Reliability analysis for determining performance of barrage based on gates operation

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Abstract. Some rivers located on a flat slope topography such as Cilemahabang river and Ciherang river in Cilemahabang watershed, Bekasi regency, West Java are susceptible to flooding. The inundation mostly happens near a barrage in the middle and downstream of the Cilemahabang watershed, namely the Cilemahabang and Caringin barrages. Barrages or gated weirs are difficult to exploit since the gate must be kept and operated properly under any circumstances. Therefore, a reliability analysis of the gates operation is necessary to determine the performance of the barrage with respect to the number of gates opened and the gates opening heights. The First Order Second Moment (FOSM) method was used to determine the performance by the reliability index (β) and the probability of failure (risk). It was found that for Cilemahabang Barrage, the number of gates opened with load (L) represents the peak discharge derived from various rainfall (P) respectively one gate with opening height (h=1m) for Preal, two gates (h=1m and h=1,5m) for P50, and three gates (each gate with h=2,5m) for P100. For Caringin Barrage, the results are minimum three gates opened (each gate with h=2,5 m) for Preal, five gates opened (each gate with h=2,5m) for P50, and six gates opened (each gate with h=2,5m) for P100. It can be concluded that a greater load (L) needs greater resistance (R) to counterbalance. Resistance can be added by increasing the number of gates opened and the gate opening height. A higher number of gates opened will lead to the decrease of water level in the upstream of barrage and less risk of overflow.

1. Introduction
Cilemahabang watershed covers two districts in West Java. The headwater is located in Cimahabang (Bogor regency) and flows northward into Ciherang river (Bekasi regency). Cilemahabang barrage is located in the segment of Cilemahabang river and across Cikarang Baru which is an important area in Bekasi. On the other hand, Caringin barrage is located in the downstream in segment of Ciherang river.

A barrage is kind of a weir with a low head consisting of several gates that can be opened or closed to control the amount of water passing through the structure. This is often built in the river on a flat terrain where the flow is tends to be slow and potentially overflow when heavy rainfall occurs. Similiar to a weir, the function of a barrage is to divert the flow for use in irrigation and other systems.

Floods in commercial areas, for example in Cilemahabang watershed, which has inundated roads, housing, and hundreds of factories in Cikarang industrial area, have detrimental effects. The most severe flood, which reached up to 1 meter high, was in Cikarang Baru, upstream of Cilemahabang barrage in
the middle of Cilemahabang watershed. Another location was in the downstream of Cilemahabang watershed, near Caringin barrage along the Ciherang river. Because the floodwater inundates several areas near the barrage, the operation of both barrages needs to be evaluated.

This study was undertaken to assess reliability of existing barrages and determine its performance by the reliability index (β) and the probability of failure (risk). Among the existing probabilistic methods, the First Order Second Moment (FOSM) was chosen with corresponding to design flood with various return periods. The reliability analysis will focussed on operational of barrages with respect to the number of gates opened the gates opening heights. At last, the result from this study can become a suggestion for local government (Head of Perum Jasa Tirta II Water Management Division I section Lemahabang) to update the Standard Operating Procedure (SOP) of those two barrages, Cilemahabang and Caringin barrages.

2. Literature Review
Analysis of uncertainties and risks in the hydraulic structures have been developed in recent decades. Reliability of hydraulic design in open channel has also been studied, e.g. the reliability analysis of composite channels was carried out using Advance First Order Second Moment (AFOSM) formulation and Monte Carlo Simulation (MCS) method. Both methods consider the channel design and run off parameters as uncertain variables [1]. The channel design parameters, including friction factors, longitudinal slope, channel width, side slope, and flow depth, while rainfall intensity and drainage as the uncertainty of run off. The study performed a detailed sensitivity study on reliability index for different values of system loading in both static and dynamic environment. It can be concluded from sensitivity analysis that flow depth and bed width were the most influencing parameters that affect the safety factor and reliability.

Another study, an assessment of storm water drain network located in northern part of Bangalore, India has done by considering three failure modes using AFOSM method [2]. There was a need to take into account the uncertainties due to natural and/or inherent randomness for designing a storm water drain network that can effectively convey the discharge. Three parameters were considered to be random variables, namely design flow, conduit dimensions, and roughness of lining material. As a result from reliability analysis, indicate a need to redesign several location of the conduits by the changing of conduit width, slope, or the roughness of conduit lining material.

Failures of hydrosystem infrastructure, as an example in two previous studies, open channel and storm drain sewer, could potentially pose significant threats to public safety and cause enormous damage on properties and the environment. In this study that a barrages or gated weir become an object of study, reliability analysis will be done for determining operation procedure or management strategy. The operation procedure of gates should be checked to minimize the floods and risk of overflow.

