# ROCK SLOPE STABILITY OF PONOROGO - PACITAN ROAD KM 232.5 USING ROCK MASS RATING (RMR) AND ROCPLANE SOFTWARE

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**ABSTRACT**

Geological structures and discontinuities in the rock are weak areas and groundwater infiltration pathways. The existence of geological structures and discontinuities will reduce the level of rock shear strength and the main implication is to increase the chance of landslides. The purpose of this study was to determine the type of sliding on rock slopes, rock mass classification, and to determine the stability of the rock slopes. Kinematic analysis obtained landslide type, rock mass classification analysis including medium rock mass class with an RMR value of 53. Slope stability analysis based on the generalized criteria Hoek & Brown failure obtained a safety factor value of 1.55 with a slope angle of 60°. Slope stability analysis based on Mohr-Coulomb criteria obtained a safety factor value of 1.59 with a slope angle of 70°.

**Keywords**: slope stability, rock mass rating, kinematics analysis, Hoek & Brown failure criteria.

## 1. INTRODUCTION

Ponorogo and Pacitan areas are included in the southern mountain route, regionally a transitional zone located between Mesozoic Paelosubduction (northeast-southwest) and the Tertiary - Resent (east-west) subduction route. The area is also a Tertiary - Resent magmatic path which shifts to the south. The rock structures that develop in the area are folds, faults, and burly. The fold system in this area generally has a fold axis that is relatively west - east or southwest-northeast and develops in the west [1]. Faults are generally downward and shear faults. Shear faults generally have a northwest-southeast and northeast-southwest direction.

The Ponorogo - Pacitan route is known as a deadly route because the road access passes through the edge of a cliff and landslides often occur around the cliffs, especially during the rainy season. This route is one of the points prone to landslides. The paths with the most frequent landslides are Ngreco Village, Tegalombo Village, Pucagombo Village in Tegalombo District, and Kedungbendo Village in Arjosari District.

Rock slope landslides can occur generally influenced by conditions of inconsistency between the existence of the formation process in nature and the bond conditions between rocks that are identified as unconformity. The effect of non-conformity can affect the value of shear strength and the strength of the slope forming material.

Rock collapse usually starts from and follows discontinuities that exist in the rock, such as joints, fractures, bedding planes, faults, and other types of cracks in the rock. According to Hencher (1987), geological structures and discontinuities in rocks are weak areas and groundwater infiltration pathways. The existence of geological structures and discontinuities will reduce the level of rock shear strength and the main implication is to increase the chance of landslides.

The purpose of this study is to know the types of landslides that occur on rock slopes, know the rock mass classification using the Rock Mass Rating method, and know the slope stability at the research location.

## 2. LITERATURE

### 2.1 Types of rock avalanches

To determine the potential failure type in a rock slope cutting activity, it is necessary to map the orientation of the discontinuity before and after the rock slope is exposed [2].

In general, the integration of rock discontinuity orientations will form the main types of landslides / collapse in rocks, namely[3]:

- Plane failure

Plane failure (**Figure 1**) is a rock slide that occurs along the sliding plane which is considered flat. The sliding area can be in the form of fractures, faults or rock layers.

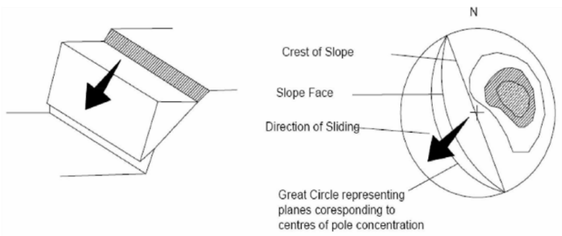


**Figure 1.** Plane failure

- Wedge failure

A wedge failure occurs when two or more weak planes intersect in such a way as to form a wedge against the slope (**Figure 2**). This wedge failure can be divided into 2 types of landslides, namely: single sliding and double sliding. For single landslides, the slide occurs in one plane, while the form of multiple landslides occurs at the intersection of the two planes.

