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Landslide Mitigation at The Bagong Dam Abutment, Trenggalek District

Gilang Bobby Hilmawan [a,*] and Ignatius Sriyana [b]

[a] PT. Brantas Abipraya, Jl. D.I. Panjaitan Kav. 14 Cawang Jakarta Timur, 13340, Indonesia
[b] Department of Civil Engineering, Universitas Diponegoro, Jalan Prof. Sudarto 13, Semarang, 50275, Indonesia

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Geoslope
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Dam

ABSTRACT

Landslides occurred continuously from July 2022 until July 2023, disrupting the construction of the Bagong Dam abutment. Geologically, the foundation of the Bagong Dam consists of a fairly thick colluvial layer, which is prone to landslides. So, the analysis of landslide mitigation at the Bagong Dam abutment is needed. The slope stability analysis carried out by Fellenius and Bishop method, then the slope modeling was carried out using Geostudio software. The analysis results on the existing slopes produced a safety factor of 0.987 (<1.07) for the Fellenius method and 1.042 (<1.07) for the Bishop method. These safety factors indicate that the existing slope is unstable and slope failure is likely to occur. In the first alternative countermeasure analysis, the slope safety factors for the cross-section of the dam at STA 0+625 were 1.715 for upstream and 1.338 for downstream; at STA 0+641, 1.321 upstream and 1.306 downstream; and for the longitudinal section of the dam, 1.525. All these safety factors greater than 1.25, indicating that the slope is stable. In the second alternative countermeasure, the slope safety factors obtained for the cross-section of STA 0+641 were 1.362 for upstream and 1.386 for downstream, and 1.657 for the longitudinal section. These safety factors are also greater than 1.25, which indicates the slope is in stable condition. The additional cost for implementing the first alternative countermeasure is 73.9 million, while for the second alternative is 35.7 million. So that, the second alternative countermeasure is the best choice by the multi-criteria decision-making analysis results.

1. Introduction

During the construction of the Bagong dam, several technical obstacles were encountered, including landslides. Landslides occurred several times during the Bagong dam construction process until May 2023. The landslides specifically affected the excavation site on the right side of the main dam abutment. Although the excavation was completed according to the approved shop drawings, a significant landslide occurred on July 7th, 2022, involving a large mass of rocky soil. This landslide damaged parts of the location planned for facility buildings.

The landslide began at STA 0+641, which was intended to be used for a helipad facility, and resulted in a decrease to a depth of 15.5 meters. This caused the surface below the elevation to be pushed up to STA 0+550. The landslide affected the entire excavation site, as illustrated in Figure 1.

One of the causes of landslides is the influence of stratigraphy (geological layers below the surface). Slope landslides on residual soil, especially on steep slopes, do not follow a deep, circular plane typical of other types of landslides. Instead, the landslide plane on residual soil slopes is relatively shallow, often forming a slight curve or nearly planar surface. Despite this, the volume of material involved in these landslides can still be very large. Based on statistical data, more than 94% of colluvial landslides occur due to the influence of rain and human activities, with continuous rainfall being a major factor. Rainwater continuously seeps into the soil and rock contact surface through the overlying gravelly soil, forming a temporary saturation zone. As rainfall duration increases, this temporary saturation zone gradually expands. The strength

*Corresponding author.
E-mail: hilmawangilangbobby@gmail.com

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of the rock and soil at the contact surface softens, the bulk density of the slope increases, pore water pressure rises, and the stability of the slope decreases, ultimately leading to a landslide [1].

Landslides can occur at the contact area between rock and clay, as well as in colluvial deposits. Colluvial material has the property of easily passing water, so when it rains, water seeps into the colluvial layer and is retained by the underlying clay. This retention causes the clay surface to become slippery, which can lead to landslides [2]. Additionally, rainwater infiltrates the soil, increasing soil pore water pressure. Positive water pressure creates capillarity, raising the groundwater level. The increased groundwater level adds to the soil mass and weakens the bonds between soil particles [3]. As the soil mass increases, the force acting on the potential landslide area grows, while the increased pore pressure weakens the bonds between soil particles, reducing the resisting force. A landslide occurs when the balance of forces is disturbed, specifically when the driving force exceeds the resisting force [2]. This research aims to analyze and determine steps to prevent landslides at the Bagong Dam abutment. Mitigating landslides at the main dam abutment involves considering many factors as design criteria for the dam foundation, including bearing capacity, slope stability, and seepage. In this case study, the focus will primarily be on slope stability.

2. Method

2.1 Bishop Method

Bishop's method assumes that the shear forces on the sides of the wedge are equal and opposite in direction, $V1=V2$. However, the normal forces on the slices are not of the same magnitude, $E1\neq E2$ [4]. Bishop's method also assumes that the forces acting on the slice have zero resultant in the vertical direction [5].

In Bishop's method, the solution is found using trial and error; the value of the safety factor on the left and right sides must be the same. Equation 1 applies for the condition without a water level.

$$F = \frac{\Sigma_{i=1}^{i=n}(C'b_i+(W_i-ui)\tan\varphi)\left(\frac{1}{Cos\ \theta i+\frac{\tan\varphi\ \sin\theta}{F}}\right)}{\Sigma_{i=1}^{i=n} W_i\ Sin\ \theta i} \quad (1)$$

Where F is the safety factor, θ_i is the angle of slice (^{0}), C' is the effective soil cohesion (kN/m^2), b_i is the width of slice-i (m), W_i is the weight of slice-i (kN), U_i is the pore water pressure at slice-i (kN/m^2), and φ is the friction angle of soil (^{0}).

