Design Analysis of Tunnel Portal in Pasir Jawa L.520 Pongkor's GMBU Underground Mine PT. Aneka Tambang Tbk.

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Abstract

PT. Aneka Tambang is a gold mines company in West Java, Indonesia. Deposit type in the Pongkor's mine of this company is low-sulphidation epithermal type and consist of four main mineralized quartz veins which are Pasir Jawa, Ciguha, Kubang Cicau, and Ciurug. This company has plan to open a new underground mine in Pasir Jawa, which is located at the north and served as smallest hydrothermal veins among the other Pongkor hydrothermal veins. To do this, geotechnical analysis is required to build a portal of the tunnel. This research focuses on geotechnical analysis which include slope stability analysis of the area around the tunnel portals. Methods used in this research are the kinematic analysis, Rock Mass Rating (RMR) and Slope Mass Rating (SMR). The result of this research includes the characteristic of rock physical, structural geology, geomechanics characteristic of the rock mass using RMR. Pasir Jawa area has RMR values of 42-47 and included in the class III (fair rock). The discussion of this research describes the type of failure based on kinematic analysis, the calculation of slope safety factor, and safe design slope.

Keywords: kinematic analysis, slope stability, factor of safety, RMR, SMR

1. Introduction

Pongkor Gold Mining Business Unit which belongs to PT. Aneka Tambang Tbk is one of the underground gold mining sites in West Java, Indonesia. Pongkor's mine consist of four main mineralized quartz vein, one of them is Pasir Jawa vein (Basuki, 1993). This company has plan to open a new underground mining in Pasir Jawa area. To do this, the geotechnical analysis is required, especially the analysis of tunnel portal (Pariseau, 2008). Tunnel portal should have a design that is safe from slope failure in the long term. Geotechnical analysis of soil and or rock mass around tunnel portal areas required to obtain a safe design. Design analysis of slope reflected in the value factor of safety (FS).

2. General

Geomechanics characteristic and evaluation of the discontinuity of the rock is necessary for the stability of a rock slope cutting. Various types of rock slope failure are associated with geological structures. Therefore, recognizing the potential for slope stability problems in the early stages of an activity which involves the cutting of the slope is very important (Hoek & Bray, 1981). Rock failure usually starts from and follow the discontinuities in the rock such as joint, fractures, and bedding planes, faults and other types of cracks in the rocks. Strike and dip discontinuities have an important role in controlling the type of failure that may occur on slopes. Besides discontinuity, an assessment of the rock mass is also very important for the estimation of the strength, deformability of rock mass, and stability of rock slope.

The location of the study area lies in Pasir Jawa, Pongkor, Bogor regency, West Java Province, Indonesia. The study area is divided into two areas, the southern slope and northern slope (see table 1). The southern slope was landslide on April 2014. The northern slope is a place which will be used as the ore production access tunnel under this slope. Rocks making up the study area consist of tuff. Geological structures that develop in research areas is shear joint and tension joint were mostly filled by minerals quartz and manganese oxide. Joint strike dominant on the southern slope is N225°-236°E, while the northern slope is N290°-306°E.

3. Methods

3.1 Rock Mass Rating (RMR)

Rock Mass Rating (RMR) system was developed by Bieniawski (1973) and has been modified over the years (Bieniawski, 1989). RMR has wide application in different rock engineering fields such as mining, hydropower projects, tunneling, and slope stability. The RMR include five input parameters to obtain basic RMR value (RMR_{basic}).

1. Uniaxial Compressive Strength (UCS) of rock mass.
2. Rock Quality Designation (RQD).
3. Spacing discontinuities.
4. Condition of discontinuities.
5. Groundwater condition.

The rating for these five parameters are assumed to yield the RMR_{basic} ranging between 0 and 100. The rating tables for these five parameters and the rock mass classes are given by Bieniawski (1989).

