EFFECTS OF FAULTS ON STABILITY OF SLOPES IN OPEN CAST MINES

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology

in

Mining Engineering

By

MASA SOREN

10605018

DEPARTMENT OF MINING ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA-769008

2010

EFFECTS OF FAULTS ON STABILITY OF SLOPES IN OPEN CAST MINES

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology

in

Mining Engineering

By

MASA SOREN

Under the guidance of

DR. SINGHAM JAYANTHU

DEPARTMENT OF MINING ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA-769008

2010

National Institute of Technology

Rourkela

CERTIFICATE

This is to certify that the thesis entitled “Effects of Faults on Stability of Slopes in Open Cast Mines
” submitted by Sri Masa Soren, Roll No. 10605018 in partial fulfillment of the requirements for the award of Bachelor of Technology degree in Mining Engineering at the National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

Date:

Dr. Singam Jayanthu
Department of Mining Engineering
National Institute of Technology
Rourkela-769008

ACKNOWLEDGEMENT

First and foremost, I express my sincere gratitude and indebtedness to **Dr. S. Jayanthu**, Professor and Head of the Department for allowing me to carry on the present topic "**Effects of faults on stability of slopes in open cast mines**" and later on for his inspiring guidance, constructive criticism and valuable suggestions throughout this project work. I am very much thankful to him for his able guidance and pain taking effort in improving my understanding of this project.

An assemblage of this nature could never have been attempted without reference to and inspiration from the works of others whose details are mentioned in reference section. I acknowledge my indebtedness to all of them.

At the last, my sincere thanks to all my friends who have patiently extended all sorts of help for accomplishing this assignment.

Date:

Masa soren

Department of Mining Engineering

National Institute of Technology

Rourkela-769008

CONTENTS

ABSTRACT

Stability of opencast mine slopes is significantly influenced by the presence of structural features in rock mass. In this work the effect of faults on slope stability is discussed in detail. The slope design in such situations needs to be made keeping in view the relative orientation of these features with respect to the slope orientation. To avoid fault-induced instability the benches should be laid across the strike of fault. Stability of slopes in opencast mines is greatly influenced by the presence of structural features in rock mass.

“Assessment of the stability of of slopes in open pit mines at different stages of mining is important for the safe and economic mining operations. Slopes are generally designed based on the geotechnical data and physio-mechanical properties of rock/soil. From geotechnical data , the rock mass quality is assessed, and from this the rock mass properties are estimated. Using the rock mass properties stability of the slopes is evaluated from empirical, analytical and numerical techniques.

Based on the numerical model analysis, it is concluded that bench failures are likely to occur because of discontinuities in the form of faults. Fault F1 located along the proposed boundary appears to pose instability problems to the high walls. Therefore, it is strongly recommended to monitor the slopes for its stability.

The analysis of stability of slopes for the ultimate pit slope indicated the safety factor exceeding 1.2 for slope angle of 48 degrees without consideration of the faults. However, the presence of fault F1 decreased the safety factor below 1. Therefore it is recommended to extend the boundary of the mine beyond the fault F1, and maintain the overall slope angle not steeper than 48 degrees.

LIST OF TABLES

LIST OF FIGURES

CHAPTER: 01

INTRODUCTION

Slope stability is a major problem in opencast mines. Slope stability in a large scale open pit mining operation is a matter of concern for the mine management so as to establish safety throughout the life of the mines. Again the profitability of the open pit mines is dependent to a large extent on the use of steepest pit slopes possible, provided they do not fail during the life of the mine. Steep slopes do need a great amount of analysis so that the whole operation is safe and profitable.

Assessment of the stability of slopes in open pit mines at different stages of mining is important for the safe and economic mining operations. Slopes are generally designed based on the geotechnical data and physico-mechanical properties of rock/soil. From geotechnical data, the rock mass quality is assessed, and from this the rock mass properties are estimated. Using the rock mass properties stability of the slopes is evaluated from empirical, analytical and numerical techniques.

In homogenous, isotropic ground conditions, the factor of safety can be determined for predefined failure modes using limit equilibrium method (Hoek. and Bray, 1981; Hoek, 1986; Piteau & Martin, 1981; Zanbak, 1983). Similarly, using analytical solution given by Xiao Yuan & Wang Sijing (1990) flexural breaking of rock mass can be determined. Design charts can be developed using limit equilibrium method. Some design charts are available for plane, wedge, circular modes of failure (Hoek & Bray, 1981), and for toppling failure (Choquet & Tanon, 1985; Zanbak, 1983). The field engineer can use them if the basic geotechnical properties are known. These charts are useful to analyse only simple types of predetermined failures, but not for determining the slope angle which depends on the rock mass stability. So there is a need to develop design charts and design guidelines to determine slope angles for different slope heights in different rock mass conditions, which can be readily used by the practicing engineer.

Under this project, the slope stability parameters in open pit mines with different geomining conditions were studied. Analysis was carried out using numerical methods, and the results compared with the empirical methods. Based on these studies, design charts and guidelines are prepared for determining slope angles under different geomining conditions.

1.1 Objective

The main objective of the study is to understand the stability of slopes in the presence of geological discontinuities such as faults in opencast mine, and to design safe slope angles for ultimate pit depth.

CHAPTER: 02

LITERATURE REVIEW

Generally, Stability of the slopes is evaluated from empirical, analytical and numerical techniques. In homogenous, isotropic ground conditions, the factor of safety can be determined for predefined failure modes using limit equilibrium method. Some design charts are available, which are useful to analyse only simple types of predetermined failures, but not for determining the slope angle which depends on the rock mass stability, particularly the unfavorable joints. If the factor of safety for the slope under analysis is above 1.2, then it is considered stable, and if it is less than 1.2, then the slope is considered to be potential hazardous horizon. Over design of slopes are not only uneconomic but also generate more waste. In view of conservation of the deposit it is necessary to design the slopes utilizing the geotechnical considerations.

Factor of safety is the ratio of stabilizing forces and destabilizing forces existing on the failure surface under study. The shear strength is mobilized to resist the shearing stress caused by the gravitational forces.

Kinematic and simple stability checks can be carried out using hemispherical projections for the joint sets identified. For the planar mode of failure, the analysis is performed for different block geometries which are kinematically possible to slide. For wedge failures, three-dimensional analysis is performed. For a wedge to be kinematically free, two planes should intersect and the dip of line of intersection must be less than the slope angle and its direction within +/ - 20^o that of slope face direction. The kinematic analysis gives a general idea about the type of failures expected, but the slope angles cannot be designed based on these results. But the failures identified by this method can be analysed in detail by limit equilibrium method. Further, it is not possible to identify circular and non-circular failures using hemispherical projections. The hemispherical projections and kinematic analyses are performed for the joint sets identified in some of the mines. A computer software named DIPS (1999) is used to assist in this analysis.

The failure surface can be planar, circular or non-circular. Different failure surfaces are analyzed to identify the surface with minimum factor of safety. Circular failure analysis is done using Bishop's method for the whole slope to assess deep seated failures, and for slopes covering a few benches to assess the local failures. On the other hand, non-circular failure analysis is done using

Sarma's method, which mainly checks the possibility of failure through different rock types. For the benches in the selected mines, two dimensional limit equilibrium analysis was performed for plane, non-circular, circular and toppling failures. For this purpose, software named GALENA, originally developed by BHP Engineering, Australia (GALENA, 1990) can be used.

In order to study the effect of in-situ stress on the stability of the slopes, stress analysis using numerical modeling is performed in mines.

An important aspect of slope stability analysis is determination of safety factor. Slope stability problems are evaluated by using empirical methods and numerical analysis methods.

2.1 Assessment of Stability of Slopes

Following examinations are being done to assess the slopes stability in opencast mines.

2.1.1 Ground investigation

Before any further examination of an existing slope, or the ground onto which a slope is to be built, essential borehole information must be obtained. This information will give details of the strata, moisture content and the standing water level. Also, the presence of any particular plastic layer along which shear could more easily take place will be noted.

Piezometer tubes are installed into the ground to measure changes in water level over a period of time.

Ground investigations also include:-

- in-situ and laboratory tests,
- aerial photographs,
- study of geological maps and memoirs to indicate probable soil conditions,
- Visiting and observing the slope.

2.1.2 Most critical failure surface:

In homogeneous soils relatively unaffected by faults or bedding, deep seated shear failure surfaces tend to form in a circular, rotational manner.

Here, it is aim to find out the most dangerous, i.e. the most critical surface, and using the assumption above, can be found this surface using "trial circles".

The method is as follows:-

- Consider a series of slip circles of different radii but the same centre of rotation. Plot the Factor of Safety (FoS) for each of these circles against radius, and find the minimum FoS.
- This should be repeated for several circles, each investigated from an array of centers. The simplest way to do this is to form a rectangular grid from the centers.
- Each centre will have a minimum FOS and the overall lowest FOS from all the centre shows that FoS for the whole slope. This assumes that enough circles, with a large spread of radii, and a large grid of centers have been investigated.
- We then have an overall failure, surface, with smaller individual ones which should not be ignored.

2.1.3 Tension cracks:

A tension crack at the head of a slide suggests strongly that instability is imminent. Tension cracks are sometimes used in slope stability calculations, and sometimes they are considered to be full of water. If this is the case, then hydrostatic forces develop as shown below:-

Tension cracks are not usually important in stability analysis, but can become so in some special cases.

We should therefore assume the cracks don't occur, but take account of them in analyzing a slope which has already cracked.

