DESIGN OF STABLE SLOPE FOR OPENCAST MINES

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

Bachelor of Technology
in
Mining Engineering

By

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CERTIFICATE

This is to certify that the thesis entitled "Design of Stable Slope for Opencast Mines" submitted by Sri Bisleshana Brahma Prakash, Roll No. 10505020 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 our supervision and guidance.

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

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My heart pulsates with the thrill for tendering gratitude to those persons who helped me in completion of the project.

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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.

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ABSTRACT

Slope stability analysis forms an integral part of the opencast mining operations during the life cycle of the project. In Indian mining conditions, slope design guidelines were not yet formulated for different types of mining practices and there is a growing need to develop such guidelines for maintaining safety and productivity. Till date, most of the design methods are purely based on field experience, rules of thumb followed by sound engineering judgment. During the last four decades, the concepts of slope stability analysis have emerged within the domain of rock engineering to address the problems of design and stability of excavated slopes. The basic objective of the project is primarily addressed towards: a) Understanding the different types and modes of slope failures b) Designs of stable slopes for opencast mines using numerical models. Analyses were conducted using the finite difference code *FLAC/Slope*. The work was aimed at investigating failure mechanisms in more detail, at the same time developing the modeling technique for pit slopes. The results showed that it was possible to simulate several failure mechanisms, in particular circular shear and toppling failure, using numerical modeling. The modeling results enabled description of the different phases of slope failures (initiation and propagation). Failures initiated in some form at the toe of the slope, but the process leading up to total collapse was complex, involving successive redistribution of stress and accumulation of strain. Significant displacements resulted before the failure had been developed fully. Based on parametric studies it can be concluded that friction angle plays a major role on slope stability in comparison to Cohesion.

Keywords: Slope stability, open pit mining, numerical modeling, rock mass strength, failure mechanisms.

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INTRODUCTION

1.1 Overview

Slope stability analysis forms an integral part of the opencast mining operations during the life cycle of the project. In Indian mining conditions, slope design guidelines are yet to be formulated for different types of mining practices and there is a growing need to develop such guidelines for maintaining safety and productivity. Till date, most of the design methods are purely based on field experience, rules of thumb followed by sound engineering judgment. During the last four decades, the concepts of slope stability analysis have emerged within the domain of rock engineering to address the problems of design and stability of excavated slopes.

In India, the number of operating opencast mines is steadily increasing as compared to underground mines. It is due to low gestation period, higher productivity, and quick rate of investment. On the contrary, opencast mining attracts environmental concerns such as solid-waste management, land degradation and socio-economical problems. In addition to that a large number of opencast mines, whether large or small, are now days reaching to deeper mining depths. As a result analysis of stability of operating slopes and ultimate pit slope design are becoming a major concern. Slope failures cause loss of production, extra stripping cost for recovery and handling of failed material, dewatering the pits and sometimes lead to mine abandonment/premature closure.

Maintaining pit slope angles that are as steep as possible is of vital importance to the reduction of stripping (mining of waste rock), which will in turn have direct consequences on the economy of the mining operation. Design of the final pit limit is thus governed not only by the ore grade distribution and the production costs, but also by the overall rock mass strength and stability. The potential for failure must be assessed for given mining layouts and incorporated into the design of the ultimate pit.

Against this backdrop, there is a strong need for good practices in slope design and management so that suitable corrective actions can be taken in a timely manner to minimize the slope failures.

1.2 Objectives

The prime objectives of the project are addressed towards:

a) Understanding the different types and modes of slope failures; and
b) Designing of stable slopes for opencast mines using numerical models.

1.3 Research Strategies

Extensive literature review has been carried out for understanding the different types and modes of slope failures. Numerical model FLAC/Slope was critically reviewed for its application to evaluation of the stability of slopes in opencast mines. Field investigation was conducted in Jindal Opencast Mine with 116 m ultimate pit depth at Raigarh in Chhattisgarh State. Laboratory tests were conducted for the rock samples taken during field investigation. Parametric studies were conducted through numerical models (FLAC/Slope) to study the effect of cohesion (140-220 kPa) and friction angle (20°-30° at the interval of 2°). Pit slope angle was varied from 35° to 55° at an interval of 5°.

1.4 Outline of Report

Following the introductory chapter, a general description of the economics of open pit mining, slope stability, failure modes and failure mechanisms, the assessment of slope stability and different methods of analysis are discussed in Chapter 2.

In Chapter 3, numerical modelling (FLAC) has been described, starting with FLAC's overview followed by summary of its features and finally analysis procedure. Application of numerical modelling is given through a case study of "Jindal Power OCP, Mand Raigarh Coalfield" in Chapter 4. Chapter 5 deals with conclusion and scope for future work.

CHAPTER: 02

LITERATURE REVIEW

CHAPTER: 02

LITERATURE REVIEW

2. Open Pit Slopes —An Introduction

In open pit mining, mineral deposits are mined from the ground surface and downward. Consequently, pit slopes are formed as the ore is being extracted. It is seldom, not to say never, possible to maintain stable vertical slopes or pit walls of substantial height even in very hard and strong rock. The pit slopes must thus be inclined at some angle to prevent failure of the rock mass. This angle is governed by the geomechanical conditions at the specific mine and represent an upper bound to the overall slope angle. The actual slope angles used in the mine depend upon (i) the presence of haulage roads, or ramps, necessary for the transportation of the blasted ore from the pit (ii) possible blast damage (iii) ore grades, and (iv)economical constraints.

2.1 Slope Stability

Slope stability problem is greatest problem faced by the open pit mining industry. The scale of slope stability problem is divided in to two types:

- **Gross stability problem:** It refer to large volumes of materials which come down the slopes due to large rotational type of shear failure and it involves deeply weathered rock and soil.

- **Local stability problem:** This problem which refers to much smaller volume of material and these type of failure effect one or two benches at a time due to shear plane jointing, slope erosion due to surface drainage.

To study the different types and scales of failure it is essential to know the different types of the failure, the factors affecting them in details and the slope stability techniques that can be used for analysis. The different types of the slope failure, factors affecting them, stability analysis techniques and software available have been described in the following sections:.

Factors Affecting Slope Stability

Slope failures of different types are affected by the following factors:

2.1.1 Slope Geometry

The basic geometrical slope design parameters are height, overall slope angle and area of failure surface. With increase in height the slope stability decreases. The overall angle increases the possible extent of the development of the any failure to the rear of the crests increases and it should be considered so that the ground deformation at the mine peripheral area can be avoided. Generally overall slope angle of 45° is considered to be safe by Directorate General of Mines Safety (DGMS). The curvature of the slope has profound effect on the instability and therefore convex section slopes should be avoided in the slope design. Steeper and higher the height of slope less is the stability. Diagram showing bench, ramp, overall slope and their respective angles is given in Fig. 2.1.

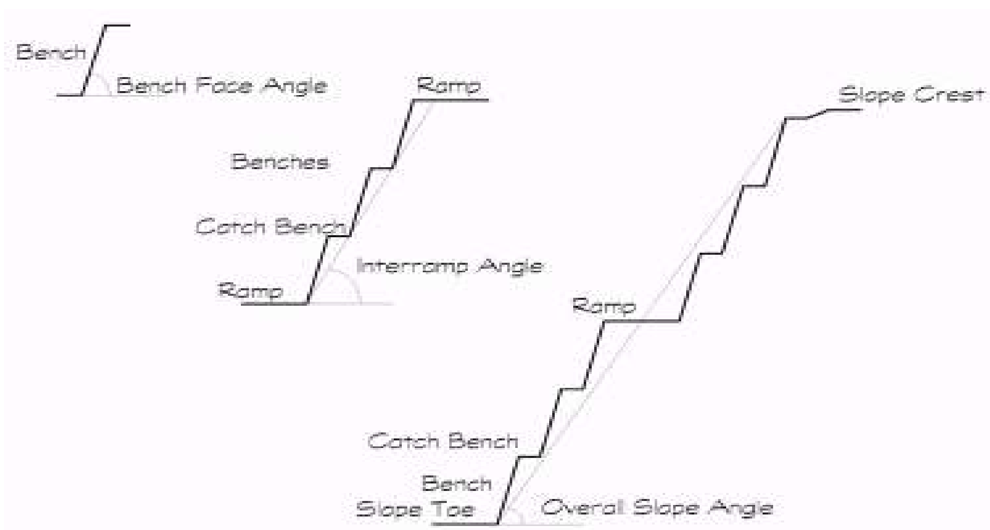


Fig. 2.1 Diagram showing bench, ramp, overall slope and their respective angles (after Coates, 1977, 1981)

2.1.2 Geological Structure

The main geological structure which affect the stability of the slopes in the open pit mines are:

1. amount and direction of dip

2. intra-formational shear zones
3. joints and discontinuities
    a) reduce shear strength
    b) change permeability
    c) act as sub surface drain and plains of failure
4. faults
    a) weathering and alternation along the faults
    b) act as ground water conduits
    c) provides a probable plane of failure

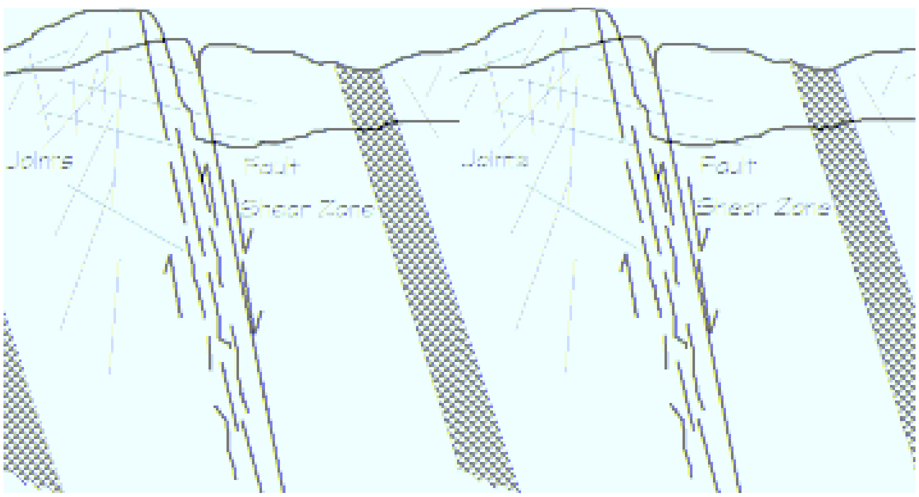


Fig. 2.2 Different types of joints and faults (partly after Nordlund and Radberg, 1995)

Instability may occur if the strata dip into the excavations. Faulting provides a lateral or rear release plane of low strength and such strata plan are highly disturbed. Localized steepening of strata is critical for the stability of the slopes. If a clay band comes in between the two rock bands, stability is hampered. Joints and bedding planes also provide surfaces of weakness. Stability of the slope is dependent on the shear strength available along such surface, on their orientations in relation to the slope and water pressure action on the surface. These shear strength that can be mobilized along joint surface depending on the functional properties of the surface and the effective stress which are transmitted normal to the surface. Joints can create a situation where a combination of joint sets provides a cross over surface.

2.1.3 Lithology

The rock materials forming a pit slope determines the rock mass strength modified by discontinuities, faulting, folding, old workings and weathering. Low rock mass strength is characterized by circular; raveling and rock fall instability like the formation of slope in massive sandstone restrict stability. Pit slopes having alluvium or weathered rocks at the surface have low shearing strength and the strength gets further reduced if water seepage takes place through them. These types of slopes must be flatter.

