Slope stability and bearing capacity analysis of disposal in open-pit coal mining

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Abstract. Open-pit mine is a mining method by excavating overburden strata to get coal seam near the surface. One of potential hazard often occur in mining area is slope failure. The hazard can be prevented by performing geotechnical study. The study is required to provide recommendations regarding safe design of slope geometries in both open-pit and disposal sites. This paper produce information related to the results of slope stability analysis especially on the disposal embankment. The analysis is performed by using the limit equilibrium method. It is also produce the bearing capacity analysis of subsurface strata in disposal site. The analysis is carried out to determine the ability of the strata to burden the load of disposal embankment above. The location was selected in a site of open-pit coal mine in Indonesia. The results will become consideration for management in designing slope disposal and choosing the location of disposal facilities.

1. Introduction

Open-pit coal mining needs to excavate the rock above the coal seam target called overburden [1]. One of the potential hazards that often occurs is the instability of mine slopes and disposal areas. The disposal area has planned to accommodate unused or overburden material [2]. Slope geometry planning in the disposal area needs based on the results of the subsurface investigation. This investigation aims to determine the type of soil, obtain disturbed and undisturbed samples for visual identification and soil testing in the laboratory, determine the depth of bedrock, observe drainage conditions, and determine the location of the groundwater table [3]. Furthermore, the method used in slope stability analysis has generally based on Limit Equilibrium Method (LEM). In this method, the Safety Factor (SF) values have been estimated based on equilibrium condition when it is beginning to calculate the failure on the specified failure plane, then compared to the strength required to maintain stability against the bodyweight of soil or rock.

Previous studies have done carried out to analyze the stability of disposal slopes using LEM, such as to engineer an increase embankment height in low-wall areas conducted by Andriyan et al. (2018) [4]. Calculation results from this study can increase the embankment height up to 100 m with an angle of 12°, and the SF value increases from 0.63 to 1.3. The change in slope geometry increases the storage volume to 8,269,529.9 LCM.

The back analysis method had used to determine the properties of mine waste dump material after a failure [5]. He also researched the effect of water balance on the stability of mine waste dumps, especially in sloping basements. In-situ test and indoor rock experiment on the waste rocks from the landfill and establishing 3D simulation models have been done based on an actual mine project in a specific open-pit mine waste dump in Tibet [6]. The Bishop method for analyzing dumping sequences
at elevations 89 – 94, 94 – 99, 99 – 104, and 104 – 109 mdpl has to do by Majid et al. (2017) [7]. These sequences make it easier to control the embankment design. The calculation results in this study obtained the SF value of 1,832 and the addition of embankment capacity to 2,668,901.50 CCM.

Comparison of two methods, namely LEM and FEM (finite element method), has been carried out to compare the SF value obtained from the two ways [8]. The calculation results from this study indicate that the predicted failure pattern and SF values tend to be the same, with a difference of 5%. The LEM method also is used to determine the stability of the coal mine disposal slopes in the Purwaraja District [9] and to estimate the stability and risk assessment of dump slope at Bhubaneswari open cast mine [10]. Whereas artificial neural network (ANN) is used to analyze and evaluate the stability of dump slope using slope profile, geotechnical and hydrological parameters [11]. Another researcher deals with a case study of failed dump slope in western coalfield limited, Nagpur, India [12].

The research area in this article is located on the area of coal mining company PT. X in East Kalimantan. The purpose of this recent study is to assess slope stability in the disposal area by slope stability simulations and also obtain the optimum safety factor number and safe dimension of slope disposal. The slope design based on safe dimension can construct embankment for the best dumping scenario suitable with the condition in the site.

2. Methodology

Data analysis uses the LEM and Spencer slice method because it meets equilibrium conditions for forces and moments, assuming that the failure plane is circular. Based on the study of Hoek and Bray, 1981, it is stated that disposal material in dumping areas with a large dimension will undergo failure in the circular pattern, that in this case, the geological structure does not significantly affect the stability of disposal [13]. The slope stability depends on the characteristics of dumping material, the slope dimensions, groundwater conditions, and external factors that influence it [14]. Laboratory tests were carried out on core samples using residual shear tests to gain cohesion and internal friction angle parameters and physical examination to gain density. These three parameters will be input in slope stability analysis with the support of software based on LEM to receive a safe dimension of a slope.