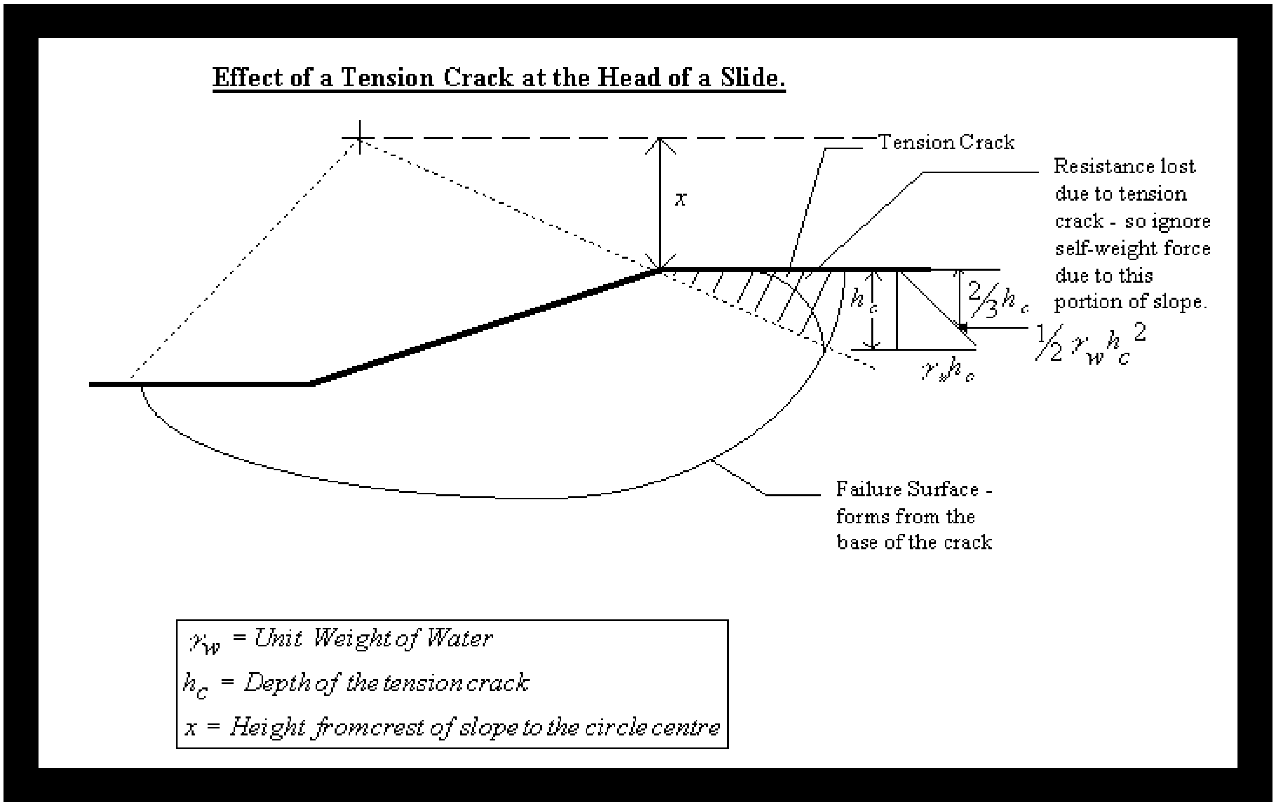

Fig.2.1 showing effects of tension crack at the head of a slide

2.1.4 Submerged slopes:

When an external water load is applied to a slope, the pressure it exerts tends to have a stabilizing effect on the slope.

The vertical and horizontal forces due to the water must be taken into account in our analysis of the slope. So we can allow for the external water forces by using submerged densities in the slope, and by ignoring water externally.

2.1.5 Factor of safety:

In slope design, and in fact generally in the area of geotechnical engineering, the factor which is very often in doubt is the shear strength of the soil. The loading is known more accurately because usually it merely consists of the self-weight of the slope.

The FoS is therefore chosen as a ratio of the available shear strength to that required to keep the slope stable.

Table 2.1 Guidelines for limit equilibrium of a slope

FACTOR OF SAFETY	DETAILS OF SLOPE
<1.0	Unsafe
1.0-1.25	Questionable safety
1.25-1.4	Satisfactory for routine cuts and fills, Questionable for dams, or where failure would be catastrophic
>1.4	Satisfactory for dams

For highly unlikely loading conditions, factors of safety can be as low as 1.2-1.25, even for dams. E.g. situations based on seismic effects, or where there is rapid drawdown of the water level in a reservoir.

2.2 Influence of Faults on Slopes Stability

Faults have following ways of influences on slopes instability in open cast mines.

Ground water in slope has important influence on slope stability, especially for high rock slope. Because of weathering, tectonization and unloading effects, joints and gaps grow and become the main flow path and water storage space. Ground water in the fractured rock can change the physical and mechanical parameters and induce fracture extend, shearing deformation through hydrostatic and hydrodynamic pressure.

Two famous slope failure accidents induced by reservoir impounding are Malpasset arch dam break and Vajont reservoir landslide. The French Malpasset arch dam failure in the initial filling in 1959, and the Italy Vajont slipped at the left side slope. In China, there are also many similar accidents. Many facts and proof indicate that the geological disaster like landslide is closely related to ground water seepage . In order to prevent this kind of disaster and reduce human life and property loss, it is necessary to do some simulation and analysis on the ground water seepage. Only by this way, we can make clearly know about the landslide failure mechanism and disaster laws.

Using finite element method, ground water seepage in rock slope was simulated and analyzed the slope stability under different water head conditions.

Mathematical model of ground water seepage

Assuming groundwater seepage complies with Darcy's law, that is

$$V = KJ \tag{2.1}$$

Where, V is seepage velocity; K is permeability of rock mass; J is hydraulic gradient. Based on Darcy law and continuity equation, mathematical model of ground water seepage can be written as

$$\frac{\partial}{\partial x}\left(KM\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(KM\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(KM\frac{\partial h}{\partial z}\right)+Q=\mu_s\frac{\partial h}{\partial t} \tag{2.2}$$

Numerical analysis on groundwater seepage in fractured rock mass

Took a left creep section A of some hydropower station for example to study the ground water flow and its influence on slope stability. According to the engineering geologic report, the strata were divided into 3 layers: full or strongly weathered bed, weak weathered bed and fresh rock bed. Weathering degree influences on rock permeability. So, seepage calculation layer is also decided by weathering degree.

Strata permeability is given by the average value of same weathering degree bed. A countertendency fault stays in the slope and its permeability is large, seepage force in the fault has important influence on slope stability. Because the slope have many faults or joints, it is difficult to consider all the structure planes effects, so we considered take two big faults F8 and F63 into our research. Parameters were listed in the Tab.2.2:

Tab.2.2 Permeability tensor of rock mass

item / soil layer	Permeability (m/s)	
	K_x	K_y
full or strongly weathered bed	4.0×10^{-4}	4.0×10^{-4}
weak weathered bed	3.9×10^{-6}	3.9×10^{-6}
Fresh rock bed	1.8×10^{-7}	1.8×10^{-7}
Fault F8	2.0×10^{-7}	3.4×10^{-4}
Fault F63	7.2×10^{-4}	5.7×10^{-7}

From the data, the permeability of fault is biggest, and fresh rock bed permeability is the smallest. Permeability of the first layer, full and strongly weathered bed is larger than the second layer, weak weathered bed because of different weathering degree. That is why we adopt the data list in table 2.2.The physical model is shown in Fig.2.2.

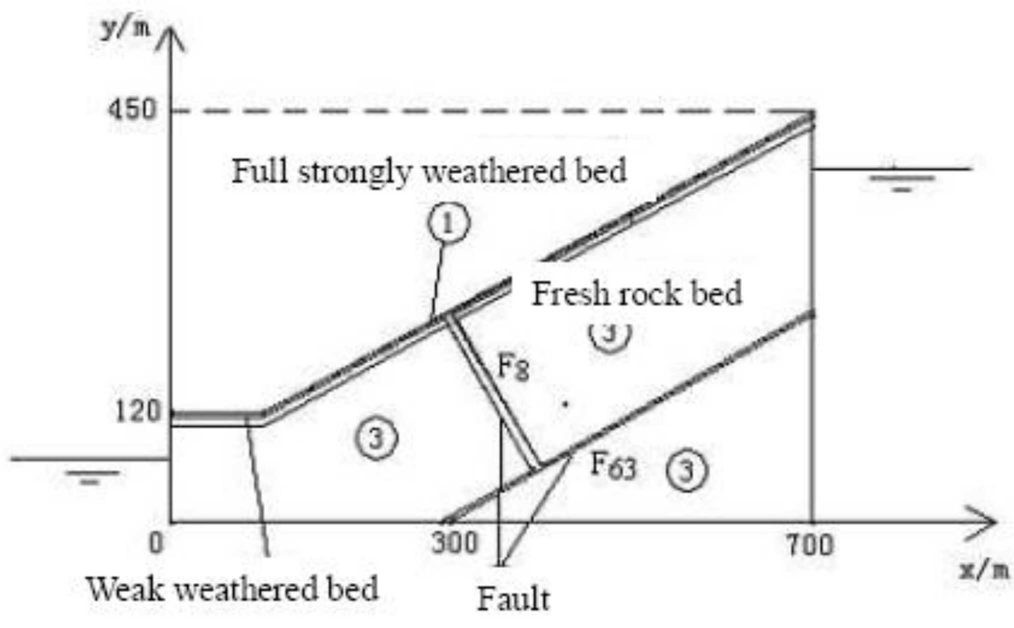

Fig.2.2 Engineering Geological zone of rock mass

The initial condition of seepage simulation is based on no pounding reservoir, and the boundary condition is the fact steady water head. Water head in the left side slope is viewed as fixed value, considered the water head change after reservoir impounding, the value is set H1=69m in the left side, and H2=390m in the right side. The slope surface was reinforced by concrete, and no leaching boundary.

