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EVALUATION OF ROCK SLOPE USING Q-SLOPE, LIMIT EQUILIBRIUM AND FAILURE PROBABILITY AT ANDESITE MINE OF SIDOMULYO VILLAGE

EVALUASI KESTABILAN LERENG BATUAN MENGGUNAKAN Q-SLOPE*, KESETIMBANGAN BATAS DAN PROBABILITAS LONGSOR PADA TAMBANG ANDESIT DESA SIDOMULYO*

WAHYU N. AKBAR and BAGUS WIYONO
Mining Engineering Department, University of National Development 'Veteran' Yogyakarta
Jl. Padjajaran, Sleman, Yogyakarta, Indonesia. 55283
Telp. (0274) 486733
e-mail: wahyunusantaraakbar@gmail.com

ABSTRACT

For an open pit mine, the rock slope stability is one of the major significant challenges at every stage in the operation. It became a concern from the planning until the mining closure. Mining activities in the research location have entered the mining closure phase and produced the final slope that consists of 4 single slopes with an overall slope height of 65m and an angle of 62° that its stability is not yet known. The actual overall slope has discontinuities which affect the potential for failure. Most of the methods used in geotechnical practice for estimating slope stability are based on the traditional limit equilibrium methods. On the other side, very few empirical techniques exist to assess the slope stability. The empirical method of the Q-Slope is a relatively new methodology for assessing the slope stability in terrains built from rock masses. This method was developed over the last decade by Barton and Bar (2015), with modifications to the original Q-System for application in rock slope stability through the parameter of RQD, Jn, Jr, Ja, O-Factor, Jwice, SRFa, SRFb and SRFc. The stability analysis by Q-Slope method has resulted the slope in stable condition because the value of $\beta_{Q\text{-}Slope} > \beta_{Slope}$. The factor of safety limit equilibrium method and probability of failure used the actual geometry and Q-Slope geometry is known in stable condition because it fulfils acceptance criteria with FoS ≥ 1.1 and PoF ≤ 37,5% according to the regulation Ministry of Energy and Mineral Resources Decree 1827/K/30/MEM/2018.

Keywords: open pit, slope stability, Q-Slope, limit equilibrium, probability of failure.

ABSTRAK

Pada tambang terbuka, kestabilan lereng menjadi salah satu tantangan utama pada setiap tahapan dalam operasional pertambangan yang menjadi perhatian mulai dari tahap perencanaan hingga tahap penutupan tambang. Kegiatan pertambangan di lokasi penelitian sudah memasuki tahap penutupan tambang dan menghasilkan lereng akhir yang terdiri dari 4 lereng tunggal dengan tinggi lereng keseluruhan 65m dan sudut 62° dengan kondisi yang belum diketahui nilai tingkat stabilitasnya. Lereng keseluruhan aktual memiliki bidang diskontinu sehingga dapat mempengaruhi potensi adanya longsor. Pada umumnya metode yang diaplikasikan dalam kajian geoteknik untuk mengetahui kestabilan lereng didasarkan pada metode kesetimbangan batas. Di sisi lain, masih kurangnya metode empiris yang diaplikasikan untuk penilaian kestabilan lereng tambang. Metode empiris Q-Slope *adalah metode baru yang diaplikasikan untuk penilaian kestabilan lereng pada massa batuan. Metode baru ini dikembangkan oleh Barton dan Bar (2015) yang merupakan modifikasi dari metode* Q-System *yang diaplikasikan untuk kestabilan lereng melalui parameter RQD, Jn, Jr, Ja,* O-Factor*, Jwice, SRFa, SRFb dan SRFc. Hasil analisis kestabilan lereng dengan metode* Q-Slope *diketahui bahwa lereng dalam keadaan stabil karena nilai* $\beta_{Q\text{-}Slope} > \beta_{Lereng}$*. Tingkat faktor keamanan (FK) metode kesetimbangan batas dan probabilitas longsor (PL) pada setiap lereng tunggal menggunakan geometri lereng aktual*

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*dan geometri lereng Q-*Slope *diketahui dalam keadaan yang stabil karena memenuhi kriteria penerimaan dengan nilai FK ≥ 1,1 dan PL ≤ 37,5% sesuai dengan peraturan pada Keputusan Menteri Energi dan Sumber Daya Mineral nomor 1827 K/30/MEM/2018.*

*Kata kunci: tambang terbuka, kestabilan lereng, Q-*Slope*, kesetimbangan batas, probabilitas longsor.*

INTRODUCTION

Analysis of rock slope stability are routinely carried out in many engineering studies, such as mining, especially in geotechnical projects, and important issue in both civil and mining engineering (Wyllie, 2018). The main objectives of the stability analysis are the determination of rock slope stability conditions, study of potential failure mechanisms, determination of the factors that affect the slope stability, and performing the optimum and safe slope designs (Gurocak *et al.*, 2017). Stability analysis and providing appropriate support systems for controlling the instabilities in the discontinuous rock slope is the main task for geo-engineers that were faced with different geotechnical projects (Azarafza, *et al.*, 2020). Assessment of slope stability in such rock mass media is a very complex task, where the failures although can occur globally affecting few berms or the entire cut, most often have local character (plane failure, wedging, local toppling) (Janevski and Jovanovski, 2021).

Various methods such as kinematic, limit equilibrium and numerical analysis are used today for evaluating slope stability of the rock. On the other hand, the Q-Slope is a relatively new empirical rock slope engineering method for assessing the stability. Such the method have initially been developed by Barton and Bar (2015) and used for quick access to rock slope stability by making optimal adjustments to slope angles as rock mass conditions. In the meantime, methodologies that allow for quick analysis with low assumptions have always been considered by professionals especially the empirical geomechanical classification methods especially rock mass rating (SMR) and Q-Slope systems were found by the researchers as a flexible procedure to achieve a suitable process in rock slope instabilities (Jorda-Bordehore *et al.*, 2018). The Q-Slope used Q-System parameters for slope stability assessments, some of which were modified by making the slope. Empirical method Q-Slope utilizes parameters RQD (*rock quality designation*), Jn (joint set number), Jr (joint roughness number), Ja (joint alteration number), Jwice (environmental and geological condition number), SRF (s*tress reduction factor*) and additional parameter O-factor (discontinuity orientation factor) based on the potential of failure (Barton and Bar, 2015). Q-Slope was used to both benefits, primarily as flexible empirical approaches to rock mass quantifications and investigate the various issues of the discontinuous to provide a suitable description in design applications (Azarafza, *et al.*, 2020). Generally, application of the geomechanical classifications for primary slope stability assessment and suggesting the in-situ supporting system can be helpful to prevent the first-time rock failures in different excavation operations (Kumar *et al.*, 2019).

Geological condition of the volcanic rocks in Kulon Progo are Old Andesite Formations and are of Oligo-Miocene in age (van Bemmelen, 1949). The mining activities in the research location (Figure 1), composed of andesite rock and have entered the mining closure phase. The final slope consisted of 4 single slopes. The overall slope is 65m in height with slope angle of 62°, its stability is not yet known. The actual overall slope has discontinuities that affect the potential of failure based on the orientation of the discontinuities, the orientation of slope and the friction angle of the rock properties as determined by kinematical analysis. Obtaining the data for these cases required detailed field surveys which are implemented by the ISRM instructions and scan-line procedure (Azarafza, Asghari-Kaljahi and Akgün, 2017).

