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Research Article

Disposal slope design based on low-plasticity rock’s shear strength in coal mining activities

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Abstract

Slope stability is the primary factor in designing a stable slope. The strength of the disposal slope relies on the characteristic of the dump materials. The rock shear strength denotes the rock's ability to withstand the burden, both constant and dynamic load. The weakest material used as a reference in designing the disposal slope is siltstone in Pulaubalang formation (location-04). The safety value in locations 04 and 02 tend to drop quickly as the inclination increase, while the safety in locations 01 and 03 tend to fall slowly. This indicates that rocks with a lower plasticity index tend to be stronger in steeper slope conditions. The slope geometry was designed to be 10, 15, 20, and 25 m high with a slope angle of 10°, 15°, 20°, 25°, 30°, 35°, 40°, 45°, 50°, 55°, and 60°. The slope geometry is considered stable and safe. However, this study suggests that the most effective slope design is the slope with 25 m high, an overall slope of 25.3°; a single slope of 35°; a berm width of 4.66 m, bench height of 5 m, with a safety factor value of 4.30 (SF= 4.30).

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Introduction

Open pit is one of the coal mining systems where all mining activities are performed relatively near the earth's surface. This mining system is profitable only if the coal layer is near the surface. The surface coal mining method typically consists of overburden removal, coal getting, and reclamation (Schissler, 2004). The removal and disposal of overburden may be affected by geological, geotechnical, and environmental factors, so these activities should be carefully planned due to their significant effect on the surface (Oggeri et al., 2019). The embankment slope at the research site consists of claystone and siltstone with low plasticity characteristics. Both materials will expand in wet conditions, shrink in dry conditions, and contain montmorillonite. This type of expansive soil will expand or increase in volume when in contact with air (Davis et al., 2003). Claystone is an aggregate of microscopic and submicroscopic-sized particles derived from the chemical decomposition of constituent rock elements and is plastic. In addition, the permeability of claystone is very low, and if it is dry, it will be hard, and if wet, it will be soft, plastic, and cohesive, expands and shrinks quickly, so it has a significant volume change (Chiarelli et al., 2003). Meanwhile, siltstone consists of coarse-grained minerals with a high rate of pore development (Yu et al., 2021).

Overburden dumping is a continuous process during the mining process that requires several considerations in designing and selecting the dumping location (Hustrulid et al., 2000). It is essential as inappropriate overburden dumping, i.e., incompliant to the standard operation, can cause unstable slopes and landslides (Behera et al., 2017). According to Jones et al. (2020), higher soil plasticity, i.e., more comprehensive water content range in the plastic area, tends to be weaker and have a higher shrink-swell, causing an unstable slope. Therefore, performing a simulation based on the physical, mechanic, and plasticity properties during the disposal slope design process is necessary to obtain optimum safety value by referring to the weakest material and serves as a guide in determining the proper disposal geometry in mine design.

Methods

Time and location

The present study was conducted from March to August 2021 in several coal mining companies in Samarinda, East Kalimantan:

a. Location-01 : CV. Tampaure Jaya Mandiri Coal with Balikpanan formation dominated by claystone.
b. Location-02 : PT. Insani Bara Perkasa with Pulaubalang formation dominated by claystone.
c. Location-03 : CV. Busur Abadi with Pulaubalang formation consisting of siltstone.
d. Location-04 : CV. Piawai Alam Bumi Perkasa with Balikpapan formation dominated by siltstone.

The Kutai Basin is located in East Kalimantan, Indonesia and consists of several rock formations (Renema et al., 2015) that vary from one location to another (Adebayo and Adetula, 2013).

Physical property test

A physical property test is carried out to determine values affecting the rock strength, including natural density. Dry density, saturated density, apparent specific gravity, true specific gravity, specific gravity, natural water content, saturated water content, saturation level, porosity, and void ratio. Rock density is a rock's physical property that may substantially change due to different mineralogy and porosity content (Carmichael, 2017). The density of pore-less sedimentary rock is determined by its mineral composition (Schön, 2015). The physical property test includes (A) Sample’s normal weight (Wn), (b) Saturated Sample Weight (Ww) (c) Saturated *Sample* weight in water (Ws) (d) dry weight (Wo) with temperature ≥110 ºC for 24 hours in an oven.

