Monte Carlo Simulation for Reliability Hydraulic Stability of Rubble Mound Breakwater Armour at Sudimoro Power Plant, Pacitan

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Abstract. In breakwater design, the conventional deterministic methods are commonly used. This method based on the concept of load, where the design load should not exceed the carrying capacity of the structure. However, the design load itself can be identified using statistics analysis, such as the average value of a 60-years return period of wave. Most of the available design formulas only provide a link between wave characteristics and some structural responses, such as run up, overtopping, or armor layer damage. In this paper, a reliability analysis of the breakwater hydraulic stability of Sudimoro Power Plant, Pacitan is performed with Monte Carlo simulation. The formulas used in hydraulic stability calculations were Hudson and Van der Meer formulas. The analysis shows that the breakwater structure in Pacitan Power Plant already meets the hydraulic stability design with reliability of 98.8% for Hudson and 95.12% for Van der Meer formula. This paper also presents the reliability of breakwater armour units proposed by authors.

1. Introduction

The study was performed on the breakwater of Sudimoro Coal power plant, located in Sukorejo village, Sudimoro sub-district, about 30 km east of Pacitan, East Java, Indonesia. The Sudimoro Power Plant support the energy diversification program for power generation to non-fuel oil by using low-calorie coal to produce steam to drive the turbines in this power plant. To protect the coal unloading processes in the jetty, a breakwater as shown in figure 1 were constructed using Dolos as armour unit.

The common method in breakwater design was the conventional deterministic method. A design method based on the concept of a design load that must not exceed the carrying capacity of the structure. The design load can be identified using a probabilistic method, for example finding the average value of the 60-year wave return period. In most cases, resistance is defined in terms of the load causing a particular design impact or damage and is not given as a major force or deformation.

A reliability-based risk assessment is needed on a coastal structure based on probabilistic methods, where the uncertainty (stochastic) of the load is involved and the strength variables are taken into account [1] The risk that the system is unable to meet demand is defined as the probability of failure over a specified period based on the specific system operating conditions. Whereas the reliability is defined as a probabilistic assessment of the likelihood that the system will perform adequately for a specified period under known operating conditions. System risk and reliability are defined as the probability of failure and non-failure over the period determined by the system.
A *monte carlo* simulation will be used to analyze the reliability of the rubble mound breakwater armour unit at Sudimoro Power Plant. The reliability of Dolos, BPPT Loc and Tetrapod as breakwater armour unit were investigated. In previous study [2] the stability of armour unit, was evaluated using Hudson method. In this study the *Van Der Meer* method were applied and compared to previous study.

![Figure 1](image-url). Sudimoro Power Plant and Waverose Diagram 1999-2019.

2. Materials and Methods
The Sudimoro Power Plant, Pacitan is located on the southern coast of Java islands which is famous for the ferocity of the waves. As shown in figure 1, the prevailing waves come from South East direction. The breakwater at Sudimoro Power Plant seems not properly designed due to frequent structural damages. The wave overtopping occurs frequently, results in unsettled waters in harbour basin.

The breakwater was designed to provide calm water for safety of coal unloading in Sudimoro Power Plant. The design load was usually defined statistically as the characteristic value of the wave as the main load. Designing with this method is usually known as conventional methods.[2]. The weakness of this method is that the design load is determined without any consideration of the uncertainties [3].

Due to the uncertainties, it is necessary to carry out a risk assessment analysis based on the reliability of the coastal structure. By using the application of probabilistic methods or reliability and risk analysis (risk & reliability analysis), one can produce an engineering system that is more efficient and also meets the expected quality standards [4].

2.1. Long Term Wave Analysis
For long term wave analysis, the Weibull distribution were used in this study. In this method, the significant wave heights with various return periods were estimated. As the Weibul distribution using arbitrary $\alpha$ parameter, the approach taken is to try several of $\alpha$ value for available data and then choose the one that gives the best fit results in the regression line. The wave data were sorted from the largest data to the smallest data. The analysis of the extreme values of the wave data set used two relationships with the statistical set analyzed by linear regression to determine the most suitable line [5]. The following equation shows the probability distribution (P) of Weibull:

$$P = 1 - \exp \left( - \left( \frac{H}{\beta} - \gamma \right)^{\alpha} \right)$$

where $H$ is the wave height, $\alpha$, $\beta$ and $\gamma$ are parameter in Weibull distribution. $\beta$ and $\gamma$ were obtained form the regression line equation in Weibull distribution analysis.

