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Design Review Of Leuwikeris Dam Acces Road Pakage 4

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ABSTRACT

The construction of the Leuwikeris Dam requires materials to be used for stockpiling. Therefore, it is necessary to access the road from the quarry in question to the location of the dam. This access road is located in Cibodas Village RT 32 / RW 15 Ciharalang Village, Cijeungjing District, Ciamis Regency, West Java. The construction of access roads needs to build bridges connecting the valleys which requires making embankments to reach the elevation of the road plan. The embankment is made up to ± 15 m using embankment material around the site. This design review was carried out because a landslide occurred on one of the embankment slopes which did not rule out the possibility that it could occur on the opposite slope. This design review uses the help of the SLOPE/W program to determine the level of slope safety and alternative slope reinforcement designs. It should be noted that the data used are secondary data obtained from related parties. Some things that can be concluded from the design review are as follows: 1) The landslide occurred due to the failure of the existing gabion foundation to withstand the load of the slope. 2) Repair the gabion foundation by enlarging and deepening the dimensions (attached image), so that it can cut the landslide area. The dimensions of the large foundation are expected to provide sufficient counter weight. 3) The landslide material must be cleaned and start compaction per layer from the beginning. Ensure that the backfill is of maximum density. 4) Improved gabion design so as to obtain sufficient volume weight of at least 1.5 t/m3. The installation of bamboo chimneys is still being carried out as additional reinforcement.

Keywords: *Safety factor; Gabion; Road; Embankment; Slopes stability; Slope-W.*

1. INTRODUCTION

1.1. Background

The construction of the Leuwikeris Dam requires materials to be used for stockpiling. Therefore, it is necessary to access the road from the quarry in question to the location of the dam. This access road is located in Cibodas Village RT 32 / RW 15 Ciharalang Village, Cijeungjing District, Ciamis Regency, West Java. The construction of access roads needs to build bridges connecting the valleys which requires making embankments to reach the elevation of the road plan. The embankment is made up to ± 15 m using embankment material around the site.

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This design review was carried out because a landslide occurred on one of the embankment slopes which did not rule out the possibility that it could occur on the opposite slope. The picture above shows a general failure from top to bottom, this is evidenced by the fact that the Gabion has been lifted to the ground. This could be due to the fact that the lower gabion foundation structure did not penetrate the landslide area, resulting in a collapse.

Fig. 1. Research Location.

The DED image of the existing slope reinforcement is shown in the following figure:

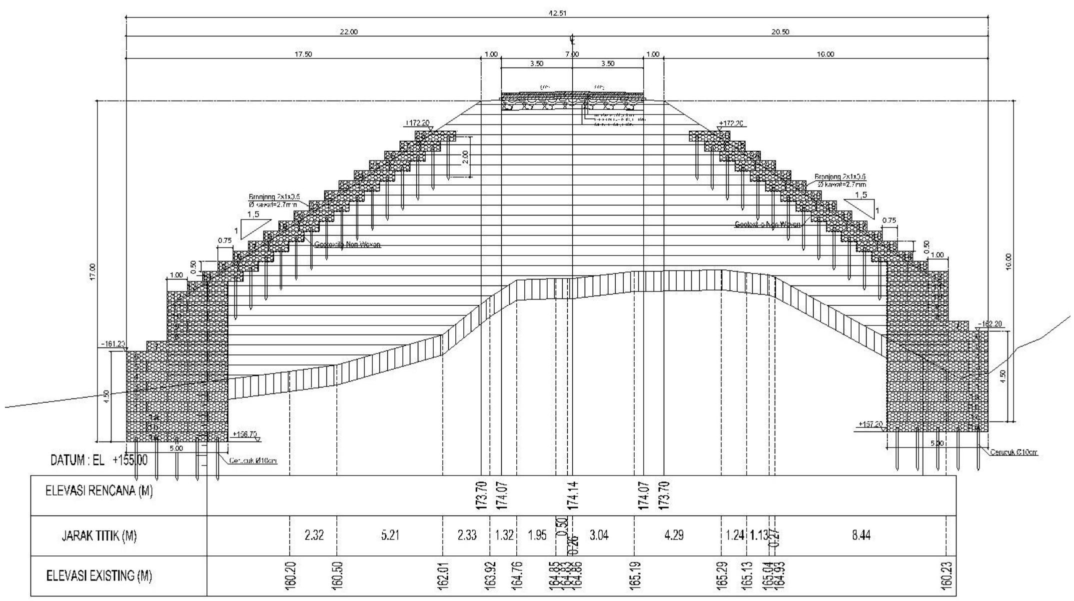

Figure 2. Slope cross section

1.2. Literature Study

2. 1.Slope Stability

Slope stability analysis is generally based on the concept of limit plastic equilibrium (limit plastic equilibrium). The purpose of the stability analysis is to determine the safety factor of a potential landslide field. The safety factor is defined as the value of the ratio between the holding force and the driving force (Hardiyatmo, 2012)

$$SF = \frac{\tau}{\tau_d} \tag{1}$$

with :