3.2 Slope Mass Rating (SMR)

Romana (1985) developed the Slope Mass Rating technique for stability assessment of the rock slopes which is primarily based on the application for RMR_{basic} and the orientation of discontinuities. This technique is suitable for preliminary assessment of slope stability in the rocks. In this SMR approach adjustment rating for joints in relation to the slope has been introduced by Romana (1985; in Singh & Goel, 1999). The adjustment rating for joints is the product of the three following factors:

- F1 is a measure of parallelism between the slope face and the joint plane or the line intersection between two joint planes.
- F2 depends on the dip of the joint plane or plunge of the line intersection between two joint planes.
- F3 depends on the relation between the dip of the slope face and dip of the joint plane or plunge of the line of intersection of two joint planes.

The rating (F4) assigned to the excavation methods range from +15 (natural slope) to -8 (deficient blasting). The adjustment ratings for joint orientation and method of excavation are given in the paper by Romana (1985). SMR can be obtained from the RMRbasic as follow:

$$SMR = RMR_{basic} + (F_1 \cdot F_2 \cdot F_3) + F_4 \qquad (1)$$

3.3 Kinematic Analysis

The kinematic analysis method, which was described by Hoek and Bray (1981), allows the investigation of potential planar, wedge, and toppling failure modes of rock slopes. This method analysis the movement of objects without considering the cause of forces. This method is based on a detailed evaluation of the structure of the rock mass and geometry of the weak areas that can contribute to slope instability. This method is based on strike and dip discontinuities and friction angle of discontinuity.

4. Result and Discussion

Engineering geological properties of the rocks exposed in the study area were determined on the basis of field observation/measurement and laboratory tests. The uniaxial compressive strength of the intact rocks were determined in according with ISRM (1985). The shear strength parameters and geomechanical properties of the rock masses were determined by Hoek-Brown failure criterion 2002 (Hoek & Brown, 2002). Geomechanics characteristic of the rock mass obtained based on five parameters contained in geomechanic classification system Rock Mass Rating (RMR) (Bieniawski, 1989). (see table 2)

To determine the stability of the portal, slope stability analysis were performed. Assessment of slope stability in rocks through kinematical analysis to determine the type and possibility of the occurrence of any kinematic failure (*Dips 5.1* Software). Next, limit equilibrium analysis were performed to provide a direct measure of stability in

Table 1 Slope geometry

Location	Slope face	Slope face angel	Slope face height	Upper slope face	Upper slope face angel	Upper slope face height	Overall height
Southern slope	N260ºE	61º	20 m	N260ºE	26º	19.5 m	39.5 m
Northern slope	N90ºE	49º	41 m	N90ºE	31º	17 m	58 m

Table 2 Mechanical Properties

Location	UCS (MPa)	RQD (%)	γ (T/m^3)	c (Mpa)	Φ (º)	RMR	RMC
Southern slope	32.02	66.22	2.549	0.260	42.31	47	III (fair rock)
Northern slope	32.52	49.14	2.549	0.285	37.16	42	III (fair rock)

terms of the factor of safety (FS) (Hearn, 2011).

Factor of safety for the failure controlled by discontinuities (regularly jointed) like planar, wedge, and toppling failure using the formula according to Hoek & Bray (1981). While factor of safety for the failure controlled by rock mass and weak rock (irregularly jointed) like circular failure were analyzed by the slope stability software *Slide 5.0* using Bishop's simplified method under dynamic conditions. The study area lies 4th seismic zone according to the earthquake zonation map of Indonesia (Irsyam, 2010).

4.1 Slope Stability Analysis for Southern Slope

Figure 1 illustrates the stereographic projection of the strike and dip of all discontinuities. The stereographic show that the pole point for the joint sets (JS1 and JS2) and the intersection point of JS1-JS2 are located inside the daylight envelope. Based on stereographic mentioned, this slope has wedge failure potential, formed by the intersection of discontinuities JS1 and JS2. The direction of sliding N277°E and plunge intersection (ψ_i) 57°.