After the reservoir being impounded, water head in the slope raised because of the reservoir water head rising. We can use unsteady seepage simulation to calculate the water head raising process. After the reservoir being impounded, the value raised to H3=120m, under this elevation, the boundary was set fixed water head. Because the water head rose about 51 meters, the head value needs to be check so as not to influence the right side seepage head. After impounding, the water head raised from 69 m to 120m. The height of slope top is 450 m, and the water head is 390 m. The simulation pressure head were shown in Fig.2.3 and Fig.2.4.

Input the simulation results of pressure head, total head and seepage velocity into slope stability analysis software, and using the sliding surface method to calculate the stability coefficients under three different water heads condition, water head of left side is 0 m, 69 m and 120m, and judge the stable states. Table 2.3 gave the stability analysis results under 3 conditions.

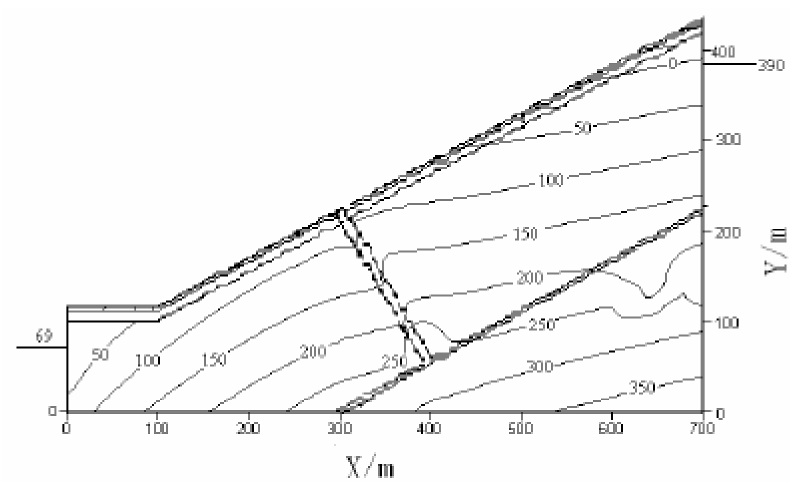

Fig.2.3 Contour map of pressure water-head before impoundment

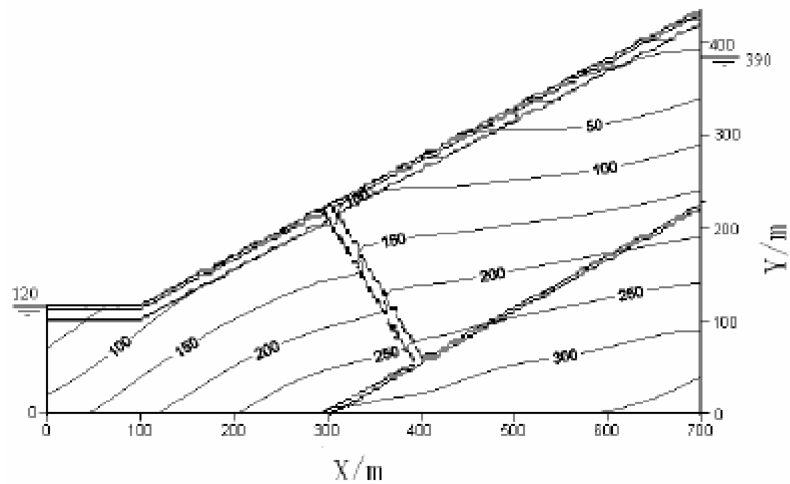

Fig.2.4 Contour map of pressure water-head after impoundment

Tab.2.3 Analysis result under different water-head conditions

Working conditions	Total down-slide force(kN)	Total anti-slide force(kN)	Stability factor
Before impounding	6356	8810	1.39
Impounding 69 m	7865	8617	1.10
Impounding 120 m	8017	8323	1.04

The stability analysis results indicated that, with the water head rise, the total down-slide force increases, total anti-slide force and stability factor decrease. Before the reservoir impounding, the factor is 1.4 and is in a stable state. When the head increase to 69m, the factor is 1.1, and when it achieves to 120m, the factor decreased to 1.04.

2.3 EMPIRICAL MODELLING

2.3.1 Rock Classification Systems and Design Norms

Classification systems have played an indispensable role in engineering for centuries (Bieniawski, 1973 & 1989). Considering the three main design approaches for excavations in rocks - analytical, observational and empirical - as practiced in mining and civil engineering, the rock mass classification today forms an integral part of the most predominant design approach, the empirical design methods. However, modern rock classifications have never been intended as the ultimate solution to design problems, but only a means towards this end. In essence rock mass classifications should be applied intelligently and used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the design objectives and site geology.

Field engineers through the years have been attempting to describe the ground condition using the rock or rock mass properties such as petrologic descriptions, general rock type, or one or a few of the physico-mechanical properties. As a result, several methods have come into usage describing the same rock in different ways. Most of the earlier systems were "intact rock classifications", that is, systems based on laboratory properties determined on a sample of rock. On the other hand, "rock mass classifications" consider discontinuities and large scale ground features.

The objectives of rock mass classifications are to:

a) Identify the most significant parameters influencing the behaviour of a rock mass.
b) Divide a particular rock mass formation into groups of similar behaviour, that is, rock mass classes of varying quality.
c) Provide a basis for understanding the characteristics of each rock mass class.
d) Relate the experience of rock conditions at one site to the conditions and experience encountered at others.
e) Derive quantitative data and guidelines for engineering design.

2.3.2 Geomechanics Classification (RMR) System

Bieniawski's geomechanics classification, also known as rock mass rating (RMR) approach, was initially developed for tunnels in South Africa, but later has been extended to coal mines in the USA. In India this system has been considerably modified for the specific use of developing support systems in coal mine bord & pillar workings.

There are five parameters in this classification. They are: uniaxial compressive strength of rock material, RQD, spacing of discontinuities, condition of discontinuities and ground water conditions. Ratings obtained for the values of the individual parameters are summed to give the basic (unadjusted for discontinuity orientations) RMR. This overall rating is adjusted for orientation of discontinuities by applying correction factors.

Table 2.4: Rock mass classification (after Bieniawski, 1973)

Sl. no.	**Para-meter**	**Range of values**				
1	Spacing of joints (cm)	< 6	6 - 20	20 - 60	60 - 200	> 200
	Rating	0 – 5	6 - 8	9 - 10	11 - 15	16-20
2	Condition of joints	slickensided; soft gouge; continuous	slickensided; 1-5 mm gouge; continuous	slightly rough; < 1 mm soft gouge	rough; fresh; dis-continuous	very rough; tight; fresh; discontinuous

	Rating	0 - 4	5 - 10	11 - 20	21 - 25	26 - 30
3	RQD (%)	<25	25 - 50	50 - 75	75 - 90	> 90
	Rating	0 - 3	4 - 8	9 - 13	14 - 17	18 – 20
4	Rock strength (kg/cm^2)	<250	250 - 500	500-1000	1000 – 2500	> 2500
	Rating	0 - 2	3 - 4	5 - 7	8 - 12	13 – 15
5	Ground water (l/min.)	> 125	25 - 125	< 25	wet	Dry
	Rating	0	1 - 4	5 - 7	8 - 10	11 – 15

	Blasting Adjustment	natural slope	pre-split	smooth	normal	Deficient
		15	10	8	0	- 8

Depending on the RMR, the rock mass can be classified as given in the following table (Table2.5).

Table 2.5: Description of RMR classes (after Bieniawski, 1973)

RMR	**Roof Class**	**Roof Description**
00 – 20	V	Very Poor

20 – 40	IV	Poor
40 – 60	III	Fair
60 – 80	II	Good
80 – 100	I	Very Good

Geomechanics classification is a versatile system, and therefore has found applications in several countries and several types of excavations in rock.

Slope Mass Rating System

Slope mass rating is a system of classification developed by Romana (1985, 1991) as an extension of Bieniawski's (1979, 1989) rock mass rating approach for application to rock slopes. RMR (widely used in tunnels) is not of immediate use for slopes due to the fact that joints are a more governing parameter for stability in slopes. In order to assess slope instability risk, some parameters are introduced to include the attitude of discontinuities, the slope failure modes and slope excavation methods.

Romana (1985) made an important contribution in applying rock mass classifications to the assessment of the stability of the rock slopes. He developed a factorial approach to rating adjustment for the discontinuity orientation parameter in the RMR system, based on field data. Recognizing that rock slope stability is governed by the behaviour of the discontinuities, his modification of the RMR system involved subtracting the newly proposed adjustment factors for discontinuity orientation and adding a new adjustment factor for the method of excavation. This approach is suitable for preliminary assessment of slope stability in rock, including very soft or heavily jointed rock masses (Bieniawski, 1979).

The RMR is computed according to Bieniawski's 1979 proposal, adding rating values for five parameters.The proposed 'Slope Mass Rating' is obtained from RMR through a factorial adjustment depending on the joints-slope relationship and adding a factor depending on the method of excavation (Laubscher, 1990; Romana, 1993), as given below:

$$\mathbf{SMR = RMR + (F1 * F2 * F3) + F4} \qquad \text{(Eqn. 2.3)}$$

The adjustment rating for joints (Table 2.6) is the product of three factors as explained below.

