located from 4° S to 7.1° S and from 106° E to 114° E. The research location is located on the east side of the IOD axis in East Sumatra (East Indian Ocean) with a distance of about 4000 km from the axis of the IOD (Figure 1).

The tools used in the processing and analysis of data in this study are a number of computer programs such as: ArcGIS, ODV, Microsoft Excel. The data used in this study can be seen in Table 1.

Table 1: Types and sources of data.

Data Types	Data Sources
SST	https://oceancolor.gsfc.nasa.gov/
Rainfall	https://apps.ecmwf.int
Dipole Mode Index	http://jamstec.go.jp/frcgc/research/d1/IOD/DATA/DMI

Data processing methods are as follows: (1). IOD data that has been downloaded through the site is processed using Microsoft Excel and is classified seasonally and is classified based on DMI values,

namely the positive and negative phases. As for the strength of the IOD phase consists of normal, moderate, and strong phases (Amri et al, 2010). (2) SST data downloaded is first processed using SeaDas software to cut images according to the location of the study, then processing SST variability and thermal front are processed using ArcGIS with the Single Image Detection method to identify the thermal front using a 0.5° threshold value. (3) Rainfall data is processed using ODV to convert the nc format to txt and seasonal grouping and rainfall calculation are performed on Microsoft Excel.

The analysis in this study uses Pearson correlation analysis which is used to determine the degree of closeness of the relationship between the two variables. The analysis is done by calculating the monthly average of sea surface temperature (SST). Rainfall data and DMI values are not calculated. Correlation test using the DMI value aims to determine the level of relationship between the data parameters to the formation of IOD, namely the value of the X variable which is assumed to be the dependent variable (IOD) and the variable Y as the independent variable namely rainfall and SST.

3 RESULT AND DISCUSSION

3.1 Indian Ocean Dipole Dynamics

The monthly IOD conditions in 2013 occurred under normal conditions with both IOD (+) and IOD (-) phases, but IOD (+) phases were more common. IOD with a weak condition occurred two years during the 2013-2017 period, namely in 2015 and 2017. In 2013 during the east season, there was one negative IOD phase and three positive IOD phases with a condition where the DMI value was 0.08-0.29. In 2014 in the east season there were two negative IOD phases and a positive IOD under normal conditions. Early in the eastern season of 2015 there was a normal positive IOD phase with normal conditions, while the following three months (July-September) a positive IOD phase occurred with a low intensity at DMI 0.53-0.86. In 2016 the IOD phase that occurred was a negative IOD phase with normal intensity. In 2017 the IOD phase that occurred the same as in 2015 which occurred three times the positive IOD phase with weak strength and once with normal strength, but the weak strength IOD phase occurred at the beginning of the east season (June-August).

SST data has been processed by NASA has validated with the conditions in the waters. NASA sails to various waters including Indonesia for retrieve insitu SST data so that can validate derived SST data from MODIS imagery. Thermal front validation uses an algorithm developed by Cayula and Cornillon. This algorithm will search for different populations in each area. The population is the warm temperature area and the cold temperature area. The result accurate to detect thermal front in regional waters.

3.2 SST Variability

The average SST for five years has a value of around 30°-28°C can be shown in Figure 2. In 2013 showed a phase change from IOD (-) with the normal nature to IOD (+) in the next three months. This causes the east season SST in that year continues to decrease every month. SST in 2014 was colder than in 2015, namely the beginning of the eastern season SST conditions around 29°C and decreased in the next two months while at the end of the east season SST experienced a slight increase due to the change from IOD (-) to IOD phase (+). In 2015 SST experienced an increase from 2014 and was hotter than in 2013, a decrease in the value of DMI from the weak phase caused SST conditions in September to increase from August. The 2016 IOD conditions underwent a phase change at the end of the eastern season, ie from the IOD (-) phase to the IOD (+) which caused the average SST conditions in September to be higher than other years which nearly reached 31°C. In 2017, it is indicated that the beginning of the east season has an IOD (-) phase and the end of the season has an IOD (+) phase, causing an increase in temperature from the previous month.

The relationship between the IOD phase and SST values over the past five years is shown in Table 2. The correlation value of IOD with SST in the Java Sea tends to have a strong interpretation relationship with the direction of the negative relationship, this can be seen in three years, namely in 2013, 2015 and 2017. high correlation value. Correlation value that has a relationship of interpretation is having a direction of a positive relationship that occurred in 2014 and 2016. The direction of a negative relationship can be stated that the increase in the value of DMI is followed by a decrease in the value of SST.

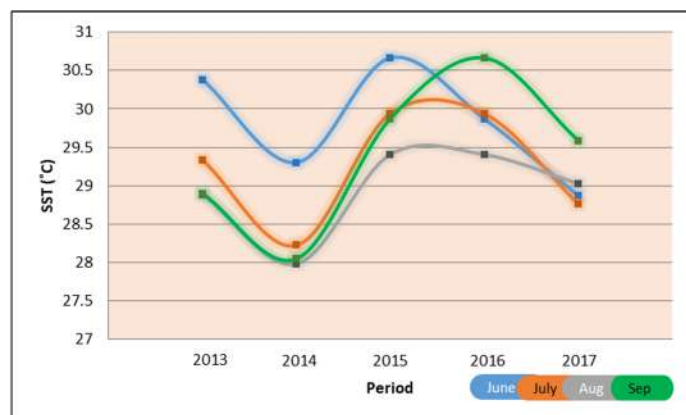


Figure 2: Average SST pattern for five years.

Table 2: Correlations between IOD and SST.

2013			
Period	DMI	SST	Correlation
June	-0.29	30.38	-0.93
July	0.12	29.34	
August	0.10	28.89	
September	0.08	28.90	
2014			
Period	DMI	SST	Correlation
June	0.18	29.3	0.40
July	-0.05	28.23	
August	-0.08	27.98	
September	0.24	28.05	
2015			
Period	DMI	SST	Correlation
June	0.50	30.66	-0.86
July	0.53	29.94	
August	0.86	29.41	
September	0.67	29.87	
2016			
Period	DMI	SST	Correlation
June	-0.23	29.87	0.33
July	-0.43	29.94	
August	-0.15	29.41	
September	-0.05	30.66	
2017			
Period	DMI	SST	Correlation
June	0.63	28.87	-0.93
July	0.83	28.77	
August	0.64	29.03	
September	0.42	29.59	

3.3 Front Thermal Distribution

The extent of the thermal front to IOD is very fluctuating as seen in Figure 3. The highest peak of the thermal front was experienced in 2016 with a normal IOD phase. The number of events of the IOD

phase under normal conditions occurred more during 5 years, while the IOD phase with a weak condition only occurred 3 months for 2 years, namely 2015 and 2017. The IOD phase not only affects the SST but also affects the distribution of the thermal front. IOD phases with normal conditions are more common with larger fronts, whereas IOD phases with weak conditions tend to have smaller front areas.

The number of thermal front events is calculated from the number of pixels formed in the processed sea surface temperature image. The image used is an image with a resolution of 4km, so the area of the formed pixel front is 4 km². Seen from table 3 the number of thermal front events over a period of 5 years in the north waters of Java, then in 2016 experienced the most thermal front events of 823 points, while the lowest thermal front events occurred in 2015 with 347 points. In 2016 the waters condition was experiencing a normal positive IOD phase, whereas in 2015 the water conditions tended to experience a weak positive IOD phase. This condition is thought to affect the distribution of thermal fronts.

3.4 Rainfall Dynamics

Rainfall that occurs in the Java region has a monsoonal rainfall pattern which is indicated by longer drought conditions with one peak drought condition between August and September (Hamada et al, 2009). The monsoonal rainfall pattern was evidenced in 2013-2017 rainfall at all stations in September decreased or did not experience a rainfall

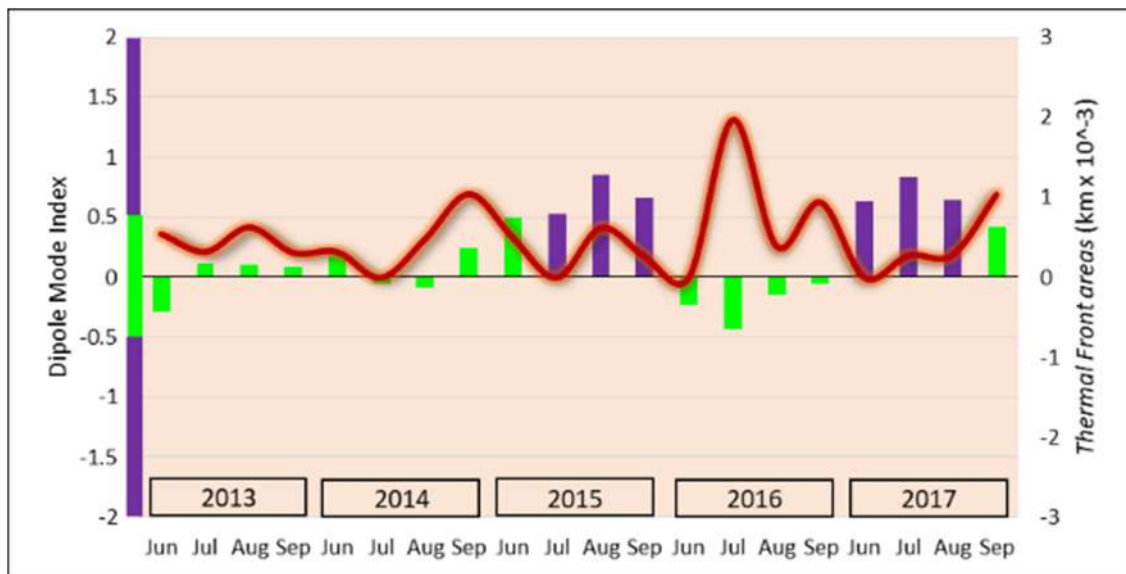


Figure 3: Relationship between IOD and thermal front area for five years.

Table 3: Number of thermal front events over a period of 5 years.

Period		DMI	Thermal Front Area	
			pixel	km ²
2013	June	-0.29	137	548
	July	0.12	81	324
	August	0.10	158	632
	September	0.08	78	312
2014	June	0.18	81	324
	July	-0.05	0	0
	August	-0.08	121	484
	September	0.24	262	1048
2015	June	0.50	125	500
	July	0.53	0	0
	August	0.86	156	624
	September	0.67	66	264
2016	June	-0.23	0	0
	July	-0.43	494	1976
	August	-0.15	98	392
	September	-0.05	236	944
2017	June	0.63	0	0
	July	0.83	70	280
	August	0.64	71	284
	September	0.42	260	1040

supplement for the month (Figure 4). In 2013 the degree of correlation was 0.38 with the direction of a positive relationship, which means that when the DMI value decreased the rainfall also decreased. In contrast to 2013, 2014 had a degree of correlation of -0.73 which means it showed a good correlation between IOD and rainfall with the direction of the negative relationship. So the correlation relationship can be stated when the DMI value rises, the rainfall will decrease. Correlations with the direction of the negative relationship also occurred in 2015. 2016, and 2017 with the degree of correlation respectively

Table 4: Correlation results between IOD and rainfall for 5 years.

Period	DMI	Rainfall (mm)						Correlation
		1	2	3	4	5	6	
June	-0.29	355	768	519	515	768	467	0.38
July	0.12	717	1022	601	700	538	638	
August	0.10	355	472	519	515	666	467	
September	0.08	340	294	189	94	55	88	
2014								
Period	DMI	Rainfall (mm)						Correlation
		1	2	3	4	5	6	
June	0.18	559	661	479	524	308	445	-0.73
July	-0.05	618	804	574	397	251	302	
August	-0.08	558	351	238	124	88	148	
September	0.24	282	69	62	13	0	8	
2015								
Period	DMI	Rainfall (mm)						Correlation
		1	2	3	4	5	6	
June	0.50	352	399	239	355	223	313	-0.13
July	0.53	259	134	44	70	22	47	
August	0.86	307	76	52	16	23	0	
September	0.67	208	39	22	0	50	0	
2016								
Period	DMI	Rainfall (mm)						Correlation
		1	2	3	4	5	6	
June	-0.23	388	769	312	647	428	443	0.34
July	-0.43	405	601	374	54	442	388	
August	-0.15	354	467	232	212	189	144	
September	-0.05	474	476	435	368	298	262	
2017								
Period	DMI	Rainfall (mm)						Correlation
		1	2	3	4	5	6	
June	0.63	483	653	436	503	406	443	0.40
July	0.83	426	629	295	503	261	439	
August	0.64	221	322	68	147	25	123	
September	0.42	304	308	255	195	164	146	

-0.13, 0.34 and 0.40 expressed by IOD and rainfall have poor relationship interpretation.

The relationship between IOD and rainfall has an insignificant correlation shown in table 4. This can be said because of the five years the correlation values obtained tend to be very low, which means IOD and rainfall have no relationship at all. The low correlation between IOD and rainfall is caused by many factors, such as irradiation time, wind direction, and the time needed from evaporation to the formation of rain.

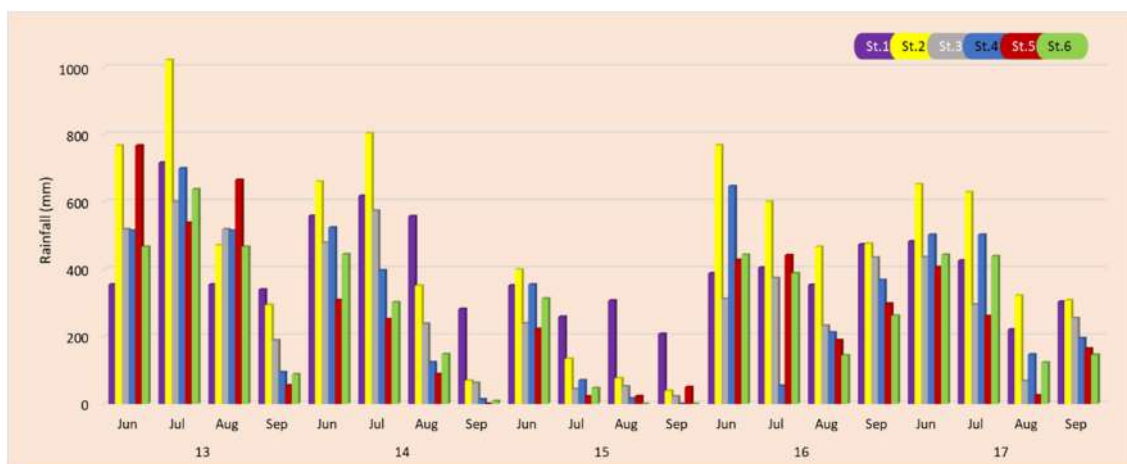


Figure 4: Fluctuations in rainfall for 5 years in the north waters of Java.

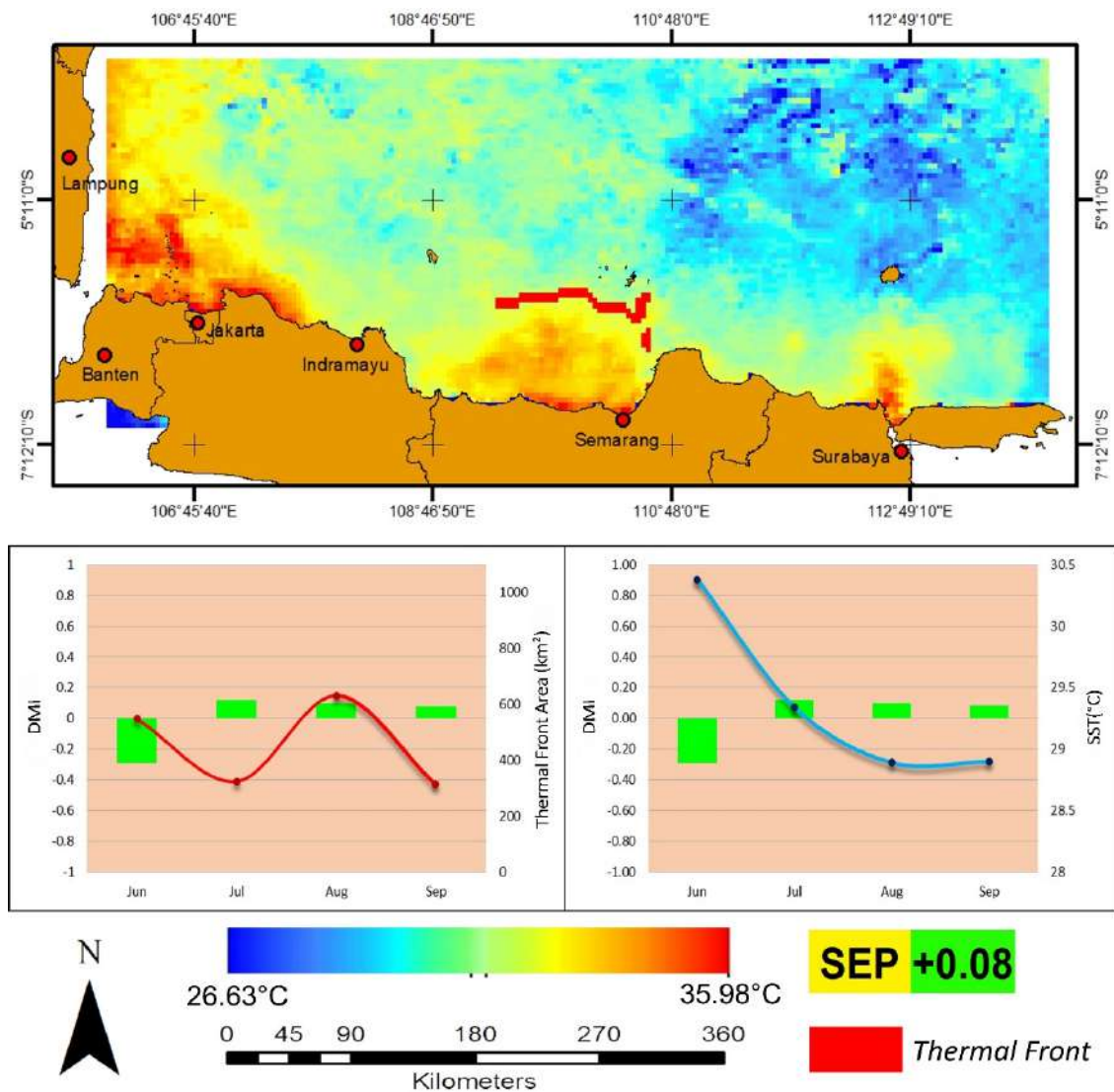


Figure 5: Variability and relationship between IOD and SST and thermal front when the positive IOD condition is normal in 2013.

3.5 Relationship among Parameters

In September 2013 the waters experienced a normal positive IOD phase with the lowest DMI value of 0.08, making the water temperatures colder. The distribution of hot temperatures concentrated near the coast with a maximum temperature of 34.42 °C and cold temperatures spread in the offshore area with a minimum temperature of 27.29 °C. Cold temperatures and warm temperatures spread in the offshore area evenly from west to east. The average temperature this month is 28.9 °C. The degree of correlation between IOD and SST is -0.93 which means it has a very good relationship with a negative relationship direction can be seen in Figure 5. The

occurrence of thermal front in that month was 78 pixels or 236 km² scattered on the north coast of Semarang.

Whereas in September 2016 a normal negative IOD phase occurred with the lowest DMI value of -0.05 causing SST to warm up in the study area. The average SST in this month is 30.66 °C with a maximum and minimum temperature of 35.50 °C and 28.28 °C shown in Figure 6. The relationship between IOD and SST this month has a degree of correlation of 0.33 which is stated IOD has no effect on the rise or fall of SST. The distribution of thermal fronts is spread in coastal and offshore areas with a total occurrence of 236 pixels or an area of 944 km².

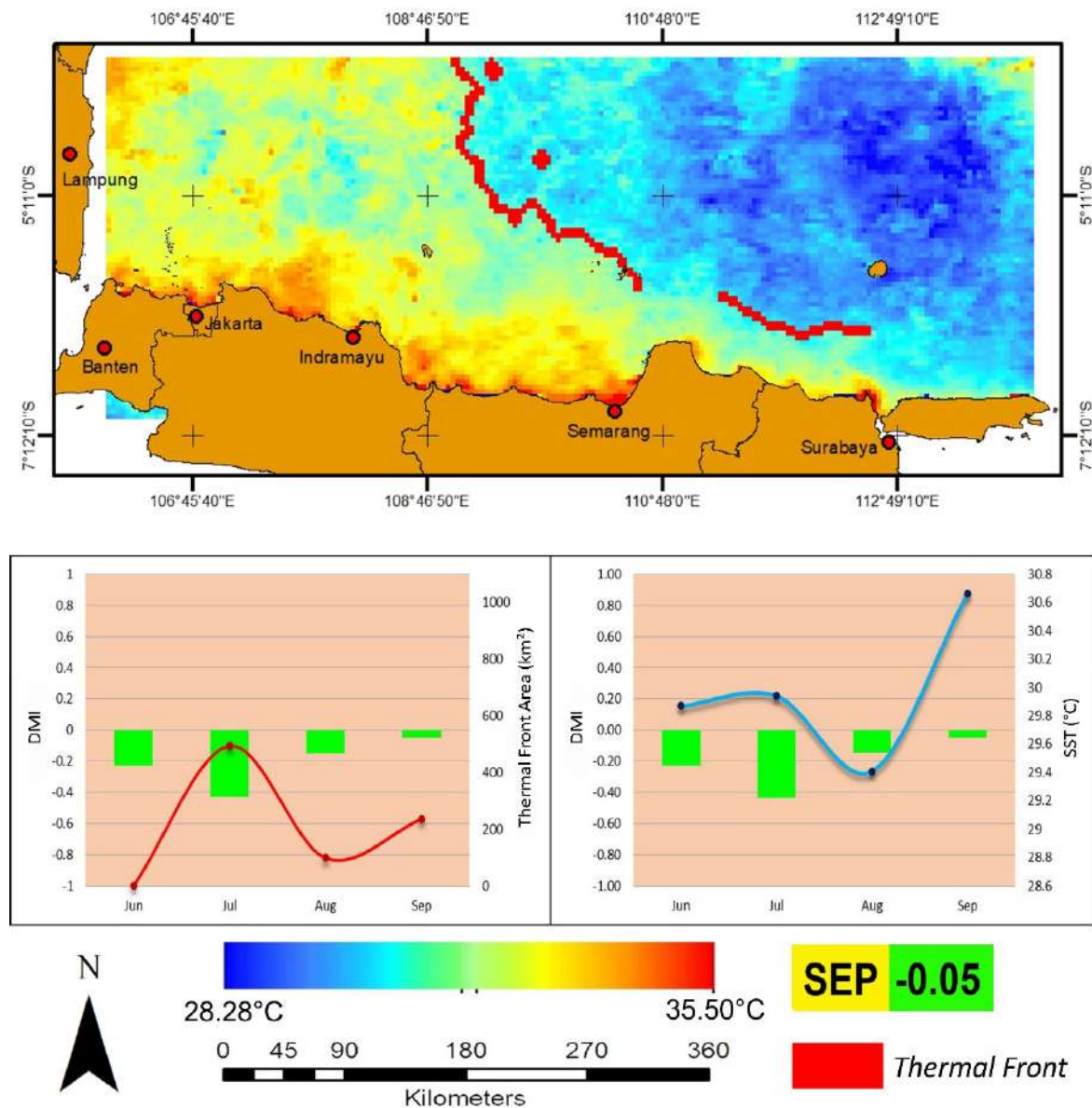


Figure 6: Variability and relationship between IOD and SST and thermal front when negative IOD is normal in 2016.

The weak phase IOD condition was experienced in August 2017 with the lowest DMI value of 0.64. These conditions make the average water temperature decreased to 29.03 °C. hot temperatures remain concentrated in coastal areas with a maximum temperature of 34.39 °C and a minimum temperature of 26.04 °C. The relationship between IOD and SST has a degree of correlation of -0.93 which means it has a very good relationship with the direction of the negative relationship. It can be seen in Figure 7 that the lower the DMI value, the higher the SST value. In this condition also affects the distribution of thermal fronts. The number of

thermal front events on weak IOD tends to be smaller. This month's thermal front is 71 pixels wide or 284 km².

The distribution of hot temperatures in 2013-2017 is concentrated in coastal areas with the highest distance of 200 km to the north and 125 km to the east. When the IOD (+) phase warm temperature distribution reaches the northern waters of East Java, while the IOD (-) phase the warm temperature is only concentrated to the northern waters of Central Java. The more eastward the distribution of heat has a smaller concentration.

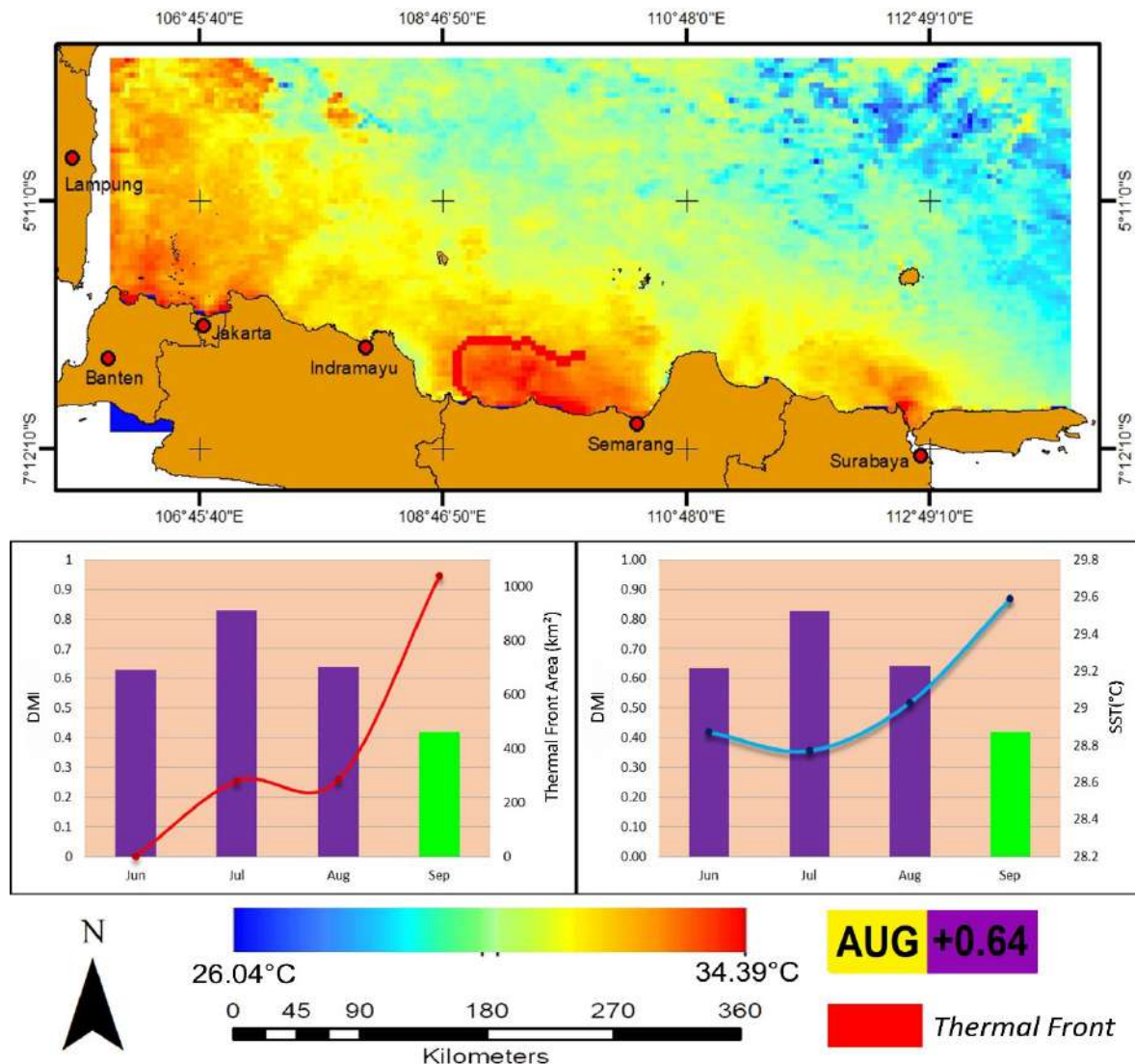


Figure 7: Variability and relationship between IOD and SST and thermal front when IOD is positively weak in 2017.

2013 and 2017 experienced an IOD (+) phase with differences in normal and weak conditions, while in 2016 experienced an IOD (-) normal. Based on the results of the third year SST image processing shows that sea surface temperatures tend to be hotter when a negative IOD phase occurs, and temperatures tend to be cooler when a positive IOD occurs. This can be reinforced by the statement of Yamagata et al., (2004) in Fadholi, (2013) the dipole mode structure characterized by SST anomalies warmer than usual in the west and colder than usual in the eastern Indian Ocean.

The distribution of thermal front tends to be more visible in normal IOD (+) or (-) phases with the highest area of 1976 km², whereas in weak IOD phases the thermal front tends to be less with a

maximum area of 624 km². Thermal front events occur more frequently in the northern waters of Central Java with an elongated and winding shape that has a distribution to the offshore.

4 CONCLUSIONS AND RECOMENDATION

Based on this research, conclusions can be obtained as follows: (1) The phenomenon of IOD in the east season during the five-year phase of IOD with normal conditions occurs more frequently than IOD in weak conditions. The decrease in SST value is caused by the occurrence of the IOD (+) phase and

the warmer SST occurs when the IOD (-) phase. (2) The correlation between IOD and SST in the Java Sea tends to have a negative relationship, which means an increase in the value of DMI is followed by a decrease in the value of SST ($r = -0.9$).

Meanwhile the correlation of IOD to rainfall is not significant which means it does not have a correlation. (3) The heat temperature is concentrated in the coastal area with the furthest distance of 200 km² to the north and is evenly distributed to the northern waters of East Java which tends to occur in the IOD (+) phase. Whereas the IOD (-) phase of the scattered heat only reaches the northern waters of Central Java. (4) Thermal front events are more common when IOD is in the normal phase with the largest area of 1976 km², whereas in the weak phase the thermal front tends to have a smaller area or number of events with a maximum area of 624 km². Several months were not found thermal front events due to temperature differences that did not reach 0.5 °C. The distribution of thermal fronts tends to form in the northern waters of Central Java with a form extending north and south.

Future studies can use data with a longer duration in order to find out the relationship between parameters and use several seasons for comparison. In addition it is necessary to add parameters such as currents, irradiation times, wind to determine the characteristics of the formation of thermal front and IOD.

The result of this study can be used as a reference for further research which is associated with high productivity waters. The location of the thermal front can be further investigated about fishing grounds, so as to avoid overfishing in waters.

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Characteristics of Wind and Wave along the Kenjeran Beach Tourism Areas in Surabaya, Indonesia

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Keywords: Wind, Sea Wave, Beach Tourism, Kenjeran Beach, Management of Coastal Tourism.

Abstract: Waves are movements of ups and downs of sea water that carry energy. This wave energy can erode the beach shore. Many and a few areas of eroded coast will depend on the magnitude of the energy of the waves. This study aims to analyse the characters of wind and waves in the Kenjeran Beach Tourism Area Surabaya. This research was carried out for 8 months, from December 2018 to July 2019 at Kenjeran Beach. The data used is wind data for 10 years (2009-2018). The method used in this study is the method of field surveys, analysis of winds, calculation and analysis of waves in the beach. This research shows that the highest sea waves caused by wind in the Kenjeran tourism area is as high as 0.8 m. The benefits of this research are as a basis in the management of Pamurbaya tourism area, both in maintaining the safety of tour operators / visitors and in the development (management) of tourist area locations.

1 INTRODUCTION

Kenjeran beach is a part of Surabaya's coast which is designated as a tourism area. Kenjeran beach tourism provides an interesting view. Some places must be protected from damage caused by scouring by waves or currents, known as coastal abrasion. In addition to coastal abrasion, accretion also occurred in the area, i.e. increased land due to sedimentation. This event occurred mainly in the mangrove area.

Kenjeran Beach is part of East Coast of Surabaya (Pamurbaya) which is mostly a mangrove area so that this area occurs erosion and accretion. Accretion in Pamurbaya was more dominant than abrasion (Prasita, *et al.*, 2019; Herdianti, *et al.*, 2017, Prasita, 2015).

Mangrove areas in addition to functioning as a protector/conservation are also used for tourism activities. Mangrove tourism provides beauty and comfort for visitors obtained from existing resources/ ecosystems. To maintain and manage the tourism area, the coastal dynamics in Pamurbaya need to be studied in more detail, especially related to characteristics of wind and wave.

Therefore, some of the problems of this study can be formulated as follows: What are the wind and sea wave patterns at the Pamurbaya tourism location for the safety and comfort of visitors and tourism operators?

The purposes of this study is to analyze wind and sea waves for the management of coastal tourism areas, mainly related to visitor safety. The results of the study are useful to find out areas that endanger tourism operators and visitors.

2 RESEARCH METHODS

This research was carried out for 8 months (December 2018 - July 2019) by taking the research location at Surabaya Kenjeran Beach. Research locations from 7° 14' 35.25" S to 7° 19' 15.60" S and from 112° 47' 44.02" E to 112° 50' 22.98" E. The research location is presented in Figure 1.

The tools and materials needed for this research are: Software ArcGis 9.3, wind data for 2009-2018, wave data for 2009-2018. The wind and wave data were processed and analyzed using Software WRPOLY View 8.0.2. The overall data processing flowchart was shown in Figure 2.

Processing of wind data for season and processing of wind data for wave forecasting, wavelength, wave height, breaking wave and wave energy. Correction of location effects was needed because wind data came from the ground stations not measured directly above sea level or on the coast. To change the wind speed blowing on the water, an existing chart was

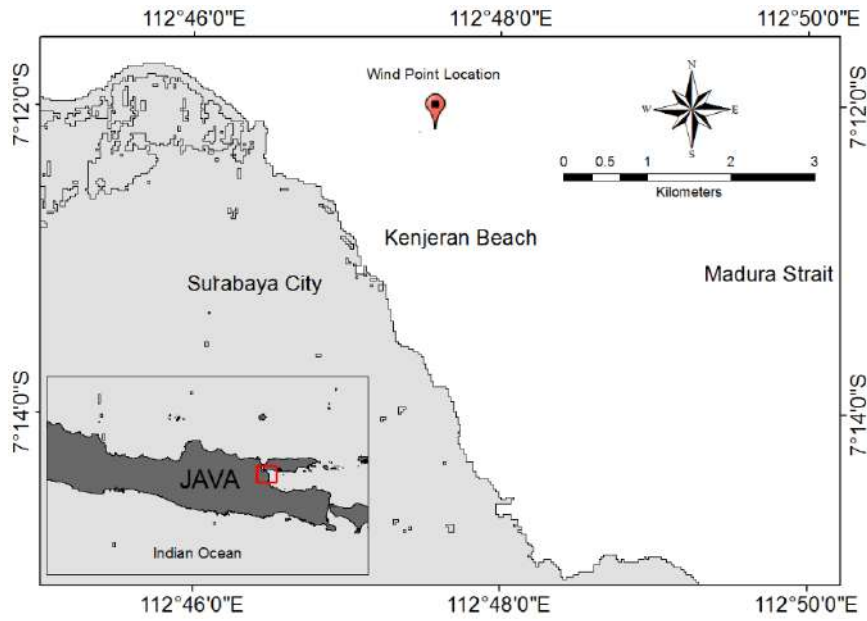


Figure 1: Research Location.

used (Permatasari, 2018; Engky *et al.*, 2018). The relationship between sea breeze and land wind was shown in Equation 1.

$$RL = U_W/U_L \quad (1)$$

Where U_L is land wind speed, U_W is sea breeze speed, RL is the relationship between sea wind speed and land wind.

After the location effect was corrected, the next step was to convert wind speed into a wind stress factor as shown in Equation 2.

$$U_A = 0.71U^{1.23} \quad (2)$$

Where, U_A is wind stress factor (m/s), U is wind speed (m/s).

Wave forecasting through graphs (Permatasari, 2018), if the fetch length (F), wind stress factor (U_A) and duration were known then the height and significant wave period could be calculated. To estimate the wave height, it was necessary to know the duration of sea wind speed, wind stress factor (U_A) and effective fetch. This estimation only applied to significant waves, namely the average of 33% of the highest waves, of all waves that occurred in waters that had been predicted of high waves (Triatmodjo, 1999).

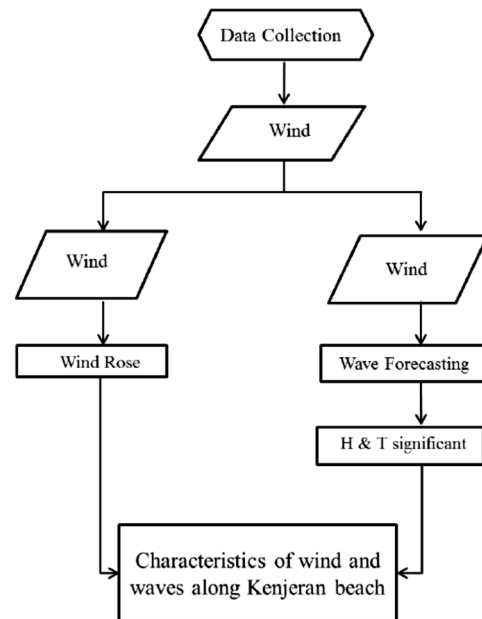


Figure 2: Flowchart of the Research.

Effective fetch is a wave generator area with constant wind direction and direction. The effective fetch equation is shown in equation 3.

$$F_{eff} = \frac{\sum X_i \cos \alpha}{\sum \cos \alpha} \quad (3)$$

Where, F_{eff} is effective average fetch, X_i is length of the fetch segment measured from the wave

observation point to the end of the fetch, is deviation on both sides of the wind direction, using an increase of 6 ° to an angle of 42° on both sides of the wind direction.

Estimation of wave forecasting did through graphs to obtain the period and height of the wind generation wave. Forecasting waves did through graphs if fetch length (F), wind stress factor (UA) and duration are known then height and significant wave period can be calculated. To estimate the wave height, it was necessary to know the duration of sea wind speed, wind stress factor (UA) and effective fetch. This estimation only applied to significant waves, namely the average of 33% of the highest waves, of all waves that occurred in waters that have been predicted of high waves (Triatmodjo, 1999).

The determination of the breaking wave was carried out using the graph. The graph stated the relationship between the calculation of H_o/gT^2 and H_b/H_o . The H_o value is the height of the equivalent waveform, g is gravity and T is the wave period. H_o was obtained from the calculation of the refraction coefficient and wave height that propagates from the deep sea as presented in Equation 4.

$$H_o = K_r \times H_b \quad (4)$$

After obtaining a break wave height value (H_b) continued with determining the depth of the breaking wave (d_b). Determination of the breaking wave depth was based on the calculated value of H_b/gT^2 .

Wave energy was calculated using Equation 5 where ρ is the density of seawater with a value of 1,030 kg/ m³, g is the earth's gravity with a value of 9.81 m/s² and H is the wave height at a certain depth. In this study, the calculated wave energy was wave energy at the depth of the breaking wave occurred.

$$E = \frac{\rho g H^2}{8} \quad (5)$$

3 RESULT AND DISCUSSION

3.1 Analysis Result of the Wind

The distribution of wind speed and direction in the District of Kenjeran for 10 years analyzed on an annual basis was shown in Table 1. The distribution of wind analyzed on an annual basis was intended to determine the significant wind speed and direction that have an effect in 10 years. The wind that occurs had an average speed of 5.31 m/s. This value

occurred during the 10 years of research, namely 2009 to 2018. The variation in the dominant wind direction movement occurred in the range of 90° to 270° but overall the wind that occurred came from the East and Southeast in accordance with the wind direction category based on Dauhan (2013), namely Southeast Wind Direction Category if they have values of 112.5° to 157.5°.

Table 1: Distribution of winds for 10 years.

Year	Wind	
	Speed (m/s)	Direction (°)
2009	1.97	135
2010	2.07	135
2011	2.98	90
2012	3.31	135
2013	3.39	90
2014	3.51	90
2015	2.59	135
2016	2.04	90
2017	2.55	135
2018	2.87	270

3.1.1 Distribution of Wind Speed and Direction in 2009

Wind speed and direction in 2009 were shown in Figure 3 (A) and the frequency distribution of wind speed was shown in Figure 3 (B). This year's wind speed consisted of a range of 0-2 m/s which was the lowest range in the 2009 wind speed and direction pattern, range 2-4 m/s, 4-6 m/s, 6-8 m/s and the highest range that occurred this year was > 8 m/s.

The dominant wind direction came from Southeast, second East and third West. Wind from the Southeast is the most dominant wind direction with a percentage of 26.4%.

3.1.2 Distribution of Wind Speed and Direction in 2010

The wind speed that occurred in 2010 consisted of a range of 0-2 m/s, the lowest range in the pattern of wind speed and direction in 2005, a range of 2-4 m/s, a range of 4-6 m/s, a range of 6-8 m/s, and the highest range was >8 m/s. The dominant wind direction came from the Southeast, East and West. Wind from the southeast was the most dominant wind direction with a percentage of 26.9%. The speed and direction of the wind above was illustrated in Figure 4 (A). Frequency distribution of wind speed in the range of 0-2 m/s at 48.8%, 2-4 m/s at 40.8%, 4-6 m/s

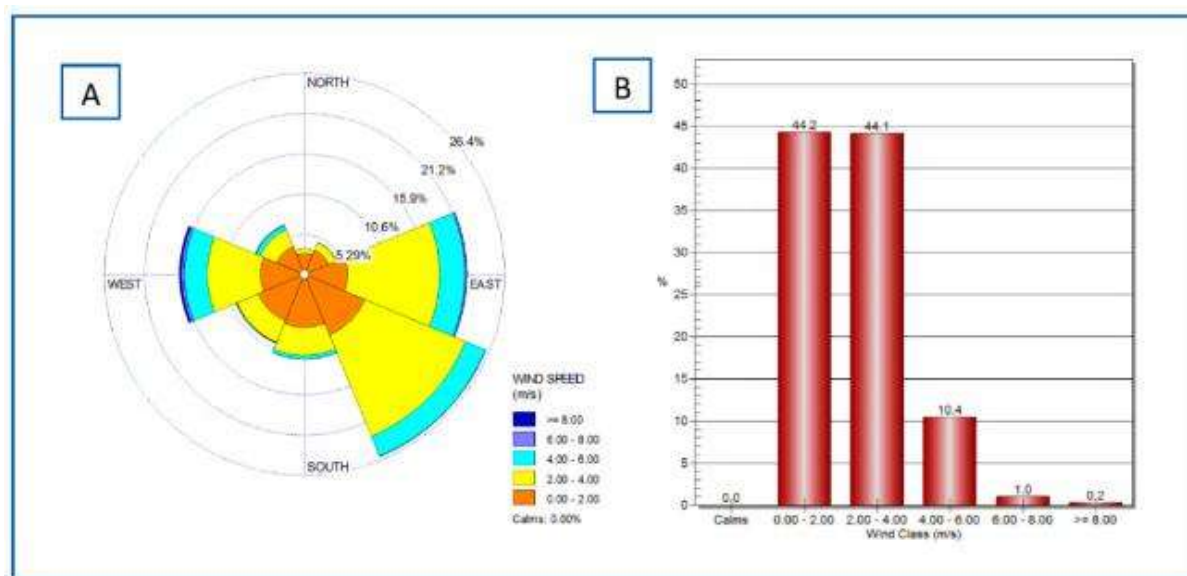


Figure 3: Patterns and distribution of wind frequencies in 2009.

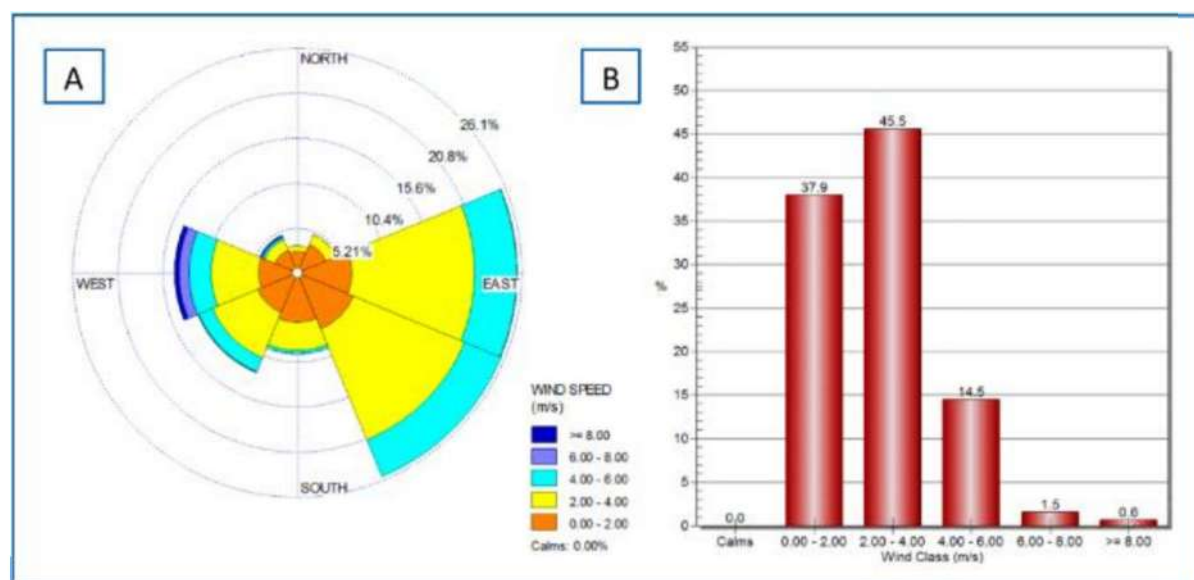


Figure 4: Patterns and distribution of wind frequencies in 2010.

at 10.4%, speed 6-8 m/s at 2.4% and >8 at 0.5%. The value of the distribution was shown in Figure 4 (B).

3.1.3 Distribution of Wind Speed and Direction in 2011

The pattern of wind speed and direction in 2011 was shown in Figure 5 (A) while Figure 5 (B) was the frequency distribution of wind speeds that occur. Wind speed was divided into several ranges. This

year's range consists of 0-2 m/s which was the lowest speed, 2-4 m/s, 4-6 m/s, 6-8 m/s and the highest range is more than 8 m/s. The direction of the dominant wind came from East then West and Southeast. Wind from the Southeast is the most dominant wind direction with a percentage of 34%. Frequency distribution of wind speed at 0-2 m/s at 20.9%, 2-4 m/s at 35.4%, 4-6 m/s at 28% then at 6-8 m/s at 9.4 % and >8 m/s at 2.2%.

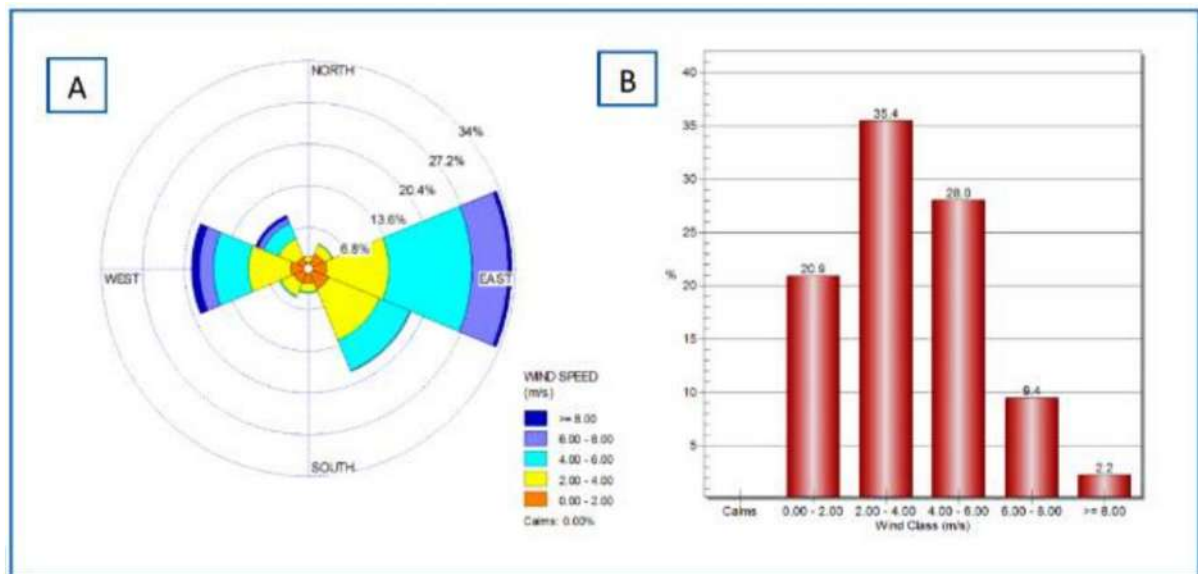


Figure 5: Patterns and distribution of wind frequencies in 2011.

