Safety evaluation of the existing Grawan dam based on hydrogeotechnical behaviour conditions to ensure the availability of water resources

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Abstract. Grawan Dam was built in 2004 to supply irrigation water and raw water. Lately, the dam has experienced various symptoms of damage to the overflow structure due to hydraulic behaviour and the possibility of overtopping of the dam, which endangers the stability of the dam. Therefore, it is necessary to evaluate the overflow structure system based on the current hydrological conditions and to review the slope stability. In this study an evaluation was carried out covering the safety of the slopes as well as the safety of the overflow building system. Dam slope safety was reviewed based on seepage and slope stability. The stability of the overflow structure security system was evaluated in normal and earthquake conditions. The results of the hydrological analysis on guard height, seepage that occurs safely to permit discharge, and slope stability showed 100% safety from permitted safety factors, and on hydraulic conditions resulted in proposed improvements for Vlugter ponds, transition channels, and energy dampers with planning for Q₁₀₀, Q₁₀₀₀, and Q₉₉₉. Analysis for the stability of the overflow structure showed that it is safe against rolling and sliding, and meets the eccentricity and carrying capacity of the soil, being 100% safe.

Keywords: Evaluation, Dam Safety, Hydro-Geotechnical

1. Introduction

Water resources are very important for humans because they are basic human needs. From all of the water on the surface of the earth, only 2.5% is in the form of fresh water. With this very limited amount, development efforts and good water management become necessary, as through the construction of reservoirs and dams to store water.

In addition to saving water, reservoirs also function to regulate water use in accordance with the planned needs so that later on the water will provide maximum results. In order for reservoirs and dams to be used according to the planned lifetime, continuous maintenance and observation become necessary [1].

In accordance with SNI No.1731-1989-F on Dam Safety Guidelines and Minister of Settlements and Regional Infrastructure Regulation No. 296/KPTS/M/2001, inspections must be carried out at least once every five years to determine the condition of dams, regarding their structural safety, hydraulics, and operations.

Grawan Dam is located in Rembang Regency in Central Java. It was built in 2004 to meet the irrigation water needs of the area (160 ha) and raw water needs in the sub-districts of Sumber and Sulang. From field monitoring results, it was seen that there were damages to the energy absorber structure and
floodway, and thus a study needed to be conducted on a thorough evaluation of the hydrological and
geotechnical conditions of the dam.

2. Materials and Methods

2.1. Study Location
The study location is Grawan Dam, which is geographically located at the coordinates 6°46’50.58” S and
111°17’24.72” E and administratively located in Grawan Village in the Sumber Sub-District of
Rembang Regency, Province of Central Java.

2.2. Data Collection
The data needed to conduct this study among others include the following:
1. Rainfall data from 1973-2017
2. Map of Indonesian earthquakes in 2017
3. Geological data of the foundation and body of Grawan Dam
4. Cross-section of Grawan Dam
5. Grawan Dam technical data

2.3. Steps of the Study
The steps of this study are arranged systematically to facilitate the completion of the analysis. The
following are the steps conducted in this study:
1. Data collection
2. Determining the regional rainfall
3. Testing the skewness of rainfall data
4. Frequency analysis testing
5. Goodness-of-fit testing
6. Analysis of planned flood discharge with the latest hydrological conditions
7. Flood routing
8. Analysis of seepage on the body of the dam using the latest geotechnical data
9. Analysis of dam body stability using the Fellenius and Bishop methods, with the latest soil
sampling data

Table 1. Soil Parameters for Slope Analysis on DB-1

| Type           | Unit weight | Permeability | Stiffness | Soil strength |
|----------------|-------------|--------------|-----------|---------------|
|                | $\gamma_d$ | $\gamma_t$ | $K_x$ (m/sec) | $K_y$ (m/sec) | $E_{ref}$ (KN/m$^2$) | $\nu$ (nu) | $C$ (KN/m$^2$) | $\phi$ (°) |
| Construction   | 13.9        | 18.6        | 5.26 x 10$^{-6}$ | 5.26 x 10$^{-6}$ | 11000 | 0.35 | 20 | 28 |
| Existing       | 12.4        | 17.5        | 4.6 x 10$^{-7}$ | 4.6 x 10$^{-7}$ | 11000 | 0.301 | 29 | 29 |
| Coarse gravel  | 23          | 24          | 3.6 x 10$^{-3}$ | 3.6 x 10$^{-3}$ | 13000 | 0.3 | 40 | 0 |
| Split Stone 2/3| 22          | 23          | 1            | 1            | 15000 | 0.3 | 50 | 0 |
| Split Stone 5/7| 21          | 22          | 1.7          | 1.7          | 15000 | 0.3 | 50 | 0 |
| Rock Toe       | 21          | 22          | 2            | 2            | 15000 | 0.3 | 50 | 0 |
| Rip-Rap        | 21          | 22          | 2            | 2            | 15000 | 0.3 | 50 | 0 |