F1 depends on the parallelism between joints and slope face strike. It ranges from 1 (when both are near parallel) to 0.15 (when the angle between them is more than 30 degrees and the failure probability is very low). These values are found to match approximately the relationship:

$$\mathbf{F1 = (1 - \sin A)^2} \qquad \text{(Eqn. 2.4)}$$

where A denotes the angle between the strikes of slope face and joint.

F2 is related to joint dip angle in the planar mode of failure. It is a measure of the joint shear strength. Its value varies from 1 (for joints dipping more than 45 degrees) to 0.15 (for joints dipping less than 20 degrees). It has been found to match approximately the relationship:

$$\mathbf{F2 = \tan^2 (B_j)} \qquad \text{(Eqn. 2.5)}$$

where B_j denotes joint dip angle. For toppling mode of failure, F2 remains 1.

F3 reflects the relationship between the slope face and the joint dip.

F4 is a factor for the method of excavation.

Table 2.6 : Adjustment ratings for joints (after Romana, 1993)

Case	Very favorable	Favorable	Fair	Un-favorable	Very un-favorable
P : $\lvert \alpha_j-\alpha_s \rvert$ T : $\lvert(\alpha_j-\alpha_s)-180^o\rvert$	$>30^o$	$30-20^o$	$20-10^o$	$10-5^o$	$<5^o$
For P/T : F_1	**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
P : $\lvert \beta_j \rvert$	$<20^o$	$20-30^o$	$30-35^o$	$35-45^o$	$>45^o$
For P : F2	**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
For T : F2	**1**	**1**	**1**	**1**	**1**
P : $\beta_j - \beta_s$	$>10^o$	$10-0^o$	0^o	0^o to -10^o	$<-10^o$
T : $\beta_j + \beta_s$	$<110^o$	$110-120^o$	$>120^o$	-	-
For P or T : F_3	**0**	**-6**	**-25**	**-50**	**-60**

(P = for plane failure; T = for toppling failure;

α_j, joint dip direction; α_s, slope dip direction; β_j, joint dip; β_s, slope dip).

The adjustment factor for the method of excavation has been fixed empirically as shown in Table 2.7.

Table 2.7: Adjustment ratings for method of excavation (after Romana, 1993)

Method	Natural slope	Presplitting	Smooth blasting	Blasting or mechanical	Deficient blasting
F4	+15	+10	+8	0	-8

Table 2.8: Tentative description of the SMR classes (after Romana, 1993)

Class	SMR	Description	Stability	Failures	Support
I	81-100	Very good	Completely stable	None	None
II	61-80	Good	Stable	Some blocks	Occasional
III	41-60	Normal	Partially stable	Some joints or many wedges	Systematic
IV	21-40	Bad	Unstable	Planar or big wedges	Important/ corrective
V	00-20	Very bad	Completely unstable	Big planar or soil like	Re-excavation

It can be seen that SMR is mostly less than or equal to the original RMR value. Only when the excavation is made by pre-splitting or smooth blasting, SMR may be more than RMR. The classification must be applied for each joint system. The lower value of SMR is retained for design of the slope. In certain evolutive rocks (marls and clay shales) the slopes are stable during the working period, and may fail afterwards (usually one to two years later). The classification must be applied twice: for the present fresh conditions (actual conditions) and future weathered conditions (based on prognosis). The worst possible water condition must be assumed (Romana, 1993).

2.3.3 Chinese Slope Mass Rating System

CSMR system (Romana, 1995; Zuyu, 1995) introduces two coefficients E and L and modifies Eqn. 2.3 as follows.

$$\mathbf{CSMR = (\xi * RMR) + [\lambda * F1*F2*F3 + F4]} \qquad \text{(Eqn. 2.6)}$$

where,

ξ is the slope height factor = 0.57+0.43*80/H

H is the height of slope in meters

λ is the discontinuity condition factor

= 1 for faults, long weak seams filled with clay

= 0.8 to 0.9 for bedding planes, large scale joints with gauge

= 0.7 for joints, tightly interlocked bedding planes

and

F1, F2, F3 and F4 are the adjustment factors from SMR.

The factor ξ is applicable only for heights greater than 40 m. However, this is not yet an accepted system of classification, and needs a number of corrections and modifications.

2.4 NUMERICAL MODELLING

Numerical models are computer programs that attempt to represent the mechanical response of a rock mass subjected to a set of initial conditions such as *in situ* stresses and water levels, boundary conditions and induced changes such as slope excavation. The result of a numerical model simulation typically is either equilibrium or collapse. If equilibrium result is obtained, the resultant stresses and displacements at any point in the rock mass can be compared with measured values. If a collapse result is obtained, the predicted mode of failure is demonstrated.

Numerical models divide the rock mass into zones. Each zone is assigned a material model and properties. The material models are idealized stress/strain relations that describes how the material behaves. The simplest model is a linear elastic model, which uses the elastic properties(Young's modulus and poisson's ratio) of the material. Elastic-plastic models use strength parameters to limit the shear stress that a zone may sustain.

The zones may be connected together. Termed a continuum model, or separated by discontinuities, termed a discontinum model. Discontinuum models allow slip and separation at explicitly located surfaces within the model.

Numerical models tend to be general purpose in nature- that is, they are capable of solving a wide variety of problems. While it is often desirable to have a general-purpose tool available, it requires that each problem be constructed individually. The zones must be arranged by the user to fit the limits of the geomechanical units and the slope geometry. Hence, the numerical models often require more time to set up and run than special-purpose tools such as limit equilibrium methods.

There are several reasons why numerial models are used for slope stability studies.

i. Numerical models can be extrapolated confidently outside their databases in comparison to empirical methods in which the failure mode is explicitly defined.
ii. Numerical analysis can incorporate key geo-logic features such as faults and ground water providing more realistic approximations of behaviour of real slopes than analytic models. In comparison, non-numerical analysis methods such as analytic,

physical or equilibrium may be unsuitable for some sites or tend to oversimplify the conditions, possibly leading to overly conservative solutions.

iii. Numerical analysis can help to explain observed physical behaviour.

iv. Numerical analysis can evaluate multiple possibilities of geological models, failure modes nad design options.

All rock slopes involve discontinuities. Representation of these discontinuities in numerical models differ depending on the type of model. There are two basic types of model: discontinuum models and continuum models discussed in Table 2.9.

Table 2.9 Numerical methods of analysis

Analysis method	Critical input parameters	Advantages	Limitations
Continuum Modelling (e.g. Finite Element, Finite Difference Method)	Representative slope geometry;constitutive criteria (e.g. elastic, elasto-plastic, creep etc.); groundwater characteristics; shear strength of surfaces; in situ stress state.	Allows for material deformation and failure. Can model complex behaviour and mechanisms. Capability of 3-D modelling. Can model effects of groundwater and pore pressures. Able to assess effects of parameter variations on instability. Recent advances in computing hardware allow complex models to be solved on PC's with reasonable run times. Can incorporate creep deformation. Can incorporate dynamic analysis.	Users must be well trained, experienced and observe good modelling practice. Need to be aware of model/software limitations (e.g. boundary effects, mesh aspect ratios, symmetry, hardware memory restrictions). Availability of input data generally poor. Required input parameters not routinely measured. Inability to model effects of highly jointed rock. Can be difficult to perform sensitivity analysis due to run time constraints.

Discontinuum Modelling (e.g. Distinct Element, Discrete Element Method)	Representative slope and discontinuity geometry; intact constitutive criteria; discontinuity stiffness and shear strength; groundwater characteristics; in situ stress state.	Allows for block deformation and movement of blocks relative to each other. Can model complex behavior and mechanisms (combined material and discontinuity behavior coupled with hydromechanical and dynamic analysis). Able to assess effects of parameter variations on instability.	As above, experienced user required to observe good modeling practice. General limitations similar to those listed above. Need to be aware of scale effects. Need to simulate representative discontinuity geometry (spacing, persistence, etc.). Limited data on joint properties available.
Hybrid/Coupled Modelling	Combination of input parameters listed above for stand-alone models.	Coupled finiteelement/ distinct element models able to simulate intact fracture propagation and fragmentation of jointed and bedded media.	Complex problems require high memory capacity. Comparatively little practical experience in use. Requires ongoing calibration and constraints.

2.4.1 Continuum Modelling

Continuum modelling is best suited for the analysis of slopes that are comprised of massive, intact rock, weak rocks, and soil-like or heavily fractured rock masses. Most continuum codes incorporate a facility for including discrete fractures such as faults and bedding planes but are inappropriate for the analysis of blocky mediums. The continuum approaches used in rock slope stability include the finite-difference and finite-element methods. In recent years the vast majority of published continuum rock slope analyses have used the 2-D finite-difference code, FLAC. This code allows a wide choice of constitutive models to characterize the rock mass and incorporates time dependent behaviour, coupled hydro-mechanical and dynamic modelling.

Two-dimensional continuum codes assume plane strain conditions, which are frequently not valid in inhomogeneous rock slopes with varying structure, lithology and topography. The recent advent of 3-D continuum codes such as FLAC3D and VISAGE enables the engineer to undertake 3-D analyses of rock slopes on a desktop computer. Although 2-D and 3-D continuum codes are extremely useful in characterizing rock slope failure mechanisms it is the responsibility of the engineer to verify whether they are representative of the rock mass under consideration. Where a rock slope comprises multiple joint sets, which control the mechanism of failure, then a discontinuum modelling approach may be considered more appropriate.

