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Criteria to Assess Rock Quarry Slope Stability and Design in Landslide Vulnerable Areas of Sri Lanka: A Case Study at Thalathu Oya Rock Quarry

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Abstract: The ultimate causative factor for the failure is rapid removal of toe support of the slope due to unplanned mining accompanied with uncontrolled blasting. There is also a natural causative factor behind, a naturally formed highly weathered slip surface, where along the slope failure has taken place. Secondary discontinuity created along the well-developed foliation plane due to an earlier disturbance of rock mass along kinematically more unstable joint planes, is the inception. This has turned into a weaker plane by groundwater seepage for a very long period facilitated by drainage pattern of the area. Intense weathering features of failure zone, chert particles found from the slip surface are good indications for this factor. Furthermore, it was identified that, the shear strength of rock joints can conveniently and rapidly be determined using Rock Mass Rating System and Empirical Equations. Even though these methods provide more conservative values, results will be very useful in initial design work. Results show that the back analysis method is more reliable compared to above two but is conditional as a similar type of a failure need to occur in the same rock mass in order to employ this method. Moreover, it was revealed that Barton's theorem can be effectively applied for local rock masses in determining the shear strength of discontinuities and is reliable in using at lower stress levels. When considering the stability of remaining slopes of the same site, these are highly venerable for same type of failure at any moment. According to site geometrical parameters and shear strength parameters found out from back analysis reveals that the natural factor of safety is only around 1.0 for slopes that remain hanging at this site. Further, study reveals that, the most economical method of stabilizing these existing unstable areas in the site is by reduction of the slope height with the use of controlled blasting techniques.

Keywords: Back analysis, empirical equations, Rock Mass Rating System

1. Introduction

Introduction

Landslides hazard is a major problem in the Central and Sabaragamuwa provinces of Sri Lanka, where high rainfalls are experienced throughout the year. Increase of pore water pressure in joints and discontinuities of rock masses and subsequent reduction in effective stress cause to reduce the shear strength of the failure plane. Furthermore, human activities such as removal of vegetation cover on steeply dipping terrains for agricultural purposes cause to trigger slope failures in overburden areas due to removal of roots network of the vegetation. Worst conditions occur when insufficient drainage is provided, where subsequent stagnation of water remaining at the crest of the slope. Bench and associated slope failures in open cast mines (especially in Rock quarries) sometimes can be turned into disasters. The main causative factors behind are the ignorance of geological structural features of rock mass, careless violation of rules and regulating conditions imposed by the mining regulating authorities, non-removal of the overburden soil mass prior to excavation, excavating and removal of toe support of slopes and improper designing of benches with poor drainage control. These may contribute either as singly or as combinations.

1.1 Scope and Objectives of the Study

This study was carried out to identify the factors related to slope failures in open cast rock quarries, which should be considered before planning and design. In this purpose, a detailed study was carried out for a particular industrial level rock quarry site at ThalathuOya, where slope failure has already occurred due to unplanned quarry face development especially with regards to slope stability considerations. Mining engineers can

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utilize the results of this study as a 'Model Case' for their future mines planning activities. The specific objectives of the study are to,

- Determine the causative factors behind the slope failure occurred in Operating face of ThalathuOya Rock quarry.
- Carryout detail field and laboratory investigations to determine the shear strength parameters of the slip surface.
- Identify remedial measures for possible vulnerable sections of natural permanent slopes of the same site for future slope failures.
- Provide recommendations to mine planning professionals, which can be used as a base model for them

2. Methodology

The following methodology was adopted in order to achieve the above mentioned objectives.

2.1 Step 01 Desk Study

An initial study on the history of quarrying and slope failures occurred in the particular area was carried out and the information was obtained from the respective organizations [3]. The Geology and the Geomorphology of the area was studied using the 1:120000 Structural Geology maps [9] and the 1:50000 Topographic maps [10]. This is in order to identify the joints and major discontinuity patterns and drainage pattern of the study area.

2.2 Step 02 Engineering geological assessment of the failure site

In order to identify the possible causative factors behind and the failure type responsible for this failure, a detailed engineering geological assessment accompanied with a rock joint analysis was performed. In rock joint analysis, readings on the slope geometry, strikes and dips of slip surface and other joints and discontinuities, discontinuity spacing, separation and their geomechanical characteristics and ground water flow of discontinuities were obtained. This data are summarised and presented in Figure 6 and Table 1 and were later used to determine the most possible type of failure pattern.

2.3 Step03 Analysis of effectiveness of the causative factors.

Hydrogeological pattern of the study area was studied through data gathered in the initial steps to analyse the influence of pore water pressure and degree of leaching of slip surface and thus the influence of ground water for the slope failure. Creeping effects of rock mass was studied by performing a rock mass classification, which assess the inherent quality of rock mass.