2.4. Literature Review

2.4.1. Calculation of Regional Rainfall
Rainfall and discharge data are very important data in dam hydrological analysis. The aim of this
analysis is to obtain the amount of rainfall. The need to calculate regional rainfall is for the preparation
of water use planning and flood control planning. The method used in calculating the average rainfall in a watershed area is the arithmetic (algebraic) average method.

The Regional Rainfall Value is determined using the following formula:

\[ R = \frac{1}{n} (R_1 + R_2 + R_n) \]  

(1)

Where:
R = Regional rainfall (mm)
\( n \) = Number of observation points
\( R_1, R_2, R_n \) = Rainfall at each observation point (mm)

**Figure 1.** Location of drilling bore and core sampling

### 2.4.2. Planned Flood Discharge

The Planned Flood Discharge is the flood discharge used to plan the safety level of danger with the largest probability number. Analysis of the planned flood discharge can be performed using the hydrograph method, which is conducted with the aid of a synthetic unit hydrograph model, and using non-hydrographic methods with the aid of frequency analysis techniques. This study involved the comparison of the Nakayasu Synthetic Unit Hydrograph method, the Gamma I Synthetic Unit Hydrograph method [2], and HSS ITB Synthetic Unit Hydrograph method.

### 2.4.3. Flood Routing

To obtain the flood water level on the dam body, it is necessary to conduct flood routing to determine the outflow discharge of the flood storage in the reservoir. The width threshold type spillway was used with the following elevation and volume:

\[ Q = C_d \times B \times H^{3/2} \]  

(2)

Where:
\( C_d \) = Discharge coefficient
\( B \) = Diversion width (m)
\( H \) = Water Level above the Spillway (m)
2.4.4. Dam Slope Stability
In many cases, in building a dam, the stability of the retaining walls is expected to be calculated in order to check their safety, whether natural, cut, or filled [3].

2.4.4.1. Fellenius Method of Slices

\[ F = \frac{\sum_{n=1}^{n=p} (c \Delta L_n + W_n \cos \alpha_n \tan \phi)}{\sum_{n=1}^{n=p} W_n \sin \alpha_n} \]  

(3)

\( \Delta L_n \) in the equation above is equal to \( \frac{b_n}{\cos \alpha_n} \) with \( b_n \) = width of n-slice section.

2.4.4.2. Bishop Method of Slices
In 1995, Bishop introduced a more thorough solution than a simple method of slices. In this method, the effect of the forces on the edge of each slice is taken into account, thus:

\[ F_s = \frac{\sum_{n=1}^{n=p} (c b_n + W_n \tan \phi) \frac{1}{m_{\alpha}} (n)}{\sum_{n=1}^{n=p} W_n \sin \alpha_n} \]  

(4)

2.4.5. Stability against Seepage
Seepage through dam bodies, foundations, abutments, or hills around a reservoir must be controlled so that excessive uplift force does not occur.

The safety of soil fill-type dams can be calculated based on the following formula:

\[ F_s = \frac{l_c}{l_e} \geq 4 \]  

(5)

3. Results and Discussion
Based on guidelines for Earth-Fill Dam Stability Analysis Due to Earthquake Load (2004, p. 4), an earth-fill dam that holds water with a large enough volume must consider safety factors toward dam stability by considering the load through either the Maximum Design Earthquake (MDE) or Operating Basis Earthquake (OBE) requirements.

3.1. Hydrological Analysis of Grawan Dam

3.1.1. Regional Rainfall
Calculation of regional rainfall is needed for the preparation of a water planning and flood control planning.