2.4.2 Discontinuum Modelling

Discontinuum methods treat the rock slope as a discontinuous rock mass by considering it as an assemblage of rigid or deformable blocks. The analysis includes sliding along and opening/closure of rock discontinuities controlled principally by the joint normal and joint shear stiffness. Discontinuum modelling constitutes the most commonly applied numerical approach to rock slope analysis, the most popular method being the distinct-element method. Distinctelement codes such as UDEC use a force-displacement law specifying interaction between the deformable joint bounded blocks and Newton's second law of motion, providing displacements induced within the rock slope.

UDEC is particularly well suited to problems involving jointed media and has been used extensively in the investigation of both landslides and surface mine slopes. The influence of external factors such as underground mining, earthquakes and groundwater pressure on block sliding and deformation can also be simulated.

2.4.3 Hybrid Techniques

Hybrid approaches are increasingly being adopted in rock slope analysis. This may include combined analyses using limit equilibrium stability analysis and finite-element groundwater flow and stress analysis such as adopted in the GEO-SLOPE suite of software. Hybrid numerical models have been used for a considerable time in underground rock engineering including coupled boundary-/finite-element and coupled boundary-/distinct-element solutions. Recent advances include coupled particle flow and finite-difference analyses using FLAC3D and PFC3D. These hybrid techniques already show significant potential in the investigation of such phenomena as piping slope failures, and the influence of high groundwater pressures on the failure of weak rock slopes. Coupled finite-/distinct-element codes are now available which incorporate adaptive remeshing. These methods use a finite-element mesh to represent either the rock slope or joint bounded block. This is coupled with a discrete -element model able to model deformation involving joints. If the stresses within the rock slope exceed the failure criteria within the finite-element model a crack is initiated. Remeshing allows the propagation of the cracks through the finite-element mesh to be simulated. Hybrid codes with adaptive remeshing routines, such as ELFEN, have been successfully applied to the simulation of intense fracturing associated with surface mine blasting, mineral grinding, retaining wall failure and underground rock caving.

2.5 GENERAL APPROACH OF FLAC

The modeling of geo-engineering processes involves special considerations and a design philosophy different from that followed for design with fabricated materials. Analyses and designs for structures and excavations in or on rocks and soils must be achieved with relatively little site-specific data, and an awareness that deformability and strength properties may vary considerably. It is impossible to obtain complete field data at a rock or soil site.

Since the input data necessary for design predictions are limited, a numerical model in geomechanics should be used primarily to understand the dominant mechanisms affecting the behavior of the system. Once the behavior of the system is understood, it is then appropriate to develop simple calculations for a design process.

It is possible to use FLAC directly in design if sufficient data, as well as an understanding of material behavior, are available. The results produced in a FLAC analysis will be accurate when the program is supplied with appropriate data. Modelers should recognize that there is a continuous spectrum of situations, as illustrated in Figure 2.5, below.

Typical Situation	Complicated geology; inaccessible; no testing budget	⟷	Simple geology; Lots of money spent on site investigation
Data	None	⟷	Complete

Approach	Investigation of mechanisms	← Bracket field behavior by parameter studies →	Predictive (direct use in design)

Fig. 2.5 Spectrum of modeling situations

FLAC may be used either in a fully predictive mode (right-hand side of Fig. 2.5) or as a "numerical laboratory" to test ideas (left-hand side). It is the field situation (and budget), rather than the program, that determine the types of use. If enough data of a high quality are available, FLAC can give good predictions.

The model should never be considered as a "black box" that accepts data input at one end and produces a prediction of behavior at the other. The numerical "sample" must be prepared carefully, and several samples tested, to gain an understanding of the problem. Table 2.10 lists the steps recommended to perform a successful numerical experiment; each step is discussed separately.

Table 2.10 Recommended steps for numerical analysis in geomechanics

Step1	Define the objectives for the model analysis
Step2	Create a conceptual picture of the physical system
Step3	Construct and run simple idealized models
Step4	Assemble problem-specific data
Step5	Prepare a series of detailed model runs
Step6	Perform the model calculations
Step7	Present results for interpretation

2.5.1 Define the Objectives for the Model Analysis

The level of detail to be included in a model often depends on the purpose of the analysis. For example, if the objective is to decide between two conflicting mechanisms that are proposed to explain the behavior of a system, then a crude model may be constructed, provided that it allows the mechanisms to occur. It is tempting to include complexity in a model just because it exists in reality. However, complicating features should be omitted if they are likely to have little influence on the response of the model, or if they are irrelevant to the model's purpose. Start with a global view and add refinement if necessary.

2.5.2 Create a Conceptual Picture of the Physical System

It is important to have a conceptual picture of the problem to provide an initial estimate of the expected behavior under the imposed conditions. Several questions should be asked when preparing this picture. For example, is it anticipated that the system could become unstable? Is the predominant mechanical response linear or nonlinear? Are movements expected to be large or small in comparison with the sizes of objects within the problem region? Are there welldefined discontinuities that may affect the behavior, or does the material behave essentially as a continuum? Is there an influence from groundwater interaction? Is the system bounded by physical structures, or do its boundaries extend to infinity? Is there any geometric symmetry in the physical structure of the system?

These considerations will dictate the gross characteristics of the numerical model, such as the design of the model geometry, the types of material models, the boundary conditions, and the initial equilibrium state for the analysis. They will determine whether a three-dimensional model is required, or if a two-dimensional model can be used to take advantage of geometric conditions in the physical system.

2.5.3 Construct and Run Simple Idealized Models

When idealizing a physical system for numerical analysis, it is more efficient to construct and run simple test models first, before building the detailed model. Simple models should be created at the earliest possible stage in a project to generate both data and understanding. The results can provide further insight into the conceptual picture of the system; Step 2 may need to be repeated after simple models are run. Simple models can reveal shortcomings that can be remedied before any significant effort is invested in the analysis. For example, do the selected material models sufficiently represent the expected behavior? Are the boundary conditions influencing the model response? The results from the simple models can also help guide the plan for data collection by identifying which parameters have the most influence on the analysis.

2.5.4 Assemble Problem-Specific Data

The types of data required for a model analysis include:

- details of the geometry (e.g., profile of underground openings, surface topography, dam profile, rock/soil structure);
- locations of geologic structure (e.g., faults, bedding planes, joint sets);
- material behavior (e.g., elastic/plastic properties, post-failure behavior);
- initial conditions (e.g., in-situ state of stress, pore pressures, saturation); and
- external loading (e.g., explosive loading, pressurized cavern).

Since, typically, there are large uncertainties associated with specific conditions (in particular, state of stress, deformability and strength properties), a reasonable range of parameters must be selected for the investigation. The results from the simple model runs (in Step 3) can often prove helpful in determining this range, and in providing insight for the design of laboratory and field experiments to collect the needed data.

2.5.5 Prepare a Series of Detailed Model Runs

Most often, the numerical analysis will involve a series of computer simulations that include the different mechanisms under investigation and span the range of parameters derived from the assembled database. When preparing a set of model runs for calculation, several aspects, such as those listed below, should be considered.

I. How much time is required to perform each model calculation? It can be difficult to obtain sufficient information to arrive at a useful conclusion if model runtimes are excessive. Consideration should be given to performing parameter variations on multiple computers to shorten the total computation time.

II. The state of the model should be saved at several intermediate stages so that the entire run does not have to be repeated for each parameter variation. For example, if the analysis involves several loading/unloading stages, the user should be able to return to any stage, change a parameter and continue the analysis from that stage.

III. Are there a sufficient number of monitoring locations in the model to provide for a clear interpretation of model results and for comparison with physical data? It is helpful to locate several points in the model at which a record of the change of a parameter (such as displacement) can be monitored during the calculation.

2.5.6 Perform the Model Calculations

It is best to first make one or two model runs split into separate sections before launching a series of complete runs. The runs should be checked at each stage to ensure that the response is as expected. Once there is assurance that the model is performing correctly, several data files can be linked together to run a complete calculation sequence. At any time during a sequence of runs, it should be possible to interrupt the calculation, view the results, and then continue or modify the model as appropriate.

2.5.7 Present Results for Interpretation

The final stage of problem solving is the presentation of the results for a clear interpretation of the analysis. This is best accomplished by displaying the results graphically, either directly on the computer screen, or as output to a hardcopy plotting device. The graphical output should be presented in a format that can be directly compared to field measurements and observations. Plots should clearly identify regions of interest from the analysis, such as locations of calculated stress concentrations, or areas of stable movement versus unstable movement in the model. The numeric values of any variable in the model should also be readily available for more detailed interpretation by the modeler.

The above seven steps are to be followed to solve geo-engineering problems efficiently.

2.6 Overview

FLAC/Slope is a mini-version of FLAC that is designed specifically to perform factor-of-safety calculations for slope stability analysis. This version is operated entirely from FLAC's graphical interface (the GIIC) which provides for rapid creation of models for soil and/or rock slopes and solution of their stability condition. FLAC/Slope provides an alternative to traditional "limit equilibrium" programs to determine factor of safety. Limit equilibrium codes use an approximate scheme — typically based on the method of slices — in which a number of assumptions are made (e.g., the location and angle interslice forces). Several assumed failure surfaces are tested, and the one giving the lowest factor of safety is chosen. Equilibrium is only satisfied on an idealized set of surfaces. contrast, it provides a full solution of the coupled stress/displacement, equilibrium and constitutive equations. Given a set of properties, the system is determined to be stable unstable. By automatically performing a series of simulations while changing the strength properties, the factor of safety can be found to correspond to the point of stability, and the critical failure (slip) surface can be located.