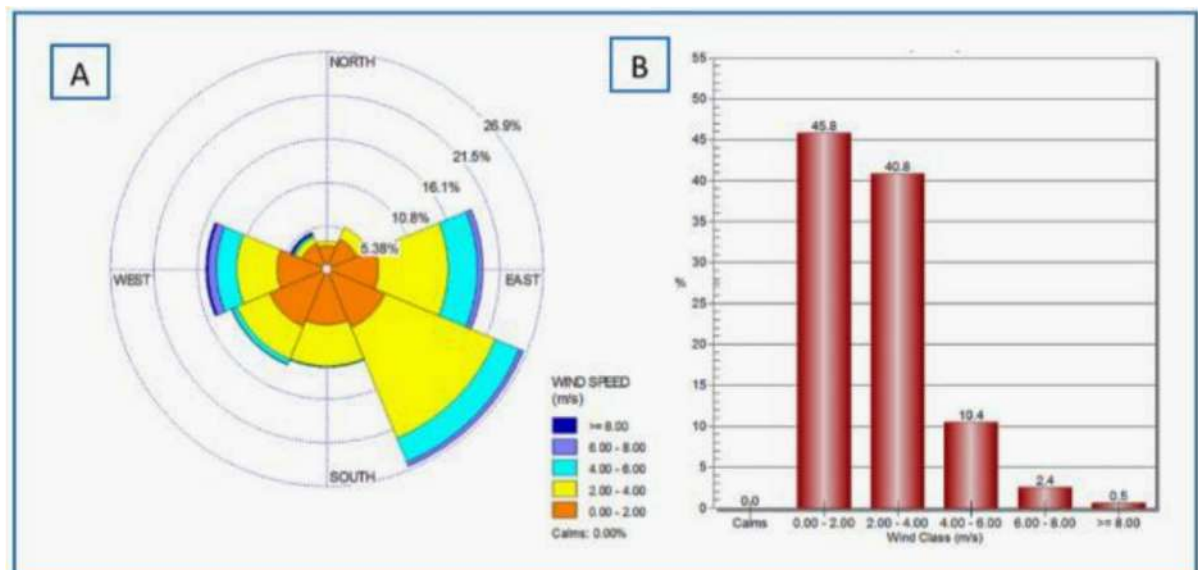


Figure 6: Patterns and distribution of wind frequencies in 2012.

3.1.4 Distribution of Wind Speed and Direction in 2012

Wind speed and direction in 2012 were shown in Figure 6(A) and the distribution of wind speed is shown in Figure 6(B). This year's wind speed consisted of a range of 0-2 m/s which was the lowest range of the wind speed and direction pattern in 2012, then a range of 2-4 m/s, a range of 4-6 m/s, 6-8 m/s and the highest range that occurred this year is > 8 m/s. The dominant wind direction comes from the Southeast and East. Wind from the Southeast

was the dominant wind direction with a percentage of 26.1% and East with the same presentation of 26.1%. Frequency distribution of wind speeds in the range of 0-2 m/s at 37.9%, 2-4 m/s at 45.5%, 4-6 m/s at 14.5%, 6-8 m/s at 1.5% and >8 m/s at 0.6%.

3.1.5 Distribution of Wind Speed and Direction in 2013

The wind speed that occurred in 2013 consisted of a range of 0-2 m/s, the lowest range in the pattern of wind speed and direction in 2013, a range of 2-4

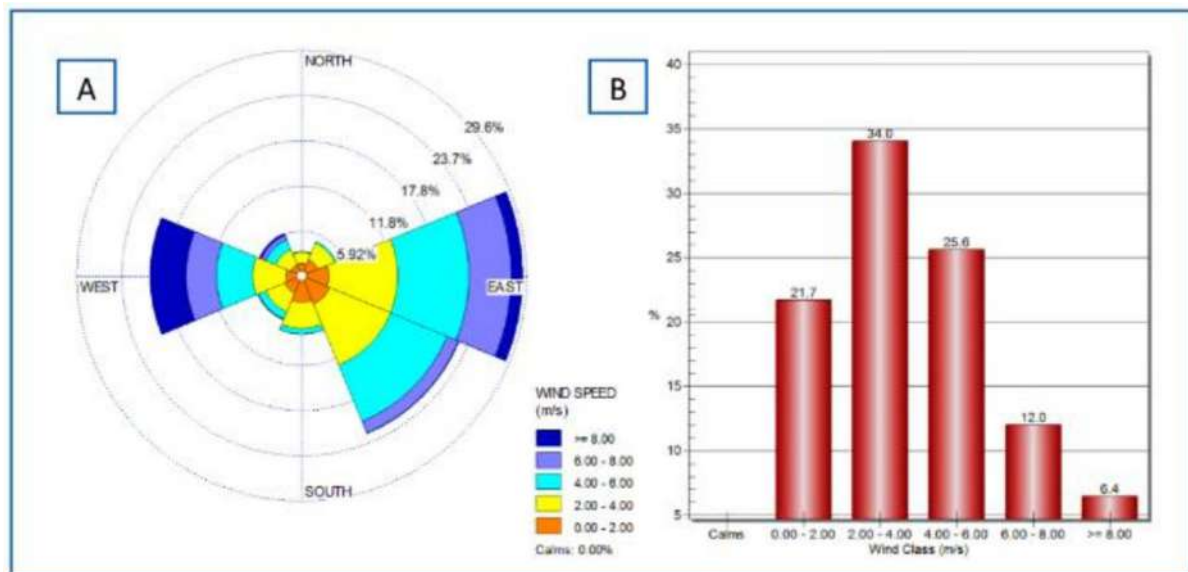


Figure 7: Patterns and distribution of wind frequencies in 2013.

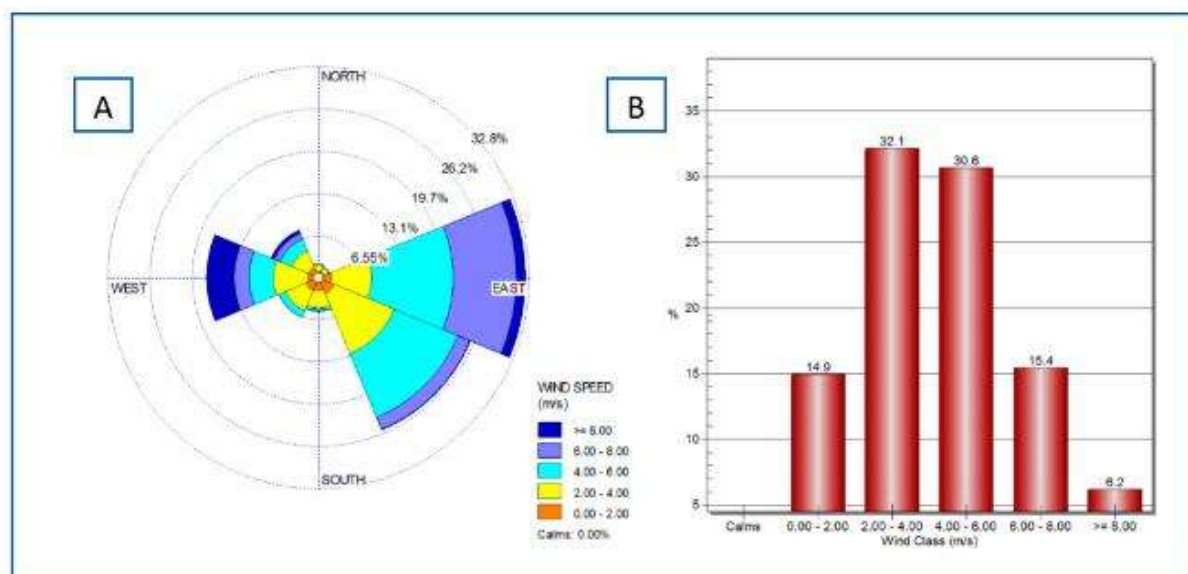


Figure 8: Patterns and distribution of wind frequencies in 2014.

m/s, a range of 4-6 m/s, 6-8 m/s and a range the highest was >8 m/s. The direction of the dominant wind originated from the East, Southeast and West.

The most dominant wind was wind direction from the East with a percentage of 29.6%. The speed and direction of the wind above was illustrated in Figure 7(A). Frequency distribution of wind speed in the range of 0-2 m/s at 21.7%, 2-4 m/s at 34%, 4-6 m/s at 25.6%, 6-8 m/s at 12% and >8 m/s at 6.4%. The frequency distribution values were shown in Figure 7(B).

3.1.6 Distribution of Wind Speed and Direction in 2014

Wind speed and direction in 2014 were shown in Figure 8(A) and the distribution of wind speed was shown in Figure 8(B). This year's wind speed consisted of a range of 0-2 m/s which was the lowest range in the 2014 wind speed and direction pattern, range 2-4 m/s, range 4-6 m/s, 6-8 m/s and range the highest occurred this year was >8 m/s. The dominant wind direction came from the East, second from the

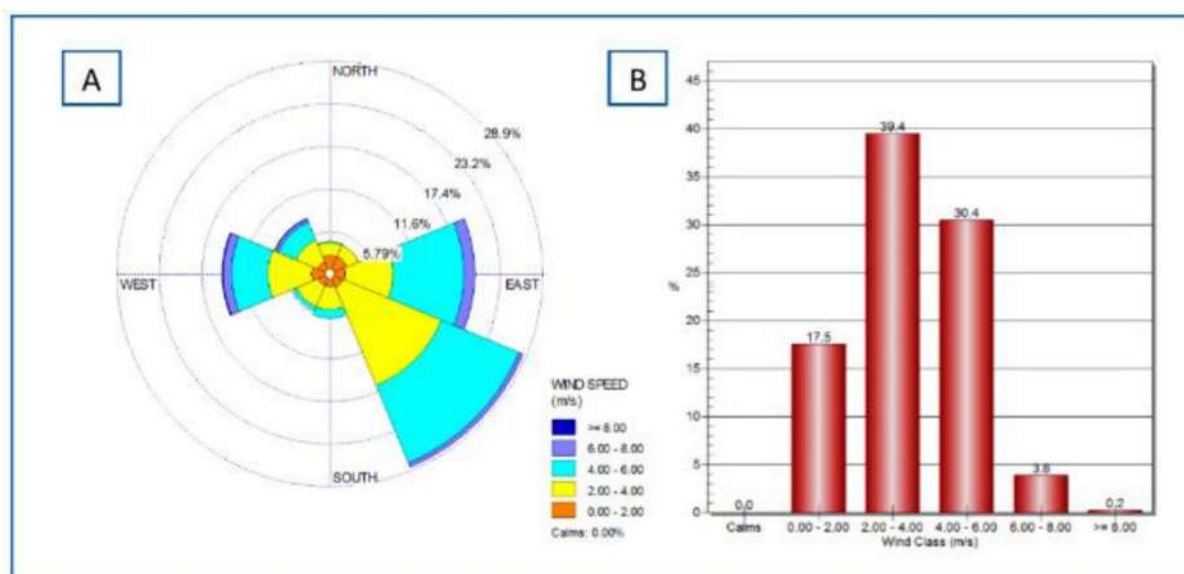


Figure 9: Patterns and distribution of wind frequencies in 2015.

Southeast and third from the West. Wind from the southeast was the most dominant wind direction with a percentage of 32.8%. Frequency distribution of wind speed was in the range of 0-2 m/s at 14.9%, 2-4 m/s at 32.1%, 4-6 m/s at 30.6%, 6-8 m/s at 15.4% and > 8 m/s at 6.2%. The frequency distribution values were shown in Figure 9(B).

3.1.7 Distribution of Wind Speed and Direction in 2015

Wind speed and direction in 2015 were shown in Figure 9(A) and the distribution of wind speed was shown in Figure 9(B). This year's wind speed consisted of a range of 0-2 m/s which was the lowest range in the 2014 wind speed and direction pattern, range 2-4 m/s, range 4-6 m/s, 6-8 m/s and range the highest occurred this year is > 8 m/s. The dominant wind direction came from Southeast, second East, and third West. Wind from the Southeast was the most dominant wind direction with a percentage of 28.9%. Frequency distribution of wind speeds was in the range of 0-2 m/s at 17.5%, 2-4 m/s at 39.4%, 4-6 m/s at 30.4%, 6-8 m/s at 3.8% and > 8 m/s at 0.2%. The frequency distribution values were shown in Figure 9(B).

3.1.8 Distribution of Wind Speed and Direction in 2016

The wind speed that occurred in 2016 consisted of a range of 0-2 m/s which was lower than the range in the pattern of wind speed and direction in 2014, a range of 2-4 m/s, a range of 4-6 m/s, 6-8 m/s and the

highest range that occurred this year was > 8 m/s as the highest wind speed this year. The first dominant wind direction came from the East and second from the Southeast. Wind from the East was the most dominant wind direction with a percentage of 29.1%. The speed and direction of the wind above was illustrated in Figure 10(A).

Frequency distribution of wind speed in the range of 0-2 m/s at 27.5%, 2-4 m/s speed at 39.4%, 4-6 m/s speed at 30.4%, speed 6-8 m/s at 3.8% and > 8 m/s at 0.2%. The frequency distribution values were shown in Figure 10(B).

3.1.9 Distribution of Wind Speed and Direction in 2017

The pattern of wind speed and direction on the coast in 2017 was shown in Figure 11(A) while Figure 11(B) was the frequency distribution of wind speeds that occurred. Wind speed was divided into several ranges. The range consisted of 0-2 m/s which was the lowest range, range 2-4 m/s, range 4-6 m/s, 6-8 m/s and the highest range that occurred this year was > 8 m/s as the highest wind speed this year. The first dominant wind direction came from the Southeast, the second West and the third South. Wind from the Southeast is the most dominant wind direction with a percentage of 29.3%. Frequency distribution of wind speeds in the range of 0-2 m / s at 21.7%, speeds of 2-4 m/s at 48.3%, speeds of 4-6 m/s at 23.7%, speeds of 6-8 m/s s at 5% and > 8 m/s at 0.6%. The frequency distribution values were shown in Figure 11(B).

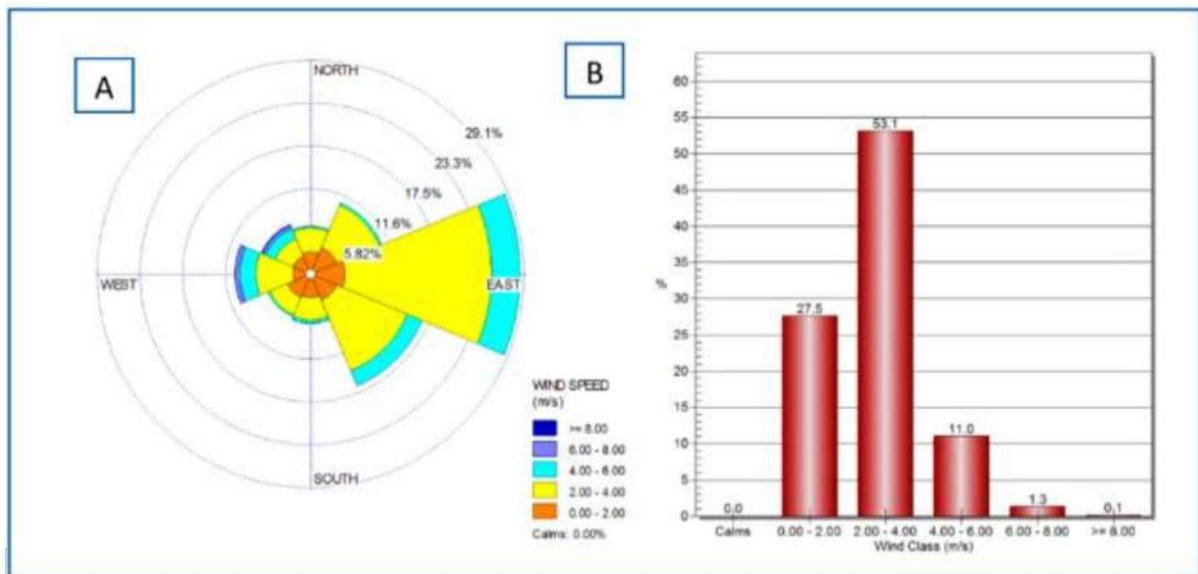


Figure 10: Patterns and distribution of wind frequencies in 2016.

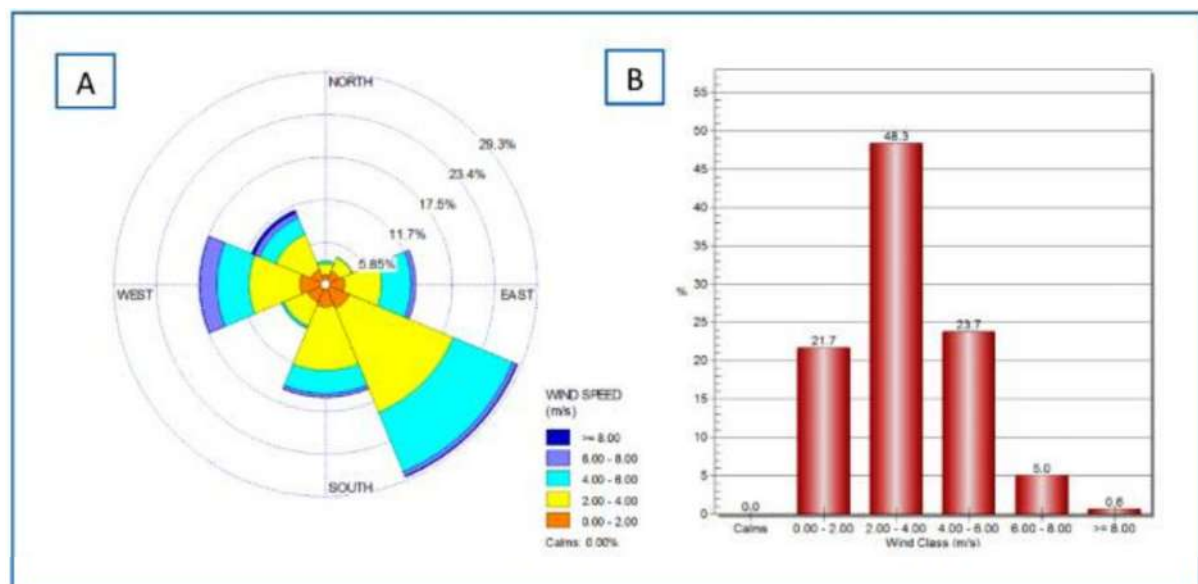


Figure 11: Patterns and distribution of wind frequencies in 2017.

3.1.10 Distribution of Wind Speed and Direction in 2018

Wind speed and direction in 2018 were shown in Figure 12(A) and the distribution of wind speed was shown in Figure 12(B). This year's wind speed consisted of a range of 0-2 m/s which was the lowest range in the pattern of wind speed and direction in 2018, range 2-4 m/s, range 4-6 m/s, range 6-8 m/s and the highest range that occurred this year was more than 8 m/s. The dominant wind direction came

from the West, then the second dominant Southeast and East third. Wind from the West was the most dominant wind direction having a percentage of 25, 7%. Frequency distribution of wind speeds was in the range of 0-2 m/s at 13.5%, speeds of 2-4 m/s at 45.1%, speeds of 4-6 m/s at 31.2%, speeds of 6-8 m/s at 9.2% and >8 m/s at 1%. The frequency distribution values were shown in Figure 12(B).

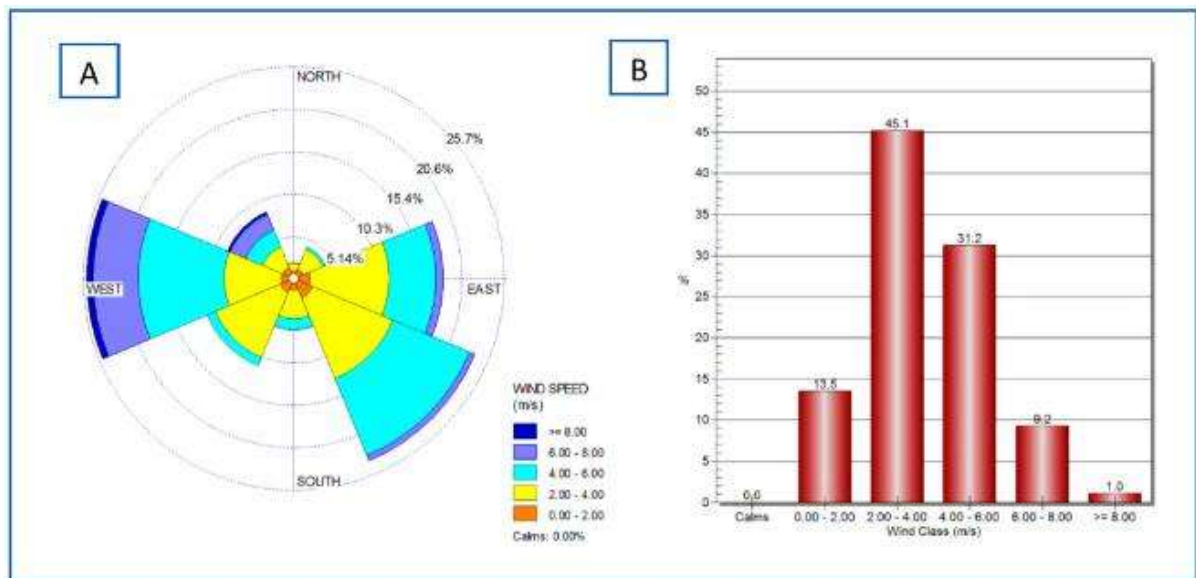


Figure 12: Patterns and distribution of wind frequencies in 2018.

3.2 Results of Effective Fetch Calculation

The drawing and calculation of the fetch was done to determine the region of wave formation caused by wind in an area. Fetch in the Kenjeran region was presented in Figure 13. Fetch occurred in the Southeast direction based on the dominant wind that occurred in the region with the calculated fetch value presented in Table 2 which was 73.87 km. The length of the fetch line at an angle of 6° to 42° with respect to the main direction line (0°) was blocked by land while the angle of -6° to -42° with respect to the line of 0° was unobstructed by land. The line that was unobstructed by the land had an infinite distance but the wind that blew on an infinite area would still have energy that was increasingly fading away and eventually disappear. If the fetch line had a distance of more than 200 km, the line was said to have a maximum fetch distance of 200 km.

3.3 Sea Wave

In forecasting waves before getting high values and wave periods required values of the wind stress factor (UA) and the fetch length that occur in the region. UA and fetch values as well as the results of forecasting height and wave periods each year were presented in Table 3. Wave fluctuations to facilitate the reading of wave fluctuations each year were presented in Figure 14.

Table 2: Calculation of fetch.

No	Direction (α)	F_i (km)	$\cos \alpha$	$F_i \cos \alpha$	Fetch Effective (km)
1	42	8.40	0.74	6.25	73.87
2	36	9.40	0.81	7.60	
3	30	10.99	0.87	9.52	
4	24	11.85	0.91	10.83	
5	18	12.88	0.95	12.25	
6	12	15.55	0.98	15.21	
7	6	19.31	0.99	19.20	
8	0	28.37	1.00	28.37	
9	6	200.15	0.99	199.05	
10	12	200.14	0.98	195.77	
11	18	144.61	0.95	137.54	
12	24	128.03	0.91	116.96	
13	30	114.01	0.87	98.73	
14	36	93.61	0.81	75.73	
15	42	87.49	0.74	65.02	
Total			13.51	998.03	

Patterns of wave fluctuations form non-repetitive curves. In the first 4 years of the study, wave heights had the same wave heights because UA formed less than 5, while in the next 2 years there was an increase then there was a decrease in wave height in 2015 and again the same in 2015 to 2018 wave heights like the 4 years earlier. Increased wave height occurred in 2013 with a difference of 0.004 m from the mean wave height while a decrease occurred in 2014 with a difference of 0.044 m from the average wave height. The wave height value each year was then searched for significant wave values that have occurred for 10 years in the Kenjeran Districts were presented in Figure 15. Significant wave results in 10

years found that the wave height occurred 0.77 m with a 4.64-second wave period. The direction of the waves was adjusted to the direction of the wind that forms them, originating from the Southeast. This result can be compared with the result from the map

of significant index, Agency for Marine and Fisheries Research, Ministry of Marine Affairs and Fisheries Republic of Indonesia. It is in the moderate level with significant height index 0.431 – 0.570 m.

Table 3: Calculation of forecasting waves.

Year	Wind Speed (m/s)	RL	UW	UA	Feff	Wave	
						H (m)	T (s)
2009	1.97	1.80	3.55	3.37	73.87	0.75	4.6
2010	2.07	1.75	3.63	3.47	73.87	0.75	4.6
2011	2.98	1.65	4.92	5.04	73.87	0.75	4.6
2012	3.31	1.60	5.30	5.52	73.87	0.75	4.6
2013	3.39	1.59	5.39	5.64	73.87	0.76	4.62
2014	3.51	1.58	5.54	5.84	73.87	0.8	4.7
2015	2.59	1.70	4.40	4.39	73.87	0.75	4.6
2016	2.04	1.75	3.57	3.40	73.87	0.75	4.6
2017	2.55	1.70	4.33	4.31	73.87	0.75	4.6
2018	2.87	1.65	4.73	4.80	73.87	0.75	4.6

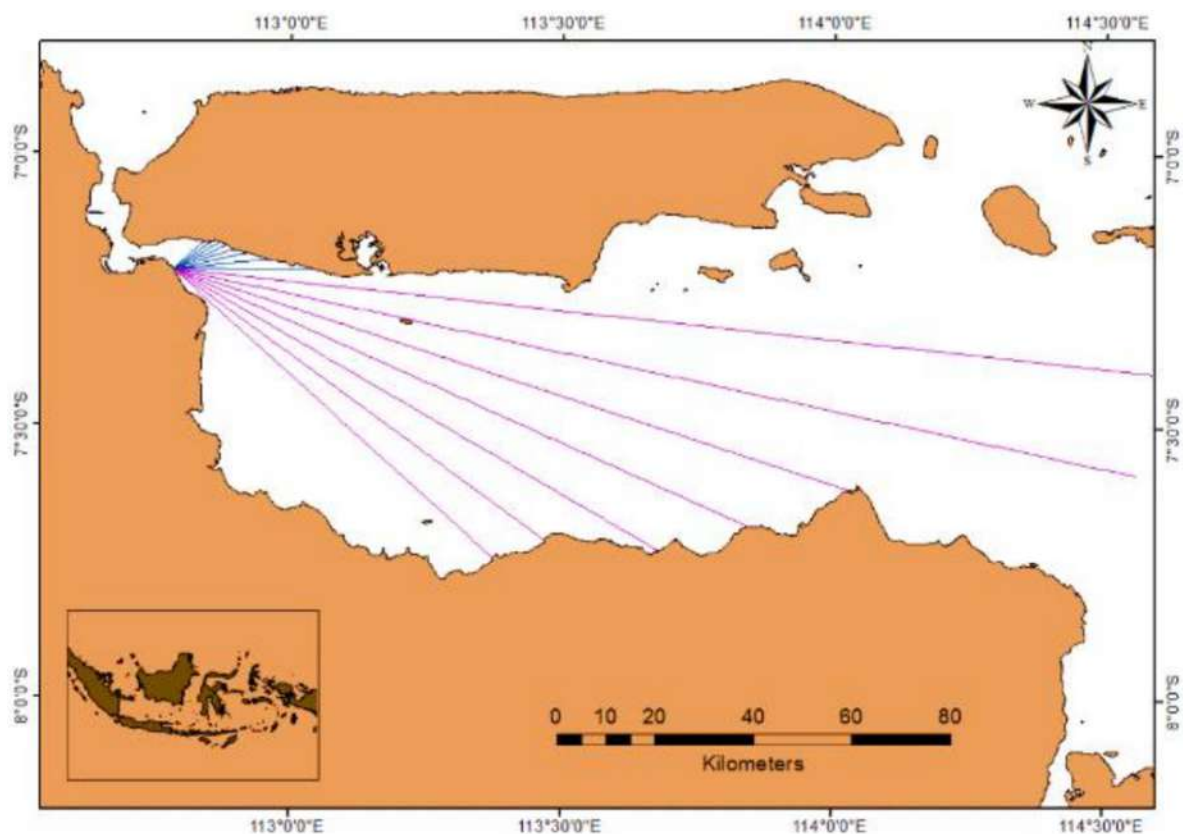


Figure 13: Fetch that occurred in the east coast region of Surabaya.

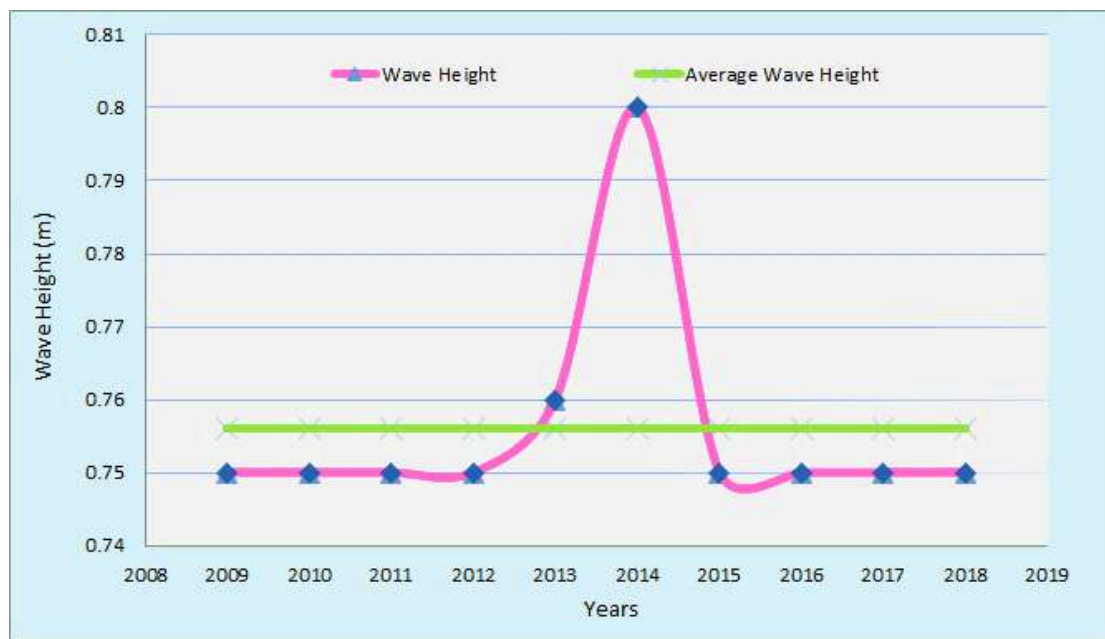


Figure 14: Wave height for 10 years in the east coast region of Surabaya.

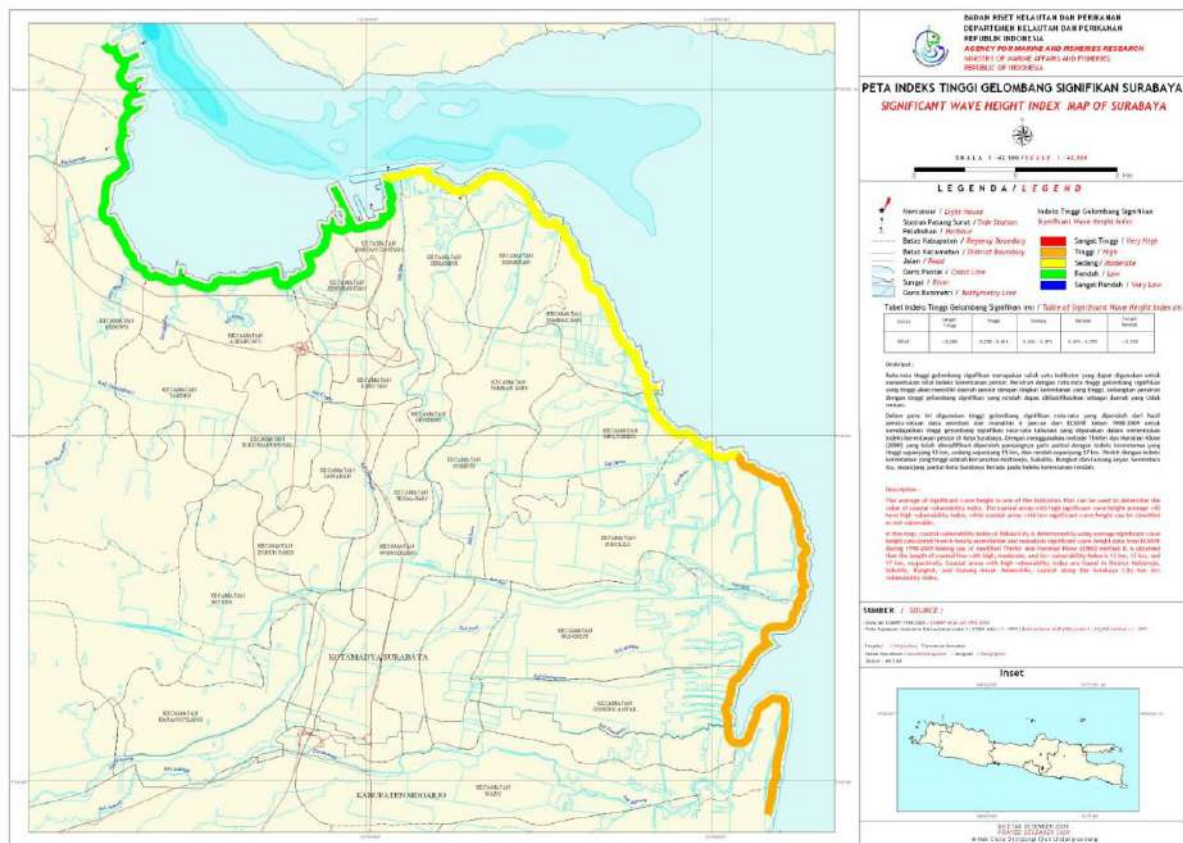


Figure 15: Map of significant wave height index in Surabaya.

4 CONCLUSIONS AND RECOMENDATION

The conclusions of this study are the sea waves in the coastal tourism area of Kenjeran are relatively small, both height and wave energy. The highest wave height of 0.8 m occurred in 2014 so that in terms of security, safety and security of visitors, the location of Kenjeran beach is very suitable for tourist sites. Further research needs to be conducted on current conditions, both currents caused by waves and tides as well as soil/geological conditions for the development of underwater tourism.

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Influence of Waters in Silvofishery Ponds on Wonorejo Mangroves That Contaminated by Heavy Metals Pb, Cd, and Cu toward Aquaculture Animals

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Keywords: Silvofishery Ponds, Heavy Metal, Toxicity, Milkfish, Wonorejo.

Abstract: Wonorejo silvofishery ponds obtained fresh water from two rivers, namely Jagir River and Afour River which flow through the city of Surabaya before entering Wonorejo mangroves area. The white shrimp and milkfish that have been cultivated in silvofishery pond often fail to be harvested. Preliminary observations found that Pb content in Wonorejo ponds reached 0.082 ppm. The purpose of this study is to determine the toxicity of waters in the Wonorejo silvofishery ponds that contaminated with heavy metals (Pb, Cd, and Cu) toward *Chanos chanos* and *Penaeus merquensis*. The results of the tests revealed that the concentrations of Pb, Cd, and Cu in R. Avour were 0.284; 0.0380; and 0.017 ppm respectively. On the other hand, the concentration of Pb in the pond water is 0.304; 0.047; and 0.024 ppm. Laboratory test shows that milkfish can survive in the condition where water is contaminated by heavy metals until the concentration increases ten times from the concentration of metals in nature, as long as the other environmental parameters are appropriate. Meanwhile, white shrimp cannot survive more than two days in these conditions.

1 INTRODUCTION

Mangrove Wonorejo is part of mangrove ecosystem located on the shore of Pamurbaya (East coast of Surabaya). Currently, the Wonorejo mangrove is utilized by the community for the cultivation of traditional ponds with shrimp and milkfish commodities. Wonorejo Mangrove which is the conservation area of Surabaya City government is also managed for Mangrove ecotourism and Mangrove Information Center (MIC). One form of mangrove conservation management from MIC is Mangrove Pond (Silvofishery Pond).

Silvofishery pond is utilized for the cultivation of milkfish and shrimp by the management of MIC. In 2017 and 2018, there were also conducted a test of silvofishery cultivation of mangrove crabs for the research of the Ministry of Research, Technology and Higher Education of Republic of Indonesia (Ristekdikti) at the site. But the harvest was not good enough because the death rate was still high. It was suspected that this high death was caused by the quality of water in the Wonorejo Mangrove Pond which was low and polluted by the heavy metals. This reason was delivered because the cultivated biota in

other farms were also experiencing mass death at the same time. The results of observations in previous studies showed that the average of heavy metals in the waters of the Wonorejo Mangrove Pond was above the quality standard of marine-life living needs. The highest heavy metal level was in cadmium elements. The heavy metal concentration of the Wonorejo mangrove pond water in the measurement of July 2018 resulted; Hg at 0.004 mg/l, Cd at 0,087 mg/l, and Pb at 0,022 mg/l. This heavy metal concentration led to the death of more than 50% of cultivated biota at the time. The regulation governing the quality threshold for marine life is the decree of the Minister of Environment No. 51 year 2004 about the quality standards for marine life, which are Hg at 0,001 mg/l, Cd at 0,001 mg/l, and Pb at 0,008 mg/l.

To find out whether this heavy metal content is always that high in the waters of Mangrove Pond Wonorejo and whether this heavy metal is indeed triggering the death of biota cultivation, it is considered necessary to conduct the toxicity test on cultivate biota in the Wonorejo Mangrove Pond, which are milkfish (*Chanos chanos*) and local white prawn (*Penaeus merguensis*). The purpose of this research is to know the level of water toxicity in the

Wonorejo Mangrove Pond that is contaminated with heavy metals against cultivation biota in the pond.

2 RESEARCH METHOD

The method used in this research was an experimental method using milkfish (*Chanos chanos*), and local white prawns (*Penaeus Sp.*) as tested biota. Water from the Wonorejo Mangrove Pond and the Avour channel was used as a medium to create a test solution.

2.1 Time and Location

University Fishery Cultivation Laboratory, on May-June 2019 during the beginning of the dry season. Thus, the surface water discharge (run off) has begun to thinning and slightly diluted by rainwater. Location of the research was in the Wonorejo Mangrove Silvofishery Pond, by taking 3 points of observation station (Figure 1).

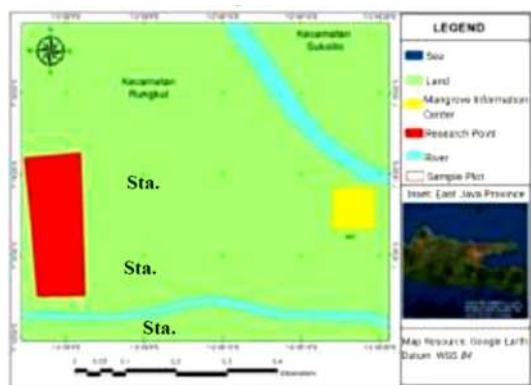


Figure 1: Location of Wonorejo Mangrove Pond.

Table 1: Tools and Materials Used during Research.

No	Tools and Material	Specification
1	Aquariums	sized 30 cm x 25 cm x 20 cm
2	Aerator	Out of gas 3L/min
3	Pond water	Clear water from silvofishery ponds
4	Sea water	Clear water from Wonorejo Beach
5	Cu solution	
6	Cd solution	
7	Pb Solution	
8	Milk fishes	<i>Chanos chanos</i>
9	<u>Prawns</u>	<i>Penaeus Sp.</i>

2.2 Research Plan

Job descriptions on this study include sampling, research preparation, toxic testing, processing and data analysis. The parameters of the heavy metals analyzed were Pb, Cd and Cu using the method of Atomic Absorption Spectrofometry (AAS). The tools and materials used in this research as shown in Table 1.

There were 12 aquariums (5 aquariums for 5 variations of concentration, 1 aquarium for control and experiments performed for 2 types of biota cultivation), 12 pieces of aerator (each aquarium was always given sufficient aeration. Each aquarium was filled with test water as much as 8 litres.

The experiment was conducted against 4-weeks-old milkfish (*C. chanos*) with 6-7 cm length, and local white shrimp (*Penaeus Sp.*) with 5-6 cm length and the age of them were approximately 2 months. Milkfish seedlings were obtained from fish breeding in Gresik, while white shrimps were obtained from the pond in Wonorejo area. Each aquarium was filled by 8 tested biotas.

The stage of the toxicity test implementation for cultivated biota was done by maintaining tested biota in the water from the Wonorejo Mangrove Silvofishery Pond with variations of concentration made. The treatment of this experiment was carried out with 6 variations of concentration in pond water dilution; 100% (8 litres of Silvofishery pond water), 75% (6 litres of pond water and 2 litres of seawater + enriched by Pb solution), 75% (6 litres of pond water and 2 litres of seawater + enriched by Cd solution), 75% (6 litres of pond water and 2 litres of seawater + enriched by Cu solution), 50% (4 litres of silvofishery pond water and 4 litres of seawater), and 25% (2 litres of silvofishery pond water and 6 litres of seawater).

The method used to determine the toxin level was determined by the amount of death of the tested biota, at the 24th hour and the 48th hour.

3 RESULTS

3.1 Distribution of Heavy Metals in the Waters of Wonorejo Silvofishery Pond

Results of laboratory analysis on the heavy metal contents of Pb, Cu, and Cd in the waters and the sediment of the Wonorejo Silvofishery Pond are presented in Figure 2.

From the results of water analysis in several locations in the Wonorejo pond, it is apparent that the

contents of three types of metals (Pb, Cd, and Cu) have metallic contents that exceed the quality standards of water for marine life when compared to the appendix III of the Ministry of Environment decree No. 51 year 2004. The results of measurement at the Wonorejo Pond are; the content of Pb is 0.304 ppm (NAB 0.008 ppm); Cd is 0.047 ppm (NAB 0.008 ppm), and Cu is 0.024 ppm, (NAB 0.001 ppm). NAB is the quality default threshold value. The average content of all three types of metals can be seen in Table 2.

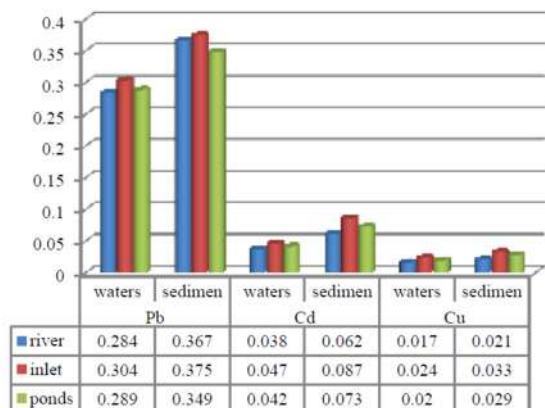


Figure 2: Distribution of heavy metals (Pb, Cd, and Cu) in Wonorejo Pond.

When compared between metal types, Pb has higher value which is 0.2923 in water samples and 0.3637 in sediment samples rather than Cd and Cu. While Cu is the lowest one than Pb and Cd whether in water and sediment samples. Metallic content on sediment samples shows the higher yield rather than on water samples.

Table 2: Average metals concentration at Wonorejo Mangrove.

		Avarage	Stdev	Treshold value of marine biota
Lead (Pb)	water	0.2923	0.0104	0.008
	sediment	0.3637	0.0133	
Cadmium (Cd)	water	0.0423	0.0045	0.001
	sediment	0.074	0.0125	
Copper (Cu)	water	0.0203	0.0035	0.001
	sediment	0.0277	0.0061	

The metal content of the Wonorejo mangrove on all stations shows that those have exceeded the quality default threshold of marine-life living needs

according to appendix III of the Ministry of Environment decree No. 51 year 2004 on the quality default of seawater for marine biota. The content of Pb (lead) has been 36 times compared to standardized quality standards. Cd (Cadmium) has even reached 42 times.

3.2 Influence of Heavy Metals against the Mortality of Cultivated Animals

Pond water with the condition of the metal concentration as presented above was then used to test the toxicity of the cultivated milkfish and shrimp. Six variations of concentration in silvofishery pond water dilution for test water is as presented in Table 3.

Table 3: Pond water quality used as test water.

Treatment	Dilution concentration	pH	DO	Temp.
		(ppm) °C		
B1	25%	7,6	7,9	26,4
B2	25%	7,7	8,0	26,1
C1	50%	7,6	7,6	25,9
C2	50%	7,7	8	25,9
APb 1	75% + Pb	7,6	7,6	25,6
APb 2	75% + Pb	7,5	7,3	25,5
ACd 1	75% + Cd	7,6	7,3	25,4
ACd 2	75% + Cd	7,6	7,8	25,1
ACu 1	75% + Cu	7,8	7,7	25
ACu 2	75% + Cu	8,1	7,6	25
K1	100%	7,7	8,2	26,5
K2	100%	7,8	7,6	26,6

Expl:

1 = milkfish
2 = white prawn
B = dilution 25%
C = dilution 50%
A = dilution 75%
K = dilution 100%

From the results of the water pond testing for the milkfish and shrimp protection, it is known that milkfish have not been found dead up to the 48th hour of observation time, on all concentrations of tested metals. While at the 24th hour observation time of shrimps, almost all prawns have been found 100% dead, except those on controls and Cd enrichment which are being dead yet. But at the 48th hour of observation time, all shrimp biota were found 100% dead.

Based on the results of this observation, it is seen that the milkfish have a higher tolerance ability to the

polluted waters rather than shrimps. The results of this observation are in accordance with the conditions in the silvofishery cultivation field in Wonorejo Mangrove. Pond farmer also stated that they often failed to harvest shrimp, while they succeeded to harvest milkfish sometimes.

3.3 Metal Content on Tested Biota

Tested biota which succeeded to live until the 5th day of subsequent testing were brought to metal level testing in its flesh. In this research, because the prawns were all dead at the 24th and the 48th hour, then the test of metal content was only done on milkfish. The test results of metal levels (Pb, Cd, and Cu) in milkfish are presented in Figure 3.

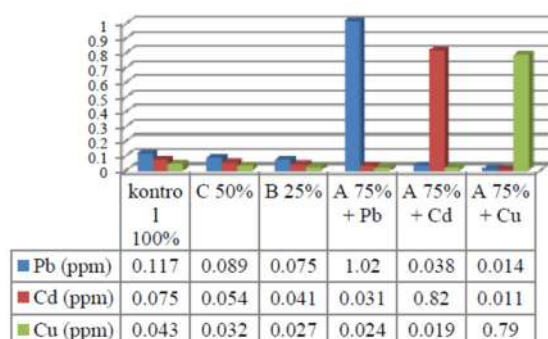


Figure 3: Graph of heavy metal content in milkfish meat in test water with different concentrations.

The test result shows that the accumulation of metal in the milkfish meat will increase as the increase of metal concentration in the test water. Even in the metal-enriched test water, the accumulation of heavy metals in milkfish meat has a quite high increase.

On concentration of test water without heavy metal enrichment, the highest metal level in milkfish meat is the lead metal type (Pb), following by Cadmium metal (Cd) and copper (Cu). The Pb metal content of milkfish that is maintained on the test water (100% of the water pond with concentration of Pb 0,292 ppm) is 0.117 ppm.

3.4 Metal Contain on Milkfish Flesh in Wonorejo Ponds

Wonorejo ponds were found that some biotas in there like milkfish, crabs, etc containing heavy metals. It's because the ponds itself got fresh water supplied by Avour river which is along of this river there are many industries and factories that suspected pollute this river by its waste. It's proven by this Table 4.

Based on the table above, we could see that Pb contain in all of the point research was bigger than the others. It's because so many activities industry are using Pb as their material. Automatically, the waste of their industry contain heavy metals like Pb. This waste suspected pollute the Avour river which is the main of supplier fresh water for tradisional ponds and silvofishery ponds in its around. Basically, this metal was accumulated into the body of aquaculture animals there especially the milkfish itself.

The research of Napitu (2010) show that the highest metal concentrations of Pb in the estuary of Blanakan River at 0.1445 ppm, while the Pb accumulation in milkfish amounted to 0.1962 ppm. Pb metal accumulation in milkfish body is higher than the concentration in the water. Pb accumulation in milkfish that cultivated in Blanakan Subang pond reaches 6,6 µg/kg, Heriyanto (2011).

Meanwhile in this study, the accumulation of Pb metal in Milkfish was still under the concentration of the Pb metal in the test water, allegedly because the time was only briefly exposed that for two days.

The content of Hg in Milkfish that live in ordinary ponds contains Hg 49 times higher than in mangrove forests and for shrimps can reach as high as twice Gunawan (2008).

