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GROUND CONTROL-I

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SURFACE MINING

Assignment # 2

Explain the various characteristics or factors of the minerals deposit that influence their mineability.

Ans. The various factors of the nature of mineral deposit that affect their mineability are as follows:

① Form and Shape of Deposit

Depending on the mode of formation of deposit the deposits may be:

(a) Stratified
(b) Veins or lobes
(c) Massive
(d) Pipe deposits
(e) Oxidised or Lateritic deposits.

(a) Stratified – These are characterised by continuity and regularity of mineralisation in the whole bed.

Common deposits of their type are syngenetic sedimentary deposits such as coal, limestone, manganese or their metamorphosed equivalents.

Stratified deposits may have dips ranging from 0° to 90°. They may be gently or highly folded or faulted. but the bed or seam continues across any structural discontinuity. A striking example of highly folded stratiform deposits is offered by the manganese ore deposits of M.P. Stratiform deposits have a large extent and terminate only at unconformities and show a wide variation of shape, extent and distribution of values depending on their mode of formation.

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② Size and extent of deposit.

(b) Veins or Lobes – They are formed by fissure filling or replacement or a combination of both. They are mainly controlled by structures such as shear or tension fractures, folds, etc. Vein deposits thus range widely between cases where the ore body is localised by a fissure cutting across a favourable bed to a blanket like replacement extending widely along a favourable formation intersecting a fissure carrying the mineralising solutions. Ore shoots generally form the commercially minable parts of veins. Most commonly ore shoots are of tabular form which are roughly elliptically in the plane of vein with pitch ~~ea~~ varying from 0° – 90°. Ore shoots can also be lenticular and the thickness of the lens vary from a few metres to few hundreds of metres in length and breadth.

(c) Massive deposits – Disseminated ore bodies often take a massive form. They are usually irregular in shape ~~at~~ although many deposits show a large lateral extent in the middle reducing progressively both towards the top and bottom.

(d) Pipe deposits – These deposits are usually of narrow cylindrical shape with near vertical axis. Volcanic diamondiferous plugs are of this shape.

(e) Oxidised or laterite deposits – These have a common feature of a more or less flat bottoming controlled by the ground water table. Valuable mineral deposits may be formed in the oxidized zone of otherwise uncommercial beds/veins ~~by~~ of primary mineralization by the leaching and removal of valueless elements causing residual concentration of valuable minerals Bauxite, iron, nickel, lead, gold deposits formed this way.

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(2) Size and extent of deposit

All forms of deposits vary considerably in size. The size of an orebody from the point of view of mining should indicate the plan area at various levels. Such data is used to get a three-dimentional view of the ore body.

Massive deposits have a large thickness both laterally as well as in a vertical direction. Pipe deposits usually have a large vertical extent compared to lateral extent. Stratified deposits have usually a large extent along dip and strike while their thickness varies. Such deposits are classified as:

(i) Very thin (< 0.75 m)
(ii) thin (0.75 – 3m)
(iii) thick (3 – 30m)
(iv) very thick (>30m)

From the mining point of view. Flat deposits of any thickness can be mined out using surface mining methods. Steep deposits of thick and very thick classes can only be mined by open cast mining.

(3) Lay of deposit

This indicates the spatial attitude of the deposit. The spatial attitude is of less significance in massive deposits but in bedded deposits it is extremely important from a mining point of view. Dips of deposits vary from 0 to 90°. Most deposits exhibit more or less consistent dip over large extent through small changes may be there.

For the purpose of mining, mineral deposits are classified as

(i) flat (0 – 30°) (ii) inclined (30° – 45°) (iii) steep (45° – 90°)

For u/g mining, it would be desirable to have a very flat subgroup with a dip less than 20° and very steep subgroup with a dip exceeding 60°.

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4) Depth of the Deposit

The depth of the deposits from the surface depends on :

(i) Mode of formation (iii) Tectonic and structural changes
(ii) Factors of deposition (iv) Changes in surface topography

Some shallow depth deposits are : oxidized and laterite mineral deposits ; Residual deposits having of gold and base metals ; Alluvial deposits.

Some deposits having a large vertical extent are :

Supergene deposits extends upto 1000m ; mesothermal and Leptothermal types of hypogene deposits extend down from 1000m to 2000m while hypothermal type of hypogene deposits extend down even further. e.g. Gold veins of Kolar gold fields

The depth of deposits greatly affect the type of mining method followed :

As in India
- When depth < 300m - Surface mining followed
- When depth > 300m - u/g mining is more feasible

5) Mineralogical Composition and Grade of the deposit

Mineral deposits may have a single valuable or several valuable minerals occuring together.

e.g. Haematite ores are usually monometallic while almost all base metal deposits are polymetallic in nature. Thus, lead and zinc often occur together in the same ore body.