3.1.2. Planned Flood Discharge
The Planned Flood Discharge is a flood discharge used to plan the safety level of danger with the largest probability value. Analysis of the planned flood discharge can be performed using the hydrograph method.
Table 2. Regional Rainfall Using the Algebraic Average Method

| Year | average rainfall (mm) |
|------|-----------------------|
| 1973 | 132.5                 |
| 1974 | 115.0                 |
| 1975 | 82.5                  |
| 1976 | 61.0                  |
| 1977 | 81.0                  |
| 1978 | 92.5                  |
| 1979 | 77.5                  |
| 1980 | 60.5                  |
| 1981 | 74.5                  |
| 1982 | 101.5                 |
| 1983 | 71.5                  |
| 1984 | 71.5                  |
| 1985 | 85.0                  |
| 1986 | 116.5                 |
| 1987 | 137.5                 |

Table 3. Summary of Planned Discharge for Basic Data from 1973-2017

| Return Period | Planned Flood (m³/s) |
|---------------|----------------------|
|               | Q inflow maximum     |
|               | Q outflow maximum    |
|               | H maximum above the spillway threshold |
|               | Flood water level elevation |
| 1.01          | 19.417               |
| 2             | 29.476               |
| 5             | 30.281               |
| 10            | 38.085               |
| 25            | 42.019               |
| 50            | 44.365               |
| 100           | 47.214               |
| 200           | 50.501               |
| 1000          | 56.698               |
| PMF           | 107.303              |
| Gamma I       | 18.556               |
| 2             | 28.218               |
| 5             | 29.703               |
| 10            | 37.017               |
| 25            | 41.337               |
| 50            | 44.109               |
| 100           | 47.198               |
| 200           | 50.785               |
| 1000          | 57.662               |
| PMF           | 110.823              |
| ITB 1         | 20.747               |
| 2             | 31.524               |
| 5             | 33.036               |
| 10            | 41.013               |
| 25            | 45.719               |
| 50            | 49.567               |
| 100           | 53.066               |
| 200           | 60.022               |
| 1000          | 118.34               |

Based on Table 3, it can be seen that the highest planned discharge was obtained from the ITB-1 synthetic unit hydrograph, where the planned discharge of $Q_{PMF}$ is 118.34 m³/s; therefore, further calculations used the values from the ITB-1 synthetic unit hydrograph.

3.1.3. Flood Routing
Flood routing through the spillway is closely related to determining the height of the dam. Based on the calculation of flood routing, the following are the obtained maximum outflows due to planned flood discharge:

Table 4. Summary of Results of Flood Routing, 1973-2017

| Return Period T (year) | Q inflow maximum (m³/sec) | Q outflow maximum (m³/sec) | H maximum above the spillway threshold (m) | Flood water level elevation (m) |
|------------------------|---------------------------|----------------------------|--------------------------------------------|-------------------------------|
| 1000                   | 62.661                    | 58.963                     | 1.511                                      | 40.511                        |
| PMF                    | 118.34                    | 112.835                    | 2.249                                      | 41.249                        |

Source: Calculation Results, 2019

Based on Table 4, it is known that from the results of flood routing, the water level in the dam is still safe toward the free board being that the maximum water level above the overflow/spillway is 2.249 m at elevation +41.249.
3.2. Seepage Analysis of the Grawan Dam
Both the body and foundation of the dam are required to be able to withstand the forces caused by the presence of filtration water flowing through the gaps between the soil grains of the embankment and the foundation.

![Figure 2: Graph of Flood Routing Hydrograph for the Spillway](source)

Source: Analysis Results, 2019

![Figure 3: (a) Observation of Piezometer Water Level at NWL; (b) theoretical model of phreatic seep/w at FWL](source)

### Table 5. Summary of Discharge Calculation

| Water level | Allowable Q Seepage (m³/s) | Initial Construction (m³/s) | Current Conditions (m³/s) | V-Notch (m³/s) | Q Seepage < Allowable |
|-------------|-----------------------------|-----------------------------|---------------------------|---------------|-----------------------|
| LWL         |                             | 2.21E-04                    | 5.08E-05                  | -             | Safe                  |
| NWL         | 0.1121                      |                             |                           |               |                       |
| FWL (Q1000) | 6.74E-03                    | 1.72E-03                    | Unreadable                | Safe          |
| FWL (QPMF)  | 1.07E-02                    | 2.53E-03                    | Unreadable                | Safe          |

Source: Analysis Results, 2019

Based on Table 5, it can be seen that the seepage discharge at the beginning of construction, during piezometer monitoring, and in current occurring conditions are still safe from the allowable seepage discharge, with the highest seepage discharge being 1.07 x 10² m³/s.