Atterberg limit test

ASTM D 4318-95b is a standard test method to determine the Atterberg limit, namely liquid limit, plastic limit, and plasticity index (Megel et al., 2006). This method is usually used for engineering and geological application (Knadel et al., 2021).

a. Liquid Limit Test
Liquid limit describes soil water content between liquid and plastic conditions, determined using the Casagrande test (Rock, 2010; Díaz et al., 2021). Soil liquid limit may decrease due to the sampling process and drying process from 60 °C to 110 °C (Huvaj and Uyeturk, 2018).

b. Plastic Limit Test
The plastic limit test is a numerical method used to analyze slope strength (Li et al., 2019) because the soil plastic limit is associated with the soil density characteristic, which helps assess the natural soil promptly (Nagaraj et al., 2015). The plastic limit test is performed by pressing and rolling on the clean glass until 3 mm diameter before cracking, and then weighed and put into the oven for 24 hours.

c. Plasticity Index
Plasticity Index (PI) represents the difference between liquid and plastic limit values. Soil with a high PI value indicates high clay particles (Coduto et al., 2010; Das and Sivakugan, 2018).

Table 1. Plasticity index classification (Coduto et al., 2010).

No	Plasticity index	Description
1	0	Non-plastic
2	1 - 5	Slightly plastic
3	5 - 10	Low Plasticity
4	10 - 20	Medium Plasticity
5	20 - 40	Hight Plasticity
6	>40	Very Hight Plasticity

Direct shear test

The direct shear test is the most common method to test the shear strength of discontinuity of rock, aiming at measuring the peak and residual direct as the normal stress function on the shear plane and finding out the limit of rock in withstanding the shear of burden (Sanei et al., 2015). During the test process, the rock sample was imposed by a certain normal force (Fn) applied perpendicular to the discontinuous plane and friction (Fs) until the rock cracked. The normal stress in each specimen can be viewed as the quotient result from the normal burden and plane area before shear (Li et al., 2015). Cohesion and internal friction angle were determined using different shear stresses, following Mohr-Coulomb's failure criteria (Tan et al., 2019).

Results

The rock physical properties

The result of physical property test used to analyze the safety of the disposal slope was the density value since density is the most important character for geomechanical analysis (Yusuf et al., 2019). It is difficult to accurately determine the density of rock materials because of changing volume, porosity, water content, and permeability (Crawford, 2013). The density test of rock for disposal slope in each location is presented in Table 2. Data presented in Table 2 show that the density of claystone and siltstone in the Balikpapan formation is lower than that of the Pulaubalang formation because the porosity value in the Balikpapan formation is greater than that of the Pulaubalang formation.

Table 2 Average rock density.

Location	Dry Density (g/cm^3)	Saturated Density (g/cm^3)	Porosity (%)
01	0.880	1.267	63.32
02	1.573	1.612	27.27
03	1.540	1.593	28.85
04	1.348	1.511	41.78

Atterberg limit test

The Atterberg limit is the consistency limit of fine-grained soil with consideration of the water content of the soil as the liquid limit, plastic limit, and shrinkage limit (O'Kelly et al., 2018). Atterberg limit is used to determine how far a material can swell or shrink (Sivakumar et al., 2009). The result of the Atterberg limit test is displayed in Table 3 and Figure 1.

Table 3. Atterberg limit.

Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
01	35.70	26.98	8.72
02	26.50	14.72	11.78
03	34.01	26.00	8.01
04	42.70	28.78	13.92

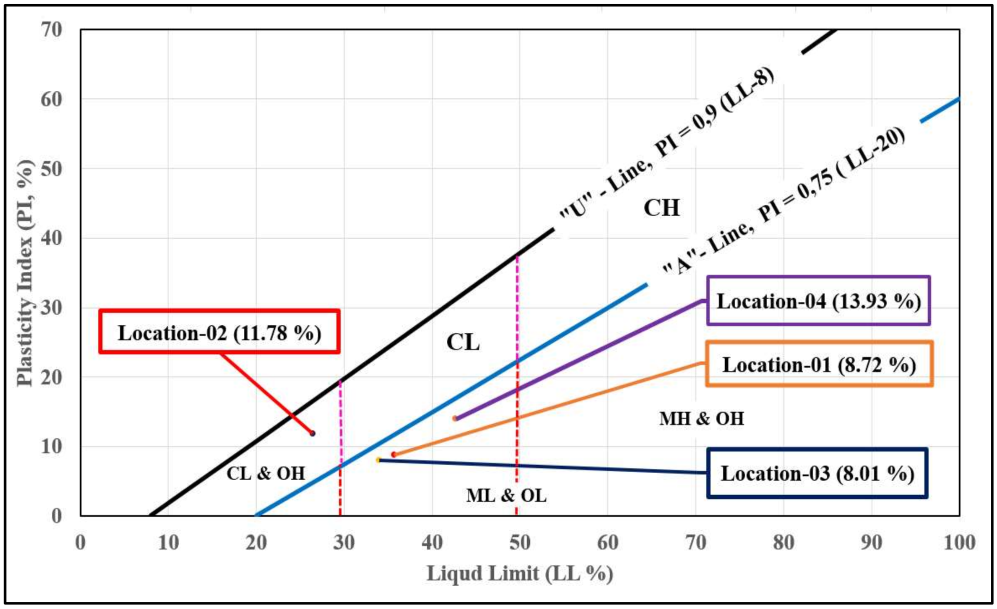


Figure 1. Plasticity chart.