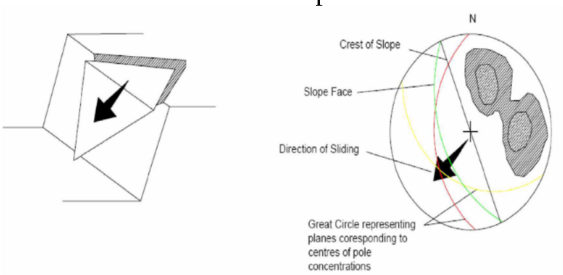


**Figure 2.** Wedge failure

- Toppling failure

Toppling failures generally occur on steep slopes and on massive rock masses where the weak areas are column-shaped. Toppling failure occur because the weak areas on the slope are in the opposite direction to the slope direction. As a result of the opposite tilt direction, the material cannot support its own mass, resulting in toppling failure. The toppling failure is shown in **Figure 3**.

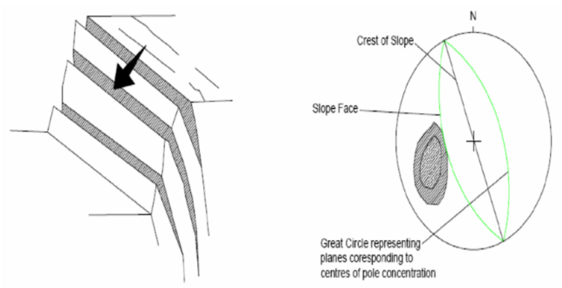


**Figure 3.** Toppling Failure

- Circular failure

Circular failure are the most common landslides in nature, especially in soil and rock materials that have been weathered so that they almost resemble soil. As the name implies, the landslide field is arc shaped (**Figure 4**). In hard rocks, circular failure can only occur if the rock has weathered and has discontinuous areas (fractures) with very tight spacing (very fluffed rock).

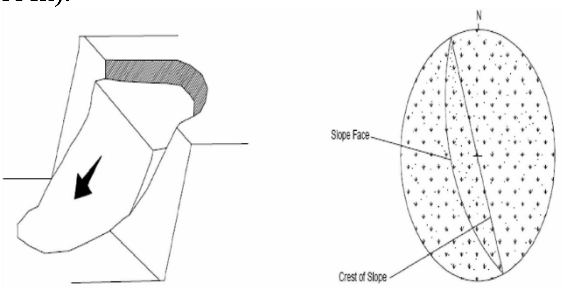


**Figure 4.** Circular failure

### 2.2 Rock Mass Rating (RMR)

Rock Mass Classification is the process of placing rock masses into groups or classes in defined relationships [4] and assigning a unique description (or number) to it based on similar properties / characteristics so that the behavior of rock masses can be predicted. Rock masses are called collections of rock material separated by rock discontinuities, mostly by joints, bed planes, intrusions and embankment faults, etc.

*Rock Mass Rating* (RMR) is a rock mass classification published and is used to determine the stability of rock mass empirically by providing an assessment of rock mass with weight and parameters based on geological conditions. The following five parameters are used to classify rock masses using the RMR system.

a. *Uniaxial compressive strength (UCS)*

The strength of intact rock in RMR is expressed by Uniaxial Compressive Strength (UCS). UCS is the strength of the intact rock obtained from the uniaxial compressive strength test results in the laboratory.

b. *Rock Quality Designation* (RQD)

The presence of discontinuity fields in the rock mass often adversely affects its mechanical properties so that the quantitative magnitude of the discontinuity plane needs to be known. Parameters that can indicate rock quality before excavation are Rock Quality Designation (RQD) developed by Deere (1964), where data is obtained from exploration drilling results in the form of drill cores ranging from BQ, NQ, and HQ.