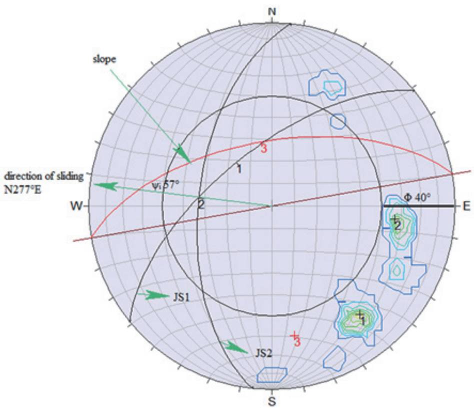

Fig. 1 Stereographic projection of Southern Slope

Natural slope stability analysis (existing slope) and the calculation of slope safety factor refers to the results of the kinematic analysis. The results of the kinematic analysis showed that wedge failure was possible on the southern slope. Wedge limit equilibrium analysis on this slope (Hoek, Bray and Boyd; in Hoek & Bray, 1981) showed that the value of factor of safety is 1.1814. According to Bowles (1989) this slope includes into critical slope (1.07<FS<1.25) with the occurrence/intensity the landslide *have* occurred. Wedge failure can be proved from failure which occurred on April 2014 ago. (Figure 2)

Fig. 2 Wedge failure on the Southern Slope

The safe slope (FS> 1.5) for southern slope use Slope Mass Rating (SMR) method Romana (1985). Based on the calculations Slope Mass Rating, obtained SMR value is 51.2° and included in class III (normal). For recommendations reinforcement on the slopes, can be used according to table rock slope reinforcement for each grade SMR (Romana, 1985), SMR 51.2 ° included in class IIIa with the type of reinforcement used toe ditch and/or nets, spot or systematic bolting, and spot shotcrete.

4.2 Slope Stability Analysis for Northern Slope

Figure 3 illustrates the stereographic projection of the strike and dip of all discontinuities in northern slope. The stereographic show that the pole point for the joint sets (JS1 and JS2) and the intersection point of JS1-JS2 are not located inside the daylight envelope. Wedge failure does not occur on the northern slope because the pole point of the discontinuities are not located inside the wedge daylight envelope. Planar and toppling also not occur on this slope. The intersection of two joint sets have smaller plunge intersection than the internal friction of discontinuities. Hoek and Bray (1981), wedge and planar failure occurred if $\psi fi > \psi i > \Phi$, where ψfi is the inclination of the slope face, ψi is the dip of the line intersection, and Φ is internal friction of discontinuity.

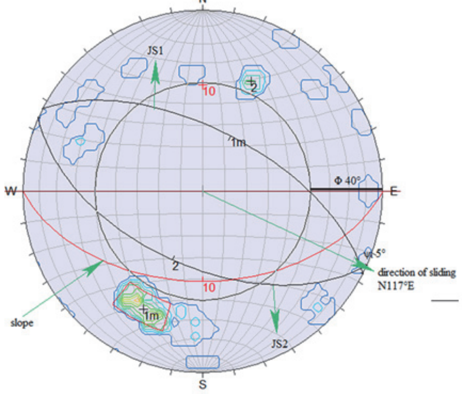

Fig. 3 Stereographic projection of Northern Slope

Kinematic analysis above shows that the northern slope are relatively stable for wedge, planar, and toppling failure. This slope has circular type failure potential because the rock consist this slope is heavily jointed rock (irregular jointed) which can be seen from the spread of discontinuities on stereoplot.

Currently, the existing slopes located at an elevation of 512 meters, while the plan of the tunnel is at an elevation of 520 meters. Therefore, as the access road into the tunnel, the slope should be cut. Slope stability analysis on the results of cutting slope design should be done to get the design slopes safe from failure with the value of the safety factor (FS) are allowed FS > 1.5.

Design of cutting slope is making steps on the slopes with a slope length of each bench that is 10 meters and width bench 3 meters. To get a factor of safety more than 1.5 (FS>1.5), conducted a simulation by varying the overall slope and single slope angle. Slope stability analysis of irregular jointed were analyzed by slope stability software *Slide 5.0* (Rocsience, 2006)*,* simulations carried out under dynamic condition, with water-saturated ground water level and also take a seismic load. Simulations start from overall slope 55° to 35°. The result of calculation using software *Slide 5.0* can be seen in Figure 5.

Based on the calculation above, cutting angle of the slope (overall slope) is relatively safe with FS>1.25 can be done by the slope angle of 55°, but to get the value of FS are allowed (FS>1.5), the maximum angle of slope cutting the angle slope design 45°, the value of FS is 1.504. At 45° slope design, single slope at 55°.