3.3. Stability against Piping
The following are the results of calculations for the safety factors from piping on the body of Grawan Dam:

\[
G_s = 2.625; \ c = 1.07
\]

\[
I_c = 0.138 \text{ (from the piezometer reading)}
\]

\[
I_c = \frac{2.625 - 1}{1.07} = 0.785
\]
Fs = 0.785
Fs = 0.138
Fs = 5.695 > 4 (safe)

3.4. Analysis of Grawan Dam Stability

The following are the results of the stability analysis for Grawan Dam in the latest conditions:

Figure 4. (a) Stability of downstream slope -- Flood Water Level (FWL) Q_{1000} — with OBE earthquake on Bishop Model (y/h = 0.25)
(b) Stability of upstream slope -- Flood Water Level (FWL) Q_{1000} — with OBE earthquake on Bishop Model (y/h = 1). (Source: Analysis Results of slope/w Model)

Table 6. Comparison of Grawan Dam Stability Safety Factors with OBE Conditions in the Upstream and Downstream Parts Using the Fellenius and Bishop Methods

| Condition | FS_{min} | y/h | Upstream Safety Factor | Downstream Safety Factor |
|-----------|----------|-----|------------------------|--------------------------|
| Steady Flow, MWL, El. +35.00, no earthquake | 1.5 | - | 3.31 | 3.432 |
| Steady Flow NWL, El. +39.00, no earthquake | 1.5 | - | 3.768 | 4.283 |
| Steady Flow FWL, Q_{1000} El. +40.511, no Earthquake | 1.5 | - | 4.477 | 5.036 |
| Steady Flow, FWL, Q_{1000} El. +41.249, No Earthquake | 1.5 | - | 4.977 | 5.022 |
| Rupture Drawdown, from NWL to MWL, no earthquake | 1.2 | - | 3.276 | 3.417 |

| Condition | FS_{min} | y/h | Upstream Safety Factor |
|-----------|----------|-----|------------------------|
| Steady Flow, MWL, El. +35.00, with Earthquake OBE | 1.2 | 0.25 | 3.197 | 3.314 |
| | | 0.50 | 2.867 | 2.972 |
| | | 0.75 | 2.684 | 2.782 |
| | | 1.00 | 2.532 | 2.625 |
| Steady Flow, NWL, El. +39.00, with earthquake OBE | 1.2 | 0.25 | 3.582 | 3.314 |
| | | 0.50 | 3.076 | 2.972 |
| | | 0.75 | 2.808 | 2.782 |
| | | 1.00 | 2.600 | 2.625 |
| Steady Flow FWL, Q_{1000} El. +40.511, with Earthquake OBE | 1.2 | 0.25 | 4.203 | 4.718 |
| | | 0.50 | 3.486 | 3.885 |
| | | 0.75 | 3.128 | 3.413 |
| | | 1.00 | 2.854 | 3.099 |
| Steady Flow FWL, Q_{1000} El. +41.249, with Earthquake OBE | 1.2 | 0.25 | 4.027 | 4.733 |
| | | 0.50 | 3.749 | 3.799 |
| | | 0.75 | 3.334 | 3.415 |
| | | 1.00 | 3.021 | 3.113 |
| operating, rapid drawdown (RDD) NWL, to MWL, with Earthquake OBE | - | 0.25 | 3.162 | 3.299 |
| | | 0.50 | 2.831 | 2.956 |
| | | 0.75 | 2.646 | 2.755 |
| | | 1.00 | 2.492 | 2.596 |