2.2 Fellenius Method

The Fellenius method (Ordinary Method of Slices) was first introduced by Fellenius in 1927. Slope analysis using this method assumes that the forces acting on the right and left sides of any slice have a zero resultant in the direction perpendicular to the landslide plane. The data needed to calculate the safety factor includes slope dimension data and soil mechanics data from the slope [6]. The forces and plane assumptions on each landslide plane are illustrated in Figure 2.

2.3 Rainfall and Pore Water Pressure

One of the causes of landslides is high-intensity rainfall. High-intensity rainfall with a long duration increases the water content in the soil. The rainfall intensity data at the Bagong Dam over the past 33 years shows a quite high intensity (heavy), >50 mm/day. High rainfall intensity can change soil conditions from unsaturated to saturated, increasing pore water pressure and reducing soil shear strength (φ) and soil cohesion (c).

Figure 1. Documentation of landslides on the right side of the main dam abutment

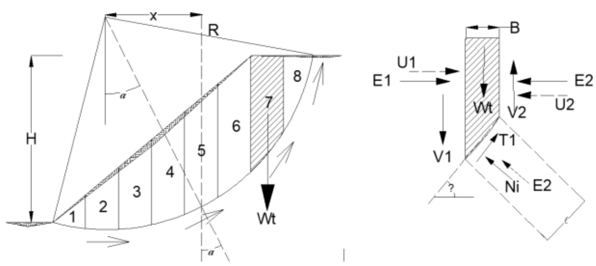

Figure 2. Forces and plane assumptions on each landslide plane slope

Pore water pressure is the pressure generated by water trapped in soil pores, which, when increased, reduces the slope safety factor [7]. Pore water pressure causes lifting forces and reduces the strength of the rock mass that makes up the slope, thereby affecting the stability of the slope [8].

Rainfall is the dominant factor influencing landslide distribution. Generally, higher rainfall intensity results in higher concentrations of landslides, with roughly exponential growth [9]. Continuous heavy rainfall causes pore water pressure to rise from previous levels, reducing the shear strength of the soil and thereby triggering extensive landslides. Landslides do not occur if rainfall is insufficient to cause the pore water pressure to reach the maximum static pore water pressure produced by heavy rainfall [10].

2.4 Soil Shear Strength

Based on the force assumptions, the slope safety factor is calculated using the Equation 2.

$$F = \frac{\Sigma_{i=1}^{i=n}(C' b_i + (W_i \cos\theta i - ui\ bi) tan\ \varphi}{\Sigma_{i=1}^{i=n} W_i\ Sin\ \theta i} \quad (2)$$

Shear strength consists of cohesion (c) and internal friction angle (ø). To analyze the slope stability, the maximum effective shear strength parameters (c', ø') is used. Shear strength parameters can be obtained from field tests such as CPT (cone penetration test) and SPT (standard penetration test), as well as from laboratory tests including unconsolidated undrained triaxial, undrained consolidated triaxial, drained consolidated triaxial tests, direct shear tests, and free compression tests.

The shear stress at failure according to Mohr's failure theory is as Equation 3 [11].

$$\tau = c + \sigma \tan \varnothing \quad (3)$$

Where τ is the landslide shear stress in all planes, σ is the normal stress in the plane, c is the cohesion, and $\varnothing$ is the friction angle

2.5 Basic Principles for Mitigating Ground Movements

Good countermeasures can effectively address problems at a relatively low cost and are easy to implement [2]. The alternative landslide management strategies include changing the slope geometry, controlling surface water, controlling seepage water, anchoring and other measures.

Changing the slope geometry, involves modifying the slope by cutting it to create a gentler incline. This approach aims to reduce the driving force by altering the slope angle and increasing the resisting force by filling material at the base of the slope. Controlling surface water

Controlling surface water, is essential to prevent or minimize seepage into landslide areas. This can be achieved by constructing drainage systems or water channels to divert surface water away from the slopes.

Controlling seepage water, reducing the groundwater level in landslide-prone areas is crucial. Methods for controlling seepage water commonly include constructing deep wells, installing vertical and horizontal drainage systems, and implementing relief wells.

Anchoring and other measures, Soil anchoring involves securing moving masses of soil using various support structures such as gabions, retaining walls, piles, soldier piles, and steel sheet piles.

2.6 Safety Factor

The safety factor is divided into several categories based on the critical Bowles collapse value [6]. The relationship between the safety factor (SF) and landslide intensity is illustrated in the Table 1.

Table 1 Relationship between Safety Factor (SF) and Landslide Intensity [4]

Safety Factor Value	Slope Conditions	Information
< 1.07	Slope Collapse Usually Occurs	Unstable Slopes
1.07 – 1.25	Slope Collapses Have Occurred	Critical Slope
Fs > 1.25	Slope Collapses Are Rare	Stable Slope

2.7 Research Site

This case study was conducted at Bagong Dam, administratively located in Sumurup Village, Bendungan District, Trenggalek Regency, East Java Province (Figure 3). Situated within the western part of the Brantas watershed, specifically in the Ngrowo-Ngasinan sub-watershed, Bagong sub-watershed, the dam's location is delineated by river boundaries. To reach the study site, a 4-wheeled vehicle was utilized from Trenggalek city, heading north to Bendungan sub-district, approximately 10 km from Trenggalek city. The focus of this research is the landslide on the right side of the Bagong Dam abutment, as shown in Figure 4.