2.1.4 Ground Water

It causes the following:

a) alters the cohesion and frictional parameters and
b) reduce the normal effective stress

Ground water causes increased up thrust and driving water forces and has adverse effect on the stability of the slopes. Physical and chemical effect of pure water pressure in joints filling material can thus alter the cohesion and friction of the discontinuity surface. Physical effects of providing uplift on the joint surface, reduces the frictional resistances. This will reduce the shearing resistance along the potential failure plane by reducing the effective normal stress acting on it. Physical and the chemical effect of the water pressure in the pores of the rock cause a decrease in the compressive strength particularly where confining stress has been reduced.

2.1.5 Mining Method and Equipment

Generally there are four methods of advance in open cast mines. They are:

(a) strike cut- advancing down the dip
(b) strike cut- advancing up the dip
(c) dip cut- along the strike
(d) open pit working

The use of dip cuts with advance on the strike reduces the length and time that a face is exposed during excavation. Dip cuts with advance oblique to strike may often used to reduce the strata dip in to the excavation. Dip cut generally offer the most stable method of working but suffer from restricted production potential. Open pit method are used in steeply dipping seams, due to the increased slope height are more prone to large slab/buckling modes of failure. Mining equipment which piles on the benches of the open pit mine gives rise to the increase in surcharge which in turn increases the force which tends to pull the slope face downward and thus instability occurs. Cases of circular failure in spoil dumps are more pronounced.

2.1.6 Dynamic Forces

Due to effect of blasting and vibration, shear stresses are momentarily increased and as result dynamic acceleration of material and thus increases the stability problem in the slope face. It causes the ground motion and fracturing of rocks.

Blasting is a primary factor governing the maximum achievable bench face angles. The effects of careless or poorly designed blasting can be very significant for bench stability, as noted by Sage (1976) and Bauer and Calder (1971). Besides blast damage and back break which both reduce the bench face angle, vibrations from blasting could potentially cause failure of the rock mass. For small scale slopes, various types of smooth blasting have been proposed to reduce these effects and the experiences are quite good (e.g. Hoek and Bray, 1981). For large scale slopes, however, blasting becomes less of problem since back break and blast damage of benches have negligible effects on the stable overall slope angle. Furthermore, the high frequency of the blast acceleration waves prohibit them from displacing large rock masses uniformly, as pointed out by Bauer and Calder (1971). Blasting-induced failures are thus a marginal problem for large scale slopes. Seismic events, i.e., low frequency vibrations, could be more dangerous for large scale slopes and several seismic-induced failures of natural slopes have been observed in mountainous areas.

Together with all these causes external loading can also plays an important role when they are present as in case of surcharge due to dumps on the crest of the benches. In high altitude areas, freezing of water on slope faces can results in the build up of ground water pressure behind the face which again adds up to instability of the slope.

2.1.7 Cohesion

It is the characteristic property of a rock or soil that measures how well it resists being deformed or broken by forces such as gravity. In soils/rocks true cohesion is caused by electrostatic forces in stiff overconsolidated clays, cementing by Fe_2O_3, $CaCO_3$, NaCl, etc and root cohesion. However the apparent cohesion is caused by negative capillary pressure and pore pressure response during undrained loading. Slopes having rocks/soils with less cohesion tend to be less stable. The factors that strengthen cohesive force are as follows:

a) Friction
b) Stickiness of particles can hold the soil grains together. However, being too wet or too dry can reduce cohesive strength.
c) Cementation of grains by calcite or silica deposition can solidify earth materials into strong rocks.
d) Man-made reinforcements can prevent some movement of material.

The factors that weaken cohesive strength are as follows:

a) High water content can weaken cohesion because abundant water both lubricates (overcoming friction) and adds weight to a mass.
b) Alternating expansion by wetting and contraction by drying of water reduces strength of cohesion, just like alternating expansion by freezing and contraction by thawing. This repeated expansion is perpendicular to the surface and contraction vertically by gravity overcomes cohesion resulting with the rock and sediment moving slowly downhill.
c) Undercutting in slopes
d) Vibrations from earthquakes, sonic booms, blasting that create vibrations which overcome cohesion and cause mass movement.

2.1.8 Angle of Internal Friction

Angle of internal friction is the angle (ϕ), measured between the normal force (N) and resultant force (R), that is attained when failure just occurs in response to a shearing stress (S). Its tangent (S/N) is the coefficient of sliding friction. It is a measure of the ability of a unit of rock or soil to withstand a shear stress. This is affected by particle roundness and particle size. Lower roundness or larger median particle size results in larger friction angle. It is also affected by quartz content. The sands with less quartz contained greater amounts of potassium-feldspar, plagioclase, calcite, and/or dolomite and these minerals generally have higher sliding frictional resistance compared to that of quartz.

2.2 Types of Slope Failure

2.2.1 Plane Failure

Simple plane failure is the easiest form of rock slope failure to analyze. It occurs when a discontinuity striking approximately parallel to the slope face and dipping at a lower angle intersects the slope face, enabling the material above the discontinuity to slide. Variations on this simple failure mode can occur when the sliding plane is a combination of joint sets which form a straight path.

This means that the solution is never any thing more than the analysis of equilibrium of a single block resting on a plane and acted upon by a number of external forces (water pressure, earth quake, etc.) deterministic and probabilistic solution in which parameters are considered as being precisely known may be readily obtained by hand calculation if effect of moment is neglected.

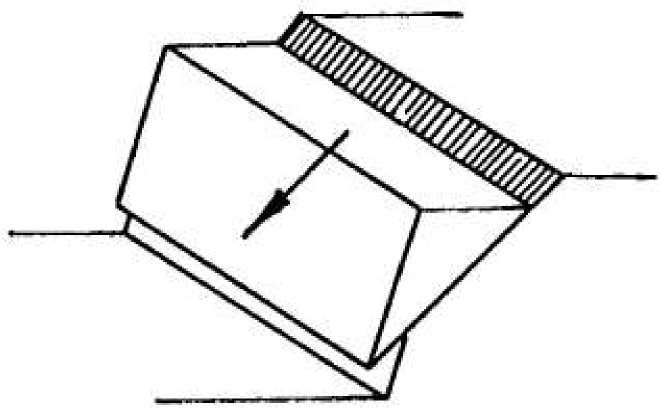


Fig. 2.3 Plane failure (after Coates, 1977; Call and Savely, 1990).

For a plane failure analysis, the geometry of the slope is very critically studied. In this connection two cases must be considered:-

(a) A slope having tension crack in the upper face.
(b) A slope with tension crack in the slope face.

When the upper surface is horizontal ($\psi_s = 0$), the transition from one condition to another occurs when the tension crack coincides with the slope crest, that is when

$$\frac{z}{H} = (1 - \cot\psi_f \tan\psi_p) \qquad (1)$$

Where 'z' is the depth of the tension crack, 'H' is the slope height, 'ψ_f' is the slope angle and 'ψ_p' is the dip of the sliding plane.

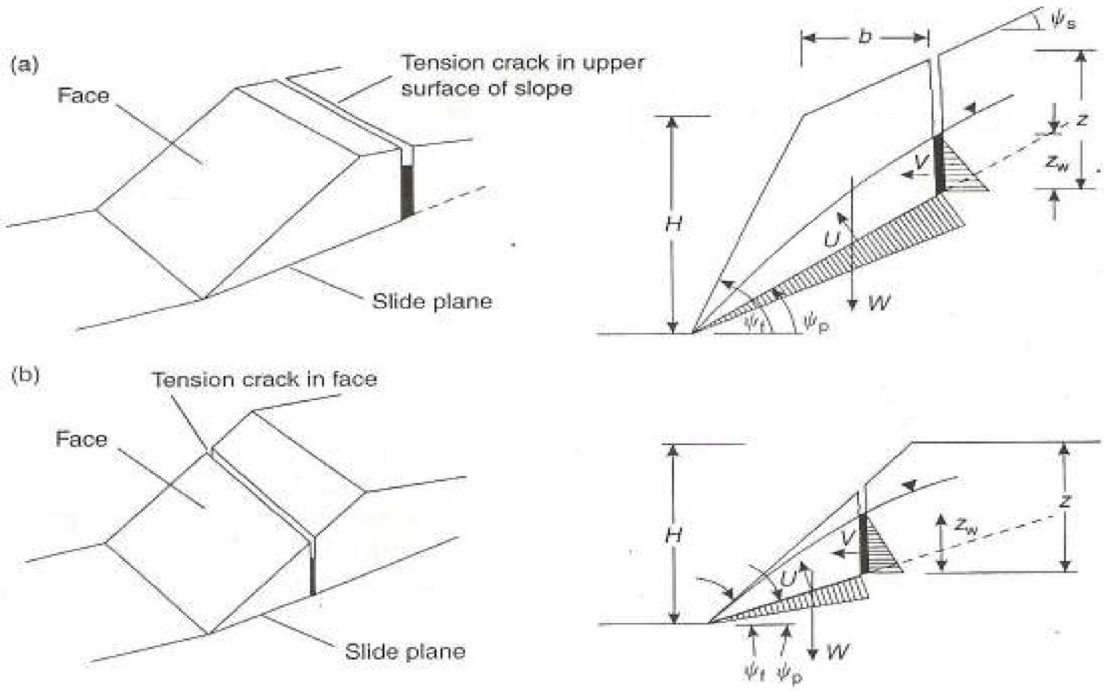


Fig. 2.4 Geometries of plane slope failure: (a) tension crack in the upper slope; (b) tension crack in the face

For the analysis, the following assumptions are to be made:-

a) Both the sliding surface and tension crack must strike parallel to the face.
b) The tension crack is vertical and is filled with water to a depth 'z_w'.
c) Water enters the sliding surface along the base of the tension cracks and seeps along the sliding surface, escaping at atmospheric pressure where the sliding surface daylight in the slope faces.
d) The forces 'W' (weight of sliding block), 'U' (uplift force due to water pressure on the sliding surface) and 'V' (force due to water pressure in the tension crack) all acts through the centroid of the sliding mass.
e) The shear strength of the sliding surface is defined by cohesion 'c' and the friction angle 'ϕ' that are related by the equation

$$\tau = c + \sigma \tan\phi \qquad (2)$$

f) A slice of unit thickness is considered and it is assumed that the release surfaces are present so that there is no resistance to the sliding at the lateral boundaries of the failure.

Calculation of factor of safety

The factor of safety for the general case of the plane failure is the ratio of the forces acting to keep the failure mass in place (the cohesion times the area of the failure surface plus the frictional shear strength determined using the effective normal stress on the failure plane) to the forces attempting to drive the failure mass down the failure surface (the sum of the component of the weight, water forces, and all other external forces acting along the failure surface). This is determined by resolving all forces acting on the on the potential failure mass in to directions parallel and normal to the potential failure surface. The general factor of safety which results is:

$$\mathbf{FS}=\frac{\textbf{Resisting force}}{\textbf{Driving force}} \tag{3}$$

$$\mathbf{FS}=\frac{\mathbf{cA}+\sum \mathbf{N}\tan\phi}{\sum \mathbf{S}} \tag{4}$$

where ‘c’ is the cohesion and ‘A’ is the area of the sliding plane.
The factor of safety for the slope configurations in Fig. 2.4 is given by

$$\mathbf{FS}=\frac{\mathbf{cA}+(\mathbf{W}\cos\psi_p-\mathbf{U}-\mathbf{V}\sin\psi_p)\tan\varphi}{\mathbf{W}\sin\psi_p+\mathbf{V}\cos\psi_p} \tag{5}$$

Where ‘A’ is given by

$$\mathbf{A}=(\mathbf{H}+\mathbf{b}\tan\psi_s-\mathbf{z})\operatorname{cosec}\psi_p \tag{6}$$

The slope height ‘H’, the tension crack depth is ‘z’ and is located a distance ‘b’ behind the slope crest. The dip above the crest is ‘ψ_s’. When the depth of water in the tension crack is ‘z_w’, the water forces acting on the sliding plane ‘U’ and in the tension crack ‘V’ are given by

$$\mathbf{U}=\frac{1}{2}\gamma_w z_w(\mathbf{H}+\mathbf{b}\tan\psi_s-\mathbf{z})\operatorname{cosec}\psi_p \tag{7}$$

$$\mathbf{V}=\frac{1}{2}\gamma_w z_w^{2} \tag{8}$$

Where ‘γ_w’ is the unit weight of water.