Apart from polymetallic there may be occurence of the same metal in different mineral forms in an ore body

e.g. Copper ore have native copper, cuprite, malachite etc.

Such multi-mineral occurences are of importance to the mining engineer in fact that they pose problem in concentration and separation of the minerals and may have influence on the mining method. Apart from mineralogical composition,

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the grade of deposit is of great importance from mining point of view. The average grade of deposit is of little imp. if distribution of values is not uniform throughout the deposit. Thus, it is necessary to determine the grade over suitable sections of the deposit. Grade together with the size and physical nature of a deposit often determines it mineability. It is not the valuable minerals alone that determine the mineability. Sometimes unwanted impurities may render a deposit unmineable. Presence of dangerous gases such as methane poses mining problems. Even mineral deposits with a high silica content may pose the hazard of silicosis in the course of their mining.

6.) Physio - mechanical properties of the ore or mineral and the wall rocks :

Physical prop. of the mineral that are of importance to the mining engineer are :

(i) Colour - ore having distinctive colour from their waste are easily amenable to sorting.

(ii) Hardness - softer ores are easier and cheaper to mine

(iii) Specific gravity - It determines the size of handling and preparation equipement

(iv) Grain size - determines the method of concentration and recovery of value.

(v) Porosity and permeability - determines water content and water influx during mining.

(vi) degree of fracturing - cleavability, friability - determines the prod. percentage of fines during mining.

(vii) Degree of fracturing and cleavage - also affect the degree of fragmentation in blasting as well as strength of ore pillar.

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Mech. properties of elastic wall rock ~~as~~ that are of prime importance are compressive, tensile and shear strength, elasticity, poisson's ratio, abrasity, grindability and drillability and creep.

7. Geological disturbances and presence of other deposits nearby :

These also influence the mineability of mineral deposits. Highly folded and faulted deposits can be difficult to mine, if not completely unmineable. A close study of the structural disturbances affecting the mineral deposit is thus called for. While single ore bodies are easily mineable, several deposits occuring close together may introduce difficulty in mining.

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ROLL NO. : 11108EN035

B.Tech Part - III

MINING ENGG.

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Class → B.Tech (Sem III)
Roll No → 11108EN034
Subject → UCM - I

ASSIGNMENT - 2

Burn Cut Method Of Drilling :-

In Burn cut, parallel holes at right angles to the face are drilled in a cluster which may take several forms one of which is shown behind in the figure. Some of the holes (which are sometimes of larger diameter than the rest) are left uncharged to give relief to the heavy concentration of explosives in charged holes. This cut is effective in hard, brittle, homogeneous ground which breaks evenly; but cannot be used in springly plastic ground.

The advantages of the Burn cut are:-

(i) Drilling time is considerably reduced and supervision in drilling is easier as holes are straight.
(ii) Depth of pull is dependent of the size of the drift
(iii) The quantity of blasted material is not projected far with a suitable form of the cluster.

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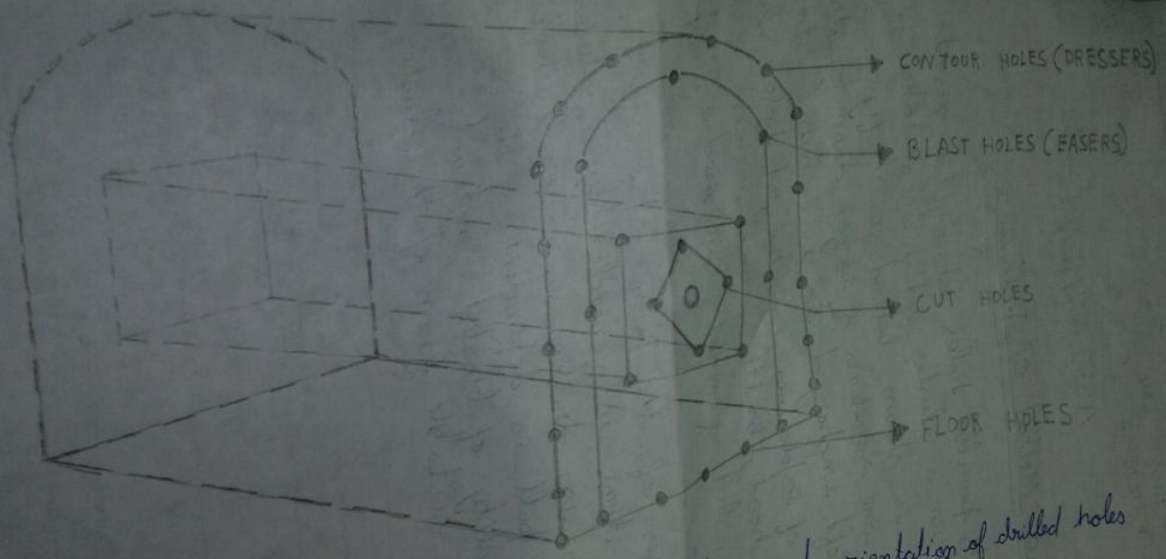

FIGURE :- Design Pattern and orientation of drilled holes in Burn cut method of drilling

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25/07/2013.