2.1. Safety Factor and Slope Stability

The basic principle of slope stability analysis simply include two matter, namely: resisting forces (strength of slope to prevent failure) and driving forces (forces that cause failure). The stability of slope can be calculated based on the ratio of resisting forces and driving forces [13]. If resisting forces is greater than driving forces, the slope will be stable, so as to if driving forces is greater than resisting forces, the slope will be unstable. Driving forces can be generated from gravity force, vibrations generated from blasting activities as well as heavy equipment an also from earthquakes. Resisting forces determined from cohesion and internal friction angle of slope material.

In determining the level of stability, the term of Safety Factor (SF) is used. The formula can be seen below.

\[
SF = \frac{\text{Resisting Forces}}{\text{Driving Forces}}
\]

where \( c \) is cohesion; \( A \) is area \( W \) is load weight above failure plane; \( \alpha \) is slope angle and \( \phi \) = internal angle

Safety factor can be adjusted according to the needs as stated in Bowles, 1997 [15] (Table 1)

| Safety Factor | Kondisi                     |
|---------------|-----------------------------|
| SF < 1,07     | Unstable (failure able to occur) |
| 1,07 < SF < 1,25 | Critical (failure once occur)         |
| SF > 1,25     | Stable (failure rarely occur)     |
2.2. LEM
LEM is a prevalent method for analyzing the stability of slope with the translational and rotational slip types. In this method, the calculation in slope stability analysis just used static equilibrium conditions and ignored the stress-strain relationship on the slope. Another assumption is that we must determine the geometry of the failure plane first. The slope stability condition in the limit equilibrium method is expressed in SF number calculated using force equilibrium, moment equilibrium, or both two equilibrium conditions.

The calculation is performed by dividing disposal material above the failure plane into slices, which is also known as the slice method. The slice method, which Fellenius first published, is the most straightforward technique in which all inter-slice forces are ignored and only consider moment equilibrium [8]. Furthermore, Bishop develops a more complex approach by putting forces around slice area and considering moment equilibrium.

2.3. Bearing Capacity Analysis
Bearing capacity is the ability of foundation layer to support the structures upon it without experiencing failures. This capacity needs to be analyzed so that the sub-base layer does not experience shear failure and excessive settlement. It is determined by the type and characteristic of foundation layer, and is also influenced by the shear strength of foundation layer that comprises of cohesion and friction angle. If the shear stress acting on a layer then simultaneously the normal stress (σ) will work, the shear stress (τ) will increase when deformation reach the limit. If the limit values are connected with different normal stress (σ), then a straight line will be obtained where cohesion as a constant and normal stress (σ) as a variable, and the slope of the line is determined as the friction angle. Then, it can be written in the equation as follows:

\[ \tau = c + \sigma \tan \phi \]

Where \( \tau \) is shear stress (kg/cm²); \( c \) is cohesion (kg/cm²); \( \sigma \) is normal stress (kg/cm²) and \( \phi \) is angle of friction (°)

From the equation above, cohesion (c) is obtained from the magnitude of cohesive between soil grains while resistance to soil particles termed as friction angle (φ). Both parameters can be determined from laboratory testings. Terzaghi, in Das (2018) [3], introduced the formula of bearing capacity of foundation layer which is calculated as the ultimate bearing capacity (\( q_{ult} \)), that is an ultimate condition of the capacity itself. If it is exceeded, it will cause failure of foundation layer. Therefore, the value of the allowable bearing capacity (\( q_a \)) must be less than the value of the ultimate bearing capacity (\( q_{ult} \)). Allowable bearing capacity (\( q_a \)) depends on the selected safety factor (F). Commonly, the value of safety factor is about 2 to 5, and the value of the allowable bearing capacity is calculated with the formula below.

\[ q_a = \frac{q_{ult}}{F} \]

Where: \( q_a \) is allowable bearing capacity (kN/m²); \( q_{ult} \) is ultimate bearing capacity (kN/m²); and \( F \) is safety factor

Furthermore, the calculation of the ultimate bearing capacity (\( q_{ult} \)) refers to the general equation of the bearing capacity of foundation layer below.

\[ q_{ult} = cN_c + qD_fN_q + 0.5\gamma BN_y \]

In which: \( q_{ult} \) is ultimate bearing capacity (kN/m²); \( c \) is cohesion (kN/m²); \( q \) is \( \gamma \times D_f \) (density × depth); \( D_f \) is depth of foundation, measured from ground surface; \( \phi \) is internal friction angle (°) and \( N_c, N_q, N_y \) is bearing capacity factors (Figure 1).
Terzaghi also developed a formula for the influence of shape factor of foundation on the general equation of ultimate bearing capacity \( q_{ult} \) and categorized into two groups:

1. Square footing, refers to the formula below:

\[
q_{ult} = 1.3cN_c + \gamma D_f N_q + 0.4\gamma BN_y
\]

2. Circular footing, refers to the formula below:

\[
q_{ult} = 1.3cN_c + \gamma D_f N_q + 0.3\gamma BN_y
\]

3. Results and Discussion

This study uses primary data from geotechnical drilling at Pit 4 and Pit 5. GT-03 borehole are located in Pit 5 while GT-09 borehole are located in Pit 4.