2.2. Hydraulic Stability in Breakwater Design
The rubble mound type breakwater were built in Sudimoro Power Plant. This type of structure is
commonly built in Indonesia. Rubble mound breakwater is a breakwater where the core material is relatively small which is covered by one or more secondary layers and on the exposed side is protected by a larger armour unit or concrete armout unit. The design process of the breakwater structure has major implications for the benefit of functional design and facility costs. Therefore numerical wave modeling needs to be carried out from the early stages of planning to determine and optimize the appropriate layout.

2.2.1. Hudson formula. The required armor unit size for concrete armor units in a double layer can be assessed by a stability formula such as Hudson formula [6]. For concrete armor units the Hudson formula can be rewritten to a form of Stability Number (Ns) using the significant wave height, Hs (m), and the nominal diameter of the unit, Dn (m) as presented in equation (2),

\[
\frac{H_s}{\Delta D_n} = (K_D \cot \theta)^{1/3}
\]

where \(\Delta\) is the specific density ratio defined as : \(\gamma_a / \gamma_w - 1\), \(\gamma_a\) is specific density of armour unit and \(\gamma_w\) is specific density of water, \(K_D\) is stability coefficient of armour unit, and \(\theta\) is the slope of breakwater.

2.2.2. Van der Meer formula. The formula on equation (2) above can be extended by including damage level as proposed by Van der Meer [7]. Equation (3) below shows as the expression for the stability number Ns.

\[
\frac{H_s}{\Delta D_{n50}} = 0.7(K_D \cot \theta)^{1/3} S_d^{0.15}
\]

where \(S_d\) is the damage level parameter (-), \(S_d = A_e / D_{n50}^2\) and \(A_e\) is the eroded area in a cross-section (m²). 2 for minor damage level and 12 for heavy damage.

2.3. Failure Function
The failure mode of hydraulic stability for the rubble mound breakwater structure is obtained from the Van der Meer and Hudson stability equations expressed in equation (2) and (3) above. Furthermore, the performance function of the breakwater based on Hudson equation:

\[
D_n = \frac{H_d}{\Delta(\sqrt[3]{K_D \cot \theta})}
\]

and the structure performance function is based on the Van der Meer equation:

\[
D_n = 0.7 \Delta^{1/3} S_d^{0.15}
\]

2.3.1. Reliability. In the evaluation of structural engineering systems safety, one must evaluate the capabilities of the structural systems designed to respond to project requirements. Structural systems can fail in carrying out their functions due to several factors. Failure can occur due to the failure of the main structure. To analyze the risk of system failure, one must clearly identify the inputs to the system and their consequent responses.

A structural safety depends on the maximum load that can be imposed during the life of the structure, and also on its strength or components. Since the prediction of the maximum load and the true strength of a structure is based on uncertainty, one cannot guarantee absolute safety, and engineers must rely on several probabilistic concepts to demonstrate the probability that sufficient available force will withstand the maximum load over the life of the structure.

2.3.2. Monte Carlo Simulation. One of the methods for reliability analysis in engineering and economics is the Monte Carlo simulation. The main characteristic in this simulation is that by inputting the random
value of each random variable the system is simulated according to its probability distribution. This method is generally applied to systems that contain random variables or parameters [8].