Topics:-

1.) Design of Mine openning
2.) Design of pillars
3.) Rock Support
4.) Caving
5.) Subsidence
6.) Slopes
7.) Rock Burst
8.) Load and Deformation Monitoring

Mine opening:-

Any opening, excavation or structure above or below the ground is a disturbance (from a ground control point of view) to the rockmass. This disturbance is significant.

1.) First type disturbance in rocks is the disturbance of insitu stress flux. in the original orientation

~~Depending on the extent of disturbance w.r.t. the ability of rock mass to resist the disturbance~~

Mine opening
- narrow
- wide

These types behave totally differently. Wide is less stable than narrow openning.

The terms have nothing to do with size. They are influenced by structure of rocks in which the openings are made.

The same opening can be narrow in one type of rockmass and wide in another type of rockmass.

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In Case of Single openings:-

narrow opening is that type of opening which is not influenced by the presence of joint structure existing in the rockmass. In contrast to this. In case of wide opening the stability or performance of the opening is essentially affected by the presence of joint structure already existing in the rockmass.

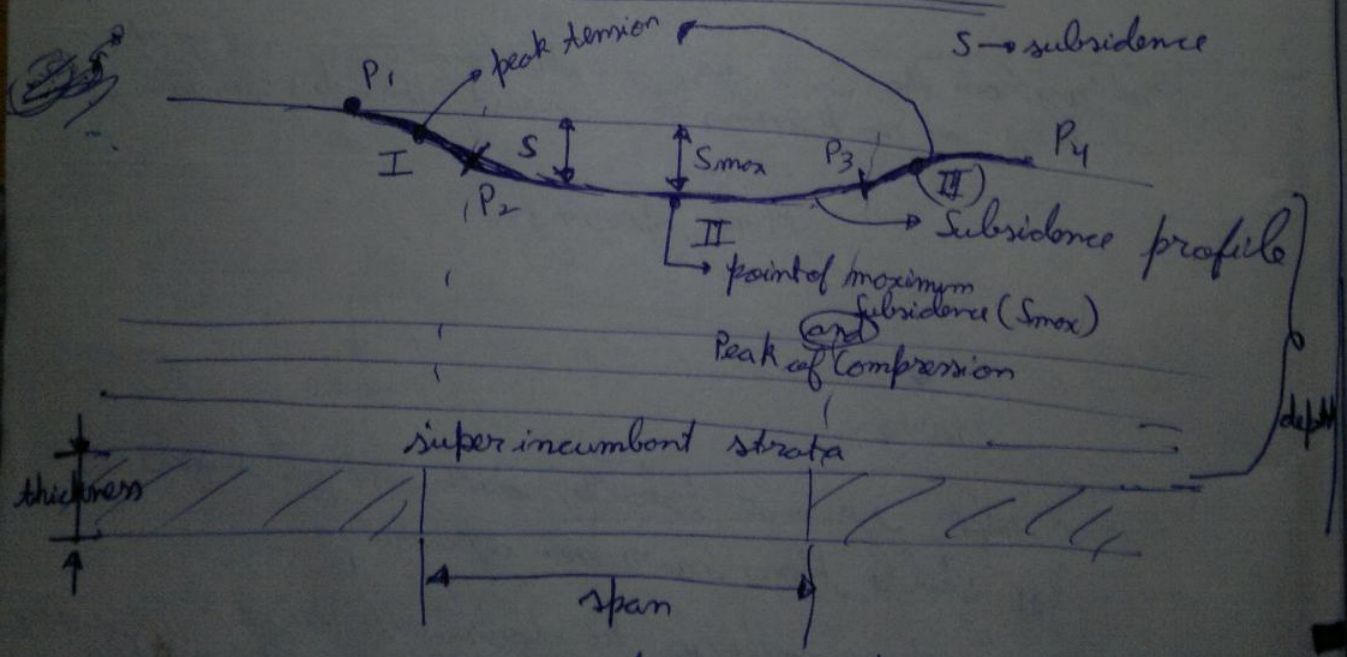
In case of multiple openings:-

an opening is treated as narrow opening only when it is neither affected by the presence of joint structure already existing in the rockmass or by the presence of other structure that has been introduced by mining operations.

If one structure affects the other structure presence then it is wide opening.