The weights of the sliding block for the two geometries shown in Fig. 2.4 are given by the equations (9) and (10). For the tension crack in the inclined upper slope surface

$$W=\gamma_r[(1-\cot\psi_f\tan\psi_p)(bH+\frac{1}{2}H^2\cot\psi_f)+\frac{1}{2}b^2(\tan\psi_s-\tan\psi_p)] \tag{9}$$

And, for the tension crack in the slope face

$$W=\frac{1}{2}\gamma_r H^2[(1-\frac{z}{H})^2\cot\psi_p\times(\cot\psi_p\tan\psi_f-1)] \tag{10}$$

Where 'γ_r' is the unit weight of the rock.

2.2.2 Wedge Failure

The three dimensional wedge failures occur when two discontinuities intersects in such a way that the wedge of material, formed above the discontinuities, can slide out in a direction parallel to the line of intersection of the two discontinuities. It is particularly common in the individual bench scale but can also provide the failure mechanism for a large slope where structures are very continuous and extensive.

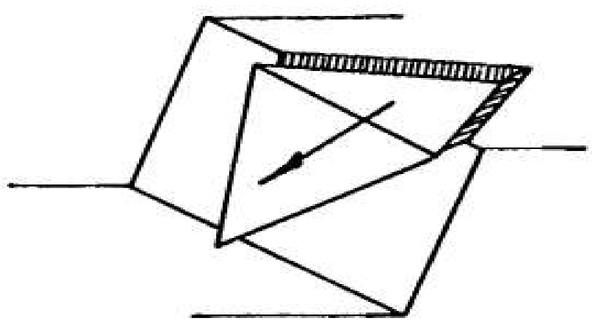


Fig. 2.5 Wedge failure (after Hoek and Bray, 1981)

When two discontinuities strike obliquely across the slope face and their line of intersection 'daylights' in the slope, the wedge of the rock resting over these discontinuities will slide down along the line of intersection provided the inclination of these line is significantly greater than the angle of friction and the shearing component of the plane of the discontinuities is less than the total downward force. The total downward force is the downward component of the weight of the wedge and the external forces (surcharges) acting over the wedge.

The wedge failure analysis is based on satisfying the equilibrium conditions of the wedge. If ‘w’ be the weight of the wedge, the vector ‘w’ can be divided into two components in the parallel and normal directions to the joint intersection, Fig. 2.6.

$$\mathbf{N = w\cos\theta,\ P = w\sin\theta} \qquad \mathbf{(11)}$$

The vector ‘N’ in the Fig. 2.7 is divided into two components ‘N_1’ and ‘N_2’, normal to the joint set surfaces 1 and 2, respectively as follows:

In Fig. 2.6 the equilibrium conditions in the directions *x* and *y* are as follows:

$$\mathbf{N_{1x} = N_2x,\ N_{1y} + N_{2y} = N} \qquad \mathbf{(12)}$$

$$\mathbf{N_{1x} = N_1 \sin\alpha_1,\ N_{2x} = N2 \sin\alpha_2} \qquad \mathbf{(13)}$$

$$\mathbf{N_{1y} = N_1 \cos\alpha_1,\ N_{2y} = N2 \cos\alpha_2} \qquad \mathbf{(14)}$$

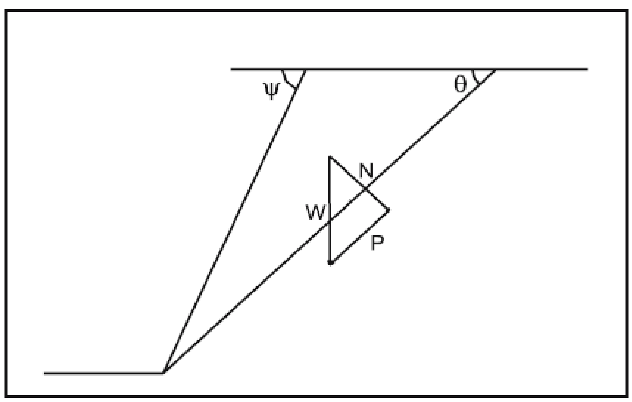


Fig. 2.6 Conditions of effective forces in the wedge failure analysis

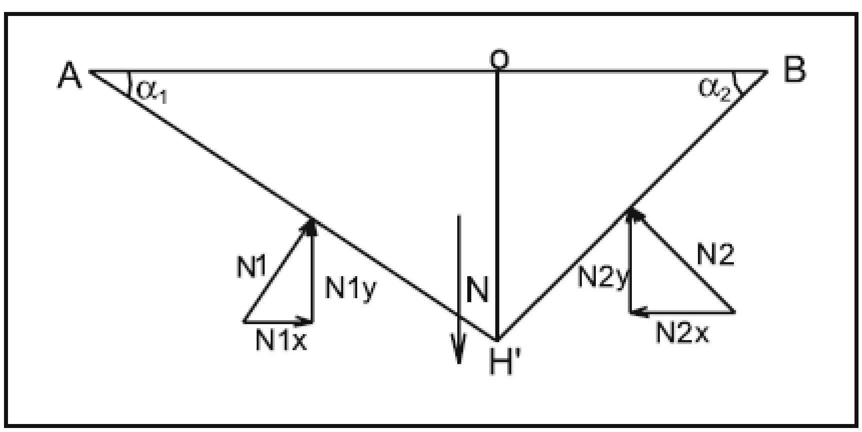


Fig. 2.7 Diagram of the plane normal to the intersection of joint sets 1 and 2

The forces 'N$_1$' and 'N$_2$' can be obtained from the Equations. (12), (13), and (14) as follows:

$$\begin{cases} \mathbf{N_1 sin\alpha_1 = N_2 sin\alpha_2} \\ \mathbf{N_1 cos\alpha_1 + N_2 cos\alpha_2 = N = Wcos\theta} \end{cases} \quad \mathbf{(15)}$$

Where

$$\mathbf{N_1 = \frac{Nsin\alpha_2}{sin(\alpha_1+\alpha_2)},\ N_2 = \frac{Nsin\alpha_1}{sin(\alpha_1+\alpha_2)}} \quad \mathbf{(16)}$$

Calculation of the angles α_1 and α_2

In Fig. 2.8 the line CC' is the intersection line of two joint surfaces 1 and 2. The segment OH is drawn vertically in the normal plane passing through the line of intersection CC'. Fig. 2.7 is drawn in the three-dimensional view as the triangle ABH'. From the point O the segment OH' normal to the intersection is drawn. The plane ABH' is the plane normal to the intersection CC' at point H'. From the points H and A on plane 1, two lines are drawn so that the first one is parallel to the strike and the second one is in the direction of dip line.

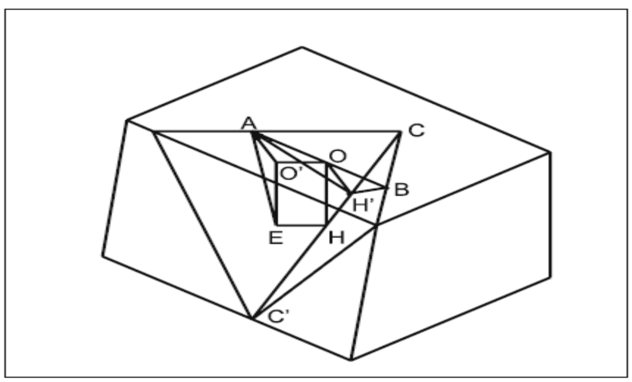


Fig. 2.8 The geometry of the sliding wedge

These two lines intersect at point E. EO' is drawn parallel and with the same size as HO. The quadrilateral OO'EH is rectangle. Using the geometric and trigonometric relationships in the triangles H'OA, OO'A, and O'AE, the angles α_1 and α_2 are obtained from the following equation.

$$\mathbf{cos\theta\ cos\ \gamma_1 tan\ d_1 = \frac{H'O}{HO} * \frac{AO'}{AO} * \frac{EO'}{AO'} = \frac{HO'}{AO} = tan\alpha_1} \quad \mathbf{(17)}$$

It can be shown in the same way that $\mathbf{tan\ \alpha_2 = cos\theta\ cos\ \gamma_2\ tan\ d_2}$, where HO = EO`, ∠EAO` = $\angle \boldsymbol{d_1}$, ∠OAH` = $\angle\boldsymbol{\alpha_1}$, ∠OBH` = $\angle\boldsymbol{\alpha_2}$, ∠HOH` = $\angle\boldsymbol{\theta}$, and ∠OAO` = $\angle\boldsymbol{\gamma_1}$ in which 'd_1' and 'd$_2$' are the slope angles of the joint set 1 and 2, respectively. The angles 'γ_1' and 'γ_2' are the angle between the dip directions of joint sets 1 and 2 and the strike of the plane normal to intersection line, respectively.

The factor of safety can be calculated from the equation (18) given below:

$$\mathbf{FS = \frac{T_1 + T_2}{w \sin\theta}} \qquad \mathbf{(18)}$$

where

$$\mathbf{T_1 = N_1 \tan(\phi_{j1} + i_1)(1-a_1) + C_{j1}(1-a_1)S_1 + N_1 a_1 \tan\phi_{r1} + C_{r1} a_1 S_1} \qquad \mathbf{(19)}$$

$$\mathbf{T_2 = N_2 \tan(\phi_{j2} + i_1)(1-a_2) + C_{j2}(1-a_2)S_1 + N_2 a_2 \tan\phi_{r2} + C_{r2} a_2 S_2} \qquad \mathbf{(20)}$$

The internal frictions of the intact rock '$Ø_{r1}$' and '$Ø_{r2}$' and the cohesion coefficients of the intact rock 'C_{r1}' and 'C_{r2}' are determined from the triaxial compressive tests and using the Mohr–Colomb criterion. The correction factor for the effect of intact rock specimen diameter on the cohesion coefficients could also be included. The internal friction angles of the joint sets 1 and 2 surfaces 'φ_{j1}' and 'φ_{j2}' are obtained from the shear tests on the polished rock joint specimens. The irregularity angles 'i_1' and 'i_2' are determined from the direct measurements on the rock outcrops using the stereographic projections of the joint sets 1 and 2.

2.2.3 Circular Failure

The pioneering work, in the beginning of the century, in Sweden confirmed that the surface of the failure in spoil dumps or soil slopes resembles the shape of a circular arc. This failure can occurs in soil slopes, the circular method occurs when the joint sets are not very well defined. When the material of the spoil dump slopes are weak such as soil, heavily jointed or broken rock mass, the failure is defined by a single discontinuity surface but will tend to follow a circular path.