2.1. Concepts of Reliability and Risk
With reference to American dictionaries defines risk as the probability of failure to achieve the intended goal. Reliability is defined mathematically as the complement of the risk [3]. Failure of an engineering system can be defined as a situation in which the load L (external forces or demands) on the system exceeds the resistance R (strength, capacity, or supply) of the system. Meanwhile, the reliability is defined as the probability of nonfailure (survive) in which the resistance of the system exceeds the load. In hydrosystem engineering designs, uncertainties associated with the resistance of the hydraulic flow-carrying capacity and the randomness of events, such floods and rainstorms as the load. The mathematical definitions of reliability and failure probability can be seen as,

$$P_{\text{survive}} = P(L \leq R) \quad \text{and} \quad P_{\text{failure}} = P(L \geq R)$$

(1)

In which the resistance R is the designated value of resistance, a deterministic quantity. By considering inherent randomness of annual maximum floods, the annual failure probability for a hydraulic structure designed with a capacity to accommodate a T-year flood, where T-year refers to various return periods.
2.2. First Order Second Moment (FOSM) method
The First Order Second Moment (FOSM) method simplify the functional relationship between independent variables and dependent variable by a truncated Taylor series expansion. The inputs and outputs of these methods are expressed as expected values and standard deviations [1]. This approaches account for uncertainty or randomness in the design parameters by constructing their probability density functions (PDFs). However, such PDFs cannot be derived in most practical applications, owing to complexity of the system and/or data limitations. Hence hydrologists resort to approximate approaches to express the uncertainty based on confidence interval or statistical moments [2].

One of the best-known probability density functions is that forming the familiar bell-shaped curved for the normal distribution, which can be expressed as,

$$f(x) = \frac{1}{\sqrt{2\pi \sigma}} \exp \left( -\frac{(x-\mu)^2}{2\sigma^2} \right)$$  \hspace{1cm} (2)

Where \(x\) = random variable, \(\mu\) = mean and \(\sigma\) = standard deviation. If loads and resistances, as well as the parameters determining loads and resistances, are normally distributed, or if their distributions can be approximated by or transformed into normal density functions, the probability of failure of the structure can also be determined from the normal distribution. And if \(Z (R-L)\) is a random variable also, the probability of failure can be illustrated as a shaded area \(P(Z<0)\).

![Figure 1. The PDFs normal distribution of resistance/strength (R), load (L), and Z (R-L)](image)

3. Methodology

3.1. Hydrology Analysis using Software HEC-HMS 3.5
This analysis was done to determine the design flood for any return periods (50 years, \(P_{50}\) and 100 years, \(P_{100}\)) and from extreme rainfall (\(P_{real}\)) during the time period 1993-2009. HEC-HMS uses a separate model to represent each component of the runoff process, including models that compute runoff volume/losses, models of direct runoff, models of baseflow, and models of channel flow/routing which method for each model is tabulated below.
Table 1. The method was used in HEC-HMS modelling [4]

| No. | Component      | Method            | Categorization                                                                 |
|-----|----------------|-------------------|--------------------------------------------------------------------------------|
| 1.  | Precipitation  | User hyetograph   | The response of a watershed is driven by precipitation. It may be observed rainfall from a historical event, a frequency-based hypothetical rainfall event, or an event that represents the upper limit of precipitation. |
| 2.  | Runoff volume  | Initial and constant-rate | Address amount about the volume of precipitation that falls on watershed including infiltration on pervious surface and runoff of the impervious surfaces. |
| 3.  | Direct runoff  | Synthetic Unit Hydrograph: Clark’s UH and Snyder’s UH | Describe what happens as water that has not infiltrated or been stored on the watershed moves over or just beneath the watershed surface. |
| 4.  | Baseflow       | Exponential recession | Simulate the slow subsurface drainage of water from the system into the channels. |
| 5.  | Routing        | Muskingum         | Modelling one-dimensional open channel flow. |

3.2. Hydraulic Analysis using Software HEC-RAS 4.1.0

The objective of this step is to determine the water surface profiles in the river near the Cilemahabang and Caringin barrages and to calculate the discharge through the gates for various combinations of the gates opened. As the river geometry models, three major tributaries are considered, namely Kali Rasmi, Kali Buntu, and Ci Pegadungan. They will all flow into the main Ciherang river as depicted in Figure 1.