3.1. Results

Geotechnical drilling at both two boreholes, GT-03 and GT-09 located in Pit 5 dan Pit 4, have resulted in core samples which were then sent to the geomechanics laboratory to obtain parameters such as physical and mechanical properties of the rocks that compose the slope. The laboratory testing results become one of the inputs for the rock properties of all slope-forming layers. The geomechanical parameters of intact rock in both pits can be seen in Tables 2 and 3.

**Table 2. The geomechanical parameters of rock in GT-03 borehole**

| Layer | Material          | Density  | Compressive Strength | Modulus of Elasticity | Poisson’s Ratio | Tensile Strength | Peak Cohesion | Peak Angle of Friction | Residual Cohesion | Residual Angle of Friction |
|-------|-------------------|----------|----------------------|-----------------------|----------------|-----------------|---------------|-------------------------|------------------|--------------------------|
|       |                   | KN/m³    | MPa                  | MPa                   | -              | MPa             | MPa           | °                       | MPa              | °                        |
| 1     | Claystone 1       | 18.25    | 2.09                 | 328.35                | 0.33           | 1.17            | 0.30          | 38.69                   | -                | -                        |
| 2     | Sandstone 1       | 23.94    | 3.64                 | 1045.64               | 0.46           | 2.10            | 1.20          | 49.65                   | 0.06             | 18.26                    |
| 3     | Claystone 2       | 24.62    | 3.79                 | 1764.47               | 0.45           | 0.97            | 0.48          | 51.64                   | 0.08             | 18.98                    |
| 4     | Siltstone 1       | 24.82    | 19.47                | 1947.62               | 0.34           | 2.85            | 4.83          | 39.75                   | 0.10             | 28.40                    |
| 5     | Sandstone 2       | 24.03    | 15.71                | 4539.22               | 0.37           | 2.71            | 2.08          | 56.69                   | 0.07             | 27.64                    |
| 6     | Claystone 3       | 23.84    | 2.33                 | 1488.05               | 0.27           | 1.69            | 0.38          | 39.41                   | 0.10             | 21.86                    |
| 7     | Coal 1            | 12.65    | 2.92                 | 424.36                | 0.39           | 1.44            | -             | -                       | -                | -                        |
| 8     | Claystone 4       | 22.96    | 13.50                | 1538.09               | 0.33           | 2.39            | 3.93          | 43.14                   | 0.05             | 32.97                    |
| 9     | Claystone 5       | 23.35    | 9.35                 | 1467.95               | 0.39           | 1.70            | 0.69          | 56.75                   | 0.04             | 21.85                    |
| 10    | Claystone 6       | 24.03    | 8.88                 | 1367.45               | 0.44           | 2.36            | 0.98          | 43.82                   | 0.05             | 25.90                    |
| 11    | Claystone 7       | 25.80    | 4.84                 | 1171.30               | 0.45           | 1.72            | 0.96          | 49.05                   | 0.10             | 20.10                    |
| 12    | Claystone 8       | 12.16    | 2.14                 | 828.20                | 0.40           | 1.26            | 0.64          | 53.45                   | -                | -                        |
| 13    | Claystone 9       | 25.21    | 14.37                | 2863.75               | 0.47           | 2.81            | 3.50          | 42.80                   | 0.05             | 29.33                    |
| 14    | Claystone 10      | 27.22    | 17.30                | 1605.65               | 0.47           | 2.71            | 3.55          | 49.47                   | 0.12             | 25.28                    |
| 15    | Claystone 11      | 24.62    | 8.24                 | 1605.65               | 0.27           | 2.06            | 2.64          | 19.52                   | 0.03             | 30.39                    |