29/07/2013

Mine Subsidence (BKS Sir)



The ground movement of the surface (vertical or horizontal) is called mine subsidence

This profile is made up of three curves. (subsidence profile)

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When seen from above:-

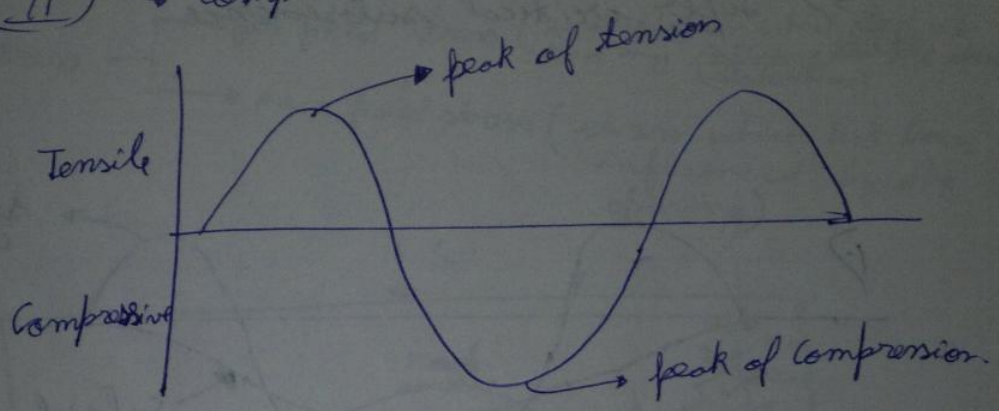
(I) → Convex curve

(II) → Concave curve

(III) → Convex curve

(III), (I) → tensile strain

(II) → Compressive strain



P_1, P_2, P_3, P_4 → Inflexion points of zero strain

Rock is very weak in tension and peak of tension lies outside the panel area so damage is seen more outside the panel area than within the panel area.

The inflexion points P_2, P_3 where there is change from tensile to compressive strain lie directly above panel and slightly inside or slightly outside the compressive strain curve.

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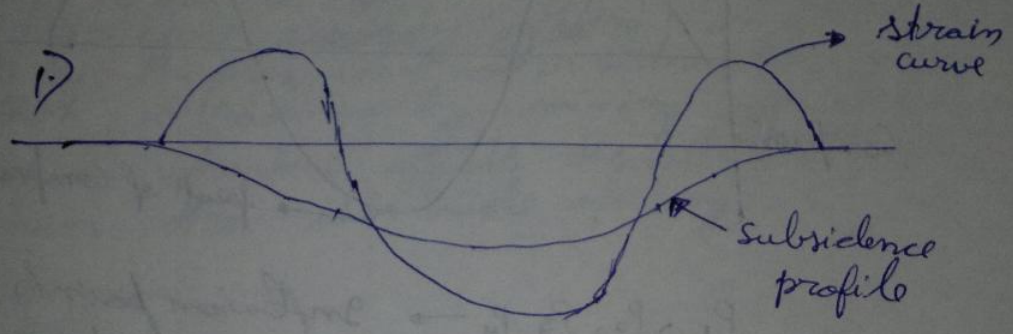
⟹ When span is increased subsidence increases and the limit of subsidence is the seam thickness but it is never the case.

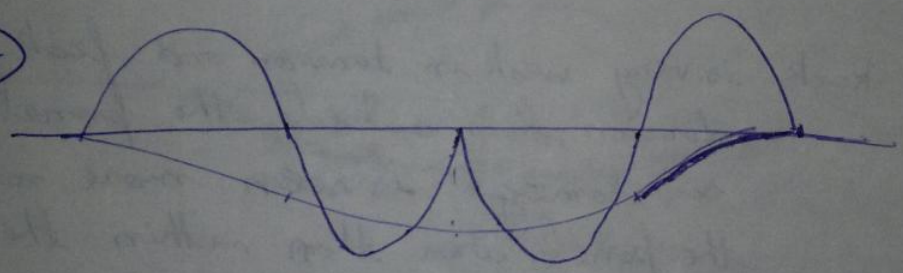
Classification of Subsidence:—

1.) Sub-critical subsidence
2.) Critical subsidence
3.) Super critical subsidence

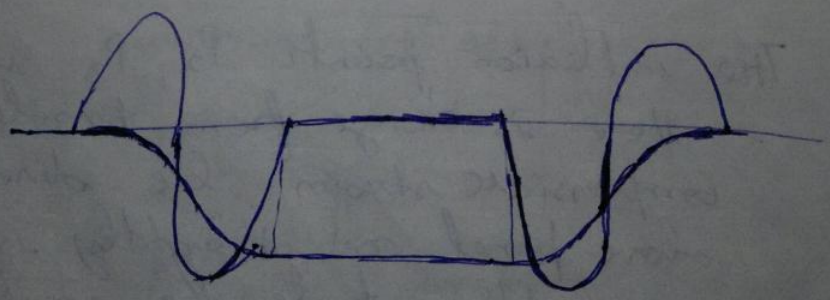
1.)



2.)



3.)