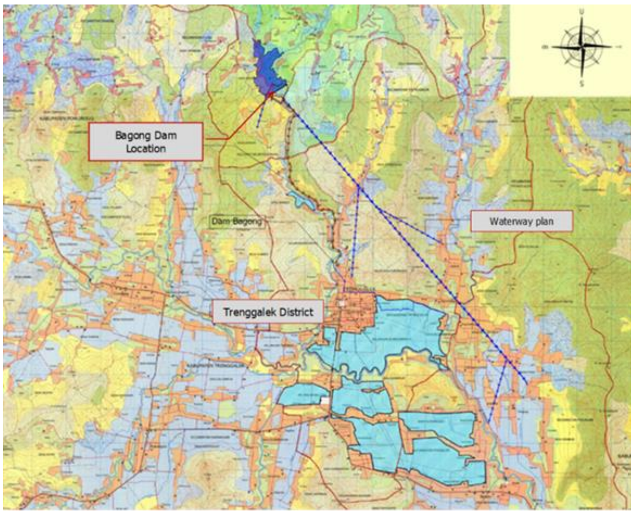

Figure 3. Location of Bagong Dam

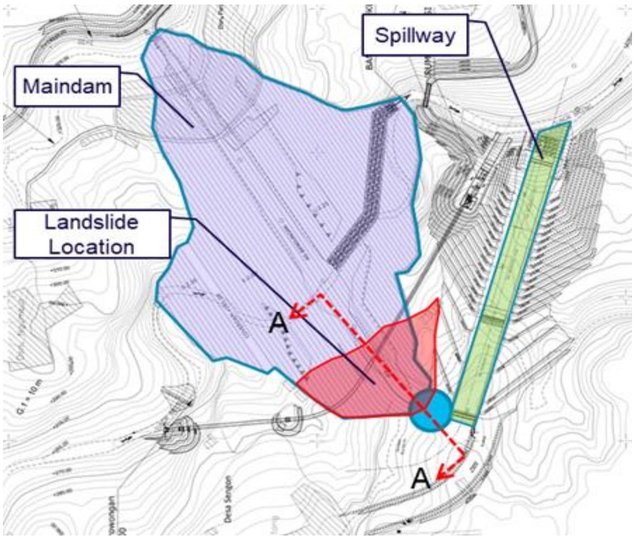

Figure 4. Location of the landslide

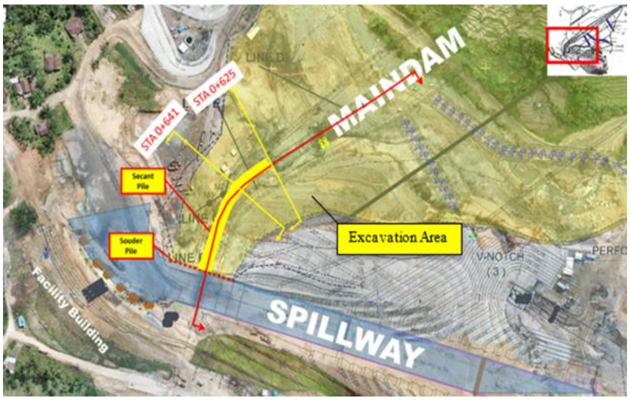

Figure 5. Alternative countermeasure Layout 1

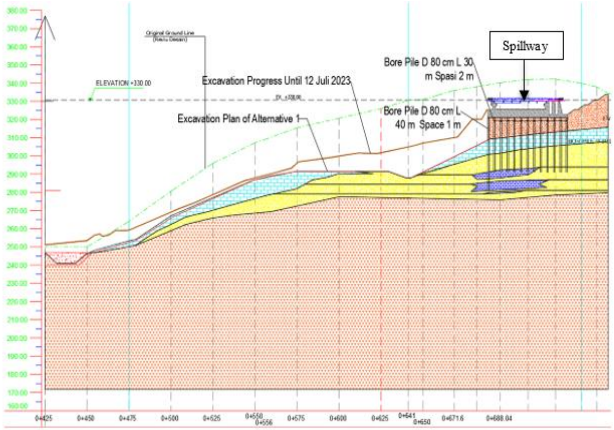

Figure 6. Longitudinal section of alternative excavation 1

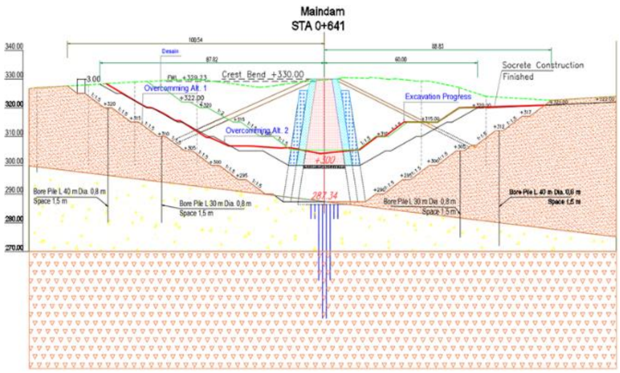

Figure 7. Typical cross section of alternative excavation 1

2.8 Research Stages

The research stages to be carried out are as data collection, landslide prevention analysis, and landslide modeling. The data required for slope stability analysis are shop drawings, geological investigation reports, soil parameter data such as internal friction angle, cohesion, and specific gravity, groundwater level data, and ground level conditions (OGL) before and after the landslide occurred.