2.4 Step 04 Determination of Shear Strength parameters of Slip surface

During the analysis in finding the type of failure pattern in Step02, it was found out that it is of planer type failure and the findings are presented under section 3.6.Hence thereon following methods were employed in determining shear strength parameters rock.

2.4.1 Using the Method of back analysis

As sample cases described by Bray & Hoek [2] and Sau Mau Ping Road, Kowloon city case in Hong Kong [11], a range of friction values were given to corresponding factor of safety equations and assuming that the Factor of Safety (F) reaches unity at failure. Different cohesion values were obtained for different internal friction angle values at different possible pore water pressure conditions. Two basic models (Model-01 and Model-02) which were earlier employed to similar cases [2] and [11] were employed in this study. These models represent most possible slope geometries where, Model 01, with a tension crack and Model 02 without a tension crack.

<u>Model-01</u>

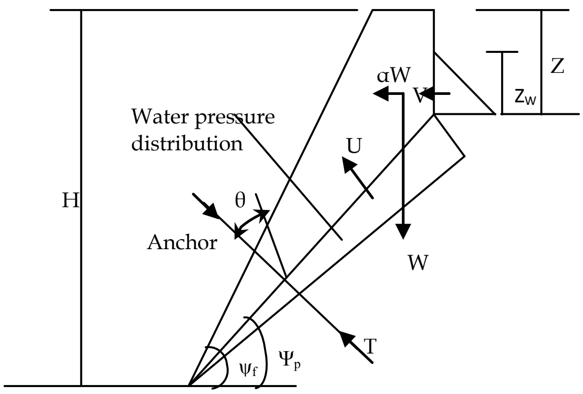


Figure 1 - Model 01: with a tension crack

$$F = \frac{cA + [W(\cos\psi p - \alpha\sin\psi p) - U - V\sin\psi p + T\cos\theta]\tan\emptyset}{W(\sin\psi p + \propto\cos\psi p) + V\cos\psi p - T\sin\theta} \quad (1)$$

Where

$$Z = H\left[1 - \sqrt{\cot\psi f - \tan\psi p}\right] \quad (2)$$

$$A = (H - Z)/\sin\psi p \quad (3)$$

$$W = \gamma_r H^2/2\{[1 - (H/Z)^2]\cot\psi p - \cot\psi f\} \quad (4)$$

$$U = \gamma_w Z_w A/2 \quad (5)$$

$$V = \gamma_w {Z_w}^2/2 \quad (6)$$

$\emptyset$– Angle of internal friction along the discontinuity

Model-02

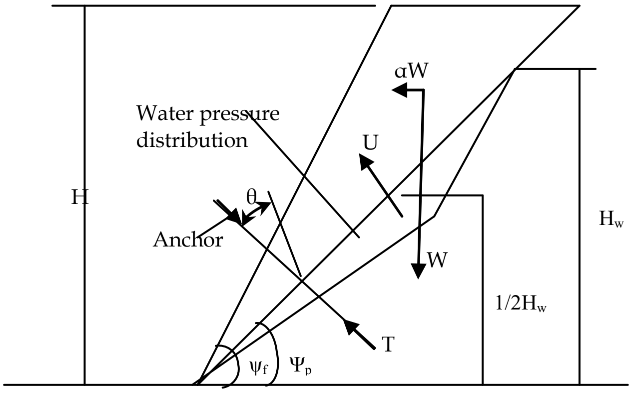


Figure 2 - Model 02: without a tension crack

$$F = \frac{cA + [W(\cos\psi p - \alpha\sin\psi p) - U + T\cos\theta]\tan\emptyset}{W(\sin\psi p + \propto\cos\psi p) - T\sin\theta} \quad (7)$$

Where

$$A = (H - Z)/\sin\psi p \quad (8)$$

$$W = \gamma_r H^2\{\cot\psi p - \cot\psi f\}/2 \quad (9)$$

$$U = \gamma_w H_w^2/4\sin\psi p \quad (10)$$

$\emptyset$ – Angle of internal friction along the discontinuity

The Figure 1 and Figure 2 and Equations 1 to 10 were used in determining the value range for cohesion.Here, c′ and Ø are the cohesion and angle of internal friction of rock and U- pore water pressure, V-thrust force generated in the tension crack , W – weight of the sliding wedge, α - coefficient seismic acceleration taken as 0.08**g** = 0.785, H-slope height, ψ_f– slope face angle, ψ_f– dip of slip plane, z- tension crack depth, Z_W- water head in tension crack in Model 01, H_W-water head in slip plane in Model 02 and T-reinforcement applied through tension bolting (if available).