The studies of this research have been carried out in three stages. In the first stage, the orientation of the discontinuity and the Q-Slope parameter have been determined at the site investigation. In the second stage, the results obtained from the field and the laboratory testing were collected to be the parameter of stability analysis. In the final stages, assessment and analysis of rock stability through the method and results of the Q-Slope are compared to the results obtained with the Limit Equilibrium Method to have data for appropriate comparison and engineering analysis. The limit equilibrium and the Q-Slope methods are applicable at terrains built from weathered and tectonically disturbed schists because the stability conditions on site were confirmed with these two methodologically different approaches (Janevski and Jovanovski, 2021).

METHOD

Orientation of Discontinuities

Orientation of discontinuities and shear strength are two parameters that play important roles in the development of instabilities that develop due to the discontinuity surfaces, these two parameters are used as input parameters in rock slope analysis such as kinematic analysis, limit equilibrium analysis (Gurocak *et al.*, 2017).

Orientation, or attitude of a discontinuity in space, is described by the dip of the line of maximum declination on the discontinuity surface measured from the horizontal, and the dip direction or azimuth of this line, measured clockwise from true north (Zhang, 2016). The measurements taken directly from the rock mass using a geologist compass are evaluated via the contour diagrams prepared to test this method using the stereographic projection method and the main orientations of the discontinuity sets contained within the rock mass are determined (Gurocak *et al.*, 2017).

Mapping techniques in the field used for detailed structural data have been used in mining and civil engineering for many years and have been well documented by some authors such as Priest and Hudson (1981); Windsor and Thompson (1997); Harries (2001) and Brown (2003) in Read and Stacey (2009). Scanline mapping involves measuring and recording the attributes of all structures that intersect a given sampling line. The measurement (Figure 2) observable structures in the outcrop or slope face are shown to the left and the structures selected for mapping are shown to the right (Read and Stacey, 2009).

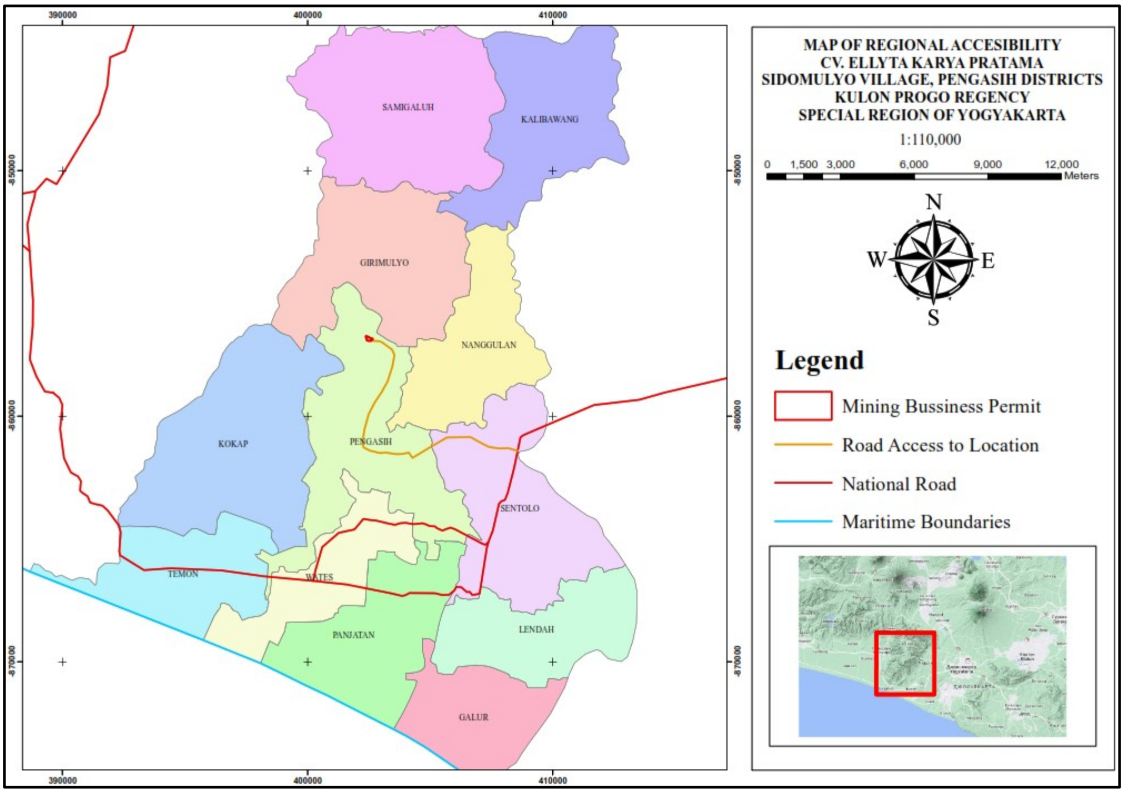
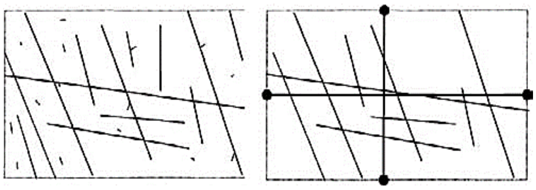


Figure 1. Research area is located in Andesite Mine, Sidomulyo Village, Pengasih District, Kulon Progo Regency, Yogyakarta Province



Source: Read and Stacey (2009)

Figure 2. Scanline mapping technique

Rock Properties

In an open pit slope engineering, the most commonly used defect properties are the shear parameters of the defect. The geomechanical properties of the intact rock that occur between the structural defects in a typical rock mass are measured in the laboratory from representative samples of the intact rock (Read and Stacey, 2009).

Rock mechanics laboratory tests are performed to determine a physical and mechanical property of the intact rock or the discontinuity, the property determined by the test is generally used for (Feng, 2017):

- classification and characterization of intact rock test (unit weight parameter and UCS parameter)
- rock engineering design test (UCS parameter and shear strength parameter)

In open pit mines the most commonly used rock properties are the following suggested methods for rock testing by ISRM (1981) in Read and Stacey (2009). The following equation and graphical concept of mechanical properties can be seen in Figure 3 and Figure 4.

$$\gamma = \frac{W}{V_T} \quad \text{...(1)}$$

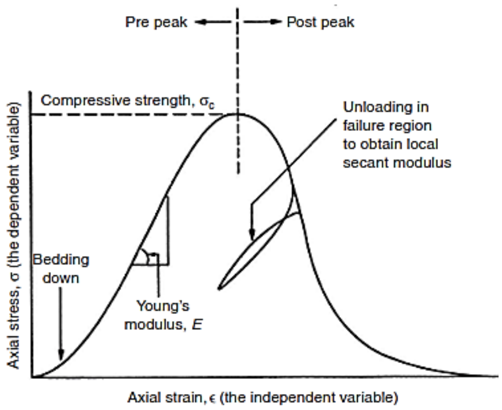
Where, γ = unit weight (kN/m^3), W = weight (kN) and V_T = total volume (m^3)

$$\sigma_c = \frac{P}{A} \quad \text{...(2)}$$

Where, σ_c = UCS (MPa), P = load (kN) and A = cross-sectional area (m^2)

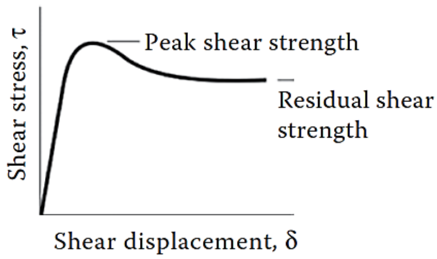
$$\tau = c + \sigma \tan \phi \quad \text{...(3)}$$

Where, τ = shear strength, σ = normal stress, c = cohesion and ϕ = friction angle.