Random Number Generator or random number generation is the basic principle in this method. This simulation is carried out by taking samples of random variables based on their probability distribution which are then used as input in the performance function (X). If the performance function value (X) < 0 and the sample is N, then the system under review is considered a crow n times [8]. Therefore, the probability of failure (P_f) is the ratio between the number of failures and the number of samples. This simulation is carried out until the failure / success probability graph shows convergence.

\[ P_f = \frac{n}{N} \]  

with n is number of failed occurrences, N is number of iterations and Pf is the chance of failure. Furthermore, the reliability (K) can be calculated by the following expression:

\[ K = 1 - P_f \]  

3. Results and Discussion

The analysis is focused on the part of breakwater at points A, B and C in figure 1. Initially the armour unit were Dolos. Currently, the existing breakwater has a quite severe damage at point B and the Dolos are replaced by BPPT Loc. There is no information on the weight of installed Dolos in the location. However, the dimensional data for the designed breakwater structure can be seen in Table 1 [9]:

| Parameter                        | Value  | Unit |
|----------------------------------|--------|------|
| Top Elevation (Elv.)             | 8,564  | m    |
| Crest Width (B)                  | 16.2   | m    |
| Main Guard Dolos Weight (W_1)    | 12     | ton  |
| Weight of Second Layer Stone (W_{10}) | 1.2 | ton  |
| Core Stone Weight (W_{200})      | 0.06   | ton  |
| Main Protection Layer Thickness (t_1) | 4,163 | m    |
| Thickness of Second Armour Layer (t_2) | 1.4 | m    |

In this study, several environmental data, namely wave data, tidal data and bathymetry data were used for analysis. The wave data was taken from ECMWF at coordinate 111.375 E and 8.375 S. The parameters obtained from the ECMWF are significant wave height, wave direction and significant wave period from September 1999 to August 2019.

Bathymetry of the study area is obtained from the reports of the Study on Strengthening the Permanent Jetty and Temporary Jetty Breakwater of PT. PJB Pacitan” [10] and also from the map obtained from Indonesian Navy Hydrographic Center.

The Weibull Distribution were employed to calculate the wave height for each return period of 1 year, 20 years, 30 years, 60 years and 100 years as can be seen in Table 2. To calculate the wave period, the following expression we applied [5]:

\[ T = 3.54H_r^{0.6} \]

| Return Period | Hs (m) | Ts (s) |
|---------------|--------|--------|
| Tr (Year)     |        |        |
| 1             | 2.54   | 6.26   |
| 20            | 3.92   | 8.14   |
The Delft3D [11] software were used to model the wave transformation from deep water to the location of breakwater. The 60-years return period of waves from three directions (Southwest, South and Southeast) will be used as the input of the Delft3D model. The input parameters in the Delft 3D model listed in Table 3.

### Table 3. Parameters input in Delft 3D wave model.

| Input parameter | Hs (m) | Ts (s) |
|-----------------|--------|--------|
| Hs (m)          | 4,42   |        |
| Ts (s)          | 8,77   |        |
| Maximum iteration | 1000   |        |

Figure 2 shows the results of Delft3D wave model for three different wave directions. The gradation of color shows the height of the wave: yellow to red shows high waves, while green to blue shows the small waves.

**Figure 2.** Significant wave height from (a) Southeast, (b) South, and (c) Southwest.

Based on the wave models above, the incoming wave from South East affect the location A, B and C on the breakwater significantly. The highest wave occurs at the point A (4,86m) when the wave coming from the Southwest. The following Table 4 shows the results of the wave model using Delft3D.

### Table 4. Wave height at points A, B, and C, from three incoming wave direction.

| Incoming wave direction | Hs (m) |
|-------------------------|--------|
|                         | A      | B      | C      |
| Southeast               | 2,14   | 1,52   | 1,48   |
| South                   | 4,19   | 3,33   | 3,29   |
| Southwest               | 4,86   | 3,57   | 3,76   |
The weight of breakwater armour unit and layers were estimated based on the wave height that occurs on the breakwater, as shown in Table 4 above. The results of the weight of breakwater armour unit based on the highest incoming waves (4.86m) were listed in Table 5 below. The specific density of concrete and water were assumed as 2.4 ton/m$^3$ and 1.025 ton/m$^3$. The slope of breakwater were 1: 1.5 and the weight of primary armour unit, secondary armour unit and breakwater core listed in column W, $W_{10}$ and $W_{200}$. The cross-section of the breakwater for various armour unit design are shown in Figure 3.