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(GSPS Sir) 01/08/2013

Stresses around Opening

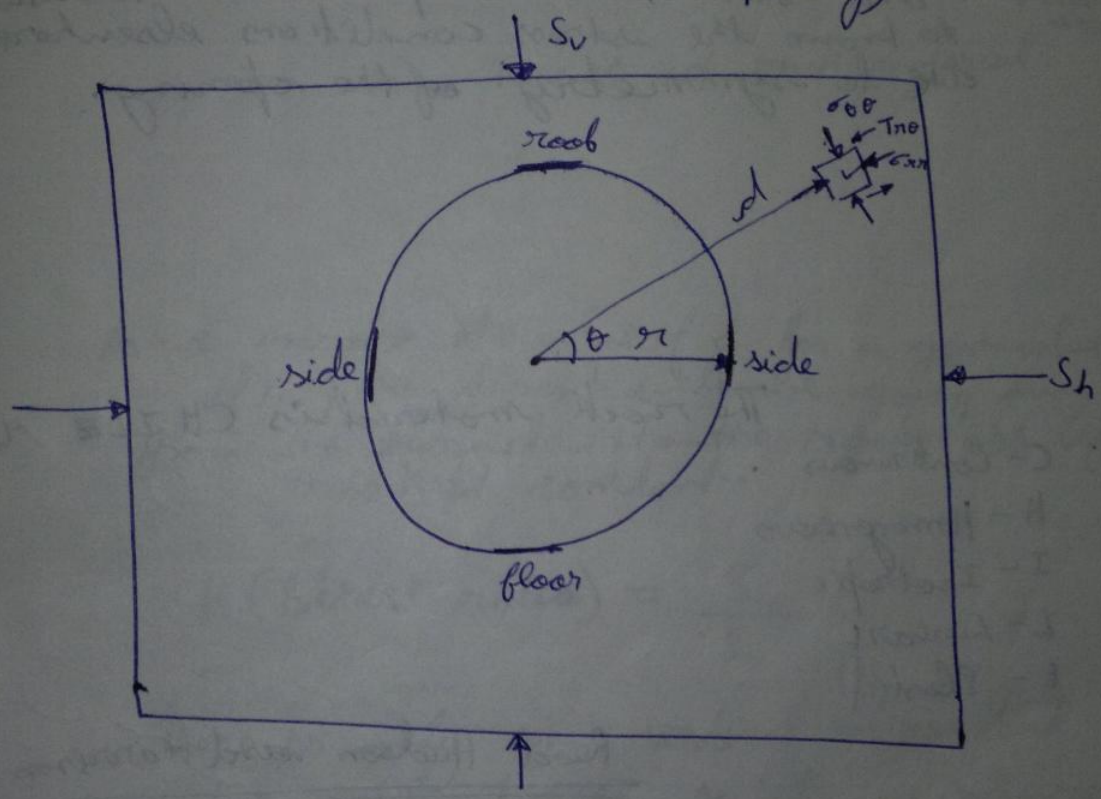
Circular opening

Driven in a rockmass with a vertical stress S_v and horizontal S_h.

S_v and S_h are far field stress (stress are acting at infinite)

Stress → far field stress (is that stress which does not have any influence (disturbance) due to the opening)

→ near field stress (stress which has been influenced by the opening)



Both S_v and S_h are compressive in nature

The radius of the opening which demarcates the size is r

we are trying to evaluate the influence of the opening on a rockmass at a distance d from the center of the opening

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we are calculating that due to the introduction of the opening to what extent the stress have been affected.

We have 3 components of stress:-

$\sigma_{\theta\theta}$ → Tangential stress

σ_{rr} → radial stress

$\tau_{r\theta}$ → shear stress

Rather than focussing on the entire periphery of the opening we can focus on ℄ roof and side and extrapolate the results to know the stress conditions elsewhere due to symmetry of the opening.

The rock material is CHILE Material.

C- Continuous.
H- Homogeneous
I- Isotropic
L- Linear
E- Elastic

Read Hudson and Harrison Vol-I

$$\sigma_{rr} = \left(\frac{S_h + S_v}{2}\right)\left(1 - \frac{r^2}{d^2}\right) + \left(\frac{S_h - S_v}{2}\right)\left(1 - \frac{4r^2}{d^2} + \frac{3r^4}{d^4}\right)\cos 2\theta$$

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$$\begin{cases} \sigma_{\theta\theta} = \left(\dfrac{S_h + S_v}{2}\right)\left(1 + \dfrac{r^2}{d^2}\right) - \left(\dfrac{S_h - S_v}{2}\right)\left(1 + \dfrac{3r^4}{d^4}\right)\cos 2\theta \\ \tau_{r\theta} = \left(\dfrac{S_h - S_v}{2}\right)\left(1 + \dfrac{2r^2}{d^2} - \dfrac{3r^4}{d^4}\right)\sin 2\theta \end{cases}$$

These set of formulae are known as closed for solution for studying stresses around a circular material.