Source: ISRM (2007) in Feng (2017)

Figure 3. UCS curve test



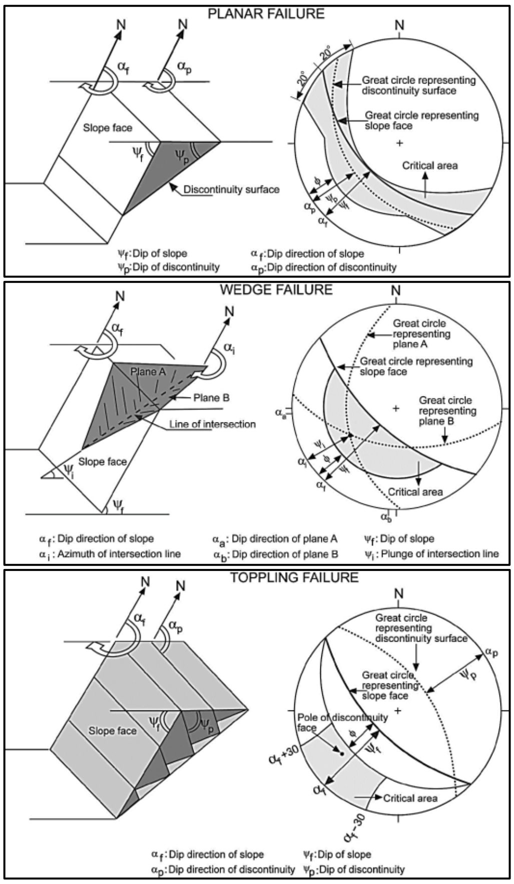
Source: Wyllie (2018)

Figure 4. Shear strength curve test

Kinematical Analysis

When the failures in the rock slopes are considered, it is observed that they occur mostly due to structural elements in the rock mass that are defined as discontinuities (Feng, 2017). The kinematic analysis method has been well documented by some authors such as Hoek and Bray (1981); Goodman (1989); Wyllie and Mah (2004) and it is used to evaluate stability analysis for failures controlled by discontinuities such as planar, wedge and toppling by taking into consideration the discontinuity orientations, slope direction and the internal friction angles of discontinuity surfaces (Gurocak *et al.*, 2017).

The main advantages of the kinematic analysis method are that the parameters used in the analysis are easy to determine, it provides an idea about the failure potential and, it is related to limit equilibrium analysis (Gurocak *et al.*, 2017). Kinematic and geometric conditions for judgment of the potential failure controlled by discontinuities can be seen in Figure 5.



Source: Gurocak *et al.*, (2017)

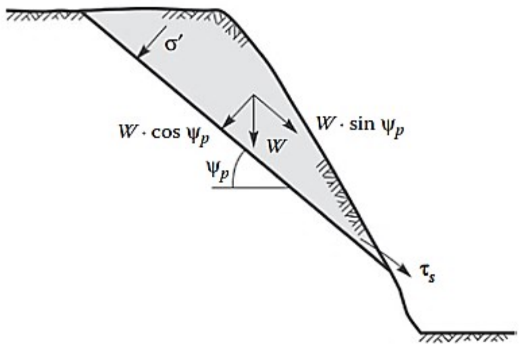
Figure 5. Kinematic analysis descriptions

Limit Equilibrium Method

Since 1930, slope stability analysis were usually conducted by limit equilibrium method (LEM). Although several other methods have been developed in the past few decades, the limit equilibrium approach remains the most widely used method in geotechnical practice, primarily because of its simplicity (Janevski and Jovanovski, 2021). The LEM has been commonly adopted in routine slope design, the conventional LEM has been widely applied in analyzing soil slopes or rock slopes that are heavily fractured or weathered (Zhang, 2020).

The stability of rock slopes under geological conditions depends on the shear strength generated along the sliding surface. For all shear-type failures, the rock can be assumed to be a Mohr–Coulomb material in which the shear strength is expressed in terms of the cohesion c and friction angle ϕ. For a sliding surface on which an effective normal stress σ′ is acting, the shear strength τ developed on this surface is given by Equation 3 (Wyllie, 2018).

Equation 3 is expressed as a straight line on a normal stress–shear stress plot in which the intercept defines the cohesion on the shear stress axis. The slope of the line is defined as the friction angle, and the effective normal stress is the difference between the normal component of the vertical stress due to the weight of the rock lying above the sliding plane (Wyllie, 2018). The calculation of the factor of safety for the block shown in Figure 6 involves the resolution of the force acting on the sliding surface into components acting perpendicular and parallel to this surface.



Source: (Wyllie, 2018)

Figure 6. Limit equilibrium method concept

That is, if the dip of the sliding surface is ψ_p, its area is A and the weight of the block lying above the sliding surface is W, then the normal and shear stresses on the sliding plane are:

Driving force, $\tau_s = \frac{W \sin \omega_p}{A}$ .... (4)

Resisting force, $\tau = c + \frac{W \cos \omega_p \tan \phi}{A}$ .... (5)

Factor of safety:

$FS = \frac{\text{resisting force}}{\text{driving force}}$ .... (6)

$FS = c\,A + \frac{W \cos \omega_p \tan \emptyset}{W \sin \omega_p}$ .... (7)

Where, W = weight of the block, A = area and ω_p = dip of the sliding surface, c = cohesion and ϕ = friction angle.

Empirical Method Q-Slope

The purpose of the Q-Slope method is to allow the engineering geologists and the rock engineers to assess the stability of excavated rock slopes in the field and make potential adjustments to slope angles as rock mass conditions become visible during construction (Barton and Bar, 2015). The Q-Slope method is intended to be used in reinforcement-free rock slopes and was developed from case records in six countries, spanning 17 rock types (igneous, sedimentary and metamorphic) and some saprolites for slope heights ranging from 5 m to 30 m (Bar, Barton and Ryan, 2016). An empirical relationship between the Q-Slope and the long-term stable slope angles is now supported through over 500 case studies from Asia, Australia, the Americas and Europe (Barton and Bar, 2020).

The Q-Slope method requires the assignment of the ratings for the rock quality designation (RQD), the joint set number (Jn), the joint roughness number (Jr), and the joint alteration number (Ja), which remain unchanged from the Q-System (Barton and Bar, 2015). For the Q-System users, the formula for estimating Q-Slope is the most familiar as shown in equation (8).

$$Q_{slope} = \frac{RQD}{J_n} \times \left(\frac{J_r}{J_a}\right)_0 \times \frac{J_{wice}}{SRF_{slope}} \quad \text{......................(8)}$$

- The first four parameters (RQD, Jn, Jr, and Ja). The Q-Slope ratings for rock quality designation, RQD (Deere 1963; Deere *et al.,* 1967), joint set number (Jn), joint roughness number (Jr), and joint alteration number (Ja) remain unchanged from the Q-System (Barton *et al.,* 1974; Barton and Bar, 2015). Tables 1, 2, 3 and 4 describe the ratings for RQD, Jn, Jr, and Ja, respectively.