SF = safety factor

τ = maximum shear resistance (kN/m^2)

τ_d = shear resistance occurs (kN/m^2)

SNI 8460-2017 recommends the following safe factor categories:

Tabel 1. Recommended slope SF value

Rock slope conditions	Recommended safety factor values
Permanent slope conditions	1.5
Temporary condition	1.3

The method of calculating the forces acting on the landslide plane has been developed by several researchers, including: Bishop's rigorous, Spencer's, Sarma's and Morgenstern-Price which provide a more complex way by taking into account the moment force balance. The forces acting on the landslide section are shown in Figure 2. As follows:

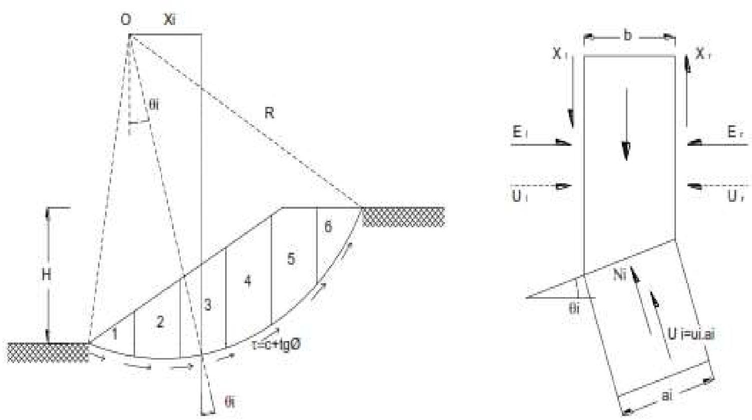

Figure 3. The force acting on the slice of the landslide plane

Soil shear strength parameters consist of cohesion (c) and internal friction angle (□). According to Mohr-Coulumb (1776) in Hardiyatmo (2012) gives the following general equation:

$$\tau = c + \sigma tg\varphi \qquad (2)$$

with :

τ = soil shear streght (kN/m^2)

c = soil cohesion (kN/m^2)

□ = friction angle (º)

σ = normal stress at the failure surface (kN/m^2) (kN/m^2)

1.2.1. Numerical Simulation

Modeling of soil and gabion materials in numerical simulations uses the Mohr-Coulumb Material Model. Determination of soil parameters and reinforcement material parameters requires data analysis, based on the results of field tests, as well as from laboratory tests. Field testing includes 5 deep drill points with varying depths of BH 1 (30m), BH 2 (42 m), BH 3 (30 m), BH4 (30m), BH 5 (30 m). In addition, a 3-point sondir test was also carried out. Laboratory testing is carried out on embankment soil including index properties, direct shear test and standard proctor. The locations for field testing are as follows:

Figure 4. Deep Drill and Sondir Test Locations

The landslide location is located in the area at the S3 and BH 5 positions, so the data we use is at that point. The results of the stratigraphy of the soil investigation are shown in the following figure:

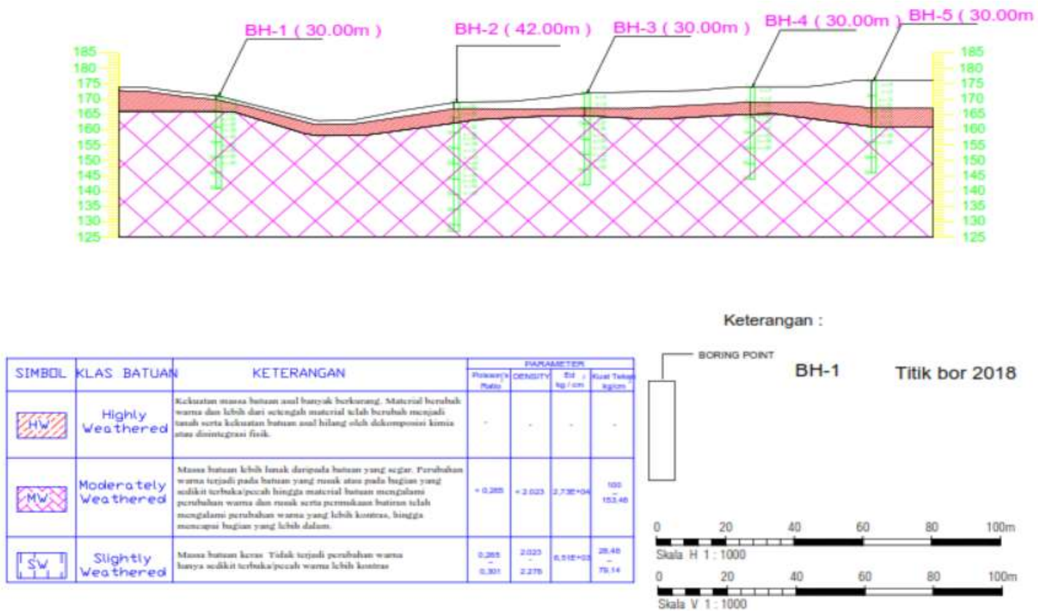

Figure 5. Rock Class Stratigraphy

1.2.2. Cone Penetration Test (CPT)

"Sondir" testing or CPT (Cone Penetration Test) in Indonesia refers to SNI 2827:2008 or ASTM D 3441-86 on how to test field penetration with sondir. The main parameters resulting from the sondir test are qc end resistance and frictional resistance. In the sondiri test, the standard cone with a diameter of 35.7 mm + 0.4 mm with a conical angle of 60° + 5° and a sliding blanket surface area of 150 cm2 + 3 cm2 is penetrated through the soil layer by being pressed both mechanically and hydraulically with a penetration speed of 10 mm/s – 20 mm /s + 5 mm which is read every 20 cm. Cone resistance is recorded from the reading of 2 pieces manometer with a capacity of 0 MPa - 2 MPa for and 0 MPa - 5 MPa for relatively soft soils, or 0 MPa - 5 MPa for and 0 MPa - 25 MPa for moderately hard soil layers. In this soil investigation work, the CPT test was carried out using a sondir machine with a capacity of 2.5 tons. Soil investigation data used is at point BH 5 and so it needs to be matched as follows:

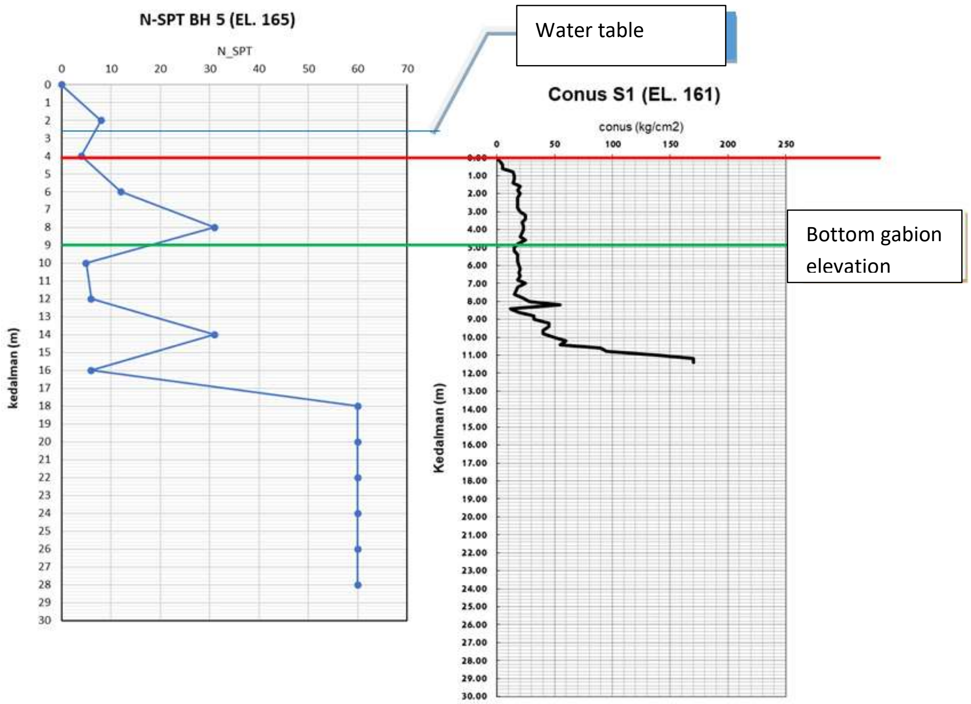

Figure 6. Comparison of Sondir and Boring Results

Figure 6 shows results that tend to match, the top layer is a layer of soft/loose soil, then at a depth of 12 m from the results of the CPT the soil begins to harden. It is also seen that the gabion foundation is still located on soft soil.