**Figure 1.** Values of bearing capacity factors [16]
Based on Table 2, the average density of claystone, sandstone, siltstone are 23.41 kN/m³, 23.36 kN/m³, 24.47 kN/m³, respectively while the average compressive strength of claystone, sandstone, siltstone are 7.18 MPa, 13.36 MPa, 108.61 MPa, respectively. In peak condition, the average cohesion of claystone, sandstone, siltstone are 1.46 MPa, 2.28 MPa, 2.82 MPa, respectively while the average friction angle of claystone, sandstone, siltstone are 45.93°, 51.94°, 34.36°, respectively. In residual condition, the average cohesion of claystone, sandstone, siltstone are 0.07 MPa, 0.08 MPa, 0.06 MPa, respectively while the average friction angle of claystone, sandstone, siltstone are 16.09 MPa, 14.04 MPa, 4.80 MPa, respectively. In peak condition, the average cohesion of claystone, sandstone, siltstone are 1.46 MPa, 2.28 MPa, 2.82 MPa, respectively while the average friction angle of claystone, sandstone, siltstone are 45.93°, 51.94°, 34.36°, respectively. In residual condition, the average cohesion of claystone, sandstone, siltstone are 0.07 MPa, 0.08 MPa, 0.06 MPa, respectively while the average friction angle of claystone, sandstone, siltstone are 16.09 MPa, 14.04 MPa, 4.80 MPa, respectively.

| Layer | Material    | Density (kN/m³) | Compressive Strength (MPa) | Modulus of Elasticity (MPa) | Poisson’s Ratio | Tensile Strength (MPa) | Peak Cohesion (MPa) | Peak Angle Of Friction ° | Residual Cohesion (MPa) | Residual Angle Of Friction ° |
|-------|-------------|-----------------|----------------------------|-----------------------------|----------------|------------------------|---------------------|-------------------------|---------------------------|----------------------------|
| 1     | Claystone 1 | 23.84           | 3.54                       | 470.39                      | 0.23           | 1.21                   | 1.08                | 23.10                   | 0.13                      | 4.29                       |
| 2     | Sandstone 1 | 23.64           | 2.59                       | 918.40                      | 0.36           | 1.72                   | 1.28                | 49.61                   | 0.08                      | 14.83                      |
| 3     | Carb. Claystone | 23.05       | 3.42                       | 809.70                      | 0.38           | 1.94                   | 1.35                | 46.45                   | 0.05                      | 10.47                      |
| 4     | Siltstone 1 | 24.23           | 3.68                       | 1008.23                     | 0.43           | 1.49                   | 1.43                | 23.60                   | 0.06                      | 8.73                       |
| 5     | Coal 1      | 14.22           | 11.56                      | 1733.43                     | 0.34           | 1.58                   | 1.95                | 52.34                   | 0.09                      | 12.82                      |
| 6     | Sandstone 2 | 21.78           | 11.77                      | 1711.46                     | 0.38           | 2.72                   | 3.98                | 42.25                   | 0.11                      | 9.49                       |
| 7     | Sandy Claystone | 23.35         | 54.21                      | 9135.88                     | 0.40           | 3.51                   | 8.41                | 52.82                   | 0.11                      | 10.04                      |
| 8     | Siltstone 2 | 24.43           | 5.61                       | 2627.32                     | 0.27           | 2.32                   | 0.92                | 46.90                   | 0.02                      | 27.61                      |
| 9     | Sandstone 3 | 23.15           | 12.93                      | 1771.21                     | 0.45           | 1.77                   | 4.44                | 50.89                   | 0.12                      | 15.59                      |
| 10    | Siltstone 3 | 25.02           | 5.10                       | 1720.06                     | 0.36           | 2.52                   | 2.32                | 46.22                   | 0.08                      | 13.99                      |
| 11    | Sandstone 4 | 21.78           | 23.13                      | 2296.90                     | 0.33           | 1.47                   | 4.08                | 35.27                   | 0.12                      | 11.25                      |
| 12    | Clayey Sandstone | 24.23       | 19.78                      | 2962.39                     | 0.36           | 3.33                   | 8.53                | 39.79                   | 0.03                      | 24.39                      |
| 13    | Coal 2      | 14.52           | 11.63                      | 1342.93                     | 0.37           | 1.79                   | 1.76                | 31.03                   | 0.07                      | 21.77                      |
| 14    | Claystone 2 | 23.74           | 3.18                       | 923.33                      | 0.34           | 1.44                   | 1.37                | 28.91                   | 0.08                      | 8.90                       |
| 15    | Coal 3      | 12.46           | 6.45                       | 977.78                      | 0.38           | 1.62                   | 2.55                | 16.21                   | 0.07                      | 19.41                      |

Based on Table 3, the average density of claystone, sandstone, siltstone are 23.47 kN/m³, 22.89 kN/m³, 24.53 kN/m³, respectively while the average compressive strength of claystone, sandstone, siltstone are 16.09 MPa, 14.04 MPa, 4.80 MPa, respectively. In peak condition, the average cohesion of claystone, sandstone, siltstone are 2.80 MPa, 4.46 MPa, 1.56 MPa, respectively while the average friction angle of claystone, sandstone, siltstone are 37.82°, 43.56°, 38.91°, respectively. In residual condition, the average cohesion of claystone, sandstone, siltstone are 0.09 MPa, 0.09 MPa, 0.05 MPa, respectively while the average friction angle of claystone, sandstone, siltstone are 8.43°, 15.11°, 16.78°, respectively.