The content of Cu is the lowest in mangrove of Wonorejo, however Cu is an essential metal for aquatic animals that is beneficial in the formation of Haemosianin blood system and in enzymatic aquatic animals (Darmono 2001). Metal Cu needed marine organisms as enzyme cofactors for the growth and development of his life. But if the amount of Cu in the body is excess, it will turn into toxins for the body (Palar, 2004).

Table 4: Metal contain on milkfish flesh.

	Pb (ppm)	Cd (ppm)	Cu (ppm)
Pond 1	0.181	0.019	0.0091
Pond 2	0.189	0.029	0.0095
Silvofishery Pond 1	0.114	0.015	0.0052
Silvofishery Pond 2	0.125	0.014	0.0072

When compared with the regulation of the Agency for Drug Control and Food No. 5 year 2018 about the maximum limit of heavy metal contamination in processed food, it is said that Pb metal is 0.20 mg/kg and Cd metal is 0.10 mg/kg. Then, Pb content in milkfish that live in the Mangrove pond water is still under the maximum threshold. Or it can be said that it is still safe to consume.

Similarly, the content of cadmium (Cd) and copper (Cu) on milkfish flesh reaches the highest of 0.075 and 0.043 respectively, which is still below the maximum limit of Cd and Cu for food. So, it is still safe to consume.

In milkfish that are kept in pond water enriched with Pb, Cd, and Cu with a concentration of 0.25 ppm, it indicates the presence of higher metal concentrations in milkfish meat; Pb reaches 1.02 ppm; Cd reaches 0.82 ppm; and Cu reaches 0.79 ppm. This concentration is still below the maximum limit for food as well. However, if food containing metals with such content is consumed continuously, it should be asked whether it has an impact on health or not. Surely, it requires further research.

4 CONCLUSIONS

The type of metals that pollute the water of the Wonorejo Mangrove Pond is the lead metal (Pb) with its highest concentration of 0.304 ppm, Cd metal with its highest one of 0.047 ppm, and Cu metal with its highest of 0.024 ppm. The metal concentration in sediment is higher than in the water pond. Pb, Cd and Cu metal pollutants in this concentration does not affect to the milkfish, as long as other environmental parameters are appropriate. But it caused more than 50% of shrimp deaths in the 24th hour.

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Dynamic Behavior Analysis of Porous Saw Floating Breakwater under Regular Waves

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Keywords: Floating Breakwater, Porous, Motion, RAO.

Abstract: Porous saw floating breakwaters are floated and tethered breakwaters for coastal protectors with typical steep, deep, and relatively large choppy contours, designed with saw porous, to be able to absorb waves efficiently and effectively. A proposed numerical model was developed for the porous saw type, while the model for the pontoon and the saw type will be used as a comparative study. All types of the structure breakwaters were physically placed inside the flume tank in which the direction of waves was perpendicular (to the structure) with 3 cm height and 1.1 seconds of period conducted in 41 cm depth of water. The numerical model shows that the Response Amplitude Operators (RAO) only affect the sway, heave, and roll motions. The porous saw floating breakwater demonstrates the smallest RAO value among other types i.e. sway 0.66 cm/cm, heave 2.1 cm/cm, and roll 0.86 deg/cm. It means the porous saw type more advantages since the ability to absorb the wave. Moreover, some of the wave energy will be reduced when the wave passes through the structure. Therefore, the wave energy received by the structure and the mooring rope becomes smaller.

1 INTRODUCTION

In recent years, many sectors attempt to manage and utilize the Indonesian coastal region. Such as industry, trade, transportation, housing, and the tourism sector. As population growth and increasing socio-economic development activities, the "value" of the coastal region continues to grow. Coastal areas, despite their high economic value, are vulnerable to many threats. One serious threat encountered is abrasion, causes a reduction in coastal areas. It may happen because of the large amount of wave energy comes directly to the coast without any absorption beforehand.

Preventing the negative effects caused by abrasion, it is very important to build and install the coastal protection structures to reduce wave energy towards the coastal area. One of the coastal protection structures that protect from abrasion and erosion is a breakwater, a structure was built to protect the area behind it from the wave attacks. There are two types of breakwaters, namely fixed breakwater and floating breakwater, both are built depending on the normal water level elevation and tidal conditions in which the structure is placed. Floating breakwater offers the level of protection needed when working in deeper

waters with exposure to natural resources that are stronger than conventional types of the breakwater. Floating breakwater applying the concepts of reflection, dissipation, and transformation to reduce wave energy so that it can weaken the up-coming wave to an acceptable level (Morey, 1998). This breakwater is a floating structure on a limited draft and depends on the interaction of building structures at the top of the water column. Moreover, another advantage of floating breakwater against the fixed breakwater, the efficiency does almost not depend on the tides and sea-level rise, then the impact to the environment is low, the cost of structure construction, installation and removal are low, short time for installation, and ability to reset the layout if there are changes in the future (Ruol et al. , 2012). However, as a floated structure, then a mooring system placed on the seabed (Fousert, 2006).

In general, floating breakwaters reduce the surface waves through the mechanism of reflection, destroying the movement of water particles and attenuation of viscosity. When the waves hit the structure, energy will be reflected, scattered, and partially muted by the structure. The greater the wave energy absorbed, the higher the intensity of the following structure's motion. Since most of the wave energy will be converted into motion energy. The

amount of energy absorbed is largely determined by the magnitude of the cross-sectional area of the structure relative to the perpendicular direction of the incident wave. Therefore, a floating breakwater design with minimum cross-section and interaction effect is an important reference for producing designs with minimum responsiveness. Such conditions can be achieved with porous surface conditions.

Based on the concept of maximum energy dissipation due to reflections and minimum energy trajectories that pass through the structure, the floating breakwater research continues to develop to find better performance. The number of researchers has conducted a study on the floating breakwater with porous types, namely porous pontoon (Lee & Ker, 1997; Williams & Li, 1998; Wang & Sun, 2010), porous boxes (Stainissie & Drimer, 2003; Koutandos & Prinos, 2011; Zeng et al., 2018), porous plates (Chwang & Dong, 1984; Wang & Ren, 1993; Cho, 2016; Fang et al., 2018), porous cylinder (William et al., 2000; Zhao et al., 2010; Shih, 2012).

This research will study the porous floating breakwater. This type of structure also reduces extreme wave loads which can affect the performance of floating structures. In an impermeable form, the structure will receive the maximum wave force so that it will transfer the wave energy to the mooring rope. This undesirable condition since mooring ropes will receive large loads so that the structure remains stable receiving wave force. The response of structural motions as a result of wave loads will affect the performance of the structure, i.e. reflection and transmission of waves. Therefore, during the design stage, it is necessary to predict the structure's motion to produce a small wave attenuation. Numerical simulations will be carried out to determine the characteristics of the 6 degrees of freedom structure motions using the Ansys Aqwa Module.

2 METHOD OF RESEARCH

2.1 Floating Breakwater Design

The material used in the floating breakwater model is High-Density Polyethylene (HDPE) with a density of 960 kg/m^3 . Figure 1 shows the pontoon, porous saw, and saw type floating breakwater have the same dimensions (length 20 cm, height 6 cm and width 6 cm). The types are slightly different, the porous saw type has a pore, which is penetrated the structure from the front to the back and there is also a triangle shape lined up in the front of the structure.

In the porous saw type, the pipes are used so that

the front and rear sides of the breakwater are perforated. The pipe used has an outside diameter of 0.4 cm. These three breakwater types above are floats with a 4 cm draft. The pore in the porous saw type floating breakwater is take up to 5% area of the overall area in the side of the structure. The difference between these three models requires the thickness of the porous saw type floating breakwater to be 0.36 cm to keep drafts are the same. The reduction was caused by the loss of buoyant force due to a hole in the floating breakwater.

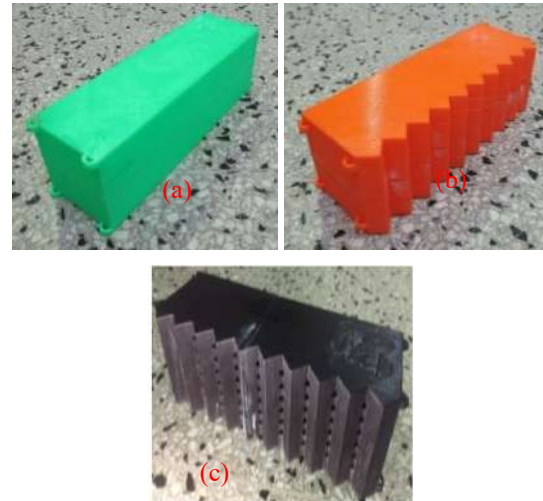


Figure 1: Floating breakwater (a) type pontoon, (b) saw and (c) porous saw.

2.2 Hydrostatic Parameters

A series of floating breakwater models arranged in 3 directions longitudinal to the width of the flume tank. The hydrostatic parameters in this numerical simulation are used as a basis for measuring the model validation. Based on the dimensions of the floating breakwater model, it can be determined the volume of the types of a pontoon, saw, and porous saw respectively 2266.58 cm^3 , 2487.5 cm^3 , and 2274 cm^3 . The mass of the floating breakwater can be determined by multiplying the volume by the density of HDPE so that the mass of the floating breakwater type of pontoon, saw, and porous saw is 1457 gr, 1592 gr, and 1535 gr respectively. The third dimension of this structure is designed to have a draft of 4 cm.

The initial step before developing a numerical model is calculating the center of gravity for each structure. The value of the gravity center in the floating structure is very important to determine the stability of the floating breakwater. The center of gravity calculation is done by dividing floating

breakwater into several parts. Then, determining the value of inertia and the radius of structure gyration. Complete data on hydrostatic parameters are shown in Table 1.

Table 1: Hydrostatic data floating breakwater.

Hydrostatic Parameters	Pontoon	Saw	Porous Saw
Vol. Displ. (cm ³)	2266.6	2487.5	2274
Massa (gr)	1457	1592	1535
Ixx	3822	4498.9	4387.6
Iyy	40812	44553.75	44546.85
Izz	43197	47276.3	47253.4
Kxx	1.62	1.68	1.64
Kyy	5.29	5.29	5.29
Kzz	5.45	5.45	5.45
WPA (cm ²)	360	372	372

2.3 Validation of the Hydrostatic Model

Numerical simulations to determine the response of floating breakwater motion are performed by computational fluid dynamics with the ANSYS/AQWA hydrodynamic simulation software. Figure 2 shows the layout of the model scenario with the direction of the incident wave perpendicular to the structure.

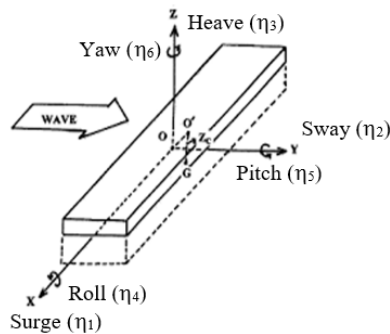


Figure 2: Degree of freedom of floating structures (Das & Das, 2005).

Before conducting the structural motion analysis, the floating breakwater model must be validated to find out the floating breakwater by the conditions in the analytical calculation. Important hydrostatic parameters that will be used for the validation of this model are the volume displacement and water plane area. Based on The American Bureau of Shipping (1998), the validation is a maximum of 2% and for other provisions a maximum value of 1%. The calibration results of the model are shown in Table 2 and 3.

Table 2: Validation of numerical models: volume displacement.

No	Floating Breakwater	Hydrostatic Parameters		
		Volume Displacement (cm ³)		
		Analytic	Numeric	Error %
1	Pontoon	1457	1457	0
2	Saw	1657	1657	0
3	Porous Saw	1663.79	1658.28	0.303

Table 3: Validation of numerical models: water plane area.

No	Floating Breakwater	Hydrostatic Parameters		
		Water Plane Area (cm ²)		
		Analytic	Numeric	Error %
1	Pontoon	360	360.01	0.0001
2	Saw	372	371.8	0.002
3	Porous Saw	372	371.8	0.002

3 RESULTS AND DISCUSSION

Numerical simulations of floating breakwater motion have been done at a depth of 41 cm, a wave period of 1.1 s and a wave height of 3 cm with the direction of the wave perpendicular to the structure. The results of numerical simulations are shown in figures 3-8.

The characteristic of surge motion in pontoons, saw and porous saw floating breakwater in the direction of waves perpendicular to the structure is very small and almost close to 0 (Figure 3). Saw type RAO is the biggest among other types because there is additional displacement in front of the structure. Generally in the surge motion, waves to the side do not have an effect on the occurrence of surge motions.

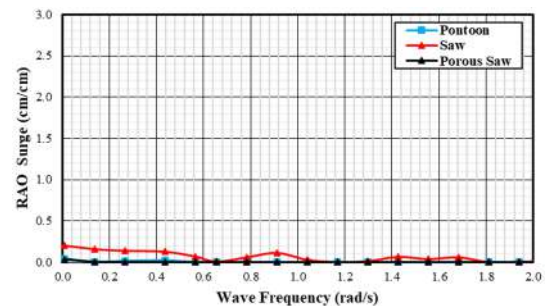


Figure 3: RAO Surge numerical prediction of various types of the floating breakwater.

Sway motion characteristics are almost the same as the surge motion where the maximum RAO value is at low frequencies (Figure 4). The highest RAO on the pontoon is 0.79 cm/cm. At the same frequency, the maximum RAO for saw-type floating breakwater is 0.98 cm/cm and porous saw-type RAO is 0.6

cm/cm. After that, the third RAO floating breakwater decreases gradually. Sway motion has a higher maximum value when compared to surge for side waves because side wave propagation has a great influence on sway motion.

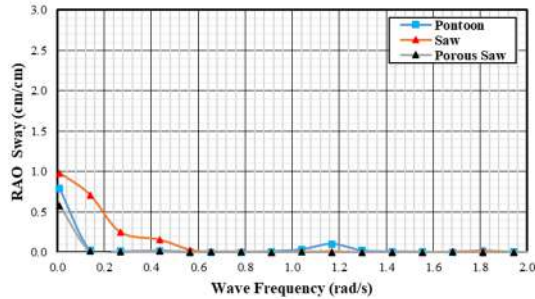


Figure 4: RAO Sway numerical prediction of various types of the floating breakwater.

Figure 5 shows the RAO heave motion is an example to explain dynamic systems that experience wave excitation. At low frequencies, the three RAO types of floating breakwater are 1.11 cm/cm, 1.08 cm/cm, and 1.13 cm/cm and gradually increase towards the resonant region at the natural frequency of 0.436 rad/s. the largest maximum RAO heave occurs in the saw type followed by porous pontoon and gergagi types respectively 2.85 cm / cm, 2.45 cm/cm, and 2.1 cm/cm. After crossing the peak, the RAO heave will decrease dramatically at high frequencies.

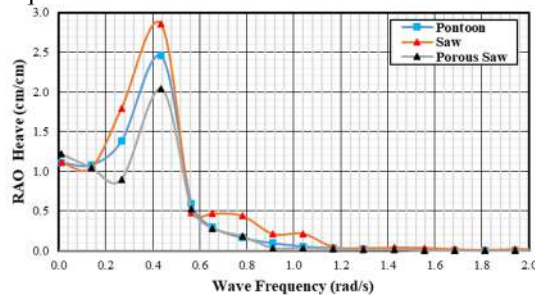


Figure 5: RAO Heave numerical prediction of various types of the floating breakwater.

Roll motion is almost the same as the characteristics of the heave motion (Figure 6). In the sub-critical region, the motion response represents the same condition in all types of the floating breakwater, then increases sharply in the resonant region and decreases significantly in the supercritical region. The natural frequency of the roll motion is 0.43 rad/s. The value of the highest response of roll motion on a saw is 11.86 deg/cm, followed by the pontoon type of 9.64 deg/cm, and the porous saw is 6.8 deg/cm.

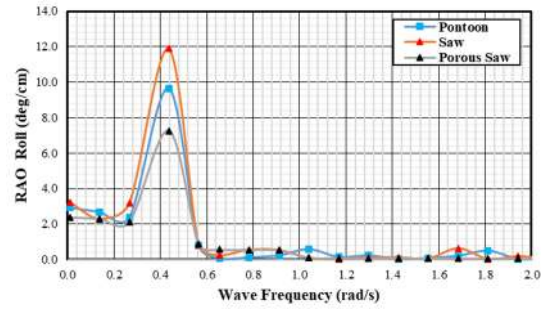


Figure 6: RAO Roll numerical prediction of various types of the floating breakwater.

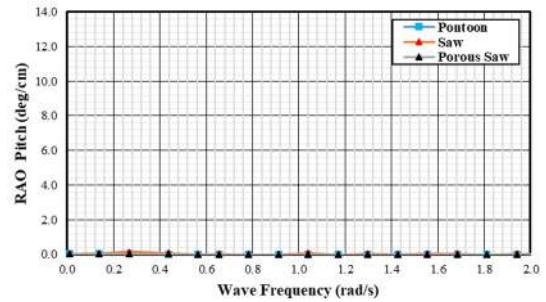


Figure 7: RAO Pitch numerical prediction of various types of the floating breakwater.

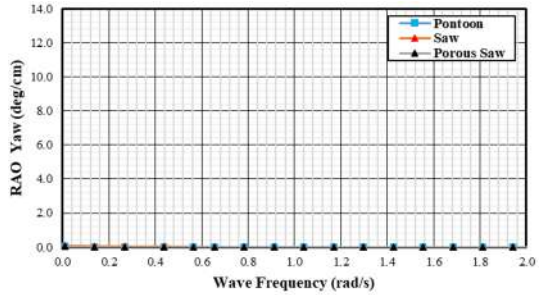


Figure 8: RAO Yaw numerical prediction of various types of the floating breakwater.

In pitch and yaw motions (Figure 7-8), it is not the same as roll motions which are both rotational mode motions. However, pitch and yaw motion intensity is not affected by side and bow waves, so that the value is 0 or close to 0 so there is almost no motions response.

Based on numerical simulations obtained floating breakwaters motion behavior. The condition of a free-floating structure without a mooring system will produce a vertical motion mode (heave, roll, and sway) which is more dominant than the horizontal motion mode (surge, pitch, yaw). This happens because all three modes of motion (vertical motion modes) have a stiffness factor due to harmonic wave excitation, the presence of this stiffness factor causes

the damping factor value to be small so that when the motion reaches its resonant frequency, the change in motion characteristics will have a sharply increased part. Whereas the horizontal motion mode which does not have a stiffness factor during free-floating conditions will produce a relatively large damping factor so that the motion will be damped by the presence of the damping factor and the horizontal motion mode does not experience a sharp increase. If there is an increase in certain parts, then the increase in the curve is influenced by the coupling effect of other motions.

6 CONCLUSIONS

This paper compares the dynamic behavior of a porous saw type floating breakwater with another types. Numerical studies were carried out with computation fluid dynamics on models of these structures at a water depth of 41 cm, wave height of 3 cm, and a wave period of 1.1 seconds. some findings may be explained as follows:

- The surge, pitch, and yaw motion modes are not affected by side waves so the RAO value is very small and almost close to 0.
- The floating breakwater motion in the direction of incoming waves perpendicular to the structure only affects the motion sway, heave, and roll.
- Sway motion has a higher maximum value when compared to surge for side waves because side wave propagation has a great influence on sway motion. At the same frequency, the maximum RAO for saw-type floating breakwater is 0.98 cm/cm, tipe pontoon is 0.79 cm/cm and porous saw-type RAO is 0.6 cm/cm. After that, the third RAO floating breakwater decreases gradually.
- The largest RAO maximum heave motion occurs in the saw type followed by the type of pontoon and porous saw by 2.85 cm/cm, 2.45 cm/cm, and 2.1 cm/cm respectively. After going through the peak, the RAO heave will decrease dramatically at high frequencies.
- The highest peak value of RAO roll occurs at natural frequency 0.43 rad / s, in floating breakwater type of saw, pontoon, and porous saw of 11.86 deg/cm, 9.64 deg/cm, and 8.6 deg/cm respectively.

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Two-dimensional Numerical Study of Scour beneath Subsea Pipeline under Regular Wave Condition

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Abstract: The interaction of pipe and soil under hydrodynamic flow becomes one of the concern in coastal engineering. Scoured seabed beneath sub-marine pipeline could develop the pipe instability and affect to its safety. The pressure difference between upstream and downstream of pipe will induce a seepage flow in the seabed underneath the pipe. When the velocity increased then the critical point reached, a mixture of sand and water divert to the gap below the pipe and leading to a large velocities in the gap. It generates a larger shear stress on the bed and increase the amount of sediment transport. Due to the complexity of the scour problem, there are many factors affecting the scour depth. Some of them are the pipe position and the wave angle of attack. This study evaluates the bed morphological evolution around pipeline, performed with variation of pipe positions and wave angles of attack.

1 INTRODUCTION

This research aims to investigate the bed evolution during scour process until it reaches a steady state (equilibrium stage). The stage reached when the bed shear stress along the bed below pipe becomes constant and equal to its undisturbed value. A computational fluid dynamics used to calculate flow and sediment transport subjected to regular wave and steady currents. The model performed the sediment erosion around pipeline that laid on erodible seabed, based on time dependent scour processes using Volume of Fluid (VoF) method with standard $k-\omega$ turbulence closure. Beside the bed deformation, the scour propagation rate discussed with respect of the pipe position.

Submarine pipelines have become a concern in marine and geotechnical engineering. The presence of pipeline though it lay out, buried or trenched in the seabed, it will change the marine environment. Wave and current action change the flow pattern when exposing the marine pipelines installed on seabed in coastal or subsea area. This change generally cause the increase of bed shear stress and the turbulence level. Both terms lead the sediment transport around pipe. Therefore interaction between pipeline, seabed,

and flow causes the occurrence of local scouring. The developing scour along the pipe will generate gap between seabed and pipe, this commonly referred to pipeline span which become a threat to the stability of the pipeline structures.

Wave-induced scour around pipelines occurs in three stages, they are onset scour, tunnel erosion, and lee wake erosion. Onset scour driven by the pressure difference between the upstream and downstream of the pipe. When the flow velocity exceeds the critical point, a mixture of water and sand break through underneath the pipe. The stage followed by tunnel erosion stage, during this stage a substantial amount of water is diverted to the gap and leading large velocities, this resulting a large shear stresses below the pipe and increasing the sediment transport. In this stage, the scour occurs extremely fast at the beginning. As a result a dune begins to form at the downstream side of pipe. However this dunes gradually migrates downstream and there will be more scour at the downstream side of the pipe than at the upstream side. This stages is lee-wake erosion (Sumer and Fredsoe, 2002).

Over decades researchers have studied the local scour around pipeline and proposed different empirical equations to describe the equilibrium scour depth, such as Mao (1986), Sumer and

Fredsoe (1990), Chiew (1991), Cevik and Yuksel (1999), Bakhtiary (2011), and Dogan (2018). While numerical methods have been developed to scour around pipeline as well as Brors (1999), Liang and Cheng (2004), Shen et al. (2013), Fuhrman et al. (2014), Shen et al. (2015), Zhang and Shi (2016).

Sumer and Fredsoe (1990) established scour below pipelines exposed by waves and found that lee-wake of the pipe is the key element in the scour process also Keulegan-Carpenter number is the main parameter that governs the equilibrium scour depth. The larger the KC-number, the larger the stream-wise extent of the area affected by the lee-wake during half period of the flow. Hansen (1992) investigated combined wave and current parameter on local scour and identified that velocity ratio (m) as the governing equation for scour in combined wave and current. Velocity ratio developed from composed U_c , flow velocity due to current, and U_w , wave orbital velocity. However numerical model predictions in developing combined wave and current condition are limited. Hansen (1992) and Larsen et al. (2016) presented modelling of the scour under combined wave and current, but the study is limited to the value of combined velocity and assume the fluid as a rigid lid not free surfaces area. Ahmad (2019) investigated pipeline scour under combined wave and current, the results showed the scour depth and horizontal extent of the scour increase with U_{cm} (combined velocity of current and wave) for a given KC number. While the experiments of Cheng et al. (2014) showed that the scour propagation under wave only condition increases with KC number and Shield parameter.

Based on the background above, the further discussion on pipeline scour under the combined action of wave and current still needed. Over more the occurrence of the actual scour beneath subsea pipeline occurs in the combined wave and current. This paper primarily concerned with numerical simulation of a single pipe that laid on sandy bed and subjected to combined wave and current loadings. The simulation was done under free surface dynamic analysis. The study discusses the generation of regular waves and steady current condition in the numerical wave tank. Wave only model and combined wave-current model validated with Airy wave theory, then the validated model used to validate pipeline scour.

2 NUMERICAL METHOD

In this study, three-dimensional wave hydrodynamics and sediment transport calculated by CFD numerical modelling.

2.1 Flow Model

The hydrodynamics module based on the continuity and motion equation. The motion equation of the fluid velocity components has been done in the three-dimensional Navier-Stokes equation which for incompressible flow. The equation of continuity written as follow.

$$\frac{\partial \rho}{\partial t} + \frac{\partial(\rho u)}{\partial x} + \frac{\partial(\rho v)}{\partial y} + \frac{\partial(\rho w)}{\partial z} = 0$$

The Navier-Stokes equations are as follows.

$$\begin{aligned} p \left(\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + w \frac{\partial u}{\partial z} \right) &= -\frac{\partial p}{\partial x} + \rho g_x + \mu \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} \right) \\ p \left(\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + w \frac{\partial v}{\partial z} \right) &= -\frac{\partial p}{\partial y} + \rho g_y + \mu \left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} + \frac{\partial^2 v}{\partial z^2} \right) \\ p \left(\frac{\partial w}{\partial t} + u \frac{\partial w}{\partial x} + v \frac{\partial w}{\partial y} + w \frac{\partial w}{\partial z} \right) &= -\frac{\partial p}{\partial z} + \rho g_z + \mu \left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} + \frac{\partial^2 w}{\partial z^2} \right) \end{aligned}$$

Where u , v , and z are the velocity components, p is the pressure, ρ is the fluid density, μ is the turbulent eddy viscosity, and g is the gravitational acceleration.

The wall boundary conditions evaluated based on turbulence closure. A turbulence model is required to calculate the turbulent viscosity μ . In this study, the turbulent viscosity calculated by the standard two-equation κ - ω of (Wilcox, 1988).

$$\begin{aligned} \frac{\partial k}{\partial t} &= \frac{\partial}{\partial z} \left\{ (v + v_t \sigma_{k\omega 1}) \frac{\partial k}{\partial z} \right\} + v_t \left(\frac{\partial u}{\partial z} \right)^2 - \beta^* \omega k \\ \frac{\partial \omega}{\partial t} &= \frac{\partial}{\partial z} \left\{ (v + v_t \sigma_{k\omega 1}) \frac{\partial \omega}{\partial z} \right\} + \gamma_1 \frac{\omega}{k} \left(\frac{\partial u}{\partial z} \right)^2 - \beta_1 \omega^2 \end{aligned}$$

The standard κ - ω closure has advantages to perform low Reynold number for near-wall turbulence. It is numerically stable in flows with adverse pressure gradient. So the advanced model of the standard closure continue to be developed, i.e. Menter (1994), Suntoyo et al. (2008), and Suntoyo and Tanaka (2009).

Using CFD model, the free surface (fluid-fluid interface) between water and air is calculated using

Eularian methods that characterized by meshes to evolve the shape of interface, i.e., Shen et al. (2013), Putri and Suntoyo (2020). VOF method proposed by Hirt and Nichols (1981) is used to track the free surface interface.

2.2 Sediment Transport Model

The physical mechanism of sediment in this paper defined as sediment scour model. It estimates the effect of the flow mechanism on the transport sediment and the erosion of surfaces. It estimates the sediment motion by approximating the sediment erosion, advection, and deposition. The numerical model estimates the sediment transportation by computing the suspended sediment transport, computing the sediment settling due to gravity, also computing the bed-load transport and motion.

Bed-load transport determined by Shield number, where bed shear stress (τ) is the important part which becomes the basis of sediment movement under flow model. Bed shear stress (τ) is determined by profiling logarithmic velocity profile near bottom.

$$\theta = \frac{\tau}{\rho g(s-1)d_{50}}$$

The sediment transport occurs when shield number exceeds the critical value. The Soulsby-Whitehouse equation used to predict the critical Shield number parameter.

$$\theta_{cr} = \frac{0.3}{1 + 1.2D_*} + 0.055[1 - \exp(-0.02D_*)]$$

Where D_* is the non-dimensional grain size and can be found by $D_* = \left[\frac{g(s-1)}{\nu} \right]^{1/3}$. According the the critical Shield number parameter, the bed-load transport rate equation calculated by Van Rijn (1984) as follow.

$$\frac{q_b}{[(s-1)g]^{0.5} D_{50}^{1.5}} = 0.053 \frac{T^{2.1}}{D_*^{0.3}}$$

3 MODEL SETUP

The scheme of this numerical simulation based on the experimental input from Suntoyo (2017). It conducted a series of experiments to study the two dimensional scour phenomenon due to combined wave and current flow over cylindrical pipe that laid on the erodible bed. This study developed the investigation to the two dimensional scour analysis. An open top flume with a hydraulically smooth pipe

made brought to the model with diameter 3,81cm of pipe and placed over the sand bed which has median grain size diameter (d_{50}) 0,55 mm. The model simulated by the regular wave flow. Wave only simulation and combined wave and current given as the variation to define the difference.

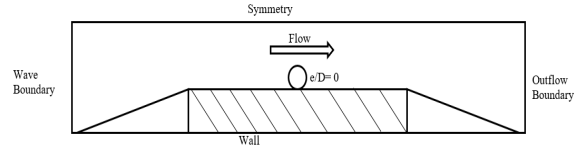


Figure 1: Domain and Boundary Scheme.

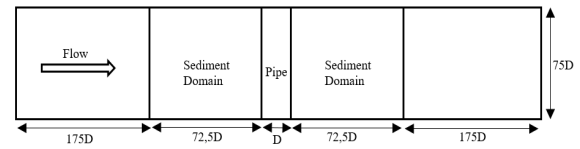


Figure 2: Top View Sketch of Computational Domain.

The governing equation of flow simulation discretized by finite difference method. A mesh of fixed rectangular cells used to subdivide the flow region, with each cells have local average values of all dependent variables.

Good meshing quality of computational domain put up the better understanding convergence of the numerical model so the better accuracy will resulted. Figure 1 (upside) illustrates the perspective view of the 3D mesh on the seabed and pipe. Figure 2 (downside) is the cross sectional view of the mesh of the model. Three different mesh sizes adopted to define flow, sediment, and pipe. Considering the flow behaviour near bed and pipe, finer mesh blocks used in respect where the velocity boundary layer near bed develop and the erosion around pipe expected. Smooth grid mesh applied near sediment around the pipe, this made the finer convergence and accuracy which dealing of the contacts between pipe, sediment and flow. The computational domain contain 7,5 million cells of mesh which define the boundary condition that aimed to get best estimation of the flow characteristic around seabed.

The numerical models were set up with assumption of hydrostatic pressure distribution from the inlet to the outlet boundaries. The top boundary of the domain is symmetry, where the air pressure assumed does not affect the numerical wave tank. At the side surfaces, the wall boundaries used with no-slip velocity condition. The inlet is set to be regular wave condition with wave height and angle variation. The outlet set to be outflow boundary with allowing flow to enter at outflow boundary condition.

Table 1: Input Data.

Module	Parameter	Notation	Magnitude	Unit
Wave	Depth	d	0.75	m
	Height	H	13	cm
	Period	T	1	s
	Angle	α	0; 45; 60	deg
Current	Velocity	v	0; 1; 1.3	cm/s

4 RESULTS AND DISCUSSIONS

4.1 Model Verification

This section discuss the validation of the proposed numerical model to predict wave propagation and scour induced wave to understand the accuracy f the model. The numerical results of the surface elevation compared with theoretical solution. The validation consists of the linear wave propagation using linear Airy wave consideration.

$$\eta = -\frac{H}{2} \cos(kx - \omega t)$$

Where H is the water depth, k is the wave number that derived from $2\pi/L$, and ω is wave frequency from $2\pi/T$. The formula shows that the water level fluctuations are periodic in respect to x and t. It shows a sinusoidal and progressive wave that propagate to the positive x direction.

Based on Liu et al. (2016) to compare the numerical result of the wave propagation with the theoretical solution that formed from potential flow theory, the viscosity is set to zero. To compare the numerical results of the wave propagation with theoretical solution, one case with inviscid fluid simulated to examine the mesh resolution in the computation. In simulation with inviscid fluid, to assume that the fluid is in ideal fluid mode so the viscosity and turbulent boundary ignored and set to zero. In contrast to the ideal fluid, the real fluid which has viscosity and turbulent boundary also proposed. The viscous fluid will be used as reference fluid in this study. Then the results of Airy wave theory, simulation in inviscid fluid, and simulation in viscous fluid are compared. Figure 3 shows the comparisons of numerical result of the time series of free surface elevation and the theoretical solution. The comparison of free surface elevation monitored at cross section $x=75D$ of time series $t=748s-752s$.

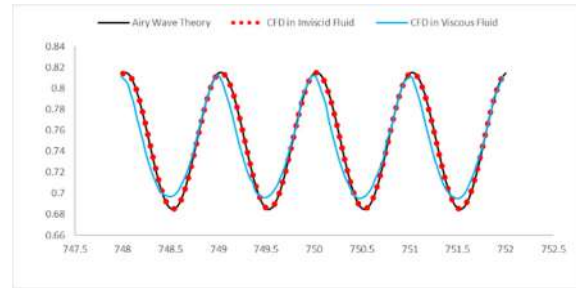


Figure 3: Comparison between Airy Formula and CFD Simulation in Wave only Condition.

The comparison graph of the simulation results show the surface elevation between airy formula and the CFD simulation results. The simulation of wave under ideal fluid mode, where viscosity and turbulent are set to be zero, shows the similar trend-line with empirical Airy Formula. The comparison of viscous fluid that simulated under viscosity and turbulent mode also shows a good agreement. The error results from both results comparison is 0,3%.

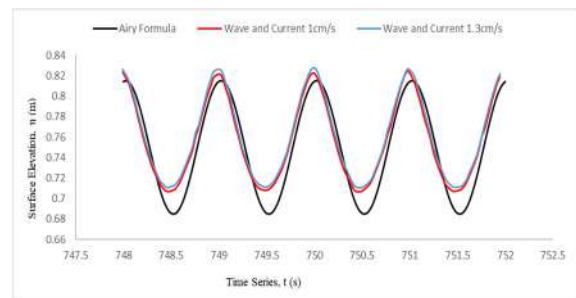


Figure 4: Comparison between Airy Formula and CFD Simulation in Wave and Current Condition.

The trend of simulation study on its surface elevation shows a slight shift to the empirical Airy trend-line. The comparison between theory and results of wave and current simulation shows that the CFD simulation results under wave and current have the upward trend shift compared to wave only simulation and theory. The greater of the current value that given, the more it leads to a shift in surface elevation.

4.2 Scour around Pipe

Calculation divided in to three parts based on the wave angle of response, 0° , 45° , and 60° . The three sections are carried out on a pipe laying on the seabed with $e/D=0$. Calculations performed under wave only condition and combined wave and current of 1cm/s and 1,3cm/s. Predicted time of equilibrium scour depth and scour deposition is about 25 minutes, this

is based on the experiments of Cheng et al. (2009) and Suntoyo et al. (2017). Maximum scour depth and deposition after 25 minutes simulation obtained. However, the calculation results in CFD for stages of scour development not as expected. The equilibrium scour depth and deposition not reached yet at the expected given time.

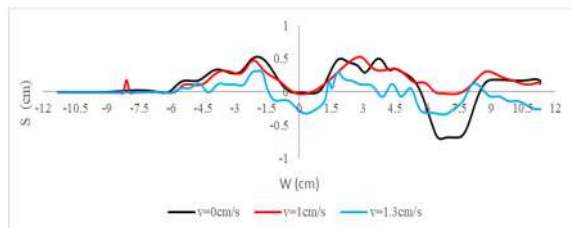


Figure 5: Bed Profile after 25 Minutes on $\alpha=0^\circ$.

Although the results have not shown the equilibrium condition at the expected given time (25 minutes), but the results show good agreement for how scour around pipe formed. The CFD calculation show the deformation movements of seabed which in accordance with Sumer et al. (2002) research. The simulation results in wave only condition shows the result of less sediment erosion beneath the pipe compared to the combination of wave and current results. However for the wave only simulation, the sand dune around pipeline that deposited is higher than the combined results.

In Figure 5, it can be seen if the flow still carry out sediment downstream the pipe, so that erosion occurs around the downstream of the pipe. Based on Sumer et al. (2002) wake pattern in oscillatory motion in wave are govern by Keulegan-Carpenter (KC) number. Larger KC number means that the water particles travel quite large distances, and resulting in separation and probably vortex shedding. The statement is compatible with the calculation above. In wave only condition, the generated erosion at the downstream pipe is greater than the combined conditions, despite the scour depth in the combined of wave and current simulations show the deeper results.

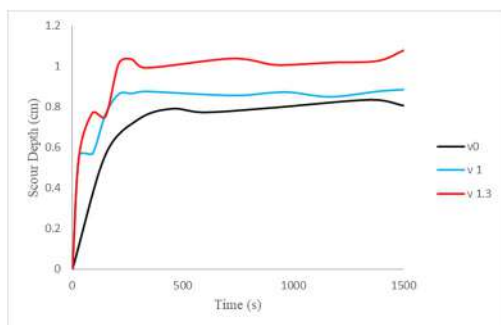


Figure 6: Typical Time-Dependent Scour on $\alpha=45^\circ$.

Typical time-dependent scour on figure 6 represents that current velocity magnitude influence the scour depth. It is clear if the greater current velocity magnitude, the deeper and wider scour produced. Different wave angle which lead to the pipe also have an influence on the width and depth of scour. The difference is quite significant. But it cannot yet be concluded whether the smaller the angle of wave attack will reduce the scour depth.

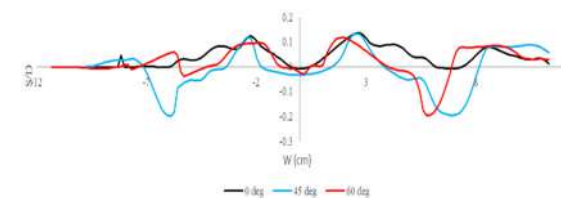


Figure 7: Bed Profile under Combined Wave and Current $v = 1$ cm/s in Different Angle of Attack.

CFD simulation on different wave angle of attack indicates that the scour depth and width trend-line of 45° wave angle has the widest and deepest scour profile. However, although wave angle 60° does not have the deepest and widest trend, the simulation results show that there is a slight difference in depth between angles 45° and 60° . Where angle 60° has the deepest scour depth compared to the other results.

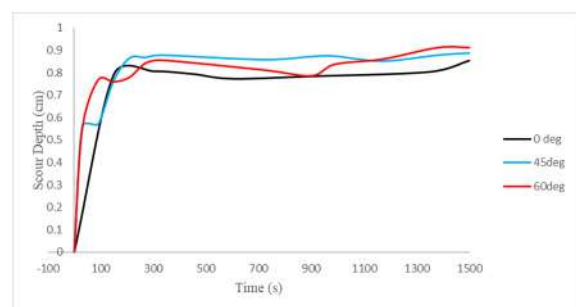


Figure 8: Typical Time-Dependent Scour in Different Wave Angle.

The bed profile deformation during scour shows if the initial stage of scour started from onset scour then followed by tunnel erosion. Initial flow that hits the pipe lead pressure difference between upstream and downstream the pipe and caused piping. Later on the gap between pipe and the bed formed as shown in Figure 9 ($t=30$ s). The initial gap continuously exposed to flow, and the gap evolves during the time. As seen in Figure 9, the scour occurs faster at the beginning ($t=0$ s until $t=180$ s).

Just as described in Lee-Wake erosion process, a dune begins to form and gradually resettle to the reverse side and deposited as the scour progressed. In this stage scour hole formed and the distance between

pipe and the bed seen as a gap in two-dimensional scour approach. If the scour hole traced and observed in three-dimensional view, it will be a free span between pipe and the bed that can disturb the stability of the pipe.

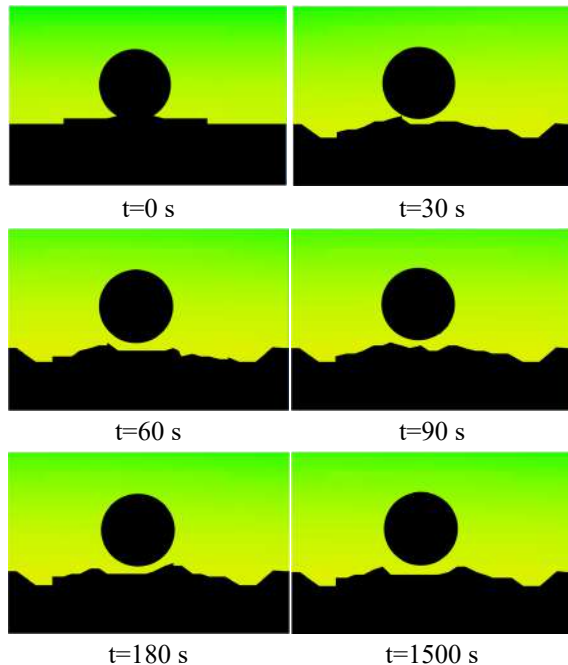


Figure 9: Bed Profile Deformation during Scour Process.

5 CONCLUSIONS

1. It should be noted that viscosity and turbulence affect flow condition. Verifying flow between CFD and empirical formula, should be done by set both conditions as zero.
2. The angle of attack is one of influencing factors of scour depth. The scour depth is reduced by decreasing of wave angle of attack.

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Effect of Cigarette Butts Extraction on the Corrosion Rate of ASTM A36 for Jacket Platform

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Keywords: Jacket Platform, Inhibitor, Cigarette Butts, Nicotine.

Abstract: Jacket platform is one type of offshore building that used for the offshore oil exploitation process. In general, the Jacket is designed with an operating life of 20 to 25 years and in operation must be guaranteed the safety and strength of the Jacket structure. One of the safety factors that must be considered is corrosion growth in all parts of the Jacket structure. To overcome corrosion problems, there is a need for alternative corrosion improvements, namely the prevention of corrosion by using corrosion inhibitors. The organic inhibitor that can be used as corrosion inhibitors, one of which is cigarette butts, because it contains a lot of nicotine. Nicotine can be used as an inhibitor by donating nitrogen atoms to nicotine to Fe^{2+} atoms so that complex compounds $[\text{Fe}(\text{NH}_3)_6]^{2+}$ are formed. This compound has a higher stability than Fe so that it can be used as protection in corrosion. The purpose of this research is to solve the cigarette butts problem by creating it a corrosion inhibitor to minimize the cost of material replacement, maintenance costs, and over-design. This study is an experiment by using ASTM 36 steel, material will be blast cleaned in advance with abrasive material Steel Grid. Second steps in this study is the coating process by mixing epoxy paint and extraction of cigarette butts, then immersed in corrosive medium namely sea water of Kenjeran. Corrosion rate testing is carried out by using the weight loss method. The highest corrosion rate occurred at 1% mixing with a corrosion rate of 24.61 mpy and the lowest corrosion rate at 2% mixing with a corrosion rate of 13.78 mpy Based on this research it can be concluded that the nicotine that can be used as cathodic protection on coatings.

1 INTRODUCTION

A Jacket Platform is a very common welded tubular space frame with three or more near vertical tubular chord legs with a bracing system between the legs. These platforms are virtue of their immobility, designed for long term use. The platform exhibits a low natural period and deflection against environmental loads (Samanta, 2016). In the generally Jacket is made of steel material because it is designed with the operating life for 20 years. One of the factors that can lower the strength of the jacket structure is the occurrence of corrosion (Ardianto, 2017).

The major failure mode of oil and gas structures is corrosion damage (Yasseri, 2017). The process of corrosion in the jacket platform occurs with high intensity because almost any aqueous environment can promote corrosion, which occurs under numerous complex conditions in oil and gas production, processing, and pipeline systems (Champ-Tech,

2012). Corrosion in the jacket platform can reduce the operating life of the jacket, this incident caused the oil and gas companies in Indonesia suffered disadvantages due to corrosion in the offshore building reach to hundreds of millions of dollars are equivalent to 2-5% of a country's Gross Domestic Product. Therefore, it is necessary to reduce the corrosion level of the jacket platform (Harjanto, 2014).

One of the most promising methods to improve the protective ability of coatings is the use of corrosion inhibitors (MCI) (Golovin, 2019). Inhibitors had been shown to be one of the major tools for tackling corrosion in the oil and gas industries. (Popoola, 2013). Microencapsulation technology greatly simplifies the process of compounding the polymer protective coating and allows to isolate the active components from the reactive groups of polymer resins and hardener at the stage of curing, correctly form the polymer base of the coating and avoid the negative effect of the inhibitor on the

adhesion of the coating to the protected substrate (Golovin, 2019). An Inhibitor with an effective corrosion reduction performance is generally an organic compound consisting of atoms N, O, P, and S (Hatch, 1984).

One of the groups of organic compounds containing such components is nicotine, on the other hand hazardous substances have been identified in cigarette butts – including arsenic, lead, nicotine and ethyl phenol (Eriksen, 2013). Nicotine is widely contained in tea, coffee, tobacco and cigarette butts, but the component of nicotine on cigarette butts more than coffee and tea is equal to 3,43% (Rodgman, 2006). Nicotine can be used as an inhibitor by donating the nitrogen atom in the nicotine to the atom Fe^{2+} so that the compound is formed $[\text{Fe}(\text{NH}_3)_6]^{2+}$. This compound has a higher stability than the Fe so that it can be used as a protection in corrosion (Haryono, 2010).

In general, tobacco is the main ingredient of cigarette composition, but when cigarettes turn into cigarette butts, the cigarette butts Filter absorbs chemical content in cigarettes as much as 80% (Novotny, 2009). This is caused by paper and cellulose acetate, that collects chemicals that are produced by smoking. (Novotny, 2014). This plastic component of filtered cigarettes may not degrade in the environment for many years (Bonanomi, 2015). There are numerous advantages of using tobacco extract as a metallic corrosion inhibitor is a natural, renewable, environmentally benign, and relatively inexpensive source. The active constituents in tobacco can be commercially extracted in a simple operation using only water as an extraction medium (Fouda, 2014).

Almost six-trillion cigarettes are produced globally each year, with approximately one-third to two-thirds of those cigarette butts being possibly deposited in the environment and ending up in parks, beaches, streets, and communities (Granados, 2019). According to the World Health Agency (WHO) survey about 36.3% of the population of Indonesia is an active smoker and Indonesia produces at least 52 million sticks of cigarette butts waste every year. Therefore, cigarette butts are potential to be used as an alternative to corrosion inhibitor material and can tackle cigarettes butt garbage in Indonesia. Previous research stated that the use of tobacco as a corrosion inhibitor on spikes still have a 23% lower corrosion rate reduction compared to using cigarette butts waste with an average value of corrosion rate of 0.16 mpy (Andeka, 2015). The greater the addition of cigarette butts extract then the higher the rate of the corrections. Therefore, this research needs to be done

to assess the utilization of waste cigarette butts as a corrosion inhibitor material on the platform jacket.

The purpose of this research is to determine the parameters that affect the use of cigarette butts as coating material and to know the reliability of coating coatings from the cigarette butts. There are also benefits achieved from this research the first is to utilize waste cigarette butts so as to reduce the amount of waste in Indonesia and for the protection of oil rig building so as to reduce the accommodation and maintenance cost. As for the potential results that can be created from this work is the scientific article. The second is a reference for scientists/societies/institutions.

2 METHOD

2.1 Time and Place of Execution

The research began on April 1, 2019 until July 13, 2019. The first stage author do is to find cigarette butts in the surrounding area Faculty of Marine Technology, ITS. Then proceed with the purchase of specimen and perform various stages of testing in the laboratory environment of ITS Metallurgical Material Department and in CV Cipta Agung.

2.2 Tools and Materials

The equipment used in this research was spectrophotometer (wet and dry thermometer), evaporator, dry abrasive blast cleaning, WFT measuring instrument (Wet Film Comb), DFT measuring device (Dry Film Thickness), (pull of test) power test equipment, FTIR (Fourier Transform (Infrared)), stereo microscope, and optical microscope.

The materials used were methanol, cigarette butts, tobacco, original powdered tea, paint jotun penguard Primer grey component A and B, thinner Jotun No. 17, Kenjeran sea water, sandpaper No. 150, glue standard setting epoxy adhesive, and ASTM A36 steel with specimen sizes is 20 mm x 20mm x 6 mm and 50 mm x 50 mm x 6 mm, where the properties of ASTM A36 shown in figure 1.