1.2.3. Material Properties

Laboratory tests were carried out on the embankment soil, based on laboratory tests of the embankment soil, including the type MH (clay silt with high plasticity). Soil with a high plasticity value is not recommended as an embankment material. This type of soil will soften when exposed to water. If the soil is soft, the shear strength of the soil will decrease drastically. However, from the test results the c and phi values are quite high so that in this analysis it is considered a stable embankment. The recapitulation of laboratory test results is as follows:

Tabel 2. Recapitulation of Laboratory Test results

No	Grainsize (%)		Atterberg (%)		γb (kN/m^3)	Direct Shear		Classification
						c (kPa)	Ø (°)	
1	Gravel	0.00	LL	73.45	17.55	61.40	23.10	MH
2	Sand	19.32	PL	37.81				
3	Silt	42.98	PI	35.64				

4	Clay	37.70	SL	67.72

One of the important steps in slope stability analysis is the determination of the parameters of the shear strength of the soil. This parameter is obtained from the results of field testing combined with laboratory testing. In this analysis, soil parameter data from laboratory test results is minimal, so this lab data is not used for analysis. Instead, data from sondir and SPT test results from deep drills are used.

Broadly speaking, the soil layer is divided into 4 parts; Embankment, Layer 1, Layer 2, and Layer 3. The embankment layer is in accordance with the results of laboratory tests, the upper layer has an average SPT value of 9, the middle layer has an average SPT of 18, and the lower layer is a rock layer with an SPT > 50.

Determination of the shear strength of the soil for each layer is determined by the correlation of the undrained strength value to the SPT value. According to Terzaghi the value of Cu can be estimated

$$Cu = 2/3\ N \tag{3}$$

with N = SPT value of soil layer

The slope safety value can be obtained by conducting a "Trial Error" on several landslide areas which are generally circular arcs and then the minimum F value is taken as an indication of the critical landslide area.

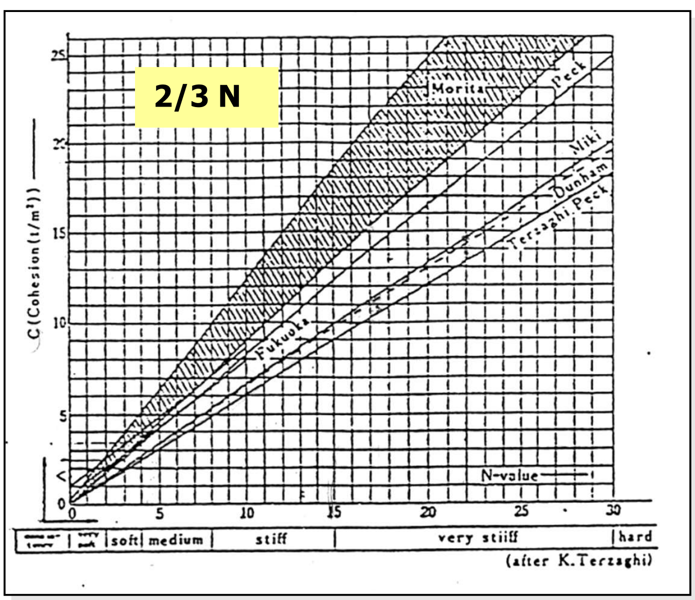

Figure 7. Correlation of Cu and SPT values

The loading used is assumed to be a uniform load of 100 kN/m2 at the top of the slope because it will be used as an access road. The seismic load used based on Seed (1979) recommends the following kh: 0.10 for close to capable of producing an earthquake of magnitude 6.5.

Gabion materials take reference from Maccaferi Gabion Product with b = 15 kN/m^3, c= 12.5 kN/m^2 and phi = 50 °. The material properties used are as follows:

Tabel 3. Material Properties

No	Layer of soil	Unit (m)	C (kPa)	Ø (º)	Material Model
1	Embankment	17.55	61.40	23.1	Mohr-Coulomb
2	1	17	60	0	Mohr-Coulomb
3	2	17	120	0	Mohr-Coulomb
4	3	17	330	0	Mohr-Coulomb
5	Gabion	15	12.5	50	Mohr-Coulomb

2. RESEARCH METHOD

All stability calculations were performed using the Morgenstern-Price method with entry-exit slip surface definition. Pore -water pressure is determined using the groundwater table.