| Condition | FS_{min} | y/h | Downstream Safety Factor |
|-----------|----------|-----|--------------------------|
| Steady Flow, MWL, El. +35.00, no earthquake | 1.5 | - | 2.474 | 2.561 |
| Steady Flow NWL, El. +39.00, no earthquake | 1.5 | - | 2.474 | 2.561 |
| Steady Flow FWL, Q_{1000} El. +40.511, no Earthquake | 1.5 | - | 2.474 | 2.561 |
| Steady Flow, FWL, Q_{1000} El. +41.249, No Earthquake | 1.5 | - | 2.474 | 2.561 |
| Rupture Drawdown, Form NWL to MWL, no earthquake | 1.2 | - | 2.474 | 2.561 |

| Condition | FS_{min} | y/h | Downstream Safety Factor |
|-----------|----------|-----|--------------------------|
| Steady Flow, MWL, El. +35.00, with Earthquake OBE | 1.2 | 0.25 | 2.413 | 2.498 |
| | | 0.50 | 2.230 | 2.972 |
| | | 0.75 | 2.123 | 2.782 |
| | | 1.00 | 2.032 | 2.625 |
| Steady Flow, NWL, El. +39.00, with earthquake OBE | 1.2 | 0.25 | 2.413 | 3.314 |
| | | 0.50 | 2.230 | 2.972 |
| | | 0.75 | 2.123 | 2.782 |
| | | 1.00 | 2.032 | 2.625 |
| | | 1.00 | 2.032 | 2.625 |
| Aliran langeng (Steady Flow) muka air banjir Q_{1000} El. +40.511, ada gempa OBE | 1.2 | 0.25 | 2.413 | 4.718 |
| | | 0.50 | 2.230 | 3.885 |
| | | 0.75 | 2.123 | 3.413 |
| | | 1.00 | 2.032 | 3.099 |
| Steady Flow FWL, Q_{1000} El. +40.511, with Earthquake OBE | 1.2 | 0.25 | 2.413 | 4.733 |
| | | 0.50 | 2.230 | 3.799 |
| | | 0.75 | 2.123 | 3.415 |
| | | 1.00 | 2.032 | 3.113 |
| Steady Flow FWL, Q_{1000} El. +41.249, with Earthquake OBE | - | 0.25 | 2.413 | 3.299 |
| | | 0.50 | 2.230 | 2.956 |
| | | 0.75 | 2.123 | 2.755 |
| | | 1.00 | 2.032 | 2.596 |

Source: Analysis Results
**Table 7.** Comparison of Grawan Dam Stability Safety Factors with MDE Earthquake Load Conditions in the Upstream and Downstream Parts Using the Fellenius and Bishop Methods

| Condition                                      | FS_{min} | y/h | Upstream Safety Factor | Downstream Safety Factor |
|------------------------------------------------|----------|-----|-------------------------|--------------------------|
|                                                 |          |     | Fellenius               | Bishop                   |
| Steady Flow, MWL, EL. +35.00, with earthquake MDE | 1.2      |     | 0.25 2.889 2.994       | 0.25 2.243 2.321         |
| Steady Flow NWL EL. +39.00, with earthquake MDE | 1.2      |     | 0.50 2.060 2.136       | 0.50 1.729 1.781         |
| Steady Flow FWL Q1000 EL. +40.511, With Earthquake MDE | 1.2      |     | 0.75 1.744 1.809       | 0.75 1.508 1.550         |
| Steady Flow, FWL QPMF EL. +41.249, with Earthquake MDE | 1.2      |     | 1.00 1.516 1.575       | 1.00 1.341 1.376         |
| Rapid Drawdown, from NWL to MWL, with earthquake MDE |          |     |                        |                          |
|                                                 |          |     | 0.25 3.109 3.514       | 0.25 2.243 2.321         |
|                                                 |          |     | 0.50 2.005 2.196       | 0.50 1.729 1.781         |
|                                                 |          |     | 0.75 1.646 1.765       | 0.75 1.508 1.550         |
|                                                 |          |     | 1.00 1.400 1.484       | 1.00 1.341 1.376         |
|                                                 |          |     | 0.25 3.532 3.937       | 0.25 2.243 2.321         |
|                                                 |          |     | 0.50 2.115 2.253       | 0.50 1.729 1.781         |
|                                                 |          |     | 0.75 1.695 1.788       | 0.75 1.508 1.550         |
|                                                 |          |     | 1.00 1.423 1.491       | 1.00 1.341 1.376         |
|                                                 |          |     | 0.25 3.802 3.857       | 0.25 2.243 2.321         |
|                                                 |          |     | 0.50 2.194 2.260       | 0.50 1.729 1.781         |
|                                                 |          |     | 0.75 1.737 1.791       | 0.75 1.508 1.550         |
|                                                 |          |     | 1.00 1.449 1.493       | 1.00 1.341 1.376         |
|                                                 |          |     | 0.25 2.853 2.979       | 0.25 2.243 2.321         |
|                                                 |          |     | 0.50 2.009 2.095       | 0.50 1.729 1.781         |
|                                                 |          |     | 0.75 1.675 1.749       | 0.75 1.508 1.550         |
|                                                 |          |     | 1.00 1.420 1.475       | 1.00 1.341 1.376         |