In this analysis, two alternative landslide countermeasure scenario are proposed as Alternative 1 and Alternative 2.

In alternative countermeasure 1, the entire colluvial layer on the right side of the main dam abutment is excavated, situated within the core zone. This removal of colluvial material will result in the formation of long slopes in the upstream and downstream sections of the core zone, particularly between STA 0+671 and STA 0+625. To reinforce these slopes at these STAs, soldier piles are necessary. Additionally, strengthening the dam foundation will involve the use of curtain grouting and consolidation grouting, while the spillway foundation will utilize a bore pile foundation, as shown in Figure 5. The longitudinal section and typical cross-section of alternative excavation 1 are illustrated in Figure 6 and Figure 7.

Alternative countermeasure 2 involves not excavating all the colluvial soil in the core zone foundation, resulting in less steep excavation slopes that do not require soldier pile security. Instead, in alternative 2, the core zone rests on the colluvial layer, particularly from the spillway to STA

0+600, necessitating reinforcement to address seepage issues and enhance carrying capacity. This reinforcement involves using secant piles, while consolidation grouting is still conducted to increase the foundation's bearing capacity. However, in this case study, the cost of secant pile reinforcement will be calculated as an additional implementation cost, and no seepage analysis will be performed. The proposed alternative two is depicted in Figure 8. The longitudinal section and typical cross-section of alternative excavation 2 are illustrated in the Figure 9 and Figure 10.

The landslide modeling carried out by Plaxis software. First, select an analysis method. The selection of the analysis method is made at the beginning of creating the worksheet. The analytical methods used in this paper are the Ordinary (Fellenius) and Bishop methods. Second, create the object geometry on the slope/w based on the actual conditions of the slope and soil layers in the field. This object can be created by importing regions from AutoCAD or by importing points for analysis in two dimensions. The object geometry is illustrated in the Figure 11.

Third, the material data that must be input into the Mohr-Coulomb modeling includes several material properties, as shown in Figure 12, namely friction angle (φ), cohesion (c), soil density (γ). This data can be seen in the Table 2. Fourth, input groundwater level data. Modeling should also incorporate the groundwater level condition to account for the influence of pore water pressure. Fifth, calculate safety factors. This involves identifying critical areas in the soil layer structure and comparing the resisting force with the driving force.