Table 5. The Weight of Armour Units.

| No. | Armour Unit | $H$ (m) | $K_D$ | $\gamma_a$ (ton.m$^{-3}$) | $\gamma_w$ (ton.m$^{-3}$) | $S_r$ | $\cot \theta$ | W (ton) | $W_{10}$ (ton) | $W_{200}$ (ton) |
|-----|-------------|---------|-------|--------------------------|--------------------------|------|--------------|---------|----------------|----------------|
| 1   | Dolos      | 4.857   | 15    | 2.4                      | 1.025                    | 2.341| 1.5          | 5.06    | 0.51           | 0.03           |
| 2   | BPPT Loc   | 4.857   | 17    | 2.4                      | 1.025                    | 2.341| 1.5          | 4.47    | 0.45           | 0.02           |
| 3   | Tetrapod   | 4.857   | 8     | 2.4                      | 1.025                    | 2.341| 1.5          | 9.49    | 0.95           | 0.05           |

Figure 3. (a) Author proposed Dolos Design, (b) Planning Dolos Design, (c) Author's BPPT Loc Design, and (d) Author's Tetrapod Design.

To simplify the simulation of reliability analysis, the process of using the software must be simplified by creating a transformation function. The transformation function is developed using Response Surface Method in the theory of the Design of Experiment [12]. Observations were made at three points (A, B, and C) and with 3 angles of waves incidence, namely Southwest (225°), South (180°) and Southeast (135°). With Minitab, 9 transformation equations for each condition were obtained. There are two input variables, namely the significant wave height ($H_s$) of the deep sea and the significant wave period ($T_s$), for that the suitable method is the First Order Response Surface Model for two variables. The response output from the wave load variable with the coming Southwest direction (225°) can be seen in Table 6. The result for South and South east direction were not presented due to limited space in this paper. Furthermore, a regression equation can be obtained for the three-observation points Ha, Hb, Hc.

Table 6. Response Surface Model for the direction of the incoming wave Southwest (225°).

| Exp | Variable | Hs(m) | Ts(s) | Ha | Hb | Hc | Ha | Hb | Hc | Ha | Hb | Hc | Ha | Hb | Hc | Error (%) |
|-----|----------|-------|-------|----|----|----|----|----|----|----|----|----|----|----|    |          |
| 1   | x1       | 3.8   | 12.2  | 4.41| 3.51| 3.79| 4.41| 3.51| 3.79| 0.003| 0.004| 0.004|    |    |    |          |
| 2   | x2       | 3.8   | 12.2  | 4.18| 3.19| 3.34| 4.18| 3.19| 3.34| 0.003| 0.004| 0.004|    |    |    |          |
| 3   | x1       | 1.3   | 12.2  | 1.52| 1.16| 1.27| 1.52| 1.16| 1.27| 0.001| 0.003| 0.006|    |    |    |          |
| 4   | x2       | 1.3   | 12.2  | 1.18| 0.97| 1.01| 1.18| 0.97| 1.01| 0.001| 0.005| 0.005|    |    |    |          |
| 5   | x1       | 2.25  | 9.58  | 2.36| 1.18| 1.99| 2.36| 1.88| 1.99| 0.003| 0.004| 0.004|    |    |    |          |
| 6   | x2       | 2.25  | 9.58  | 2.36| 1.18| 1.99| 2.36| 1.88| 1.99| 0.003| 0.004| 0.004|    |    |    |          |
In Table 6 above, nine running wave transformations were performed, with different variables, the results of different responses are also obtained.

| Exp | Variable | Basic Variable | Delft3D Response (m) | Transformation Function (m) | Error (%) |
|-----|----------|----------------|----------------------|-----------------------------|-----------|
|     |          |                | Hs(m) | Ts(s) | Ha | Hb | Hc | Ha | Hb | Hc | Ha | Hb | Hc | Ha | Hb | Hc | Ha | Hb | Hc | Ha | Hb | Hc |
| 6   | 0        | 0              | 2.25  | 9.58  | 2.36 | 1.18 | 1.99 | 2.36 | 1.88 | 1.99 | 0.003 | 0.004 | 0.004 |
| 7   | 0        | 0              | 2.25  | 9.58  | 2.36 | 1.18 | 1.99 | 2.36 | 1.88 | 1.99 | 0.003 | 0.004 | 0.004 |
| 8   | 0        | 0              | 2.25  | 9.58  | 2.36 | 1.18 | 1.99 | 2.36 | 1.88 | 1.99 | 0.003 | 0.004 | 0.004 |
| 9   | 0        | 0              | 2.25  | 9.58  | 2.36 | 1.18 | 1.99 | 2.36 | 1.88 | 1.99 | 0.003 | 0.004 | 0.004 |