To quantify the core of the box, the RQD must be calculated. RQD is calculated from the percentage of core drill bits obtained with a minimum length of 10 cm and the number of core cuttings is usually measured at a 2 m long core, an example of RQD calculation of drill core can be seen in Figure 5. Cuts due to drilling handling must be ignored from the calculation and the core a soft and not good drill weight = 0 and the calculation is as follows.

$$\mathrm{RQD} = \frac{\Sigma\ \text{Length of core pieces} \geq 10\ \text{cm}}{\text{Total length of the core (cm)}} \times 100\% \quad (1)$$

If a core drill is not available, the RQD can be calculated indirectly by measuring the orientation and distance between discontinuities in the rock outcrop. Priest & Hudson (1976) proposed an equation to determine the RQD from the line stretch data as follows [5].

$$\mathrm{RQD} = 100\ e^{-0.1\lambda}(0.1\lambda + 1) \quad (2)$$

Where, λ = the ratio of the number of discontinuities per meter.

c. *Discontinuity Distance*

Discontinuity distance is determined from the average distance between rock fractures along the scanline measurement span.

d. *Discontinuity Field Conditions*

Discontinuity conditions are determined from the description of each discontinuity plane, in the form of weathering level, discontinuity plane surface roughness, continuity of the fracture plane, opening width, and discontinuity field fill material.

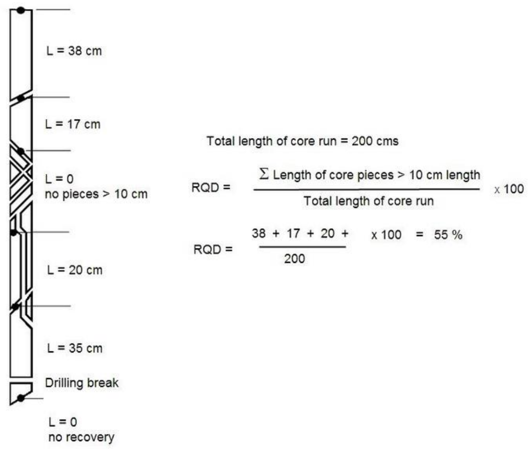


**Figure 5.** RQD Determination of Drill Core Samples

e. *Groundwater Conditions*

The presence of this water will reduce the shear strength between the two discontinuity surfaces. The weight of groundwater parameters can be determined in several ways, one of which is direct observation in the field by determining the general condition of groundwater.

## 2.3 Rock Failure Criteria

Hoek and Brown introduced their failure criteria in an attempt to provide input data for the analyzes necessary for the design of underground excavation in hard rock. These criteria are derived from the results of research into intact rock collapse by Hoek and studies of rock mass behavior model led by Brown.

These criteria start from the properties of the intact rock and then introduce factors to reduce these properties based on discontinuity characteristics in a rock mass. There are 2 rock collapse criteria based on Hoek & Brown's proposal.

1. Generalized Criteria Hoek & Brown Failure

In 1995 Hoek et al included the concept of the Geological Strength Index (GSI) which provides an estimate of the reduction in rock mass strength due to differences in geological conditions. This criterion became known as the Generalized Hoek-Brown criterion with the equation [6]:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(M_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \qquad (3)$$

Where σ1 'and σ3' are the maximum and minimum effective stresses at failure, σci is the compressive strength (UCS) of the intact rock. Meanwhile, Mb is the reduction factor of the rock type constant mi, and s and a are the rock mass constants obtained by the following equation.

$$\mathrm{M_b} = \mathrm{m_i}\exp\left(\frac{GSI-100}{24-14D}\right) \qquad (4)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \qquad (5)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \qquad (6)$$

The GSI value is a rock strength index classification obtained in two ways, the first is by observing the rock classification in the field with the help of the GSI value estimation table based on field observations put forward, the second is by reducing the RMR classification results proposed by Hoek & Brown through the following equation.

$$\mathrm{GSI = RMR - 5} \qquad (7)$$

*2. Mohr-Coulomb Criteria*

Most geotechnical software still adhere to the Mohr-Coulomb collapse criteria, so it is necessary to determine the inner shear angle and the equivalent cohesion value for each rock mass. The adjustment process requires a balance between the upper and lower areas on the Mohr-Coulomb plot, resulting in an equation to obtain the value of the inner shear angle and cohesion:

$$c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma'_{3n}](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+(6am_b(s+m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}} \qquad (8)$$

$$\phi' = \sin^{-1}\left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}}\right] \qquad (9)$$

Determination of the value of σ3max on different slopes and tunnels, for slopes the value of σ3max is obtained based on the formula:

$$\frac{\sigma'_{3max}}{\sigma'_{cm}} = 0.72\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.91} \qquad (10)$$

Where,

$$\sigma'_{cm} = \sigma_{ci}\frac{(mb+4s-a(mb-8s))\left(\frac{mb}{4}+s\right)^{a-1}}{2(1+a)(2+a)} \qquad (11)$$

$$\sigma'_{3n} = \frac{\sigma'_{3max}}{\sigma_{ci}} \qquad (12)$$

## 3. RESEARCH LOCATION AND METHOD

The location of the research was conducted at KM 232.5 on Ponorogo - Pacitan road, Tegalombo district, Pacitan. The research location is shown in **Figure 6**.