The conditions under which circular failure occurs are follows:

1. When the individual particles of soil or rock mass, comprising the slopes are small as compared to the slope.
2. When the particles are not locked as a result of their shape and tend to behave as soil.

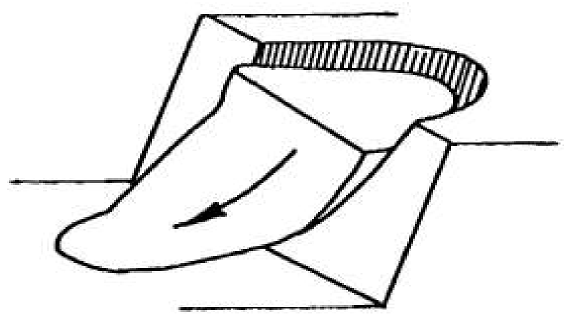


Fig. 2.9 Three-dimensional failure geometry of a rotational shear failure (after Hoek and Bray, 1981).

Types of circular failure

Circular failure is classified in three types depending on the area that is affected by the failure surface. They are:-

(a) **Slope failure:** In this type of failure, the arc of the rupture surface meets the slope above the toe of the slope. This happens when the slope angle is very high and the soil close to the toe posses the high strength.

(b) **Toe failure:** In this type of failure, the arc of the rupture surface meets the slope at the toe.

(c) **Base failure:** In this type of failure, the arc of the failure passes below the toe and in to base of the slope. This happens when the slope angle is low and the soil below the base is softer and more plastic than the soil above the base.

2.2.4 Two Block Failure

Two block failures are much less common mode of rock slope failure than single block failures such as the planes and the 3D wedge and, consequently, are only briefly considered here. Several methods of solution exist and each may be appropriate at some level of investigation.

2.2.5 Toppling Failure

Toppling or overturning has been recognized by several investigators as being a mechanism of rock slope failure and has been postulated as the cause of several failures ranging from small to large ones.

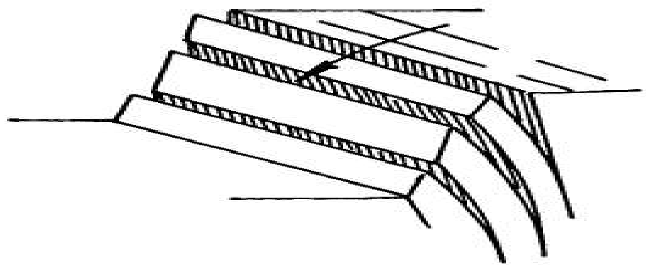


Fig. 2.10 Toppling failure

It occurs in slopes having near vertical joint sets very often the stability depends on the stability of one or two key blocks. Once they are disturbed the system may collapse or this failure has been postulated as the cause of several failures ranging from small to large size. This type of failure involves rotation of blocks of rocks about some fixed base. This type of failure generally occurred when the hill slopes are very steep.

2.3 Factors to be Considered in Assessment of Stability

2.3.1 Ground Investigation

Before any further examination of an existing slope, or the ground on 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 and shear planes. 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.

2.3.2 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. The aim is to find the most critical surface using "trial circles".

The method is as follows:

- A series of slip circles of different radii is to be considered but with same centre of rotation. Factor of Safety (FOS) for each of these circles is plotted against radius, and the minimum FOS is found.
- 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.
- An overall failure surface is found.

Fig. 2.11(a) & (b) shows variety of slope failure circles analysed at varying radii from a single centre and variation of factor of safety with critical circle radius respectively.

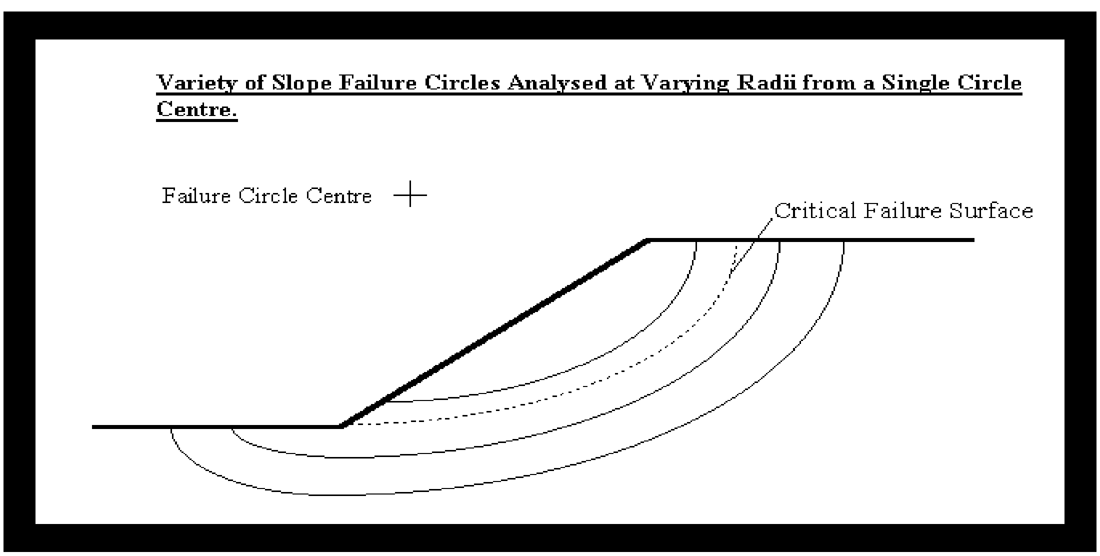


Fig. 2.11(a) Variety of slope failure circles analysed at varying radii from a single centre

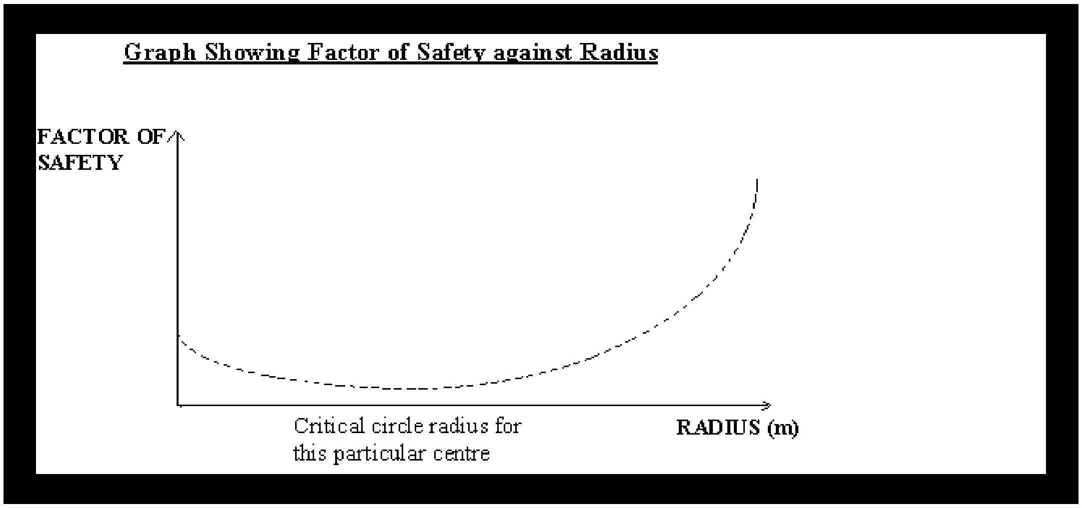


Fig. 2.12(b) Variation of factor of safety with critical circle radius

2.3.3 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 in Fig. 2.12.

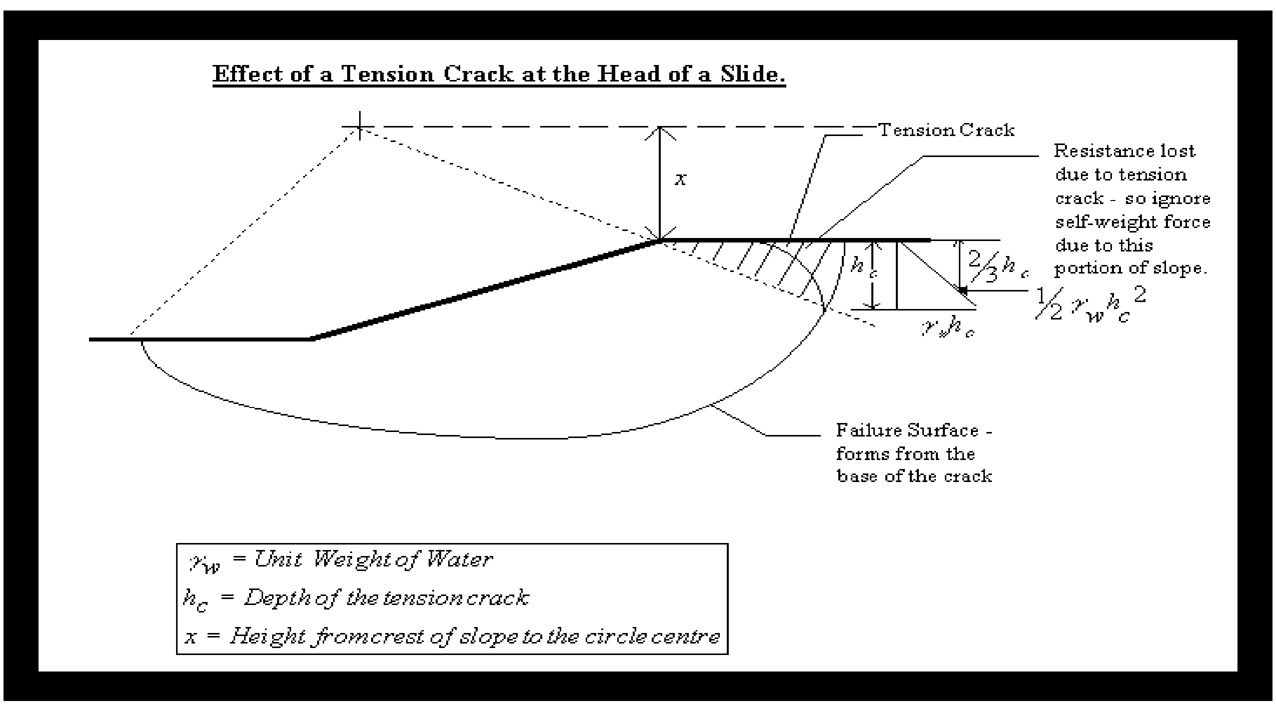


Fig. 2.12 Effect of tension crack at the head of a slide

Tension cracks are not usually important in stability analysis, but can become so in some special cases. Therefore assume that the cracks don't occur, but take account of them in analyzing a slope which has already cracked.

2.3.4 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 analysis of the slope. Thus, allowing for the external water forces by using submerged densities in the slope, and by ignoring water externally.

2.3.5 Factor of Safety (FOS)

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

Table 2.1 Guidelines for 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.

2.3.6 Progressive Failure

This is the term describing the condition when different parts of a failure surface reach failure at different times. This often occurs if a potential failure surface passes through a foundation material which is fissured or has joints or pre-existing failure surfaces. Where these fissures occur there will be large strain values, so the peak shear strength is reached before other places.

2.3.7 Pre-Existing Failure Surfaces

If the foundation on which a slope sits contains pre-existing failure surfaces, there is a large possibility that progressive failure will take place if another failure surface were to cut through them. The way to deal with this situation is to assume that sufficient movement has previously taken place for the ultimate state to develop in the soil and then using the ultimate state parameters. If failure has not taken place, then a decision has to be made on which parameters to be used.