Figure 2. (a) The layout of Cilemahabang river channel networks (main channel and tributaries); (b) The barrage with low broad crested and several gates which set between flanking piers; (c) example sluice gate with broad crested spillway

Gated weir (barrage) within HEC-RAS was modeled as vertical lift gates (sluices gates). The equations used to model the gates openings that can handle both submerged and unsubmerged conditions at the inlet and outlet of the gates. The spillway crest under the gate was specified as broad crested spillway. The flow through the gate is considered to be “free flow” when the downstream tailwater elevation \(Z_D\) is not high enough to cause an increase in the upstream headwater elevation for a given flow rate, the equation is as follows [5]:

\[
Z_C = \frac{Z_D}{2} + \frac{Q^2}{8gB^2} - \frac{Q^2}{8gB^2} \tan \theta
\]
\[ Q = CW R \sqrt{2gH} \]  

Where:  
- \( H \) = upstream energy head above the spillway crest \((Z_U - Z_{sp})\)  
- \( C \) = coefficient of discharge, typically 0.5 to 0.7  
- \( W \) = weir coefficient

When the downstream tailwater increases to the point at which the gate is no longer flowing freely (downstream submergence is causing a greater upstream headwater for a given flow), the equation becomes:

\[ Q = CW R \sqrt{2g3H} \]  

Where:  
- \( H \) = \( Z_U - Z_D \)

Submergence begins to occur when the tailwater depth above the spillway divided by the headwater energy above the spillway, is greater than 0.67. Equation (4) is used to transition between free flow and fully submerged flow. This transition is set up so the program will gradually change to the fully submerged orifice equation below when the gates reach a submergence of 0.80.

\[ Q = CA \sqrt{2gH} \]  

Where:  
- \( A \) = area of the gate opening  
- \( H \) = \( Z_U - Z_D \)  
- \( C \) = discharge coefficient (typically 0.8)

When water is flowing through the top of gate which the gate is closed and at low flow condition (overflow happens through the weir crest under the gate), the flow generally modeled with the standard weir equation:

\[ Q = C L H^{3/2} \]  

Where:  
- \( C \) = Weir flow coefficient, typically 2.6 – 4.0  
- \( L \) = length of the spillway crest  
- \( H \) = upstream energy head above the spillway crest

### 3.3. Reliability Analysis

This analysis was carried out to determine its performance based on gates operation. Equations below were used to calculate the reliability index (\( \beta \)) and the probability of failure or risk (Tung et.al., 2006).

Reliability index:

\[ \beta = \frac{\mu_x - \mu_r}{\sigma_x} \]  

\[ = \frac{\mu_x - \mu_r}{\sqrt{\sigma_{R}^2 + \sigma_{L}^2}} \]  

Probability of failure: \( P\text{(failure)} = 1 - \Phi(\beta) \)

Probability of survive: \( P\text{(survive)} = \Phi(\beta) \)

Where \( 1 - \Phi(\beta) \) is risk and \( \Phi(\beta) \) is reliability with its values are tabulated in PDFs normal distribution tables, and these values may be approximated by the following polynomial [6].

\[ B = \frac{1}{2} \left[ 1 + 0.196854|\beta| + 0.115194|\beta|^2 + 0.000344|\beta|^3 + 0.019527|\beta|^4 \right]^4 \]

Where \(|\beta|\) is the absolute value of reliability index \( \beta \), and the standard normal distribution has \( \Phi(\beta) = B \) for \( \beta < 0 \) and \( \Phi(\beta) = 1 - B \) for \( \beta \geq 0 \).
The error in $\Phi(\beta)$ as evaluated by this formula is less than 0.00025.

4. Results
The design flood was calculated as the Load (L) and the discharge both free flowing and submerged weir flow through the gate openings as the Resistance (R). To avoid a risk of overflow, the water elevation upstream should be kept below the surrounding ground elevation (+11,126 m for Cilemahabang barrage and +5,200 m for Caringin barrage). The result about water elevation upstream of the barrage considering several combinations of the gates operation is summarized in Table 2.

Table 2. The water elevation upstream with several combinations of the gates opened based on various rainfalls Preal, P50, and P100 (CLB = Cilemahabang barrage with 7 gates; CRB = Caringin barrage with 8 gates)

| number of gates (pc) | opening height (m) | P_{real} | P_{50} | P_{100} |
|---------------------|-------------------|---------|-------|--------|
|                     |                   | CLB     | CLB   | CLB    |
| 0                   | -                 | 11,13   | 11,21 | 11,18  |
| 1                   | 1 m               | 11,08   | 11,07 | 11,17  |
| 2                   | 1 m; 2,5 m        | 11,08   | 10,12 | 10,96  |
| 3                   | #gate 2,5 m       | 9,69    | 9,39  | 9,59   |
| 4                   | #gate 2,5 m       | 9,13    | 8,85  | 8,48   |
| 5                   | #gate 2,5 m       | -       | 3,46  | -      |
| 6                   | #gate 2,5 m       | -       | 3,46  | -      |
| 7                   | #gate 2,5 m       | -       | 3,46  | -      |
| 8                   | #gate 2,5 m       | -       | 3,46  | -      |

Table 2 shows a decrease of water elevation as the effect of increasing number of gates opened. The operating procedure of gates is opened gradually start from no gate opened until all gates opened. Afterward, its response is monitored due to the change of water elevation upstream of barrage. The opening height is uniform for each opening (2,5 m), which is the maximum height opening.