**Figure 6.** Location of Research on The Rock Slope of the Ponorogo - Pacitan Road

For this research the steps include:

- Field survey which includes field observations and measurement of the orientation of the field structure.
- Data collection techniques, data collected includes primary data and secondary data.
- Data analysis techniques, the data analysis process is carried out based on field and secondary data.
- Lab tests, lab tests that will be carried out include testing the physical properties of rocks, and uniaxial rocks.
- Kinematic analysis is in the form of stereographic analysis to analyze the types of landslides that occur.
- Rock mass classification analysis using the Rock Mass Rating (RMR) method.
- Landslide stability analysis, in this research modeling will be used the Limit Equilibrium Method with the help of software Rocplane.

## 4. RESULTS AND DISCUSSION

### 4.1 Geology

Based on the geological map of the Pacitan sheet (**Figure 7**) [7], the research location is around the Tegalombo fault with Watupatok formation units in the form of inserts of sandstones and claystones, and breakthrough rock units in the form of andesite.

Andesite rocks are structured with a gray brown color and porifiritic texture, which consists of orthoclase, quartz, ore minerals in the plagioclase period. These andesite rocks then undergo alteration caused by the hydrothermal activity of minerals present in the rock. After experiencing alteration, there is a change in color, namely reddish brown.

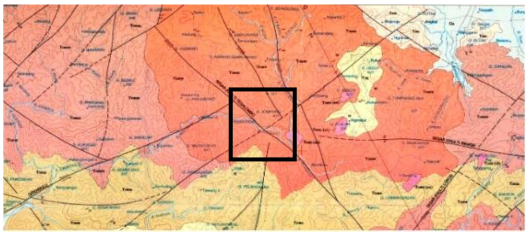

**Figure 7.** Geological Map of Research Location KM 232.5

## 4.2 Identification of Type of Slides

To identify the types of landslides that occurred on the slopes of KM 232.5 Ponorogo - Pacitan road, an observational survey was carried out and measurement of the orientation of the rock structure on the slope. The orientation measurement was carried out at a span of 4 meters, and the data obtained was in the form of a strike dip slope N154°E / 66° and the strike dip of the rock solid structure are shown in **Table 1**.

**Table 1.** Data for Measuring the Orientation of The Rock Solid Structure

| Joint | Strike | Dip | Distance (m) |
|---|---|---|---|
| 1 | 134 | 49 | 0 |
| 2 | 160 | 25 | 0.2 |
| 3 | 120 | 54 | 0.5 |
| 4 | 340 | 28 | 0.5 |
| 5 | 165 | 36 | 0.9 |
| 6 | 130 | 51 | 1.2 |
| 7 | 140 | 49 | 1.6 |
| 8 | 118 | 54 | 1.8 |
| 9 | 140 | 29 | 1.9 |
| 10 | 129 | 29 | 2.1 |
| 11 | 100 | 44 | 2.4 |
| 12 | 135 | 50 | 2.4 |
| 13 | 130 | 50 | 2.7 |
| 14 | 143 | 42 | 2.9 |
| 15 | 154 | 34 | 3 |
| 16 | 115 | 45 | 3.2 |
| 17 | 145 | 55 | 3.5 |
| 18 | 150 | 48 | 3.5 |
| 19 | 143 | 45 | 3.8 |
| 20 | 134 | 65 | 4 |

From the data obtained, a kinematic analysis was carried out in the form of a stereographic method using Dips software to determine the type of landslide. The results of stereographic analysis can be seen in **Figure 8**.