2.4 Methods of Analysis

2.4.1 Wedge Failure Analysis

The 3D nature of the wedge failure analysis complicates the analysis. The different methods of analysis are given as follows:

2.4.1.1 Spherical Projection Solution using Factor of Safety

The 3D wedge problem can be very easily analyzed using spherical projection techniques. When the shear strength of the shear surface is entirely frictional and there is no external force, the problem becomes dimensionless and can be analyzed very simply by the means of a stereo net analysis alone. The introduction of water pressure or the external forces requires the use of side calculations to determine the orientation of the resultant forces acting on the wedge.

Use of spherical projection rapidly establishes a zone of orientations for the resultant force for which the wedge will remain stable. The orientation of the line of intersection of the wedge is defined by the intersection of the great circles which defines the joints.

To determine the factor of safety against sliding, the great circle containing both the resultant force acting on the wedge and the resultant shear force is drawn. The intersection of this great circle with and through both the normal and both the reactions on the shear planes define the position of the resultant of these normal and reactions. The factor of safety can be defined as the ratio of the resultant shear force acting along the line of intersection of the wedge to the resultant shear strength available to resist sliding in the same direction.

2.4.1.2 Chart Solution

Hoek and Bray (1980) produced a series of charts which can be used to rapidly access the stability of rock wedges for which there is know cohesion or external forces. Under these condition and for a given friction angle, the factor of safety is a function only of the dip and direction of the shear plane. These charts are convenient to use for use simple wedge problem but suffer from the disadvantage that it does not give the feel of the problem.

2.4.1.3 Spherical Projections Solutions using Probabilistic Approach

Monte Carlo analysis of the wedge failure gives, with a specified confidence level, the uncertainty in the orientations of the shear planes. When the orientations of the shear planes are known then the spherical projection technique can be used to find out the orientation of the failure plane.

2.4.2 Circular Failure Analysis

The stability of the slopes of finite extent like that in the case of circular is analyzed by the method of dividing the whole suspected failure area in to slices and further analyzing the sequence of events that may follow thereafter. There are several methods of slices in their new advancement together with friction circle method and tailors stability number method.

2.4.2.1 Method of Slices

This method was advanced by the Swedish geotechnical commission and developed by W.Fellienius (1936). By dividing the mass above an assumed rupture surface of failure in to vertical slices and assuming that the forces on the opposite sides of each slice are equal and opposite, a statistically determinate problem is obtained and semi graphical method have been devised by which the stability of the mass may be analyzed for any given circle. The main objection of this method is that the most dangerous of infinite number of circles are to be found out for which graphical method is to be used for a number of time.

2.4.2.2 Modified Method of Slices

When there are several dangerous circles to be analyzed usual procedure by the slice method is quite tedious. N.C.Coutrney of U.S.A. has developed simple graphical solutions by which the forces that are inherent in the method of slices such as the forces acting on the vertical sides of the slices.

2.4.2.3 Simplified Method of Slices

This method takes in to account the forces acting on the vertical sides of the slices in the development of an equation for determining the factor of safety. However, the simplified equation proposed by Bishop (1955) does not contain the forces acting on the vertical sides and there by simplifies the computation.

2.4.2.4 Friction Circle Method

It is a very convenient method which takes in to account the total forces acting on the whole mass lying above the assumed circular surface of failure. This method eliminates the indeterminate forces that are inherent in the method of slices such as acting on the vertical sides of the slices.

2.4.2.5 Taylor's Stability Number

Taylor (1937) made a mathematical trial method using the friction circle method. Charts as formulated by Taylor give the relationship between stability number and the slope angle for various angle of friction. This method is applicable to homogeneous simple slopes without seepage.

2.4.3 Two Block Failure Analysis

2.4.3.1 Stereographic Solution

A stereographic analysis is convenient way of determining weather or not a two block configuration will stable (Goodman, 1975 and Kuykendall and Goodman, 1976). Any form of shear strength envelope can be accounted for by use of the secant angle of friction.

2.4.4 Toppling Failure Analysis

Base friction models can be useful insight in to the mechanism of failure. They can also be used to provide a quantitative assessment of the effect of possible slope stabilization procedure such as reducing the slope angle or installing horizontal reinforcements. The difference conditions are taken in to account to ascertain sliding and toppling of block in inclined plane.

2.4.5 Other Methods of Analysis

2.4.5.1 Limit Equilibrium Method

In limit equilibrium method of analysis, static force is applied to analyze the stability of the rock mass or soil above the failure surface. If failure has already occurred, the geometry of the failure surface can be determined and the analysis of the failure can be done and is known as back analysis. If it is a design situation, however the failure surface is potential rather than actual, many potential surface may have to be analyzed to find the critical geometry before an acceptable slope geometry can be accounted for.

In the case of plane failure, 3D wedge failure, circular failure, the material above the failure surface will be on the point of slipping when the disturbing forces due to gravity are just counterbalanced by the forces tending to restore equilibrium. The ratio of the two forces defines the factor of safety of the failure surface.

2.4.5.2 Stress Analysis Method

Failure does not necessarily occur along a well defined failure surfaces. The situation where the structural condition does not permit sliding along the discontinuity surface, crushing of the rock occurs at the points of the highest stress. Progressive failure of the rock mass can subsequently deform the slope and may cause the catastrophic failure.

The objectives of the stress analysis method are to represent the rock mass by a series of structural elements (finite element method) or cells of constraint of materials (one finite different method) and perform an analysis to determine to stresses at points within the slope. The stress distribution can be examined to determine where rock failure is likely to occur, rock failure occurs when the stresses to which the rock is subjected more than its strength.

CHAPTER: 03

NUMERICAL MODELLING

CHAPTER: 03

NUMERICAL MODELLING

3.1 Introduction

Many rock slope stability problems involve complexities relating to geometry, material anisotropy, non-linear behaviour, *in situ* stresses and the presence of several coupled processes (e.g. pore pressures, seismic loading, etc.). Advances in computing power and the availability of relatively inexpensive commercial numerical modelling codes means that the simulation of potential rock slope failure mechanisms could, and in many cases should, form a standard component of a rock slope investigation.

Numerical methods of analysis used for rock slope stability may be conveniently divided into three approaches: continuum, discontinuum and hybrid modelling. Table 2 provides a summary of existing numerical techniques.

Table 3.1 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	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

		analysis.	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 behaviour and mechanisms (combined material and discontinuity behaviour coupled with hydro-mechanical and dynamic analysis). Able to assess effects of parameter variations on instability.	As above, experienced user required to observe good modelling 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 finite-element/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.

3.1.1 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.

3.1.2 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. Distinct-element 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.

3.1.3 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.

3.2 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 3.1, 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 behaviour by parameter studies →	Predictive (direct use in design)

Fig. 3.1 Spectrum of modeling situations

FLAC may be used either in a fully predictive mode (right-hand side of Fig. 3.1) 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 3.2 lists the steps recommended to perform a successful numerical experiment; each step is discussed separately.

Table 3.2 Recommended steps for numerical analysis in geomechanics

Step 1	Define the objectives for the model analysis
Step 2	Create a conceptual picture of the physical system
Step 3	Construct and run simple idealized models
Step 4	Assemble problem-specific data
Step 5	Prepare a series of detailed model runs
Step 6	Perform the model calculations
Step 7	Present results for interpretation

3.2.1 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.

3.2.2 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 well-defined 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.

3.2.3 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.

3.2.4 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.

3.2.5 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.

3.2.6 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.

3.2.7 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.

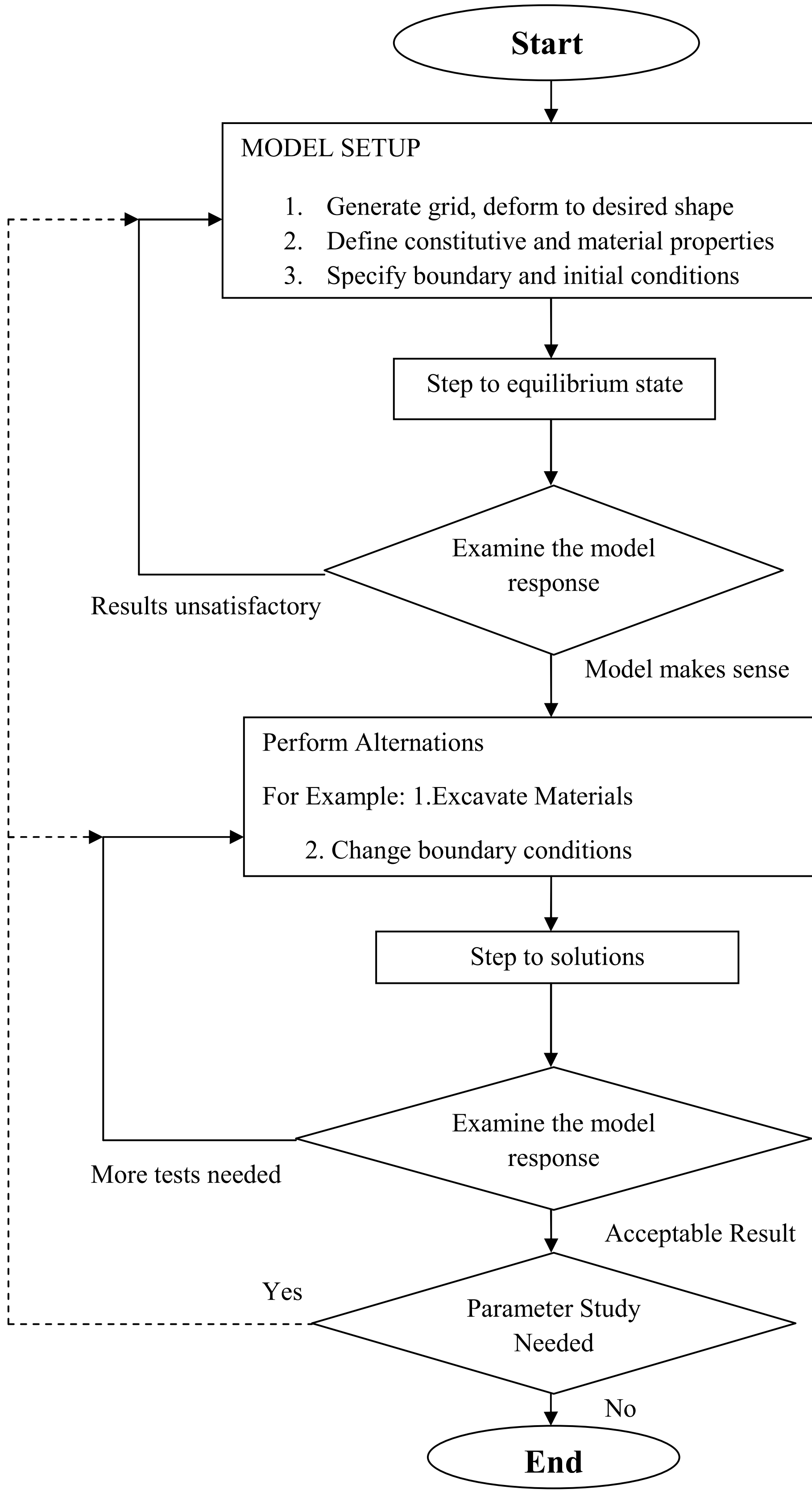


Fig. 3.2 Flow chart for determination of factor of safety using FLAC/Slope

3.3 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 of 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. In 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 or 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.)