FLAC/Slope does take longer to determine a factor of safety than a limit equilibrium program. However, with the advancement of computer processing speeds (e.g., 1 GHz and faster chips), solutions can now be obtained in a reasonable amount of time. This makes FLAC/Slope a practical alternative to a limit equilibrium program, and provides advantages over a limit equilibrium solution:

1. Any failure mode develops naturally; there is no need to specify a range of trial surfaces in advance.
2. No artificial parameters (e.g., functions for interslice force angles) need to be given as input.
3. Multiple failure surfaces (or complex internal yielding) evolve naturally, if the conditions give rise to them.
4. Structural interaction (e.g., rock bolt, soil nail or geogrid) is modeled realistically as fully coupled deforming elements, not simply as equivalent forces.
5. The solution consists of mechanisms that are kinematically feasible. (The limit equilibrium method only considers forces, not kinematics.)

CHAPTER: 03

CASE-STUDY

Empirical models, numerical modeling studies were conducted for understanding the stability of the pit slopes and presented in this report for the depth of 120 m for the SRP-OC1 mine under SCCL. Overview of srirampur opencast mine is shown in Fig 3.1.

Fig 3.1: Overview of srirampur opencast mine

3.1 DETAILS OF THE MINE

The SRP OCP-1 mine is situated in the southern part of Somagudem indaram coal belt in Karimnagar District of Andhra Pradesh **.**The pit geometry and the identified cross sections were provided by the mine management. Presently the mine is working seam no II, III, IIIA, and IIIB with shovel – dumper combination, and planned to be extended up to a depth of 120 m in the first phase and to be extended later as a part of SRP-II project. During the process of coal mining the overlying strata consisting of top soil and sedimentary rock formation has been removed as

Over burden during different stage of mining. The present studies are to assess the stability of the overall pit slope.

The area is underlain by deep black cotton soil ranging in thickness from 5 to 8 m, below which brown medium grained alluvial sand of about 10 m thickness is present. Below this soil cover the lower Gondwana group of Permian age comprising Talchir, Barakar and Barren Measures formations occur resting unconformably on the Sullavai group of rocks of Proterozoic age. The general trend of the Gondwana formations is NW-SE, with north-easterly dips of about 8 to 10^o. The average RMR for SRP OCP rock formations was estimated to be around 35. The SRP OCP block is highly disturbed by faults and joints and bedding planes. According to the RMR classification, it is designated as poor quality rock. The more prominent joint set is bedding plane with a dip of 5^0 to 15^0 and dip direction of 30^0 to 60^0.

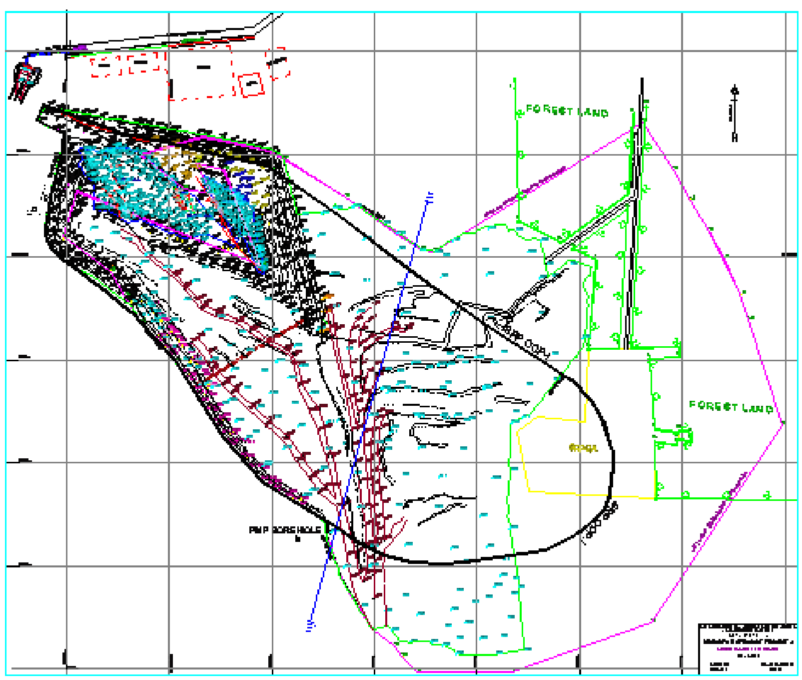

Fig 3.2 Present working condition of the mine

Present condition of the mine is shown in the Fig 3.2 along with the location of PMP Bore hole made for the purpose of obtaining physico-mechanical properties of the strata.

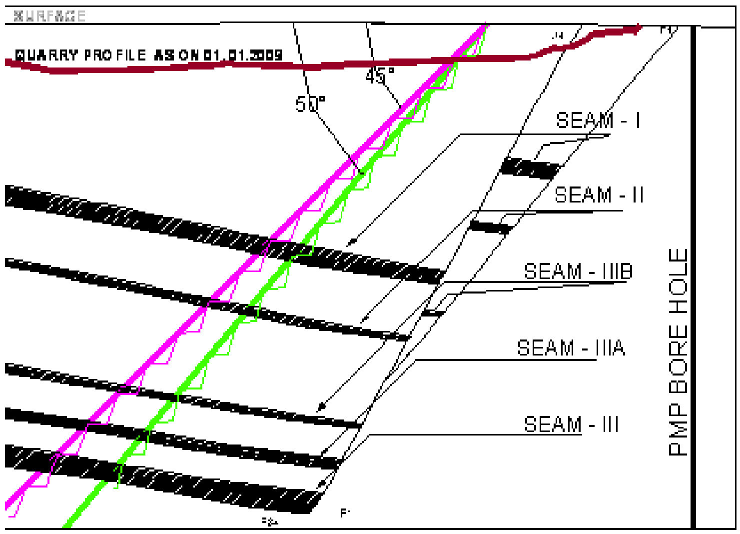

Fig.3.3: current working seams in srirampur mine

3.2 PHYSICO-MECHANICAL PORPERITES

The litho units observed in PMP hole and the working benches are Greenish grey shale with boulders, Fine grained to coarse grained sandstone, shale with calcareous filling etc with the coal seams. Density of the strata varying from 2.1 to 2.68 g/cc while compressive strength varying from 78 to 324 kg/sq.cm. Young’s modulus and tensile strength was about 0.24 - 0.71 , 10 to 25 kg/sq.cm. The properties of the strata for Bore hole SBH 357 and SBH 358 presented in Table 3.1 and Table 3.2, respectively.

Table 3.1: Physico Mechanical properties of the strata (B.H No. SBH 357)

Depth		Thickness	Strata	Diameter(cm)	Density(gm/cm^{3})	Tensile strength	Compressive Strength	Young's modulus	Shear strength	Impact strength No.	Protodyknov strength index
From	To										
11	36	25.0	Vfg SST shaly matrix	4.62	2.34	47.68	343	0.75	92.6	51.7	
36	40	4.0	F to Mg huge quartz boulder	4.62	2.62	25.65	296	0.66	63.1	50.9	1.26
40	51	11.0	F to Mg SST huge quartz boulder	4.62	2.3	0.0	95	0.28	0.00	47.3	0.3

51	53	2.0	Vfg to Mg SST with pebbles at bottom	4.62	2.4	-	196	0.77	-	49.1	
53	58	5.0		4.62	2.74	47.47		0.9		53.7	
58	94	36.0	Vfg to Mg SST with pebbles at bottom	4.62	2.46	27.44	357	0.77	71.7	51.9	
94	99	5.0	Boulder bed silico hard compact	4.62	2.64	27.0	218	0.51	55.6	49.5	

118	130	12.0	Shale with quartzite huge boulders	4.62	2.29	-	195	0.3	-	17.5	
		100									

Table 3.2: Physico Mechanical properties of the strata (B.H No. SBH 358)

Depth		Thickness	Strata	Diameter(cm)	Density(gm/cm$^{3)}$	Tensile strength	Compressive Strength	Young's modulus	Shear strength	Impact strength No.	Protodyknov strength index
From	To										
12.0	19.0	7.0	Shale with boulders	4.64	2.15	-	78	0.24	-	47.01	0.21
		7.45	Shaly SST, huge boulders	4.64	2.19	-	147	0.38	-	48.24	0.5
37.69	47.5	9.0	Vfg SST	4.64	2.2	-	214	0.50	-		
47.5	53.3	5.8	Vfg SST MH,	4.64	2.22	-	260	0.59	-	50	

			quartzite with pebbles								
54.04	66.0	11.96	Vfg SST MH thinly laminated	4.64	2.23	-	207	0.49	-	49.31	
66.0	67.84	1.84	Vfg SST MH massive	4.64	2.33	-	221	0.52	-	49.56	0.8
67.84	70.76	2.92	Mg SST MH	4.64	2.14	10.4	131	0.34	26.7	47.96	0.47
70.76	87.76	17	F to Mg SST MH shaly	4.64	2.26	18.9	161	0.40	39.9	48.49	
87.76	90.3	2.54	Boulder bed at bottom	4.64	2.68	38.8	256	0.58	72.2	50.19	
90.3	100.6	10.32	Shale sandy at middle	4.64	2.20	-	192	0.46	-	49.05	
100.6	108.6	8.02	SST with quartzite boulders	4.64	2.24	-	276		-		
108.6	117.7	9.06	Shale with calcareous filings	4.64	2.25	-	311		-		
117.7	134.8	17.16	Shale with quartzite boulders	4.64	2.25	-	324	0.71	-	51.40	
134.8	151.0	15.13	Vfg SST MH thinly	4.64	2.31	25.7	295	0.66	63	50.88	

			laminated								
		127.0									

A number of fault/shear planes were observed. The fault plane dipping into slope face and whose direction is same as that of slope face is potential for plane failure. Some of the fault planes are dipping against slope face whose direction is not that of slope face may not be potential for failure. Therefore, precautions need to be taken for stability of the benches and there may be a need to extend the mine boundary beyond the fault F1 & F34 dipping 53 & 58 degrees at a distance of 30 and 60m from the existing boundary. There may be a scope of slip along the fault planes due to slenderness of the strata with the interface –Fault and the ultimate pit slope profile.