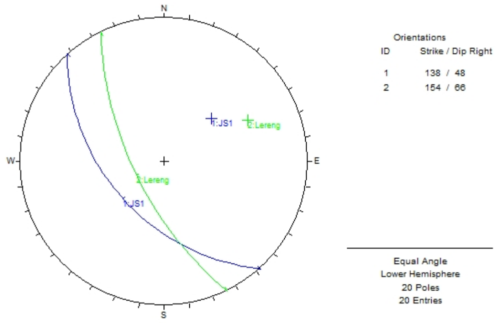


**Figure 8.** The Results of The Joint Projection Stereographic Analysis Using the Dips Software

From the picture above, it can be said that the type of landslide that occurred was planar. This is because there is only 1 joint set, where the plane of the joint set 1 is almost parallel to the slope and the joint set 1 is in the same direction as the slope, and the difference in the strike of the two planes is less than 20°.

## 4.3 Rock Mass Classification

Rock mass classification for the slope of KM 232.5 is calculated and analyzed the Rock Mass Rating using five parameters [3] which include intact rock strength, rock quality designation (RQD), distance between discontinuities, discontinuity field conditions, groundwater.

*a. Intact rock strength*

The intact rock strength parameters were carried out using laboratory testing, namely the Uniaxial Compressive Strength (UCS) test. In this case the rock type is andesite, the samples taken were tested by UCS, so the rock compressive strength for andesite rock types was 6.5 Mpa. So that the strength of the intact rock based on **Table 2** is obtained at 2.

**Table 2.** Weighted Intact Rock Strength

| Parameter | | Value Hose | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Intact rock strength | **PLI (Mpa)** | > 10 | 10-4 | 4-2 | 2-1 | For low compressive strength, UCS is needed | | |
| | **UCS (Mpa)** | > 250 | 100-250 | 50-100 | 25-50 | 25-5 | 5-1 | <1 |
| **Weight** | | 15 | 12 | 7 | 4 | **2** | 1 | 0 |

*b. RQD*

The calculation of RQD is carried out by direct observation in the field by looking at the muscular structure on the slope. Observations were made on a slope span of 4 meters, in which observations obtained a total number of fractures along the length of 20 fractures, then the ratio of the average number of fractures (λ) can be calculated.

$$\lambda = \frac{20}{4} = 5 \text{ joint / meter}$$

Then the RQD can be calculated as follows.

RQD = 100e-0.1 × λ (0.1λ + 1)
= 100 e (-0.1 × 5) × (0.1 × 5 + 1)
= 90.87%

From the above calculations, the RQD weighting can be found using Table 3.With the RQD calculation value = 90.87%, the RQD weighting value is 20.

**Table 3.** RQD Weighting

| Parameter | Value Hose | | | | |
|---|---|---|---|---|---|
| RQD (%) | 90 - 100 | 75 - 90 | 50 - 75 | 25 - 50 | <25 |
| **Weight** | **20** | 17 | 13 | 8 | 3 |

*c. Distance Between Discontinuities*

Determination of the distance between discontinuities was carried out using the scanline method, namely measuring the orientation of the rock mass in the field. Based on the measurement data of rock mass orientation, the average distance between discontinuities is 20 cm. Based on Table 4, it is known that the weight of the distance between discontinuities on the slope is 8.

**Table 4.** Weighted Distance Between Discontinuity

| Parameter | Value Hose | | | | |
|---|---|---|---|---|---|
| Discontinuity distance | > 2 m | 0.6 - 2 m | 0.2 - 0.6 m | 0.06 - 0.2 m | <0.06 m |
| **Weight** | 20 | 15 | 10 | **8** | 3 |

*d. Discontinuity Field Conditions*

Based on field observations, the condition of the discontinuity plane is rather rough with the separation of discontinuities less than 1 mm and the rock conditions are very weathered. Based on **Table 5**, it is known that the weight of the discontinuity conditions on the slope is 13.