Physical Properties	Metric
Density	7.85 g/cc
Mechanical Properties	Metric
Tensile Strength, Ultimate	400 - 550 MPa
Tensile Strength, Yield	250 MPa
Elongation at Break	20.0 %
	23.0 %
Modulus of Elasticity	200 GPa
Compressive Yield Strength	152 MPa
Bulk Modulus	140 GPa
Poissons Ratio	0.260
Shear Modulus	79.3 GPa
Component Elements Properties	Metric
Carbon, C	0.260 %
Copper, Cu	0.20 %
Iron, Fe	99.0 %
Manganese, Mn	0.75 %
Phosphorous, P	<= 0.040 %
Sulfur, S	<= 0.050 %

Source: <http://www.matweb.com/>

Figure 1: The Properties of ASTM A36 Steel.

ASTM A36 steel based on figure 1 including steel that has a low carbon composition, which is equal to 0, 26 % (low carbon steel).

3 RESULTS AND DISCUSSION

3.1 Extraction Inhibitor Solution Result



Figure 2: Extraction results of inhibitors from (a) cigarette butt waste (b) tea (c) tobacco.

After extraction process is done, then obtained result of cigarette butts extract, tea extract, and tobacco extract. Solution of extract corrosion inhibitor has thick brown characteristic with the volume of each cigarette butts extract as much as 15 mL with pH 6.2, tobacco extract 13 mL with pH 7.8, and as much as 18 mL with pH 7, 3 On every 1 scale of the reaction size (batch).

3.2 Surface Preparation and Blast Cleaning Testing Results

The result of the surface preparation test was determined by using a value parameter of RH

(humidity ratio) and dew point. This parameter derived from dry bulb and wet data respectively at 30 °C and 25 ° C. According to the test parameters, it can be noted that cigarette butts have a RH and Dew Point value of 67% and 23% respectively.

This indicates that the cigarette butts extract has a normal moisture level in accordance with the standard namely less than 85% (ASTM D-3451-06, 2017). The results of visual observation Blast cleaning SA2 1/2 shown in Figure 3, namely Steel SS 400/JIS G3101/ASTM A36 is a type of Steel with low carbon content so that the Blasting process is needed to clean the surface of the Steel due to oxidation process.



Figure 3: (a) Steel A36 before the blast cleaning process SA 2¹/₂ (b) Steel A36 after blast cleaning process SA 2¹/₂.

3.3 Surface Roughness and Coating Measurement Results

After the blasting process is done on the steel, the next process is the measurement of roughness done using ASTM D4417-2014 standard. According to the ASTM D4417-2014 standard range of steel roughness is 37 µm to 137 µm. In this roughness measurement the average roughness value obtained was 94.6 µm for specimens measuring 50 mm x 50 mm x 6 mm and 96.3 µm for specimens measuring 20 mm x 20 mm x 6 mm. This indicates that the surface roughness value has been accordance with standards. At this stage the WFT (Wet film Thickness) and DFT (thick dry film) values were obtained at the same time, 120 µm and 55 µm in compliance with ISO 2808-2007(R2010) standard.

3.4 Adhesion Test Result

The nature of adhesion between cigarette butts extract with steel is an important parameter in its application when coated on the jacket platform and in this research the adhesion properties are measured through the "Pull Off" method in accordance with D4541 2017 standard, which the results can be shown in Figure 4. Overall, the variation of cigarette butts extract 2% has a high adhesion power compared with other varieties. The result from the tobacco extract has the adhesion of lower. This becomes the advantage of cigarette butts as a corrosion inhibitor

on the jacket platform in physical terms because it has a high adhesive power due to great adhesion power.

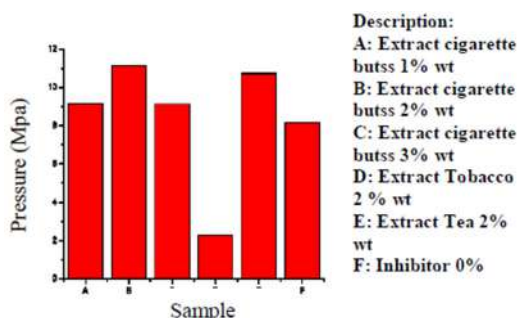


Figure 4: Adhesion Testing Results.

3.5 Measurement of Salinity of Seawater Results

The sea water of Kenjeran as much as 500 mL is heated at temperature 200 °C using the electric heater until it shrinks and becomes granules salt. At this stage, the salinity of seawater obtained is 3.32‰, which means that in seawater there are 0.032 grams of salt/mL of seawater so that the salinity obtained is 0.032 ppt with pH 8.2.

3.6 Corrosion Rate Testing Results

Testing of steel corrosion rate with specification of dimensions 20 mm x 20 mm x 6 mm is done through weight loss test with data obtained Inserted into the equation (1).

$$CR (M) = \frac{W \times K}{D \times A \times T} \quad (1)$$

Description:

CR = Rate of corrosion (miles per year (mpy))

W = Weight change from specimen (gram)

K = Constant factor (3, 45x106)

D = Density of specimen in units (g/cm³)

As = Area of specimen (cm²)

T = Time (hours)

The constant parameters in equation (1) are the factor constant (K) is 3.45 x106, the material density is 7.85 g / cm³ and the surface area of all materials is 12.8 cm², while the observation time is 144 hours (6 days). Based on the calculation obtained in equation (1), the corrosion rate values are obtained for all variations, both variations in mass percent of cigarette butt extract and other inhibitor type extracts which can be shown in Figure 5.

The lowest corrosion rate obtained in cigarette butts extracts of 2 % which is 28.6 mpy, even that value is still lower than tea and tobacco extracts at the

same mass percent which shows that cigarette butts extract is a type of corrosion inhibitor that has the highest corrosion inhibition rate compared to other inhibitor materials.

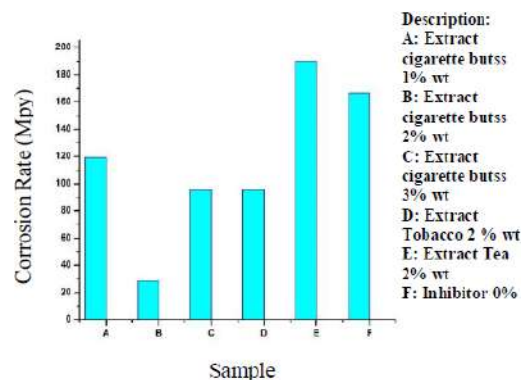


Figure 5: Corrosion Rate Results (mpy).

The calculation of corrosion rate can be evidenced by the efficiency of inhibitors calculated through the equation (2).

$$EI = \frac{X_A - X_B}{X_A} \times 100\% \quad (2)$$

Description:

X_A = rate of corrosion without inhibitors

X_B = corrosion rate with inhibitors

By using the equation (2), the result of the efficacy of B inhibitors was obtained 83%. It is also reinforced by the use of FTIR test results to identify the bonds contained in the compound, which can be shown in Figure 6. The red waves show FTIR spectra results from cigarette butts extract inhibitors. Based on these results, the sharp peaks are shown in the number of waves 3323.12 cm⁻¹ and 1019.79 cm⁻¹. The sharp and strong ribbon spectrum is shown at 1019.79 cm⁻¹. It shows the C-N group stretching and on the 3323.12 cm⁻¹ ribbon showing the NH stretching cluster.

The blue color waves show FTIR spectra results of mixing of cigarette butts extract inhibitors with epoxy paint with sharp peaks at 2918.79 cm⁻¹, 1455.32 cm⁻¹, and at 1011.27 cm⁻¹. On the Spectrum 2918.79 cm⁻¹ tape shows the N-H cluster stretching. As for 1011.27 cm⁻¹ indicates the C-N group stretching. Wave of black color showed FTIR results from paint 100% without added inhibitors.

Based on figure 6, it can be concluded that the nicotine substance found in inhibitors does not lose its ability as an inhibitor of resistor corrosion, because when added to the paint, the nicotine is still present and not lost, which means the ability of the inhibitor is not interrupted, according to the desired results.

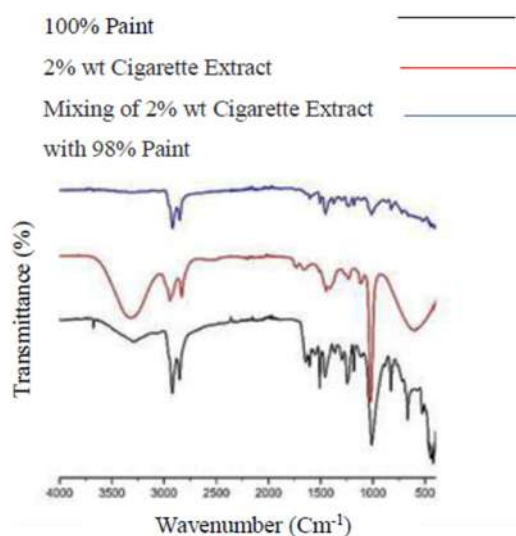


Figure 6: FTIR Spectra of Cigarette Butts Extract, 100% Paint, 2% wt Cigarette Extract, and Mixing of 2% wt Cigarette Extract with 98% Paint.

3.7 Material Characterization Results

Physical characterization of morphology of the steel surface is performed to compare the corrosion behaviour on the surface of the steel before and after coated by a corrosion inhibitor. Characterization is performed with the stereo microscope that the result can be shown in Figure 7. Based on the characterization, the addition of a 2% cigarette butts Extract can be seen that the corrosion is inflicted at the lowest in comparison to other variations. Corrosion spots can be seen clearly in the variation of the addition of cigarette butts extract by 1% which indicates that the inhibitory efficiency of the highest corrosion rate is obtained in the addition of a 2% cigarette butts extract. It is in accordance with the results Obtained at the corrosion rate measurement that the lowest corrosion rate is also obtained when the addition of 2% cigarette butts extract.

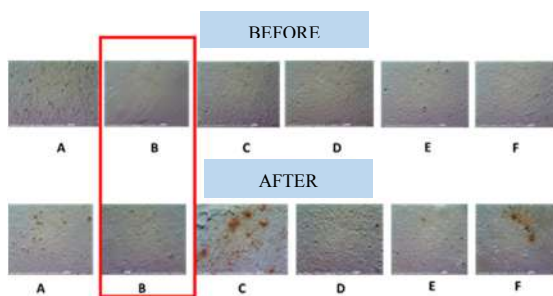


Figure 7: Observation Results of Stereo Microscope on Extract Sample (A) Cigarette butts 0.3% (B) Cigarette butts 0.2% (C) Cigarette butts 0.1% (D) Tobacco (E) Tea (F) Cat 100%.

4 CONCLUSIONS

Based on research the extract of cigarette butts potentially as alternative corrosion inhibitors on the jacket platform. The higher the percentage of mass extract cigarette butts, the smaller the rate of corrosion with the optimum cigarette butts extract of 2% with a corrosion rate of 28.6 mpy. In addition, cigarette butts extract has a higher corrosion rate reduction efficiency than in other types of corrosion inhibitors.

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Training of Fiberglass Boat Repair for Gresik's Fisherman to Improve Fiberglass Boat Quality of Boat Construction following the Indonesian Classification Regulations

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Keywords: Fiberglass, Damage, Classification, Training.

Abstract: There are several types of damage to the fiberglass boat, including holes, cracks, and deformation on the fiberglass boat hull. At this time, fiberglass boat's damages are repaired by fishermen with their knowledge without considering classification regulations. Therefore this paper will explain training on repairing fiberglass damage using a syllabus and methods by Indonesian classification bureau (BKI) regulations.

1 INTRODUCTION

Gresik Regency is between 7 degrees and 8 degrees south latitude and between 112 degrees and 113 degrees east longitude. Most of the area is lowland with an altitude between 0-12 meters above sea level, except a small part in the north (Panceng District) has a height of up to 25 meters above sea level. The Java Sea borders the northern part of the Gresik Regency; the Madura Strait and Surabaya City-bound the eastern part; the southern part is bordered by Sidoarjo Regency and Mojokerto Regency, while Lamongan Regency borders the western region.

Gresik Regency has an archipelago of Bawean Island and several smaller islands around it. The total area of Gresik is 1,192.25 km² consisting of 996.14 km² land area plus 196,11 km² area of Bawean Island while the area of the waters is 5,773.80 km² which is very potential from the marine fisheries subsector.

Gresik Regency is one of the buffer cities of Surabaya. Where the city of Surabaya is the capital and economic center of East Java and East Indonesia. Nearly one-third of the Gresik Regency is a coastal area. As a coastal area that has also been facilitated by a large port, Gresik Regency has access to regional, national, and even international trade.

The mapping of the marine fisheries sector in Gresik has the second-highest potential of marine

fisheries from the mapping of the marine fisheries sector in East Java with the acquisition of catches of 362,624 tons per year with this development the movement of the economic industry of the Gresik district community leading to the fisheries sector is also increasing (Norromadani Y, 2016).

With the use of boat for fishing activities, some damages caused by collisions or the age of the boat, so the boat needs to get maintenance and repairs. In general, repairs activities are replace the entire damaged part, but it will take a long time by reviewing the damages for hull repairs effectively. Repairing fiberglass boats usually still traditional way based on limited knowledge about fiberglass boat repair. Therefore, as an institution engaged in maritime, ITS is doing community service to develop and improve the welfare of the community with a form of training support for maintenance and repair of ships following the standards used by the Indonesian Classification Bureau for resistance to fiberglass ship structures.

2 LITERATURE REVIEW

2.1 Fiberglass Boat

FRP ships are generally made of fiberglass or glass fiber. This is because fiberglass is relatively cheaper when compared to other reinforcing materials.

Fiberglass also has a relatively strong strength with a light weight. Additionally fiberglass has chemical resistance and is easy to process. However, the strong tensile strength of this fiberglass decreases when it receives a continuous load over a long period of time.

Table 1: Composition of E-Glass and S-Glass per unit weight.

<i>Composition</i>	<i>E-Glass</i>	<i>S-Glass</i>
<i>Silicone Dioxide</i>	52-56 %	64-66 %
<i>Calcium Dioxide</i>	16-25 %	0-0,3 %
<i>Aluminium Oxide</i>	12-16%	24-26 %
<i>Boron Oxide</i>	5-10 %	-
<i>Sodium Oxide & Potassium Oxide</i>	0-2 %	0-0,3 %
<i>Magnesium Oxide</i>	0-5 %	9-11 %
<i>Iron Oxide</i>	0,05-0,4 %	0-0,3 %
<i>Titanium Oxide</i>	0-0,8 %	-
<i>Fluorides</i>	0-1 %	-

The table 1 shows the ratio of E-glass and S-glass composition by weight. In marine use applications, E-glass (lime aluminum borosilicate) is the most commonly used reinforcement material because it has good tensile strength, is resistant to water degradation, and is cheap. S-glass (silicon dioxide, aluminum, and magnesium oxides) has 33.33% better tensile strength and generally has more resistance to fatigue. (Ship Structure Committee, 1990).

Reinforcement fiberglass material is divided into several types, namely Chopped Strand Mat (CSM), Woven Roving (WR), and Multiaxial:

- Chopped Strand Mat (CSM)

Chopped Strand Mat (CSM) is a type of fiberglass made of glass fibers that are placed and arranged randomly between one another, as in Figure 2.4. A comparison of the use of resin against fiberglass Chopped Strand Mat is 2.5 ~ 3 Resin: 1 CSM. CSM Fiberglass is generally distinguished by weight per square meter. For example, CSM 300 means that each square meter of fiberglass weighs 300 grams.

- Woven Roving (WR)

Woven Roving (WR) is fiberglass made of glass fiber that is woven in two directions continuously at an angle of 90 °. Fiberglass WR is the reinforcement material most often used for the construction of marine structures. It is because WR fiberglass is available in quite a significant weight (24 ounces per square yard or around 800 grams

per square meter is the most commonly used weight) so that the thickness of the structure can be met quickly. Besides fiberglass, WR also has resistance to impact because the material is woven.

- Multiaxial

Multiaxial fibers consist of two or more layers of fibers with different direction orientations (0 °; 90 °; 45 °; -45 °), each report is sewn with fine polymer yarn. Multiaxial fibers can be combined with Chopped Strand Mat (CSM) and Woven Roving (WR). Multiaxial fiber is usually applied to wind turbines, fast boats, recreational products, racing cars, aerospace and defense system (defense equipment). (Baskoro, 2018)

2.2 Types of Fiberglass Boat Damage

Various reasons can cause damage caused to fiberglass vessels. Based on Greg Davis's survey in 1996, from 3000 respondents 61% damage to fiberglass boats owned by respondents occurred due to exposure to foreign objects submerged in the sea, 17% occurred due to collisions, 14% occurred due to lack of ship maintenance, 5% occurred due to aground and 3 % occurred due to manufacturing problems. Following are the types of damage that can occur to fiberglass vessels. (Greene, 2006)

- a. Tear on Boat Skin

Tear damage generally occurs when a ship collides with another boat or foreign object when the boat is at sea or the dock. Grounding or aground of the boat can also be a cause of this type of damage. This type of damage needs to do a visual inspection to find out how severe the injury and delamination that occurs. Tear damage was initially inspected visually. After the investigation carried out and the need for repairs is made, holes are built around the area of damage. The hole is what will be re-laminated during the repair process.

- b. Delamination

Delamination is damage that occurs when the loss of bond between the lamination of the skin of the ship and its core material. Delamination damage generally cannot be identified visually. To determine this damage, ultrasonic evaluation or tap testing is necessary. In the tap testing evaluation, a part of the delamination is suspected to be knocked using a metal or plastic hammer and listen to the sound it makes. Areas that occur delaminated will cause a higher tone.

- c. Cracks on the Gelcoat Layer

These cracks only occur on the surface of the gel coat layer, and rarely this damage causes

structural failure. However, cracks in the gel coat layer can cause water to seep into the fiberglass laminate layer. To detect, it can be done the visual evaluation and dye penetrant test.

2.3 Indonesian Classification Bureau Regulations

In Indonesia, the bureau that regulates fiberglass ship regulation is the Indonesian Classification Bureau (BKI). BKI published two regulations about fiberglass ships, namely BKI 2016, Rules for Fiberglass Reinforced Plastic Ships, and BKI 2014, Rules for Non-Metallic Material. BKI in 2016 regulates the calculations performed on the construction of fiberglass ships while BKI in 2014 regulates the manufacturing process, inspection, and repair of fiberglass vessels.

The rules used as a reference for the repair of FRP ships are the 2014 edition of the Indonesian Classification Bureau (BKI), Rules for Non-Metallic Material. According to (BKI, 2014), there are several conditions for repairing FRP vessels. Some of them are:

- The reparation process should only be carried out by personnel who have sufficient experience in this field, generally carried out by staff who have training certificates.
- The resin used for the repair process must be the same as the resin used in the production process. This is done to prevent residual stresses in the repair area.
- The reinforcement materials used are the same as those in the production process.

3 METHOD

Stages that need to be done in community service begins with communicating with fiberglass boat builders in the Lumpur Village, Gresik District, Gresik Regency. Next, a visit to the location of the people's boat port and fiberglass shipbuilding was made.

The meeting held with the fiberglass shipbuilders produced several things that needed to be used as information and input for community activities. Some of these include a) Obtained the location of community service for the repair of fiberglass vessels, namely at the community port of the Lumpur Village, Gresik District, Gresik Regency. b) The shape of the ship that is commonly used in the Lumpur area, ship construction, and the type of

fiberglass material used. c) The process of maintaining fiberglass fishing boats. d) Damages that occur as well as the location of damage to the ship. e) Plan for the operation of ship repair.

The method offered in this community service is to provide information transfer and training on fiberglass ship repair by the Indonesian Classification Bureau standards. The activity is carried out by providing material and interactive discussions so that the trainees can understand and facilitate the understanding of the content provided.

Evaluation of the implementation of the activities carried out after the ship repair training event is completed. Matters evaluated include a) The method of preparing the ship before repairs are made. b) Methods for preparing tools and materials for ship repair c) Methods for repairing vessels that are good and right. As for the analysis of the data used to assess the success of the training through a descriptive study based on participant responses to the process and results of the training activities carried out.

4 RESULTS AND DISCUSSION

Community service activities related to the maintenance and repair of fiberglass fishing vessels by following the regulations of the Indonesian Classification Bureau in the context of increasing the strength of the structure of fishing vessels in Gresik Regency are carried out through the following stages:

4.1 Preparation

The implementation of the training program was initiated by coordinating with the chief fisherman members in the Lumpur harbor area of the Lumpur Village to inform the members of the fishermen's association that there would be a training activity to repair fiberglass fishing boats.



Figure 1: Discussion with the chief fisherman.

4.2 Training Implementation

After purchasing materials that will be used in fiberglass boat training activities, the next event is to carry out fiberglass boat repair training. Fifteen participants were attending the training activities from fishers who participated in the activities of the local community. The ship used in training is also one of the training participants. During the training, participants were guided directly to carry out the stages in the fiberglass boat repair process.

Before the ship repair work is carried out, the ITS team briefs and explains the reparation process and stages that will be carried out. The trainees were divided into three groups to simplify and share the work to be done to save repair time. The first group was tasked with cleaning all ships to be given layers of fiber using a grinder. The second group is in charge of preparing the fibers that will be used. The fibers used are WR 600, CSM 350, and CSM 450. The third group is tasked with developing putty, which will be used to patch the hole in the ship and also to prepare the resin mixture to be used.



Figure 2: Briefing of participants before the repair process.



Figure 3: Cleaning of the hull before the repair process.

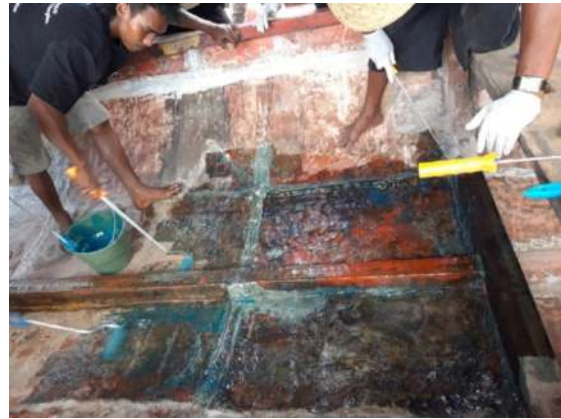


Figure 4: Provision of resin on the surface of the boat.



Figure 5: Installation of WR 600 at keel of the boat.



Figure 6: Photo group after the training.

4.3 Training Evaluation

Evaluation is carried out during the process and at the end of the activity. During the ship repair training process, participants were able to see extraordinary enthusiasm.

Before getting this training, they did not know how to mix resins and catalysts properly, did not know what percentage of catalysts had to be given and how to process the coating between fiberglass and resin properly. After attending this training, they can carry out the above work well.

Many participants held discussions with presenters while the training was taking place to solve problems they found during the ship repair training process. The results of an evaluation of 15 participants in the repair of fiberglass fishing vessels on a scale of 1 to 5 can be presented as follows:

Table 2: Evaluation table for training implementation.

No	Questions	Average	Criteria
1	Material given is needed by participants	4.53	Good
2	Achieve the objectives of the training program	4.00	Good
3	Ship repair methods used	4.66	Good
4	The ability of the speaker in delivering material	4.66	Good
5	Participation of participants in fish boat repair training	4.00	Good
6	Ship repair training materials can be applied for further improvements	4.86	Good
7	Adequate training facilities (tools and materials, places)	4.33	Good
8	Benefits of training for participants	4.80	Good
	Average	4.48	Good

5 CONCLUSIONS

The fiberglass fish boat repair training in the Lumpur Village, Gresik District, was well implemented, and the participants who attended the training showed high satisfaction. This activity can also improve the abilities and skills of fishers in the process of repairing their vessels when damaged.

Participants' responses to training activities based on the results of quantitative evaluations are in the outstanding category with an average rating of 4.48 seen from the aspects of the material needed by the participants, program objectives, improvement

methods, ability of speakers, facilities, and benefits for participants.

The results of the evaluation indicate that fiberglass fishing boat repair training activities are needed by the fishermen so that later, they can be used as a provision for boat repair knowledge if there is damage to their boat.

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Analysis on the Effect of Subsea Buoy to the Tension of Spread Mooring System

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Keywords: FSO, Spread Mooring, Subsea Buoy, Tension.

Abstract: The problem that might occur in floating structures with mooring systems is clashing between mooring lines with subsea equipment, for example pipelines. Addition of subsea buoys on the mooring line can lift the mooring line so that it can avoid clashing. The addition of the subsea buoy can affect tension on the mooring line. This research discusses the effects of subsea buoy to the tension of mooring line with a variation position of subsea buoy. Variations on the position of one subsea buoy is arranged at the distance of 605 m, 577.5 m, 550 m, 522.5 m from anchor and two subsea buoys at the distance 605 m and 467.5 m from the anchor. The analysis was carried out on stand alone and offloading conditions with wave directions 0°, 45°, 90°, 135°, 180°. The results after the addition of subsea buoys it has a smaller tension on the mooring line. The result of the variation of subsea buoy position, the optimum position to get the smallest tension value is when the variation of two subsea buoys with a distance of 605 m and 467.5 m from the anchor. From the results of the analysis there is also no clashing between the mooring line and pipeline.

1 INTRODUCTION

Natuna is an area at the northern end of the Karimata Strait. This area is one of the largest oil and gas reserves in the world. Natuna is an area where there are many offshore structure for oil and gas exploration, either fix structure or floating structure. At present, the development of offshore structure design technology is continuing to explore oil and gas in the Natuna area. One of them is the construction of Floating Storage and Offloading (FSO).

Floating Storage Offloading (FSO) is a floating structure in the form of a ship which serves to store hydrocarbons and transfer to vessels or barges. In its operation the FSO structure is movement caused by environmental loads, such as waves, wind and currents. So that the mooring system is needed on the FSO structure. The purpose of this mooring system is to limit movement and keep the FSO in place.

One type of mooring system that is usually used is spread mooring. The mooring system consists of several mooring lines that spread and are moored to the seabed using anchors. This system does not allow the ship to move or spin to reach a position where environmental effects such as wind, current, and waves are relatively small.

The construction of mooring systems there are many factors that must be considered, one of the distance between the mooring line and the mooring line or with other subsea equipment. Clashing between the mooring line and the pipe is one of the problems that can be found. The addition of the subsea buoy on the mooring line can avoid clashing between the mooring line and the pipe, because the subsea buoy can lift the mooring line so that clashing does not occur. Addition of the subsea buoy can affect the tension on the mooring line. This research discusses the effect of adding subsea buoys to the tension of mooring line with a variation of one subsea buoy with four variations of position and two subsea buoys.

2 LITERATURE REVIEW

Many studies and research analyze variations in mooring system designs, such as subsea buoy analysis on mooring systems. Examples of numerical analysis on hybrid mooring systems with clump weights and buoys by Yuan Z.M. et al. (2014) which analyzed the type of new mooring line, hybrid mooring system with clump weight and buoys (HMSWB). In this study Yuan Z.M only analyzed the

effect of buoys on (HMSWB) because previously the influence of the clump weight was discussed by Ji C.Y. et al. (2011). This study concluded that installed buoys can reduce tension on the mooring line.

Sundaravadivelu (1991) has a study that the increase in submerged buoy net buoyancy can reduce the excursion of the buoy. The buoys used in the study were single point subsurface mooring. Fitria, Favi Ainin (2018) conducted a research on adding clump buoys to mooring systems which aimed to see the effect of adding clump buoys to mooring line tension and avoiding the potential for clashing between mooring lines. The results of the study that the addition of a clump buoy on the mooring line can reduce tension and also clashing between the mooring lines. Mavrakos (1997) analyzed the effect of adding submerged buoys in the deep sea and has variations in the number, size, and position of the submerged buoy. The next analysis is the effect of adding submerged buoy to the tension and dry length on a single point mooring mooring system (Suseprasetyo, 2013). This analysis get the results of the submerged buoy displacement in proportion to the amount of dry length and the farther position of the submerged buoy from fairlead has smaller tension.

3 OBJECTIVES AND SCOPE OF STUDY

The objective of this research is to comprehend how the subsea buoy in the mooring system affects the tension of mooring line. The responses which to be analysed are tension of mooring line, maximum offset, and clashing between mooring line and pipeline. The scope of study and boundaries of this research are as follows.

- The mooring system used is spread mooring.
- FSO is analyzed in full load and ballast condition.
- Environmental data uses data in Natuna.
- Collinear environmental loading conditions.
- Variation of one subsea buoy at a distance of 605 m, 577.5 m, 550 m, 522.5 m from the anchor.
- Variation of two subsea buoys at a distance of 605 m and 467.5 m from the anchor.
- The size of the subsea buoy is fixed.
- Dynamic analysis using time domain method.

4 METHODOLOGY

The flow and procedure of this research was conducted in stages as follows.

- Structural data uses Belida FSO data which is a conversion from tankers, shuttle tanker data, & environmental data using data in Natuna.
- Modelling FSO & shuttle tanker
- Model validation is to ensure the modeling is accordance with the original structure. Validation by comparing the hydrostatic data from the software with the original hydrostatic data. Model validation using reference from ABS (2018).
- Mooring system modeling, which uses a spread mooring type with eight mooring lines and 45 ° & 60° angle configurations.
- The mooring line analysis conducted in this study is the analysis of tension, offset, and clearance between the mooring line and pipeline.
- Analyses were performed without subsea buoys and with subsea buoys.

5 RESULTS AND DISCUSSION

5.1 Structur Modeling

FSO modeling uses software by entering FSO coordinates.

Table 1: FSO Data.

Parameter	Unit	Value	
		Full load	Ballast
LOA	m	244.60	244.60
LPP	m	233.00	233.00
Breadth	m	42.20	42.20
Depth	m	22.20	22.20
Draft	m	15.50	7.00
KG	m	13.71	10.08
Displacement	Ton	12588.60	58833.87

Table 2: Shuttle tanker Data.

Parameter	Unit	Value
LOA	m	240.50
LPP	m	230.00
Breadth	m	42.00
Depth	m	21.20
Draft	m	14.85
KG	m	12.48
Displacement	Ton	118643.87

Table 3: Mooring Line Data.

Parameter	Unit	Value
Type	-	Chain, R4 Studless
Length of chain	m	914
Size	mm	87 mm dia
MBL	mT	783.35

Table 4: Mooring Hawser Data.

Parameter	Unit	Value
Type	-	Rope/Nylon
Size	mm	96 dia
MBL	mT	154.076

Table 5: Subsea Buoy Data.

Parameter	Unit	Value
Weight	kg	5600
Tinggi	m	4.6
Diameter	m	2.8

Table 6: Environment Data.

Direction	NE	E	SE	S	SW
Wind Speed (m/s)	18	11	10	13	13
Wave Data					
Hs	4.4	2.0	1.8	2.0	2.0
Tp	9.9	8.6	8.5	8.6	8.6
Current Speed (m/s)					
Surface	0.89	0.80	0.62	0.62	0.76
30 m	0.69	0.62	0.48	0.48	0.67
3 m	0.45	0.41	0.35	0.35	0.43

This is modelling of FSO & shuttle tanker.

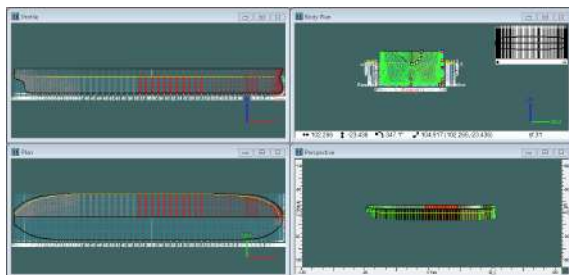


Figure 1: Modeling FSO with Maxsurf.

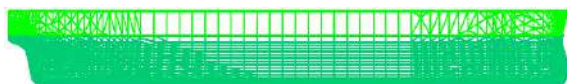


Figure 2: Side view FSO with Moses.

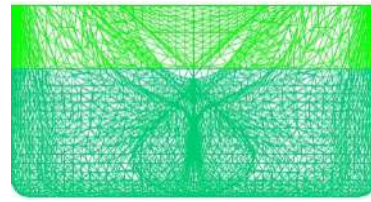


Figure 3: Front view FSO with Moses.

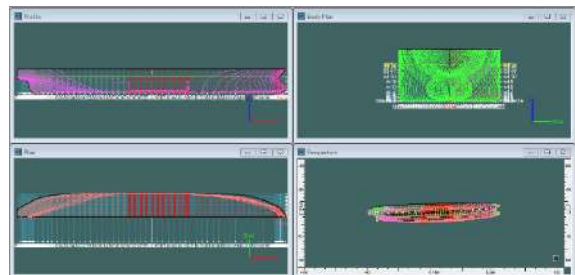


Figure 4: Modeling shuttle tanker with Maxsurf.



Figure 5: Side view shuttle tanker with Moses.

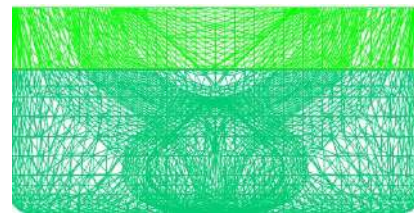


Figure 6: Front view shuttle tanker with Moses.

Mooring systems were modeling by software and modeled in 6 variations in two conditions, stand alone and offloading. The Variations were mooring line without subsea buoy, mooring line with one of subsea buoy at a distance from the anchor 605 m, 557.5 m, 550 m, 522.5 m, and mooring line with two subsea buoy at a distance from the anchor 605 m and 476.5m.

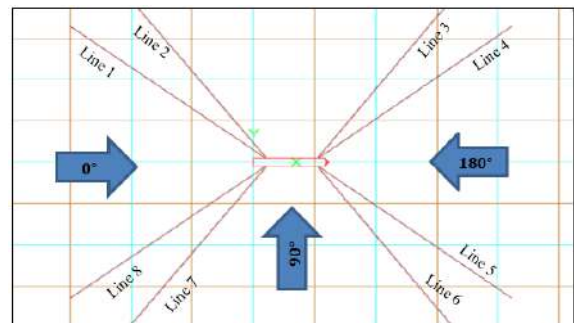


Figure 7: Stand alone conditions.

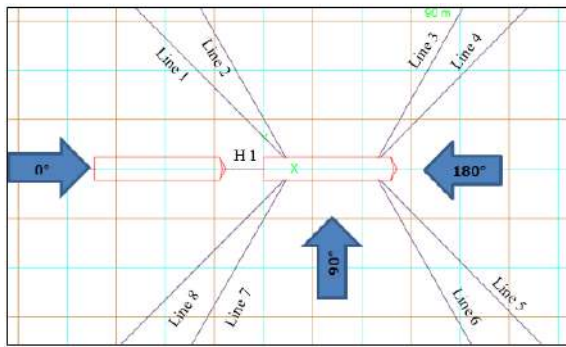


Figure 8: Offloading conditions.

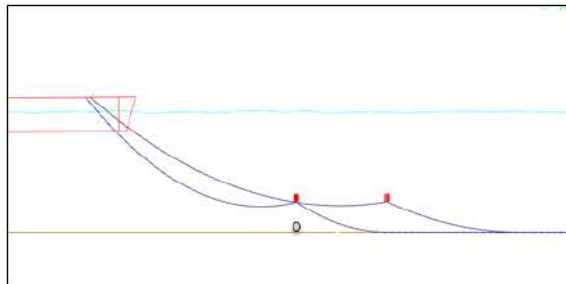


Figure 9: Mooring Line with one Subsea Buoy.

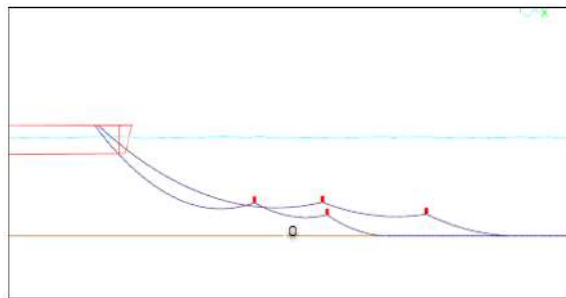


Figure 10: Mooring Line with two Subsea Buoy.

5.2 Model Validation

The model was validated based on the ABS (*American Bureau of Shipping*) MODU (*Mobile Offshore Drilling Unit*) rules, the difference of displacement modeling not exceed 2%.

Table 7: FSO Validation.

Load	Data	MOSES	Validasi
Full Load	128588.6	128561	0.02%
Ballast	58796.11	57589.1	2.00%

Table 8: Shuttle Tanker Validation.

Load	Data	MOSES	Validasi
Full Load	118644	118787	0.12%

5.3 Responses Amplitude Operator Analysis

RAO (Response Amplitude Operator) analysis is performed motion characteristics of FSO and shuttle tanker. This analysis carried out when free floating and moored condition in 6 degrees of freedom, namely surge, sway, heave, roll, pitch and yaw. The following is RAO of FSO and shuttle tanker during full load and ballast conditions.

1. RAOs Free Floating Condition

Table 9: Max. RAO FSO Full Load Condition.

Motion	Unit	RAO Max.				
		0°	45°	90°	135°	180°
Surge	m/m	0.97	0.69	0.00	0.69	0.97
Sway	m/m	0.00	0.70	0.99	0.70	0.00
Heave	m/m	1.00	1.00	1.45	1.00	1.00
Roll	deg/m	0.01	1.60	2.21	1.59	0.01
Pitch	deg/m	0.79	0.97	0.37	0.93	0.79
Yaw	deg/m	0.00	0.30	0.03	0.32	0.00

Table 10: Max. RAO FSO Ballast Condition.

Motion	Unit	RAO Max.				
		0°	45°	90°	135°	180°
Surge	m/m	0.98	0.69	0.00	0.69	0.98
Sway	m/m	0.00	0.70	0.99	0.70	0.00
Heave	m/m	1.00	1.00	1.09	1.00	1.00
Roll	deg/m	0.00	2.20	4.65	2.31	0.00
Pitch	deg/m	0.73	0.78	0.12	0.77	0.73
Yaw	deg/m	0.00	0.32	0.04	0.33	0.00

Table 11: Max. RAO shuttle tanker Full Load Condition.

Motion	Unit	RAO Max.				
		0°	45°	90°	135°	180°
Surge	m/m	0.97	0.68	0.00	0.68	0.97
Sway	m/m	0.00	0.70	0.99	0.70	0.00
Heave	m/m	1.00	1.00	0.45	1.00	1.00
Roll	deg/m	0.01	1.94	2.69	1.92	0.01
Pitch	deg/m	0.85	1.01	0.36	0.88	0.88
Yaw	deg/m	0.00	0.31	0.05	0.32	0.00

5.4 Mooring Line Tension Analysis

Analysis of the mooring line tension was carried out without subsea buoys and with subsea buoys in two

conditions, namely stand alone conditions and offloading conditions. Mooring line 6 has the largest tension value compared to the mooring line 3, 4, 5 which has been added to the subsea buoy. So that a comparative analysis of the position of the subsea buoy is carried out on the mooring line 6.

1. Stand Alone Condition

For analysis of tension on the mooring line carried out with conditions without subsea buoy, one subsea buoy with four variations of position and two subsea buoys in five of wave directions, namely 0° , 45° , 90° , 135° , 180° .

a. Wave Direction 0°



Figure 11: Max. Tension 0° .

b. Wave Direction 45°



Figure 12: Max. Tension 45° .

c. Wave Direction 90°



Figure 13: Max. Tension 90° .

d. Wave Direction 135°

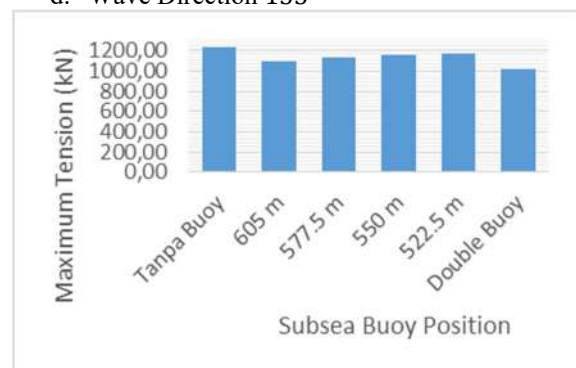


Figure 14: Max. Tension 135° .

e. Wave Direction 180°

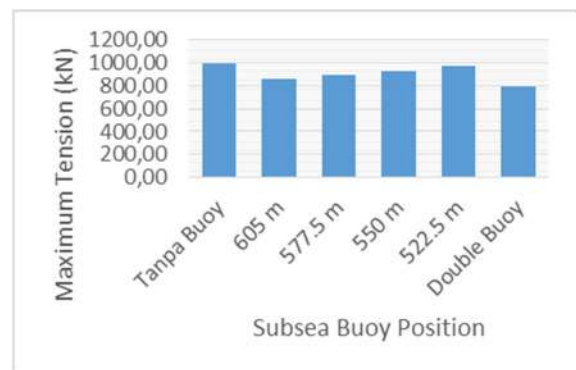


Figure 15: Max. Tension 180° .

From Figure 11 until Figure 15 indicates that the largest mooring line tension on the mooring line without subsea buoys. The mooring line with the subsea buoy from the smallest to the largest is the mooring line with two subsea buoys, the mooring line with one subsea buoy with a distance of 605 m, 577.5 m, 550 m, 522.5 m from the anchor.

2. Offloading Condition

For analysis of tension on the mooring line carried out with conditions without subsea buoy, one subsea buoy with four variations of position and two subsea buoys in five of wave directions, namely, 0° , 45° , 90° , 135° , 180° .

a. Wave Direction 0°

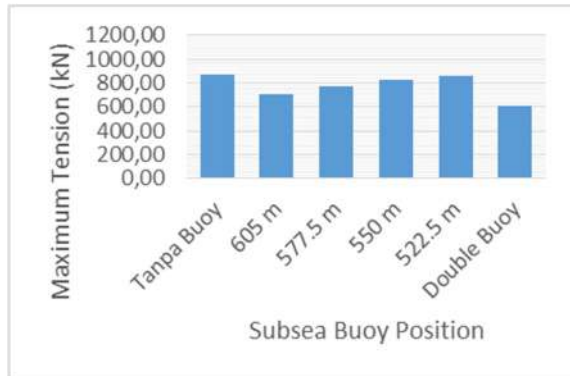


Figure 16: Max. Tension 0° .

b. Wave Direction 45°

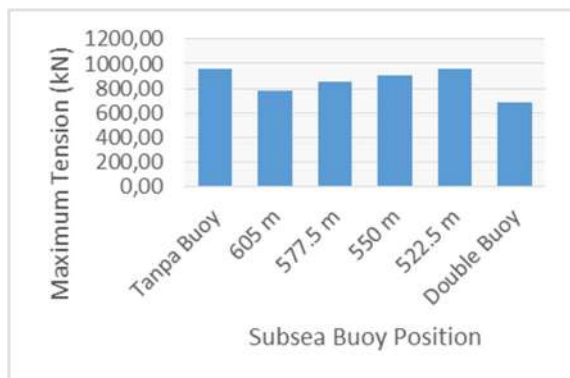


Figure 17: Max. Tension 45° .

c. Wave Direction 90°

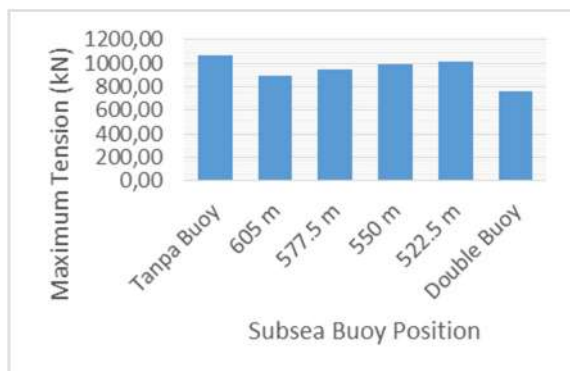


Figure 18: Max. Tension 90° .

d. Wave Direction 135°

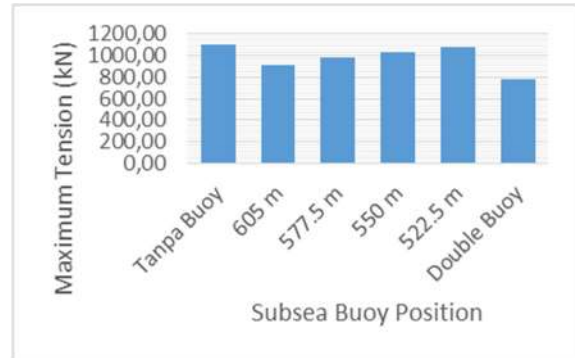


Figure 19: Max. Tension 135° .

e. Wave Direction 180°

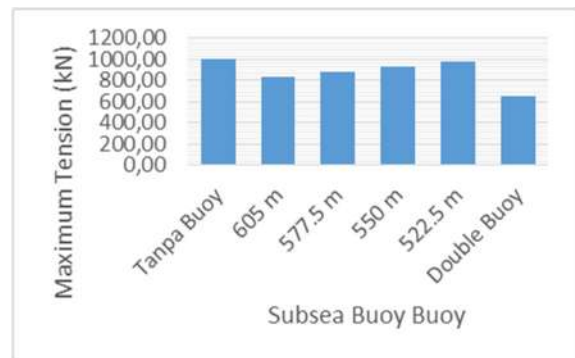


Figure 20: Max. Tension 180° .

From Figure 16 until Figure 20 indicates that the largest mooring line tension on the mooring line without subsea buoys. The mooring line with the subsea buoy from the smallest to the largest is the mooring line with two subsea buoys, the mooring line with one subsea buoy with a distance of 605 m, 577.5 m, 550 m, 522.5 m from the anchor.

5.5 Offset Analysis

Stand alone and offloading condition are used in the analysis.

1. Stand Alone Condition.

This analysis was carried out with 5 load directions include 0° , 45° , 90° , 135° and 180° .

Table 12: Max. Offset Stand Alone Condition.

Wave Directions	Offset x & y	Maximum Offset (m)		
		Tanpa Buoy	605 m	577.5 m
0°	x	0.80	1.67	1.05
	y	0.00	0.00	0.00
45°	x	0.34	2.28	1.67
	y	2.84	3.49	3.49
90°	x	0.41	2.34	1.72
	y	2.69	2.90	2.82
135°	x	0.31	2.06	1.47
	y	1.49	1.57	1.30
180°	x	1.43	2.89	2.24
	y	0.14	0.08	0.08

Table 13: Max. Offset Stand Alone Condition.

Wave Directions	Offset x & y	Maximum Offset (m)		
		Tanpa Buoy	605 m	577.5 m
0°	x	0.50	0.47	2.53
	y	0.00	0.00	0.00
45°	x	1.02	0.57	3.54
	y	3.46	3.44	3.68
90°	x	1.09	0.46	3.53
	y	2.70	2.62	3.45
135°	x	0.86	0.50	3.08
	y	1.28	1.28	2.13
180°	x	1.58	0.90	4.54
	y	0.08	0.08	0.08

2. Offloading Condition.

This analysis was carried out with 5 load directions include 0°, 45°, 90°, 135° and 180°.

Table 14: Max. Offset Offloading condition.

Wave Directions	Offset x & y	Maximum Offset (m)		
		Tanpa Buoy	605 m	577.5 m
0°	x	0.81	1.67	1.56
	y	0.00	0.00	0.00
45°	x	1.75	3.90	3.19
	y	5.19	5.92	5.62
90°	x	2.41	4.78	3.92
	y	6.72	7.75	7.28
135°	x	3.10	5.94	4.90
	y	7.97	8.91	8.77
180°	x	1.44	3.55	2.67
	y	0.14	0.00	0.00

Table 15: Max. Offset Offloading condition.

Wave Directions	Offset x & y	Maximum Offset (m)		
		Tanpa Buoy	605 m	577.5 m
0°	x	1.37	0.66	1.66
	y	0.00	0.00	0.30
45°	x	2.42	1.61	5.18
	y	5.34	5.15	6.40
90°	x	3.14	2.34	6.38
	y	6.84	6.52	8.40
135°	x	4.02	3.19	7.87
	y	8.26	7.85	9.11
180°	x	1.84	1.13	4.89
	y	0.00	0.00	0.00

5.6 Clearance between Mooring Line and Pipeline

Stand alone and offloading condition are used in the analysis.

1. Stand Alone Condition.

Table 16: Clearance between Mooring Line and Pipeline.

Line	Clearance (m)		
	No Buoy	605 m	577.5 m
3	0.00	6.51	10.10
4	0.19	17.42	14.41
5	0.19	17.42	14.41
6	0.00	6.51	10.10

Table 17: Clearance between Mooring Line and Pipeline.

Line	Clearance (m)		
	550 m	522.5 m	Double Buoy
3	12.45	4.77	25.73
4	10.84	3.20	29.90
5	10.84	3.20	29.90
6	12.45	4.77	25.73

From Table 16 and 17 shows that there is no clashing between mooring line and pipeline after the addition of the subsea buoy, but the clearance that matches the criteria of DNV OS E301 which is in the variation of one subsea buoy with a distance of 577.5 m from the anchor, one subsea buoy with a distance of 577.5 m from anchor, and on the condition of two subsea buoys. The biggest clearance occurred in the condition of two subsea buoys, namely on lines 3 and 6 valued at 25.73 m and on lines 4 and 5 worth 29.9m.