3.2. Discussion

Recently, there is a method in determining parameters of disposal especially the strength parameters of the material. The method proposed by Hoek E and Bray J, 1981 and the detail can be seen in example 3 [13]. In this article, an empirical approach is proposed to predict the parameter. The approach is intended to estimate one equivalent parameter that can represent the admixture material of disposal. Then, equivalent parameters used as input parameter for disposal material.

3.3. Determination of Disposal Density

The type of material to be dumped consist of claystone, siltstone and sandstone, assuming a mixture ratio of 30% : 30% : 40%, can be seen in Table 4.
Table 4. Composition of claystone, siltstone, sandstone in a disposal material on Pit 5 (GT-03) and Pit 4 (GT-09)

| Lithology | Pit 5 (GT-03) | Pit 4 (GT-09) |
|-----------|---------------|---------------|
|           | Percentage (%) | Density (kN/m³) | Percentage (%) | Density (kN/m³) |
| Claystone | 30            | 23.41         | 30            | 23.47         |
| Siltstone | 30            | 24.47         | 30            | 24.53         |
| Sandstone | 40            | 23.36         | 40            | 22.89         |

Equivalence density

\[ \text{Equivalent density}_{\text{in situ}} (\text{GT-03}) = (0.3 \times 23.41) + (0.3 \times 24.47) + (0.4 \times 23.36) = 23.71 \text{ kN/m}^3 \]

\[ \text{Equivalent density}_{\text{in situ}} (\text{GT-09}) = (0.3 \times 23.47) + (0.3 \times 24.53) + (0.4 \times 22.89) = 23.56 \text{ kN/m}^3 \]

Determination of equivalent density of disposal material is achieved by using the swell factor of mixed material as 85% (Peurifoy, et al., 2018), then the density of the material is as follows:

Density of embankment (compacted) on Pit 5 (GT-03) = 0.85 \times 23.71 = 20.15 \text{ kN/m}^3 = 2.06 \text{ gr/cm}^3

Density of embankment (compacted) on Pit 4 (GT-09) = 0.85 \times 23.56 = 20.03 \text{ kN/m}^3 = 2.04 \text{ gr/cm}^3

3.4. Determination of Internal Friction Angle

Internal friction angle is determined by using a graph based on the grain size. The graph being used is as follows. The graph used to specify internal friction angle of coarse-grained type like sandstone is a graph that correlates density and internal friction angle in Figure 2. Of this graph, the friction angle (\( \phi \)) of disposal material is derived as 45°. Based on Table 4 (Pit 5), in peak condition the average friction angle of sandstone is 51.94° while in residual condition the average friction angle of sandstone is 23.73°. Based on Table 4 (Pit 4), in peak condition the average friction angle of sandstone is 43.56° while in residual condition the average friction angle of sandstone is 15.11°. It can be seen that in peak condition the predicted friction angle of this method is near equivalent with the laboratory friction angle (45°~51.94° and 45°~43.56°) while in residual condition the average friction angle of this method is higher than the laboratory friction angle (45°>23.73° and 45°>15.11°).

Figure 2. Correlation between friction angle versus unit weight [15]

The graph used to specify internal friction angle of fine-grained type like claystone and siltstone is a graph that correlates PI (Plasticity Index) and internal friction angle in Figure 3. In general, the PI number...
of fine-grained material has a value between 15 – 25 %. This study used a value of 15% based on the value at the minimum condition. From this graph, the number of friction angle (\(\phi\)) of the disposal material is derived as 15°.

![Figure 3. Correlation between friction angle versus plasticity index Ip [15]](image)

The ratio of claystone, siltstone and sandstone being dumped on disposal area is 30 : 30 : 40, so that the number of friction angle of disposal material is received, and termed as \(\phi_{\text{waste}} : \phi_{\text{waste}} = (0.3 \times 15^\circ) + (0.3 \times 15^\circ) + (0.4 \times 45^\circ) = 27^\circ\).