Table 1. Rock quality designation

Rock Quality Designation		
	RQD Description	RQD (%)
A	Very poor	0 - 25
B	Poor	25 - 50
C	Fair	50 - 75
D	Good	75 - 90
E	Excellent	90 - 100
where RQD is reported or measured as B10 (including zero), a nominal value of 10 is used to evaluate Q-Slope. RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate		

Source: Barton and Bar (2015)

Table 2. Joint set number

Joint Set Number		
	Joint Set Number Description	Jn
A	Massive, no or few joints	0.5 - 1
B	One joint set	2
C	One joint set plus random joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	12
H	Four or more joint sets, random, heavily jointed	15
J	Crushed rock, earthlike	20

Source: Barton and Bar (2015)

- Discontinuity orientation: O-factor. The discontinuity orientation factor described in Table 5 provides orientation adjustments for discontinuities in rock slopes (Barton and Bar, 2015). The Set A O-factor is applied to the most unfavourable discontinuity set. If required, the Set B O-factor is applied to the secondary discontinuity set in case of potentially unstable wedge formations.
- Environmental and geological condition: Jwice. The environmental and geological condition number, Jwice, is more sophisticated than Jw of the original Q-System since slopes are outside and exposed to the elements for a very long time (Barton and Bar, 2015). As in Table 6, Jwice has a new structure for slopes, including tropical rainfall erosion-effects and ice-wedging effects. Adjustment factors for slope reinforcement and drainage measures are also included.
- Strength reduction factor: The SRFslope is obtained by using the maximum of SRFa, SRFb and SRFc described in the subsequent tables. Table 7 describes strength reduction factors (SRFa) for the physical condition of the slope surface (now or expected) due to susceptibility to weathering and erosion. Table 8 describes strength reduction factors (SRFb) for adverse stress-strength ranges in the slope. Table 9 describes strength reduction factors (SRFc) for major discontinuities such as faults, weakness zones and joint swarms which may also contain clay filling that adversely affects slope stability.

Table 3. Joint roughness description

Joint Roughness Number		
	Joint Roughness Number Description	Jr
(a) Rock-wall contact, (b) Contact after shearing		
A	Discontinuous joint	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
(c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1.0

Source: Barton and Bar (2015)

Table 4. Joint alteration description

Joint Alteration Number		
	Joint Alteration Number Description	Ja
(a) Rock-wall contact (no clay fillings, only coatings)		
A	Tightly healed, hard non-softening, impermeable filling, i.e., quartz or epidote	0.75
B	Unaltered joint walls, surface staining only	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0
D	Silty- or sandy-clay coatings, small clay disintegrated rock, etc.	3.0
E	Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also, chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.	4.0
(b) Rock-wall contact after some shearing (thin clay fillings, probable thickness & 1–5 mm)		
F	Sandy particles, clay-free disintegrated rock, etc.	4.0
G	Strongly over-consolidated non-softening clay mineral fillings	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings	8.0
J	Swelling-clay fillings, i.e., montmorillonite. Value of Ja depends on percent of swelling clay-size particles and access to water	8 - 12
(c) No rock-wall contact when sheared		
M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	5
OPR	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10. 13, or 13-20

Source: Barton and Bar (2015)

Table 5. O-factor description

Discontinuity Orientation Factor - O-factor			
	O-factor Description	Set A	Set B
A	Very favourably oriented	2.0	1.5
B	Quite favourable	1.0	1.0
C	Unfavourable	0.75	0.9
D	Very unfavourable	0.50	0.8
E	Causing failure if unsupported	0.25	0.5

Source: Barton and Bar (2015)

Table 6. Environmental and geological description

Environmental and Geological Condition Number					
	Jwice	Desert Environment	Wet Environment	Tropical Storms	Ice Wedging
A	Stable structure; competent rock	1.0	0.7	0.5	0.9
B	Stable structure; incompetent rock	0.7	0.6	0.3	0.5
C	Unstable structure; competent rock	0.8	0.5	0.1	0.3
D	Unstable structure; incompetent rock	0.5	0.3	0.05	0.2
When drainage measures are installed, apply Jwice x 1.5, when slope reinforcement measures are installed, apply Jwice x 1.3, and when drainage and reinforcement are installed, apply both factors Jwice x 1.5 x 1.3					

Source: Barton and Bar (2015)

Table 7. SRFa physical description

SRF_a Physical Condition		
	Description	SRFa
A	Slight loosening due to surface location, disturbance from blasting or excavation	2.5
B	Loose blocks, signs of tension cracks and joint shearing, susceptibility to weathering, severe disturbance from blasting or excavation	5
C	As B, but strong susceptibility to weathering	10
D	Slope is in advanced stage of erosion and loosening due to periodic erosion by water and/or ice-wedging effects	15
E	Residual slope with significant transport of material downslope	20

Source: Barton and Bar (2015)

Table 8. SRFb stress and strength

SRF_b Stress and Strength			
	Description	σ_c/σ_1	SRF_b
F	Moderate stress-strength range	50-200	2.5-1
G	High stress-strength range	10-50	5-2.5
H	Localized intact rock failure	5-10	10-5
J	Crushing or plastic yield	2.5-5	15-10
K	Plastic flow of strain softened material	1-2.5	20-15

Source: Barton and Bar (2015)

Table 9. SRFc major discontinuity

SRF_c Major Discontinuity					
	Description	Favourable	Unfavourable	Very Unfavourable	Causing Failure
L	Major discontinuity with little or no clay	1	2	4	8
M	Major discontinuity with RQD_{100} due to clay and crushed	2	4	8	16
N	Major discontinuity with RQD_{300} due to clay and crushed	4	8	12	24

Source: Barton and Bar (2015)

Q-Slope and Slope Angles

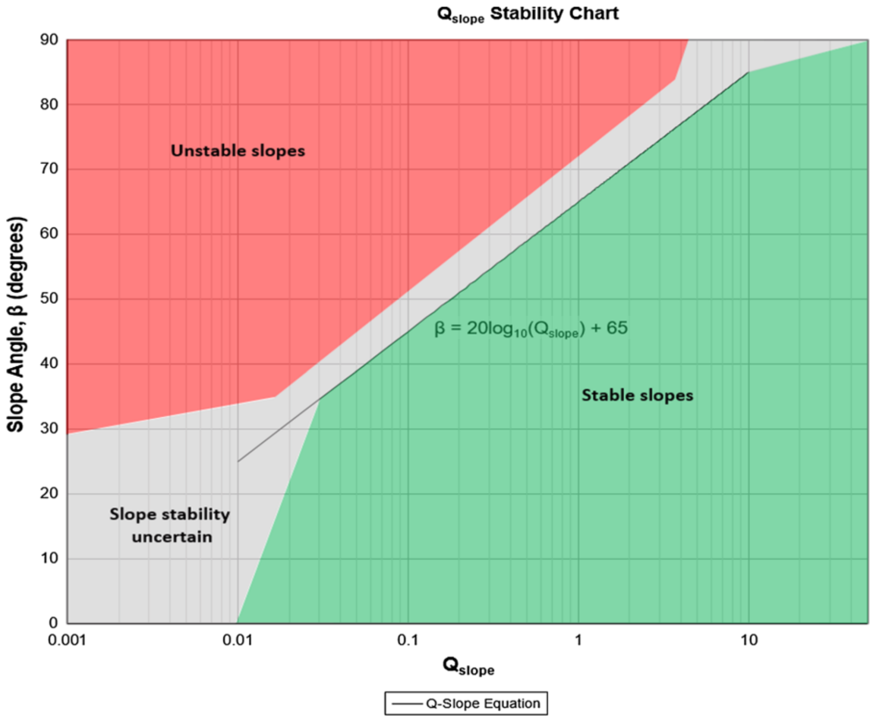
Barton and Bar (2015) derived a simple formula for the steepest slope angle (β) not requiring reinforcement or support for slope heights less than 30 m. This formula is now extended to all slope heights:

$\beta=20\log_{10}Q_{slope}+65^{o}$................................(9)

Figure 7 presents the Q-Slope stability chart with an uncertain slope stability 'corridor' in grey. The unstable area is shown in red, and the conservative stable slope area is shown in green (Bar, Barton and Ryan, 2016). The probability of failure (PoF) was calculated and displayed considering only the failed and quasi-stable slopes, using iso-potential lines in

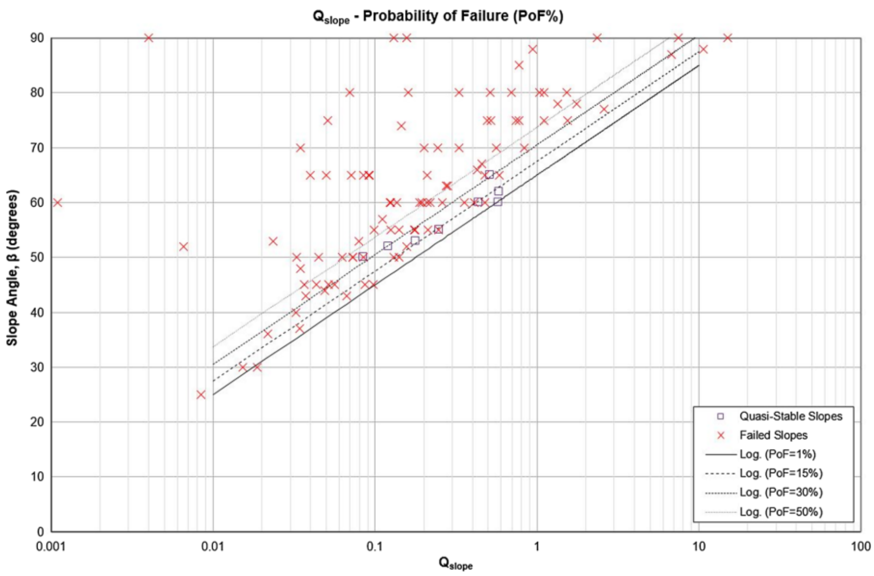
Figure 8. Bar and Barton (2017, 2018) acknowledge that these iso-potential lines are one possible interpretation of the data and that other similar interpretations are also possible. If certain degrees of failure is accepted, such as percentages of individual benches in opencast mines, then the following equations can be derived:

PL = 1% : $\beta = 20 \log_{10} Q_{slope} + 65°$.....(10)
PL = 15% : $\beta = 20 \log_{10} Q_{slope} + 67.5°$..(11)
PL = 30% : $\beta = 20 \log_{10} Q_{slope} + 70.5°$. (12)
PL = 37.5% : $\beta = 20 \log_{10} Q_{slope} + 72°$ .... (13)
PL = 50% : $\beta = 20 \log_{10} Q_{slope} + 73.5°$..(14)



Source: Bar, Barton and Ryan (2016)

Figure 7. Q-Slope stability chart



Source: Bar and Barton (2017, 2018)

Figure 8. Q-Slope – probability failure chart

RESULTS AND DISCUSSION

Geotechnical Characteristics

The research location lies in CV. Ellyta Karya Pratama in the west area of Yogyakarta and the southwest area of Kulon Progo regency. The research location is on the final slope with the actual geometry shown in Figure 9, where the mining activities have entered the mining closure phase and produced the final slope consisting of 4 single slopes with an overall slope height of 65m and an overall slope angle of 62°. Table 10 is illustrated the general description of the studied cases.

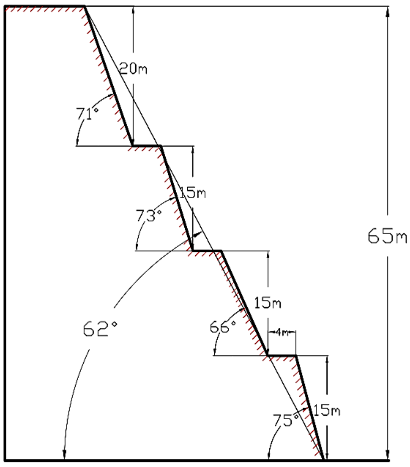


Figure 9. Final slope geometry

In various stages of research, as shown in Figure 10, within the complete activities, a significant geotechnical database has been collected by site investigation (discontinuity mapping) and laboratory testing, which is the geotechnical data collection for the geotechnical stability analysis and evaluation. The geotechnical data consisted of 445 discontinuity mapping within a 15m scanline on every slope and five boulders of rock samples for laboratory testing (rock physical and rock mechanic). Besides that, there are four additional data from the previous laboratory test for the material properties parameter within all laboratory test results shown in Table 11.

Table 10. General description research location

No	Characteristic	Description
1	Main lithology	andesite
2	Slope condition	dry and excavated
3	Slope height	15 m to 20 m
4	Slope angle	66° to 75°
5	Joint orientation	unfavourable
6	Joint number	2 to 3
7	Joint spacing	0.21 m to 0.25 m
8	Joint roughness	rough & undulating
9	Joint alteration	unaltered
10	Joint aperture	closed-filled discontinues
11	Infilling	mostly clay
12	Seepage	mostly dry

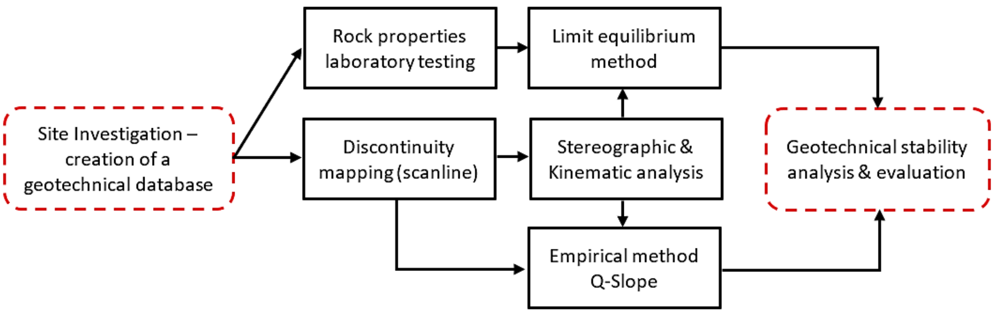


Figure 10. Research flow chart

Table 11. Result of rock laboratory test

Parameter	Rock Sample									Mean
	1	2	3	4	5	6	7	8	9	
C (Kpa)	91.7	130.0	133.9	105.3	99.6	102.7	113.8	128.9	151.9	117.55
Friction angle (°)	21.3	25.5	23.6	22.7	22.0	24.2	30.2	26.2	28.8	24.95
Density (KN/m^3)	23.1	22.9	21.7	21.2	21.7	26.1	26.4	26.2	25.9	23.88
UCS (Mpa)	86.6	76.4	71.3	56.1	76.4	113.4	114.5	98.4	72.3	85.04

Kinematical Analysis

Besides the rock strength parameter, the discontinuity has influenced a controlled type of failure in the jointed rock mass and has a significant role in rock slope stability. Discontinuity mapping has conducted through the scanline mapping method by the Priest & Hudson, 1976 in Read and Stacey (2009) within 15 m line each the single slope. Discontinuities in the research location are categorized in non-systematic joints because of irregular form, spacing, and orientation. The orientation of the joint set can be known based on the stereographic projection method with the input parameter of orientation mapping along the scanline. Result of stereographic projection can be seen in Figure 11, 12, 13 and 14. The recapitulation of stereographic projection is shown in Table 12.