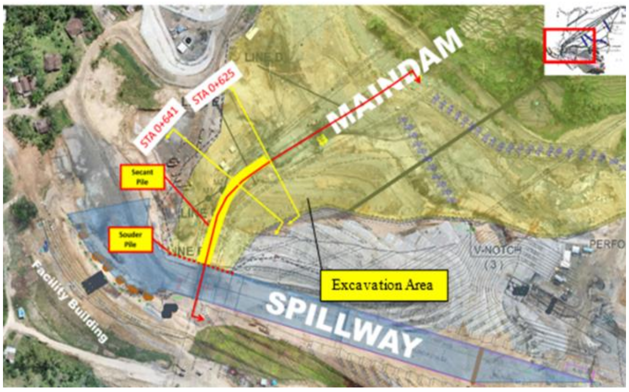

Figure 8. Alternative countermeasure layout 2

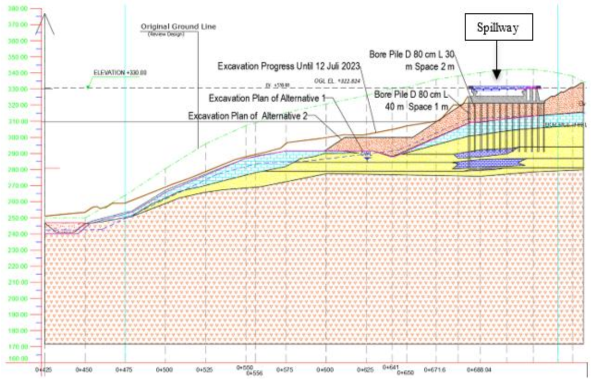

Figure 9. Longitudinal section of Alternative 2

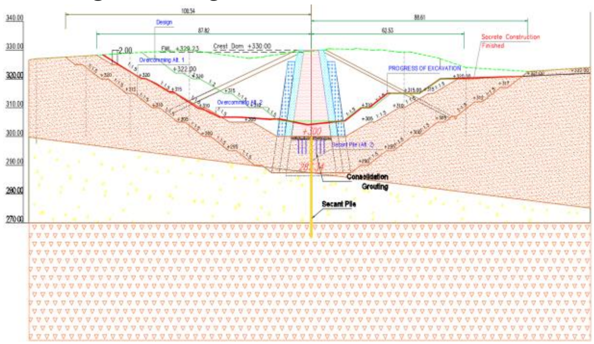

Figure 10. Typical cross section of Alternative 2

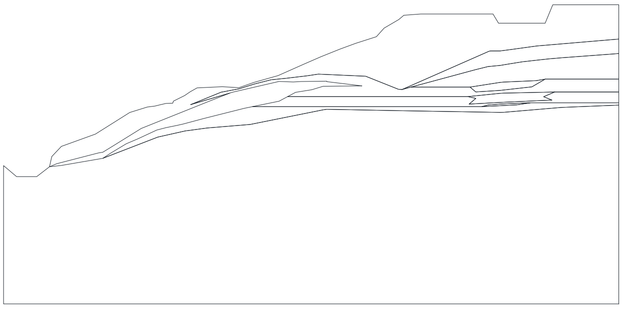

Figure 11. Object geometry

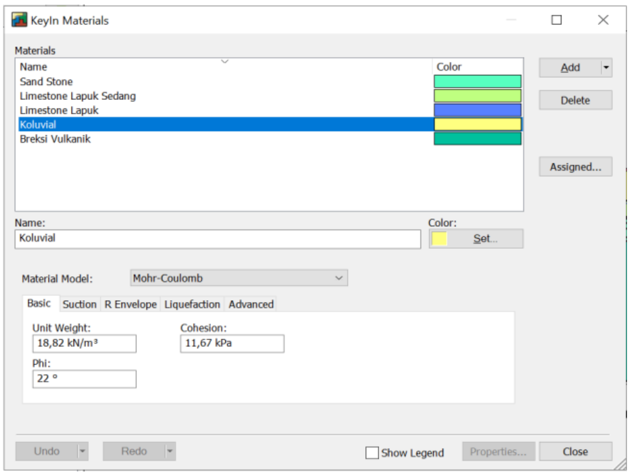

Figure 12. KeyIn materials data

3 Results

3.1 Geological Conditions

On Pedestal Hill, on both the right and left sides of the slope, ancient landslides in the form of colluvial deposits exist. These sedimentary deposits are formed by the weathering of soil and its parent rock (limestone of the Wonosari formation). This condition makes the two supporting hills prone to landslides. The lithological composition of colluvium readily retains rainwater in shallow groundwater aquifers, thus triggering landslides [12].

The results of geoelectric measurements in the main dam landslide area indicate that the residual soil layer (colluvium) is saturated with water and contains limestone rock fragments. Beneath the colluvium layer, interbedded claystone with sandstone is also saturated with water and is affected by seepage occurring in the main dam [12]. A longitudinal section image illustrating the geological condition of the main dam abutment on the right side is provided in Figure 13. Soil parameter data obtained from investigative drill tests yielded the following results, as shown in Table 2.

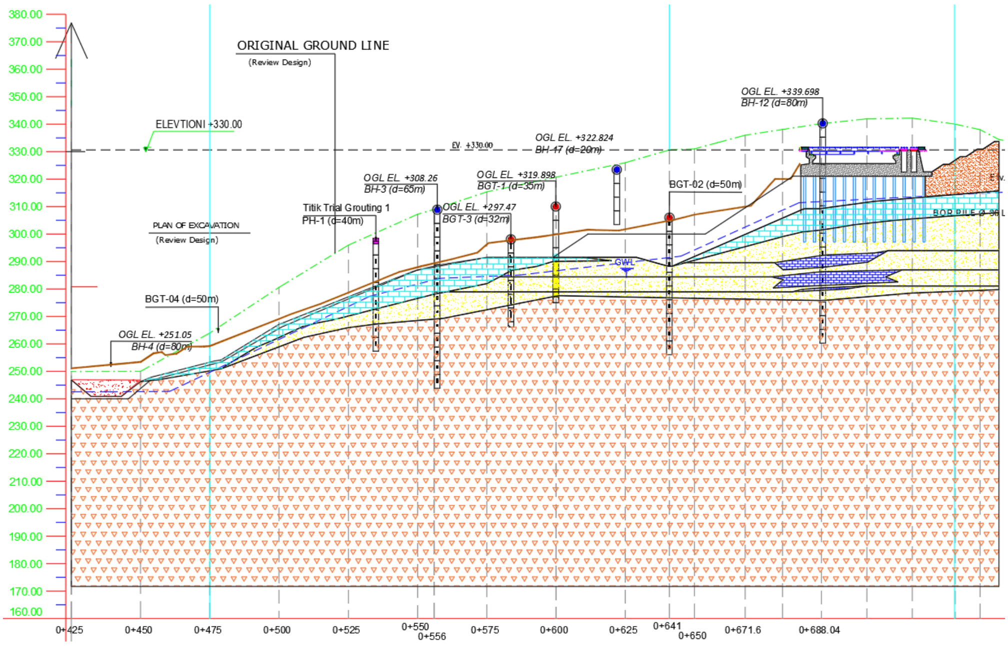

Figure 13. Longitudinal section of right-side abutment geology [13]

Table 2. Landslide parameter data [13]