3.3 MONITORING

Cracks were observed along the fault plane F1 during First week of Feb10, and monitoring of the cracks indicated a maximum vertical and horizontal displacement of 10 cms, and 11 cms , respectively during 10.2.10 to 24.3.10. Horizontal and vertical displacement was not perceptible after 24.2.10, and 10.3.10, respectively. Based on field studies, laboratory test results and analysis results the bench parameters and slope angles were designed and are given in this report. Fig. 3.4 shows cracks on the surface due tro failure of slope because of presence of faults at srirampur OCP-1 mine.

Fig. 3.4: Cracks in bed of srirampur mine

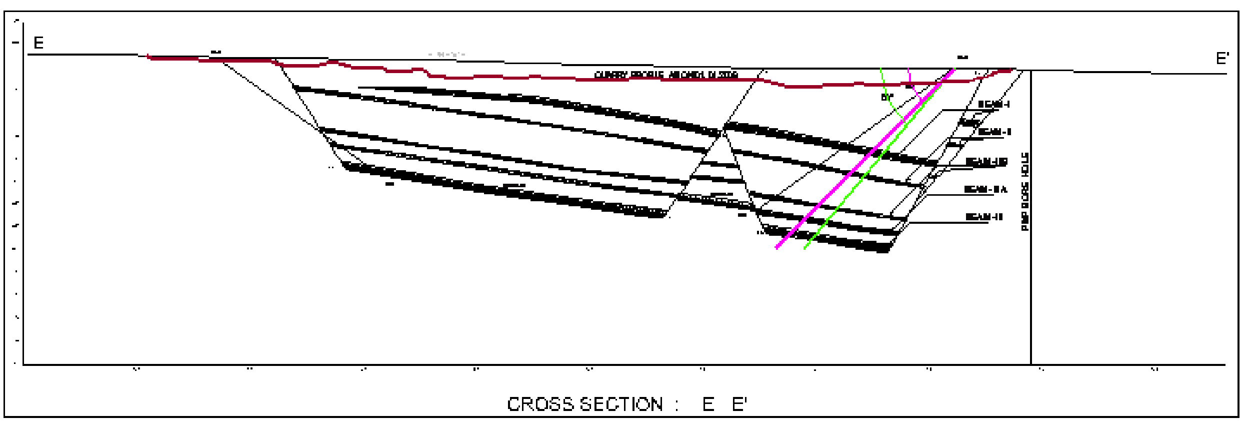

Fig.3.5: faults present in Srirampur mine

Monitoring of movement of cracks was conducted from 10.02.10 to 24.03.10. Total number of Cracks observed were 27, and six monitoring Stations were fixed for understanding the extent of slope instability and displacement along the cracks/fault plane. Details of monitoring is presented in Table 3.3.

Table 3.3: Details of monitoring of displacement along cracks.

Station Number	Horizontal displacement			Vertical displacement			Remarks
	Starting	Ending	Difference	Starting	Ending	Difference	
01	11Cms	13cms	2cms	22.5cms	27cms	4.5cms	Horizontal displacement constant from25.02.09 Vertical displacement constant from 10.03.09
02	7cms	12cms	.5cms	14.5cms	24cms	9.5cms	Horizontal displacement constant from10.03.09 Vertical displacement constant from 10.03.09
03	6cms	9cms	3cms	6cms	16cms	10cms	Horizontal displacement constant from26.02.09 Vertical displacement constant from 10.03.09

04	7cms	9cms	2cms	7cms	16cms	9cms	Horizontal displacement constant from24.02.09 Vertical displacement constant from 10.03.09
05	7cms	12cms	5cms	13.5cms	19cms	5.5cms	Horizontal displacement constant from10.03.09 Vertical displacement constant from 10.03.09
06	7cms	10cms	3cms	9cms	17cms	8cms	Horizontal displacement constant from07.03.09 Vertical displacement constant from 06.03.09
00	12cms	23cms	11cms	12cms	15cms	3cms	Horizontal displacement and Vertical displacment constant from14.03.09

Following are the salient observations of the above monitoring of displacements along the cracks.

1. Depth of the cracks increased from 0.7 meters to 1.13meters (Max) in some of the Points.

2. Horizontal displacement is constant from 24.02.10
3. Vertical displacement is constant from 10.03.10 in most of the stations
4. The Survey station no. FNL-6 at the south edge of the quarry was observed for the movement of cracks from 11.02.10 to 16.02.10 and the difference observed in the station is 0.49 meters from the original position.

The overall slope angle is formed by line joining the crest of the top most bench and toe of the bottom bench of the pit. For the purpose of this project the factor of safety of 1.2 is considered for long term stability, and it was assumed that controlled blasting would be undertaken at ultimate pit limits.

CHAPTER 4

ANALYSIS AND RESULTS

The overall slope angle is formed by line joining the crest of the top most bench and toe of the bottom bench of the pit. For the purpose of this project the factor of safety of 1.2 is considered for long term stability, and it was assumed that controlled blasting would be undertaken at ultimate pit limits.

4.1 SLOPE MASS RATING

The lowest value of MSMR was related to toppling failure mode, whereas the MSMR exceeded 40 for plane failure mode These values indicate normal condition of the slope in accordance with the field observation. Modified slope mass rating (MSMR) is estimated as follows:

JS = joint set no. (1, 2, 3, .. are joint sets; S = schistocity; B = bedding plane)

Jd = joint dip amount ; Sd = slope dip amount;

RMR = Rock Mass Rating ; H = bench height, in m

F1, F2, MF3, F4 = adjustment factors for conversion of RMR to MSMR;

MSMR = Modified Slope Mass Rating

Jdd = joint dip direction; Sdd = slope dip direction;

SMR = Slope Mass Rating

MF3 = modified F3 factor ; F4 = 0 in all cases

Table 4.1 Estimation of MSMR

JS	Jd	Jdd	Sdd	Sd	RMR	H	Planar Mode of Failure					Toppling Mode of Failure				
							F1	**F2**	**MF3**	**SMR**	**MSMR**	**F1**	**F2**	**MF3**	**SMR**	**MSMR**
B	10	45	45	55	35	67.5	1	0.15	25	37.75	**33.4**	0.15	1	-25	31.25	**41.41**

As per the above MSRMR, slope angles suggested for the site are as follows:

Accordingly, the following trends could be established for relating the MSMR with the individual bench angle (S_b) and the overall slope angle (S_o) :

For individual bench angle -

$$\mathbf{S_b = 22 * \ln (MSMR) - 18} \qquad \text{(Eqn. 4.1)}$$

S_b = 22* ln(41.41) – 18

S_b = **63.9** $^{\mathbf{0}}$

For overall slope angle -

$$\mathbf{S_o = 14 * \ln (MSMR) - 16} \qquad \text{(Eqn. 4.2)}$$

S_o = 14* ln(41.41) – 16

S_o = **36.12** $^{\mathbf{0}}$

Thus the safe slope angle for the bench and the overall slope are **63.9**$^{\mathbf{0}}$ and **36.12**$^{\mathbf{0}}$ respectively. But the above value of MSMR could not consider the effect of faults directly, therefore, numerical modeling by simulation of major fault was conducted.

4.2 NUMERICAL MODELLING

The typical analysis results for the failure planes passing through rock mass for the slope angel of 45 degrees and the fault plane are given in Fig. 4.1 .

Velocity vectors and plasticity indicators showed that the slope with fault is unstable, and the safety factor is below 1.2. Therefore, the analysis of slope stability without fault was undertaken as parametric study with overall slope angles ranging from 40 to 55 degrees. Table 4.2 shows the safety factor of slopes with varying overall slope angles. The analysis of stability of slopes for the ultimate pit slope indicated the safety factor exceeding 1.2 for slope angle of 48 degrees without consideration of the faults. The typical analysis results for the slope angel of 45 degrees without considering fault plane is given in fig 4.1 . However, the presence of fault F1 decreased the safety factor below 1.2. Therefore it is recommended to extend the boundary of the mine beyond the fault F1, and maintain the overall slope angle not steeper than 48 degrees. Cracks observed along the fault plane F1 during First week of Feb10, and monitoring of the cracks indicated a maximum vertical and horizontal displacement of 10 cms, and 11 cms respectively during 10.2.10 to 24.3.10. Horizontal and vertical displacement was not perceptible after 24.2.10, and 10.3.10, respectively.

Table 4.2: safety factors of slopes with varying slope angles

Slope angle, degrees	Safety factor
40	1.37
45	1.29
48	1.2
50	1.13
55	1.02

Therefore, it is proposed to extend the boundary from the south side edge of the quarry up to 130 m (Max) from the previous edge of the quarry. In view of the instability due to faults, it is also recommended that the extraction of the OB in the first bench shall at a stretch all along the boundary and then only excavation in the 2nd bench should start. It is also strongly recommended to monitor the movement along fault plane till the excavation reaches the proposed extended boundary.