Based on Table 2, in peak condition the number of friction angle of disposal material and termed as \(\phi_{\text{waste}}\) can be calculated as: \(\phi_{\text{waste}} = (0.3 \times 45.93^\circ) + (0.3 \times 34.36^\circ) + (0.4 \times 51.94^\circ) = 44.86^\circ\), while in residual condition \(\phi_{\text{waste}}\) can be calculated as: \(\phi_{\text{waste}} = (0.3 \times 24.18^\circ) + (0.3 \times 28.23^\circ) + (0.4 \times 23.73^\circ) = 25.22^\circ\).

Based on Table 3, in peak condition the number of friction angle of disposal material and termed as \(\phi_{\text{waste}}\) can be calculated as: \(\phi_{\text{waste}} = (0.3 \times 37.82^\circ) + (0.3 \times 38.91^\circ) + (0.4 \times 43.56^\circ) = 40.44^\circ\), while in residual condition \(\phi_{\text{waste}}\) can be calculated as: \(\phi_{\text{waste}} = (0.3 \times 8.43^\circ) + (0.3 \times 16.78^\circ) + (0.4 \times 15.11^\circ) = 13.61^\circ\).

It can be seen that in peak condition the predicted friction angle, \(\phi_{\text{waste}}\), of this method is lower than the laboratory friction angle (27°<44.86° and 27°<40.44°) while in residual condition the average friction angle, \(\phi_{\text{waste}}\), of this method is near equivalent with the laboratory friction angle (27°~25.22° and 27°~13.61°).

3.5. Determination of Disposal Cohesion

Determination of cohesion number of disposal material is established by calculating 60% of minimum cohesion parameter of claystone, siltstone and sandstone. Based on Table 2, in peak condition the average cohesion of claystone, sandstone, siltstone are 1.46 MPa, 2.28 MPa, 2.82 MPa, respectively while in residual condition, the average cohesion of claystone, sandstone, siltstone are 0.07 MPa, 0.08 MPa, 0.06 MPa, respectively. Based on Table 3, in peak condition the average cohesion of claystone, sandstone, siltstone are 2.80 MPa, 4.46 MPa, 1.56 MPa, respectively while in residual condition, the average cohesion of claystone, sandstone, siltstone are 0.09 MPa, 0.09 MPa, 0.05 MPa, respectively. The minimum cohesion in peak condition are 1.46 MPa and 1.56 MPa while in residual condition are 0.06 MPa and 0.05 MPa.

The cohesion number of disposal material is received as follow: \(C_{\text{waste}}\) in peak condition = 1.46 × 0.6 = 0.88 MPa. \(C_{\text{waste}}\) in residual condition = 0.05 × 0.6 = 0.03 MPa.
3.6. Modelling of Disposal Embankment
In modelling the slope disposal, the approaches taken are as follows:

1. Single slope of disposal embankment is constructed with 2 (two) variables as:
   - Height of slope : 5, 10 meter
   - Angle of slope : 10°, 20°, 30°, 40°

2. Overall slope of disposal embankment is constructed with 2 (two) variables as:
   - Height of slope : 10, 15, 20 meter
   - Angle of slope : 30°, 40°

3.7. Slope Stability Analysis of Disposal
The slope stability analysis of disposal embankment was carried out by using limit equilibrium method-based software. The models being analyzed were eight models for single slope and 6 (six) models for overall slope. The result of analysis is Safety Factor (SF) number with several variations of slope angle and embankment height. SF number being used is based on Kepmen ESDM No. 1827K/2018 [17]. The SF number for single slope can be seen in Table 5 while for overall slope can be seen in Table 6. Simulation of single slope can be seen in Figure 4-B and of overall slope can be seen in Figure 4.A. Parameter correlations of angle and height of slope with Safety Factor number can be seen in Figure 5.

Table 5. Safety Factor of single slope analysis

| Slope Angle (°) | Slope Height (m) | Dynamic Safety Factor (SF) | Probability of Failure, PoF (%) |
|-----------------|------------------|-----------------------------|---------------------------------|
| 10              | 5                | 2.61                        | 0.00                            |
|                 | 10               | 1.79                        | 0.00                            |
| 20              | 5                | 2.25                        | 0.00                            |
|                 | 10               | 1.46                        | 0.00                            |
| 30              | 5                | 2.10                        | 0.00                            |
|                 | 10               | 1.27                        | 0.00                            |
| 40              | 5                | 1.93                        | 0.00                            |
|                 | 10               | 1.10                        | 0.40                            |