**Table 5.** Weighting of The Field Conditions of The Discontinuity

| Parameter | Value Hose | | | | |
|---|---|---|---|---|---|
| Discontinuity condition | Very rough, not continuous, no separation, the stone walls are not weathered | Slightly coarse, separation <1 mm, walls slightly weathered | Slightly rough, <1 mm separation, very weathered walls | Slicken sided / gouge thickness <5 mm, or separation of 1 - 5 mm, continuous | Soft gouges> 5 mm thick, or> 5 mm separation, continuous |
| **Weight** | 20 | 17 | **13** | 8 | 3 |

*e. Groundwater Conditions*

Groundwater parameters are directly observed in the field by observing the general condition that the slopes have found conditions on the slopes, namely humid. From the observation at the location, the weighting of groundwater conditions on the slope can be found, from **Table 6** it is known that the weight of groundwater conditions on the slope of KM 232.5 is 10.

**Table 6.** Weighting of Groundwater Conditions

| Parameter | | Value Hose | | | | |
|---|---|---|---|---|---|---|
| Groundwater | Flow / 10 m tunnel length (lt / min) | None | <10 | 25 - 10 | 25 - 125 | > 125 |
| | Mask stock pressure σ1 | 0 | <0.1 | 0.1 - 0.2 | 0.2 - 0.5 | > 0.5 |
| | General condition | Dry | Moist | Wet | Dripping | It flows |
| **Weight** | | 15 | **10** | 7 | 4 | 0 |

f. RMR total weight classification

Each result of the weighting of the five parameters that has been obtained is then added up to get the total RMR weight value that will be used to classify the rock mass class.

RMR = compressive strength weight of intact rock + weight of RQD + weight of fracture distance + weight of stocking condition + weight of groundwater

RMR = 2 + 20 + 8 + 13 + 10 = 53

Based on **Table 7**, the RMR weight value is 53. Then the rocks on the slope of KM 232.5 are included in the classification of class III rock masses (medium rocks). It can be said that the rock slopes belonging to the class III rock mass class are susceptible to weathering so they are prone to landslides.

**Table 7.** Rock Mass Classification is Based on The Total RMR Weighted Value

| **RMR weight** | 100 - 81 | 80 - 61 | **60 - 41** | 40 - 21 | <20 |
|---|---|---|---|---|---|
| **Class** | I | II | **III** | IV | V |
| **Description** | Very good rock | Good rock | **Medium rock** | Bad rock | Rocks Are Very Bad |

## 4.4 Slope Stability Analysis

Slope stability analysis is calculated based on 2 rock failure criteria, namely Hoek & Brown's general criteria and Mohr-Coulomb criteria.

*a. Hoek & Brown Failure*

The general criteria for Hoek & Brown collapse include several parameters, namely rock compressive strength ($\sigma ci$) from the compressive strength test results, the GSI value obtained from the RMR reduction (Bieniawski, 1989), rock mass constant (mi) based on rock type, and the value of the disturbance factor. (D) based on stress relaxation.

The GSI value can be determined from the reduction in the weighting result of the RMR classification according to Bieniawski (1989), namely GSI = RMR - 5. Previously, the RMR was obtained at 53, then the GSI value was obtained:

GSI = 53 - 5 = 48

As for several other parameters, namely Mb, s, and a which are calculated based on the value of mi, GSI and the disturbance factor (D) using the formula proposed by Hoek & Brown as follows.

$$\mathrm{M_b} = \mathrm{m_i}\exp\left(\frac{GSI-100}{24-14D}\right)$$

$$\mathrm{M_b} = 25 \times \exp\left(\frac{48-100}{28-14(1)}\right) = 0.609$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right)$$

$$s = \exp\left(\frac{48-100}{9-3(1)}\right) = 0.0002$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{48}{15}} - e^{-\frac{20}{3}}\right) = 0.507$$

The data is inputted into the Rockplane software, and then Slope modeling performed with Rockplane software. The results of slope stability analysis with Rocplane software are shown in **Figure 9**.

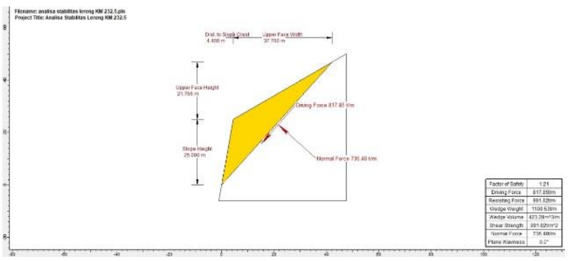

**Figure 9.** The Results of The Analysis of The Slope Stability of KM 232.5

From the results of the analysis, the safety factor of slope stability is 1.21, it can be said that the slope is unstable. This is because the slope SF value is smaller than the standard or does not meet the specified standards, namely SF ≥ 1.5.