(Figure 6 and 7)

4.3 Desain of Portal Tunnel

Tunnel located in northern slope, which will be used as the entrance to the underground mining production. Location studies have joint conditions set over 3 and included into the low-stress environment ($\sigma 1/\sigma c<0.15$). Type of tunnel that will be made on this plan includes the development of the tunnel with the standard dimensions 4.5m x 4.5m and the roof is arc with the aim to increase the compression stress on the roof. To anticipate rock falling from slopes, a concrete roof could be made 2.5m from the slopes on the portal of tunnel.

(Figure 7)

5. Conclusions

The southern and northern slopes area consist of jointed tuff, were mostly filled by quartz and manganese oxide minerals. The result of the kinematic analysis, the southern slope has a wedge-type failure, the value of factor of safety is 1.1814 (critical slope). The safe slope for southern slope use Slope Mass Rating (SMR) method for wedge-type failure, the safe slope for southern slope is 51.2°. On the northern slope, which are the place that will be used as the ore production access tunnel under this slope has a circular-type failure. The safe slope obtains from simulations of cutting slope angle with a bench. The design for tunnel portal have overall angle slope is 45°, single slope angle 55°, width of each bench 3m, and length of each bench 10m. The geometry of tunnel is 4.5m x 4.5m and the roof is an arc. On the tunnel portal made a concrete roof for anticipate rock falling from the slope.

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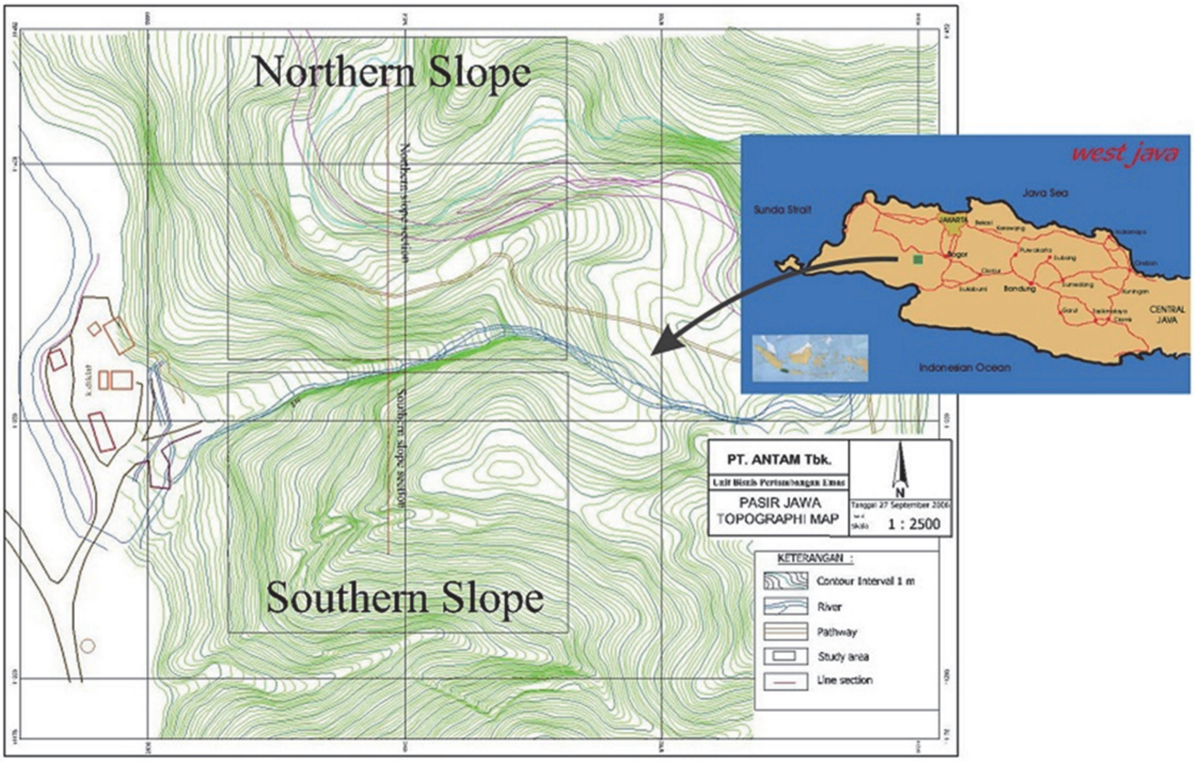