3.4 Summary of Features

FLAC/Slope can be applied to a wide variety of conditions to evaluate the stability of slopes and embankments. Each condition is defined in a separate graphical tool.

1. The creation of the slope boundary geometry allows for rapid generation of linear, nonlinear and benched slopes and embankments. The Bound tool provides separate generation modes for both simple slope shapes and more complicated non-linear slope surfaces. A bitmap or DXF image can also be imported as a background image to assist boundary creation.
2. Multiple layers of materials can be defined in the model at arbitrary orientations and non-uniform thicknesses. Layers are defined simply by clicking and dragging the mouse to locate layer boundaries in the Layers tool.
3. Materials and properties can be specified manually or from a database in the Material tool. At present, all materials obey the Mohr-Coulomb yield model, and heterogeneous properties can be assigned. Material properties are entered via material dialog boxes that can be edited and cloned to create multiple materials rapidly.
4. With the Interface tool, a planar or non-planar interface, representing a joint, fault or weak plane, can be positioned at an arbitrary location and orientation in the model. The interface strength properties are entered in a properties dialog; the properties can be specified to vary during the factor-of-safety calculation, or remain constant.

FLAC/Slope is limited to slope configurations with no more than one interface. For analyses which involve multiple (and intersecting) interfaces or weak planes, full FLAC should be used.

5. An Apply tool is used to apply surface loading to the model in the form of either a real pressure (surface load) or a point load.
6. A water table can be located at an arbitrary location by using the Water tool; the water table defines the phreatic surface and pore pressure distribution for incorporation of effective stresses and the assignment of wet and dry densities in the factor-of-safety calculation.
7. Structural reinforcement, such as soil nails, rock bolts or geotextiles, can be installed at any location within the model using the Reinforce tool. Structural properties can be assigned individually for different elements, or groups of elements, through a properties dialog.
8. Selected regions of a FLAC/Slope model can be excluded from the factor-of-safety calculation.

3.5 Analysis Procedure

FLAC/Slope is specifically designed to perform multiple analyses and parametric studies for slope stability projects. The structure of the program allows different models in a project to be easily created, stored and accessed for direct comparison of model results. A FLAC/Slope analysis project is divided into four stages which is described below.

a) Models Stage

Each model in a project is named and listed in a tabbed bar in the Models stage. This allows easy access to any model and results in a project. New models can be added to the tabbed bar or deleted from it at any time in the project study. Models can also be restored (loaded) from previous projects and added to the current project. The slope boundary is also defined for each model at this stage.

b) Build Stage

For a specific model, the slope conditions are defined in the Build stage. This includes: changes to the slope geometry, addition of layers, specification of materials and weak plane, application of surface loading, positioning of a water table and installation of reinforcement. Also, spatial regions of the model can be excluded from the factor-of-safety calculation. The build-stage conditions can be added, deleted and modified at any time during this stage.

c) Solve Stage

In the Solve stage, the factor of safety is calculated. The resolution of the numerical mesh is selected first (coarse, medium and fine), and then the factor-of-safety calculation is performed. Different strength parameters can be selected for inclusion in the strength reduction approach to calculate the safety factor. By default, the material cohesion and friction angle are used.

d) Plot Stage

After the solution is complete, several output selections are available in the Plot stage for displaying the failure surface and recording the results. Model results are available for subsequent access and comparison to other models in the project. All models created within a project, along with their solutions, can be saved, the project files can be easily restored and results viewed at a later time.

CHAPTER: 04

CASE STUDY

CHAPTER: 04

CASE STUDY

JINDAL POWER OCP, MAND RAIGARH COALFIELD

4.1 Introduction

Jindal Power Opencast Coal Mine is captive mine of Jindal's 1000 MW (4 x 250 MW) thermal power plant. The block is located between Longitudes - 83°29'40" to 83°32'32" (E) and Latitude - 22°09'15" to 22°05'44" (N) falling in the topo sheet no. 64 N/12 (Survey of India). Administratively, the block is under Gharghoda Tahsil of Raigarh District, Chhattisgarh. The block is well connected by Road. It is about 60 km from Raigarh town, which is the district head quarter and nearest railway station on Mumbai - Howrah Main Line.

4.2 Geology

In general area of the coal block - Jindal Power Open Cast Coal Mine is almost flat with small undulations from surface the lithological section comprises about 3-4 m unconsolidated loose soil/alluvium. Below the top soil there is weathered shale/sandstone up to 6–8 m depth. The weathered shale/sandstone is competitively loose in nature and can be excavated without blasting. Below weathered mental (which varies from 3 – 10 m), the rock is hard, compact and massive in nature it can be excavated only after blasting.

In the sub-block IV/2 & IV/3 only lower groups of Gondwana seams have been deposited. The general strike of the seams in NW-SE is almost uniform throughout the block. Two normal faults of small magnitude have been deciphered based on the level difference of the floor of the seams, though the presence of some minor faults of less than 5 m throw cannot be overruled.

The Mand Raigarh basin is a part of IB River - Mand - Korba master basin lying within the Mahanadi graben. Sub block IV/2 & IV/3 of Gare-Pelma area is structurally undisturbed except one small fault (throw 0-15 m) trending NE-SW with westerly throws. The strike of the bed is NW-SE in general with dip varies from 2° to 6° southwesterly. In the sub block IV/2 & IV/3, total 10 coal seams have been established. They are seam X to I in descending order. The lithology of the seams and details of the seams are given in Table 4.1 and Table 4.2 respectively.

Table 4.1 Lithology of the seams

Coal Seam/Parting	Parting(m)
Banded fine grained sandstone	0.4
Carb shale	
Sandstone	0.4
Grey shale	
[OB]	4
Banded sandstone	1.5 - 2.5
Shale	1
Shaly coal	0.5
Banded sandstone	0.2
Shaly coal	0.2
Sandstone	0.2
Coal	0.2
Sandstone	2
[Seam IX A]	6
Coal	0.2
Shaly coal	0.5
[Seam IX]	4.2
Fine grained banded sandstone	0.3
Carbonaceous shale	0.4
Fine sandstone	2.5
Grey shale	0.4
[Parting]	4 - 5
Seam VIII	4
Grey shale	2
Fine grained sandstone	4.5
[Parting]	6.5
Seam VII	5 - 5.5

Table 4.2 Details of the seams

Name of seam	Details
Seam VII	1. Spacing of joint: 90 cm. 2. Joint direction: 220- 310 ^{0}. 3. Dip of joint: 70 ^{0}.
Parting	1. Vertical joints: every 1m of dip 5-9 ^{0}. 2. Maximum joint spacing: 23cm.
Seam VIII	1. Joint Spacing: 89 cm 2. Joint dip: 93^{0} 3. Strike: 135 ^{0} SE 4. Dip of seam: 5-9 ^{0}
Parting	1. Dip amount: 5^{0}. 2. Joint direction: 210- 260^{0} 3. Joint dip amount: 70^{0}
IX Seam	1. Strike: 170^{0} 2. Joint Orientation: 85^{0} 3. Joint dip: 162^{0}
IX A Seam	1. Joint Spacing: 4 m approx. 2. Bench slope angle: around 70^{0}. 3. 7 joints/m 4. One dip side joint (4 joints /m) 5. One strike side joint (5 - 7 joints / m)

4.3 Data Collection

The objective of the investigation was to design stable slopes so that it facilitates safe operations. The typical analysis ingredients are cohesion and angle of internal friction. These data represent the engineering properties of the area under investigation.

4.4 Laboratory Test

4.4.1 Sample Preparation

Three rock samples are taken from undisturbed specimens by boring. After boring the samples are cut into required dimension (Length/Diameter is greater than 2). The dimensions are given in the Table 4.3.

Table 4.3 Dimensions of the tested samples

Sl. No.	Average Length(cm)	Average Diameter (cm)	Length/Diameter Ratio
Sample 1	11.5	5.38	2.1
Sample 2	11.49	5.43	2.1
Sample 3	11.38	5.55	2.1

4.4.2 Triaxial Testing Apparatus for Determination of Sample Properties

The equipment is designed for testing rock samples with a cell which is designed to withstand a lateral pressure of 150 bar (150kgf/cm^2) and can be used in AIM-050, Load Frame 500 kN (50,000 kgf) capacity. Lateral pressure can be applied with the help of AIM – 246, Constant Pressure System, 150 bar (150 kgf/cm^2).

The equipment consists of a base which houses four valves these valves can be used for measurement of pore pressure, top drainage, bottom drainage, and for entry /exit of cell water. Base has a hole in the center for fixing the locating g pin and bottom pedestal of various sizes. It also has ten threaded holes and two locating pins for aligning and clamping chamber with bolts to base. Chamber has ten free holes and two lifting handles. Top cap is fixed with the chamber. Top cap has an air plug and a pressure inlet plug. A grooved and lapped plunger which can be lifted with the help of two pins provided on the top of the plunger.

Fig. 4.1 A typical triaxial test apparatus

Setting Up

1. First the cell is to be cleaned to be free from all foreign particles.
2. The chamber is removed by unscrewing ten Allen bolts with the help of Allen keys.
3. The base is cleaned and a thin layer of oil is applied on it.
4. The chamber is cleaned from inside and smeared with oil.
5. The locating pin is placed in the center hole. While the right size of pedestal is placed with suitable combination on the locating pin.
6. The sample to be tested is placed on the pedestal. The same size loading pad is kept on the top of the sample. The suitable copper tubes are connected with the bottom pedestal and top loading pad. The chamber is placed in the locating pin and clamps it to the base with the help of Allen bolts.

4.4.3 Test Procedure

1. The water is filled in the cell with the help of funnel and rubber tube through the valve meant for this purpose.
2. The hose pipe from AIM – 246 is connected with constant pressure system to pressure inlet plug and apply required lateral pressure around the sample.
3. Designed level of cell pressure is built up using AIM - 246. The lateral pressure is to be maintained constant while samples are subjected to different consolidation stress history as well as during shear tests. The readings are recorded.
4. After the test is over, remove the loading pad, copper tube connections and pedestal. The cell is cleaned and a thin layer of oil is put on the base and inside of chamber.