Minimum value of $d/r = 1$ because for $d < r$ the area is a void/constructed area.

There is not rock material.

As we increase the value of d for a given value of r, the element will be going away from the tunnel into the rockmass if θ is kept constant.

$$K \text{ (Stress ratio)} = \frac{S_h}{S_v}$$

There is only one vertical stress but 2 horizontal stresses so the S_h we are considering here is the mean horizontal stress.

$$\left(S_h = \frac{S_{h\,max} + S_{h\,min}}{2} \right)$$

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$\underbrace{S_v}_{MPa} = \gamma H$ → cover depth (m)

γ → unit weight of rock. MPa/m

$\boxed{Pa/m \equiv N/m^3}$

γ → the amt of the weight the ~~ground~~ rock ~~will have~~ having 1 m^2 surface area of unit thickness

$\gamma = 0.025$ for practical purposes but depends on the density of rock.

we can have value of $\underbrace{K > 1}_{\text{horizontal stress} > \text{Vertical stress}}$ or $\underbrace{< 1}_{\text{horizontal stress} < \text{Vertical stress}}$ or $K = 1$

If this condition is encountered then while designing it has to be taken into consideration. This is a very special case. Otherwise it will create safety and maintainance problems throughout the life of the tunnel

if $S_v = P$ then $S_h = KP$

$\frac{\sigma_{rr}}{P}$

$\frac{\sigma_{\theta\theta}}{P}$

$\frac{\sigma_{r\theta}}{P}$

This gives us the extents of intensification of stress condentrations

if ratio $> 1 \Rightarrow$ intensified

if ration $< 1 \Rightarrow$ reduction in concentration

Lab	K
1	0.1
2	0.2
3	0.5
4	0.[illegible]
5	0.8
6	1
7	1.2
8	1.4
9	[illegible]
10	2.[illegible]
11	3
12	5

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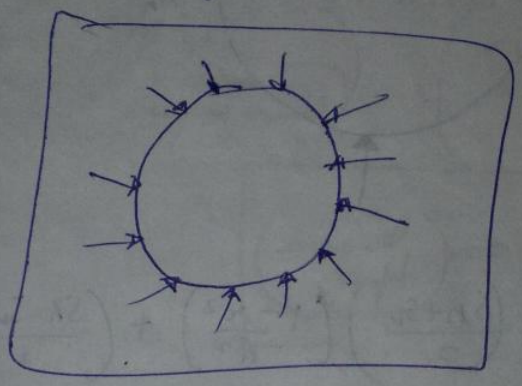
when K = 1 → This type of stress condition is known as hydrostatic stress condition.

if K = 1 then the amount of Far field stress that the opening is subjected to is equal.



Lab	K values
1	0.1
2	0.2
3	0.5
4	0.7
5	0.8
6	1
7	1.2
8	1.4
9	2
10	2.5
11	3
12	5

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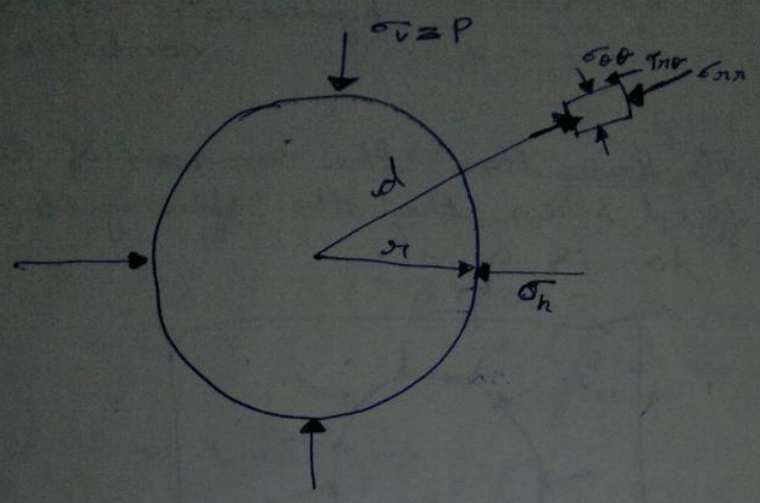
02/08/2013 (GSPS Sir)



$$\sigma_{rr} = \left(\frac{S_h+S_v}{2}\right)\left(1-\frac{r^2}{d^2}\right) + \left(\frac{S_h-S_v}{2}\right)\left(1-\frac{4r^2}{d^2}+\frac{3r^4}{d^4}\right)\cos 2\theta$$

$$\sigma_{\theta\theta} = \left(\frac{S_h+S_v}{2}\right)\left(1+\frac{r^2}{d^2}\right) - \left(\frac{S_h-S_v}{2}\right)\left(1+\frac{3r^4}{d^4}\right)\cos 2\theta$$

$$\tau_{r\theta} = \left(\frac{S_h-S_v}{2}\right)\left(1+\frac{2r^2}{d^2}-\frac{3r^4}{d^4}\right)\sin 2\theta$$