2.4.2 *Using Empirical Equations*

Shear strength of highly jointed rock mass was determined using empirical equation proposed by Barton [1]. According to section 3.4, the stress levels applied on slip surface due to overburden is comparatively low and hence Barton's method was employed in this analysis, which is more effective in low stress levels. The Barton's equation can be given as,

$$\tau = \sigma_n \tan[\mathrm{JRC}\log_{10}(\mathrm{JCS}/\sigma_n) + \emptyset_r] \quad (11)$$

Where,

JCS - Joint compressive strength
JRC - Joint roughness coefficient
τ - Shear strength of the joint
$\emptyset_r$ - Residual friction angle
σ_n - Normal effective stress

Residual friction angle represents the theoretical minimum strength value of a planer and slickensided surface obtained when the roughness is completely worn away. The JRC was determined according to the roughness profiles [1]. The residual friction angle ($\emptyset_r$)was determined from the Equation 12[1]; and it is with the use of Schmidt hammer rebound value of corresponding weathered and fresh rocks.

$$\emptyset_r = (\emptyset_b - 20^0) + 20^0(r/R) \quad (12)$$

Where, $\emptyset_b$ is the basic friction angle of the rock, 'R' the schmidt hammer rebound value of fresh rock surface and 'r' the schmidt hammer rebound value of corresponding weathered rock surface

2.4.3 *Using Rock Mass Rating System*

The shear strength of the rock mass can also be determined through rock mass classification. This is by using the geomechanics classification system (Rock Mass Rating RMR System) [4] and Slope Mass Rating (SMR) System [6], which the final score of rock class was correlated to the shear strength parameters of most unfavourable joint set of the rock mass. Here RMR is adjusted into SMR by,

$$SMR = RMR + (F_1 * F_2 * F_3) + F_4 \quad (13)$$

Where $F_1, F_2, F_3$and $F_4$are the adjusting factors for, dip direction of most vulnerable joint set relative to dip direction slope, dipangle of most vulnerable joint set, dip angle of most vulnerable joint set relative to dip angle slope and the blasting/ excavation method employed respectively.

2.5 Step 05 Introduction of remedial measures

Considering all the analysed data and assuming the worst possible combinations of causative factors for the failure, different slope stabilization techniques were introduced to stabilize the unstable portion that remained hanging at other part of the quarry site. Stabilization techniques such as reduction of slope angle, slope height, providing adequate drainage, and other miscellaneous methods such as tension bolting were analyzed against the factor of safety that can be gained and the cost effectiveness of the remedial action.

2.6 Step 06 Design of safe slope for failed slope

In design of safe slope for already failed slope, the basic concept of achieving the required factor of safety 1.50 for permanent slope was the main objective. In determining the safe slope angle, the Equation 14 proposed by Orr, 1992 [5] was used as the base.

$$S = 0.65RMR + 25 \quad (14)$$

Where, S is slope angle in degrees and RMR is the Rock Mass Rating of particular rock mass.
When S< 40°; one of the following options can be adopted.
Option 01- Give-up slope design and stop the mining activity.
Option 02- Take, S= 40° and minimize the pore water pressure development by improving the drainage simultaneously.

3. Engineering Geological Assessment

3.1 Location

Scale (1: 50000)
Figure 3 - Topography of the site

The particular site under study is at ThalathuOya in the Patahewaheta Divisional Secretariat of Kandy District, at a distance of two kilometers from ThalathuOya town to the left side of the Moragolla road, at an altitude of approximately 600MSL. The study area where slope failure has occurred extends to about two hectares.

3.2 Rainfall

The rainfall data for the particular area pertaining to the time of failure (i.e. July 2001) is plotted in Figure 4. As the rain fall graph depicts, area has received 95mm rapid precipitation on 26th July 2001. This may have caused the sudden increase of pore water pressure on the slip surface of the site, which ultimately caused to initiate the failure as a debris flow on 28th July 2001 (according to the information given by the neighbors).

3.3 Geology and geomorphology

The main soil types overlain are residual soils, which formed as a result of weathering of underlying parent rock, charnockiticbiotite gneiss. Due to intense dipping nature of the terrain, the natural vegetation cover consists mostly of grass and few isolated tall trees.Lands have mainly used for cultivation of intercrops such as pepper and cloves, where the dipping terrains are favourable for growth of these crops. The lands that were affected due to the particular slope failure have been used previously for pepper cultivation.

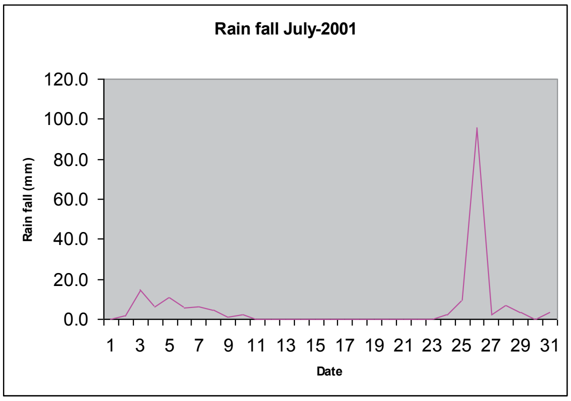
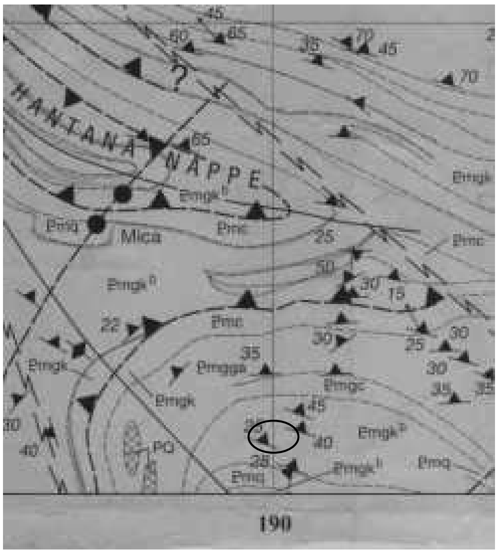