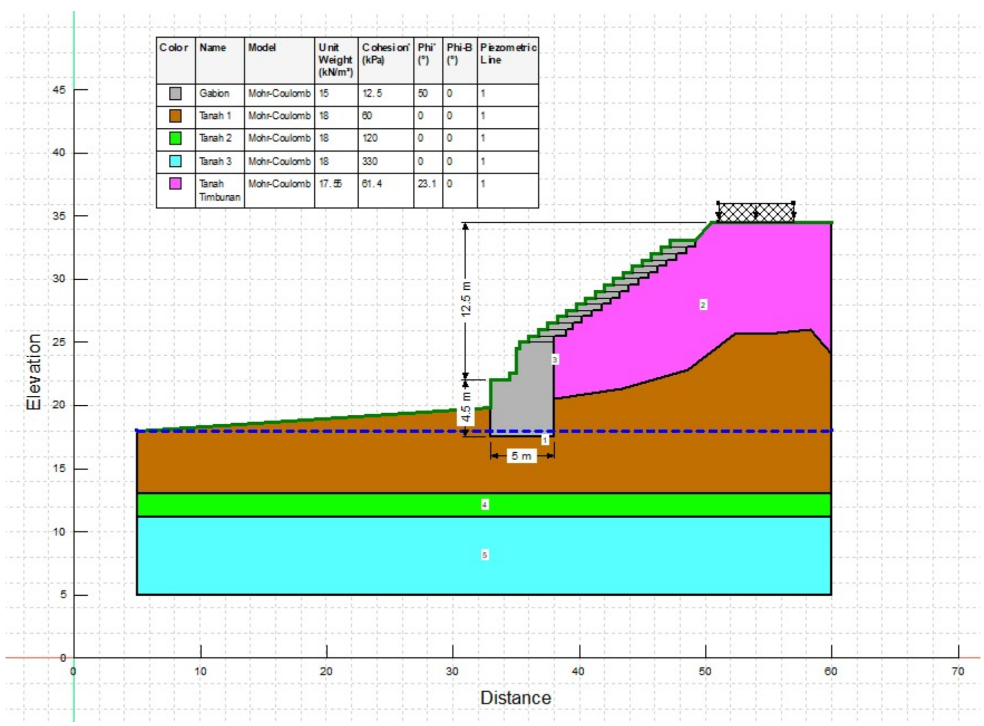

Figure 8. Geometry and Material Properties Slope/W Analysis

Model analysis carried out include:

1. Unreinforced slope

2. Unreinforced slopes with earthquake loads
3. Slope with existing gabion reinforcement without earthquake load
4. Slope with existing gabion reinforcement with earthquake load
5. Slope with gabion reinforcement Alternative without earthquake load
6. Slopes with alternative gabion reinforcement with earthquake loads

Performing multiple simulations is a good practice to analyze each failure mode independently. Various possibilities can occur that can result in a landslide. This is generally achieved by adjusting the location of the slip surface search zone. However, it is equally important to make assumptions about the effect of the reinforcing components on the system stability. In this case study, the additional shear resistance of the gabion basket steel net was not considered. Only the shear resistance mobilized by the rock in the gabion basket is considered.

Fig. 9. Reseach Flow

3. RESULT AND DISCUSSION

3.1.1. Slope/W Simulation Existing Condition

Unreinforced slopes have a factor of safety < 1.5 (Figures 10 and 11). As expected, the critical failure mode is an active wedge-like failure with an exit point located at the end.