Source: Analysis Results

3.5. Analysis of Spillway Hydraulics

3.5.1. Overflow/Spillway Threshold

In reviewing the hydraulics aspect, according to the as-built drawing, the overflow or spillway threshold type is of the ogee I type, but after review, the overflow threshold type is more suitable as the width threshold type. The width at the overflow threshold still meets to make the discharge flow in accordance with the current hydrological conditions and does not result in overtopping.

3.5.2. Vlugter-Type Energy Absorbers

These are the hydraulic calculations used in Vlugter pond planning if the length of the Vlugter pond floor ≤ 8.00 m and Z ≤ 4.50 m: For 1/3 ≤ Z/He ≤ 4/3, then D = L = R = He + 1.4 Z, and for 4/3 ≤ Z/He ≤ 10, then D = L = R = He + 1.1 Z. The length of the Vlugter pool on the as-built drawing does not yet meet the criteria. The length of the Vlugter pool on the as-built drawing is 4.6 m, while the length from the calculation results should be 5.539 m.

3.5.3. Transition Channels

Calculation of the transition channel utilized the energy equation formula. These are the calculation conditions for the transition channel:
- Outflow Q_{1000} hydrological data for 1973-2017 = 58.963 m³/sec
- Initial channel length: 15 m
- Length of the planned channel: 14 m
- Slope: 0
- Base elevation: +35.55
- Width of the upstream base: 15 m
- Width of the downstream base: 6 m
• Narrowing angle: 18° (qualify > 12.5°)
  To find out the length of the transition channel using the condition of the water level in the
upstream part of the transition channel, the water level profile in the transition channel is calculated
using the $Q_{1000}$, which is then controlled by the $Q_{PMF}$.

3.5.4. Floodway
The water level profile on the floodway was calculated using $Q_{1000}$, which is then controlled by the
$Q_{PMF}$. These are the calculation conditions for the floodway:
• Outflow $Q_{1000}$ hydrological data for 1973-2017 = 58.963 m$^3$/sec
• Length of floodway I: 10 m
• Length of floodway II: 35 m
• Slope of floodway I: 0.031
• Slope of floodway II: 0.18
• Channel width: 6 m

3.5.5. Stilling Basin
By considering the safety conditions of the river toward scouring, sub-critical flow is then planned to
occur at the final channel. In order for sub-critical flow to occur in the final channel, the slope in the
final channel must not exceed the critical slope. Therefore, the critical slope can be calculated using
the critical water level design with the $Q_{100}$ outflow discharge.
• Outflow $Q_{100}$ for 1973-2017: 47.464 m$^3$/sec
• Channel width: 6 m
• Manning roughness coefficient (layers of stone) = 0.025
In the as-built drawing, the type of energy damper is unknown, and thus the energy damper or
absorber is planned using the $Q_{100}$ discharge, which is then controlled by the $Q_{1000}$ discharge. From the
results of the analysis, the following values were obtained [4]:
• Discharge per unit of channel width $q = 17.26$ m$^3$/sec/m
• Flow velocity at the end of floodway $V = 20.10$ m/sec
• Froude number at the end of floodway $F_1 = 6.38$
From the evaluation, the plan is to use the USBR Type II Stilling Basin ($q > 45$ m$^3$/s/m, $V > 18$ m/sec,
Fr > 4.5).