Fig. 4 Location map of study area

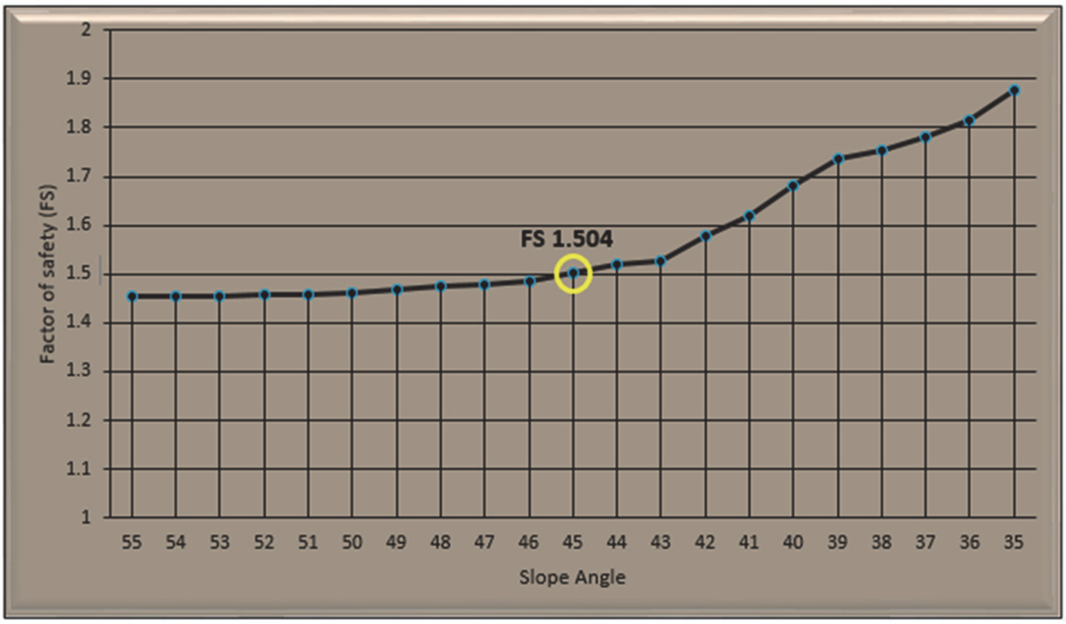

Fig. 5 Factor of safety value for Northern Slope

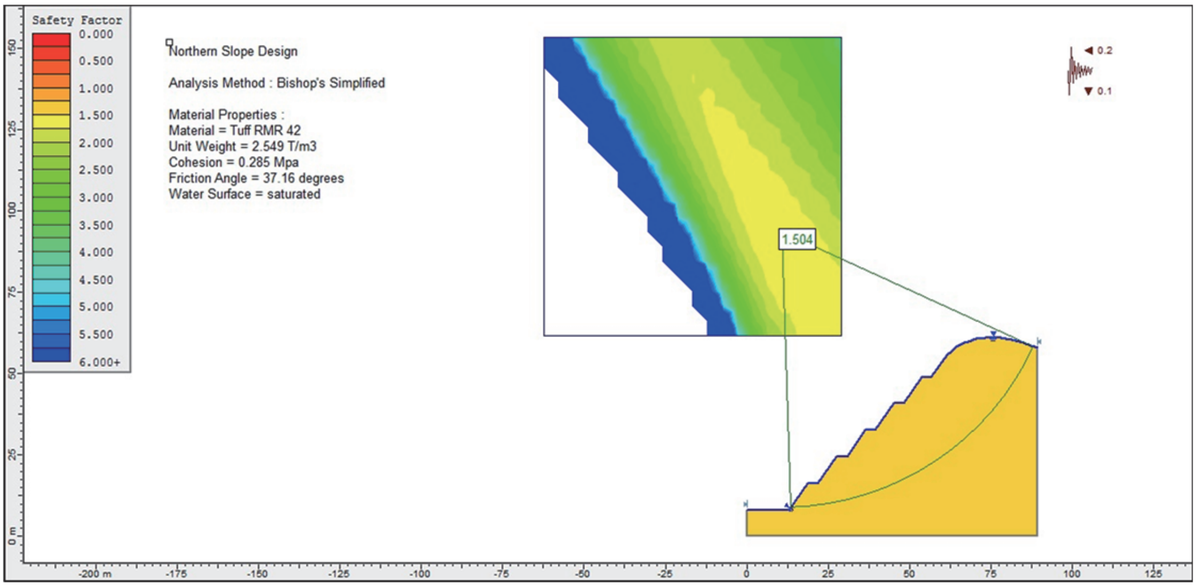