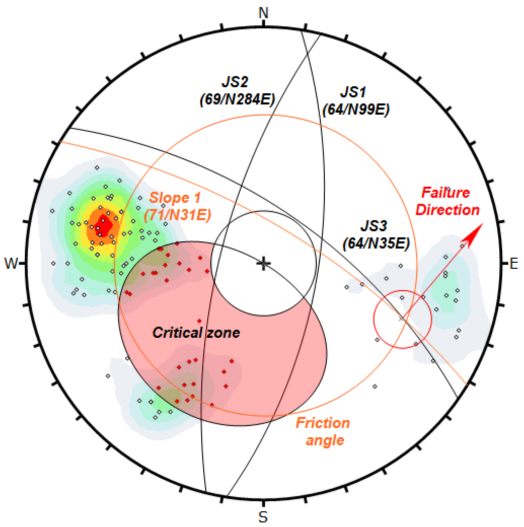


Figure 11. Stereographic projection single slope 1

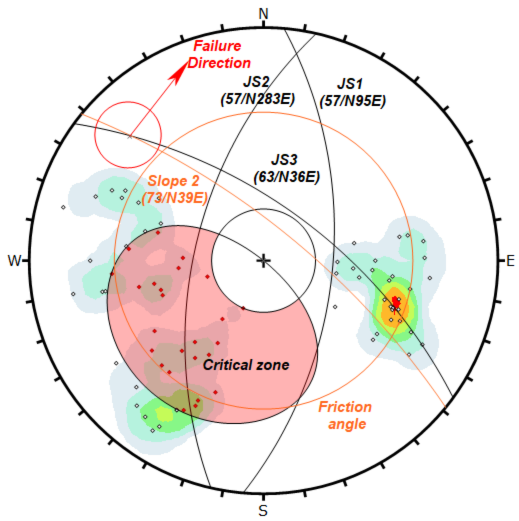


Figure 12. Stereographic projection single slope 2

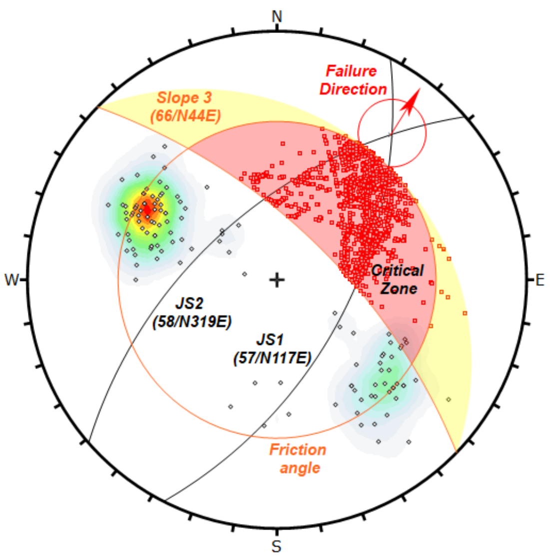


Figure 13. Stereographic projection single slope 3

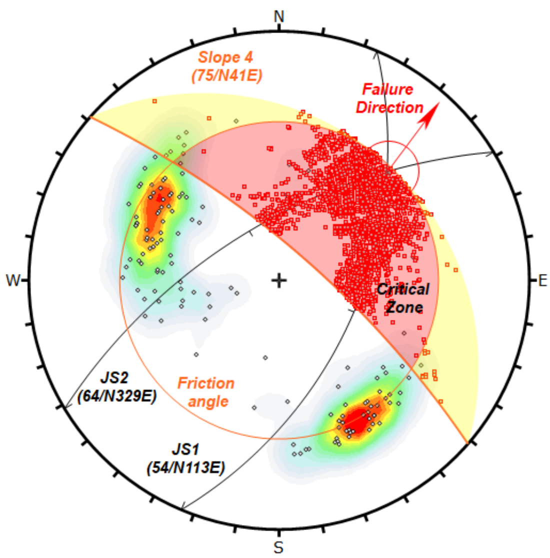


Figure 14. Stereographic projection single slope 4

Table 12. Result of discontinuity orientation at research locations

Location	Joint Orientation			Slope	Slope Height (m)	Slope Angle (°)	Rock Type
	J1	J2	J3				
Slope 1	64°/N099°E	69°/N284°E	64°/N035°E	71°/N031°E	20	71	Andesite
Slope 2	57°/N095°E	57°/N283°E	63°/N036°E	73°/N039°E	15	73	Andesite
Slope 3	57°/N117°E	58°/N319°E	-	66°/N044°E	15	66	Andesite
Slope 4	54°/N113°E	64°/N329°E	-	75°/N041°E	15	75	Andesite

The kinematic analysis method is widely preferred to analyze the type of failure controlled by the orientation of the discontinuities, the orientation of slope, and the friction angle. The excavated slope has the type of failure planar and wedges based on the criteria of the kinematic analysis. The direction of the failure and the critical zone can be seen in Figure 11, 12, 13 and 14.

According to Hoek & Bray, 1981 in Wyllie (2018) the following geometrical conditions must be provided for planar failure on a single plane to occur, there is the dip direction of the discontinuity that must be within ±20 degrees of the dip direction of the slope face ($\alpha_f \approx \alpha_i$; ±20°), the dip of discontinuity must be less than the dip of the slope face ($\psi_i < \psi_f$), and the dip of discontinuity must be greater than the friction angle of the failure plane ($\psi_i > \phi$).

Wedge failures occur when a rock mass slides along two intersecting discontinuities. According to Hoek & Bray, 1981 in Wyllie (2018), three conditions are required for wedge failures. There is the direction of the line of intersection that must be similar to the dip direction of the slope ($\alpha_f \approx \alpha_i$), the plunge of line of intersection must be less than the dip of slope ($\psi_i < \psi_f$), and must be greater than the friction angle ($\psi_i > \phi$). According to Brawner and Milligan (1971) in Kliche (2018) there are two types of wedge namely, wedge block that can be failure, and the wedge block that cannot be a failure. The result of the kinematical analysis of the actual single slope in the research location can be seen in Table 13 and the projection in the research location can be seen in Figure 15.