No.	Soil Parameters	Mark	Unit
A.	Colluvial Soil		
	1. Specific gravity (γ)	18.82	kN/m^3
	2. Internal Shear Angle (φ_{ef})	22	^{0}
	3. Cohesion (c_{ef})	11.67	kPa
B.	Weathered Limestone		
	1. Specific gravity (γ)	22	kN/m^3
	2. Internal Shear Angle (φ_{ef})	23	^{0}
	3. Cohesion (c_{ef})	35.5	kPa
C.	Weathered Limestone		
	1. Specific gravity (γ)	22	kN/m^3
	2. Internal Shear Angle (φ_{ef})	17	^{0}
	3. Cohesion (c_{ef})	35.5	kPa
D.	Sandstone		
	1. Specific gravity (γ)	18.28	kN/m^3
	2. Internal Shear Angle (φ_{ef})	29	^{0}
	3. Cohesion (c_{ef})	27	kPa
E.	Volcanic Breccia		
	1. Specific gravity (γ)	23.64	kN/m^3
	2. Internal Shear Angle (φ_{ef})	49.70	^{0}
	3. Cohesion (c_{ef})	34	kPa

3.2. Existing Slope Stability Analysis

The results of slope stability analysis in existing conditions, using both the Geoslope program and manual calculations, demonstrate suitability. The existing slope is critical for groundwater level conditions based on initial investigations, where the safety factor based on the manual Fellenius method is 1.169 < 1.25 and modeling is 1.17 < 1.25 In calculations using the Bishop Manual method, the safety factor was 1.23 < 1.25, and modeling was 1.24 < 1.25. The slope is unstable after an increase in groundwater levels due to rainwater infiltration, where using the manual Fellenius method, it is 0.987 < 1.07, and modeling is 0.98 < 1.07. The same thing was also shown by calculations using the Bishop method, with manual calculations producing a safety factor of 1.042 <1.07 and modeling 1.044 <1.07.

This is consistent with the previously conducted analysis, which indicated that groundwater levels significantly influence slope stability by affecting the safety factor value. Specifically, higher groundwater levels lead to lower safety factor values [14]. A comparison of safety factors is presented in the Figure 14.

3.3. Analysis of The Alternative

From this design, the acceptable shear capacity can be calculated as Equation 4, Equation 5, and Equation 6.

$$Vn = Vc + Vs \quad (4)$$

$$Vc = \frac{\sqrt{Fc'}}{6} bw.\, d \quad (5)$$

$$Vc = 414{,}712.13\ N$$

$$Vs = \frac{Av \,.\, Fy \,.\, d}{s} \quad (6)$$

$$Vs = 1{,}329{,}727.83\ N$$

$$Vn = 1{,}754.439\ kN$$

The shear force will be used as input data on slope/w as the soldier pile data used. The design soldier pile as shown in Figure 15.

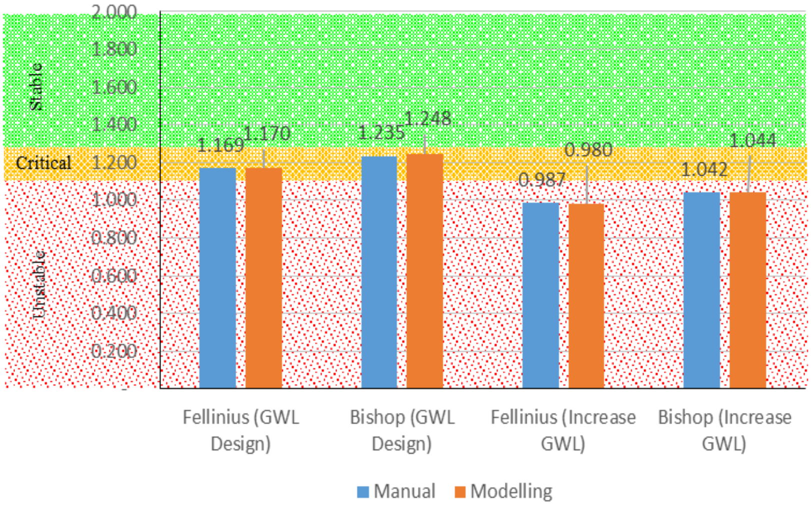

Figure 14. Comparison of existing slope safety

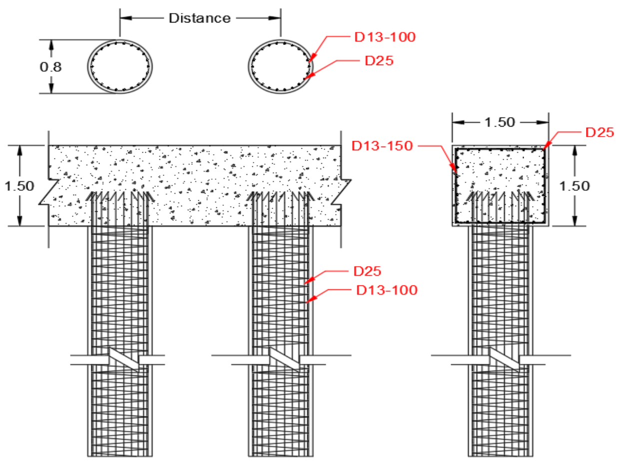

Figure 15. Design soldier pile

The alternatif countermeasure 1. From the results of the slope stability analysis in alternative one countermeasure without soldier pile reinforcement, the slope at STA 0+641 upstream obtained a safety factor of 0.98 <1.07 and downstream of 0.96 <1.07, which shows that the slope is in an unstable condition, so it requires reinforcement with soldier's pile. At STA 0+625, the upstream slope is stable with a safety factor of 1.715 > 1.25, so no strengthening is needed. safety factor number for alternative countermeasure 1 can be seen in the Figure 16. However, the safety factor downstream is 1.134 < 1.25, indicating the slope is in a critical condition, so strengthening is still needed. The results of the slope stability analysis after strengthening according to alternative design 1 showed that at STA 0+641 upstream, the safety factor increased to 1.32 > 1.25 and downstream 1.306 > 1.25, at STA 0+625 downstream 1.338 > 1.25.