The graph shows that the claystone in the Balikpapan formation is included in organic silt and silty clay with low plasticity. The claystone of the Palaubalang formation is included in the category of organic silt and organic silty clay with low plasticity. The siltstone of the Palaubalang formation is included in non-organic clay with low to moderate plasticity, gravel clay, sandy clay, silty clay, and lean clays. At a clay content of less than 40%, the plasticity index of soil containing a specific type of clay will be directly proportional to the clay content.

Direct shear test

Rock shear strength refers to the rock's ability to withstand the burden, either constant or dynamic burden. The peak shear stress and residual shear strength can be calculated using a linear failure envelope that can be described using Mohr-Coulomb Criteria (Anubhav and Basudhar, 2010). The shear stress, cohesion, and internal friction angle values are presented in Tables 4 and 5. Table 5 shows that the higher the cohesion value, the lower the internal shear angle, and the lower the cohesion value, the faster the rock will reach a plastic condition.

Table 4. Shear stress average.

Location	F Shear (kN)		σ_n (MPa)	τ (MPa)	
	Peak	Residual		Peak	Residual
01	1.075	0.313	0.056	1.123	0.328
02	0.988	0.425	0.056	1.070	0.463
03	0.888	0.425	0.056	0.884	0.427
04	1.150	0.338	0.056	1.220	0.359

Table 5. Cohesion and internal friction angle.

Location	Curve	Cohesion (MPa)	Internal Friction Angle (°)	UCS (MPa)	UTS (MPa)
01	Peak	0.8972	76.180	14.800	0.220
	Residual	0.1140	65.682	1.870	0.030
02	Peak	0.7111	81.200	18.490	0.110
	Residual	0.0953	71.576	2.760	0.020
03	Peak	0.3711	83.810	13.736	0.040
	Residual	0.0196	72.512	0.646	0.003
04	Peak	1.0168	74.680	15.120	0.270
	Residual	0.1461	65.329	2.350	0.050

Note : UCS = Unconfined Compressive Strength, UTS = Ultimate Tensile Strength.

Discussion

Disposal geometric planning

The rock properties (physical and mechanical properties) are two important factors determining slope stability (Hustrulid et al., 2000). It is difficult to determine the parameter value of slope stability due to the heterogeneity of rock mass. However, it is possible to estimate the general description of the physical and geometric characteristics of the slope. Therefore, measuring slope stability using the limit equilibrium method needs to determine the critical form and location that suits the surface (Ahangar-Asr et al., 2010). The slope geometry is designed to be 10, 15, 20, and 25 m high with 10°, 15°, 20°, 25°, 30°, 35°, and 40° angles. The safety of each height and angle is calculated using an exponential equation. The result is presented in Table 6. The table represents the simulation of slope stability analysis for each location in terms of slope steepness. The simulation shows that each location exhibits different safety levels, depending on the angle. The safety in Locations 04 and 02 tends to drop drastically as the inclination increases, while locations 01 and 03 tend to drop slowly. This indicates that rocks with a lower plasticity index tend to be stronger on a steeper slope.

Table 6. Safety factor.

Location	Angle (°)	Height (m)			
		10	15	20	25
1	10	6.528	5.063	4.423	4.007
	15	6.026	4.686	3.975	3.508
	20	5.820	4.440	3.669	3.178
	25	5.671	4.172	3.381	2.854
	30	5.467	3.891	3.071	2.580
	35	5.164	3.623	2.788	2.305
	40	4.895	3.350	2.499	2.025
2	10	8.183	7.081	6.473	6.075
	15	7.210	6.027	5.382	4.973
	20	6.493	5.243	4.615	4.168
	25	5.856	4.670	3.973	3.559
	30	5.364	4.151	3.435	3.044
	35	4.887	3.630	2.978	2.581
	40	4.410	3.186	2.561	2.164
3	10	8.182	6.589	5.835	5.362
	15	7.540	6.026	5.188	4.655
	20	7.222	5.604	4.726	4.161
	25	6.940	5.226	3.343	3.754
	30	6.615	4.879	3.946	3.404
	35	6.273	4.555	3.617	3.069
	40	5.943	4.226	3.284	2.756
4	10	5.396	5.042	4.868	4.756
	15	4.198	3.808	3.613	3.481
	20	3.355	3.005	2.761	2.616
	25	2.730	2.330	2.105	1.958
	30	2.163	1.775	1.552	1.415
	35	1.683	1.290	1.082	0.925
	40	1.232	0.855	0.655	0.488