Figure 4 - Rainfall in July 2001 to ThalathuOya
(Source- Meteorological Department of Sri Lanka)

When considering the site geological setting, the area belongs to the Highland Series of metamorphic rocks in the central part of the Sri Lanka. Furthermore the structural geology map of the area in Figure 5 reveals that, there is a shear zone running at the North-Eastern boundary of the site. This may be the initiation of the disturbance in the rock mass, which later act as the ground water seepage zone through the rock slope, along a weaker plane of foliation planes, which ultimately act as the slip surface. This can further be justified by the degree of weathering of the rock. The upper and lower parts of this seepage zone is weathered to a lesser degree than the seepage zone, whilst the mid slip surface is excessively weathered and the surface minerals leached due to this same ground water movement. Moreover, chert particles detected from mobilized debris as wells as thin crustal formations at the bottom of the right hand relief surface is good indication of groundwater movement for very longer period, where chert is formed by the precipitation of silica under hydrostatic pressure for longer periods.

The overburden thickness of the soil varies from 1.0m at the lower part of the slope to 0.25m at the upper part of the slope. The weathered thickness of the rock varies from 1.0m at the left failure slope to more than 6.0m at the right failure slope. The geomorphology of the area is with high relief with highly undulated terrain. The morphological depression at the vicinity of the site is a good indication of an existence of a major discontinuity.



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Scale (1: 120000)

Figure 5 - Structural Geology of the area

3.4 Geometry of failed slope

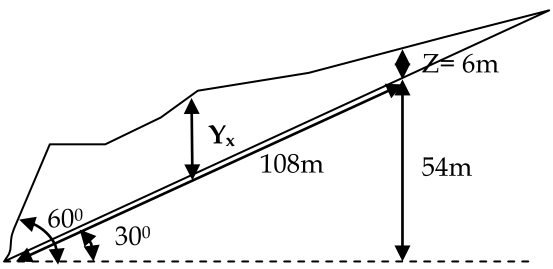

Figure 6 - Approximate Geometry of failed slope

Approximate geometrical parameters of failed slope were obtained from existing relief faces. The overburden thickness (Y_x) varies from 4.50m to 6.50m. The location of probable tension crack was detected at the upper crest of the slope, where it can be traced from remaining extensions in the two relief wedges. The depth of the tension crack was around 6.00m.

3.5 Hydrology and hydrogeology

The ground water movement in the particular site is shown in Figure 7 and as it depicts, large volume of ground water as well as surface water is passed through the site in the rainy season. There is a catchment area of around fifteen acres which elevates to more than 40m from the crest of the slope. The existence of Bamboo trees in the upper and upper left side parts of the slope crest is a good indication to prove the shallow ground water existence throughout the year. Also existence of surface water draining canals pointing towards the site location is a good indication for water stagnation effect in the rainy season, which ultimately may have caused to increase the pore water pressure conveniently. As mentioned in section 3.3, this process was in existence for very longer period until the ultimate failure.

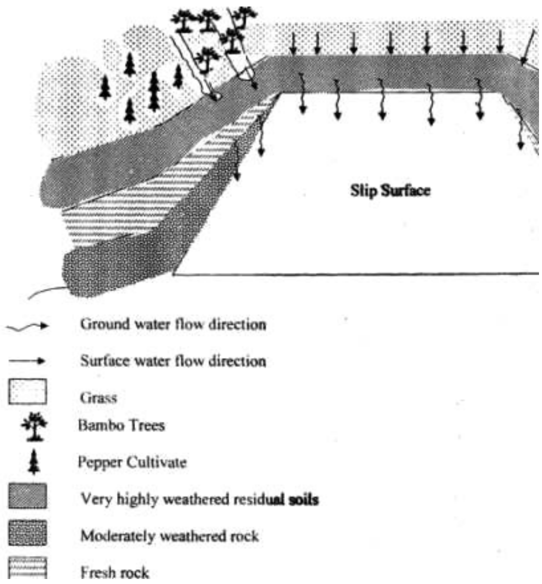

Figure 7 - Hydrogeological setting of the site

3.6 Rock Joint Analysis and Failure type

Dip and strike of all the joint sets were measured from the exposed relief areas. Moreover, conditions of these discontinuities were also obtained to classify the rock mass.