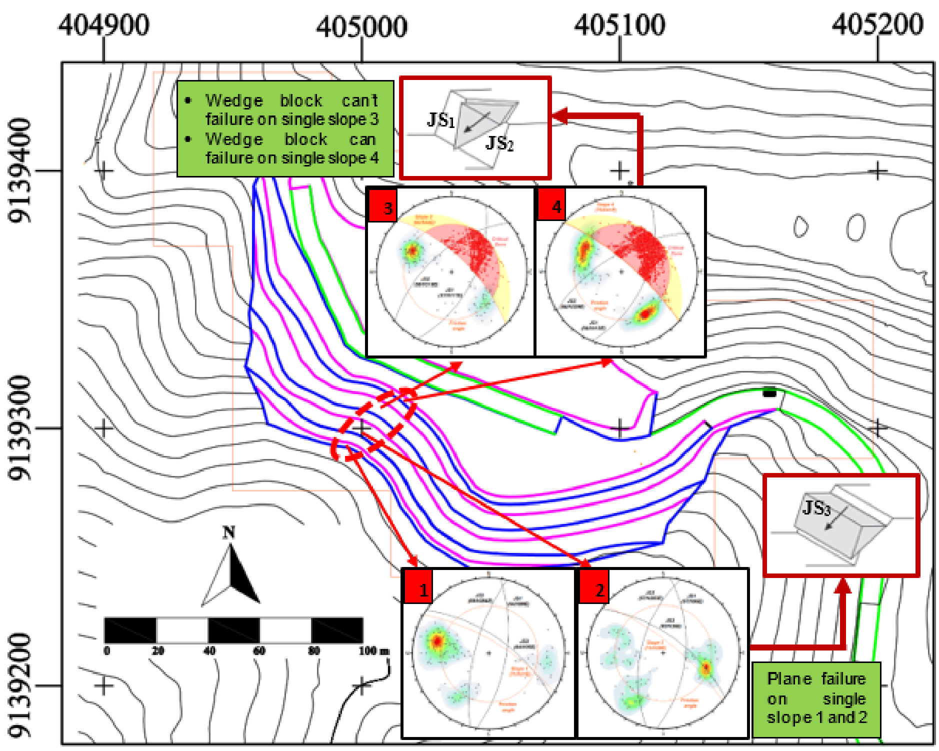


Figure 15. Research location and the projection of kinematical analysis

Table 13. Kinematical analysis

Location	α_f (N...°E)	α_i (N...°E)	ψ_f (°)	ψ_i (°)	φ (°)	Requirement Kinematical		Result
						$\alpha f \approx \alpha i$; ±20	$\psi_f > \psi i > \varphi$	
Slope 1	31	35	71	64	24.95	Accepted	Accepted	Planar failure
Slope 2	39	36	73	63	24.95	Accepted	Accepted	Planar failure
Slope 3	44	38	66	17	24.95	Accepted	Not accepted	Wedge block
Slope 4	41	45	75	27	24.95	Accepted	Accepted	Wedge failure

From the kinematical analysis can be known, the potential of planar controlled by the joint set 3 (64°/N035°E) on the single slope 1 & joint set 3 (63°/N036°E) on the single slope 2 and the potential of wedge block is controlled by the intersection of joint set 1 (57°/N117°E) and the joint set 2 (58°/N319°E) on the single slope 3 and intersection of joint set 1 (54°/N113°E) and the joint set 2 (64°/N329°E) on the single slope 4.

Empirical Method Q-Slope

The Q-Slope method was applied to the research location with the lithology andesite. Q-Slope method helps to execute and identify more quickly stability as conditions of rock mass through the maximum slope angle with the safety factor and probability based on the Ministry of Energy & Mineral Resources Decree 1827/K/30/MEM/2018 (Menteri Energi dan Sumber Daya Mineral, 2018). For FS ≥ 1.1 and PF ≤ 37.5% on the moderate risk criteria, the following Equation (13) was used for potential adjustable slope angle calculation. The ratings for the different parameters used for Q-Slope calculation, as presented in Equation (8), were estimated in Table 14, and the calculation of the Q-Slope is shown in Table 15. Further, the Q-Slope value, angle ($\beta_{Q\text{-}Slope}$), and actual slope angle (β_{Slope}) of the investigated slopes are shown in Table 15 according to the empirical stability method Q-Slope of Barton and Bar (2015). There is the following description of the Q-Slope parameter:

- RQD in the research location was known based on the calculation discontinuity spacing with an average result of 0,23m with the condition in moderate spacing. Therefore, the value of the RQD is in the range of 90-100% with the classification of excellent condition.
- Joint numbers were known based on the stereographic projection to know the joint set/family discontinuity, and the result is shown in Table 12.

Table 14. Estimated Q-Slope parameters

Location	RQD (%)	Jn (rating)	Jr (rating)	Ja (rating)	Jwice (rating)	O-factor (rating)	SRFa (rating)	SRFb (rating)	SRFc (rating)
Slope 1	91.62	9	3	1	0.5	0.75 (set A)	5	1.5	2
Slope 2	93.65	9	3	1	0.5	0.75 (set A)	5	2.25	2
Slope 3	93.29	4	3	2	0.5	0.5 (set A) 0.8 (set B)	5	2.4	4
Slope 4	93.06	4	3	2	0.5	0.5 (set A) 0.8 (set B)	5	2.5	4

Table 15. Estimated Q-Slope values

Location	RQD/Jn	Jr/Ja	Jwice/SRF	$(Jr/Ja)_{O.factor}$	Q_{slope}	Q_{slope} Angle (°)	Slope Angle (°)	Stability condition
Slope 1	10.18	3.00	0.10	2.25	2.291	79.20	71	Stable
Slope 2	10.41	3.00	0.10	2.25	2.341	79.39	73	Stable
Slope 3	23.32	1.50	0.10	0.90	2.099	78.44	66	Stable
Slope 4	23.27	1.50	0.10	0.90	2.094	78.41	75	Stable

- Joint roughness and joint alteration were known based on the condition of contact between the rock wall influences by the aperture and the infilling. In general condition, the rock-wall is in contact condition because the joint aperture was described in closed discontinuity. Besides that, the level of roughness in the rough and undulating description with the alteration of discontinuities is unaltered and slightly altered joint walls within the surface staining only.
- Jwice was known based on the stability affected by the structure and rock strength parameter through the wet environmental condition the in-research location. From the kinematic analysis, there are structures controlled the potential of failure, and the average UCS value is 85.04 MPa with the description of the competent rock. The O-factor parameter has been described in unfavorable joint conditions because the joint controlled the potential of failure.
- SRF parameters were known from the maximum value between SRFa, SRFb and SRFc. The SRFa (physical condition) is known with that the lithology andesite is strong against the weathering based on UCS value and there are plane & wedge blocks due to the influence of discontinuity. SRFb (a stress condition) is known from the calculation of σ_c/σ_1, with the maximum stress (σ_1) can be estimated by considering the material density and the slope height. The SRFc (major discontinuity) is known by joint aperture in closed discontinuity conditions and filling material (clay) in unfavourable conditions.

Based on the result in Table 15, the slope is in stable condition because the maximal slope angle of the empirical method Q-Slope is greater than the actual slope angle ($\beta_{Q\text{-}Slope}$ > β_{Slope}). The comparative result of slope angle of the actual geometry at the research location in Figure 9 and the result of the empirical method Q-Slope can be seen in Figure 16.

Analysis of Slope Stability

Slope stability analysis is carried out in fully saturated (filled with water) conditions or wet environmental conditions, where this condition assumes that the analysis is pessimistic of slope stability. The analysis of slope stability with the LEM is carried out based on the potential of failure in the kinematic analysis.