Case - I :-

If $S_h = S_v$ (Hydrostatic condition)

$$\sigma_{rr} = S_v\left(1-\frac{r^2}{d^2}\right)$$

$$\sigma_{\theta\theta} = S_v\left(1+\frac{r^2}{d^2}\right)$$

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if $r = 2m$ $\quad 2 \leq d \leq 10$

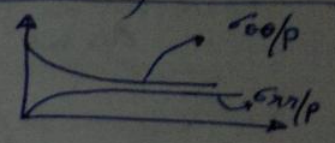
at $\theta = 0$

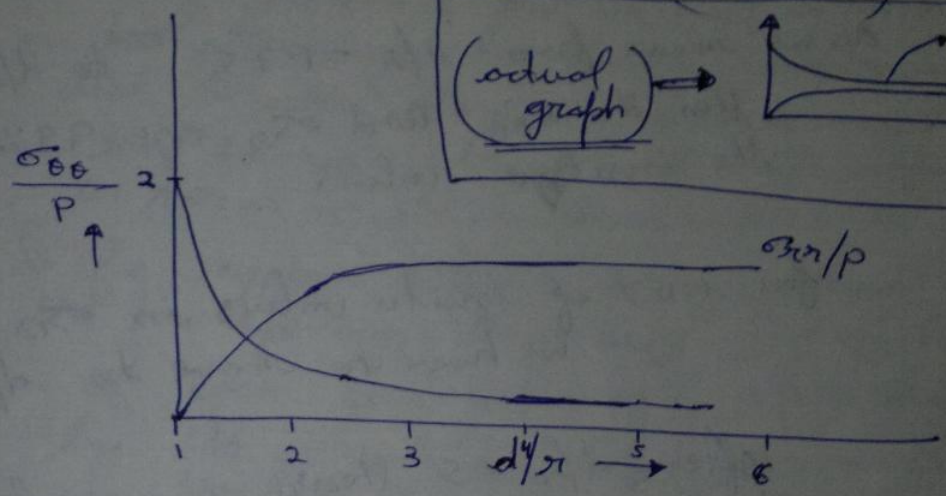
	$d=2$	$d=4$	$d=6$	$d=8$	$d=10$
$\sigma_{\theta\theta}$	$2P$	$1.25P$	$1.111P$	$1.0625P$	$1.04P$
σ_{rr}	0	$0.75P$	$0.889P$	$0.9375P$	$0.96P$

($\sigma_{\theta\theta}$ (↓) at a ~~new~~ variable ~~constant~~ rate)

(σ_{rr} (↑) at a variable rate)

(actual graph) ⟹





$\frac{d}{r}$ → indicates how far our rock element is from the periphery of the tunnel.

⟹ The results are independent of the value of θ

At the surface :-

⟹ radial stress is zero at all points on the surface

⟹ tangential stress is 2 times the virgin stress (i.e. the vertical stress)

⟹ In hydrostatic condition only.

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As we go into the rockmass the value of $\sigma_{\theta\theta}$ decreases but with a variable rate and the value of σ_{rr} increases with a variable rate

As we exceed the points of $d/r = 1.75$ the value of $\sigma_{\theta\theta}$ or σ_{rr}, then they are achieving almost the insitu value.

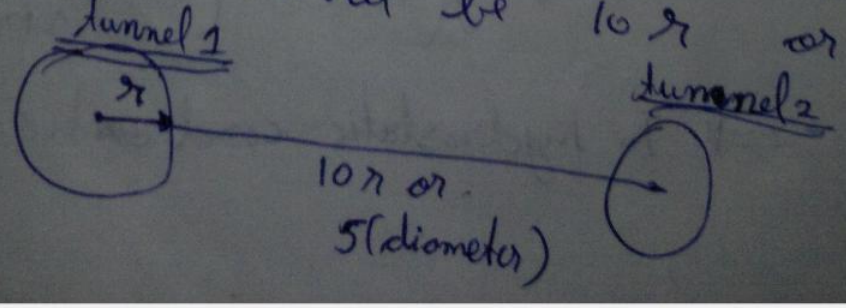
As we move from $d/r = 1.75$ to $d/r = 3$ then the $\sigma_{\theta\theta}$ and σ_{rr} are 99% of the insitu values

for 100% of insitu values in σ_{rr} and $\sigma_{\theta\theta}$ we have to move to $d/r = 5$

After $d/r = 5$ there is no further chances of saturation in the values of σ_{rr} and $\sigma_{\theta\theta}$

In Hydrostatic conditions only:-

If we want to have two companion excavations which remain in proximity of each other without any influence on each other then the minimum distance between the two should be $10r$ or 5 (diameter)



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Zone of influence → the dist away from the surface of the structure upto which the original state of stress is likely to be disturbed.