3.6.1 Stereonet Analysis results of Rock Joints

The stereonet plot as shown in Figure 8 was developed based on data in Table 1 and the back analysis results of section 4.1, which was used to plot the friction circle. When considering the kinematic conditions of the possible rock slope failures, it is clear that vertical joint sets 1 and 2 may have contributed to form possible tension cracks due to toppling movements. Individual plane failures are possible for joints 2, 3, 4, 5 and slip surface. Out of them, most vulnerable is joint 5 and least is along slip surface, which has the least dip. Wedge failures are possible along the combination of joint sets 2&4, 2&5, 3&4, 3&5 and 4&5. From kinematic point of view the most vulnerable of them is the wedging along the intersection of joints 2&5.

Table 1 - Orientation and Properties of discontinuities

Joint No. (Dip direction/ Dip angle)	Joint Spacing (m)	Joint water condition (l/minute)
Slip surface (Foliation Plane) ($60^0/30^0$)	0.20-1.50	0.80
Joint No.01 ($73^0/90^0$)	0.30-3.00	0.80
Joint No.02 ($180^0/90^0$)	0.30-3.00	0.80
Joint No.03 ($30^0/40^0$)	5.00-6.00	< 0.80
Joint No.04 ($84^0/50^0$)	14.00-15.00	< 0.80
Joint No.05 ($70^0/60^0$)	> 20.00	< 0.80
Slope Face ($60^0/30^0$-40^0)	According existing relief area geometry	

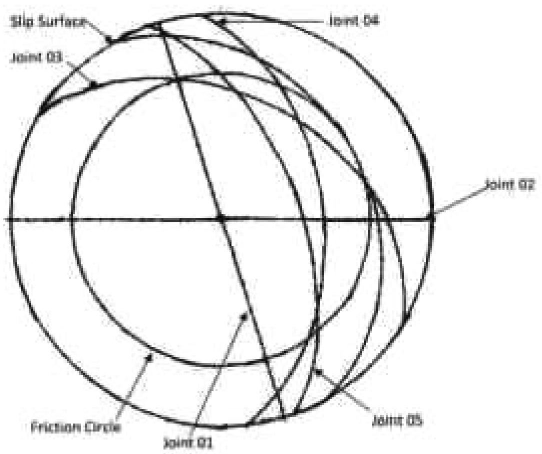

Figure 8 - Stereonet plot of great circles of rock discontinuities

3.6.2 Rock Joint condition Analysis
According to Table 1 results, the **Joint Separation** for all the joints, including slip surface is ***<0.10 mm*** and **Joint Smoothness** is ***Irregular to Planer*** for all. It was very difficult to find any **Joint Gouge** in all discontinuities. The highest **joint water condition** is observed in slip surface and joints 1 and 2. Apart from these, it was quite evident that the rock mass is extremely weathered along the slip surface compared to other joints and embedded minerals in rock texture is leached out of their parent rock in this section, which extends to about 2.00m in thickness.

3.6.3 Most Possible type of Failure Pattern
Stereonet analysis depicts that the failure type is combination of wedge and plane failures. Moreover, slip surface (which is parallel to foliation planes), which has the least possibility of contributing for the failure. However, initial micro level failures may have opened up these rock joints and later may have act as pipes of drains which brought surface runoff into an underlying weaker-well developed-foliation plane. It is evident that the failure has occurred along the presently exposed slip surface and hence the most contributory joint set out of above joint sets is a well-developed foliation plane and thus is a **plane type failure**.

4. Shear strength parameters of slip surface

4.1 Shear strength determination from back analysis

Equations 1 and 7 in section 2.4.1 were rearranged by keeping the factor of safety (F) equals to unity (which is at limit equilibrium) and range of values were given to "ϕ"in order to obtain the variation of "c" for two basic slope geometries, which depicts in Figure 9. Previous experimental studies have shown that "ϕ" for gneissic rocks is ranged between $27^0$and 34^0 [8].This range is highlighted from the ellipse in Figure 9, from which, the mid value of the range for "ϕ", which is **30^0** was taken as the angle of internal friction of the slip surface and the corresponding lowest possible value for "c" from the curves become 10.35T/m^2 or **0.1035MPa**.