2. Offloading Condition.

Table 18: Clearance between Mooring Line and Pipeline.

Line	Clearance (m)		
	No Buoy	605 m	577.5 m
3	0.00	6.13	10.04
4	0.19	15.92	12.74
5	0.19	15.92	12.74
6	0.00	6.13	10.04

Table 19: Clearance between Mooring Line and Pipeline.

Line	Clearance (m)		
	550 m	522.5 m	Double Buoy
3	10.58	5.22	24.88
4	10.11	4.39	27.55
5	10.11	4.39	27.55
6	10.58	5.22	24.88

From Table 18 and 19 shows that there is no clashing between the mooring line and the pipe after the addition of the subsea buoy, but the clearance that matches the criteria of DNV OS E301 which is in the variation of one subsea buoy with a distance of 577.5 m from the anchor, one subsea buoy at 577.5 m from anchor, and on the condition of two subsea buoys. The biggest clearance occurred in the condition of two subsea buoys, namely on lines 3 and 6 valued at 24.88 m and on lines 4 and 5 worth 27.55 m.

6 CONCLUSIONS

The findings of the study could be revealed as follows:

- Mooring line without subsea buoys at stand alone conditions and in all loading directions has the maximum tension. The largest tension from the direction of 45° on line seven with a value of 1503.09 kN. The offloading condition, the mooring line without the addition of subsea buoys in all loading directions also has the maximum tension value. In the direction of 135° on line seven has the largest tension with a value of 1743.05 kN. For all tension it matches the criteria of API RP 2SK, which is a safety factor less than 1.67.
- From the research it is known that the addition of the subsea buoy reduces the tension on the mooring line. On the mooring line with one subsea buoy the further distance from the anchor has smaller tension. At the mooring line with two

subsea buoys with a distance of 605 m and 467.5 m has the minimum value. This condition occurs in all of wave directions when stand alone and offloading.

- From the research it is known that the addition of the subsea buoy can affect the offset of the FSO. In stand alone and offloading conditions, the highest offset is on the mooring line with two subsea buoys and a mooring line with one subsea buoy with a distance of 605 m from the anchor. The value of all offsets that occur is in accordance with the API RP 2P criteria.
- From the research it is known that the addition of subsea buoys can avoid clashing between mooring line and pipeline. However, for the clearance between mooring line and pipeline that matches the criteria of DNV OS E301 which is in the variation of one subsea buoy with a distance of 577.5 m from the anchor, one subsea buoy with a distance of 550 m from the anchor, and in the conditions of two subsea buoys. The biggest clearance occurred on the mooring line with two subsea buoys.

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Hydrodynamic Modelling of Flood Inundation on Welang River, Indonesia

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Keywords: A 1D-2D Coupled Hydrodynamic Model, Flood Inundation, Welang River, MIKE FLOOD.

Abstract: A flood inundation in a river can be caused by an increase in river water level due to the addition of water discharge from both the rainfall and the tides from the sea. Flood inundation simulation can be done by using a numerical flood model flood model. In the Welang basin, floods occur every year and cause many problems. Floods that occurred on April 29, 2019, caused the closure of national and regional roads and many houses that were flooded. In the present study, flood model was carried out by using MIKE FLOOD software. Semare Village and Tambakrejo Village are critical points where floods occur. The depth of the water from the simulation model occurs 0.3 m - 2.2 m compared to the flood map report by the Indonesian National Disaster Management Agency which estimates it at 0.1 m - 1.5 m.

1 INTRODUCTION

The Welang Basin is threatened with flooding every year. It happened during the rainy season from December to June. The higher the surface of the river water from the increasing intensity of rainfall and rising sea level due to the tide cycle, this results in a higher risk of flooding. The Welang River has many tributaries (about 21) with a total watershed area of 509.5 km² and a length of about 53 km, so that the river's water level becomes high when the intensity of rainfall increases.

(Wahyudi et al., 2018) investigated a method to apply eco-drainage to control flooding that often occurs in the Welang river, however, the flood can't be predicted how far it can spread of eventhough the flood affects Pasuruan districts so that flood control using the method of harvesting rainwater, by using infiltration ponds requires a large number of wells to be applied. (Wahyudi and Sumirman, 2019), showed how the flood management in the Welang basin can be done by injecting flood water into ground water using Artificial Storage Recharge (ASR). However, there have been no studies on two-dimensional flood inundation models in the Welang River Basin.

MIKE FLOOD 1D-2D coupled model is a powerful software for integrated hydraulic (1D) – hydrodynamic (2D) modelling purpose. A lot of research has been done using MIKE FLOOD 1D-2D coupled model (e.g., Dat, et al., 2019, CTCN, 2017, Timbadiya et al., 2014, Kadam and Sen, 2012, Patro et al., 2008). In the present paper, the existing condition of flood in the Welang river is simulated by using the MIKE FLOOD 1D-2D coupled model. The expected final result is a flooding map.

2 STUDY AREA

The modelling area is located at the downstream of Welang River. The desired model area is located at Kraton sub-district, Pohjentrek sub-district (Pasuruan district), and Gadingrejo sub-district (Pasuruan City) from Dhompo village to river mouth on the north, where river levels data is measured at Dhompo village in the coordinate system (-7.665908 latitude, 112.856342 longitude). The model area showed in Figure 1 within the red area.

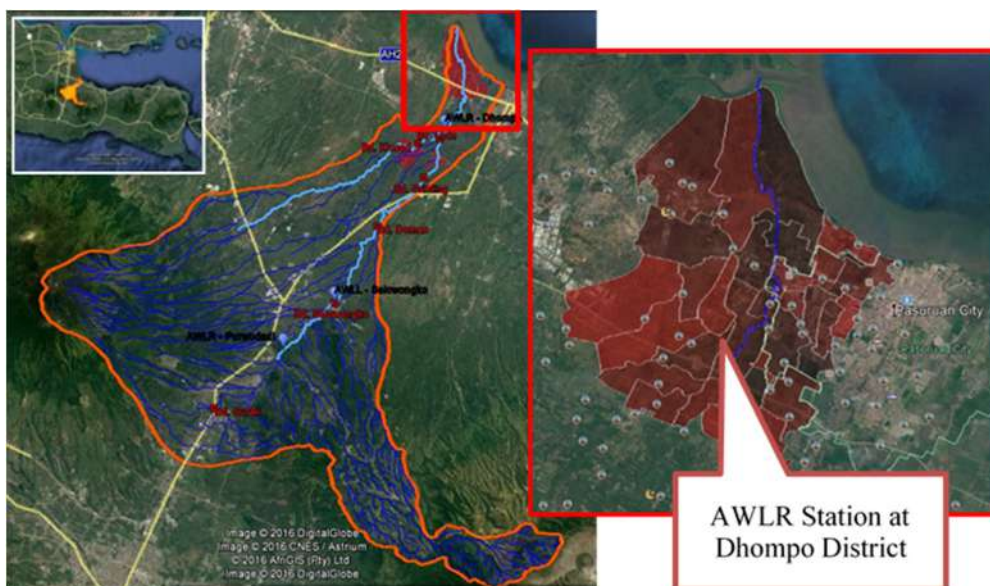


Figure 1: Modelling area of Welang River (Google Earth Pro).

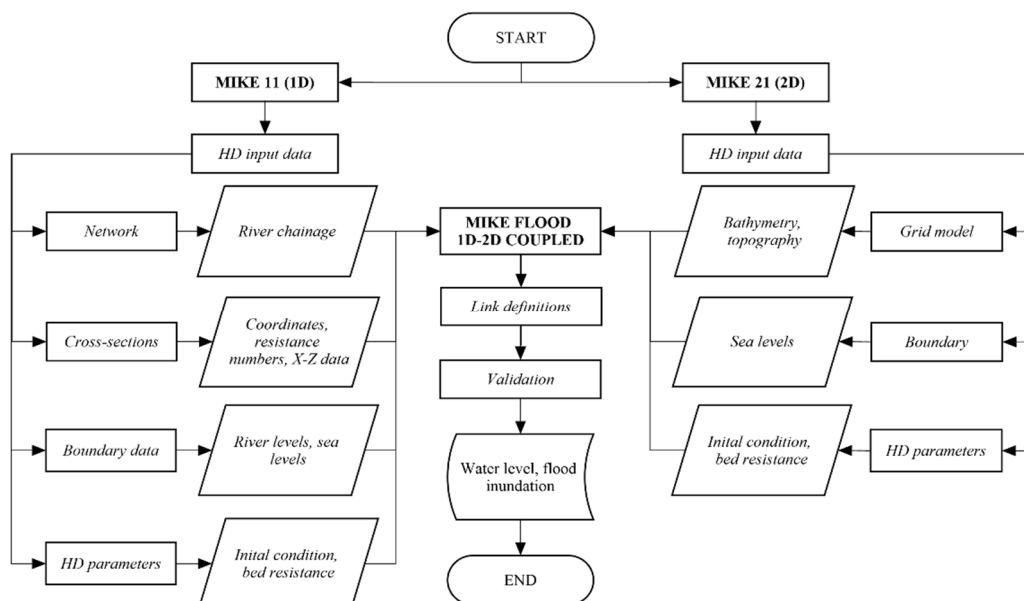


Figure 2: Flow chart of MIKE FLOOD 1D-2D coupled model.

3 METHOD

The methodology applied for solving this problem consists of the data collection, the model settings on MIKE 11 1D and MIKE 21 2D, the models added MIKE FLOOD 1D-2D, and the model validation can be seen in Figure 2. The MIKE FLOOD 1D-2D coupled model consists of upstream river surface and sea level at the river mouth (Figure 3). Hourly river

level data collected for 1 year in 2019 and sea level forecast data were collected from measurements for 15 days in 2014 in Grati, Pasuruan.

Bathymetry and topography data were collected from the Indonesian Geospatial Information Agency. Surface height and cross-section data collected from the Water Resources Public Works Department of East Java Province. The model runs from 27 April 2019 11:00 PM and finishes on 29 April 2019 6:00 AM.

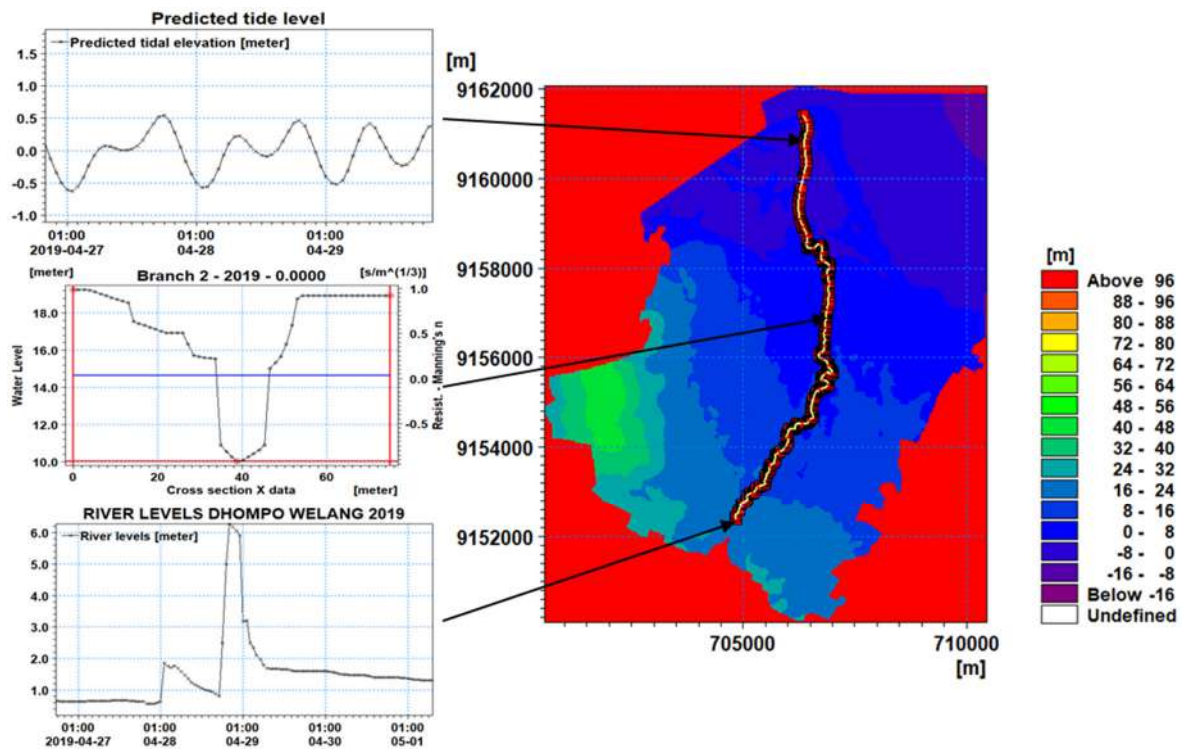


Figure 3: Modelling setup of MIKE FLOOD coupled model.

Manning coefficient is used for determining the bed roughness parameters. Applied bed roughness coefficients are divided into river, landuse and sea bed areas, which use a uniform distribution. The bed roughness coefficient values are estimated based on (Chow, 1959) and the MIKE 21 recommendation summarized in Table 1.

Table 1: Applied Manning number.

Natural stream	Manning's n
River upstream	0.040
River downstream	0.045
Landuse	Manning's M
Vegetation, natural areas	22
Sea	Manning's M
Seabed	32

Table 2: Average Bed Slope of Welang River.

Area	Stations (m)	Slope
Dhomp	0 – 1000	0.002408
Sungi Wetan	1000 – 3000	0.001601
Sukorejo	3000 – 5200	0.001890
Tambakrejo	5200 - 6000	0.002657
Semare	6000 – 11400	0.000870

According to the Manning equation, the slope of the river bed also contributes to the increase in river water flow and flow discharge. Areas that have a low slope are at risk of water deposition, causing flooding to occur. Semare village has a low slope shown in Table 2. Because the location of the Sumare village close to the river mouth causes a very high tidal effect as seen in Figure 4.

Model validation was performed using the flood map data from National Disaster Management Authority of Indonesia (BNPB, 2019) shown in Figure 5.

4 RESULTS AND DISCUSSION

Comparison of flood maps from flood models and flood map reports from BNPB is shown in Figure 6. Green polygons represent inundation in Tambakrejo Village and yellow polygons represent inundation in Sukorejo Village. At 01:00 28 April 2019, this model showed good results compared to the flood map report from BNPB, 2019. The validation process is carried out through water depth data where the results of this validation are shown in Table 3.

The time of the flood occurs when the highest tide is 0.47 m MSL at 20:00 28 April 2019. At 20:00, 28



simulation. However, the temporal variation of flood map on Welang River has not been examined in detail, yet.

Type	Water depth (m)
Model	0.3 – 2.2
BNPB	0.1 – 1.5

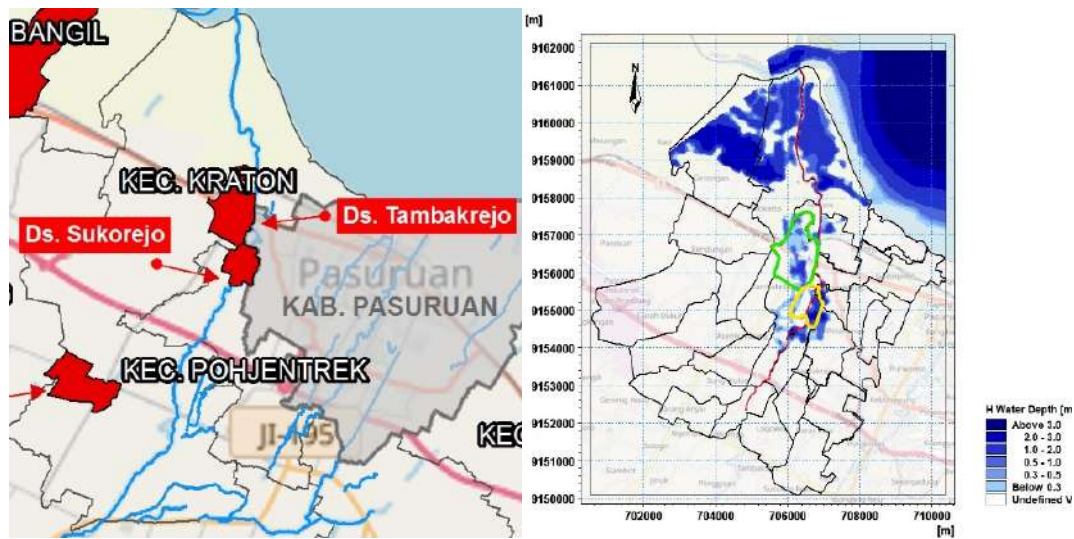


Figure 6: Flood model validation at 1:00 AM 28 April 2019.

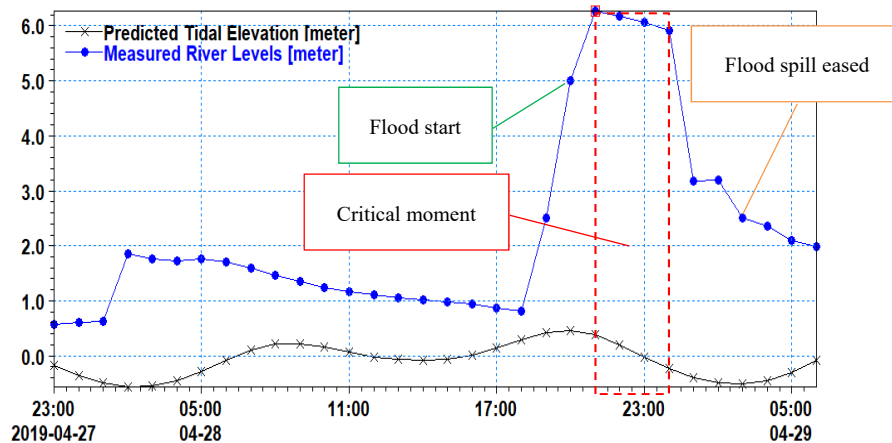


Figure 7: The relation of the river levels and tide levels when flood occurred on Welang River.

5 CONCLUSION

Based on the modeling results and analysis in the paper, a few conclusions can be drawn as follows:

1. From the results of the hydrodynamic model it has been shown the critical point at which floods begin to overflow. Semare Village and Tambakrejo Village are critical areas where floods begin to overflow.
2. This Kali Welang flood modeling simulation is validated using water level data from the flood map report of the Indonesian National Disaster Management Agency (2019), which shows quite valid results, the simulation model results show 0.3 m - 2.2 m while the data used for this tie tie is 0.1 m - 1.5 m.

3. River water level 6.27 m occurs during tidal height 0.39 m MSL until 0:00 April 29, 2019 with river height 5.91 m at tidal height -0.24 m MSL. At 3 am on April 29, 2019, the height of the river did not exceed the height of the riverbank so that the flood spill subsided.

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Modelling Water Quality in Welang River Estuary, Pasuruan

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Keywords: Modelling, Water Quality, Welang River, Estuary, Pasuruan.

Abstract: Land use activities for agriculture, plantations, industrial areas, fishponds and settlements in the Welang watershed can affect water quality in the Welang estuary. Waste disposal from these activities into the river can reduce water quality. The purpose of this study is to apply a numerical model to assess the water quality in the Welang estuary in accordance with permitted water quality standards in the coastal water. 2D numerical model with Hydrodynamic and Ecolab module is used to simulate the distribution of DO and pH as water quality parameters. Result of the hydrodynamic model showed that the current velocity flows into upstream at the tide, whereas at the ebb the current flows into downstream. The water quality modelling results show the minimum value of DO is 1.80 mg/l, and pH between 7.74 -7.87, while, the DO quality standard must be more than 3 mg/l, and pH is 6-9. Modelling results show that water quality in the Welang estuary is lightly polluted, because DO values are not in accordance with the water quality standards for class III based on government regulation number 82/2001.

1 INTRODUCTION

Welang River is one of the major rivers in East Java that crosses Malang Regency, Pasuruan Regency, and Pasuruan City. The upstream river is located at Lawang, Malang Regency and estuary in the Madura Strait in Kraton Regency, west of Pasuruan City. The Welang watershed from upstream to estuary has an area of 518 km² with the main river length of 36 km and an average width of 25 m (Public Works and Water Resources Pasuruan, 2019). Upstream of the Welang river is agriculture, plantation, industrial and residential area. Midstream of the Welang river is residential area and agriculture. The downstream is an agricultural area, fishpond, and industrial area. Various land-use activities in the Welang watershed can affect the river water quality. Organic and inorganic waste generated from these activities directly or indirectly enter the river will accumulate in the estuary causing the water quality in the estuary to decrease.

Previous research by Suntoyo et al. (2015) aims to determine the condition of water quality in the Porong river estuary due to mud disposal in Sidoarjo. The parameters used in modelling are chemical demand (COD), total suspended solid (TSS), phosphate and nitrate. The results showed that the water quality in Porong estuary still within safe criteria, because the water quality parameters none of which exceed the

quality standard based on government regulation number 82/2001. Another research was carried out by Gopal et al. (2018) to find out the water quality at the mouth of the Uppanar river. This river is an agricultural area, settlement and fishing port downstream, barren land and industrial centre in the upstream and middle. The results showed that the values of nitrate, phosphate, iron and lead exceed the permissible limit of WHO drinking water guidelines.

Research about water quality at the Welang estuary has never been done before. Therefore, research is needed to assess whether water quality in the Welang estuary in accordance with permitted water quality standards. The parameters used to assess water quality are dissolved oxygen (DO) and power of hydrogen (pH). DO is a very important parameter for knowing water quality (Thomann and Mueller, 1987).

2 STUDY AREA

The study area is located in the Welang Estuary, Pasuruan East Java Province (Figure 1). Geographically located between 7.57-7.60 south latitude and 112.86-112.89 east longitude.

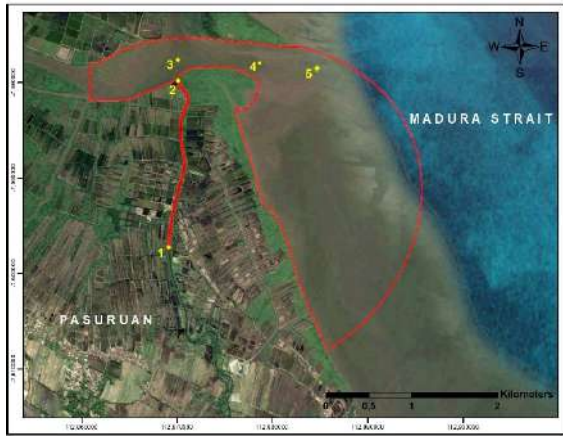


Figure 1: Study area in Welang estuary.

3 MATERIAL AND METHOD

3.1 Data Collection

Water quality measurements were performed at 5 points in the surface (Figure 1). The measurement point is along the river to the coastal. Measurement of water quality parameters are DO and pH.

3.2 Method

Modelling simulation was done with input bathymetry, river flow discharge, tides, and water quality parameters. River flow discharge data is used as boundary conditions in rivers, while tides are used as boundary conditions at sea. The simulation was carried out for 30 days with the time step interval is 3600s, number of steps is 720. The boundary condition shown in Figure 2. The hydrodynamic model is used for water surface and currents 2D modelling. The continuity equation and the momentum equation used for hydrodynamic modelling as follows:

Continuity equation:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = \frac{\partial d}{\partial t} \quad (1)$$

Momentum equation:

- Axis – x

$$\begin{aligned} \frac{\partial u}{\partial t} + \frac{\partial}{\partial x} \left(\frac{u^2}{h} \right) + \frac{\partial}{\partial y} \left(\frac{uv}{h} \right) + gh \frac{\partial \zeta}{\partial x} + \\ \frac{gu\sqrt{u^2 + v^2}}{C^2 h^2} - \frac{1}{\rho_w} \left[\frac{\partial}{\partial x} (h\tau_{xx}) + \frac{\partial}{\partial y} (h\tau_{xy}) \right] \\ - \Omega_v - fVV_x + \frac{h}{\rho_w} \frac{\partial}{\partial x} (p_a) = 0 \end{aligned} \quad (2)$$

- Axis – y

$$\begin{aligned} \frac{\partial v}{\partial t} + \frac{\partial}{\partial y} \left(\frac{v^2}{h} \right) + \frac{\partial}{\partial x} \left(\frac{uv}{h} \right) + gh \frac{\partial \zeta}{\partial y} + \\ \frac{gv\sqrt{u^2 + v^2}}{C^2 h^2} - \frac{1}{\rho_w} \left[\frac{\partial}{\partial y} (h\tau_{yy}) + \frac{\partial}{\partial x} (h\tau_{xy}) \right] \\ - \Omega_u - fVV_y + \frac{h}{\rho_w} \frac{\partial}{\partial y} (p_a) = 0 \end{aligned} \quad (3)$$

Where ζ = water surface (m), t = time (s). g = gravitation acceleration (m/s^2), x dan y = space coordinat (m), u dan v = flux density in direction of x and y ($\text{m}^3/\text{s/m}$), $f(V)$ = wind friction factor, P_a = atmospheric pressure (kg/m^2), ρ_w = water density (kg/m^3), Ω = parameter of Coriolis, h = water depth (m), d = water depth varied with time (m), C = resistance coefficient Chezy (m/s), τ_{xx} , τ_{xy} , τ_{yy} = component of effective shear stress, V , V_x , V_y = wind velocity in direction of x and y (m/s).

ECO Lab equation used to simulate the DO and pH distribution pattern in a given time is shown as follows:

$$P_c = \frac{dc}{dt} = \sum_{i=1}^n process_i \quad (4)$$

Where P_c = ECO Lab process, c = the concentration of the ECO Lab state variable, n = number of processes involved for specific state variable, $process$ = user specified expression containing argument such as mathematical function, build in function, number, forcing, constants, and state variable.

The calibration of the hydrodynamic model and water quality was conducted by comparing the data of modelling results with the measurement data. The calibration of hydrodynamic modelling uses the root mean square error (RMSE) and mean absolute percentage error (MAPE) methods, whereas for ECO Lab modelling only uses MAPE. The RMSE equation as follows (Wei, 2006):

$$RMSE = \sqrt{\frac{1}{M} \sum_{l=1}^M e_l^2} \quad (4)$$

Where, M is amount of data and e_l is obtained from the following equation:

$$e_l = Z_{n+l} - \hat{Z}_n(l) \quad (5)$$

Where, Z_{n+l} is measurement data and $\hat{Z}_n(l)$ is modeling data. The equation for calculating MAPE as follows:

$$MAPE = \left(\frac{1}{M} \sum_{l=1}^M \left| \frac{e_l}{Z_{n+l}} \right| \right) 100\% \quad (6)$$

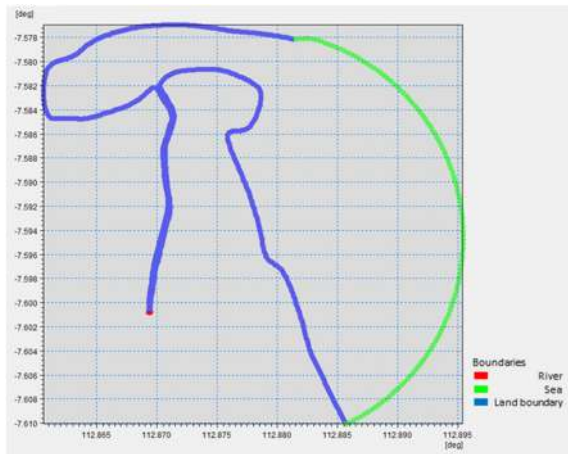


Figure 2: Boundary condition of modelling.

4 RESULTS AND DISCUSSION

4.1 Bathymetry

Bathymetry modelling in the Welang estuary is shown in Figure 3. The depth at the Welang estuary ranges from 0-5.5 meters.

4.2 Model Validation

The calibration of the hydrodynamic model was conducted by comparing simulation results at point 3 with the measurement data. For the hydrodynamic model, the root mean square error (RMSE) value is 0.01 and the mean absolute percentage error (MAPE) value is 0.64%.

Table 1 shows the measurement data at point 3 and the results of the water quality modelling. The difference in value is relatively small. DO value of the field measurement result is 4.08 mg/l, while the modelling result is 4.75 mg/l. pH value of the field measurement result is 7.87, while the modelling result is 7.84. MAPE value for DO is 16.42% and for pH is 0.38%. It can be concluded that the results of water quality modelling give good results.

Table 1: Validation of modelling results with measurement data at point 3.

Water Quality Parameters	Field measurements	Modelling Result	MAPE (%)
DO	4.08 mg/l	4.75 mg/l	16.42
pH	7.87	7.84	0.38

4.3 Hydrodynamic Simulation

The types of tides that occur at the Welang estuary is mixed tide prevailing semidiurnal. The water surface elevation at the highest tide reaches 2.8 m in the simulation step of 35. While at the lowest tide, water surface elevation is 0.5 m in the simulation step of 42. Figure 4 shows the tidal graph at the Welang estuary.

Current speed at high tide between 0-0.06 m/s, whereas at low ebb 0.0-0.28 m/s. Current flow into upstream at the highest tide, while at the lowest ebb the current flow into downstream. Currents flow to upstream or towards river mouths during high tide conditions with the maximum current speed of 0.13 m/s (Figure 5a). During the low ebb, current flow into downstream or toward the coastal. Maximum current speed is 0.30 m/s (Figure 5b).

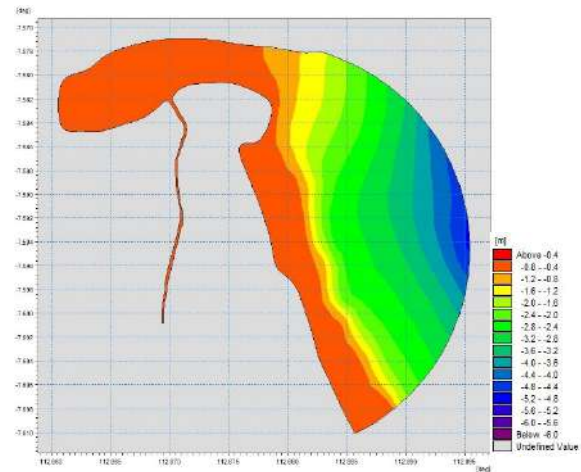


Figure 3: Bathymetry in Welang estuary.

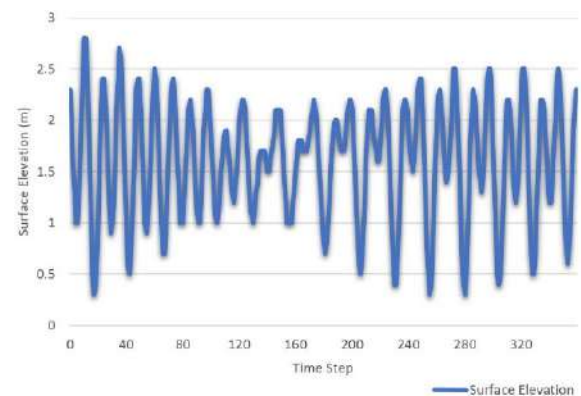


Figure 4: Water Surface Elevation.

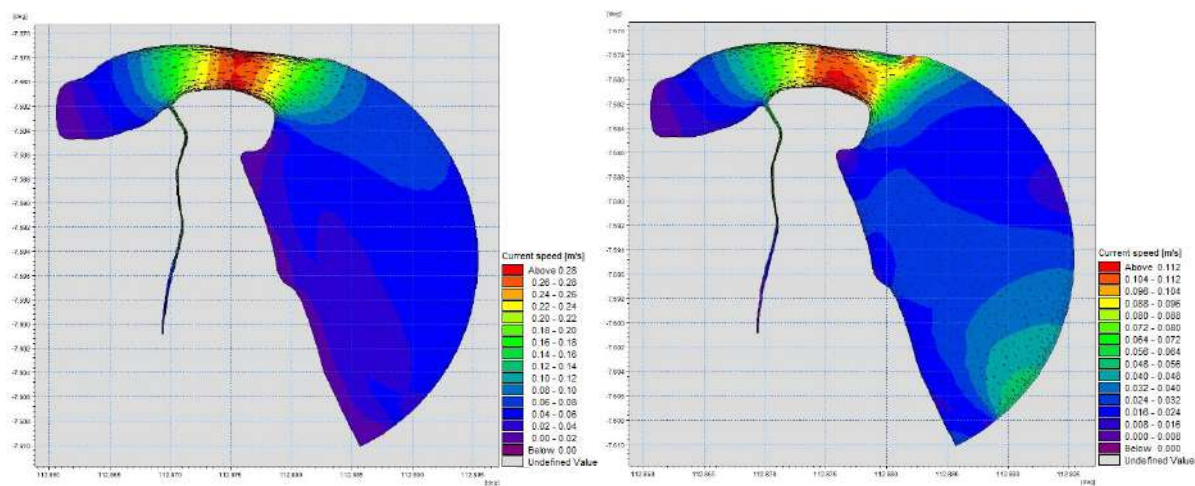


Figure 5: Current distribution at highest tide (a) and lowest ebb (b).

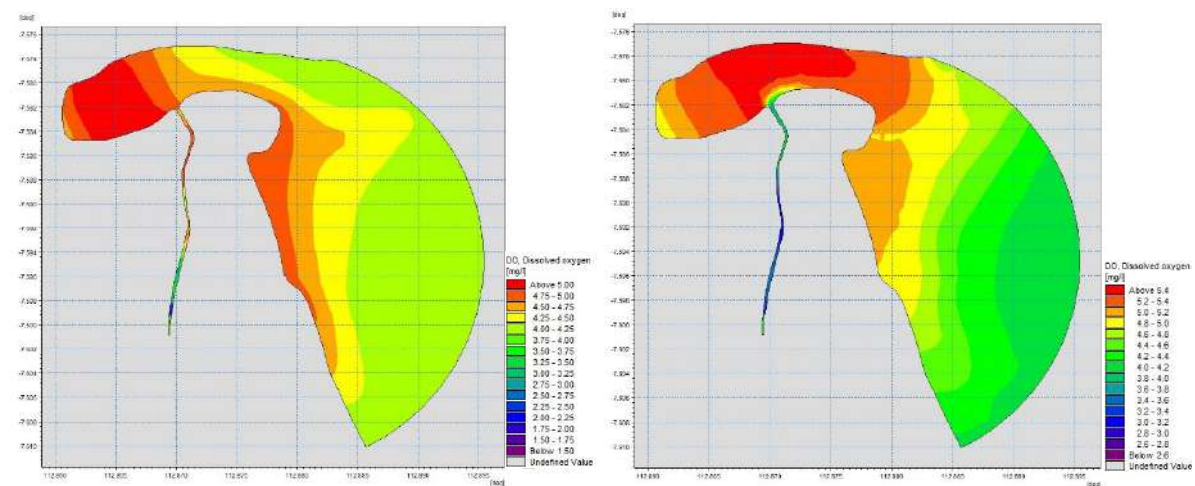


Figure 6: Distribution of DO during tide condition (a), and during ebb condition (b).

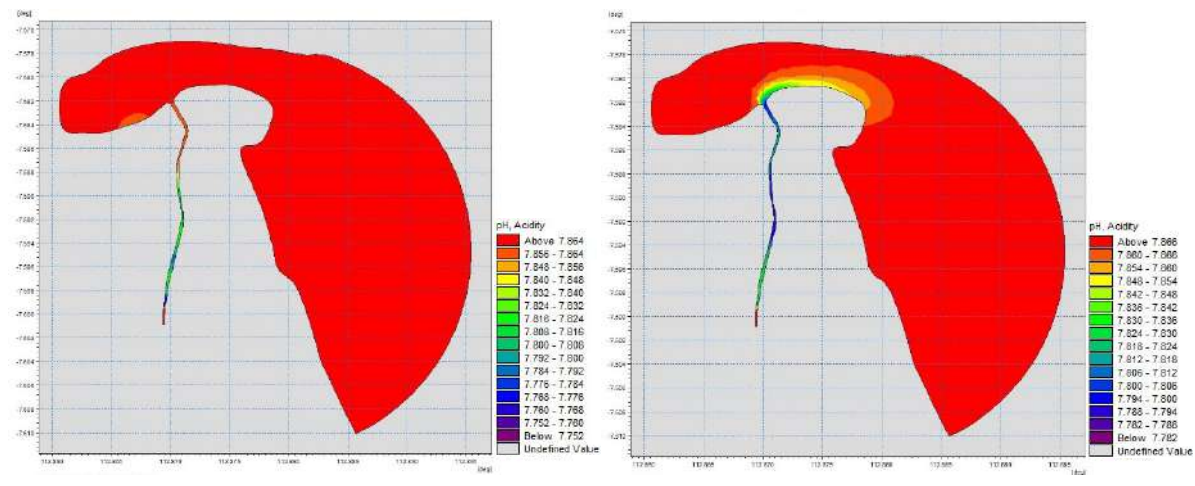


Figure 7: Distribution of pH during tide condition (a), and during ebb condition (b).

4.4 Water Quality Parameters

Water quality measurements have been conducted for 5 points in the study area (Figure 1), where measurement data are given in Table 2. The highest concentration of DO at station 3 reaching 4.08 mg/l, while the lowest at station 1 with a value of 1.07 mg/l. The highest pH at station 5 reached 7.941, while the lowest at station 1 with a value of 7.735.

The modelling results of DO shown in figure 6. DO concentration at tide is lower than at ebb. DO distribution varies between 1.80-5.24 mg/l in the river body to the coastal at tide (Figure 6a). DO concentrations in river bodies are smaller than in river mouth and coastal area. At the river bodies, maximum DO concentration is 4.97 mg/l, the minimum DO concentration is 1.80 mg/l. In the estuary, concentration of DO maximum value is 5.24 mg/l and minimum is 4.01 mg/l. When at the ebb, the spreading of DO does not vary much with at tide. Concentration of DO ranged from 3.01-5.46 mg/l (Figure 6b). The highest DO concentration is 5.46 mg/l around the estuary. The maximum DO concentration in the river is 4.23 mg/l, while the smallest concentration is 3.01 mg/l. In the estuary, concentration of DO maximum value is 5.46 mg/l and a minimum of 3.7 mg/l. DO concentrations at the river mouth of and coastal area are greater than in river bodies during tide and ebb.

Figure 7 shows the distribution of the pH in for tide and ebb condition, it was indicated that the value of pH in the river body and estuary are not much different with range 7.741-7.864 (Figure 7a). The value of pH in the river body is lower than in the estuary and coastal area. At high tide, the maximum value of pH at the coastal area which is 7.860, while maximum value at the river is 7.743. At ebb, the distribution of pH also has the same tendency as the current tidal conditions (Figure 7b). The maximum value of pH is 7.866 at estuary and the minimum value is 7.866 at river body.

Table 2: Water quality measurement data.

Station	DO (mg/l)	pH
1	1.07	7.735
2	1.18	7.778
3	4.08	7.876
4	3.43	7.93
5	3.14	7.941

4.5 Water Quality of Welang River Estuary based on Quality Standard

Water quality in Welang estuary analysed under the government regulation number 82/2001 on the management of water quality and water pollution control. Classification of Welang estuary included in water quality class III. The modelling result of water quality parameter compared with the value of the water quality standard provided in the regulation (Table 3). Table 3 shows the DO value is less than the water quality standard. Based on government regulations, the DO quality standard must be more than 3 mg/l, while the modelling results show the lowest DO value is 1.80 mg/l. pH value of the modelling result showed still relatively safe. Water quality standards of pH are 6-9, while modelling results show the values between 7.743-7.87.

Table 3: Water quality in Welang estuary.

Water Quality Parameters	Water Quality Standard (Water class III)	Value of modelling Result
DO	>3 mg/l	1.12 - 6.93 mg/l
pH	6 - 9	7.743 - 7.87

5 CONCLUSIONS

The modelling results show the minimum value of DO is 1.80 mg/l, and pH between 7.743-7.87, while, the DO quality standard must be more than 3 mg/l, and pH is 6-9. Modelling results show that water quality in the Welang estuary is lightly polluted, because DO values are not in accordance with the water quality standards for class III based on government regulation number 82/2001.

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Distribution of Current and Temperature at Welang Estuary, Pasuruan

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Keywords: Estuary, Temperature Distribution, Current Velocity, 3D Modelling, Welang River.

Abstract: Welang river estuary has many benefits to the surrounding community, i.e for the fish and shrimp farming. Estuary ecological management in this area is important. In which, it is necessary to understand the dynamics of the waters so that the estuary area can be managed properly. The purpose of this study was to determine the characteristics of the welang estuary using 3D modeling. Welang river estuary influenced by tidal conditions. The current pattern that occurs in the welang estuary moves from southeast to northwest. Current velocity around the welang estuary at the surface of the water tends to be greater than at the bottom of the water. This is because of the influence of the wind. The horizontal temperature distribution tends to be uniform and vertically shows no thermocline layer at the welang estuary.

1 INTRODUCTION

The coastal region of Indonesia has a fairly extensive estuary area, one of which is on the north coast of Java, some researchers have studied related to the estuary environment in several locations, e.g., Porong, Wonorejo and Wonokromo river. (Suntoyo et al., 2015; Suntoyo and Sakinah, 2017; Sakinah et al., 2017; Pahlewi et al., 2017; Kauh et al., 2018). Welang watershed is located in Malang Regency, Regency and Pasuruan City. The Welang River empties into the Kraton Subdistrict, Pasuruan Regency which is directly related to the Madura Strait (Febriyanto et al., 2018). Welang estuary is an estuary ecosystem that has an important function as a biota habitat, such as fish. In addition, this area is used by local residents as fish and shrimp ponds. The more activities that occur in estuary areas, the area will have an influence on the quality of estuarine waters, directly or indirectly.

Until now, some informations regarding the characteristics of the welang estuary is currently limited. So the need for further research in the estuary area to be able to develop its designation, such as ports, transportation routes, tourist areas and fish farming. The characteristics of flow patterns,

temperature and salinity are important parameters in water dynamics that have an influence on water circulation (Pahlewi et al., 2017; Suntoyo and Sakinah, 2017; Sakinah et al., 2017).

Determination of estuary type is the initial stage in carrying out development planning in the estuary by taking into account its characteristics such as current, temperature and salinity. Therefore the purpose of this study is to analyze the characteristics of the waters in the welang estuary based on the parameters of current, temperature and salinity and can determine the type of estuary in the estuary of the Welang.

Current and temperature distribution were modeled by using 3-dimensional models in order to explain the distribution horizontally and vertically. Furthermore, these results was analyzed to explain the characteristics of Welang estuary waters.

2 STUDY AREA

The research location is in the Welang River estuary in the Kraton and Ketug Pasuruan Districts. Welang River itself crosses Malang Regency, Pasuruan Regency and Pasuruan City, East Java Province empties into Madura Strait.

The modeling location in this study is shown in Figure 1, where the blue area indicates the modeling area. Direct measurements consist of current, temperature and salinity carried out at 5 sampling stations with 3 different depths (0.2d, 0.6d and 0.8d). The model simulation was carried out for 6 months, from July 2019 to January 2020. Geographically, the Welang Estuary is located at 112,861°E - 112,896°E and 7,584°S - 7,590°S.

3 METHOD

The methodology used in this research, the first step is the study of literature to support the study to be carried out, followed by the collection of data which includes primary data and secondary data. Primary data were obtained by performing direct measurements in the field which included temperature, flow velocity and salinity data. Whereas secondary data was obtained from various related agencies and this data used for input simulation models such as bathymetry, tides, wind, and river discharge data. After data collection, the next step is to process bathymetry data which will become input data in the simulation of current and temperature distribution using 3D modeling where the output is current and horizontal and vertical temperature

distribution. Modeling and simulation was done using computer software with a simulation time of 30 days or 720 hours in November 2019, to obtain the good results, the model validation was conducted. Then analyze the results and finally conclusions.

4 MODEL VALIDATION

The results of the modeling of the current pattern are validated using tidal data obtained from measurement results on November 1, 2019 - December 1, 2019 at the Grati Power Plant and the current velocity measured at point St 21 (Figure 2) by using current meter. In general, the results of the model and measurement data show a trend that is equal to the MAPE (Mean Absolute Percentage Error) value of 0.35% (Figure 3). Current measurements are carried out on 14 November 2019 at 12.00-21.00 WIB within all three depths (0.2d, 0.6d, and 0.8d) using a current meter (Figure 4). The results of the current velocity from the three depths are then averaged. The results of the current validation at point St 21 show a MAPE value of 27.5%, while the RMSE value is below 1.00. Model validation is acceptable because the error is still below 30%. The validation of current velocity modeling is classified as Reasonable forecasting.

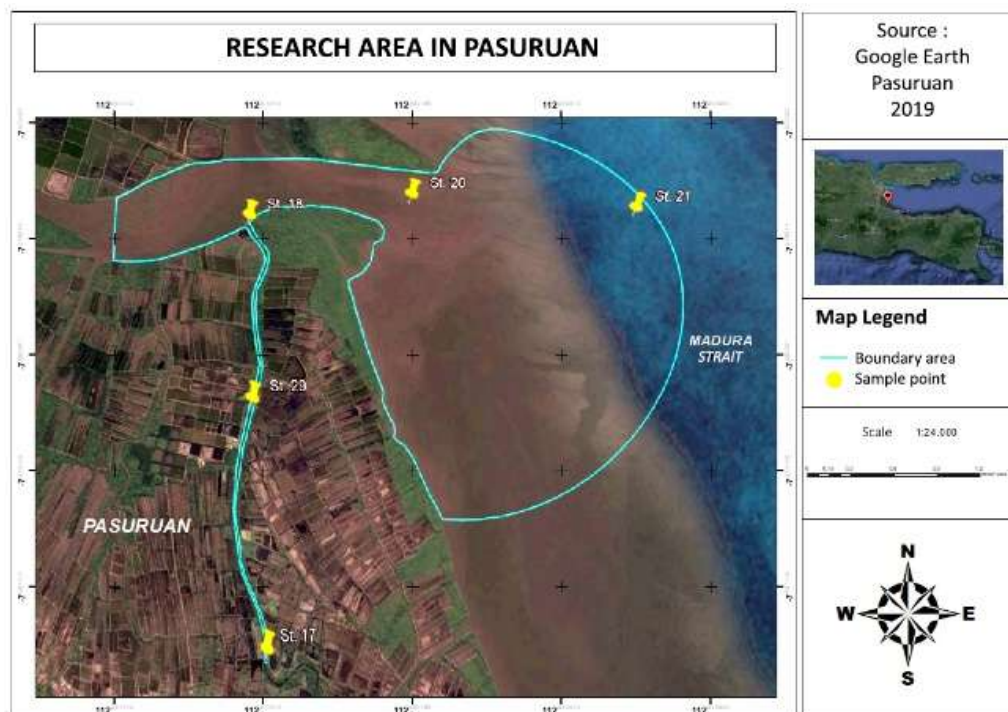


Figure 1: Study Area Including of Observational Stations (Google Earth).

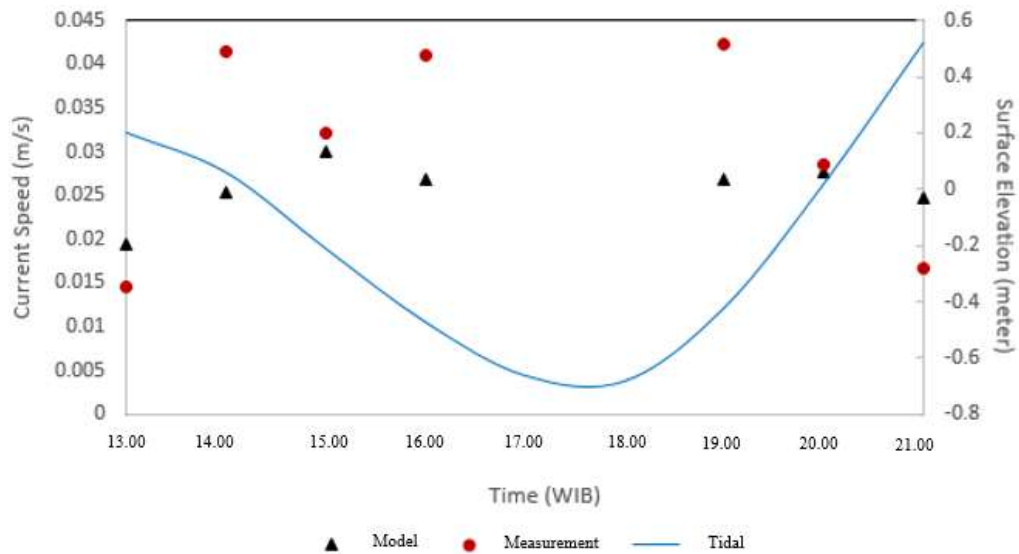


Figure 2: Validation of Current Velocity at Point St 21.