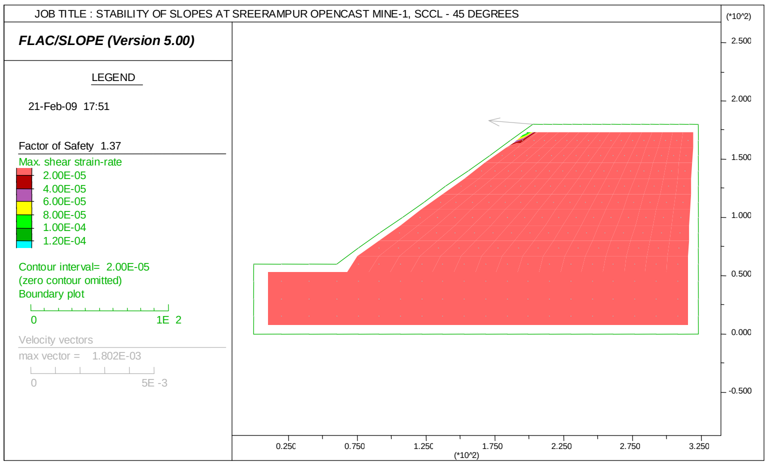

Fig.4.1 Analysis result for the slope angel of 45 degree for the high wall in the absent of faults

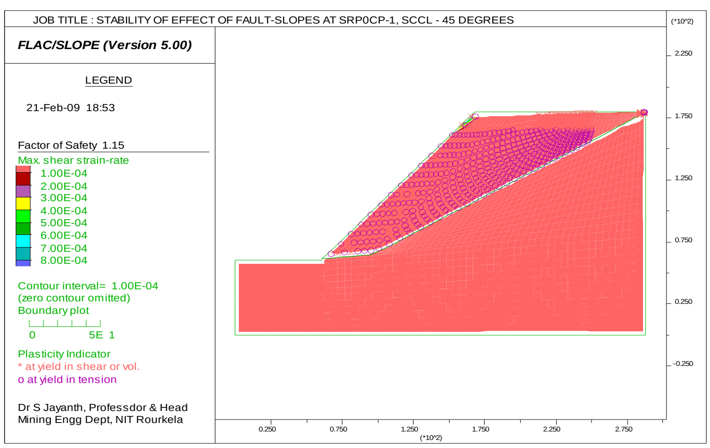

Fig.4.2 Shows unstable slope with 45 degree angle due to presence of fault (FoS< 1.2)

Fault was situated in the slope (Fig4.2), and the safety factor of slope was 1.15 with 45 degree angle, which is unstable. The analysis of stability of slopes for the ultimate pit slope indicated the safety factor exceeding 1.2 for slope angle of 48 degrees without consideration of the faults. However, the presence of fault F1 decreased the safety factor below 1.

Therefore it is recommended to extend the boundary of the mine beyond the fault F1, and maintain the overall slope angle not steeper than 48 degrees.

CHAPTER 5

CONCLUSION & RECOMMENDATIONS

Based on numerical model analysis results for the case it is concluded that bench failures are likely to occur because of discontinuities in the form of faults. Fault F1 located along the proposed boundary appears to pose instability problems to the high walls. Therefore, it is strongly recommended to monitor the slopes for its stability.

The analysis of stability of slopes for the ultimate pit slope indicated the safety factor exceeding 1.2 for slope angle of 48 degrees without consideration of the faults. However, the presence of fault F1 decreased the safety factor below 1. Therefore it is recommended to extend the boundary of the mine beyond the fault F1, and maintain the overall slope angle not steeper than 48 degrees.

Therefore, it is proposed to extend the boundary from the south side edge of the quarry up to 130 m (Max) from the previous edge of the quarry. In view of the instability due to faults, it is also recommended that the extraction of the OB in the first bench shall at a stretch all along the boundary and then only excavation in the 2nd bench should start. It is also strongly recommended to monitor the movement along fault plane till the excavation reaches the proposed extended boundary.

In view of the instability due to faults, it is recommended to extract the disturbed area in the following manner;

1. It is recommended to extend the boundary from the south side edge of the quarry upto 130 m (max) from the previous edge of the quarry.
2. It is necessary that the dip amount and dip direction of prominent faults be determined. Such information will help in better planning of mine workings.
3. External loading on the top of slope in the form of overburden dumps be avoided, as such situations adversely affects the stress equilibrium inside the slope. The overburden dumps should be preferable placed in the decoaled areas or away from the mine boundary.

4. The extraction of the OB in the first bench is to be removed at a stretch all along the boundary and then only excavation in the 2nd bench starts.
5. Whenever a fault or slip is encountered the area will be compacted and made level before deploying shovel .
6. The benches will be worked from top to downwards.
7. The shovel should be positioned perpendicular to the benches.
8. In addition to this, geo-technical mapping be carried out periodically to ascertain the new exposures and the impact of structural features on slope stability. To avoid sliding along the fault plane, the benches may be laid in such a way that they don't strike parallel to the strike of fault.
9. The water , especially the water pressure inside the slope significantly reduces the available shear strength and plays a critical role in determing the stability of slopes. Effective drainage measures are thus necessary to avoid or minimize water-induced instabilities.
10. No work in the lower benches should be done at the time of extraction of the disturbed area.

REFERENCES

1. Bieniawski ZT, 1973. Engineering classification of jointed rock masses. Trans. S Afr. Inst. Civil Eng. Vol. 15, pp. 355 – 344

2. Bieniawski ZT, 1979. The geomechanics classification in rock engineering applications. Proc 4th Int Cong Rock Mech, Montreux, AA Balkema, Rotterdam, Chapter 5, pp. 55 – 95

3. Bieniawiski ZT, 1989. Engineering Rock Mass Classifications. Wiley, New York

4. Choquet P & Tanon DDB, 1985. Nomograms for the assessment of toppling failure in rock slopes. Proc 26th US Symp Rock Mech., Rapid City, SD (Ed. E Ashworth) Balkema, Rotterdam, pp. 19 – 30

5. DIPS, 1999. DIPS – Software for plotting, analysis and presentation of structural data using hemispherical projection techniques. RocScience Inc., Toronto (Canada)

6. GALENA, 1990. Geomechanics Algorithm Limit Equlibrium Analysis, Version 2.0. BHP Engineering, Australia.

7. Hains A & Terbrugge PJ, 1991. Preliminary estimation of rock slope stability using rock mass classification systems. Proc 7th Int. Cong Rock Mech., Vol. 2, pp. 887 – 892

8. Hall BE, 1985. Preliminary estimation of slope angles. Proc Symp Rock Mass Characteristics, S Afr. National Group on Rock Mechanics, Johannesburg, pp. 120 -121

9. Hoek E & Bray J, 1981. Rock Slope Engineering. The Inst. of Mining & Metallurgy, London, pp. 358

10. Hoek E, 1986. General two-dimensional slope stability analysis. Analytical & Computational Methods in Engg. Rock Mech. (Ed. ET Brown) Allen & Unwin Ltd., London

11. Jayanthu S, et al, 2002. Application of rock mass classification systems for understanding stability of jointed rock masses in opencast mines. Mining Engineers J (communicated)

12. Laubscher DH, 1977. Geomechanics classification of jointed rock masses - Mining applications. Trans. Instn. Min. Metall. (Section A) 86, pp. A1 - A8

13. Laubscher DH, 1990. A geomechanics classification system for the rating of rock mass in mine design. J South Afr Instt. Mining, Vol. 90, No. 10, pp. 257 – 273

14. Moon BP & Selby MJ, 1990. Rock mass strength and scarp forms in Southern Africa. Geografisika Annaler. Ser. A, 65:135 -145

15. Orr CM, 1992. Assessment of rock slope stability using the Rock Mass Rating (RMR) system. Proc Aus. IMM, 297 : No. 2, pp. 25 – 29

16. Piteau DR & Martin DC, 1981. Mechanics of rock slope failure. Proc 3rd Int. Conf Stability in Surface Mining (Ed. CO Brawner), pp. 133 -169

17. Romana M, 1985. New adjustment rating for application of the Bieniawski classification to slopes. Proc Int. Symp Role of Rock Mechanics ISRM, Zacatecas, pp. 49 – 53

18. Romana M, 1991. SMR classification. Proc 7th Int Cong Rock Mechanics, Vol. 2, pp. 955 – 960

19. Romana M, 1993. Geomechanical classification for slopes: Slope Mass Rating. Comprehensive Rock Engg., Vol. 3, (Ed. John A Hudson) Pergamon Press, pp. 575 – 600

20. Romana M, 1995. The SMR geomechanical classification for slopes: A critical ten years review. Proc 8th Int Conf Landslides, Granada (Spain). (Eds. Chacon, Irigaray & Fernandez), AA Balkema, Rotterdam, pp. 255 – 267

21. Selby MJ, 1980. A rock mass strength classification for geomorphic purposes: with tests from Antarctica and New Zealand. J Geomorph. NF, Vol. 24, No. 1, pp. 31 – 51

22. Selby MJ, 1982. Controls on the stability and inclination of hill slopes formed on hard rock. Earth Surface Processes & Landforms, Vol. 7, pp. 449 – 467

23. Selby MJ, 1987. Rock slopes in slope stability. (Eds. MG Anderson & KS Richards) John Wiley, New York, pp. 475 – 504

24. Xiao Yuan & Wang Sijing, 1990. A study on the flexural breaking of rock mass. Proc Symp Mechanics of Jointed and Faulted Rock (Ed. Rossmanith), Balkema, Rotterdam, pp. 265 – 269

25. Zanbak C, 1983. Design charts for rock slopes susceptible to toppling. J Geotech Engg, Vol. 1009, No. 8

26. Zuyu Chen, 1995. Recent developments in slope stability analysis. Proc 8th Symp Int Soc Rock Mech, Tokyo (Ed. T Fuji), Vol. 3, pp. 1041 – 1048

27. Http://www.seiofbluemountain.com/upload/product/201002/12657047366i78z6t.pdf