Furthermore, the performance of barrage can be checked by defining the reliability index and the probability of failure for Cilemahabang and Caringin barrages. Table 3 shows the results of reliability analysis due to gates operation with the design flood was derived from extreme rainfall ($P_{real}$). The conclusion of these calculation is a higher number of gates opened will lead to minimize risk of overflow.

Two graphics in Figure 3 and Figure 4 give information about relationship between number of gates opened and a risk (%). It compares also about a risk (%) and water elevation upstream on the same graphic.
Table 3. Results of reliability analysis based on gates operation

| number of gates (pc) | opening height (m) | Reliability analysis |  |  |  |  |  |  |
|----------------------|-------------------|----------------------|---|---|---|---|---|---|
|                      |                   | $\mu_x - \mu_z$      | $\sigma_x - \sigma_z$ | $\mu_x - \mu_z \sigma_z$ | $\Phi(\beta_x)$ (%) | $R_{30} - 1 - \Phi(\beta_x)$ (%) |
|                      |                   | CLB                 | CRB | CLB | CRB | CLB | CRB | CLB | CRB |
| 1                    | 2,5 m             | -56,9              | -107 | 17,77 | 29,40 | - | 3,20 | - | 3,63 | 0,09 | 0,02 | 99,91 | 99,98 |
| 2                    | #gate 2,5 m       | -36,9              | -70,1 | 17,83 | 29,34 | - | 2,07 | - | 2,39 | 1,90 | 0,85 | 98,10 | 99,15 |
| 3                    | #gate 2,5 m       | 0,003              | -35,4 | 24,46 | 30,36 | 0,00 | - | 1,17 | 50,01 | 12,18 | 49,99 | 87,82 |
| 4                    | #gate 2,5 m       | 4,99               | -18,9 | 22,56 | 29,91 | 0,22 | - | 0,63 | 58,75 | 26,39 | 41,25 | 73,61 |
| 5                    | #gate 2,5 m       | 10,4               | 4,16  | 21,29 | 30,48 | 0,49 | 0,14 | 68,72 | 55,41 | 31,28 | 44,59 |
| 6                    | #gate 2,5 m       | 20,55              | 31,83 | 19,98 | 32,54 | 1,03 | 0,98 | 84,79 | 83,58 | 15,21 | 16,42 |
| 7                    | #gate 2,5 m       | 66,97              | 59,91 | 18,22 | 33,28 | 3,68 | 1,53 | 99,98 | 93,71 | 0,022 | 6,29 |
| 8                    | #gate 2,5 m       | -70,02             | -     | 34,18 | -     | 2,05 | -     | 97,99 | -     | 2,01 |  

Figure 3. A Graphic relationships between number of gates opened and a risk (%); a risk (%) and water elevation upstream (m) for Cilemahabang barrage
Figure 4. A Graphic relationships between number of gates opened and a risk (%); a risk (%) and water elevation upstream (m) for Caringin barrage

The line graphs in both figures above illustrate changes in a risk of overflow and the water elevation upstream with the combination of different gates operation, which the value of a risk or the probability of failure is showed in Table 3. For both barrages, Cilemahabang and Caringin barrage respectively, a risk declined moderately as the number of gates opened got increased. There was an optimum gate should be opened when the graph of gates opened and a risk overtook the graph of a risk and water elevation upstream, namely three gates for Cilemahabang barrage and five gates for Caringin barrage. These number of opening gates are become the ideal operating procedure of barrage to maintain a safe condition without any inundation or overflow.

5. Conclusions
From the calculation and analysis, it can be concluded that: (1) For Cilemahabang barrage, the number of gates opened should be one gate with opening height (h=1m) for \( P_{real} \), two gates (h=1m; h=1,5m) for \( P_{50} \), and three gates (h=2,5m) for \( P_{100} \). (2) For Caringin barrage, there should at least be three gates opened (h=2,5 m) for \( P_{real} \), five gates (h=2,5m) for \( P_{50} \), and six gates (h=2,5m) for \( P_{100} \). (3) The reliability index will increase if more gates were opened. (4) The optimum number of gates should be opened are three gates and five gates for Cilemahabang and Caringin barrages respectively.

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