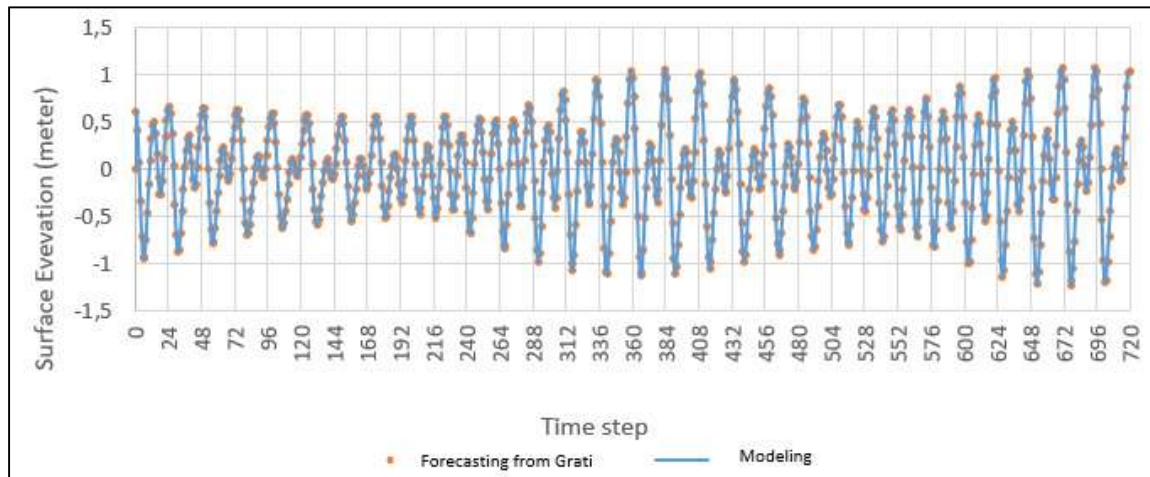


Figure 3: Comparison of Tidal Elevations between Forecasting Data and Model Results.

5 RESULTS AND DISCUSSION

The modeling results presented in this paper are the horizontal and vertical distribution of velocity and temperature. From the simulation results, the sea level with the highest tide is 1.052 m in step 384, while the lowest tide with -1.132 m in step 366. Where the highest tide conditions occur on November 17, 2019 at 00.00 WIB and the lowest tides occur on November 16, 2019 at 06.00 WIB. Based on tidal data analysis, the Welang estuary is a valid semi-diurnal tidal type (Figure 4).

At highest tide conditions, the current pattern moves from southeast to west toward the estuary,

while the current pattern at lowest ebb moves east towards the sea. The frequency of the direction current is more dominant in the west-northwest direction, this is due to the east monsoon wind pattern as well as shown by (Sugianto, 2009). The current velocity at point 18 show that the highest tide occurs between 0-0,06 m/s and the lowest tidal current between 0.015-0.105 m/s presented in Figure 5.A and 5B. The current velocity at the highest tide tends to be lower than at lowest ebb. The Reason is at high tide, seawater moves into the river and at the same time, the river water pushes upstream. At lowest ebb, currents tend to be higher because the water just

pushes out towards the river mouth and increases the current velocity.

From the distribution of current at the highest tide and lowest ebb shows that the value of the current speed at the top is greater than at the bottom of the waters. this is due to the effect of the wind coming from the southeast as well as the bottom shear stress reduced the velocity in the sea bottom (Figure 6).

In the vertical distribution, reviewed at 2 points namely point IP1 and IP2 (see Figure 7) which are around the point 18 estuary. Current distribution at the highest tide conditions (384 step), at a distance of 0-80 m from IP 1, the speed current is varies from surface to bottom waters. On the surface the current velocity ranges from 0.008 to 0.096 m/s while the current velocity at the bottom of the waters is around 0.008-0.072 m/s. at a distance of 80-220 m, uniform current velocity distribution from surface to bottom. Where the current speed is around 0-0.016 m/s (Figure 8 A.).

Current distribution at the lowest ebb condition (step 366), at a distance of 0-80 m from IP 1, the current speed is varies from surface to bottom of the waters. On the surface the current speed ranges from 0.048-0.120 m/s. while at the bottom of the waters, the current speed is around 0-0,024 m/s. At a distance of 80-220 m, the current surface velocity values are around 0-0,048 m/s and at the bottom of the waters the current speed is around 0-0,032 m/s (Figure 8.B).

At the highest tide conditions, high temperatures from the sea towards the river mouth. The temperature in the river body area ranges from 30.10-30.50 °C. Whereas in river mouths the temperature is around 30.55-30.65 °C, and at sea temperatures above 30.65 °C (Figure 9.A.). The pattern of

temperature distribution at the lowest ebb conditions shown in Figure 9.B. In the river body area, the temperature value is around 30.10-30.2 °C, while the temperature value in the river mouth area is around 30.2-30.45 °C and in the sea area the temperature value is around 30.5-30.7 °C.

Overall the temperature at highest tide in the estuary area is caused by the push coming from the sea which has high temperature. Whereas at lowest ebb the temperature in the estuary tends to be low because at ebb the flow is only influenced by the river discharger that goes to the sea, so that the temperature carried from the upstream of the river is low towards the sea gives the effect of decreasing the temperature in the estuary to the sea

The distribution of temperature at the highest tide conditions at a distance of 0-20 meters, the temperature value is around 30.58-30.61 °C. at a distance of 20-210 meters the temperature value is around 30.61-30.64 °C and the temperature value at a distance of 210-280 meters is around 30.64-30.67 °C (Figure 9.A). The distribution of temperature at the lowest ebb conditions at a distance of 0-110 meters around 30.25-30.28 °C, while at a distance of 110-220 meters the temperature value tends to vary at around 30.31-30.52 °C (Figure 9.B).

In general, the vertical temperature distribution pattern in both conditions in the welang estuary tends to be uniform from top to bottom. Although the value of the surface layer is greater than the middle and base layers. This is due to the influence of direct sunlight. Distribution of temperature from surface to base there is no thermocline layer. This is because in this area including shallow water so the temperature tends to be uniform as well as shown by (Alosairi et al. 2018).

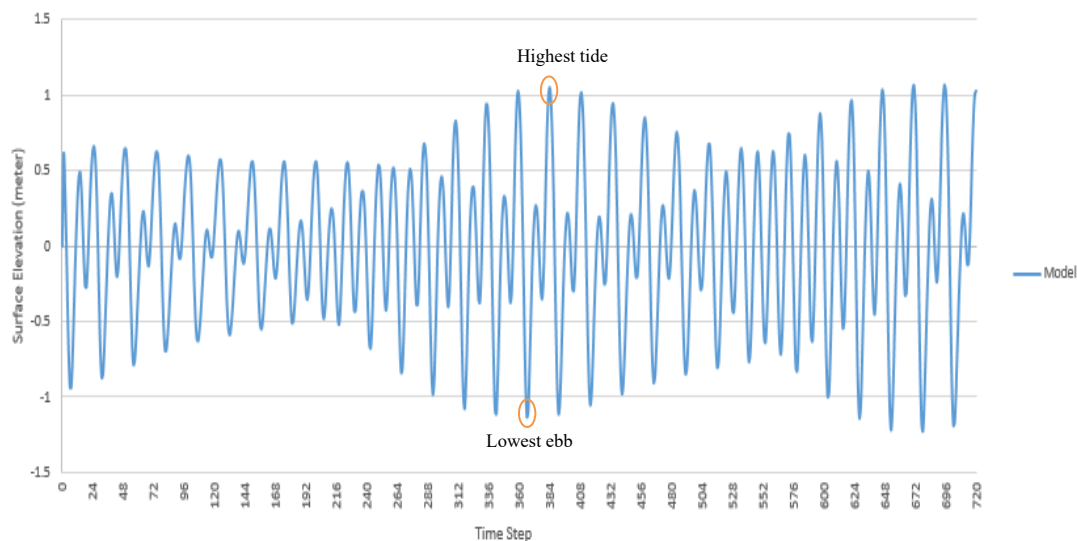


Figure 4: Water Surface Elevation.

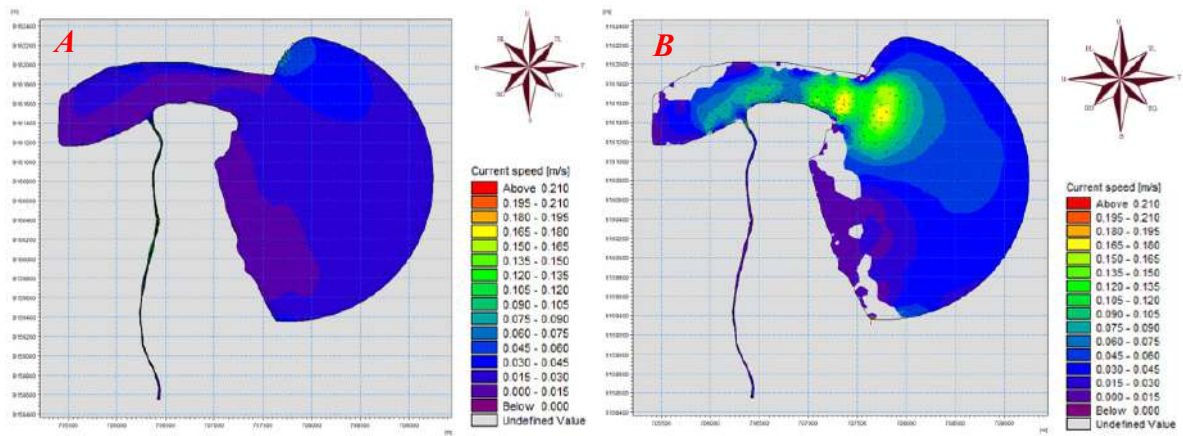


Figure 5: Horizontal Current Distribution in A: the Highest Tide, B: the Lowest Ebb.

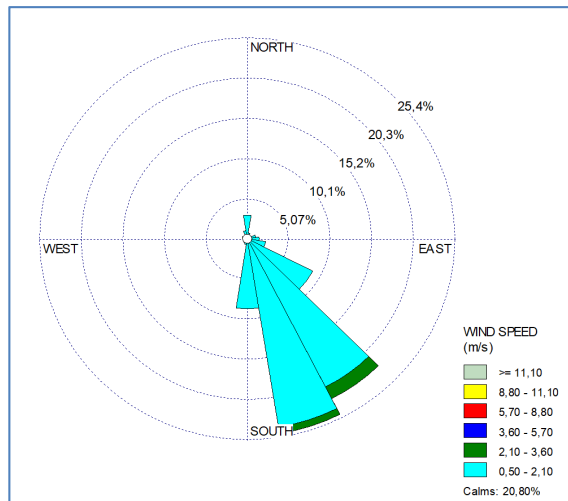


Figure 6: Wind Rose Direction and Speed Current Distribution.

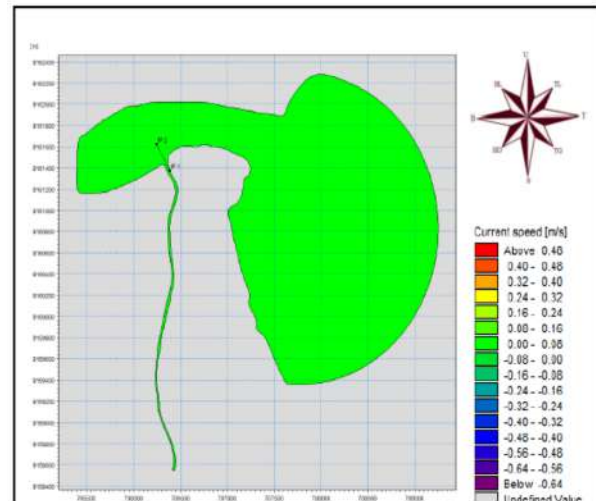


Figure 7: Position IP1 and IP2 Cross-section.

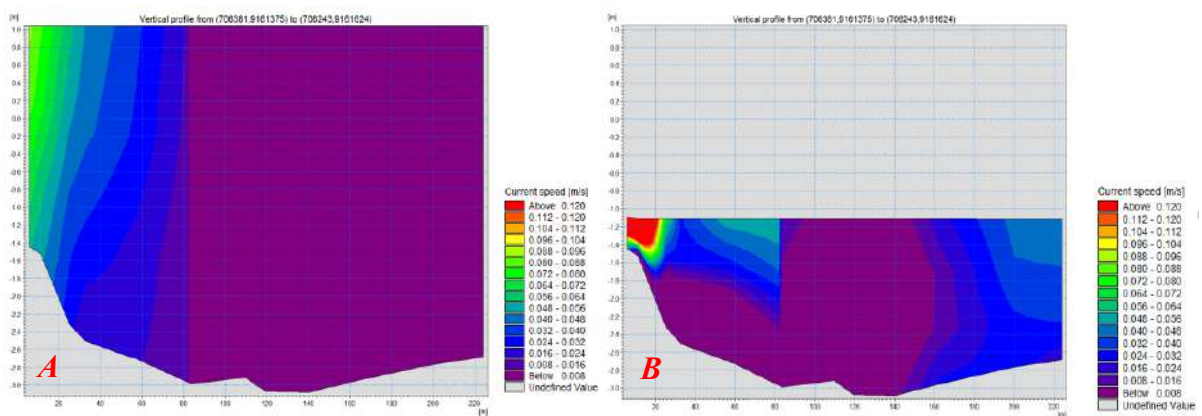


Figure 8: Horizontal Temperature Distribution in A: the Highest Tide, B: the Lowest Ebb.

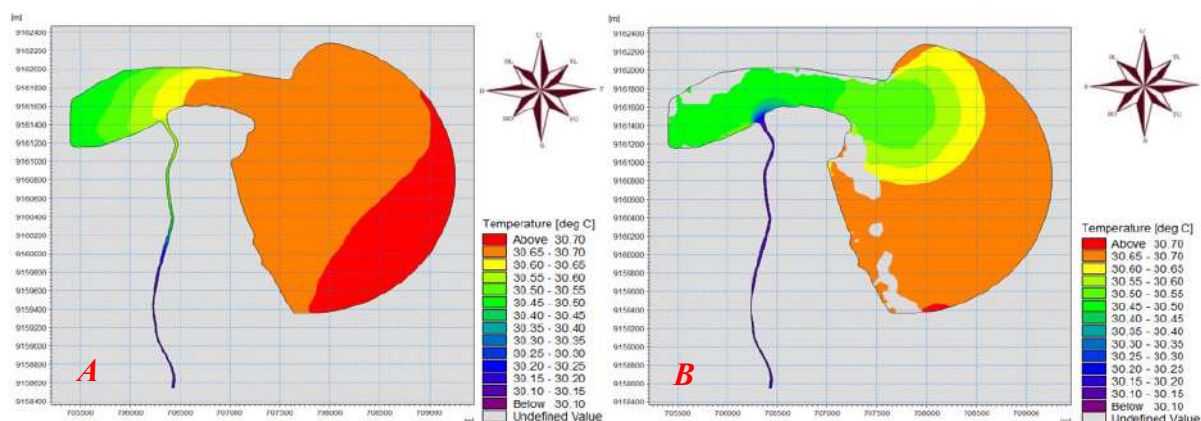


Figure 9: Horizontal Temperature Distribution in A: the Highest Tide, B: the Lowest Ebb.

6 CONCLUSION

The current pattern that occurs in the welang estuary is moving from the southeast toward the northwest. The speed current around the welang estuary at the surface of the water tends to be greater than at the bottom of the water. This is due to the effect of the wind. The horizontal temperature distribution tends to be uniform and vertically shows there is no thermocline layer in the welang estuary and the temperature distribution from the surface to the bottom does not differend so that the distribution is evenly distributed only slightly higher on the surface due to direct sunlight so that the surface is hotter compared to the bottom .

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Bathymetric Survey for Estimating the Local Scour at Suramadu Bridge

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Keywords: Bathymetri, SBES, MBES, Scouring.

Abstract: The Suramadu Bridge is the longest bridge in Indonesia, located at Madura Strait causes an increase in the speed of ocean currents as well as obstacles to seawater flow acceleration of ocean currents due to the reduce of the wet cross-sectional area of seawater flow. This influence cause changes in the seabed bathymetry along the Suramadu Bridge and the potential for local scouring on the bridge foundation. In dangerous stage of scouring, can affect the stability and carrying capacity of the foundation structure under the bridge. The Suramadu Bridge has operated since 2009, has been almost 10 years, and bathymetric surveys is needed to ensure the actual conditions on the seabed under the Bridge. The purpose of the bathymetric survey activity is to obtain the latest bathymetric data and the scouring patterns around the main pillar (P46 and P47) of the Suramadu Bridge. The Bathymetric Survey method used are SBES and MBES. The first Bathymetric survey was obtained at the year of 2016 since the bridge had operated, shows that the scouring was indicated at the main pillar the increase is about 7 – 8 m. Based on the bathimetric survey data of the 2017, shows the scouring is increased is about 1 meter below the result of bathymetric survey of 2016 at the seabed in the main bridge.

1 BACKGROUND

Many bridges around the world failed or damaged because of extreme scour around piers and abutments. Any constructed bridges must be continuously monitored for any changes in the structure of the bridges that usually focus on the changes of the soil in the area of the bridges constructed. The typical changes can be seen is from the bridge scouring or in this case (Akib, 2011).

Suramadu Bridge is located in the northern part of East Java Province, Indonesia. With an overall length of 5.4 km, it spans Madura Straits and connects Surabaya and Madura Island (Jatnika et al, 2006).

Suramadu Bridge in Madura Strait reduces the wet cross sectional area of sea water, increase the ocean current speed and acceleration. The seabed along Suramadu Bridge can be change after more than 10 years after the last study of bathymetri in 2005.

Scour is a natural phenomenon caused by the erosive action of flowing water on the bed and banks of alluvial channels. The local flow around a hydraulic structure such as a bridge pier or abutment is associated with an enhanced sediment-carrying capacity, such that scour may occur near a structure even when there is no transport of sediment away

from the structure. At a bridge site, scour around bridge piers or abutments may lead to reduced support and hence constitute a potential catastrophic hazard (Masjedi, 2010).

Scour is local lowering of streambed elevation that takes place around structures that are constructed in flowing water (Akib et al, 2014). Local scour is the removal of sediment from around bridge piers due to flowing of water. A large amount of local scour is dangerous to the bridge piers and causes the structure tend to collapse and loss of life without any warning (Ahmad et al, 2016).

Local scour around a bridge pier is largely depending on the shape of the bridge pier and how the design is fashionable from the view of construction. Local scour is a complex phenomenon which depends on the discharge, depth of flow, geometry of the pier and type of sediment particle (Roy, 2017).

Scouring depth assessment for Approach Bridge and Main Bridge of Suramadu Bridge for a 20-year return period carried out in 2005 for pier foundation P37 - P56 by Yu (2005) from the Department of Hydraulic Engineering of Tsinghua University Figure 1 and Table 1. In 2016 was the first study of Bathymetri on Suramadu Bridge by Bathymetry Consultants was hired by Ministry Of Public Work

and Housing, and the result was the local scouring 7 – 8 meters at the main pillars (P.46 and P.47). So, that to ensure that the scheme of local scouring not damaged the bridge, in 2017 the survey of Bathymetry was continued by Bathymetry Consultants was hired by Ministry of Public Work and Housing.

The location of activities was along the Suramadu Bridge corridor and around the main pillar (P46 - P47) with a corridor width of 500 m to the left and right of the bridge. Conduct a detailed bathymetry survey along the Suramadu Bridge corridor and around the main pillar (P46 - P47) with corridor width of 1 km.

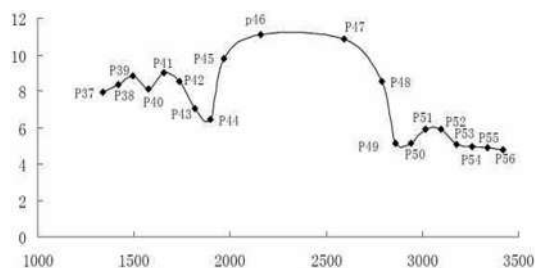


Figure 1: Maximum Scouring Depth on Pier 37 - Pier 56 of Suramadu Bridge for a 20-Year Period Based on 2005 Study Results.

Table 1: Scour depth at major piers under tide with 100-year return period.

Pier Number	Scour depth (m)	Pier Number	Scour depth (m)	Pier Number	Scour depth (m)	Pier Number	Scour depth (m)
P37	8,35	P42	8,94	P47	11,36	P52	6,16
P38	8,79	P43	7,41	P48	8,9	P53	5,28
P39	9,27	P44	6,7	P49	5,34	P54	5,19
P40	8,88	P45	10,2	P50	5,35	P55	5,09
P41	9,42	P46	11,5	P51	6,17	P56	4,99

The purpose of the bathymetry survey activity is to obtain data on the latest bathymetry conditions and scouring patterns in the main pillar of the Suramadu Bridge. In addition, this paper gives recommendation and mitigation if needed. Protecting the bridge piers against scour is a crucial step to preventing bridge failure because there is a close relationship between bridge failure and scour at bridge foundations (Tang et al, 2009).

2 METHOD

2.1 Data Collection

Bathymetry is a method or technique in determining sea-depth or seabed profile from the result of sea-

depth analysis (Anugroho et al, 2017). From late 2017 the survey using Single Beam Echo Sounder (SBES) and Multi Beam Echo Sounder (MBES). Survey location along Suramadu Bridge with corridor width of 1 km (500 meters to the right and left side of the bridge). Tide measurements carried out at these locations used for correction when carrying out bathymetry measurements.

Data processing of multibeam depth survey results was carried out using Qinsy software and single beam echo sounder was processed using Hydro pro navEdit and Terramodel software. The License of the software was licensed to Bathymetry Consultant that hired by Ministry of Public Works and Housing. And the Author was a part of Bathymetry Consultant Members.

2.2 Determination of Bench Mark (BM)

This survey started with data collection. The first data in this study is determine of Bench-Mark (BM) points as measurement reference points that represent the bridge spans and survey corridors. Mapping changes in seabed elevation along the Suramadu Bridge, especially in the area around the bridge foundation.

The navigation system used to determine the position of moving objects such as ships when conducting bathymetry surveys known as DGPS (Differential GPS). This differential correction can either be a pseudo range correction (such as RTCM SC-104) or coordinate correction. With differential correction, the coordinates obtained have relatively higher horizontal position accuracy compared to the absolute method (Abidin, 2000).

2.3 Bathymetry Survey

According to IHO (International Hydrographic Organization), the bathymetry survey is "measured or charted depth of water or the measurement of such depth". Tides also affect the survey due to sea level variations, so tidal observations needed to reduce the results of the survey on the dynamics of the seawater (Rinaldy et al, 2014).

Sounding is one of the methods of determining depth using the principle of the reflection of an aquatic wave (Qhomariyah and Yuwono, 2016). The device used for this activity is echo sounder. The using of this device is an indirect measurement of depth by measuring the travel time of acoustic wave pulses emitted by Transducer (Figure 2).

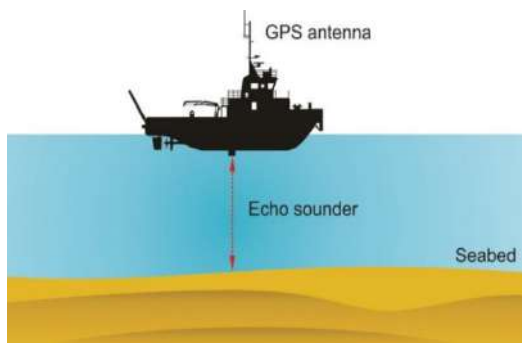


Figure 2: Bathymetri Measurement.

Bathymetry surveys use a combination of Multi-Beam Echo sounder (MBES), is a survey equipment for areas > 3 m deep and Single-Beam Echo sounders (SBES) for areas < 3 m deep with 100% coverage for locations near bridge foundations. Using Real Time Kinematic (RTK) Global Positioning System (GPS) equipment.

Bathymetry surveys carried out using a combination of Single beam Echo sounder (SBES) and Multi beam Echo sounder (MBES) equipment. Furthermore, SBES and MBES work methods and work principles explained in the following sub-chapters:

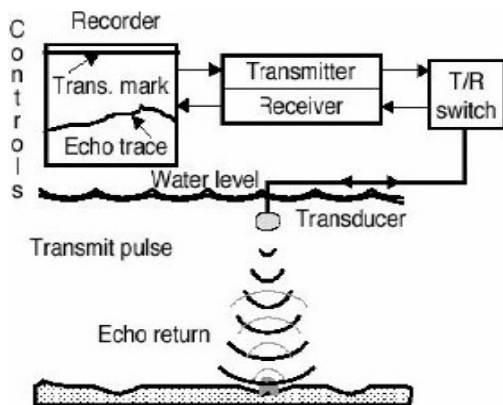


Figure 3: Echo Sounder Working Principle.

Acoustic waves with a frequency of 5 kHz or 100 Hz will maintain their intensity loss to less than 10% at a depth of 10 km, while acoustic waves with a frequency of 500 kHz will lose their intensity at depths of less than 100 m. The principle of this method is distance measurement by utilizing acoustic waves emitted from the transducer (transmitter). A transducer is a part of an echo technology device that converts electrical energy into mechanics (to generate sound waves) and vice versa. Acoustic waves travel through the water to the seabed and reflected back to the transducer (received by the receiver).

Bathymetry measurements influenced by the dynamics of seawater media in the form of tides, making it very difficult to determine the same object at different times. Thus on the measurement of the depth of the seabed it is necessary to do three measurements at the same time, namely the measurement of the depth, the measurement of the position of the depth measuring instrument and the measurement of tides.

2.4 Determine of Point Control

The control points used in the bathymetry survey activities along the Suramadu Bridge were determined using BM VKS2. The use of this control point is intended as a verification point for Differential GPS that is used as a horizontal positioning system or navigation during the survey. Here are the results obtained.

Point Name : BM VKS2
 Stipulated by : BPPT
 Location Point : Tambak Wedi Surabaya
 Geodetic coordinates
 WGS 84
 Latitude : $7^{\circ} 12' 29,201''$ S
 Longitude : $112^{\circ} 46' 40,130''$ E
 UTM Zone (Zone 49S)
 East : 696303,803
 North : 9202862,814
 Point Condition: Good

Verification of the coordinates obtained from the VSPS Hemisphere VS330 positioning system is carried out against the BM VKS2 Benchmark coordinates located on the west side of the Suramadu Bridge in Tambak Wedi. Verification was carried out on September 13, 2017 before the start of water depth measurements.

This verification is intended to ensure that the position system used, including the geodetic parameter system entered in the navigation software is in accordance with the coordinate system and the position value of the coordinates of the control points. If the difference between the coordinates is more than the tolerance required, then the equipment used must be replaced or calibration must be repeated while ensuring that there are no wrong parameters in the software used.

The DGPS antenna is placed above the existing control point, then the coordinates obtained from the position of this antenna are recorded in the navigation software that is used for approximately one hour with an observation interval every 5 seconds (Figure 3 and 4). The coordinate data obtained are then averaged

and the results are compared with the coordinates of the existing control points.



Figure 4: Determine Point Control.



Figure 5: Output of Determine Point Control.

2.5 Singlebeam Echosounder

Single beam echo sounder is a measurement device that uses a single beam as the sender and receiver of sound wave signals. The working principle of SBES is to use the principle of measuring the phase difference in pulses, which is calculating the time difference from the time of the emission and reception of the acoustic pulses. SBES is also quite accurate, where SBES is able to provide accuracy to 0.1 meters at depths of less than 100 meters (Lekkerkerk et al, 2006).

Bathymetry systems using a single beam generally have an arrangement: a transducer (transmitter/receiver) mounted on the hull or the bearing side of the ship. This system measures the depth of water directly from the inquiry vessel. Transmitter mounted on the hull sends acoustic pulses with high frequency contained in the beam (sound waves) directly down the water column. The receiver will capture acoustic energy emitted from the transmitter.

The data obtained from the process is the time interval the waves begin to emit and the waves received again, so that the depth data obtained by the recording device is a function of the time interval. The process described in the equation (Poerbandono et al, 2005):

$$d = \frac{1}{2} (v\Delta t) \quad (1)$$

Where :

- d : see water level (m)
- $v(t)$: sound wave velocity (m/s)
- Δt : interval (s)

Transceiver consists of a transmitter that has a function as a control of the wavelength of the emitted pulse and provides electrical power for a given frequency and the receiver receives repeatedly reflected waves at high speeds, up to the order of the speed of the millisecond. Continuous water depth recording from under the ship results in a high-resolution depth measurement along the surveyed lane.

The single beam echo sounder used to make 3D combined with the physical surface of the base location, the physical surface of the seabed location, which used to conduct preliminary surveys before using multiband sonar. Single beam echo sounder consists of two types, namely:

- a. Single frequency is a single echo sounder that uses only one frequency, namely high frequency.
- b. Dual frequency is a single echo sounder that uses two frequencies, namely high frequency and low frequency. High frequency provides more accurate depth in relation to shipping safety, while low frequency is able to penetrate into the seabed mud deep) so it is not safe for shipping.

The frequency range used in this system according to the WHSC (Woods Hole Science Centre) Sea-floor Mapping Group operates the frequency range from 3.5 Hz to 200 kHz. Single beam echo sounder is relatively easy to use, but this device only provides depth information along the track line traversed by the ship. This activity the use of Single beam Echo sounder (SBES) for areas that covered by Multiband Echo sounder (MBES) because, the ship is less than the minimum MBES operation.

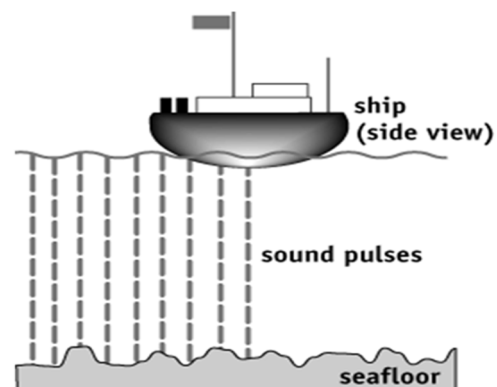


Figure 6: Bathymetry SBES Method.

Correction for bathymetry measurements by the SBES method with the following equation:

$$D = Du + s - h + BC \quad (2)$$

Where :

D : *definitive of hight*

Du : *echosounder water level measuring*

s : *transducer*

h : *water level hight*

BC : *barcheck correction*



Figure 7: Echo sounder SBES Filed Survey.

2.6 Multibeam Echosounder

Multi beam Echo Sounder (MBES) is a used in seafloor mapping along with a Single Beam Echo Sounder (SBES). MBES can measure a wider range of water depths and can acquire high resolution images simultaneously through the beamforming process. (Jung et al, 2017).

Multi beam sonar (MBS) technology developed to examine in detail large stretches of the seafloor surface, providing accurately positioned and excellent 2-D and 3-D images of features as small as a few centimetres or covering areas as large as hundreds of square meters (Stanley et al, 2011).

The principle of operation of this device based on the sound beam (beam) that emitted and reflected directly toward the seabed, and captured again through sensors that are on the MBES device. The detection algorithm on the seabed, to determine the depth and distance of the transversal, calculates the two-way propagation time between sending and

receiving. Multi beam Echo sounder can produce bathymetry data with high resolution.

Unlike the side, scan sonar beam pattern that owned by MBES it widens and crosses the hull. Each beam emits one sound pulse and has its own reception. When the ship moves, the MBES sweep results in an area of the seabed surface area (Moustier, 2005).

The transducer contained in the multi beam sonar consists of a series of elements that emit sound pulses at different angles. Usually only one beam is transmitted but it produces a lot of reflected energy from each of the transmitted sound pulses. The ability of each transducer element to reclaim the reflected sound pulse depends on the calibration method of the ship's motion applied.

This MBES has excellent accuracy in measuring depth. In addition to having excellent accuracy, the advantage of using this MBES is a wide range of measurement areas (Moustier, 2005). Coverage of the seabed area that surveyed by ships using multi beam in one sweep is called swath. The swath width for each type of multi beam echo sounder can vary.

The principle of MBES is roughly the same as measurement devices that use acoustic waves, such as SBES and sonar side scans, which is simply by emitting acoustic waves through its reflected wave transmitter.

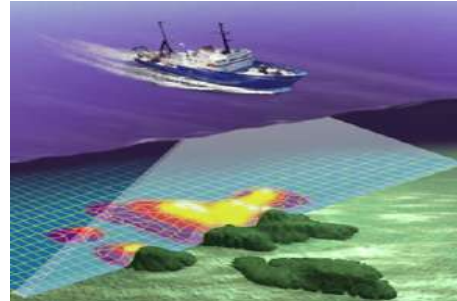


Figure 8: Bathymetry MBES Method.

Then measure the time difference between the waves when the initial emitted until the reflection of the wave is captured again by the sensor. With the previously determined speed of sound propagation in the study area, so that it can be calculated the depth of the seabed.

Transducer configuration is a combination of several projectors arranged like an array (matrix). The projector is part of the MBES transducer which functions as a channel to emit acoustic pulses towards the seabed. All acoustic signals from all directions reflecting objects on the seabed will be received again by the hydrophone array. All acoustic signals will be

received by each hydrophone simultaneously with a difference in time of reception of $4/3$ to $8/3$ mill seconds depending on the length and duration of the acoustic signal.

In general, MBES uses interferometric techniques to detect the direction of reflected waves as a function of time. By using the accumulation of acoustic signals received on two separate arrays, an interference pattern will be formed.

Based on the existing relationship a direction will be determined. When this information is combined with distance, depth data will be generated (Sasmita, 2008). The geometrical image of the acoustic waves emitted by MBES can be seen in Figure 7.

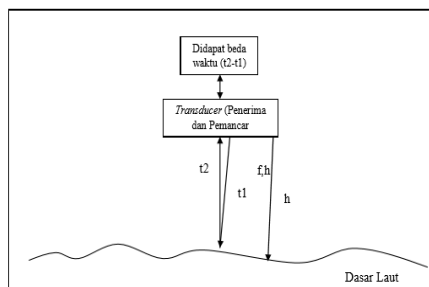


Figure 9: Transducer Mechanism.

The phase difference of pulses in MBES means that it is a function of the difference between the transmitting and receiving time phases. Then the calculation of the travel time and direction of the beam angle of each stave is determined from the measurement of the MBES pulse phase difference.



Figure 10: Echo sounder MBES Field Survey.

3 RESULT

The results of the bathymetry survey obtained around the seashore can be concluded as follows:

- The area around Surabaya was surveyed as far as 1.7 km in front of the beach. The area surveyed is in accordance with the planned area in the 500 m corridor to the left and right of the Suramadu Bridge.
- According 2016 bathymetry data Table 2, that the water sea level was 7 – 8 m.
- In the results of the cross section overlay data for 2016 with 2017 across the position of Pillar 46 also obtained changes in the form of sedimentation and scouring at certain spots. The part that forms the ridge or embankment tends to occur sedimentation while the lower part occurs scouring. Sedimentation values that occur along cross sections 500 west and 500 east are 0-0.7m per year and the highest sedimentation rates are seen in the cross section of STA 575 and 600. While the scouring value is 0 - 0.9 m per year especially in the STA 700 area up to 825.

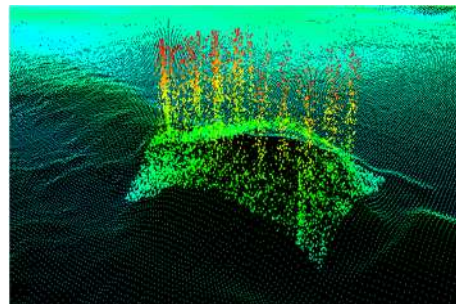


Figure 11: Seabed in P46 Pillar.

- When looking at changes in the results of the cross section on Pillar 47, the back of the west and east pillars is very visible in the 3D image as shown below, where changes occur in sedimentation at the top of the embankment and scouring at the bottom of the embankment such as dark appearance on the underside of the embankment. The value of this change varies with the sedimentation value of 0 -1 m per year as seen in the cross-section and sedimentation 0 - 1.1 m per year.

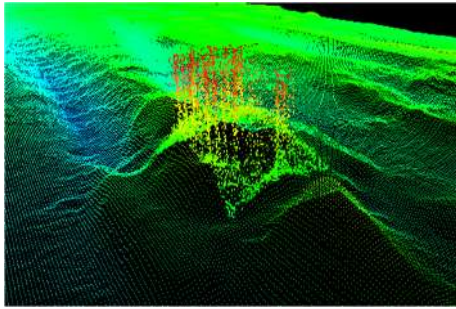


Figure 12: Seabed in P47 Pillar.

Besides the seawater depth map, from measurements with Multi beam echo sounder, you can also see the profile of the seabed surface around Pillars P-46 and P47. The condition of the profile seen in the 3D image below.

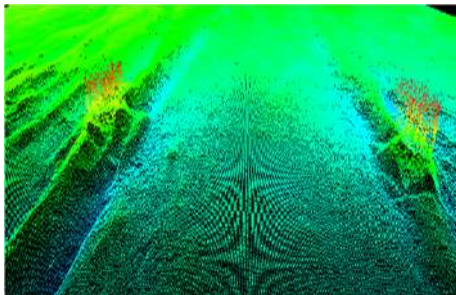


Figure 13: 3D Image Sea bed in Main Bridge.

Table 2: Bathymetry Data.

No. Of Pillars	Preliminary data	Design	2016	2017
46	-19,995	-31,495	-27,495	-28,395
47	-15,600	-26,960	-23,100	-24,200

From the table above presented in the graph, in Figure 13. From the data obtained from preliminary data, bathymetry survey data in 2016 and 2017, which can explain that scouring, occurred in the main span until 2016 occurred at 7-8 meters, but in 2017, the scouring has stabilized result 0 – 1 meters.

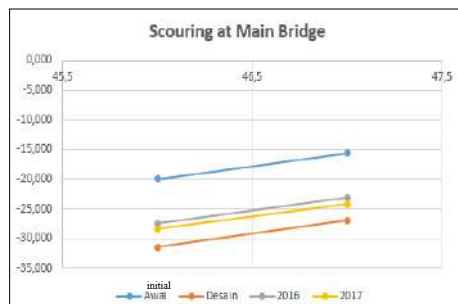


Figure 14: Scouring Graphic in Main Bridge.

4 CONCLUSION

The survey results are also presented in the form Point Cloud with a grid size of 1.0 m. All depth data use the MSL reference so that it will be commensurate when compared to 2016 measurement data that use the same reference. The maximum water depth that measured in the corridor of the survey area is 22.8 m and this depth find to the west of the Main Pillar of P-47.

The water depth of 22.8 m is in the elongated scouring area that occurs along the southern part of the P-47 pillar where this scour is parallel to the sedimentation, which is located side by side in the northern part of the P-47 pillar. Underneath there is a scoring direction to the west of Pillar 47.

The depth of water between the main pillars to a distance of 300 m from the bridge ranges from 15 to 22 m with a composition that is almost evenly distributed and regularly in the West and East. Sedimentation occurs forming embankments longwise to the west and followed on the main pillars that range between 0 - 1 meter from the embankment which can be seen clearly on the appearance of 3D images.

5 RECOMMENDATION

To ensure that the local scouring is not gives damage to the bridge in the further, bathymetry survey has to periodically, that level of local scouring can be estimated as reference to maintenace the bridge.

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Analysis of Soil Retaining Wall from Geosynthetic for Sea Reclamation to Become a New Land in the Beach

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Keywords: Reclamation, Retaining Wall, Geosynthetic, Geobag.

Abstract: Reclamation is a business to get land that was covered by water into dry land and to be used as useful land in accordance with its purpose. This reclamation can be carried out on lands that are on the coast, swamps, and offshore. In this study, land reclamation is intended as a port development area so that land requirements are in accordance with the port development plan on the coast. The location of this research is in the southern area of Ketapang Regency, West Kalimantan, which is 18 km from the Ketapang City Center. This regency does not yet have a coastal port, all of its ports are in the river, which certainly often occurs siltation due to sedimentation. Land reclamation is carried out to pursue the depth of the waters in accordance with the berthing of ships to be anchored so that no trestle construction is needed but by reclaiming it, it can save the cost of building the structure. The research method used in this study, by analyzing wind data as a reference for wave generation and calculating the wave deformation that occurs, bathymetry data as a reference for water depth, tidal data is intended as one in the design of the retaining wall height, estimated loading above the reclamation structure land, and the external forces that occur on the retaining wall. The design of the retaining wall uses geobags in a stacked manner, has a total height of 6.3 m buildups from sea level to the deepest seabed and forms a trapezoidal structure with a trapezoid peak width of 5.09 m (3 geobag lines) and a trapezoid base width 26.47 m (11 lines/stack). The size of Geobag 2 x 1.5 x 0.6 m. Geobag size is obtained from the calculation of the effect of waves on the geobag structure. So that the weight is determined by the required volume size and then the size is further reviewed by the effect of soil pressure due to the embankment for reclamation held by the geobag structure by analysis of overturning, sliding, and bearing capacity permitted exceeded.

1 INTRODUCTION

1.1 Background

Ketapang Regency is one area that has abundant natural resource potential, as a support in the distribution of goods, this area needs a port for goods and passengers loading and unloading facilities. This Regency actually has a port on the Pawan River, but as time goes by, the shipping lanes to and from the port experience siltation due to sedimentation. This area needs a new port that is no longer in a river but on the coast.

Ketapang Regency itself in general has a fairly long coastline, but to be used as a port with a specific ship weight needs to be modified beach conditions.

Based on the results of a feasibility study in 2012 with 3 port locations, the selected ones were located in the southern area of Ketapang Regency (KRTD, 2012) but in those areas they have shallow water depths, to obtain appropriate water depths, about ± 1 km from the shoreline. So in this study, to solve the problem we use the option to reclamation the portland because the cost is more economical and materials will be taken from nearby locations.

In this reclamation, geosynthetic is used as one of the structural planning materials. Geosynthetic used is the type of geobag. Geobag is used as a retaining wall of land reclamation. The geobag material used is from NAUE Germany whose geobag output material is SECUTEX®H or SECUTEX® SOFT ROCK

which is a special fiber nonwoven geotextile for hydraulic engineering.

1.2 Formulation of the Problem

1. How to determine the dimension of retaining wall?
2. How to determine the dimension of geobag?
3. How does the influence of the stability of the external forces of the geobag as a retaining wall of land reclamation?

1.3 Restricting the Problem

1. Land area for land reclamation plan is 372.417,5 m x 389.011 m
2. Land reclamation location in in the southern area of Ketapang Regency, West Kalimantan, which is 18 km from the Ketapang City Center
3. Planning a retaining wall in the form of geosynthetic geobag types using data:
 - a. Wind data was obtained from BMKG Ketapang in the years 2004 - 2013
 - b. Bathymetry data of land reclamation area with the direct survey to the field as a reference for water depth
 - c. Tidal data was obtained from a 15-day field survey as a reference for the reclamation land retaining wall height
 - d. Soil type data were obtained from field samples and examination tests at the Soil Mechanics Laboratory, Faculty of Engineering, Tanjungpura University, as a reference to soil types in filling geobags
4. Does not calculate the strength of the external geobag material used

2 THEORY

2.1 Reclamation

Reclamation is a job/business utilizing areas or land that are relatively useless or still empty and watery into useful land by draining. For example in coastal areas, swampy areas, offshore / in the sea, in the middle of a wide river, or in a lake. Reclamation is carried out following the procedures since the planning stages (pre), implementation and development (process), and its use (post) both above and or below the reclaimed land (PWD, 2008).

The reclamation of large amounts of land in the future may be needed to meet the food and adequate space requirements of the developing population.

Land reclamation for housing, industry, or recreation requires ecological, hydrological, and geological data in addition to technical analysis. Geologists can contribute to the planning of reclamation of floodplains, wetlands, landmines, and geologically dangerous areas. Geological data can be used to predict the environmental impacts deposited by a reclamation project (McComas, 1972). From a commercial perspective, reclamations have been used for airport expansions and new airports as well as port expansions and new ports. Residential and recreational developments along waterfronts have been successfully constructed on reclaimed land (Duan et al, 2016).

2.2 Retaining Wall

Retaining walls are structures used to provide stability for earth or other materials at their natural slopes. In general, they are used to hold back or support soil banks and water or to maintain a difference in the elevation of the ground surface on each of the wall sides. Also, retaining walls are often used; in the construction of buildings having basements, roads, or bridges when it is necessary to retain embankments or earth in a relatively vertical position. Retaining walls are commonly supported by soil (or rock) underlying the base slab, or supported on piles; as in case of bridge abutments and where water may erode or undercut the base soil as in waterfront structures.

The conventional retaining walls can be divided into four types, gravity, semi-gravity, cantilever, and counterfort retaining walls. Stability analysis of retaining wall is influenced by loads which burden the structures. This load in the mechanic analysis can be known as force. This force mainly can be divided into two kind, lateral pressure and vertical pressure. Lateral pressure itself comprises of soil and water pressure. There are two types of lateral soil pressure, namely active and passive pressure. The active pressure commonly located behind the retaining wall as the slope which wants to retain. On the other hand, the passive pressure in front of the retaining wall is an additional force on the retaining wall to maintain the slope from collapse. If there is no soil in front of the retaining wall, there will not any passive pressure. Moreover, the lateral pressure also includes water pressure if there is water existence in slope. Furthermore, the vertical pressure of retaining wall structure is influenced by the load above the slope (Sari et al, 2020). Design Considerations of Gravity as follows ; Scope, Limit State Design, Partial Loading and Material Factors, Load Combinations

and Factors for Stability, Load Combinations and Factors for Strength of Components, Live Loads, Earthquake Loads, Wind Loads, Hydraulic Loads, Drained v Undrained Parameters, Capacity Reduction Factors, Soil Analysis Model, Active Pressure, Pressure at Rest, Passive Pressure, Bearing Failure, Sliding Failure, Overturning, Global slip, Foundation Material, Lean Back (CMAA, 2013)

2.3 Geosynthetic

Geosynthetics can be defined as planar products manufactured from polymeric material, which are used with soil, rock, or other geotechnical engineering-related material as an integral part of a manmade project, structure, or system. Geosynthetics are widely used in many geotechnical, environmental, and hydraulic applications related to groundwater quality and control. Geosynthetic types include ; Geotextiles, Geomembranes, Geogirds, Geosynthetic Clay Liners, Geocomposite Sheet Drains, Geocomposite Strip (Wick) Drains, Geocells. (Zornberg and Christopher, 2007).

NAUE geosynthetic is geosynthetic materials from Germany (NAUE, 1967). The innovations of NAUE are leading the geosynthetic industry into the future. We offer project-specific product development, geotechnical engineering support, and provide solutions to complicated challenges, simplifying your project. Geosynthetic types, among others:

- Geosynthetic Clay Liner Bentofix® – the “Bentonite liner”
- Secutex® multifunctional geotextile
- Geomembrane CARBOFOL® the HDPE liner
- Secugrid® geogrids for soil stabilisation and reinforcement
- Combigrid® geogrids with integrated nonwoven geotextile component
- Secumat® erosion control mat
- SECUDRAIN® WD
- SECUTEX® SOFT ROCK

2.4 Geobag

Geotextile bag or geobag, a geosynthetic product made of polyester, polypropylene or polyethylene has been used world wide for protecting riverbanks and hydraulic structures from severe scouring and erosion. The use of sand filled geobags underwater in erosion protection work is being popular due to its cost-effectiveness, availability, and easier implementation. Some recent studies support that its

technical efficiency is similar or sometimes better than the use of hard material (Wahed et. al, 2011).