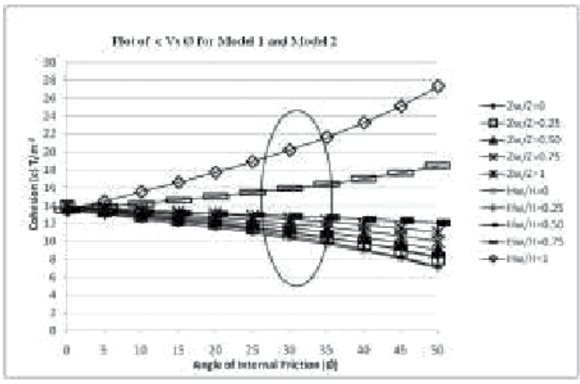

Figure 9 - Cohesion Vs Angle of Internal friction for both failure Models

4.2 Shear strength determination from empirical equations

Experimental results of unconfined compressive strength [7] of rock core samples obtained from slip surface are presented in Table 2. The average unconfined compressive strength of the slip surface from Table 2 is **43.50MPa**, which is the Joint Compressive Strength (JCS) of the slip surface. The average Schmidt Hammer rebounce value on weathered rock surface (r) of slip surface was **31.875** and average Schmidt Hammer rebounce value on Fresh rock surface (R) of same slip surface was found to be **46.409**.

Table 2 - UCS results of rock samples

Sample Number	Failure load (kN)	UCS value (MPa)
01	100.00	58.65
02	475.00	41.80
03	290.00	35.71
04	250.00	49.52
05	400.00	41.42
06	200.00	42.79
07	75.00	38.51
08	175.00	39.61

The basic angle of internal friction for coarse granite (ϕ_b) can be assumed as **26^0** [1]. Joint Roughness Coefficient (JRC), is **05** for joints which are Irregular –planer type [1]. Substituting these values to the Equation 12, ϕ_r becomes **19.73^0**.According to the slope geometrical parameters described in section 3.4, the wedge of the slope exerts low normal effective stress (σ'_n) level ranges from 08 MPa to 15 MPa on the slip surface. Applying this stress range to Equation 11 results the shear strength profile presented in Table 3.

Table 3- Shear strength range for different effective normal stress levels

Normal effective stress level(σ'_n) MPa	Shear strength (τ') MPa
08	3.45
10	4.23
12	4.98
14	5.71
16	6.43

Table 4- Shear strength deviation from two different methods

Normal effective stress (σ'_n) MPa	Empirical equations $(\tau')_{Em}$MPa	Back analysis $(\tau')_{Ba}$MPa	Difference ($\Delta\tau'$) $(\tau')_{Em}$- $(\tau')_{Ba}$ MPa
08	3.45	4.718	1.27
10	4.23	5.873	1.64
12	4.98	7.032	2.05
14	5.71	8.183	2.47
16	6.43	9.337	2.91

Using the results of back analysis discussed under section 4.1, the shear strength profile can be developed for the same effective normal stress levels, based on the Mohr –Coulomb criterion. As depicts in above Table 4, the results from two methods provide closer results at lower normal effective stress levels which, indicates the validity of the assumptions and evaluation. Theoretically, Barton's method generally provides more conservative values compared to other methods and is more valid for low normal stress levels [1]. This has been proved by the results of Table 4 as the Barton's method provide lower shear strength values compared to the values provided by the Mohr-Coulomb criterion and the difference diverges as the normal stress level increases.

4.3 Shear strength determination from Rock Mass Classification

Table 5 - Rock Mass Rating results

Parameter	Results	Rating(%)
Average Intact Rock Strength	43.50MPa (from Table-02)	04
RQD %	42.40 %	08
Joint spacing	Minimum Joint Spacing = 0.30m	10
Joint Condition	Irregular & planer joint surfaces; Continuous; Joint Separation<1.0mm; Soft joint wall rock	20
Ground water condition	0.80 (l/minute) (Damp)	10
Total Rating		52

The Rock Mass Rating (RMR) value for particular rock mass is **52%** (Table 5). Section 3.6 has shown that the most vulnerable joint set contributed to the failure was the well-developed foliation plane, which acts as the failure plane. When considering the data in Table 1, the adjustment factors (F_1, F_2, F_3andF_4) of Slope Mass Rating (SMR) value become **01**, **0.60**, **-50** and **0**(since blasting has employed in previous mining process)respectively. Substituting these values to the equation 13, the SMR value of the rock mass becomes **22.**Reverting back to the adjusted RMR, i.e. SMR with rock mass shear strength properties the Net RMR (or SMR) is within21-40 and thus, Rock Class Number belongs to **category IV** and can be concluded as a ***Poor Rock*** when considering the stability of a rock slope in an open pit mine. For rock class IV, the corresponding cohesion and the angle of internal friction of the rock mass are **0.100MPa-0.200MPa** and **15^0-25^0** respectively.