To make the slope stable, it is necessary to make changes to the slope angle by reducing the slope angle. Changes in the slope angle of the slope are made with several variations in the angle until the slope becomes stable, with the specified slope angle variation of 75°, 70°, 65°, 60°, and 55°. The results of the analysis of the safety factor of slope stability based on variations in the slope are shown in **Table 8**.

**Table 8.** The Results of The Value of The Safety Factor Based On Variations In The Angle of The Slope

| Slope (°) | Safety factor | Information |
|---|---|---|
| 80 | 1.21 | Not safe |
| 75 | 1.26 | Not safe |
| 70 | 1.33 | Not safe |
| 65 | 1.42 | Not safe |
| 60 | 1.55 | Safe |
| 55 | 1.79 | safe |

Based on these results, if the slope angle is reduced or smaller than the actual slope angle, the slope will approach a stable condition. At the specified slope angle variation, the slope becomes stable at an angle of 60° and 55° with the value of the safety factor that meets the standard, namely SF = 1.55 for the slope angle of 60° and SF = 1.79 for the slope angle of 55°.

b. *Mohr-Coulomb Criteria*

The Mohr-Coulomb criterion parameters used in the slope stability analysis are cohesion (c ') and inner shear angle (ϕ') determined by the equations of Hoek & Brown. With several parameters from the general Hoek & Brown collapse criteria used in determining the cohesion value (c ') and the inner shear angle (ϕ'), can be calculated as follows.

$$\sigma'_{cm} = \sigma_{ci}\frac{\left(mb + 4s - a(mb - 8s)\right)\left(\frac{mb}{4} + s\right)^{a-1}}{2(1+a)(2+a)}$$

$$\sigma'_{cm} = 6.5\,\frac{\left(0.609 + 4(0.0002) - 0.507\left(0.609 - 8(0.0002)\right)\right)\left(\frac{0.609}{4} + 0.0002\right)^{0.507-1}}{2(1+0.507)(2+0.507)}$$

$$\sigma'_{cm} = 0.657\ MPa$$

$$\frac{\sigma'_{3max}}{\sigma'_{cm}} = 0.72\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.91}$$

$\gamma_{andesite}$ = 0.026 MN /m$^3$ and the slope height (H) = 25 m.

$$\frac{\sigma'_{3max}}{0.657} = 0.72\left(\frac{0.657}{0.026 \times 25}\right)^{-0.91}$$

$$\sigma'_{3max} = 0.468\ MPa$$

$$\sigma'_{3n} = \frac{\sigma'_{3max}}{\sigma_{ci}}$$

$$\sigma'_{3n} = \frac{0.486}{6.5} = 0.072\ MPa$$

$$\phi' = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma'_{3n})^{a-1}}\right]$$

$$\phi' = \sin^{-1}\left[\frac{6 \times 0.507 \times 25(0.0002 + 25 \times 0.072)^{0.507-1}}{2(1+0.507)(2+0.507) + 6 \times 0.507 \times 25(0.0002 + 25 \times 0.072)^{0.507-1}}\right]$$

$$\phi' = 32.25°$$

$$c' = \frac{\sigma_{ci}[(1+2a)s + (1-a)m_b\sigma'_{3n}](s + m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}}$$

$$c' = \frac{6.5\left[\left(1 + 2(0.507)\right) \times 0.0002 + (1 - 0.507) \times 0.609 \times 0.072\right](0.0002 + 0.609 \times 0.072)^{0.507-1}}{(1+0.507) \times (2+0.507)\sqrt{1 + (6 \times 0.507 \times 0.609 \times (0.0002 + 0.609 \times 0.072)^{0.507-1})/((1+0.507)(2+0.507))}}$$

$$c' = 0.098\ MPa$$

The value of cohesion (c ') and the inner angle of shear (ϕ') is then inputted into the Rocplane software for slope stability stability analysis. The results of the slope stability analysis are shown in **Figure 10**.