A durable and effective alternative is a scour protection system with sand-filled geotextile containers Secutex® Soft Rock. The dynamic interaction between waves and waterfront soils and structures complicates hydraulic engineering. Beachfront requires longterm protection, and the solutions must be flexible, durable, and minimize the impact on marine environments. Lightweight, robust nonwoven geotextiles enable these engineered installations to encapsulate sand (including local fill) for the creation of long-term performing bags, containers, and tubes. The impact of a sand container solution made of Secutex® Soft Rock with respect to the lifecycle consideration is only a fraction of the impact of a conventional solution in an offshore scours protection system. Secutex® nonwovens are filter-stable and extremely robust. A special two-layered nonwoven composite is available, which features an integrated surface protection layer made from rough fibers for exposed or covered designs. The available sizes and mass per unit area of Secutex® nonwoven geotextile containers enable flexibility in infill selection and installation methods. Bag size and arrangement depending on the application and expected hydrodynamic conditions. Secutex® Soft Rock standard design approaches are available. Secutex® Soft Rock Applications:

- Sea walls, beach and dune revetments
- Groynes
- Submerged breakwaters and jetties
- Scour protection for waterfront structures, walls, bridge piles and offshore wind turbines
- Offshore cable protection
- Erosion control in flowing waters
- Filling of washed out material in dams
- Soil stabilisation in dams (NAUE, 1967)

3 METHOD

1. Analyze wind data on land from BMKG of Ketapang Regency Airport:

- a. Calculate wind speed at 10 m elevation

$$U_{(10)} = U_y \left(\frac{10}{y} \right)^{1/7} \quad (1)$$

- b. Calculate the wind factor
- c. Calculates the percentage of wind direction and is made into windrose
- d. Calculating Wind Tension Factor

The wind tension produces a wave generator at sea with a comparative approach to existing wind data on land (converting wind on land to wind at sea):

$$U_A = 0,71U^{1,23} \quad (2)$$

e. Calculating Effective Fetch (Feff)

$$F_{eff} = \frac{\sum X_i \cos \alpha}{\sum \cos \alpha} \quad (3)$$

where :

F_{eff} = Fetch average effective

X_i = the length of the fetch segment measured from the wave observation point to the final of the fetch

α = deviation on both sides of the wind direction, using an increase of 6° to an angle of 42° on both sides of the wind direction

f. Calculate the wind reset period

g. Calculate periods and wave heights in the deep sea

2. Analyze Wave Deformation

a. Calculate the value of H_0

Determine H_0 based on H_s obtained from previous calculations.

b. Calculate wave period (T)

Determine the Period (T) also based on previous calculations that have been carried out a repeat period with the approximating method

c. Calculate the length of a wave in the deep sea (L_0)

$$L_0 = 1,56T^2 \quad (4)$$

d. Calculate the value of the coefficient of refraction (K_r)

$$K_r = \sqrt{\frac{\cos \alpha_0}{\cos \alpha}} \quad (5)$$

e. Calculate the value of the Shoaling coefficient (K_s)

$$K_s = \sqrt{\frac{n_0 L_0}{nL}} \quad (6)$$

f. Calculate wave height (H'_0)

$$H'_0 = K_s \cdot K_r \cdot H_0 \quad (7)$$

The concept of sea wave height in this equivalent is used in the analysis of breaking waves which include the height and depth of the breaking waves

g. Calculate the slope of the seafloor (m)

$$m = \frac{s}{d} \quad (8)$$

where :

s = distance from deep shoreline

d = deep sea

h. Calculate the breaking wave height and depth on formula and graphs

$$H_b = \frac{d_b}{1,28} \quad (9)$$

3. Analyze the average high and low tide for 15 days and plot it into the elongated figure, according to the area of land and the depth of the waters from the bathymetry survey results.

4. Analyzing load on land reclamation

5. Analyzing the dimensions of the geobag

a. The weight of the geobag content, using the calculation of the stability of the protective stone with the Hudson formula

b. The geobag dimension uses the volume weight formula with a trial and error process

Wall height is known from the depth of the waters to be reclaimed to the height of the original land. The slope of the retaining wall is adjusted to the determination of the contents of the geobag grains against waves. Geobag heaps from the bottom of the waters to the height of the plan. Can be the width of the top (a) and bottom width (B), the retaining wall of the soil. Apply to AutoCAD to make it easier to adjust the installation/stacking of geobags.

6. Analyzing Soil Pressure around the site

a. Calculate the coefficient of active soil pressure (K_a) (Bachtar, 2010) :

$$K_a = \tan^2 \left(45 + \frac{\phi}{2} \right) \quad (10)$$

b. Calculate soil pressure due to load on the structure

$$\sigma = q \cdot K_a \quad (11)$$

c. Calculate soil pressure without ground water level

$$\sigma = \gamma_d \cdot H \cdot K_a \quad (12)$$

- d. Calculate soil pressure for ground water level

$$\sigma = \gamma_s \cdot H \cdot K_a \quad (13)$$

- e. The active forces on retaining walls:

1. Due to the load on the structure (distributed load)

$$P_a = H \cdot \sigma \quad (14)$$

2. Due to the presence and absence of ground water level

$$P_a = \frac{1}{2} H \cdot \sigma \quad (15)$$

- f. The force center point:

1. Due to the load on the structure (distributed load)

$$\frac{H}{2} \quad (17)$$

2. Due to the to the presence and absence of ground water level

$$\frac{H}{3} \quad (18)$$

- g. Calculate the height and distance at each midpoint of each geobag layer from the base to the top and determine the number of geobags of each layer.

- h. Calculate each force in each layer, in each plane due to soil pressure. The force due to the load on the structure (distributed load)

$$F = P_a \quad (19)$$

7. Analyze the stability of external forces (Bachtiar, 2010) :

- a. Overturning

$$FS_{\text{overturning}} = \frac{\sum M_R}{\sum M_o} \geq 2 \quad (20)$$

- b. Sliding

$$FS_{\text{sliding}} = \frac{(\sum F_R)}{\sum F_d} > 1,5 \quad (21)$$

- c. Bearing Capacity Failure

$$FS_{\text{bearingcapacity}} = \frac{q_{\text{ult}}}{q_{\text{maks}}} > 3 \quad (22)$$

4 RESULT AND DISCUSSION

The results of the image as an area that will be reclamation in the form of breadth and depth of water. The area to be reclamation is adjusted to the land plan for multipurpose buildings, warehouses, offices and container land, which is 37,241, 75 m x 38,885 m. The depth of the waters at the far end of the land area is approximately 6 m below the highway elevation. (Figure 1)

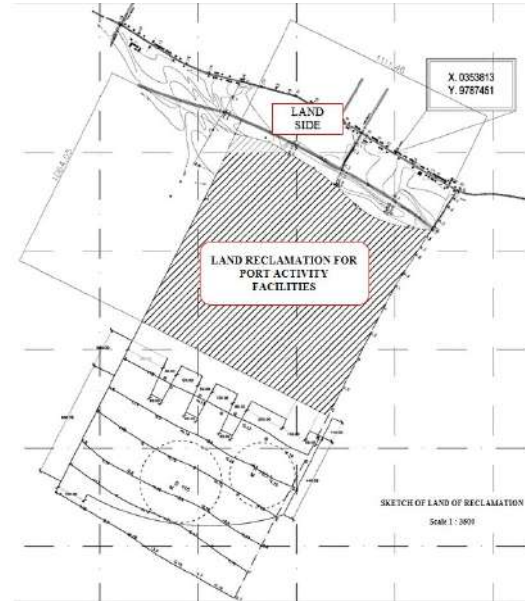


Figure 1: Land Reclamation Plan.

4.1 Wind Data

Analysis of wind data on land from BMKG Ketapang Airport with a height of 9 m above sea level. In planning structures at sea must use wind data above the sea. Therefore, the data is reprocessed to get the largest wind speed direction in the percentage scale, effective fetch, period, and wave height from the deep sea (Figure 2).

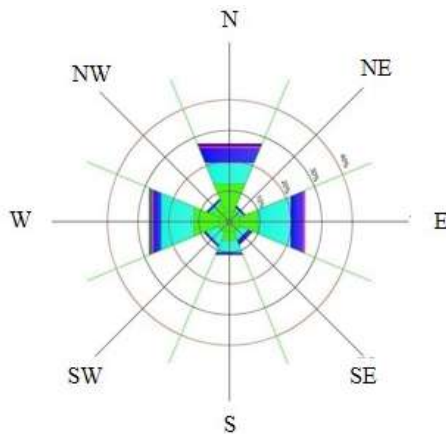


Figure 2: Windrose.

Table 1: Percentage of Speed and Direction of Wind.

Direction	North	North East	East	South East
0 - 10	0,00%	0,00%	0,00%	0,00%
10 - 20	10,00%	0,83%	6,67%	0,83%
20 - 30	7,50%	0,83%	11,67%	2,50%
30 - 40	5,00%	0,83%	4,17%	1,67%
40 - 50	0,83%	0,00%	0,83%	0,00%
50 - 60	0,00%	0,00%	0,00%	0,00%
60 - 70	0,00%	0,00%	0,00%	0,00%
70 - 80	0,00%	0,00%	0,00%	0,00%
80 - 90	0,00%	0,00%	0,00%	0,00%
90 - 100	0,83%	0,00%	0,83%	0,00%
Σ	24,17%	2,50%	24,17%	5,00%
South	South West	West	North West	Σ
0,00%	0,00%	0,00%	0,00%	0,00%
3,33%	0,83%	9,17%	3,33%	35,00%
4,17%	4,17%	11,67%	0,83%	43,33%
0,83%	0,83%	2,50%	0,83%	16,67%
0,00%	0,00%	0,83%	0,00%	2,50%
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,83%	0,00%	0,83%
0,00%	0,00%	0,00%	0,00%	0,00%
0,00%	0,00%	0,00%	0,00%	1,67%
8,33%	5,83%	25,00%	5,00%	100,00%

So, the biggest wind direction comes from the West by 25%, after that from the north and east by 24.17% as given in Table 1 and Figure 2).

Measurement of wind data is carried out on land at the airport. The measurement of the wind is not yet suitable for wave forecasting, then the wind data needs to be corrected. After that, it is converted into a wind stress factor every year as given in Table 2.

Table 2: Wind Tension Factor Average (knots).

No	Year	X (U_A)
1	2004	25,02
2	2005	23,32
3	2006	35,92
4	2007	20,32
5	2008	19,91
6	2009	23,87
7	2010	27,83
8	2011	22,00
9	2012	30,65
10	2013	27,28

The determination of waves with certain return periods requires wave data in a long enough measurement period. Representative data for several years of wind observation can be used to estimate the expected wave is equaled or exceeded once in T years and the wave is known as the return wave period, T or annual wave period, T (Pratikto et al., 2014). There are 3 methods used to calculate the wave return period, namely Normal, Gumbell and Log Person distribution as given in Table 3.

Table 3: Wave Return Period.

	2 year	5 year	10 year	
Normal	25,61	29,78	31,96	Knots
Gumbell	4,27	9,99	13,78	Knots
Log Person Type III	16,04	3,04	1,72	Knots
Average (U_A)	15,31	14,27	15,82	Knots
Average (U_A)	7,87	7,33	8,13	m/d
Duration	4	4	4	jam
Wave Height (H_s)	0,65	0,59	0,68	m
Wave Period (T)	3,75	3,55	3,75	s
Fetch	27	26	27	km
Wave Length (L_0)	21,97	19,69	21,97	m
d/L_0	0,8194	0,9143	0,8194	m

4.2 Wave Deformation

Calculation of wave deformation is done to get the wave breaking from the deep waters towards to the shoreline, the calculation results is given in Table 4, Table 5 and Table 6 for the 2 years, 5 years and 10 years return periode prediction, respectively.

Table 4: Wave deformation calculation results for the 2 years wave return period.

d(m)	H_0 (m)	T(s)	L_0 (m)	Kr	Ks	H_0 (m)	m(slope)	H_0/gT^2
20	0.65	3.75	21.98	1.00	1.00	1.54	0.001	0.011
18	0.65	3.75	21.95	1.00	1.00	0.65	0.093	0.005
16	0.65	3.75	21.91	1.00	1.00	0.65	0.074	0.005
14	0.65	3.75	21.86	1.00	1.00	0.65	0.056	0.005
12	0.65	3.75	21.78	1.00	0.99	0.65	0.186	0.005
10	0.64	3.75	21.61	1.00	0.98	0.65	0.048	0.005
8	0.62	3.75	21.24	0.99	0.95	0.65	0.053	0.005
6	0.59	3.75	20.39	0.98	0.90	0.65	0.085	0.005

Table 5: Wave deformation calculation results for the 5 years wave return period.

d(m)	H ₀ (m)	T(s)	L ₀ (m)	K _r	K _s	H ₀ '(m)	m(slope)	H ₀ ''/gT ²
20	0.59	3.55	20.00	1.01	1.01	1.71	0.001	0.014
18	0.60	3.55	20.00	1.00	1.00	0.60	0.093	0.005
16	0.60	3.55	20.00	1.00	1.00	0.60	0.074	0.005
14	0.60	3.55	19.99	1.00	1.00	0.60	0.056	0.005
12	0.60	3.55	19.98	1.00	1.00	0.60	0.186	0.005
10	0.60	3.55	19.93	1.00	0.99	0.60	0.048	0.005
8	0.59	3.55	19.76	1.00	0.97	0.61	0.053	0.005
6	0.57	3.55	19.22	0.99	0.92	0.62	0.085	0.005

Table 6: Wave deformation calculation results for the 10 years wave return period.

d(m)	H ₀ (m)	T(s)	L ₀ (m)	K _r	K _s	H ₀ '(m)	m(slope)	H ₀ ''/gT ²
20	0.68	3.75	21.98	1.00	1.00	0.68	0.001	0.005
18	0.68	3.75	21.95	1.00	1.00	0.68	0.093	0.005
16	0.68	3.75	21.91	1.00	1.00	0.68	0.074	0.005
14	0.67	3.75	21.86	1.00	0.99	0.68	0.056	0.005
12	0.67	3.75	21.78	1.00	0.98	0.68	0.186	0.005
10	0.65	3.75	21.61	1.00	0.96	0.68	0.048	0.005
8	0.63	3.75	21.24	0.99	0.93	0.67	0.053	0.005
6	0.58	3.75	20.39	0.98	0.88	0.66	0.085	0.005

The wave deformation analysis involved the shoaling, the refraction and the breaking waves. Then, the relationship among the deep-sea wave height (H_0), the incident angle of the wave (α), and the wave height in the deep sea ($H = 0.78d$) is used to determine the breaking wave.

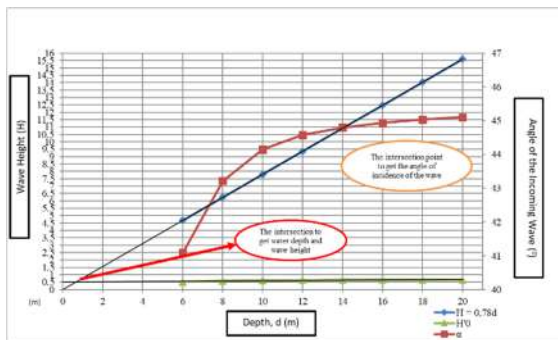
Figure 3: The relationship among the deep-sea wave height (H_0), the incident angle of the wave (α), and the wave height in the deep sea ($H = 0.78d$) for the 2 years of wave return period.

Figure 3 showed the results of the conditions in which waves propagate from the deep sea and break with wave height approaching the coastline, producing a value; sea depth (d) = 0,7 m, wave height (H_0) = 0,5 m and angle of the incident wave (α) = 42° for the 2 years of wave return period.

For the 5 years of wave return period prediction condition, waves propagate from the deep sea and break with wave height approaching the coastline within the water depth (d) = 0,2 m, wave height (H_0) = 0,2 m and angle of the incident wave (α) = 46° , while within the water depth (d) = 0,9 m, wave height (H_0) = 0,6 m and angle of the incident wave (α) = $42,5^\circ$, for the 5 years of wave return period prediction

condition as given in Figure 4 and Figure 5, respectively.

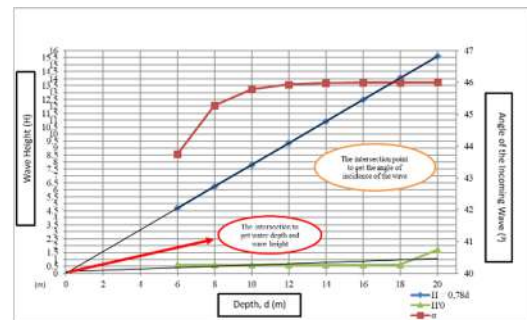
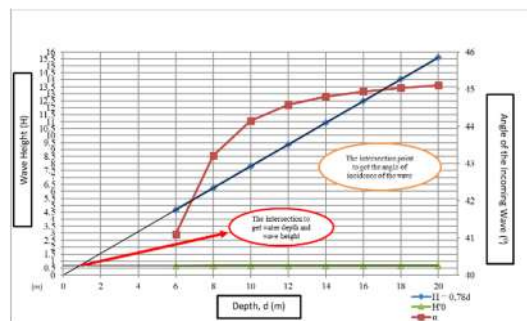
Figure 4: The relationship among the deep-sea wave height (H_0), the incident angle of the wave (α), and the wave height in the deep sea ($H = 0.78d$) for the 5 years of wave return period.Figure 5: The relationship among the deep-sea wave height (H_0), the incident angle of the wave (α), and the wave height in the deep sea ($H = 0.78d$) for the 10 years of wave return period.

Table 7: The calculation results of the structural load on the reclamation area.

1	Live Load of Container 6 @30,480 kg (uk.13,716 x 2,438 m ²)	=	6,58	kg/m ²	=	2,70	kg/m
	A	=	33,44	m ²			
	W	=	30,48	kg			
	Wide	=	2,44	m			
2	Live Load of Container Truck 45 (W=45000kg, A= 2,75 x 9,0 m ²)	=	1.818,2	kg/m ²	=	661,16	kg/m
	A	=	24,75	m ²			
	W	=	45.000	kg			
	Wide	=	2,75	m			
3	Live Load off Mobile Crane (W=79560 kg, A=5,79 x 5,79 m ²)	=	2.376	kg/m ²	=	410,35	kg/m
	A	=	33,52	m ²			
	W	=	79.650	kg			
	Wide	=	5,79	m			
4	Live Load of Building Structure	=	3.050	kg/m ²	=	3.050	kg/m
	Live Load of Office Floor (@3 tingkat)	=	250	kg/m ²	=	750	kg/m ²
	Live Load of Floor and Meeting Room	=	400	kg/m ²			
	Live Load of Ladder	=	300	kg/m ²			
	Live Load of Warehouse, Tools Room and Archive	=	400	kg/m ³			
	Live Load Parking Floor Building Lower	=	800	kg/m ²			
	Live Load Parking Floor Building Level Other	=	400	kg/m ²			
5	Dead Load of Paving Block (Rectangle 10x10,5x21, W=3,7kg)	=	167,80	kg/m ²	=	1.598,10	kg/m
	A	=	0,02	m ²			
	W	=	3,70	kg			
	Wide	=	0,11	m			
6	Dead Load for Heap Sand Paving Block	=	91,06	kg/m ²	=	0,09	kg/m
	ysand	=	1.517,72	kg/m ³			
	Thick of Heap	=	0,06	m			
	Wide of Land Heap	=	1.061,00	m			
	TOTAL	=	8.259,53	kg/m ²	=	5.722,39	kg/m

Source of Value : (ILRB, 1983)

4.3 Tidal

Based on the tidal observations data, it can be obtained the characteristics of water level in the form of a formzal value related to the highest tides and lowest tides that occur. The elevation of the original land in the form of a highway is used as a benchmark for an elevation of 0.00 meters. From the tide analysis, it is obtained that the highest water level is -1.00 m and the lowest water level is -3.20 m. So that the height of the reclamation pole can be planned to be parallel to or higher than the elevation of the highway because the highest tidal elevation is still 1 meter below the road elevation.

4.4 Load

This reclaimed land will be used as land for the construction of containers, offices, warehouses and passenger waiting rooms. In planning for the construction of the structure, a total load on the soil is required so that the structure does not collapse. The structural load planning to be built on the reclamation area is given in Table 7.

4.5 Geobag

The geobag material used is the German NAUE type Secutex® Soft Rock (NAUE, 1967). To determine the dimensions of the geobag, the type of soil used to fill into the geobag should be known in advance. In this paper, the type of soil is given in Table 8 obtained from the laboratory analysis of Soil mechanics laboratory belong to Engineering Faculty, Tanjungpura University. After that, the grain weight of the geobag is calculated using the stability of the protective stone.

Table 8: Type of soil.

Subsoil	Depth (m)	ϕ^0	γ_d (volkering)		γ_{sat}	
			(t/m^3)	(kg/m^3)	(t/m^3)	(kg/m^3)
3	3 - 3,5	11,31	1,23	1.116	1,673	1.517,72

A classic formula for the stability of rocks/stones under breaking waves at a sloping surface is given by the Hudson formula. The weight of geobag grain content calculation steps are given, as follows:

1. γ_r , obtained from the density of geobag fillers, namely the type of soil (sand) on the seabed. The data was obtained from the laboratory, which is $1,517.72 \text{ kg/m}^3$
2. γ_a , obtained from the specific gravity of seawater 1.025 kg/m^3
3. H , is the wave height of the plan

4. K_D , is the Stability Coefficient depending on the shape of the protective stone. Obtained from the table and used the number is 1.1

$$S_r = \frac{\gamma_r}{\gamma_a} = \frac{1.517,72}{1.025} = 1,48$$

6. θ , is the angle of slope of the retaining wall. Enter data according to the desired angle or with the "trial and error" technique

$$\cot \theta = \frac{1}{\tan \theta} = \frac{1}{\tan(38)} = 3,223$$

$$W = \frac{\gamma_r \cdot H^3}{K_D \cdot (S_r - 1)^3 \cot \theta} \quad (\text{Pratikto, et.a., 2014})$$

$$W = \frac{1.517,72 \times 0,6}{1,1 \cdot (1,48 - 1)^3 \cot(38)} = 2.674,5 \text{ kg}$$

According to the results of the data above, the weight of the contents of one geobag uses (W) = 2.674,5 kg as given in Table 9 and the geobag dimension size is shown in Figure 6.

Table 9: Weight of Geobag grain content.

W(kg)	γ_r (kg/m ³)	γ_a (kg/m ³)	H (m)
-2683,01	1.517,72	1.025	0,6
-2678,62	1.517,72	1.025	0,6
-415,50	1.517,72	1.025	0,6
-35,62	1.517,72	1.025	0,6
2605,36	1.517,72	1.025	0,6
2674,50	1.517,72	1.025	0,6
-1915,39	1.517,72	1.025	0,6
-1564,88	1.517,72	1.025	0,6
1473,76	1.517,72	1.025	0,6
K_D	S_r	θ	$\cot \theta$ (1/tan θ)
1,1	1,48	25	-7,489
1,1	1,48	28	-3,553
1,1	1,48	30	-0,156
1,1	1,48	33	-0,013
1,1	1,48	35	2,111
1,1	1,48	38	3,223
1,1	1,48	40	-0,895
1,1	1,48	43	-0,667
1,1	1,48	45	0,617

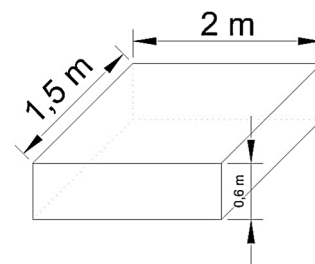


Figure 6: Geobag dimension size.

The height, width and length of the geobag size are determined from the height requirements of the retaining wall that are planned for the geobag

stacking or stacking place. Therefore, the size of this geobag is determined using the "Volume Weight Formula" with the "trial and error" technique until the volume weight above is in accordance with or slightly larger than the volume weight plan.

4.6 The Earth Pressure and Gravity Retaining Walls

To obtain structural resistance, the active earth (soil) pressure coefficient and the forces acting on each layer of geobag that will be stacked can be obtained so that the height and width of the structure are in accordance with the dimensions planned. The calculation results of the soil pressure and is given in Table 10, while the working force on the geobag layer is given in is given in Table 11. The diagram of soil pressure and the cross-section of retaining wall are given in Figure 7 and Figure 8, respectively. The top view of geobag formation used to construct the retaining wall is given in Figure 9.

Table 10: Soil pressure calculation results.

No	H (m)	Ka	Pressure σ (Kg/m ²)	Pa (Kg/m)	Point of Force (m)
1	6.3	0.67	3845.88	24229.07	3.15
2	6.3	0.67	4724.55	14882.33	2.10
3	5	0.67	5100.12	12750.31	1.67

Table 11: The working force on the geobag layer.

Geobag Layer			The working force of each geobag layer due to load is evenly distributed (kg/m)	The working force of each dry soil geobag layer (kg/m)	The working force of each wet soil geobag layer (kg/m)
Height (m)	Distance (m)	Total			
0.3	0.3	13	24229.07	15626.45	13564.16
0.9	0.6	12		17362.72	15549.16
1.5	0.6	11		19533.06	18214.73
2.1	0.6	10		22323.50	4207.60
2.7	0.6	9		10630.24	6502.66
3.3	0.6	8		12756.28	8797.71
3.9	0.6	7		14882.33	11092.77
4.5	0.6	6		17008.38	13387.83
5.1	0.6	5		19134.43	15682.88
5.7	0.6	4		21260.47	17977.94
6.3	0.3	3			
Total			24229.07	170517.86	124977.43

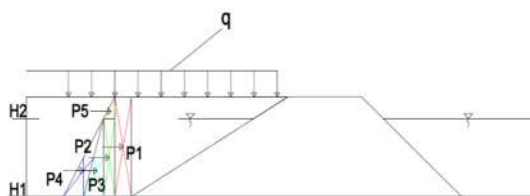


Figure 7: Diagram of Soil Pressure.

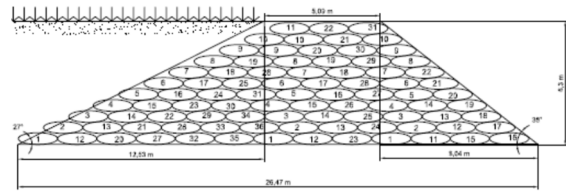


Figure 8: Retaining Walls.

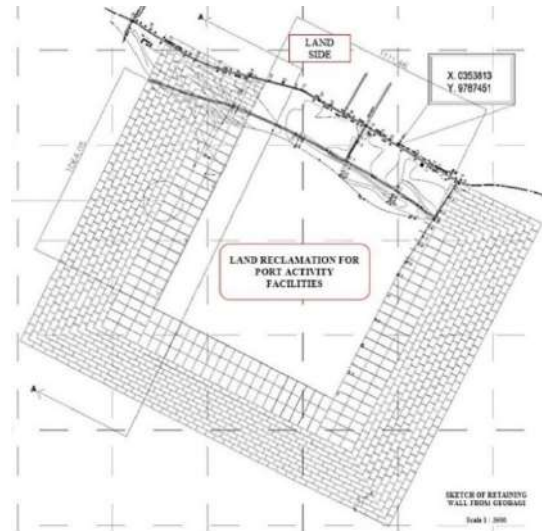


Figure 9: Top View of Geobag Formation.

The height of the retaining wall is determined by the deepest point from the coastline to the length of the reclaimed land and the width of the retaining wall depending on the geobag stacking pattern adjusted to the ground level.

4.7 Stability of External Forces

a. Overturning

Moment of prisoner due to own heavy construction (ΣM_R) = 2.423.370 kg.m

Overturning moment due to active force (ΣM_0) = 589.210 kg.m.

$$FS_{\text{overturning}} = 4,11 \geq 2 \quad \text{OK!}$$

Retaining Wall Design is safe against collapse due to overturning.

b. Sliding Failure

where :

$$\Sigma F_R = 240,33$$

$$\Sigma F_D = 128,825$$

$$FS_{\text{sliding}} = 1,87 > 1,5 \quad \text{OK!}$$

Retaining Wall Design is safe against collapse due to sliding.

c. Bearing Capacity Failure

Where :

$$q_{ult} = 34.774 \text{ kg}$$

$$e = 5,61 < 4,4$$

$$q_{maks} = 9.084 \text{ kg}$$

$$FS_{\text{bearing capacity}} = 4,41 > 3 \quad \text{OK!}$$

Retaining Wall Design is safe against collapse due to bearing capacity failure.

5 CONCLUSION

The dimensions of the retaining wall are determined based on the water depth from the results of the bathymetry and tide surveys. Meanwhile, the material of filled geobag can be obtained from the results of the type of soil that will be filled as geobag filling material dan dimension use trial and error technique until the volume weight above corresponds to or is slightly greater than the volume weight plan. From the results of the calculation of the strength of the geobag retaining wall, the following points can be concluded:

1. The height of the retaining wall is taken from the deepest point of bathymetry, which is 6.3 m and the break wave height per 10 year return period is 0.6 m. It's width is adjusted by the geobag stacking to the height of the retaining wall until it forms a trapezoidal shape of 26.47 m. This retaining wall has 11 geobag layers with the most bottom layer are 13 geobags and the top layer are 3 geobags. The slope of the angles on the left and right sides are different, on the left side which attaches to the land which is 27° and the right side on the seafront is 38° .
2. Geobag dimensions are 2 x 1.5 x 0.6 m using trial and error until the weight is in accordance with the grain weight of the protective stone.
3. Geobag retaining walls are strong against external force stability due to overturning, sliding, and bearing capacity. The result exceeds the standard threshold value.

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Tidal Analysis for Planning the Tidal Flood Management and the Moveable Weir, Case Study in Parit River, Kawunganten Cilacap

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Keywords: Tidal Flood, Water Sea Level, Observation, Analysis.

Abstract: The seawater level always rises globally. Sea level rise has an impact on human life. One of the effects of rising sea levels is the tidal flood which is one of the adverse natural phenomena. Kawunganten District, Cilacap Regency is one of the areas affected by the tidal flood. The construction of the moveable dam is a solution to prevent damage to the region due to tidal flooding. In carrying out dam design planning required accurate tidal data. This study aims to obtain data and analysis of tides along the river to the river mouth. In determining tidal data, observations are made using an automatic survey tool called DIVER. The tidal analysis is done by applying the least square method to obtain the tidal harmonic constant at the study site. Base on observation data for 75 days, the results of tidal data analysis as a reference for planning flood management and moveable. The analysis gives the value of MSL (Mean Sea Level) of 0,996 m, MHWS (Mean High Water Spring) of 1,7 m, MLWS (Mean Low Water Spring) of 0,292 m, HHWS (Highest High Water Spring) of 1,992 m and LLWS (Lowest Low Water Spring) of 0,00 m as elevation reference for the infrastructure development.

1 INTRODUCTION

Indonesia has diverse water characteristics. The diversity of waters in Indonesia is due to Indonesia's geographical condition which consists of various kinds of islands. Flood disaster is one of the disasters that often hit the Indonesian archipelago which has low land elevation. One type of flood that can provide many losses in various aspects is tidal flooding. The parameter that is very influential on the occurrence of seawater is sea level rise (Yudi R. K. et al., 2017).

Cilacap is one of the districts in Indonesia that is vulnerable to the occurrence of tidal flood. This is because the Cilacap area is a lowland area with very slow soil infiltration capability (Yani and Sumunar, 2019). Cilacap Regency is located in Central Java Province. Geographically, Cilacap Regency is on the southern side of Java, which faces the continuation of the Indian Ocean. The Cilacap district area that was harmed due to the tidal flood was Ujungmanik Village, Kawunganten District. This area has a distance of 16,662 m from the mouth of the Cilacap river. The tidal flood that occurred in this region came from the overflow of the Parit river along the Kawunganten district. Tidal floods reduce social and

economic activities of the people in Kawunganten sub-district. One of the disturbed community activities is farming due to tidal floods that have risen to the ground causing sea water to damage crops in agricultural and plantation areas. In addition, the access road used by the community to carry out their activities was also affected due to sea level rise reaching the road surface. Housing, places of worship and schools were also affected by tidal flood that came from the overflowing moat river. Flood disasters always have widespread impacts on the destruction of infrastructure, the environment, ecosystems and human life (Salami et al., 2017). High levels of community resilience to flood disasters are represented at low levels of flood risk (Isa et al., 2018).

Floods have given various kinds of adverse effects to the community and therefore preventive measures are needed to reduce these impacts (Subiyakto et al., 2019). To overcome the problem of tidal floods, there are several solutions implemented. The application is done by structural and non-structural methods (Wahyudi N. R. et al., 2019). In solving the problem of tidal flooding that occurred in Kawunganten Village, it is necessary to apply the structural method.

The structure needed is a polder system and a door that will be a solution to solving this problem. The polder system is effective in reducing the rise in water level due to tidal flooding and can reduce the risk caused by tides. Polder system is a technology in water control that is often applied in various cities with tidal flood problems. To optimize the performance of a polder system, it is necessary to have a good channel for the flow of water (Wahyudi et al., 2019). In the construction of polder systems, several infrastructures are needed which include retention ponds, drainage systems, water pumps, and dams/floodgates (Nugroho et al., 2016). To develop infrastructure to handle tidal floods it needs to be done in cooperation with the management and mitigation of tidal floods (Adi and Wahyudi, 2018).

In addition to building various types of infrastructure, parameters are also needed in planning infrastructure development, so that it can produce optimal and efficient results (Hunter J. R. et al., 2017). By considering tidal parameters, it can determine the number of costs incurred for the construction of the weir and as a reference for the amount of weir needed to overcome tidal floods (Hall et al., 2019). A very important parameter in polder system planning is tidal. Estimated water levels within the tidal range are a suitable tool for environmental management and risk management for tidal flood disasters (Sadeghian et al., 2016). Tides have the most dominant influence on the occurrence of tidal floods in Cilacap. A tide that comes from downstream across the river to the upstream direction. The increase in high seawater is increasing over time due to the influence of global warming that occurs.

This study aims to obtain primary data in the form of tidal conditions along the Parit river and Cilacap waters. The data that has been obtained is analysed to determine the type of tides. Tidal data is calculated and analysed to find important heights needed. Important elevation obtained is adjusted to the weir planning for handling tidal flooding and as a consideration in determining the structure of the weir.

2 METHOD

2.1 Water Level Measurement

Data is collected by using the DIVER survey tool. This tool is equipped with a pressure sensor, so the measurement of water elevation also takes into account air pressure. DIVER is a survey tool applying the principle of measuring water pressure at the

location under study (Eijkelkamp, 2016). To calculate the water level, the pressure data are calculated using the following formula:

$$WC = 9806.65 \frac{P_{Diver} + P_{Baro}}{\rho \cdot g} \quad (1)$$

Where WC = Water Column, P = pressure in cmH₂O, g = the acceleration due to gravity (9,81 m/s²). The water level can be calculated by the following equation:

$$WL = TOC - CL + WC \quad (2)$$

Where WL = water level, TOC = pipe depth, CL = diver depth

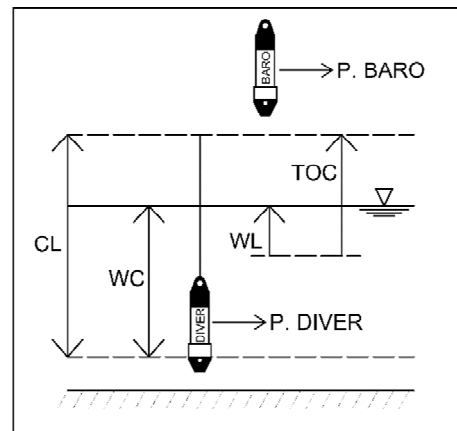


Figure 1: DIVER installation scheme.

2.2 Calculation of Tidal Harmonic Constants

Tidal data is processed using worldwide with the MATLAB programming language (Boon, 2007). In the tidal analysis, the least square method is used. The least-square method is a method for tidal calculations with the principle of minimizing the tidal elevation equation so that a simultaneous equation is obtained. The simultaneous equation is solved using a numerical method to produce harmonic constants (Dalpan and Pratomo, 2015). The least square method is one of the most widely used approaches and continues to be used in the tidal analysis (Foreman et al., 2009). The equation used in the least square method is:

$$h(t) = h_o + \sum_{j=1}^m f_i H_j \cos(\omega_j t + u_j - k_j) \quad (3)$$

Where t = time (in hours), $h(t)$ = predicted water level, h_o = average water level, f_i = nodal correction

factor for amplitude, H_j = average amplitude in the nodal cycle (18,6 years), ω_j = angular velocity of the tidal component to j, u_j = odal correction factor for the phase, k_j = phase lag between the equilibrium tide at the observation site and the equilibrium tide at Greenwich at 00.00, m = the number of tidal harmonics to be analysed (Williams, 2013).

Harmonic components that have been calculated are used as parameters in the calculation of important tidal elevations. The elevation is useful in analysing dam planning for the location under study because the water level is always changing at any time (Joesidawati et al., 2017). The calculated elevations are MSL (Mean Sea Level), MHWS (Mean High Water Spring), MLWS (Mean Low Water Spring), HHWS (Highest High Water Spring), and LLWS (Lowest Low Water Spring).

2.3 Tidal Types Calculation

The harmonic component is also used as a tidal type analysis at this research location. In determining the type of tides, it is necessary to calculate the value of formzahl numbers with the following equation:

$$F = \frac{A_{K1} + A_{O1}}{A_{M2} + A_{S2}} \quad (4)$$

Where F = formzahl numbers, A_{K1} and A_{O1} = the amplitude of a single daily tidal component, A_{M2} and A_{S2} = the amplitude of a double daily tidal component. classification of tidal types based on the value of formzahl numbers is divided into 4 types. Where the classification is a semidiurnal tides ($0 < F < 0,25$), mixed mainly semidiurnal tides ($0,25 < F < 1,5$), mixed mainly diurnal tides ($1,5 < F < 3,0$) and diurnal tides ($F > 3,0$).

3 RESULTS AND DISCUSSION

3.1 Comparison of Water Level in the River and the Sea

Based on observations that have been made, it is known that the water elevation value at the study site is shown in figure 2 for 75 days from January-April 2020. The river observation locations are selected based on the area planned as tidal flood protection in Ujungmanik Village. Whereas the observation location at sea is based on the connectivity of the flow with the Parit river. The results of river and sea elevation are compared in graphical form by

equalizing the MSL value in each data. These results are shown in Figure 3.



Figure 2: Tidal elevation monitoring locations.

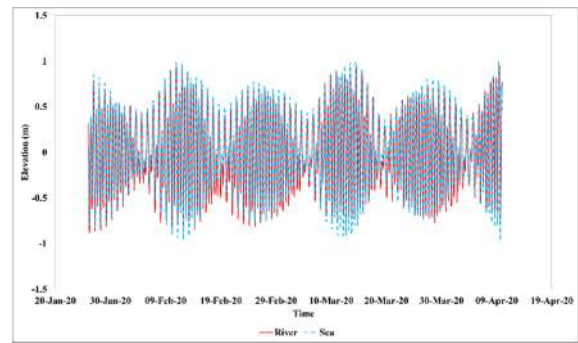


Figure 3: Water levels in the Parit River and Teluk Penyuh Beach in Cilacap.

The results in Figure 3 show the different phases in the water elevation in the Parit river and Cilacap Bay Coastal Waters. The pattern of movement of tidal in the two study sites has the same tendency. The sea level is higher than the river water level. At ebb condition, the sea level is also dominantly lower than the river elevation value. This proves that the tidal conditions at sea affect the elevation of water in the river. The graph shows the time of the rise and fall of the water level in the sea is always faster than the water level in the river. This proves that there are differences in the tide and ebb levels in water elevations in rivers and sea.

3.2 Analysis of Least Square Tidal Results

Data of water elevation in the Parit River for 75 days previously obtained shown in Figure 4 was calculated using the least square method. With the least square method, the results of 9 tidal harmonic constants are shown in Table 1.

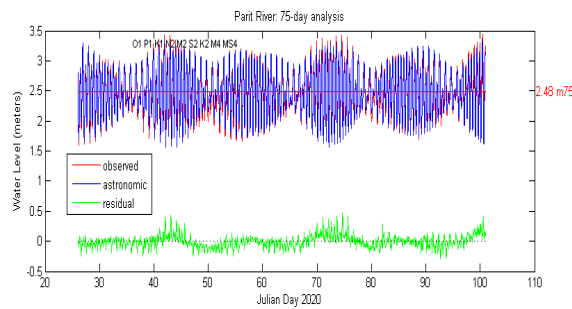


Figure 4: Comparison between observation data and tidal forecasting results.

Based on the comparison, the graph mentions of tidal forecasts with primary data obtained, and an error value of 0,101%. The error value obtained comes from the least square calculation which ignores the meteorological influence, so that the astronomical effect also does not participate in the effect of tidal generation on the model.

Table 1: Harmonic constants.

Constanta	A(m)	g°
M ₂	0,457	201,470
S ₂	0,247	331,730
N ₂	0,088	94,540
K ₁	0,179	288,920
O ₁	0,113	230,300
M ₄	0,037	176,130
MS ₄	0,040	309,880
K ₂	0,042	134,240
P ₁	0,036	306,740

Where A is amplitude and g° is a phase. From the 9 harmonic constants that have been obtained in table 1, we get the value of the formzahl number is 0,4148. With this value, the tidal type at the study site is classified as a mixed mainly semidiurnal tides because the value of formzahl numbers ranges from $0,25 < F < 1,5$. The characteristics of tidal types at this location occur two times the tide and two times the tide in a period of 1 day (24 hours). However, the water level elevation value that occurs is irregular and has a difference in elevation between two tides and two ebbs in one day.

From the calculated harmonic constants, we can get important tidal elevation results at the study site. MSL (Mean Sea Level) value of 2,48 m, MHWS (Mean High Water Spring) of 3,184 m, MLWS (Mean Low Water Spring) of 1,776 m, HHWS (Highest High Water Spring) of 3,476 m and LLWS (Lowest Low Water Spring) of 1,484 m.

In planning LLWS dams it is used as a reference. So the LLWS value becomes 0. Then the results of other elevations become MSL (Mean Sea Level) value of 0.996 m, MHWS (Mean High Water Spring) of 1.7 m, MLWS (Mean Low Water Spring) of 0.292 m, HHWS (Highest High Water Spring) of 1,992 m and LLWS (Lowest Low Water Spring) of 0 m. The tidal graph with reference to +0.00 is LLWS can be seen in Figure 5.

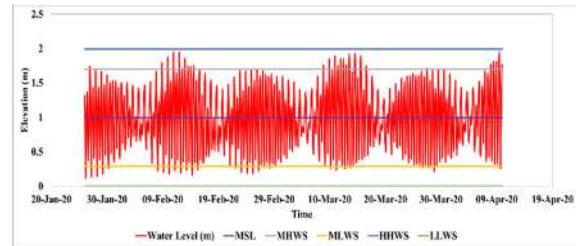


Figure 5: The water level in the parit river → LLWS = 0.

3.3 Infrastructure Analysis with Tidal Elevations

Peil Scale as a tidal reference is placed under the bridge. The bridge's elevation based on Peil Scale is +4,20 m. Based on the receding elevation, the bridge floor elevation is around +2.72 or around 70 cm from HHWS. One function of tidal observation and analysis is to determine the +0.00 LLWS elevation used as a reference for bathymetry measurements. The results of bathymetry measurements can be seen in Figure 6.





Figure 7: Tidal flood affected areas.

Based on the results of tidal measurements and analysis, the elevation of water structures including weirs, embankment, and floodgates can be determined. Assuming a maximum tidal height of 1,99 m, the height of the water structure plus a freeboard (guard height), for example for embankment and weir is added 1,0 m or about = + 3,0 m.

4 CONCLUSIONS

1. Tides and ebb elevations of water at sea are always faster than elevations of water in rivers. This proves that there are differences in the tide and ebb levels in water elevations in rivers and seas. The sea level is dominantly higher than in rivers. Therefore tidal conditions in the sea affect the occurrence of tidal floods that occur to the inhabitants along the river area.
2. Formzahl numbers obtained from tidal analysis in the Parit River amounted to 0,4148, from this value known tidal types at the study site are classified as mixed mainly semidiurnal tides because the value of formzahl numbers ranges from $0,25 < F < 1,5$.
3. Tidal elevation obtained from the calculation of harmonic constants is the MSL (Mean Sea Level) Value of 0,996 m, MHWS (Mean High Water Spring) of 1,7 m, MLWS (Mean Low Water Spring) of 0,292 m, HHWS (Highest High Water Spring) of 1,992 m and LLWS (Lowest Low Water Spring) of 0 m.
4. The bridge's elevation based on Peil Scale is +4,20 m. Based on the receding elevation, the bridge floor elevation is around +2.72 or around 70 cm from HHWS. So in determining the height of the weir can be used +3.0 m elevation to prevent high tides flowing through the Parit River.

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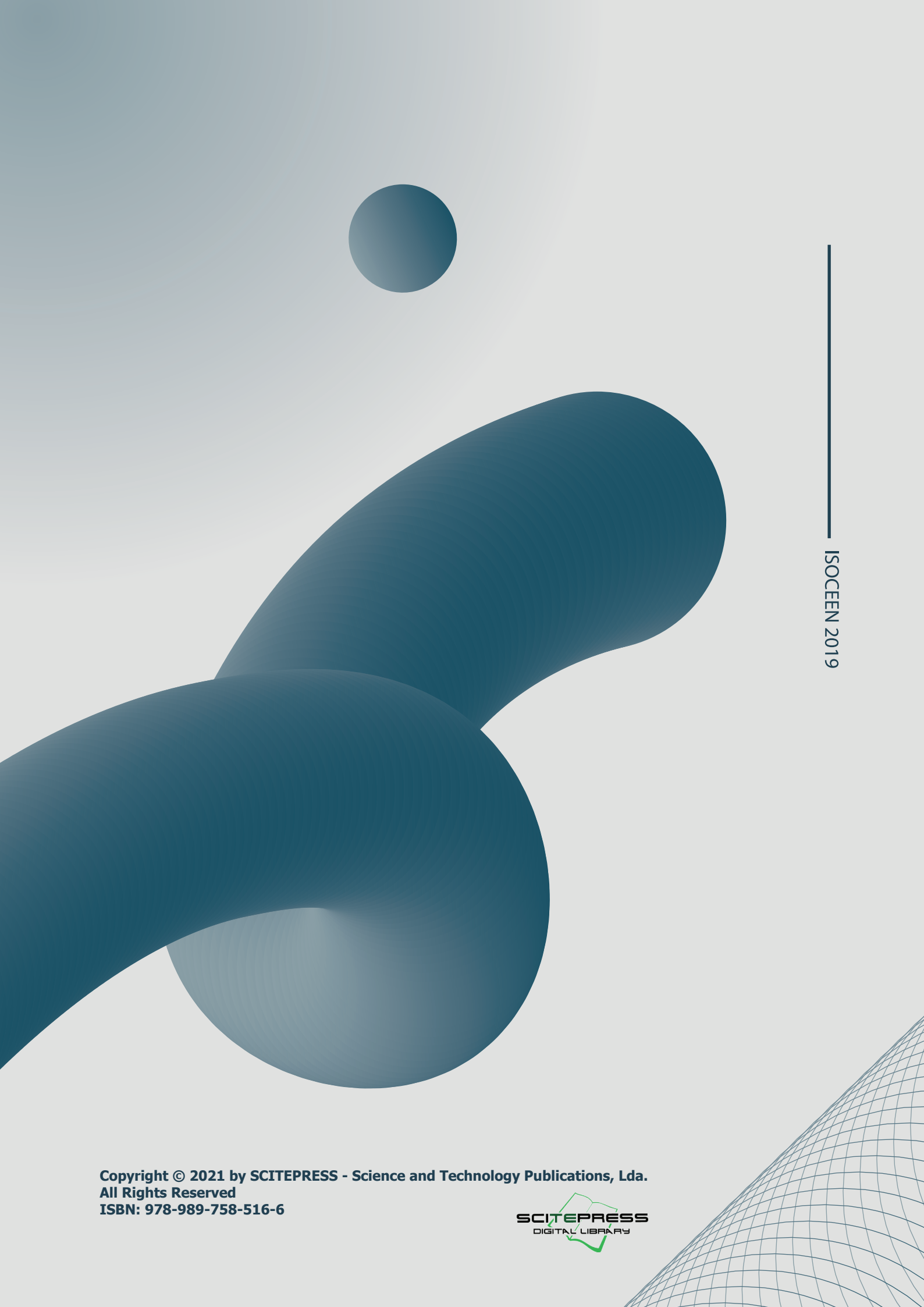
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The 5th International Seminar on Ocean and Coastal Engineering, Environmental and Natural Disaster Management

LETTER OF ACCEPTANCE

27th October 2017

Primary Author : Muhammad Zubair Muis Alie

**Paper Title: Comparative Study of Structural Geometric to the Ultimate Strength on
Fixed Offshore Platform**

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Abstract **Proceeding of** **The 7th International Seminar** **on Ocean and Coastal Engineering,** **Environmental and Natural** **Disaster Management**

PREFACE

The 7th International Seminar on Ocean and Coastal Engineering, Environmental and Natural Disaster Management (ISOCEEN) 2019 was held in Surabaya, Indonesia, on 6 – 7th November 2019. The Organizing Committee acknowledged for their dedicated preparation over many years that led to successful seminar with broad participant. This event is held by cooperation among Institut Teknologi Sepuluh Nopember (ITS) especially Department of Ocean Engineering, Tohoku University, Japan, TU Delft, Netherlands, UNESCO-IHE Delft, Netherlands and HZ University of Applied Sciences, Netherlands. The papers contained in This Proceedings cover the topics including the field of Ocean, Offshore, Coastal engineering, Environmental and Disaster Management. This forum is more focused on creating and expanding professional network to foster the relationship between the University, industry, business and communities across the country.

Finally, on behalf of the organizing committee of the seminar, sincere appreciation is expressed to all authors contributing to our seminar. Special thanks are also due to all keynote speakers, invited talks and chairpersons for the efforts in preparing the manuscripts and managing the sessions, respectively.