Table 4.4 Readings of proving and deviator and dial gauge

Sl. No.	Dial Gauge Reading	Corrected Area	σ_3 = 100 kPa			σ_3 = 200 kPa			σ_3 = 300 kPa		
			Proving Reading	Deviator Reading	Deviator stress (kPa)	Proving Reading	Deviator Reading	Deviator stress (kPa)	Proving Reading	Deviator Reading	Deviator stress (kPa)
1	0	12.19	0	0	0	0	0	0	0	0	0
2	50	12.25	16	54.4	44.40	16	54.4	44.4036	21	71.4	58.279
3	100	12.31	34	115.6	93.88	21	71.4	57.9869	33	112.2	91.122
4	150	12.38	39	132.6	107.14	33	112.2	90.662	49	166.6	134.62
5	200	12.44	45	153	123.0	62	210.8	169.47	59	200.6	161.269
6	250	12.50	56	190.4	152.28	87	295.8	236.591	86	292.4	233.87
7	300	12.57	68	231.2	183.97	109	370.6	294.899	112	380.8	303.015
8	350	12.63	99	336.6	266.46	121	411.4	325.678	149	506.6	401.04
9	400	12.70	114	387.6	305.24	143	486.2	382.897	176	598.4	471.258
10	450	12.76	132	448.8	351.60	157	533.8	418.194	205	697	546.05
11	500	12.83	139	472.6	368.31	179	608.6	474.299	244	829.6	646.529
12	550	12.90	146	496.4	384.82	196	666.4	516.61	265	901	698.478
13	600	12.97	152	516.8	398.51	219	744.6	574.179	283	962.2	741.975
14	650	13.04	163	554.2	425.08	241	819.4	628.498	301	1023.4	784.970
15	700	13.11	192	652.8	498.03	258	877.2	669.234	319	1084.6	827.463
16	750	13.18	211	717.4	544.37	268	911.2	691.436	346	1176.4	892.674
17	800	13.25	234	795.6	600.45	276	938.4	708.226	379	1288.6	972.528
18	850	13.32	252	856.8	643.12	298	1013.2	760.523	389	1322.6	992.763
19	900	13.40	261	887.4	662.45	314	1067.6	796.978	406	1380.4	1030.48
20	950	13.47	265	901	668.91	332	1128.8	838.034	411	1397.4	1037.44

21	1000	13.54	267	907.8	670.23	341	1159.4	855.997	417	1417.8	1046.77
22	1050	13.62	267	907.8	666.51	345	1173	861.226	421	1431.4	1050.94
23	1100	13.70	-	-	-	345	1173	856.415	423	1438.2	1050.03
24	1150	13.77	-	-	-	-	-	-	424	1441.6	1046.60
25	1200	13.85	-	-	-	-	-	-	424	1441.6	1040.69

4.4.4 Plotting of Mohr's Circle

With σ_3 = 100 kPa, 200 kPa and 300 kPa respectively and the total stress σ_1 = 670 kPa, 861 kPa and 1050 kPa the respective Mohr's circles are drawn. Mohr's circle showed cohesion and angle of internal friction as 180 kPa, and 26 degrees, respectively.

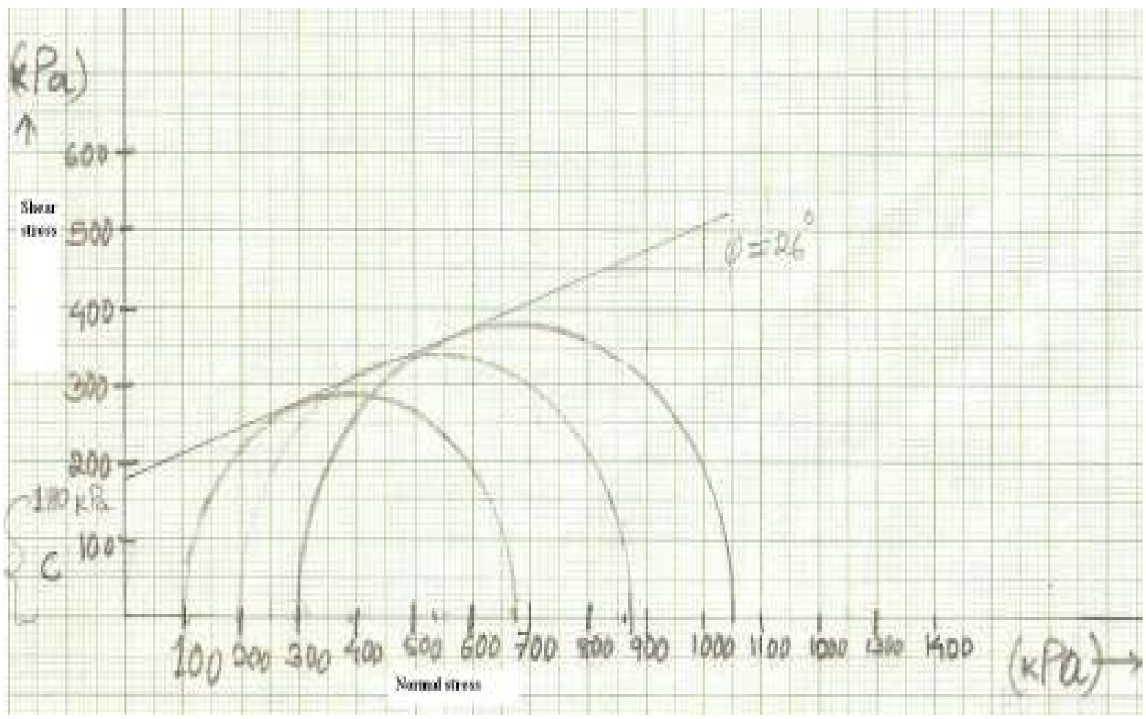


Fig. 4.2 Mohr's circle for determination of cohesion and angle of internal friction

4.5 Parametric studies

Parametric studies were conducted through numerical models (FLAC/Slope) to study the effect of cohesion (140-220 kPa) and friction angle (20°-30° at the interval of 2°). Pit slope angle was varied from 35° to 55° at an interval of 5°.

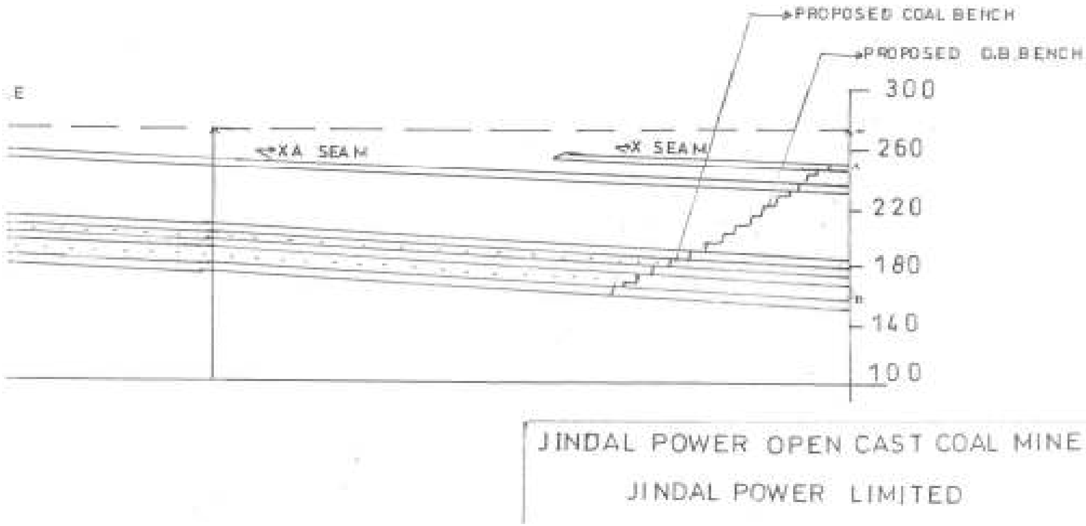
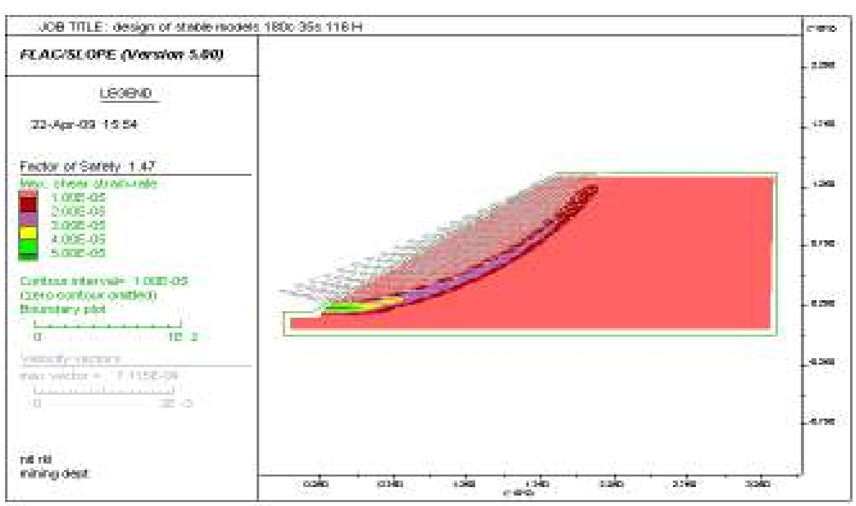


Fig. 4.3 Projected pit slope

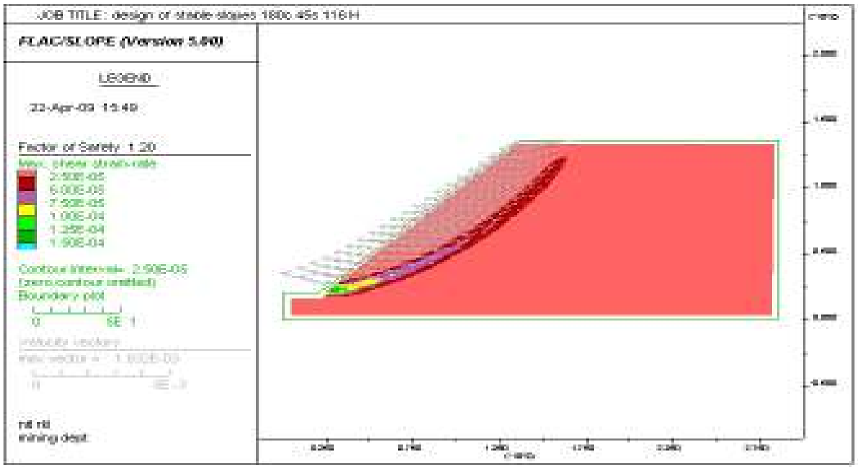
Table 4.5 Safety factors for various slope angles (Depth= 116m)

Sl. No.	Slope angle(°)	Cohesion(kPa)	Friction angle(°)	Factor of Safety
1	35	180	26	1.47
2	40	180	26	1.32
3	45	180	26	1.2
4	50	180	26	1.09
5	55	180	26	1.0

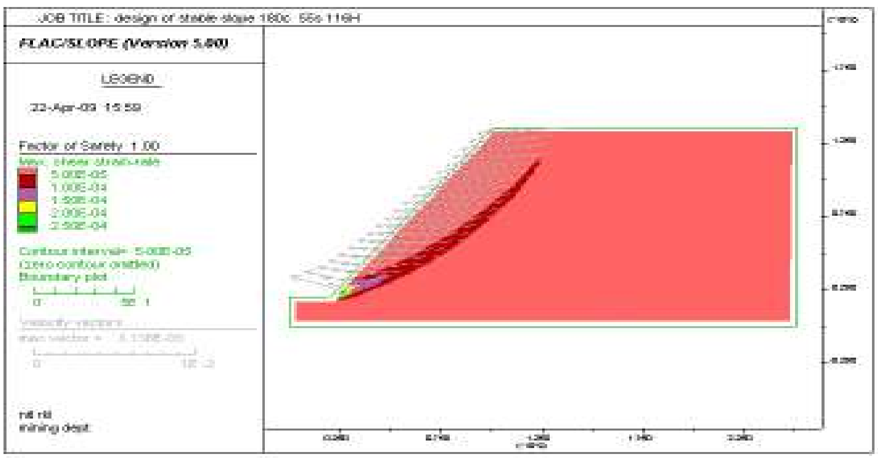
Fig. 4.4 Some models developed by FLAC/Slope with varying cohesion and friction angle