Disposal geometric design

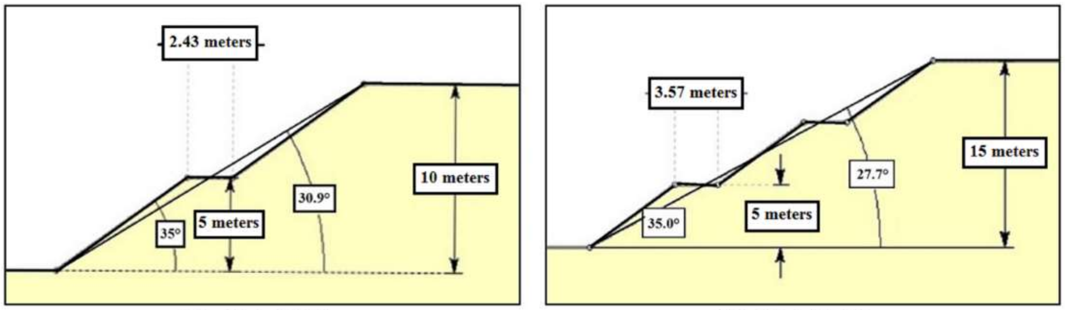
The slope geometry has three important components affecting slope stability: bench height, overall slope angle, and surface area, which are the basis for designing slope geometry (Chaulya and Prasad, 2016). One of the most common methods to analyze slope stability is the simplified Bishop method for its acceptability and plausibility (Cho and Song, 2014).The disposal design was taken based on the weakest rock type by considering several aspects:

a) Less optimal dumping causes the pore to be filled by water quickly, resulting in suboptimal water content in dump material.
b) The weak disposal material, i.e., clay shale.
c) Heavy equipment activity during the reclamation process.
d) Weak disposal ground floor.

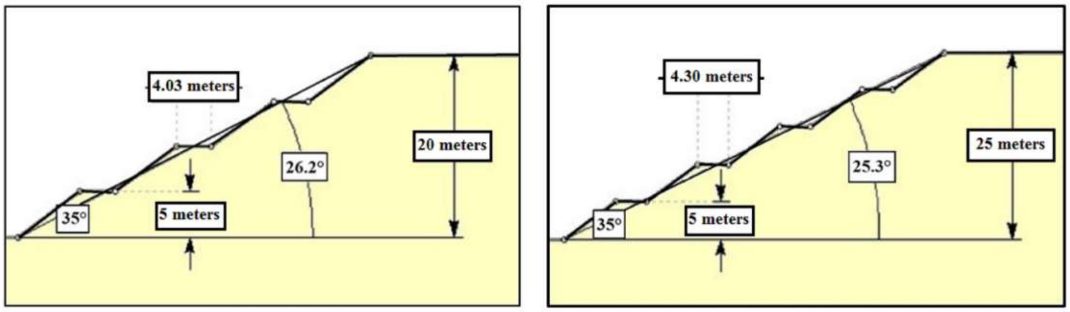
Based on the physical property and direct shear test results, the weakest material for designing the disposal slope was siltstone in the Balikpapan formation (location-04), as presented in Table 7 and Figure 2. As shown in Figure 2, the slope geometry (i.e., overall slope angle and slope height) exhibited the most significant effect on the slope stability at the height of 25 m and the overall slope angle of 25.3°. The slope geometry was considered stable and safe (SF= 4.30) and withstood the disposal burden.

Table 7. Disposal slope geometry design.

Slope Height (m)	Overall slope (°)	Bench Height (m)	Single slope (°)	Berm Width (m)
10	30.9	5	35	2.43
15	27.7	5	35	3.57
20	26.2	5	35	4.03
25	25.3	5	35	4.30



(a) Height 10 m (b) Height 15 m



(c) Height 20 m (d) Height 25 m

Figure 2. Disposal slope design.

Conclusion

Cohesion and internal friction angle in Atterberg Limits were found to affect the safety value of the slope design indirectly, and rocks with lower plastic index tend to be stronger to withstand the steeper condition. A safe slope design (SF≥2), based on the result of this study, should be 10°-30° high overall, overall slope of 24.4°-30.1°, berm width of 2.43-4.66 m, and bench height of 5 m. The most effective slope design is the slope with 25 m high, an overall slope of 25.3°; a single slope of 35°; berm width of 4.66 m; bench height of 5 m with a safety factor value of 4.30.

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