Sincerely yours

Chairman of the Organizing Committee ISOCEEN 2019
Department of Ocean Engineering, Faculty of Marine Technology
Institut Teknologi Sepuluh Nopember

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Tsunami Hazard Assessment in Indonesia Case Study: The 2018 Palu Earthquake and Landslide Tsunami and 2018 Krakatau Volcanic Tsunami

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Abstract. Indonesia has a very complicated tectonic setting, produced a very high seismic activities or frequent large earthquakes, some of them can generate tsunamis especially by large shallow earthquakes along subduction zones. In Indonesia, at least 118 tsunami events that have been occurred in the period of 1600-2018, around 90% of them are generated by earthquake, while 9% was cause by volcanic eruption, and 1% that generated by landslide. Although landslide tsunamigenic and volcanic tsunamigenic are rare, however in 2018 there were landslide tsunami triggered by earthquake occurred in Palu at 28 September 2018, and flank collapse volcanic tsunami at 22 December 2018. These two tsunamis events are unique and interesting things to study.

In this paper, the tsunami hazard along the coastline in Indonesia are conducted by PTHA and also the 2018 Palu Tsunami and the 2018 Krakatau volcanic tsunami are studied by numerical model as case studies.

Keywords: *PTHA, landslide Tsunami, volcanic tsunami, Palu, Krakatau.*

Turbulent Boundary Layer Modelling to Develop the Marine Pipeline Scouring Model and Its Countermeasures

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Abstract. Research related to turbulent boundary layer modeling for the application of bedload sediment transport due to non-linear waves has been carried out including Suntoyo et al. (2008), Suntoyo and Tanaka (2009a) and Suntoyo and Tanaka (2009b), while Suntoyo et. al. (2015) have applied the bottom shear stress calculation method and the bedload sediment transport rate due to non-linear wave motion (Suntoyo et al 2008, Suntoyo and Tanaka 2009a) to model suspended sediment concentrations and sediments. While the development of the bottom shear stress model, the bedload sediment transport due to irregular wave movement and its application to changes in coastal morphology have been carried out, among others, by Suntoyo et al (2016) and Wijaya et al. (2016) which showed that the proposed shear stress formula and bedload sediment transport provide sufficient accurate results compared to the measurement data.

Therefore in this study turbulent boundary layer models and sediment transport rates that have been developed previously for non-linear and irregular waves will be developed to model the propagation rate and scouring depth in the pipeline, new methods for propagation, calculation of width and depth of the proposed scouring are validated with measurement data in the Flume Tank and then It is applied to the problem of scouring on marine pipeline in Indonesian waters. It is hoped that the developed model can improve the accuracy and reliability of the pipeline scouring model due to non-linear and irregular waves that occur in many coastal and marine environments in Indonesian waters and can appropriately use the techniques and its countermeasures.

Using Citizen Science in Monitoring Plastic Litter in The Marine Environment

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Abstract. Globally, rivers and oceans have to deal with a growing amount of plastic waste, which is nonbiodegradable, causing a lot of negative effects on flora and fauna and eventually human wellbeing. Lebreton et al. (2017) estimated that between 1.15 and 2.41 million tonnes of plastic waste enters the oceans every year from rivers. A better understanding of the origin and pathways of plastic waste is a prerequisite for a sound plastic waste management system, but gathering sufficient data on the amount of litter being transported through rivers to the sea, as well as in the type and origin of plastic waste is a challenging task, due to the time and money needed to research the riverine and coastal areas within any country. However, there is a growing public interest in the impact of plastic on the riverine and marine environment and beach clean-ups and river shoreline clean-ups by volunteers take place more often. This provides the opportunity to ask volunteers to join in participatory monitoring or citizen science programmes. In this way volunteers can provide repeated sampling for different periods of time (summer-spring-autumn-winter; high tide-low tide) as well as over more extensive geographic areas. A key challenge with any citizen science programme is to ensure standardized methods and quality control so that the samples and data can legitimately be compared and used in peer-reviewed research. Nevertheless, a study by Van de Velde et al. (2016) showed that a citizen science programme can broaden the coverage and increase the sampling power of coastal litter without compromising the data.

In the South-Western delta of the Netherlands an elaborate and growing network of volunteers participate in beach clean-ups. Some started to use digital apps to monitor the amount and type of plastic waste found on the beaches. These groups are being used to test several methods for monitoring of plastic waste (OSPAR method, Literati app, Crowdwater app, etc.) and based on their findings apps and monitoring methods will be adapted and an advice for standardization will be developed.

The most important findings from the plastic waste monitoring on the beaches along the Western Scheldt until current date are:

1. Per stretch of 100 meter of beach an average of 380 pieces of waste is found;
2. 90% of the amount of waste on the beaches is plastic waste;
3. More than half of the plastic waste originates from fisheries and shipping.

The top 5 of monitored plastic waste objects are (macro and meso plastics) given in the following table:

	Plastic waste item	Average/100 m
1	Fishing nets and ropes	147
2	Little pieces of polystyren <50 cm	64
3	Caps	19
4	Food packaging for candy, crisps, snacks	13
5	Balloons and ribbons	12

Effect of Cigarette Butts Extraction on The Corrosion Rate of ASTM A36 for Jacket Platform

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Abstract. Jacket platform is one type of offshore building used for the offshore oil exploitation process. In general, the Jacket is designed with an operating life of 20 to 25 years and in operation must be guaranteed the safety and strength of the Jacket structure. One of the safety factors that must be considered is corrosion growth in all parts of the Jacket structure. To overcome corrosion problems, there is a need for alternative corrosion improvements, namely the prevention of corrosion by using corrosion inhibitors. The organic inhibitor that can be used as corrosion inhibitors, one of which is cigarette butts, because it contains a lot of nicotine. Nicotine can be used as an inhibitor by donating nitrogen atoms to nicotine to Fe^{2+} atoms so that complex compounds $[\text{Fe}(\text{NH}_3)_6]^{2+}$ are formed. This compound has a higher stability than Fe so that it can be used as protection in corrosion. The purpose of this research is to solve the cigarette butts problem by creating it a corrosion inhibitor to minimize the cost of material replacement, maintenance costs, and over-design. This study is an experiment by using ASTM 36 steel, material will be blast cleaned in advance with abrasive material Steel Grid. Continued with the coating process by mixing epoxy paint and extraction of cigarette butts, then immersed in corrosive medium namely sea water of Kenjeran. Corrosion rate testing is carried out by using the weight loss method. The highest corrosion rate occurred at 1% mixing with a corrosion rate of 24.61 mpy and the lowest corrosion rate at 2% mixing with a corrosion rate of 13.78 mpy Based on this research it can be concluded that the nicotine that can be used as cathodic protection on coatings.

Spatial Distribution of Pollutant in Jakarta Bay from Hydrodynamic-2D Transport Coupled Model

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Abstract. Being a center of marine activities such as shipping and fishing which are accompanied by massive development and industrial, Jakarta Bay is susceptible to the environmental degradation from the pollutants. Observation was conducted in Jakarta Bay during the southeast monsoon (Juli-Agustus) 2015 to collect physical properties data such as sea surface temperature and heavy metal component. Pollutants distribution such as Pb, Ni, Zn, Polychlorinated biphenyl (PCB), Polycyclic aromatic hydrocarbons (PAH) and Dichloro-diphenyl-trochloroethane (DDT) are simulated using Horizontal 2D hydrodynamic model from Princeton Ocean Model (POM) with the Arakawa-C grid system. The model domain consists of 400 x 260 grid and 110x110m grid resolution. With the high accuracy rate (92%) for the tidal validation and mean sea level residual 0.074 m, the simulation shows that pollutant distribution is mainly influenced by ocean current condition. Observed eddies in the simulation, which are triggered by significant difference of bathymetry and shear current velocity, indicate that the pollutants from Jakarta Bay are not easily transported by tidal current into the Java Sea.

The Influence of Additional Equipment to the Ultimate Strength of FPSO

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Abstract. Nowadays, there have many ship conversions, for example, from double hull tanker to Floating Production Storage Offloading (FPSO). The conversion is conducted due to the change function of the ship. On the other hand, due to the change function of the ship, some items of the ship such as equipment need to be evaluated because of those related to the ultimate strength of the ship. The objective of the present study is to analyze the influence of additional equipment to the ultimate strength of FPSO under vertical bending moment for hogging condition. The cross-section of FPSO is taken by considering one-frame space. The numerical method is used and the Multi-Point Constraint (MPC) is applied to both sides of the cross-section and it is assumed to have remained plane. The shell element type is used for modeling the cross-section. The material properties are set to be constant. As a simple calculation, the ship's cross-section is analyzed in intact. The additional equipment is calculated and included in the analysis to know the influence of it. The analysis of the ultimate strength, including their influence of additional equipment, is conducted under longitudinal bending. To compare the ultimate strength obtained by the numerical method, the analytical method is adopted. It is found that the ultimate strength gained by the numerical method is almost identical with the analytical method. The behavior of the ship's cross-section is also presented in term of stress distribution and deformation.

Size Optimization of Foil-Shaped Center Bulb on Catamaran Hull Form to Reduce Resistance

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Abstract. The interference of resistance between the two catamaran hulls is a very popular topic to discuss. Errors in designing hull form and determining the distance between the hull will result in a large amount of interference that causes the resistance of the catamaran increased. Some researchers try to reduce the resistance of catamaran due to interference such as changing the shape of the demihull, giving a bulbous bow, and adding a center bulb. The idea to use center bulb on catamarans is still relatively new and still requires a lot of studies. Therefore, in this study the elliptical center bulb will be replaced with a foil-shaped center bulb. The purpose of this study was to obtain the optimal length (X_1) and diameter (X_2) foil-shaped centrebulb on the NPL hull catamaran scale model with Fr 0.7 using Response Surface Methods (RSM). The optimal size of foil-shaped center bulb is determined by the size that the most makes of ship wave resistance (R_w) to a minimum. The results of the regression equation in Order 1 are $R_w = -0.168X_1 + 0.378X_2 + 14.862$ and the results of the regression equation in Order 2 are $R_w = -0.020239X_1 + 0.061318X_2 + 0.201557(X_1)^2 + 0.261325(X_2)^2 + 0.015(X_1X_2) + 13.34806$. Optimal center bulb size is obtained at a length of 103.78 mm and a diameter of 26.30 mm with a wave resistance value of 13.23 N. The center bulb size can reduce wave resistance by 11.74% from the initial center bulb model and reduce 4.72% compared to not using a center bulb.

Optimizing the Vertical Bow Motion of Ship Hull Forms Using Response Surface Methodology

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Abstract. Many aspects to analyzing seakeeping performance, one of which is the vertical bow motion of the ship. As is well known that vertical motion, along with its derivatives namely vertical velocity and vertical acceleration, will be related to other aspects of seakeeping performance such as slamming, deck wetness and Motion Sickness Incident (MSI). This study discusses the shape of the hull that has small vertical bow motion using the Response Surface Methodology optimization method. The purpose of this research is to minimize the vertical motion of the ship, so that the ship's performance will be better. In addition, to apply this RSM optimization method in the naval architecture field. The shape of the hull used in this study is the standard hull, namely Wigley Hull, S60 Hull and NPL Hull with the same ship length. The variable used for the optimization process is the ship's L/B and B/T ratio by changing it $\pm 10\%$ with a fixed displacement. Seakeeping analysis is carried out at Fr 0.25, at the heading angle of 180 degrees and a significant wave height of 1.25 meters.

Identification of Coastal Problem along the East Coast of Lampung Indonesia

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Abstract. The process of sedimentation and erosion that occurs along the coast is generally a dynamic natural process. To identify the problems of erosion and sedimentation along the coastal, it so related to wave and sediment characteristic data. The wave parameters for instance significant wave height, wave period and wave direction are forecast from daily data of wind speed from European Centre for Medium-Range Weather Forecasts (ECMWF). The daily wind data is converted to significant wave height and significant wave period using Shore Protection Manual (SPM) method. The grain sediment, that parameters performed such as diameter and settlement velocity of sediment grain. To predict the erosion and sedimentation, the Kraus's Equation is used. Sediment grain size at the East Coast of Lampung Province, which is along the coast of Ketapang District, South Lampung Regency ranged between 0.0258 – 0.033 mm. The scale of sediment size is classification in to Mud type. The significant wave height (Hs) and significant wave period (Ts) was calculated using Shore Protection Manual (SPM) method and forecasting based on wind data from ECMWF relatively is around 2.57 m/s with a wave period of 8.015 s. So that the grain size is relatively will sedimentation.

Characteristics of Wind and Wave along the Kenjeran Beach Tourism Areas in Surabaya – Indonesia

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Abstract. Waves are movements of ups and downs of sea water that carry energy. This wave energy can erode the beach shore. Many and a few areas of eroded coast will depend on the magnitude of the energy of the waves. This study aims to analyse the characters of wind and waves in the Kenjeran Beach Tourism Area Surabaya. This research was carried out for 8 months, from December 2018 to July 2019 at Kenjeran Beach. The data used is wind data for 10 years (2009-2018), The method used in this study is the method of field surveys, analysis of winds, calculation and analysis of waves in the beach. This research shows that the highest sea waves caused by wind in the Kenjeran tourism area is as high as 0.8 m. The benefits of this research are as a basis in the management of Pamurbaya tourism area, both in maintaining the safety of tour operators / visitors and in the development (management) of tourist area locations.

Thermal Front in the North of Java Sea, Indonesia

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Abstract. Indian Ocean Dipole (IOD) is a phenomenon in the Indian Ocean caused by differences in the anomaly of Sea Surface Temperature (SST) between the West Coast of Sumatra and the East Coast of Africa. The phase difference of the formation of positive IOD (+) and negative IOD (-). The IOD phase can be known by using the Dipole Mode Index (DMI). This research aims to identify the effect of IOD phase on SST and distribution thermal front. Identification of distribution using the Pearson method, and analysis of the distribution thermal front using the Single Image Detection (SIED) method. Area research in North Java Seas during wet season 2013-2015. IOD phase with normal condition more often occur than IOD phase with weakly condition. Strong correlation occurred with negative direction, and moderate correlation occurred with positive direction. Meanwhile, correlation IOD with rainfall not significant or uncorrelation. During five years the heat temperature was concentrated in the coastal area. IOD with normal condition has a larger thermal front area 1976 km², while IOD with weakly condition has a 624 km² thermal front area. Thermal front distribution more often occur on north area of Central Java with a longwise and widening distribution.

Characteristics of West Season Wind and Wave for Ferry Cruise Security Prediction in The Kalianget-Kangean Cruise Route, Madura, Indonesia

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Abstract. In the Indonesian archipelago the West Season is a wet period when bad weather usually takes place. Bad weather at sea is often marked by strong winds and high waves making it dangerous for inter-island cruise. One of the busy cruise routes is the Kalianget-Kangean cruise route which is part of the Java Sea where the musonal process, especially the West Season, is ideal. This study aims to analyze the characteristics of wind speed and wave height for 10 years (2008-2017) and weekly patterns for three months (December 2017-February 2018) representing the West Season in the waters of the Kalianget-Kangean ferry cruise route, as well as predicting security status the voyage. The data used was obtained from Surabaya Maritime BMKG which was then processed to obtain descriptive statistics and visualization of rose charts and maps of wind and wave patterns at the study site. The results show that the wind and wave characteristics of 2008-2017 formed two peaks that marked the West and East monsoons, as well as two valleys during the transition period between them. Wind speed and wave height of ten years are generally below the danger threshold, ie <10 knots and <2 m, respectively. Exceptions occur in the West Season, especially in January which is the peak of the West Season. When the wind and waves have strengthened with a steady blowing direction. The maximum wind speed reaches 29 knots and the maximum wave height reaches 6.7 m. Weekly wind and wave conditions from December 2017 to February 2018 are relatively safe, to be sailed. Extreme conditions that are dangerous for cruise occur on January 23-29 2018 when the maximum wind speed and maximum wave height reach 25 knots and 3.8 m, respectively. The Kalianget-Kangean cruise channel is relatively safe except in the time period of the Western Season in December, January, February when wind speeds exceed 21 knots and wave height of 2.5 m.

Study of Submerged Breakwater for Coastal Protection in Mantiung Beach

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Abstract. Mantiung Beach, a tourism beach, is located at Pandeglang city, Banten. Mantiung Beach has a erosion problem due to wave. In this paper, a submerged breakwater is proposed to protect the beach from erosion and to keep natural view of the sea. Ten years wind data is collected for hindcasting analysis to calculate the design wave height. The design wave height that used to design the submerged breakwater is the wave height in breakwater location which is determined from wave simulation using Delft3D Wave. Wave simulation is also run for two submerged breakwater configurations. Based on wave simulation result, the elevation of top breakwater at – 1 m LWS give better transmission coefficient.

Dealing with the Port Impacts to Ensure the Sustainability of Port-Cities Development by Employing Integrated Policy: A Case Study About Land Use and Transport in Semarang

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Abstract. The role of port-cities has increased along with increasing the function of ports as backbone of global economy in distributing logistics throughout the world. Port development has impacts on the growth of port-city. The positive impacts are economic growth and social interaction. While negative impacts affected on aspects of land use, transport and environment. Semarang is a major port-city in Indonesia that takes advantage of positive impacts and cannot be separated from negative impacts. The aim of this research is defining role of integrated policy making in achieving sustainable development in Semarang. The research design is a case study using qualitative data analysis and con/text analysis methods. Fieldwork has been conducted in 2018 for 2.5 months by observation and in-depth interviews. This paper discusses land use and transport policy of Semarang and its response to impacts of port in Semarang City only. The preliminary results indicate complexity of authorities, establishment of cross-sectoral team, land use and transport response so that policy integration process will be more challenging. Ultimately, the paper observes that to come to integrated policy making a framework for sustainable land use and transport in port-cities relation needs to get a priority to realize the desired policy outcomes.

Study on the Influence of Tool Rotating Speed on the Weld Joint Strength of Friction Stir Welding Method

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Abstract. Aluminum 6061 is a lightweight metal and has a corrosion resistance and good conductivity. In the marine industry, aluminum 6061 is widely used for ship construction. Friction Stir Welding (FSW) uses the principle of utilizing friction from a rotating work piece with another stationary work piece so that it is able to melt the stationary work piece and finally connect it together. The focused of this study is to determine the impact strength, tensile strength and micrographic structure of the butt joint of FSW Welding. The feed rate is determined as 10 mm / min with the variations of rpm speed is defined as 1640 rpm, 2620 rpm, and 3820 rpm. Tensile strength, impact strength and micrographic evaluation will be discussed to assess the effect of rotational speed on the strength of the welded joint produced by Friction Stir Welding.

Hydrodynamics Stiffness for Structural Analysis Boundary Condition of Floating Cylindrical Structure in Waves

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Abstract. Motion analysis is one of the mandatory aspects to predict the performance of a floating structure, as well as how its structural strength under certain wave load. However, in majority of floating body performance prediction, the calculation of motion and strength performance is done separately. Practically, engineers calculate the motion and hydrodynamics forces that work on the structure, then do separate calculations on the structure to predict structure's strength. These separate calculations often use assumptions that tend to be unrealistic, either over constrained or under constrained. This paper provides an alternative to the constraint problem by introducing hydrodynamic stiffness as boundary conditions, instead of using fixed or simply supported boundary conditions, spring boundary conditions are applied with hydrodynamic stiffness of floating body properties. It is expected that this model will provide a more realistic constraint to the analysis that will be carried out. Several configurations have been made to assess the best configuration to represent the proper boundary condition. The results achieved are very promising, where the boundary condition resulting a close natural frequency approximation compared with the analytical calculation. This configuration is hoped to be the baseline of more complex structure to be carried out in this research, in order to represent a more realistic structural boundary condition in future modelling.

Analysis of the Effect of Addition of Bilge Keel, Bulbous Bow, Skeg and Combination on Value of Ship Resistance and Movement of Ship Using CFD Method

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Abstract. Resistance and ship movement have an important role in the design of the ship because it is related to the power requirements and the ability of the ship to deal with waves in the sea. The large value of resistance and ship movement causes the ship to be ineffective and uncomfortable to use. Therefore, during the design and analysis process, the addition of components was carried out so that the value of the resistance and ship movement was not large. Therefore the hull design must be made as well as possible and additional components can also be added to the hull such as the Bulbous Bow, Skeg, and Bilge Keel which can improve the ship's ability to operate. Based on several studies it is known that the best bulbous bow model is the V model, the best Skeg form is the Foil model and the most optimal Bilge Keel form is the hollow triangle shape, where the most optimal variation is to reduce the value of the resistance and ship movement. To achieve these objectives, several stages are carried out, namely making a model, then analyzing the model using CFD software. The results of this study indicate that the best resistance value occurs in the ship model with Bilge keel with a value of 8.8 kN or a decrease of 13.72% of the ship without the addition of components. Then for the value of the Motion obtained Heaving value of 0.84774 m on the ship without the addition of components, for the best pitching value that is on the ship with the addition of Bulbousbow, Skeg and Bilge keel with a value of 0.080775 radians and for the best Rolling value is on the Ship with additions Bilge keel with a value of 0.0000082404 rad.

Hydrodynamic Modelling of Flood Prediction in Welang Estuarine

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Abstract. Inundation of river mouths can be caused by an increase in river water level due to increased discharge and tides from the sea. In this study, modelling is carried out to determine the effect of river discharges by tides during the monsoon on the risk of flooding. The hydrodynamic model (MIKE21) and the hydrological model (HEC-HMS) are combined to get the parameters that cause flooding such as tides, river discharge, and the interaction of both. MIKE21 is used to calculate the prediction of the highest tides, and HEC-HMS is used to calculate river flows downstream. West and East Monsoon winds often cause extreme weather changes such as high rainfall which causes a significant increase in river discharge. West and East monsoon winds are the determining parameters. The proposed model is expected to be of great aid in producing the flood hazard mapping with enhanced accuracy.

Wind and Significant Wave Height at Indonesian Seas Based on ERA5 Reanalysis Data from 2008 to 2018

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Abstract. Wind and waves are important elements in coastal engineering and oceanography. Research on variations off in-situ winds and wave's measurements is still very limited in Indonesia. Therefore, wind speed and significant wave height data from ERA5 reanalysis data, provided by the European Centre for Medium-Range Weather Forecast (ECMWF), and is used in this study. ERA5 data that has a spatial resolution of $0.25^\circ \times 0.25^\circ$ is used to analyze correlation between wind speed and significant wave height at Indonesian seas in 2008 to 2018. The results show that wind speed and significant wave height in the closed water (inter-island in Indonesian archipelago) has a higher correlation compared to the Indonesian open sea that facing the Indian Ocean or Pacific Ocean.

Distribution of Current and Temperatur at Welang Estuary, Pasuruan

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Abstract. This research was conducted at the Welang River Estuary located in Pasuruan. The Welang River has a water catchment area of about 518 km² with a river length of around 36 km and an average width of about 25 meter start flows from Lawang District to Malang Regency and empties into the Madura Strait in the Kraton District west of Pasuruan. Welang River often experiences floods. Therefore, a study was conducted to study the dynamics of the Welang river estuary using MIKE 3 modeling. The simulation was conducted to analyze several parameters such as flow patterns, salinity distribution patterns and temperatures around the welang river estuary. How the results of the analysis will determine the type of Welang river estuary based on its characteristics. So the results of research data are expected to be material information and consideration in overcoming the Welang River which is the main problem of flooding. Especially for the government and Pasuruan regional stake holders.

Modelling Water Quality in Welang River Estuary, Pasuruan

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Abstract. Land use activities for agriculture, plantations, industrial areas, fishponds and settlements in the Welang watershed can affect water quality in the Welang estuary. Waste disposal from these activities into the river can reduce water quality. The purpose of this research is to apply numerical modelling for assess turbidity, pH, and Dissolved Oxygen (DO) as water quality parameters in Welang river estuary. The research method by 2D models with Hydrodynamic and Ecolab module for distribution of DO and acidity. Current flow into upstream at the tide. At the ebb, the current flow into downstream. Water quality in Welang estuary none exceed the standard quality with maximum DO value is 6.93 mg/l and pH value is 7.8.

Mechanical Behaviour of Coir and Glass Fibre Reinforced Polymer Composites Material: A Literature Study

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Abstract. L Glass fibre reinforced polymer (GFRP) has been commonly adopted in marine engineering for vessel hull as a high performance and durable material. Otherwise, the several efforts to replace the synthetic fibres with the natural fibres due to the increase of environmental awareness and oil resources depletion have been conducted. The natural fibres such as coir, jute, hemp, ramie and bamboo which are available in abundance, biodegradable and low density is the main reason to explore the application potential in shipbuilding sectors. In this paper, the mechanical behaviour of coir and glass fibre reinforced polymer composites material is reviewed. The surface modification of the natural fibre are also reviewed and it is found that the surface treatment is an alternative method to improve the natural fibre mechanical properties for successfully utilizing the natural fibre in marine and shipbuilding applications. From this review of both coir and glass as a reinforced fibre, it is concluded that a focus on the material properties of coir fibre combined with the glass fibre as a hybrid material is needed if the behaviour of vessel hull composite structures during sailing operation and other sources of external load is to be fully understood. and use activities for agriculture, plantations, industrial areas, fishponds and settlements in the Welang watershed can affect water quality in the Welang estuary. Waste disposal from these activities into the river can reduce water quality. The purpose of this research is to apply numerical modelling for assess turbidity, pH, and Dissolved Oxygen (DO) as water quality parameters in Welang river estuary. The research method by 2D models with Hydrodynamic and Ecolab module for distribution of DO and acidity. Current flow into upstream at the tide. At the ebb, the current flow into downstream. Water quality in Welang estuary none exceed the standard quality with maximum DO value is 6.93 mg/l and pH value is 7.8.

Numerical Modeling of Vertical Axis Hydro Turbine With Experimental Validation

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Abstract. Ocean current energy is an energy source that has great potential in Indonesia but is dominated by low current velocity. Many potential locations such as Riau, Boleng, and Mansuar strait only have the maximum current speed by 1.39 m/s, 1.5 m/s, and 1.79 m/s. Therefore, energy conversion technology is needed that is able to exploit the potential of low-speed current energy into electricity. Darrieus turbine is a type of vertical axis hydro turbine that is suitable to be developed in Indonesian waters because it can work at a lower current speed than the other types. To study the Darrieus turbine performance on a low-speed ocean current, numerical modeling based on computational fluid dynamics (CFD) which is validated with experimental results must be done. The numerical validation uses Darrieus turbine experimental data with diameter 0.91 m, chord blade 0.07 m and span 0.7 m. The validation process starts with defining boundary geometry dimension, meshing construction and turbulence model suitable. The simulation is carried out at the current speed of 1.5 m/s with TSR variations. The validation parameter is the torque value that correlates with the coefficient of turbine performance. As the result obtained validation rate with the average error of 7.3%.

Variation Number of Blades for Performance Enhancement for Vertical Axis Current Turbine in Low Water Velocity in Indonesia

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Abstract. Asosiasi Energi Laut Indonesia (ASELI) has provided the results of the ratification of the energy potential of ocean currents in Indonesia amounting to 17,989 MW. This amount is very large considering that Indonesia is classified as in low water velocity. The vertical axis current turbine is proposed by many researchers because they have lower performance than the horizontal axis current turbine. So this study proposes research to improve the performance of the vertical axis current turbine for low water velocity by varying the number of blades. The type of blade NACA 634021 chose because has good performance for the vertical axis current turbine of Darrieus type. The model of the turbine is simulated by Computational Fluid Dynamics (CFD) with variations of the number of blades namely, 3, 4, and 5. The totals of the statistic element of meshing are 16,090, 167,020, and 174,375 at the number of blades 3, 4, and 5, respectively. The inlet velocity set at 1.5 m/s and tip speed ratio (TSR) are 1.5, 2, and 2.5. The final results of this study show that the blade numbers can improve the performance of the turbine model on all TSR ranges. The blades 4 and 5 are increased by 21.8% and 15% respectively from the blades 3 on TSR 2.5. The highest performance is obtained by the number of blades 4 on TSR 2.5 with the value of the power coefficient (C_p) 0.26.

The Effect of Shortcut Channel on Flooding in Welang River

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Abstract. In coastal areas, the co-occurrence of high sea level and precipitation resulting in large runoff may cause flooding. This report will mainly focus on the downstream of Welang river which overflowed more than ten times per year due to high sedimentation rates. To reduce sedimentation rate, a study will be done on the meander in the downstream to find out to what extent is the meander affecting the flow rate and increasing flooding. A new design will be made, a shortcut channel, with models that shows the difference between the new channel and the existing channel concerning flood rates, water velocity and sediment rate. The results are showing high water velocity with less sediment rates since it is flushed out towards the sea. Moreover, flood rate decreases in comparison to the existing channel. In addition to that, the increase in water velocity will increase bank erosion, but it will be controlled by including mangroves on the banks. It will also maintain water quality and clarity, filtering pollutants and support threatened and endangered species.

Evaluation of Underwater Pipe/Cable Design Criteria Due to Safety Requirement Based Hydrodynamic Load.

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Abstract. Many things can be done by piping engineers to protect pipes so they are resistant to environmental and man-made hazards. Factors that can transport the pipe are usually shipwrecks, faults in dropping and pulling anchors, dredging activities, fishing activities, and the consequences of exploring. Burial pipes are the solutions most often used by engineers for protective pipes. However, the burial of the pipeline has already begun due to unfavorable seabed problems or needs to be done if there are other routes. This paper will explain the test results related to pipes with free spans and calculation of rock dimensions to protect pipes from environmental and external loads for pipes.

Analysis of the Implementation of the Lamong Bay Reclamation Based on the Implementation of UU No. 27 of 2007

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Abstract. Lamong Bay coast is a rich coastal area, this is because there are many fish habitats, marine biota habitats, mangroves, and other fisheries activities. But in 2010 there were reclamation activities by PT. Pelindo III which aims to increase the loading and unloading capacity of container terminals, waterfront city development, and multi-purpose terminals. By implementing UU No. 27 of 2007, the community service paper based on this research will explain the results of the study and evaluation in the implementation of the reclamation in Lamong Bay which will then be socialized to the community so that harmony is created between business permit holders and coastal communities of Lamong Bay. any things can be done by piping engineers to protect pipes so they are resistant to environmental and man-made hazards. Factors that can transport the pipe are usually shipwrecks, faults in dropping and pulling anchors, dredging activities, fishing activities, and the consequences of exploring. Burial pipes are the solutions most often used by engineers for protective pipes. However, the burial of the pipeline has already begun due to unfavorable seabed problems or needs to be done if there are other routes. This paper will explain the test results related to pipes with free spans and calculation of rock dimensions to protect pipes from environmental and external loads for pipes.

Room for the Welang river

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Abstract. With the change in climate heavy rain events are becoming more frequent in the past years. Excess run-off water usually ends up in rivers creating an increase water volume in the river for a short time period. This short and intense increase of water usually creates flash floods which damage nearby infrastructure and villages. This was the case for the Welang river located on the eastside of Java which flooded in 2011 after a period of heavy rain. This report will look into one policy which will help to prevent flooding in the future, Room For The River. By constructing flood plains which can be used for water storage the risk of flooding can be reduced and thus create room for the river. Because these flood plains are only in use for a short time span secondary functions can be applied to them. Some suggestions would be temporary farms or sediment catcher for brick making.

Welang River- a Scheme for Plastic Recycling and Reuse

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Abstract. The Welang River suffers from severe problems due to pollution, especially with plastics, and a high flood risk. This research was conducted to examine the possibilities of decreasing the level of pollution in the downstream area of the river while utilizing the retrieved materials to improve the resilience against flooding. A scheme for retrieval, recycling and application possibilities for the reclaimed material was created, with a special focus on being a low cost solution and not asking for high tech machinery to make it possible to boost the local economy and create opportunities for local communities by getting involved in his project. Floating barriers are used to trap the waste from the river on the riverbanks to make an easy retrieval of waste material from the stream possible and after a recycling process plastic granulates can be created. The application of recycled plastic granulates in concrete and brick making is a promising solution for the two main problems of the area. Improved resilience and strength of these materials in comparison to traditionally used materials makes them suitable for the implementation in revetments and safety structures such as dams.

Risk Analysis of Delay on Offshore Pipeline Installation Study Case at Mahakam Blok

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Abstract. Pipeline has an important role in oil and gas industry to transfer oil or gas that have been discovered to production station, one of which is pipeline system belong to PT. X located in Mahakam Block, East Borneo, Indonesia. Considering it's important role, pipeline installation can't experience delay and needs to operate as planned. This offshore pipeline installation experienced 21 days delay. This offshore pipeline installation project was done by PT. K as contractor and supervised directly by PT. X. This project started at December 5th 2018 and finished at February 3rd 2019. To analyze the delay of this project, this research uses 2 combine methods, which are fuzzy trapezoidal and bowtie analysis. The fuzzy fault tree analysis (FTA) diagram will generate the probability value of top event project delay of offshore pipeline installation. The fuzzy event tree analysis (ETA) diagram will generate how much fine that the contractor has to pay and the risk level from every factor that causes the delay. The results from bowtie analysis will determine preventive value which will be used as precaution and mitigation which will be used as reduction due to delays.

Two-Dimensional Numerical Study of Scour beneath Subsea Pipeline under Regular Wave Condition

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Abstract. This paper presents analysis of the results of a numerical model for local scour that consider a pipeline laid on an erodible bed, combined the hydrodynamics and morphological processes. By using commercial CFD computational code the governing equations of the flow model using two interfaces in the simulation domain, the interface of the free surface tracked by Volume of Fluid (VOF) method and the bed interface solved by a finite area mesh developed from the finite volume mesh of the bed boundary. The morphological change integrated with the hydrodynamics field through deformation of the computational mesh. The results prove the stage of onset scour is followed by the stage of tunnel erosion and lee-wake erosion, in this stage velocities break in the gap and resulting a large shear stress on the bed below the pipe. Based on these, the present results have relevance to the prediction of span growth rates which is the basic of the three-dimensional scour propagation.

Influence of Waters in Silvofishery Ponds on Wonorejo Mangroves that Contaminated by Heavy Metals Pb, Cd, and Cu Toward Aquaculture Animals

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Abstract. Wonorejo silvofishery ponds obtained fresh water from two rivers, namely Jagir River and Afour River which flow through the city of Surabaya before entering Wonorejo mangroves area. The white shrimp and milkfish that have been cultivated in silvofishery pond often fail to be harvested. Preliminary observations found that Pb content in Wonorejo ponds reached 0.082 ppm. The purpose of this study is to determine the toxicity of waters in the Wonorejo silvofishery ponds that contaminated with heavy metals (Pb, Cd, and Cu) toward milkfish (*C. chanos*) and white shrimp (*P. merquiensis*). The research method includes two stages; (1) measuring the heavy metal concentration in ponds water and rivers in Wonorejo and (2) testing the toxicity of waters silvofishery ponds toward aquaculture animals in the laboratory. The results of the tests revealed that the concentrations of Pb, Cd, and Cu in R. Afour were 0.284; 0.0380; and 0.017 ppm respectively. On the other hand, the concentration of Pb in the pond water is 0.304; 0.047; and 0.024 ppm. Laboratory test shows that milkfish can survive in the condition where water is contaminated by heavy metals until the concentration increases ten times from the concentration of metals in nature, as long as the other environmental parameters are appropriate. Meanwhile, white shrimp cannot survive more than two days in these conditions.

Environmental Conditions in Karimunjawa to Support the Integrated Multi-Trophic Aquaculture (IMTA) Program

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Abstract. Karimunjawa National Park is a nature conservation area managed by the zoning system. Of the 9 zones that have been determined, one of them is the Maritime Cultivation Zone which is intended to support the interests of aquaculture. Aquaculture based on Integrated Multi-Trophic Aquaculture (IMTA) is a development system that supports cultivation by integrating a variety of different species. This study aims to analyze the suitability of Karimunjawa waters based on the measurement of environmental parameters such as temperature, DO, Ph and current salinity to support IMTA in Karimunjawa waters and observe the coral reef ecosystem as a supporter of assessing the condition of the aquatic environment. Based on the results of measurements of environmental parameters, the results obtained are still relatively good quality based on water quality standards. The average water temperature is 29.7 °C, dissolved oxygen (DO) is 7 mg / l, pH is 8.1 and salinity is 32.7 ppt.

Experimental Study on the Effect of Mooring Angle on Mooring Tension of Porous Floating Breakwater

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Abstract. Floating breakwater is commonly used as an alternative solution to conventional fixed breakwaters and can be used effectively in coastal areas in environments with relatively small waves, poor soil conditions, deep sea depth, and shore with intense erosion rate. Because floating breakwater is a floating structure, a mooring system is used to maintain its position. This study aims to determine the mooring tension that occurs on each mooring rope with a physical model tests. This research was conducted on the type of pontoon and porous. In this study it was found that on a porous breakwater, the greatest maximum tension occurred at the mooring angle 30⁰ (3.98 N) while the smallest maximum tension occurred at the mooring angle 60⁰ (0.73 N). The pontoon type also gives the same result where the 30⁰ mooring angle gives the highest maximum tension (4.91 N) while the 60⁰ mooring gives the smallest maximum tension (1.27 N). In addition, porous floating breakwaters can provide smaller mooring tension than the pontoon type.

Numerical Study of Bilge Keel Length Variations of Floating Breakwater to Optimize Transmission Coefficient

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Abstract. Waves and winds that move dynamically often cause damage on the coast so a protective beach building is needed. Breakwater is a coastal protection structure to destroy the incoming wave energy before reaching the coast. One type of breakwater is floating breakwater, this structure has an advantage compared to the fixed breakwater. Research on floating breakwater has been developed with the main goal of being the most efficient structure and can absorb waves well. In this research, floating breakwater simulation modelling with variations in bilge keel length was conducted. Validation is done by comparing the results of (GH Dong, 2008) with numerical results of the breakwater pontoon type. There are four variations of bilge keel length, namely 0 m, 0.6 m, 0.9 m, and 1.2 m. So, four variations can be concluded that the longer the bilge keel, the better absorb the waves.

Transmission Coefficient Analysis of Notched Shape Floating Breakwater : A Numerical Study Using Volume of Fluid Methods

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Abstract. Breakwater is one of the coastal structures that was built with the aim to protect the coastal area against waves. This structure is generally designed to protect ships and facilities at the Port. Breakwaters that are often applied are conventional breakwaters such as the rubble mound type. Climate change that causes tidal variations and sea level height which tends to change at any time, as well as unsuitable soil conditions to receive large structural loads will be costly and more economical if using floating breakwater. Over time, the demand for floating breakwater development increased so that scientists and engineers did a lot of research development both in physical experiments and numerical models. In this study numerical simulations will be conducted by exploring the porous shape of floating breakwater from the Christensen experiment. The aim is to get the most optimal transmission coefficient. Numerical simulations used using the Computational Fluid Dynamics (CFD) method with the help of Flow 3D Software. Validation is done first between numerical tests and experiments to get a valid approach before exploring the development of shape.

Time Acceleration Analysis of The Kijing Terminal Development Project in Mempawah, West Kalimantan

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Abstract. Every project leader wants their project run smoothly according to plan. In order to run project smoothly, all phases of project, from planning until controlling must be considered essential. The success or failure of a project depends much on accurate planning and effective controlling. Failure to create such planning and controlling will create delay in project, quality decreasing, and added cost. Project scheduling management is one of the important factors in completing a project to get optimum results. Kijing Terminal Development Project In Mempawah, West Kalimantan is the object analyzed in this final project. Kijing Terminal will be developed in 3 (three) stages, namely: Initial Phase, Phase I and Phase II. Kijing Terminal has offshore and onshore facilities. In this research, the time acceleration analysis of the project is done by using Presedence Diagram Method to find out activities that are on the critical path. There are 12 activities that are in a critical path on this project. After identifying the critical path, the analysis was carried out using the Monte Carlo Simulation method to find out the probable duration of project completion from several duration targets. Next, the time acceleration is carried out using Time Cost Trade Off method with the scheme of adding overtime hours to workers. In this analysis, three scheme of adding overtime hours to workers were made.

Offshore Pipelines Replacement Project Planning Study Case At North West Java

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Abstract. PT. X plans to carry out offshore pipeline repair and replacement. This pipeline repair and replacement project covers aspects of engineering, procurement, construction, and installation. The location of the pipeline is located 80 - 200 km northeast of Jakarta. The core of this project is to replace pipes that are no longer suitable for use by installing new pipelines in existing deteriorated pipelines. This project is a complex project because it has 3 project objects with different pipe sizes and lengths. There are 3 pipelines that are planned to be replaced, namely: (1) Pipeline A with a size of OD 12" and length of 1 km, (2) Pipeline B with a OD size of 10" and a length of 7.2 km, (3) Pipeline C with an OD size of 8" and a length of 7.2 km. Then it is necessary to anticipate project risks in the form of overtime (project delays) which can have an impact on project over cost (cost overruns). Many factors can make a project too late, for example the weather factor that makes it impossible to do work or the factor of lack of utilities and construction equipment, late purchase of material orders, lack of experienced and skilled human resources in the field, late costs down for activities purchase on the project. In addition, the occurrence of delays in critical activities because critical activities have a large impact on the duration of the project. Therefore it is necessary to have a delay analysis by looking for activities that are critical activities on the project and the factors that can cause overtime in this new pipeline installation project because it can harm the owner and contractor. This research will discuss the determination of critical paths, the determination of root causes or factors that can cause overtime, the risk of overtime, and what improvements can be made in the new replacement pipeline project of PT. X. There are 4 indications of delays in project work. The indication of delays occurred in December 2017, March 2018, September 2018, and October 2018. The largest deviation between planned value and earned value occurred in October 2018, which was 5.05%. This study uses six sigma methods, critical path, fault tree analysis, and event tree analysis. The results of the critical path method will determine the critical project activities. The results of the fault tree analysis will get the probability value for the top event of the project delay. The results of the event tree analysis will get the risk level of each factor causing the delay. The results of the six sigma will get the root cause, the value of sigma, and what corrective measures are taken.

Training of Fiberglass Boat Repair for Gresik's Fisherman to Improve Fiberglass Boat Quality of Boat Construction Following the Indonesian Classification Regulations

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Abstract. There are several types of damage to the fiberglass boat, including holes, cracks, and deformation on the fiberglass boat hull. At this time, fiberglass boat's damages are repaired by fishermen with their knowledge without considering classification regulations. Therefore this paper will explain training on repairing fiberglass damage using a syllabus and methods by Indonesian classification bureau (BKI) regulations.

Implementation of Lean Project Management in Offshore Pipeline Installation Project

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Abstract. Project implementation is inseparable from the risks that can affect the project will go according to plan or not. Therefore, good project management is very important to avoid project problems such as delays. Project delays can be caused by unproductive activities and the elements involved. The unproducibility becomes something that does not have added value or in Lean Project Management known as Non Value-Added Activities (waste). In the implementation of the Offshore Pipeline Installation Project undertaken by PT.X, there was a delay in the process, therefore a risk analysis of the causes of the delay is required with the Lean Project Management approach to identify waste by applying the Fishbone Analysis method and project risk analysis using the Fuzzy Failure Mode and Effect Analysis method that will prevent or detect the failure mode earlier. And to estimate the project, it is applied by using the Critical Chain Project Management (CCPM) method which will produce project optimization to meet the LPM criteria, namely eliminating invaluable activities in order to make the project more effective and efficient.

Investigation Method for Determining Potential Liquefaction Hazards in Coastal Areas

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Abstract. Indonesia is an archipelago country that has a very long cost line and has many coastal cities. In addition, Indonesia has many active and non-active volcanoes. Sediment materials resulting from the eruption of the volcano carried over by the river and settled in the estuary or coastal area and formed a land that later become the coastal city. Indonesia is also an area prone to earthquakes because of the location close to the active fault areas. One of the disasters caused by earthquakes is liquefaction, where the soil loses its bearing capacity. Some factors that cause liquefaction are soil condition, ground water table, earthquake intensity and so on. This research will determine the method used to assess the level of insecurity of an area against liquefaction hazard. The method is working step that must be done to an area so that it will facilitate the authorities and stakeholders to take decisions.

Design of OWC Type Owc (Oscillating Water Column) Power System Based on Automatic Valve Integrated Thermoelectric Generator with Scada Technology

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Abstract. The vast sea area certainly has great energy potential, especially ocean wave energy, especially in Gunung Kidul. The marine energy can be converted into electrical energy that can be enjoyed by the people around Gunung Kidul. Gunung Kidul already has a type of Oscillating Water Column (OWC) wave power plant. This type of generator has advantages that are simple technology, relatively low maintenance costs, and the distribution of power generated. However, OWC type power plants in Gunung Kidul have an efficiency level of only 11%. The reason is that OWC only produces electrical energy during tidal waves, during low tide, OWC cannot produce other than that, the utilization of solar energy has also not been utilized around the power plant. In addition, OWC has not been facilitated by SCADA (Supervisory Control And Data Acquisition) so that when the interruption of the time required is at least 30 minutes and the load situation is not monitored in realtime. The purpose of this research is to design a prototype of an OWC (Oscillating Water Column) ocean wave power generation system integrated thermoelectric generator with SCADA (Supervisory Control And Data Acquisition) Technology. After testing the prototype, the system experienced a 46% increase in power compared to OWC BPPT Gunung Kidul. So that if implemented the system is able to optimally increase the efficiency of electric power generated by the OWC BPPT Gunung Kidul plant.

Stress Concentration Factor of A Two-Planar Double KT Tubular Joint Due To Out-of-Plane Bending Loading

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Abstract. An over or under prediction of Stress Concentration Factor (SCF) of atubular joint could decreasing the accuracy of fatigue life prediction of the structures. Therefore the SCF is one of the most important parameters in the tubular joint fatigue analysis. This situation leads to this study to investigate the proper SCF of multi-planar tubular joints as partof offshore structures especially due to out-of-plane bending loading. In this paper a 60° two-planar double KT (DKT) tubular joint of a wellhead offshore platform is modelled as a finite element model with bending loads acting on the braces. The effect of dimensionless geometrical parameters (β , τ , γ) on the SCF distribution along the weld toe of braces and chord on the joint are investigated. The finite element model of the joints cover a wide range of geometrical parameter. Validation of the finite element model has shown a good agreement to the global structural analysis results. The result of this study is the SCF value of any non-dimensional variation due to in-plane bending loading case with the SCF was significantly differ to the SCF of a uni-planar KT-joints. The results of parametric studies show that increasing the β leads to decrease of the SCF. For load case #3, peak of the SFC slightly shift from outer saddle point into positions of around 113° for all the β . The peak of the SCF mostly occurred at the outer saddle point.

Analysis on The Effect of Subsea Buoy to The Tension of Spread Mooring System

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Abstract. The problem that might occur in floating structures with mooring systems is clashing between mooring lines with subsea equipment, for example pipelines. Addition of subsea buoys on the mooring line can lift the mooring line so that it can avoid clashing. The addition of the subsea buoy can affect tension on the mooring line. So in this final project will analyze the effect of subsea buoy to the tension of mooring line with a variation position of subsea buoy. Variations on the position of one subsea buoy is arranged at the distance of 605 m, 577.5 m, 550 m, 522.5 m from anchor and two subsea buoys at the distance 605 m and 467.5 m from the anchor. The analysis was carried out on stand alone and offloading conditions with wave directions 0 °, 45 °, 90 °, 135 °, 180 °. The results after the addition of subsea buoys it has a smaller tension on the mooring line. The result of the variation of subsea buoy position, the optimum position to get the smallest tension value is when the variation of two subsea buoys with a distance of 605 m and 467.5 m from the anchor. From the results of the analysis there is also no clashing between the mooring line and pipeline.

Experimental Study of the Effect of Waves on SPAR Responses with and without Heave Plate in Intact and Damaged Mooring Systems

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Abstract. This undergraduate thesis discusses the effects of heave plate utilization against dynamic responses of SPAR due to random waves. The analysis has been done by numerical and experimental methods. The experimental method was held in Maneuvering Ocean Engineering Basin of Balai Teknologi Hidrodinamika (BTH), BPPT. The model scale of SPAR was constructed based on the classic SPAR prototype with taut mooring systems using four identical moorings in a 1:125 scale factor. Both experimental and numerical studies were conducted in time domain analysis with heave plate utilization in the keel of SPAR as the main variable. The diameter of the heave plate has a 1.5 ratio to the outer diameter of SPAR. Each analysis was conducted in intact and damaged mooring systems. The analyzed variables of dynamic responses are the surge, heave, pitch, and maximum offset of the SPAR. Results of both the experimental and numerical studies were then compared, showing that the heave plate does not affect surge and pitch responses significantly. On the other hand, it impactfully and consistently reduces the SPAR heave responses in every condition of analysis in both the numerical and experimental methods. The most significant reduction of SPAR heave response occurred when the mooring systems were in damaged condition, i.e., 33.29% and 27.84% heave reduction in the experimental and numerical method, respectively. The study also shows that the heave plate reduces SPAR maximum offset up to 31.69% in experimental analysis and 11.22% in numerical analysis.