5. Stabilizing the vulnerable areas

5.1 Determination of Best Mitigation Technique

To mitigate future slope failures in the post failure vulnerable areas in the same site, attempts were made to achieve a minimum factor of safety of 1.5 by employing different types of improvement techniques such as,

01. Reduction of Slope height (reduction of overburden stress)
02. Reduction of Slope angle (reduction of overburden stress)
03. Drainage improvement (reduction of pore water pressure)
04. Reinforcement of slope.(increase of shearing resistance of slip surface)

The required factor of safety was calculated by assuming the worst possible conditions, i.e. when the pore water pressure is at maximum for both of the assumed failure models (Model 01 and Model 02). The shear strength parameters deduced from back analysis were used in conjunction with Equation 1 and Equation 7 in calculating the respective safety factors and plotted the gained factor of safety against percentage improvement by the respective stabilizing technique. It is evident in Figure 10, that the required factor of safety, 1.5 (which is highlighted by the dotted line) can only achieved through reduction of slope height and the introduction of reinforcements. Out of these two, the most rapid (most economical) and practical method could be the reduction of slope height, as tension bolts will need to penetrate to grater depths than in normal circumstances, since the underlying bedrock is weathered down to a substantial depth. The reason behind lack of effectiveness in reduction of slope angle and drainage improvement can be explained through Figure 6, where slope geometry suggest that reduction slope angle will reduce the overburden weight in very low proportions as natural slope surface is running almost parallel to the slip surface after initial slope angle of 60^0.

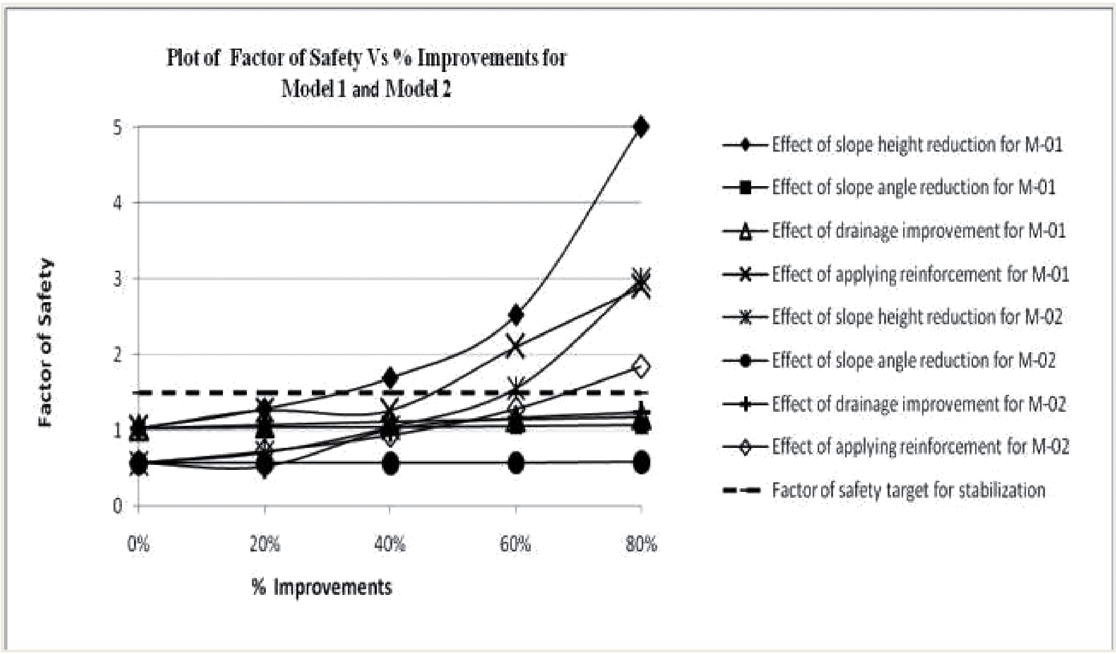

Figure 10 - Variation of Factor of Safety with Percentage improvement for both Failure Models