Fig. 6 Slope stability analysis for Northern slope use software *Slide 5.0*

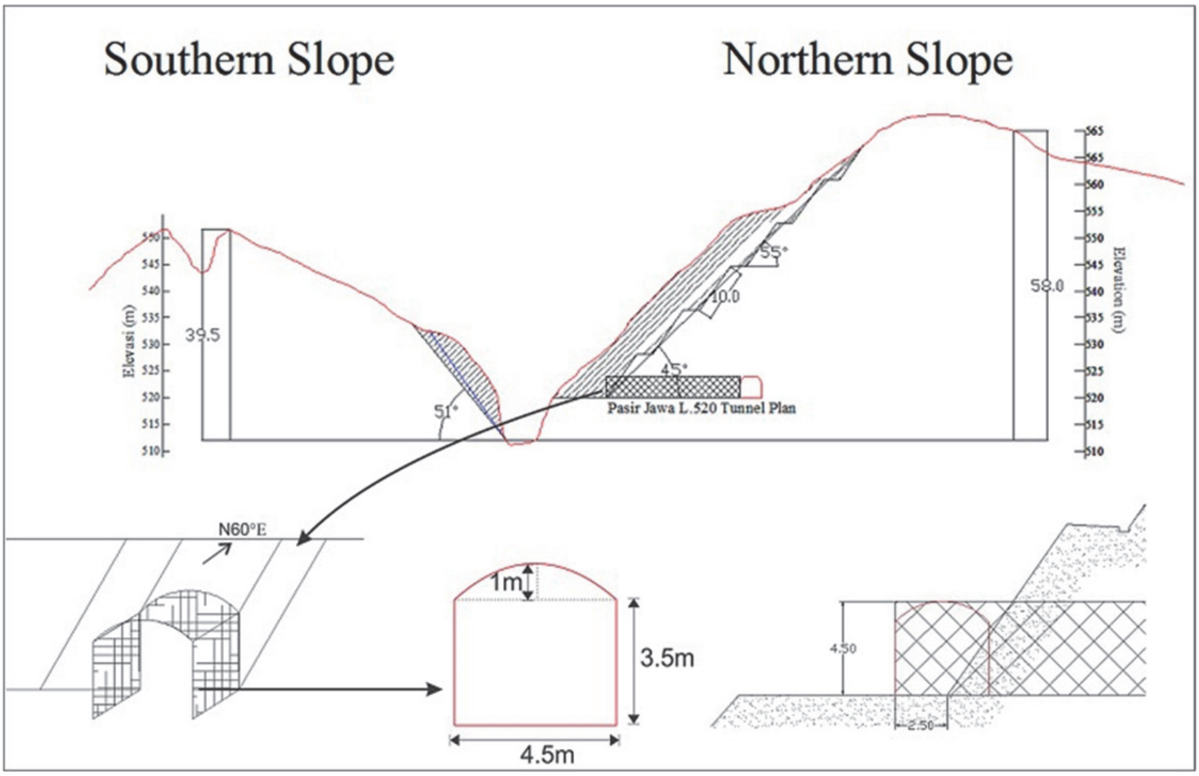

Fig. 7 Design of slopes and tunnel plan