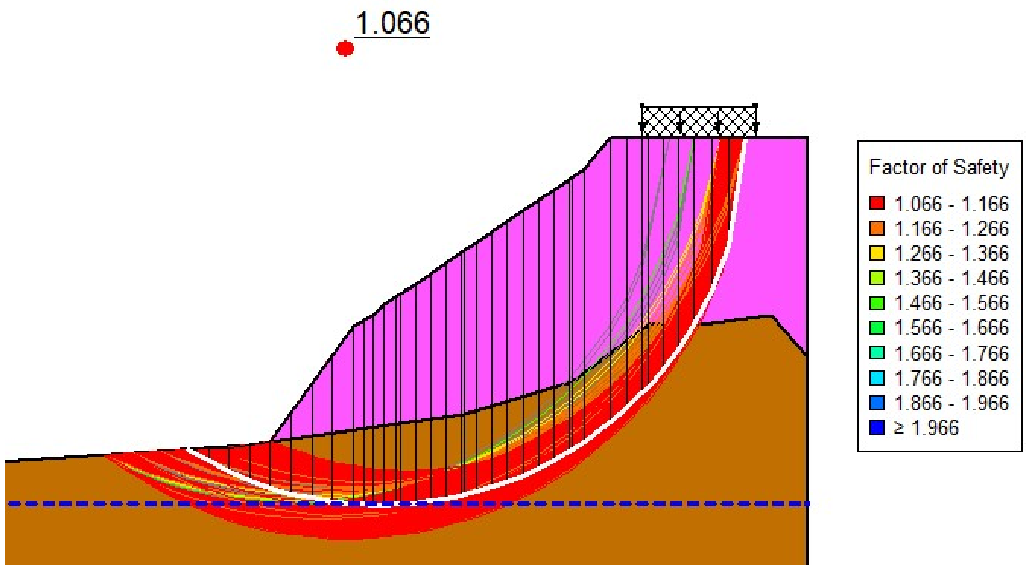

Figure 10. Slope without reinforcement

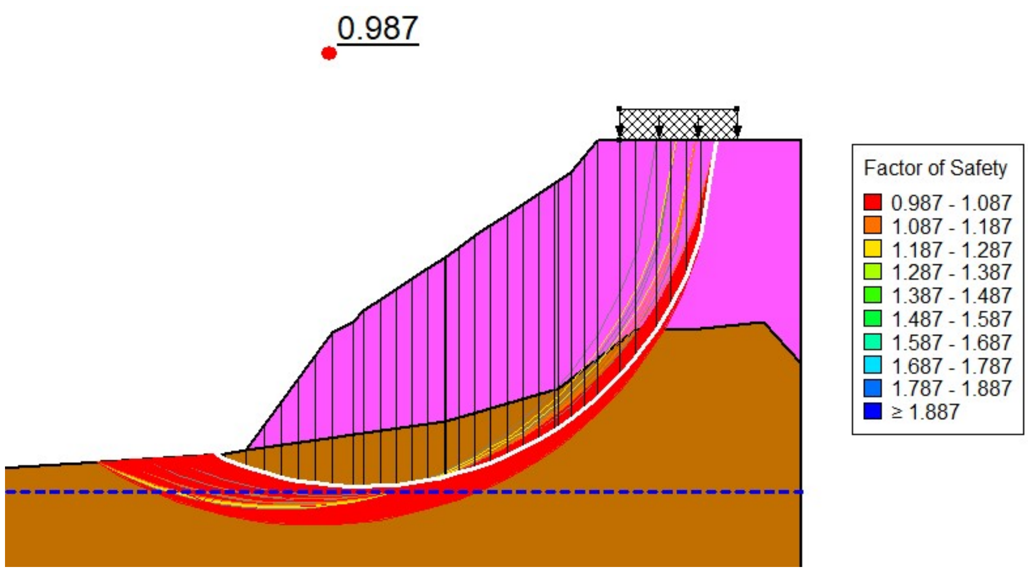

Figure 11. Unreinforced slopes with earthquake load

The slip surface color map, and the associated legend, show that the FOS is less than 1.5 for all slip surfaces located within the embankment and subgrade.

Figures 12 and 13 show the results of the existing reinforcement analysis, it appears that the FOS increases compared to the unreinforced slope, but the FOS is < 1.5 so the slope is in an unsafe condition. The critical slip surface occurs under the gabion sub-base. The collapse starts from under the foundation and then collapses above it. This can be seen in the existing photo (Figure 1). The gabion foundation is lifted to the ground level.