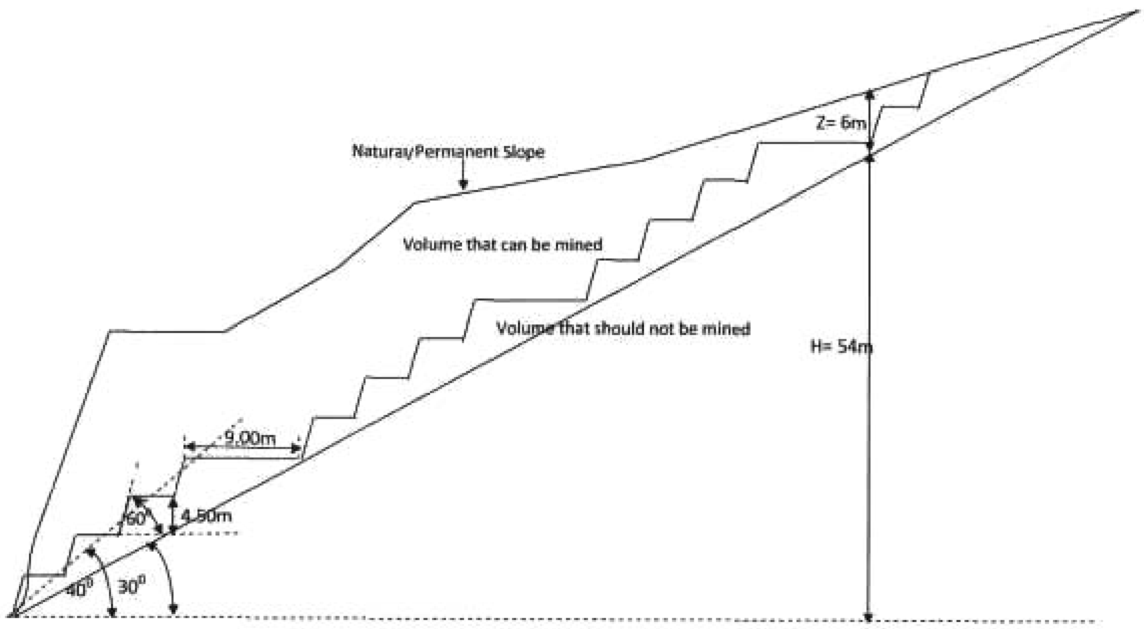

Figure 11 - Model Slope Design for failed slope

The same applies for drainage, where improvements in drainage is less effective compared to pore water pressure generated by large extent of slip plane having about 128m in length.

5.2 Sample Safe Slope Design for the failed slope

This will serve as a model for Engineers in better planning of their mining activities. Using the shear strength parameters determined from the back analysis, with a factor of safety of 1.50 for open pit mine slope, one can carry out the design in iterative basis until the required factor of safety is achieved. As a basis, it was tried to minimize disturbing force by decreasing the weight of the wedge (which happens normally as mining progresses) and simultaneously maintaining the resisting force as constant as possible, by maintaining the effective contact area of the wedge as constant as possible. For the failed slope it is reasonable to assume the Model-01, slope with tension crack behaviour. Different slope parameters were taken in design of the slope and using the Equation 1 the factor of safety was tested against these parameters. This procedure was carried out until required factor of safety was achieved.

The final design is shown in Figure 11 and it was arrived in following manner.
The Equation 14 gives overall slope angle,
S = 0.65*22+ 25 as 39.3°; S < 40° thus, taking S = 40° and **slip surface angle =30⁰**; **Overall Bench angle** (or **Inter Ramp Angle** since multiple sets of benches need to be employed with several ramps due to slope height) (S) = **40⁰**; **Individual Bench angle= 60⁰**; **Individual Bench height = 4.50**; **Number of Individual Benches per inter ramp= 04**, **Number of Ramps used= 03** and **Ramp Length = 9.00m**.The initial weight of the wedge = 1323.52 T. Reduction of up thrust force and the horizontal tension crack force can be achieved by employing proper drainage methods on upper crest of the slope. Thus, U= 0.00 T/m^2 and V= 0.00 T/m^2. Reduction of the weight of the wedge due to introduction of the benches = 695.474T. Substituting the above values to the Equation 01, gives a Factor of Safety, F = 1.57. However it should be emphasized that the initiation of the bench mining should be started from top of the slope and control basting should be carried out until the required geometry of the slope is achieved.

6. Conclusions and Recommendations

6.1 Causative factors for the slope failure

Structural Geology of the area reveals previous disturbances in this same area due to tectonic movements in adjacent shear zone. Hydrogeological pattern of the area has further worsened the situation by stagnation of water on this disturbed rock mass, may have caused to seep large quantities of flow through opened up more unstable joint sets down to more stable well developed weaker foliation plane, which finally became as the slip surface of the case under consideration. This fact has been further proved by the presence of chert particles on the slip surface and excessive weathering caused to leach the mineral grains of the slip surface and thus the reduction of the shear strength of the slip surface. Uncontrolled blasting, removal of toe support of the slope and non-employment of drainage facilities have created the reactivation of the previously disturbed area and increase of effective upward force, which ultimately may have caused to displace the material above the slip surface as a debris flow.

6.2 Shear strength parameters in slip surface

According to below Table 6 most conservative and economical testing method is the employment of RMR system in initial design stages. The back analysis can be employed only if a similar type failure has occurred in the vicinity of the site, under the same conditions and provides least conservative values. Use of empirical equations proposed in this study yield more conservative values compared to back analysis method, but cheaper and economical in practical situations. The results of the Table 4 and Table 6 prove the applicability of Barton's theorem for local rock masses, where the Barton's equation is reliable in using at lower stress levels and it deviates from the actual values at higher stress levels.

Table 6 - Comparison of different methods for Shear strength parameters

Method	Shear strength parameters	
	Angle of internal friction (ϕ)	Cohesion MPa
Back analysis	30^0	0.1035
Empirical equations	$\phi_b = 26^0$ $\phi_r = 19.73^0$	-
RMR system	15^0-25^0	0.100 -0.200

6.3 Remedial measures to stabilize the vulnerable areas of the site

As the most immediate measure, the canals pointed towards the slip surface should be diverted to elsewhere and sufficient underground draining system should be implemented to reduce the excessive pore water pressures that will be developed in rainy seasons. Furthermore, analysis has revealed that the most economical, rapid and practicable method is to reduce the slope height and this should be carried out with the use of controlled blasting.

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