Table 6. Safety factor of overall slope analysis

| Single Slope Angle (°) | Overall Slope Height (m) | Overall Slope Angle (°) | Dynamic Safety Factor (SF) | Probability of Failure, PoF (%) |
|------------------------|--------------------------|-------------------------|----------------------------|---------------------------------|
| 30                     | 10                       | 22                      | 1.50                       | 0.00                            |
|                        | 15                       | 20                      | 1.23                       | 0.50                            |
|                        | 20                       | 19                      | 1.08                       | 15.50                           |
| 40                     | 10                       | 27                      | 1.42                       | 0.00                            |
|                        | 15                       | 24                      | 1.15                       | 2.60                            |
|                        | 20                       | 23                      | 0.99                       | 50.30                           |
Figure 4. Stability analysis of single and overall slope

Stability analysis of single slope with $H = 5 \text{ m}, \alpha = 40^\circ$ (A)

Stability analysis of overall slope with $H = 20 \text{ m}, \alpha = 40^\circ$ (B)

Figure 4. Stability analysis of single and overall slope
Figure 5. Correlation between single slope angle (A), overall height (B) and overall slope angle (C) versus safety factor

3.8. Bearing Capacity Analysis of Disposal Embankment

The condition of subsurface strata beneath the disposal embankment is bedding stratum so that the determination of the bearing capacity parameter must consider the condition. One of the approach being used is weighting factor [18]. The approach utilized borehole data in GT-03 of Pit 5 and GT-09 of Pit 4. Allowable bearing capacity ($q_{all}$) is specified with the formula as follow:

$$q_{all} = \frac{q_{ult}}{FS}$$

with FS number $\geq$ 2.0. In this study, FS number being used is 2.5.

The embankment load is usually assumed as uniform load ($q$) and the value is calculated as follow:

$$q = \gamma \times H$$

wherein $q$ is uniform load of embankment (kPa); $\gamma$ is density of embankment (kN/m$^3$) and H is height of embankment (m)
In this study, γ number is established based on calculation in Sub. Section 3.2.1, which is 20.15 kN/m³ in Pit 5 (GT-03) and 20.03 kN/m³ in Pit 4 (GT-09). Based on the slope stability analysis, the overall height of embankment is 20 m the overall angle of slope is 19°. The value of uniform load can be calculated as follow.

Pit 5 (GT-03) location : \( q = \gamma \times H = 20.15 \frac{kN}{m^3} \times 20 \ m = 403.0 \frac{kN}{m^3} = 403.0 \ kPa = 0.40 \ MPa \)

Pit 4 (GT-09) location : \( q = \gamma \times H = 20.03 \frac{kN}{m^3} \times 20 \ m = 400.6 \frac{kN}{m^3} = 400.6 \ kPa = 0.40 \ MPa \)

This \( q \) number is then used as a criterion if the bearing capacity number of sub surface strata beneath the disposal embankment is smaller of than this embankment load (\( q \)). If the bearing capacity is greater than the embankment load, the sub surface strata is stable in reinforcing the load above.

\[
\begin{align*}
\gamma &= 403.0 \ kPa \\
\gamma &= 400.6 \ kPa \\
H &= 20 \ m \\
B &= 650 \ m \\
q_1 &= 403.0 \ kPa \\
q_2 &= 400.6 \ kPa
\end{align*}
\]

**Figure 6.** Calculation of bearing capacity based on borehole GT-03

Weighting factor:
Layer of claystone 1 : \((9.80 - 3.50) / (44.60 - 3.50) = (9.80 - 3.50) / 41.1 = 0.15\)
Layer of sandstone 1 : \((12.00 - 9.80) / (44.60 - 3.50) = (12.00 - 9.80) / 41.1 = 0.05\)
Layer of claystone 2 : \((15.90 - 12.00) / (44.60 - 3.50) = (15.90 - 12.00) / 41.1 = 0.09\)
Layer of sandstone 2 : \((19.71 - 15.90) / (44.60 - 3.50) = (19.71 - 15.90) / 41.1 = 0.09\)
Layer of claystone 3 : \((24.50 - 19.71) / (44.60 - 3.50) = (24.50 - 19.71) / 41.1 = 0.12\)
Layer of sandstone 3 : \((26.55 - 24.50) / (44.60 - 3.50) = (26.55 - 24.50) / 41.1 = 0.09\)
Layer of claystone 4 : \((30.70 - 26.55) / (44.60 - 3.50) = (30.70 - 26.55) / 41.1 = 0.10\)
Layer of sandstone 4 : \((32.07 - 30.70) / (44.60 - 3.50) = (32.07 - 30.70) / 41.1 = 0.03\)
Layer of claystone 5 : \((41.25 - 32.07) / (44.60 - 3.50) = (41.25 - 32.07) / 41.1 = 0.22\)
Layer of siltstone : \((44.60 - 41.25) / (44.60 - 3.50) = (44.60 - 41.25) / 41.1 = 0.08\)