The alternatif countermeasure 2. In the landslide prevention analysis presented in alternative two, depicted in Figure 17, there is no need to strengthen the slope in the direction of the dam cross-section. This is because the safety factor for the slope in the direction of the cross-section at STA 0+641 upstream is 1.32 > 1.25 and downstream is 1.366 > 1.25, indicating stable conditions. In the longitudinal section of the dam with bore pile reinforcement and a spillway foundation, the safety factor isis 1.657>1.25. The comprehensive results of the analysis are provided in Table 3.

3.4. Calculation of Implementation Costs.

The calculations of implementation costs for alternative 1 and 2 can be seen in Table 4 and Table 5. Subsequently, the assessment results are presented in Table 6, where alternative 2 is recommended.

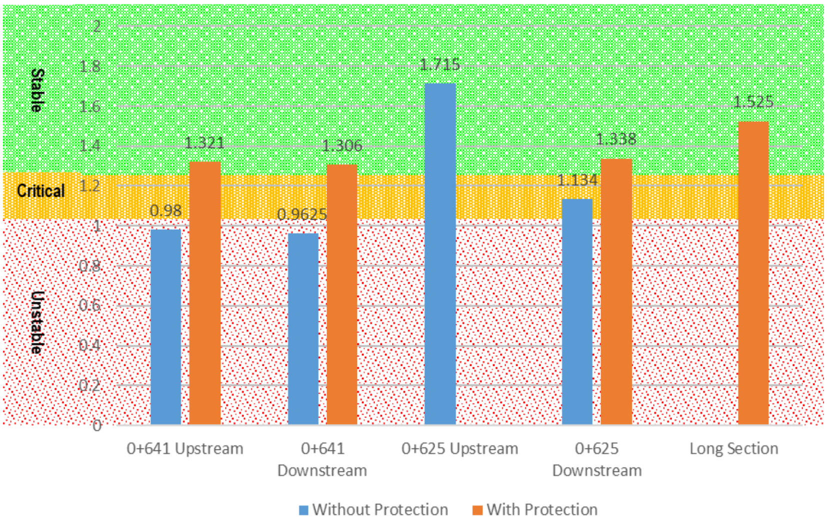

Figure 16. Safety factor number for Alternative Countermeasure 1

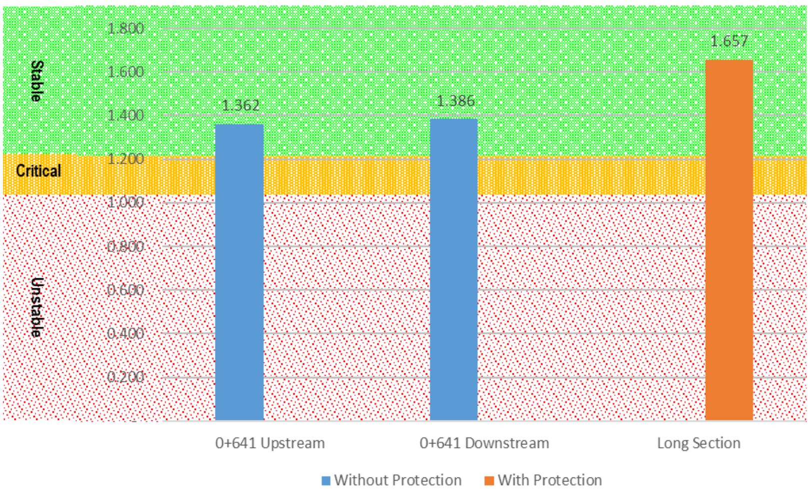

Figure 17. Safety factor number for Alternative Countermeasure 2

Table 3. Results of slope stability analysis

No		Description	Sf	
1	Slope Existing	Long Section Maindam	1.248	<1.25 Critical
		Long Section Maindam	1.170	<1.25 Critical
		Long Section Maindam	1.044	<1.05 Unstable
		Long Section Maindam	0.980	<1.05 Unstable
2	Alt. 1	Long Section Maindam	1.625	>1.25 Stable
		Crossection 0+641 (U/S)	0.980	<1.05 Unstable
		Crossection 0+641 (D/S)	0.965	<1.05 Unstable
		Crossection 0+641 (U/S)	1.321	>1.25 Stable
		Crossection 0+641 (D/S)	1.308	>1.25 Stable
		Crossection 0+625 (U/S)	1.715	>1.25 Stable
		Crossection 0+625 (D/S)	1.134	<1.05 Unstable
		Crossection 0+625 (D/S)	1.338	>1.25 Stable
3	Alt. 2	Long Section Maindam	1.657	>1.25 Stable
		Crossection 0+641 (U/S)	1.362	>1.25 Stable
		Crossection 0+641 (D/S)	1.386	>1.25 Stable