Figure 16. Comparative result of the actual geometry and the Q-Slope on the single slope

Slope stability analysis is carried out on two geometric conditions, actual geometry and the geometry of the empirical method Q-Slope which the different in the slope angle. As a result of empirical method in Table 15 can be known the slope angle of Q-Slope ($\beta_{Q\text{-}Slope}$) has average value 78.86° greater than the actual slope angle (Figure 9) with average value 71.25°.

The material properties of the rock are based on the laboratory testing shown in Table 11. Slope stability analysis was carried out by statistical method in the variation of the random number of the parameter's cohesion and the friction angle. Total of random number are 100 data of each parameter according to the selected uniform data distribution. Therefore, the slope stability calculation used software tools in Excel to produce a safety factor of 10.000 data to determine the failure probability.

Thus, the analysis results in Table 16 show that the FS and the PF values from the actual slope geometry (Figure 9) and the Q-Slope geometry (Table 15) give stable or safe conditions and fulfill the acceptance criteria with the safety factor (FS) ≥ 1,1 and failure probability (PF) ≤ 37.5%. The graph of the slope stability analysis can be seen in Figures 17 and 18.

Table 16. Result of factor of safety and probability of failure

No	Location	Geometry			FS	PF	Stability Condition
		Condition	Height (m)	Angle (°)			
1	Slope 1	Actual	20	71.00	3.432	0.009%	Stable
2		Q-Slope	20	79.20	1.441	25.287%	Stable
3	Slope 2	Actual	15	73.00	3.277	0.017%	Stable
4		Q-Slope	15	79.39	1.836	4.085%	Stable
5	Slope 4	Actual	15	75.00	4.842	0.000%	Stable
6		Q-Slope	15	78.41	4.615	0.000%	Stable

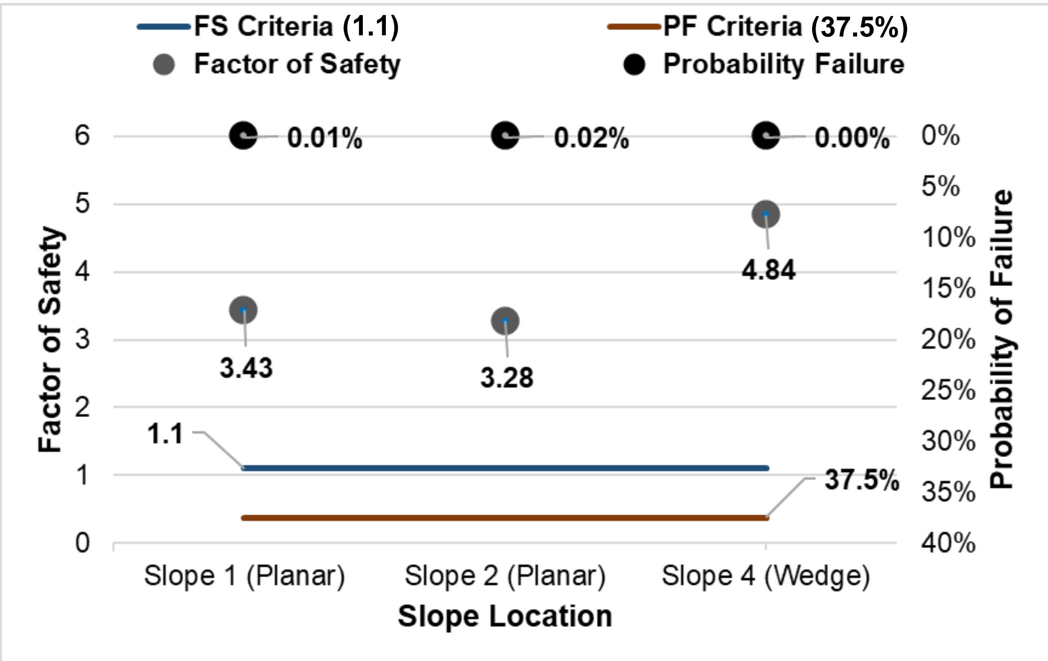


Figure 17. FoS and PoF of actual slope geometry

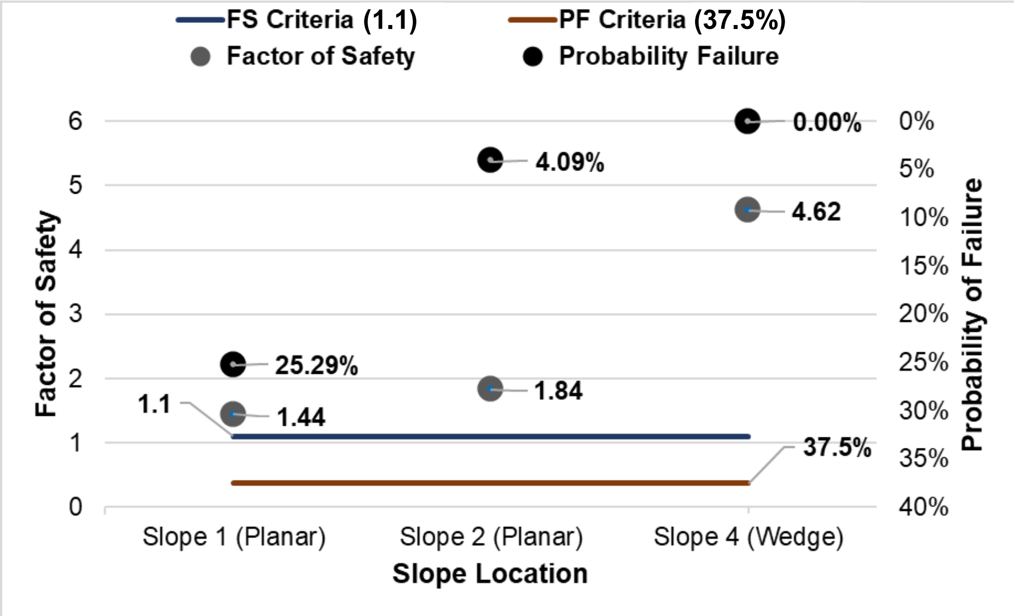


Figure 18. FoS and PoF of Q-Slope geometry

The different results of the safety factor and probability of failure are affected by the different geometry of the slope angle, because the maximum slope angle of the empirical method Q-Slope ($\beta_{Q\text{-}Slope}$) is greater than that of the actual slope angle ($\beta_{Q\text{-}Slope} > \beta_{Slope}$), so the actual slope has a higher safety factor value and lower probability of failure.

CONCLUSION AND SUGGESTION

This research, can be known for the different potential failure mechanisms due to its discontinuities, the use of the new empirical method of Q-Slope, the use of the limit equilibrium, and the probability of the failure to validate the actual and Q-Slope geometry presented.

The Q-Slope is a new empirical method and relatively fast for assessing the stability of the excavated slopes. The main advantage of Q-Slope against the other empirical methods is that it helps estimate the long-term stable slope angles without reinforcement.

According to the results of the analysis, it can be known that two methodologically different approaches Q-Slope and Limit Equilibrium are in stable condition. The two methods are applied to the comparative assessment of slope stability in mining rock slopes at the mine closure phase because it gives the same linear result for the factor of safety and probability of failure.

Therefore, if the slope's geometry uses the Q-Slope recommendation, the andesite potential that can be mined will be more optimized than the actual slope geometry if the mine operations continue.

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