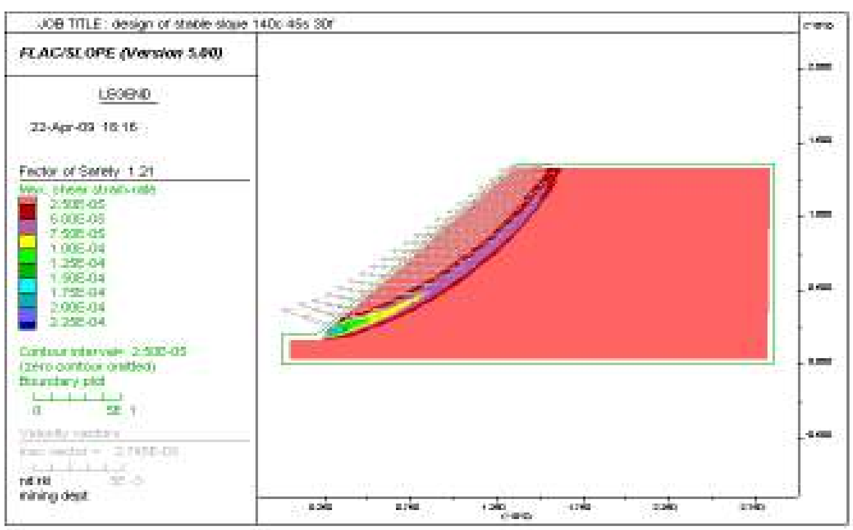
a) Depth= 116m, C=180 kPa, Slope angle= 35°, Friction angle = 26° (FOS = 1.47)



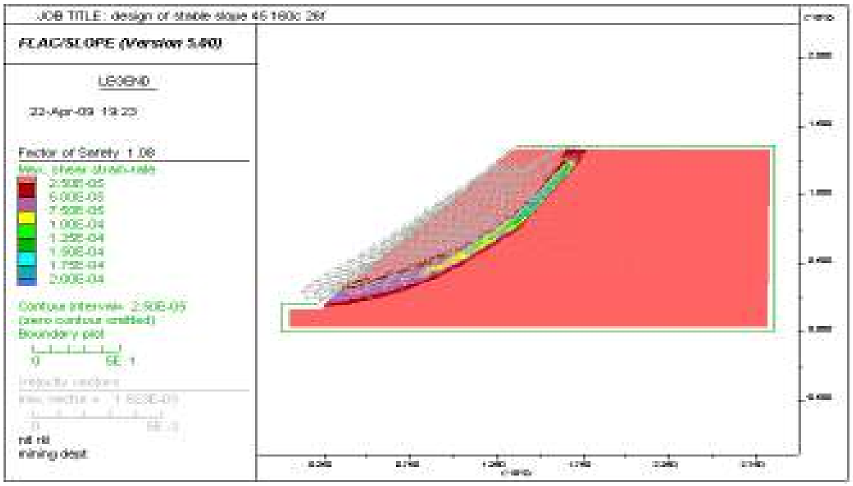
b) Depth= 116m, C=180 kPa, Slope angle= 45°, Friction angle = 26° (FOS = 1.2)



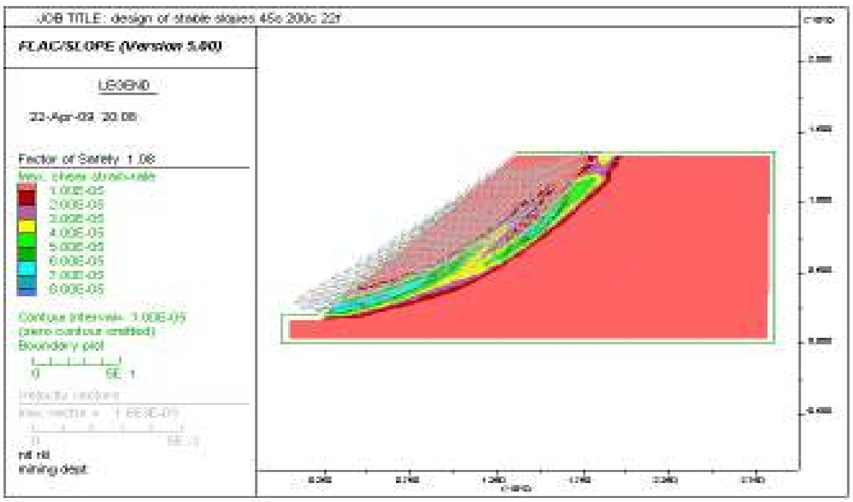
c) Depth= 116m, C=180 kPa, Slope angle= 55°, Friction angle = 26° (FOS = 1.0)



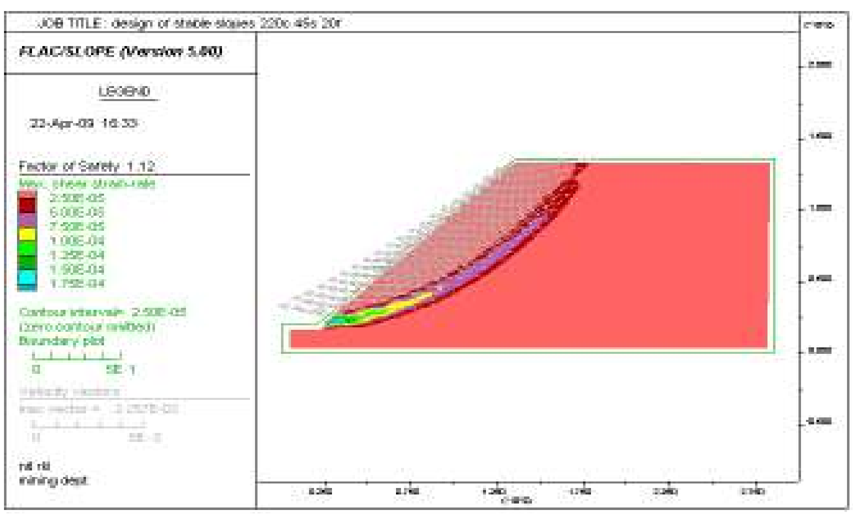
d) Depth= 116m, C=140 kPa, Slope angle= 45°, Friction angle = 30° (FOS = 1.21)



e) Depth= 116m, C=160 kPa, Slope angle= 45°, Friction angle = 26° (FOS = 1.08)



f) Depth= 116m, C=200 kPa, Slope angle= 45°, Friction angle = 22° (FOS = 1.08)



g) Depth= 116m, C=220 kPa, Slope angle= 45°, Friction angle = 20° (FOS = 1.12)

Table 4.6 Safety factors for various C and Ø values (Depth= 116m)

Sl. No.	Cohesion (kPa)	Friction Angle (˚)	Factor of Safety
1	140	20	0.91
		22	0.92
		24	0.97
		26	1.03
		28	1.09
		30	1.21
2	160	20	0.92
		22	0.97
		24	1.03
		26	1.08
		28	1.14
		30	1.2
3	200	20	1.03
		22	1.08
		24	1.13
		26	1.19
		28	1.25
		30	1.31
4	220	20	1.12
		22	1.13
		24	1.19
		26	1.25
		28	1.31
		30	1.44

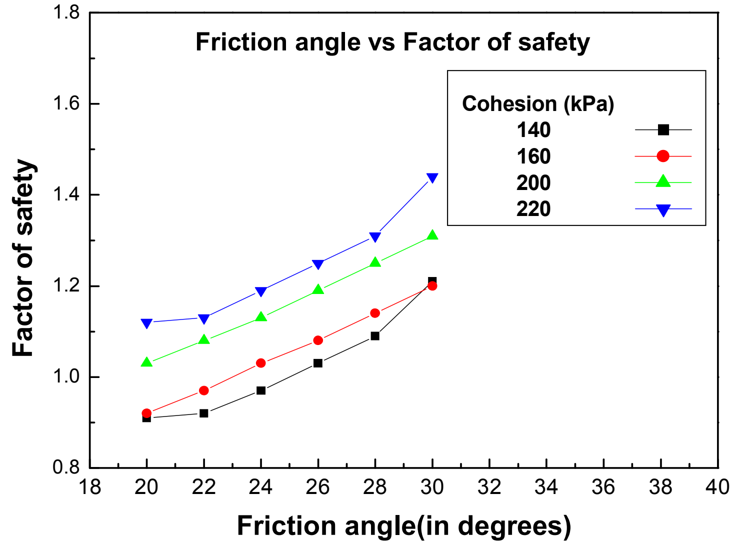


Fig. 4.5 Variation of factor of safety with friction angle for different cohesion

4.6 Result and Discussion

1. Based on Table 4.5 it is concluded that as the pit slope angle increases, the stability of the slopes decreases. The slope angle of 45° is having a factor of safety of 1.2 which is quite safe and matches with theory. Lower the pit slope angle, higher is the stripping (mining of waste rock), which will in turn have direct consequences on the economy of the mining operation.

2. Based on Table 4.6 it is concluded that as the cohesion and angle of internal friction increases, the factor of safety increases. As the cohesion increases, the binding property enhances which makes the slopes stable. High water content can weaken cohesion because abundant water both lubricates and adds weight to a mass. Moreover alternating expansion by wetting and contraction by drying of water reduces strength of cohesion.

3. While running the numerical model FLAC/Slope it was observed that factor of safety changes with change in the resolution of the numerical mesh (coarse, medium and fine). Incase of coarse mesh the factor of safety is quite approximate, while in fine mesh the factor of safety converges to the nearest possible value making it more accurate. However, calculation in coarse mesh is faster than in fine mesh. So depending upon the requirement and time availability of modeler, the mesh has to be selected.

CHAPTER: 05

CONCLUSION

CHAPTER: 05

CONCLUSION

5.1 Conclusion

Opencast mining is a very cost-effective mining method allowing a high grade of mechanization and large production volumes. Mining depths in open pits have increased steadily during the last decade which has the increased risk of large scale stability problems. It is necessary to assess the different types of slope failure and take cost effective suitable measures to prevent, eliminate and minimize risk.

The different types of the slope stability analysis techniques and software are available for slope design. Numerical modelling is a very versatile tool and enables us to simulate failure behavior and deforming materials. FLAC/Slope is user friendly software which is operated entirely from FLAC's graphical interface (the *GIIC*) and provides for rapid creation of models for soil/rock slopes and solution of their stability condition. Moreover it has advantages over a limit equilibrium solution like any failure mode develops naturally; there is no need to specify a range of trial surfaces in advance and multiple failure surfaces (or complex internal yielding) evolve naturally, if the conditions give rise to them. In this project, an attempt has been made to get acquaintance with the powerful features of FLAC/Slope in analysis and design of stable slopes in opencast mines. Data was also collected from Jindal Opencast Mine with 116m ultimate pit depth at Raigarh in Chhattisgarh State to assess the effects of cohesion and angle of internal friction on design of stable slope using FLAC/Slope.

The parametric study which was carried by varying the cohesion, angle of internal friction and ultimate slope angle showed that with increase in ultimate slope angle, the factor of safety decreases. Moreover cohesion and angle of internal friction are quite important factors affecting slope stability. With increase in both the parameters the stability increases. Conduct of slope stability assessment in Indian mines is mostly based on empirical and observational approach; hence effort is made by statutory bodies to have more application of analytical numerical modelling in this field to make slope assessment and design scientific. This will ensure that suitable corrective actions can be taken in a timely manner to minimize the slope failures and the associated risks.

5.2 Scope for Future work

For the parametric studies, only cohesion and friction angle have been considered. However this study can be extended to individual bench angles where all the benches may not be of same height. The conditions assumed during this analysis are such that there is no effect of water table and geological disturbances. Along with cohesion and friction angle other parameters like effect of geological disturbances, water table and blasting can be carried out. For slope stability analysis other numerical models such as UDEC and Galena can also be used in order to compare the sensitivity and utility of the different software.

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