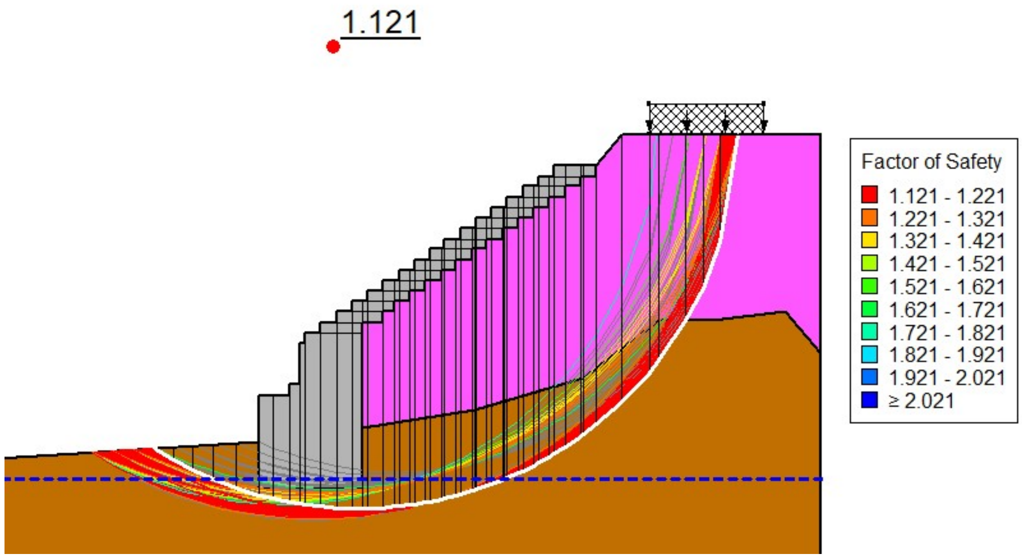

Figure 12. Slope with gabion reinforcement without earthquake load

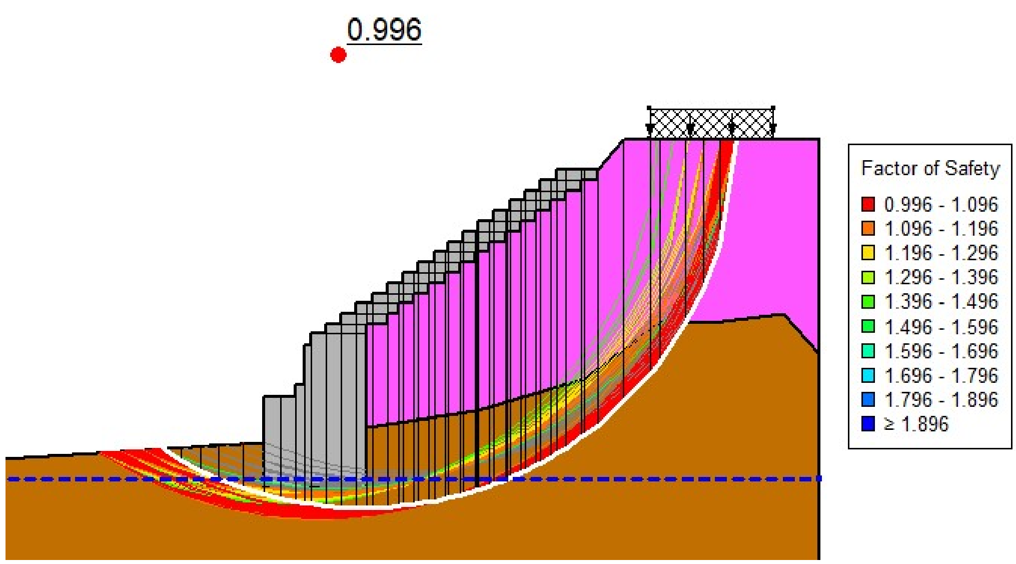

Figure 13. Slope with gabion reinforcement with earthquake load

3.1.2. Slope/W Simulation Alternative Reinforcement

Efforts so that the foundation can withstand the load of the slopes is one of them by increasing the depth of the foundation so that it can cut the landslide field. In addition, it increases the mass of the sub foundation so that it can be used as a Counter Weight. The simulations are shown in Figures 14 and 15. The simulation of the modification of the gabion foundation is quite significant, as seen in the FOS value > 1.5, which means the slope is in a safe condition.