3.6. Base Protection of Approach Channel
The following are results of scour depth analysis through the empirical approach:

| Table 8. Scour Analysis Results |
|--------------------------------|
| Return period | Discharge (m$^3$/s) | Veronese (m) | Lacey (m) | Schoklist (m) |
|----------------|----------------------|--------------|----------|--------------|
| 100            | 47.464               | 5.141        | 1.992    | 4.442        |
| 1000           | 58.963               | 6.072        | 2.474    | 5.285        |
| PMF            | 112.835              | 7.313        | 4.735    | 8.775        |

Source: Analysis Results

To protect from planned scour, the materials for rip-rap are composed of various types of rocks
with a minimum diameter of 0.3 m or 30 cm with a minimum rock specific gravity of 500.57 kg/m$^3$.
The rip-rap is planned to consist of 6 layers of rock that can reduce scour by 1.8 m. The length of the
planned channel is $4Y_2 = 4 \times 4.604 \text{ m} = 18.416$ or 18.5 m.

3.7. Stability Analysis
According to Bowles, as the above equation increased by less than 50%, and because the width of the foundation is ≥ 1.2 m (the foundation width of the overflow/spillway threshold is 9 m), then given the foundation depth factor (Kd) with an overflow/spillway threshold (D) foundation depth equal to 2.5 m, the following is the calculation of the net allowable bearing capacity:

\[ q_d = 12.5N \left( \frac{B + 0.3}{B} \right)^2 K_d \]

\[ q_d = 12.5(19.5) \left( \frac{9 + 0.3}{9} \right)^2 \left( 1 + 0.33 \frac{2.5}{9} \right) \]

\[ q_d = 228.88 \text{kN/m}^2 = 22.88 \text{ton/m}^2 \]

In the same way, the bearing capacity of the soil for the retaining wall becomes:

- Transition channel retaining wall: 25.56 ton/m²
- Flood way retaining wall: 25.56 ton/m²
- Energy damper/absorber retaining wall: 27.15 ton/m²

### Table 9. Overview of Soil Bearing Capacity for the Spillway Threshold

| No. | Stability analysis overview | Maximum stress (σ max) | Minimum stress (σ min) | Allowable stress (σ allow) | Bearing capacity control (ton/m²) |
|-----|----------------------------|------------------------|------------------------|---------------------------|----------------------------------|
| A   | Normal condition           |                        |                        |                           |                                 |
| 1   | Empty channel condition    | 21.22                  | 1.44                   | 22.89                     | OK                               |
| 2   | Full channel condition     | 5.12                   | 1.87                   | 22.89                     | OK                               |
| 3   | Flooded channel condition  | 1.46                   | 1.42                   | 22.89                     | OK                               |

| B   | Earthquake condition       |                        |                        |                           |                                 |
| 1   | Empty channel condition    | 8.78                   | 3.06                   | 22.89                     | OK                               |
| 2   | Full channel condition     | 7.44                   | 1.46                   | 22.89                     | OK                               |
| 3   | Flooded channel condition  | 2.43                   | 0.53                   | 22.89                     | OK                               |

Source: Calculation Results, 2019

From the analysis of the security or safety of stability toward overturning and sliding in the overflow or spillway structure and retaining wall, it was found that the structures are in a secure or safe condition and exceed the required critical safety value.

### 4. Conclusion

Based on the results of the study from the formulated problems, these conclusions can be made:

1. The current hydrological conditions of Grawan Dam, based on the results of flood routing, shows a safe condition toward the free board with the maximum water height above spillway being 2,499 m at the elevation of +41.249.
2. The seepage condition of Grawan Dam, with various water conditions and a seepage allowable limit of 1% x average inflow discharge = 0.01121 m³/sec, shows that the initial design condition, piezometer monitoring, and the current seepage condition are still in safe conditions, with a maximum seepage discharge of 1.07 x 10⁻² m³/sec.
3. The stability of the Grawan Dam slope with various water level conditions were analysed based on the 2017 Earthquake Map with FK ≤ 1 and shows that in the earthquake-free condition, the condition of Operating Basis Earthquake (OBE) with a 100-year return period, and the condition of Maximum Design Earthquake (MDE) with a 5000-year return period, the slope is 100% safe.
4. These are the evaluation results for the overflow or spillway structure system:
   a. The spillway or overflow threshold type is adjusted to the structure type, being of the width threshold type.
   b. The length of the Vlugter stilling basin requires a channel extension.
c. The transition channel should be shortened to 14 m.
d. The type of energy absorbers should be USBR type III.

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