Table 4. Calculation of implementation costs for Alternative 1

No	Items	Unit	Total (Rp)
Alternative 1			
	Additional Items		
A	Excavation Work		
A.1.	Excavation Of Earth	m^3	5,180,391,453.13
B	Embankment Work		
B.1	Embankment Of Core	m^3	2,267,343,168.33
B.2	Filter Embankment	m^3	1,768,656,903.48
B.3	Coarse Filter	m^3	1,142,654,587.28
B.6	Rock Embankment	m^3	12,166,049,441.21
B.7	Rip-Rap Embankment	m^3	460,773,445.95
C	Bore Pile		
C.1	Pengeboran Pondasi Tiang Bor Diameter 80 cm	m'	29,955,095,280.00
C.2	Pembesian	Kg	17,390,432,989.92
C.3	Pengecoran K225	m^3	3,605,186,347.78
Sub Total 1			73,936,583,617.07

Table 5. Calculation of implementation costs for Alternative 2

No	Items	Unit	Total Price (Rp)
Alternative 2			
	Additional Items		
A	Diafragma Wall		
A.1	Earth Drilling	M'	25,023,463,740.00
A.2	Baturock Drilling	M'	7,689,531,205.00
A.3	Plastic Concrete	M^3	7,705,961,289.49
Total Additional Items (Rp)			40,418,956,234.49
	Substraction Items		
	Drilling Dan Grouting Works		
B	Drilling Curtain Grouting Hole		
B.1	Drilling Of Curtain Grouting, Depth 0 M - 10 M	M	285,338,352.00
B.2	Drilling Of Curtain Grouting, Depth 10 M - 20 M	M	322,131,040.00
B.3	Drilling Of Curtain Grouting, Depth 20 M - 30 M	M	345,802,168.00
B.4	Drilling Of Curtain Grouting, Depth > 30 M	M	234,487,323.00
C	Drilling, Untuk Check Hole Dan Pilot Hole, And Core Sampel:		
C.1	Core Drilling Pilot & Check Hole, Depth 0 - 10 M	M	435,381,960.00
C.2	Core Drilling Pilot & Check Hole, Depth 10 - 20 M	M	447,114,200.00
C.3	Core Drilling Pilot & Check Hole, Depth 20 - 30 M	M	320,495,670.00
C.4	Core Drilling Pilot & Check Hole, Depth > 30 M	M	232,320,958.00
D	Grouting (Operational And Materials)	Ton	1,360,868,700.00
E	Lugeon Tes For Pilot Hole Dan Checkhole		
E.1	Water Pressure Test For Curtain Grouting Hole (1 Pressure)	Kali	78,599,700.00
E.2	Water Pressure Test For Pilot & Check Hole (7 Pressure)	Nos	99,110,337.00

No	Items	Unit	Total Price (Rp)
F	Redrilling Untuk Lubang Curtain Dan Sub Curtain Grouting		
F.1	Redrilling For Curtain Grouting Hole, Depth 0 M - 10 M	M	142,669,176.00
F.2	Redrilling For Curtain Grouting Hole, Depth 10 M - 20 M	M	135,013,600.00
F.3	Redrilling For Curtain Grouting Hole, Depth 20 M - 30 M	M	130,811,328.00
F.4	Redrilling For Curtain Grouting Hole, Depth > 30 M	M	60,051,631.50
Total Subtraction Items (Rp)			4,630,196,143.50
Sub Total 2			35,788,760,090.99

Table 6. Multicriteria Analysis of Alternative 1 and Alternative 2

No	Description	(%)	Alt. 1		Alt. 2	
1	Influence on ease of quality control	35%	3.00	105%	2.00	70%
2	Durability	25%	3.00	75%	4.00	100%
3	Influence on implementation costs	20%	2.00	40%	3.00	60%
4	Influence on implementation time	10%	2.00	20%	4.00	40%
5	Ease of Implementation	10%	2.00	20%	3.00	30%
Total		100%		2.60		3.00

*Criteria 1 = Bad
2 = Not Good
3 = Good
4 = Very Good

4 Conclusion

Based on the identification of the landslide cause, it is evident that the main cause is the engineering geological condition of the main dam's abutment on the right side, comprising colluvial deposits formed by the weathering of soil and limestone. The colluvium layer readily retains rainwater in shallow groundwater aquifers, exacerbating landslide risks.

The existing slope stability analysis results, the safety factor was 1.170 < 1.25. Based on analysis using the Bishop method showed a safety factor of 1.248 < 1.25. This condition illustrates that the slope condition is in a critical condition. The increase in ground water level due to rainwater infiltration affects slope stability. Slopes are in critical condition at normal groundwater levels, and slopes are unstable when groundwater levels rise.

The proposed landslide prevention design with alternative 1 produces safety factor in the cross-section direction sta 0+641 upstream 1.321 > 1.25, downstream 1.306 > 1.25 and at sta 0+625 upstream 1.715 > 1.25, downstream 1.338 > 1.25 and longitudinal cuts of 1.525 > 1.25 so that it is safe from landslide hazards. While, proposed landslide prevention design with alternative two at sta 0+641 on the upstream side of 1.362 > 1.25 downstream 1.386 > 1.25 and longitudinal cuts of 1.657 > 1.25 to protect it against landslide hazards. The estimated additional cost with alternative countermeasures is IDR. 73,936,583,617, and the estimated additional cost with alternative two countermeasures is IDR. 35,788,760,090.

Based on multi-criteria decision analysis, alternative 2 is the preferred option.

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