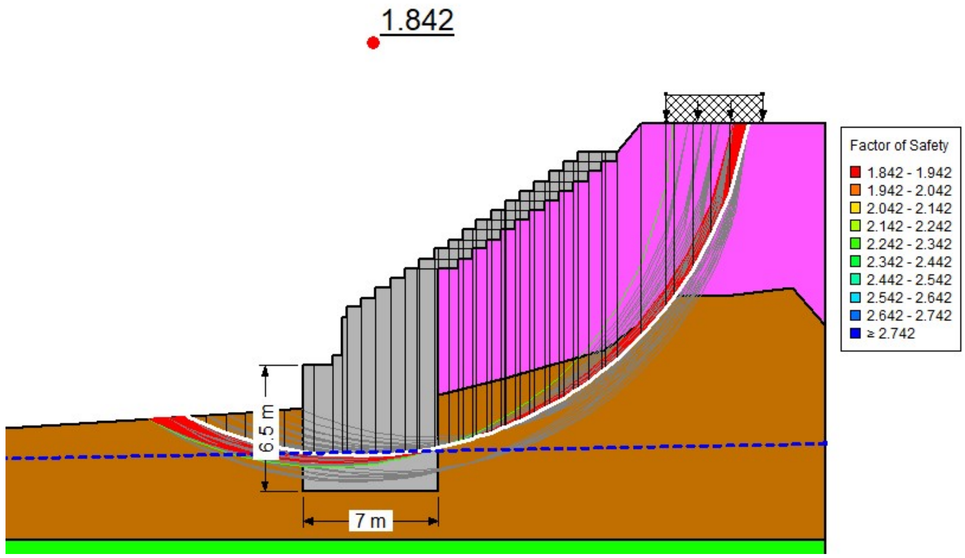

Figure 14. Slope with gabion reinforcement alternative without earthquake load

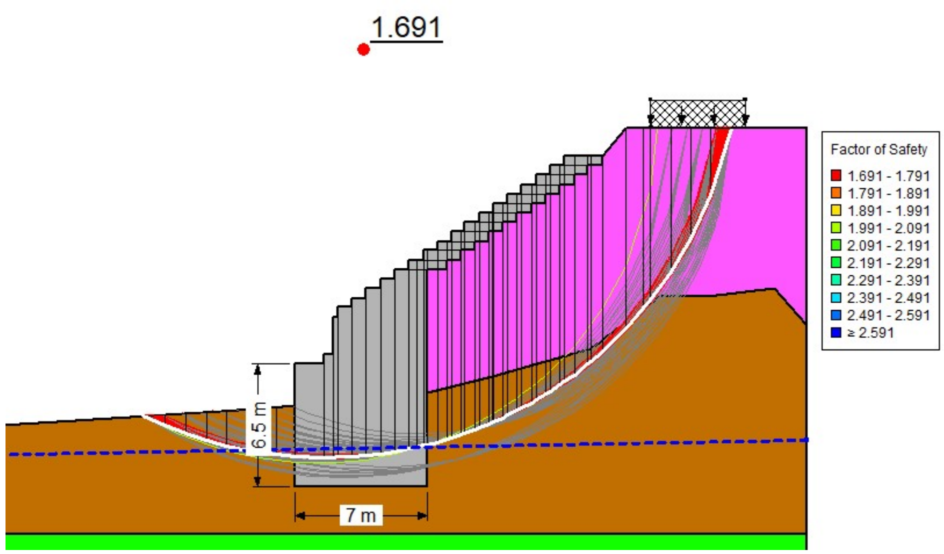

Figure 15 Slope with gabion reinforcement alternative with earthquake load

Figure 16 presents the FOS vs Lambda convergence plot for Analysis 6. The cross-over between the FOS moment line and the FOS force is unambiguous, indicating an acceptable convergence. In addition, most of the slip surface in the desired zone shows a clean convergence.

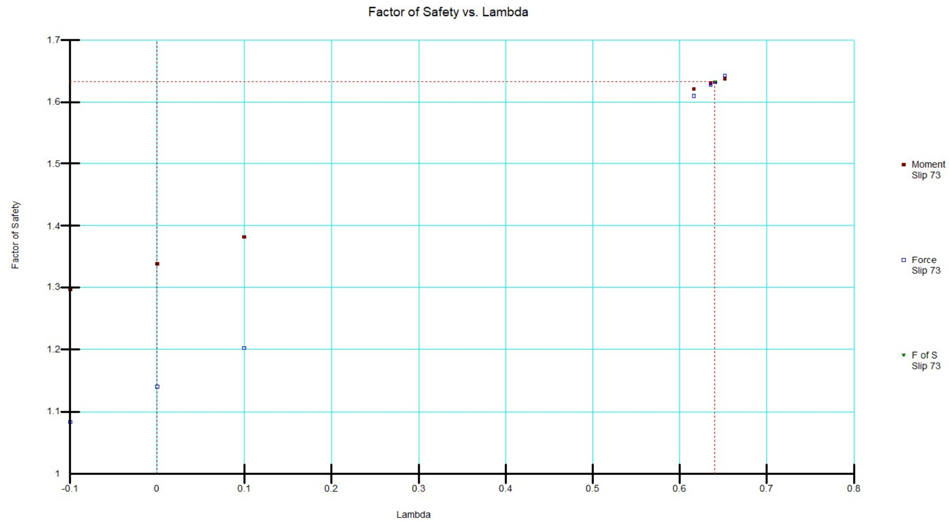

Figure 16. FOS vs Lambda on model analysis number 6

3.1.3. Recapitulation Slope Stability

The recapitulation of the results of the slope stability analysis with Slope/W is as follows:

Tabel 4. Material Properties

No	Type of Model's	Description	Safety Factor	Explanation
1	Model 1	Slope without reinforcement	1.066	Not Safe
2	Model 2	Unreinforced slopes with earthquake loads	0.987	Not Safe
3	Model 3	Slope with existing gabion reinforcement without earthquake load	1.121	Not Safe
4	Model 4	Slope with existing gabion reinforcement with earthquake load	0.996	Not Safe
5	Model 5	Slope with gabion reinforcement Alternative without earthquake load	1.842	Safe
5	Model 6	Slope with gabion reinforcement Alternative with earthquake load	1.706	Safe

4. CONCLUSION

The slope stability analysis above tries various things then happens. Some things that can be concluded from the design review are as follows:

1. The landslide occurred due to the failure of the existing gabion foundation to withstand the load of the slope. This is evidenced in the simulation results in SF = 0.996 – 1.121.
2. Modification of the form of gabion reinforcement can increase the safety factor to 1.706 - 1.842.

5. RECOMMENDATION

1. Repair the gabion foundation by enlarging and deepening the dimensions (attached image), so that it can cut the landslide area. The dimensions of the large foundation are expected to provide sufficient counter weight.
2. The landslide material must be cleaned and start compaction per layer from the beginning. Ensure that the backfill is of maximum density.
3. Improved gabion design so as to obtain sufficient volume weight of at least 1.5 t/m3. The installation of bamboo chimneys is still being carried out as additional reinforcement.

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