Determination of horizontal subsurface layer by using weighting factor:
\[
\gamma' = (0.15 \times 23.25 \ kN/m^3) + (0.05 \times 23.94 \ kN/m^3) + (0.09 \times 24.62 \ kN/m^3) + (0.09 \times 23.94 \ kN/m^3) \\
+ (0.12 \times 24.62 \ kN/m^3) + (0.05 \times 23.94 \ kN/m^3) + (0.10 \times 24.62 \ kN/m^3) + (0.03 \times 23.94 \ kN/m^3) + \\
(0.22 \times 24.62 \ kN/m^3) + (0.08 \times 24.82 \ kN/m^3) = 22.86 \ kN/m^3
\]

\[
c' = (0.15 \times 1699.38 \ kPa) + (0.05 \times 60.80 \ kPa) + (0.09 \times 78.45 \ kPa) + (0.09 \times 60.80 \ kPa) + (0.12 \times 78.45 \ kPa) + (0.05 \times 60.80 \ kPa) + (0.10 \times 1699.38 \ kPa) + (0.03 \times 60.80 \ kPa) + \\
(0.22 \times 78.45 \ kPa) + (0.08 \times 99.04 \ kPa) = 488.21 \ kPa
\]

\[
\phi' = (0.15 \times 23.26°) + (0.15 \times 18.26°) + (0.15 \times 18.26°) + (0.15 \times 18.26°) + (0.15 \times 18.26°) + (0.15 \times 24.69°) + (0.15 \times 18.26°) + (0.15 \times 18.98°) + (0.15 \times 28.40°) = 21.03°
\]
\[ q = \sigma_D' = \Sigma \gamma \cdot H = 0 \]

By using Terzaghi formula, ultimate bearing capacity \( q_{ult} \) based on GT-03 is obtained as:

\[
q_{ult} = c'N_c + \sigma_D'N_q + \frac{1}{2} \gamma'BN_y
\]

\[
= (488.21 \text{kPa} \times 12.39) + (0 \times 4.18) + (0.5 \times 22.86 \times 650 \times 1.36)
\]

\[ = 16153.2 \text{kPa} = 16.15 \text{MPa} \]

By using SF = 2.5, allowable bearing capacity \( q_{all} \) based on GT-03 is obtained as:

\[
q_{all} = \frac{q_{ult}}{SF} = \frac{16153.2}{2.5} = 6461.3 \text{kPa} = 6.46 \text{MPa}
\]

\[ q_{all} = 6461.3 \text{kPa} > 403 \text{kPa} \]

The number of \( q_{all} > 403 \text{kPa} \) state that the bearing capacity of the subsurface layer below the embankment is greater than the load of embankment yielding on it. It is clear that the disposal slope at Pit 5 is stable.

Calculation of bearing capacity based on borehole GT-09 is the same approach as in borehole GT-03. The summary of the bearing capacity calculation on both two boreholes can be seen below. The following is summary of bearing capacity analysis in both Pit 4 and 5:

| Borehole | \( \gamma' \) kN/m\(^3\) | \( c' \) kPa | \( \phi' \) degree | \( N_c \) | \( N_q \) | \( N_y \) | \( q_{ult} \) kPa | \( q_{all} \) kPa | \( q \) kPa | Condition |
|----------|-----------------|-------|----------------|------|------|------|-------------|-------------|-------|---------|
| GT-03    | 22.86           | 488.21| 21.03          | 12.39| 4.18 | 1.36 | 16153.2     | 6461.3      | 403.0 | Stable  |
| GT-09    | 23.30           | 86.64 | 10.31          | 13.68| 4.93 | 1.80 | 14818.52    | 5927.41     | 400.6 | Stable  |

4. Conclusions
From the results of slope stability analysis, the dimension of slope disposal recommended are every bench has a single height of 5 meters with single slope angle of 30\(^\circ\). The overall height of slope is 20 meters with overall slope angle is 19\(^\circ\). The slope has bench while its width is 8 meters. The load of embankment is 403.0 kPa in Pit 5 (GT-03) while in Pit 4 (GT-09), it is 400.6 kPa. From the results of bearing capacity analysis, the bearing capacity of sub surface strata is 6461.3 kPa in Pit 5 (GT-03) while in Pit 4 (GT-09) it is 5927.41 kPa. The bearing capacity number in Pit 5 is larger than 403.0 kPa while in Pit 4 it is also larger than 400.6 kPa. This means that the slope disposal is stable both on upper and beneath of the embankment.

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