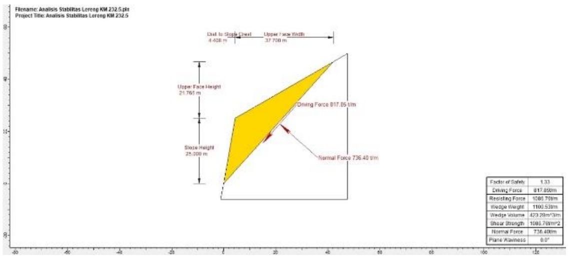

**Figure 10.** The Results of The Analysis of The Slope Stability of KM 232.5

From the results of the analysis, the safety factor of slope stability is 1.33, it can be said that the slope is unstable. This is because the slope SF value is smaller than the standard or does not meet the specified standards, namely SF ≥ 1.5.

To make the slope stable, it is necessary to make changes to the slope angle by reducing the slope angle. Changes in the slope angle of the slope are made with several variations in the angle until the slope becomes stable, with the specified slope angle variation of 75°, 70°, 65°, 60°, and 55°. The results of the analysis of the safety factor of slope stability based on variations in the slope of the slope are shown in **Table 9.**

**Table 9.** The Results Of The Value Of The Safety Factor Based On Variations In The Angle Of The Slope

| Slope (°) | Safety factor | Information |
|---|---|---|
| 80 | 1.33 | Not safe |
| 75 | 1.44 | Not safe |
| 70 | 1.59 | Safe |
| 65 | 1.83 | Safe |
| 60 | 2.26 | Safe |
| 55 | 3.30 | Safe |

Based on these results, if the slope angle is reduced or smaller than the actual slope angle, the slope will approach a stable condition. At the specified slope angle variations, the slope becomes stable at an angle of 70° and 65° with the value of the safety factor that meets the standard, namely SF = 1.59 for the slope angle of 65° and SF = 1.83 for slope angle of 65°.

From the two methods used in analyzing the safety factor of slope stability, namely the Hoek & Brown collapse general criteria and the Mohr-Coulomb criterion, different safety factor values were obtained. The value of the safety factor of the two methods is shown in Table 10.

Based on the table above, for the general criteria for Hoek & Brown failure the slope is said to be stable on a slope of 60° with the SF value = 1.55, while for the Mohr-Coulomb criteria the slope is said to be stable at a slope of 70° with the SF value = 1.59. With reference to the rock mass classification, the result of the safety factor is taken based on the general criteria for Hoek & Brown collapse with a slope angle of 60°.

**Table 10.** The Value Of The Safety Factor Is Based On The General Criteria For The Failure Of Hoek & Brown And The Value Of The Safety Factor Is Based On The Mohr-Coulomb Criteria

| Slope (°) | SF Hoek & Brown's general criteria | SF Mohr-Coulomb criteria |
|---|---|---|
| 80 | 1.21 | 1.33 |
| 75 | 1.26 | 1.44 |
| 70 | 1.33 | 1.59 |
| 65 | 1.42 | 1.83 |
| 60 | 1.55 | 2.26 |
| 55 | 1.79 | 3.30 |

## 5. CONCLUSIONS AND SUGGESTIONS

### 5.1 Conclusion

1. Identification of the type of landslide by using stereographic kinematic analysis, landslides that occur on the slopes of KM 232.5 are in the form of planar landslides.
2. Classification of rock mass class on the slope of KM 232.5 based on the classification analysis of rock mass RMR is known to be a medium rock class with a weight value of RMR is 55.
3. Analysis of the safety factor of slope stability based on the parameters of Hoek & Brown's failure criteria in the Rocplane software, the safety factor value of SF = 1.55 with a slope angle of 60°. Meanwhile, based on the Mohr-Coulomb criteria, the SF value = 1.59 with a slope angle of 70°.

### 5.2 Suggestion

1. More detailed geological research is needed on the structure and classification of rock masses in order to obtain more specific results related to rock mass classification.
2. Periodic monitoring of slopes is necessary to detect any movement or fracture in the rock that may occur, so that if there is a symptom of instability, preventive efforts can be made immediately.

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