Figure 4 above shows a contour shape of the model where Hs and Ts are input variables and Ha, Hb, and Hc are the outputs of the three equations. These three equations is presented in equations (8), (9), and (10) as follows:

\[
\begin{align*}
    Ha &= 0.07400 \ Hs^2 - 0.008389 \ HsTs + 0.8810 \ Hs + 0.07573 \ Ts - 0.5412 \\
    Hb &= 0.03612 \ Hs^2 + 0.009914 \ HsTs + 0.6348 \ Hs + 0.02334 \ Ts - 0.1683 \\
    Hc &= 0.04845 \ Hs^2 + 0.014490 \ HsTs + 0.5841 \ Hs + 0.03073 \ Ts - 0.1760
\end{align*}
\]

Using Microsoft Excel, a simulation is performed using random numbers as a large combination of load variables in the design. The result of the transformation equation will be a significant wave height at three observation points. The wave height which is taken as the design wave height is the highest wave height of the three observation points. The simulation results of the design wave height are then entered into the performance function in equation (4) and (5) to obtain the \( Dn \)-magnitude. If the system fails, it will be marked with "0", otherwise if the system is success, it will be marked with "1". Iteration is performed in several times to get a stable probability of success. Table 7 shows the failure limit of various armour unit based on Hudson Formula.

| Table 7. Failure limits of various type of armour stone (based on Hudson formula). |
|----------------------------------|
| Armour Units | Dn (m) |
|----------------|--------|
| Existing Dolos | 1,71  |
| Author proposed Dolos | 1,28  |
| BPPT Loc | 1,23 |
| Tetrapod | 1,58 |

The reliability value of hydraulic stability from the design of the breakwater with the author proposed Dolos protection stone is 94.00% for the Hudson equation, 79.28% for the Van Der Meer equation with minor damage, and 94.44% for the heavy damage. After 12000 simulations, the probability of success of the system will be checked for stability by using a graph comparing the probability of success with the number of iterations as shown in typically in figure 5 above. A complete result of the reliability
analysis for each armour unit listed in Table 8 below

![Graph](image)

**Figure 5.** Convergence graph of the probability of success for the author proposed Dolos armour unit.

**Table 8.** Reliability of concrete armour unit at Sudimoro Power Plant.

| Armour Units         | Hudson | VdM minor damage | VdM heavy damage |
|----------------------|--------|------------------|------------------|
| Designed Dolos       | 98.8%  | 95.12%           | 98.88%           |
| Author proposed Dolos| 94%    | 79.28%           | 94.44%           |
| BPPT Loc             | 94.03% | 79.11%           | 94.54%           |
| Tetrapod             | 94.1%  | 78.94%           | 94.73%           |

It is seen from Table 8 above, that the reliability analysis of breakwater armour unit using Van der Meer formula for heavy damage analysis have higher reliability compared to Hudson formula, while Hudson formula has higher reliability than Van der Meer formula for minor damage. The lower the reliability percentage result in greater degree of rigidity in the armour unit design.

4. Conclusion

Based on the Monte Carlo simulation for Reliability Hydraulic Stability of Rubble Mound Breakwater armour unit at Sudimoro Power Plant, Pacitan, the following conclusions can be drawn:

1. The design of the Dolos’ weight for breakwater armour unit at Sudimoro Power Plant was 12 tons. The design weight of the Dolos has met the design criteria for hydraulic stability with a 237% better than the author proposed Dolos design, which only need 5.06 tons based on 4,857m incoming wave height. There is no information on the weight of installed Dolos in the location.

2. The reliability of existing breakwater armour unit in Sudimoro Power Plant has been presented. Based on the comparison of reliability results, the Van der Meer formula with minor damage has a greater degree of design rigidity than the Hudson formula.

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