The Concept of zone of influence is very important for mine design because it may provide considerable simplification of a design problem.

Our design problem is that we want to establish 2 ~~two~~ openings by having minimum dist. between them in such a way that they do not interact with each other in terms of their stress flux.

So if a required distance is maintained between the two openings then ~~only~~ the stability of each opening is only challenged by its own size, shape and other attributable parameters and not by the size or shape of the nearby opening. Then this design problem will become simple.

➡ Our mining structures are not simple openings because they have influence on each other.

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Zone of influence defines the domain of significant disturbance of the pre-mining stress field by an excavation. It differentiates between the near field and far field stress of an opening. For hydrostatic stress condition it is observed that the state of stress is not significantly disturbed beyond a distance $d = 5r$. Therefore if two openings are kept at a spacing of $10r$ it is possible to maintain those openings without their mutual interaction. Zone of Influence can be a handy tool for design of simple openings.

05/08/2013

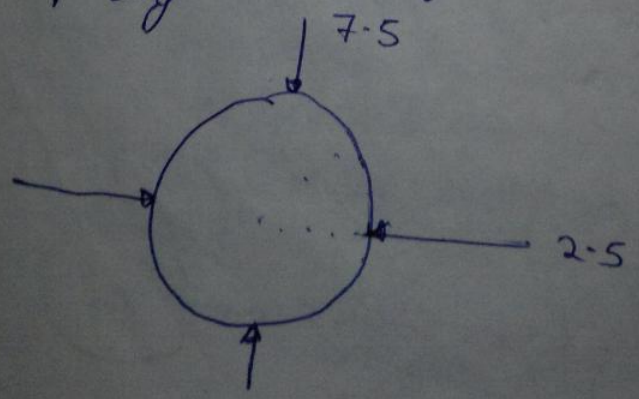
GROUND CONTROL (BKS Sir)

08/08/2013

GROUND CONTROL (GSPS Sir)

Estimation of zone of failure:- (IInd assignment)

Q: A circular tunnel of 4 m diameter is driven in CHILE rockmass under field stress as given in diagram below. The Compressive strength of the rockmass is 16 MPa. Estimate the zone of failure along periphery of the opening



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+ve stress → compressive
-ve stress → tensile } always.

Ist assignment:-

Using Closed form solution available for circular opening to establish the fact that stresses induced along the periphery of the opening does not depend on the size of the opening or the elastic constant of the rockmass.

DESIGN OF PILLARS:-

In U/G working we have to have a form of pillar designed for a purpose.

Depending on purpose of pillar to be designed, pillars can be:-

1.) Support pillar
2.) Protective pillar
3.) Control pillar.

Support pillar → While designing purpose of structure should be kept in mind and should be taken into consideration. The pillar has the objective that while mining in some area the pillar has to take the additional load induced due to excavation till we are working in the area and then they should fail.

ex.) working pillar in a panel.

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Control pillars → To have a control on the damage that is likely to be developed due to mining.

ex.) Suppose we have a railway line passing through the surface. While mining the ore, the overlying strata is going to be damaged. On seeking permission there is a condition that there should ~~only~~ be no adverse effect on the safety of ~~surface~~ the railway line. We have to understand the engineering parameter that will affect the safety of the railway line. We will come to known that it is the tensile strength that will affect the railway. So we will plan for a design such that the tensile ~~strength~~ stress induced by subsidence is less that the max. tensile ~~allowable~~ ~~subsidence~~ strength ~~in the area~~ of the rockmass and the subsidence is less than the maximum subsidence allowed.

Safety pillar → pillar which is required ~~the~~ to ~~secure~~ separate the area which is potentially dangerous due to the environmental or other condition. We do not want the danger to propagate to the other part where we are working.

ex.) 1.) Pillar required to separate old workings from main active working.

2.) Shaft pillar left around the shaft column for its ~~self~~ support requirement

3.) ~~Tos~~ To separate areas which are facing problem of water inundation

4.) To separate areas ~~a~~ affected by fire in the mine.

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Support pillars:-

- Includes ~~all~~ situations where the main support to the undermined hanging wall is provided by a system of pillars, usually laid out in a systematic manner.

- Being employed in one or other form to extract a great variety of minerals such as coal, chrome, potash, iron ore etc.

- Aim is to minimize the ore locked up in pillars to a level commensurate with the safety of the mine.

⇒ on developing we are trying to ~~developing~~ commensurate with a minimum size of pillar given by CMR. But recovery of ore is less so we extract more coal from the pillars but later on the ~~rep~~ there can be ~~over~~ premature collapse of the pillars so we have to improve recovery but not at the cost of safety

Protective pillars:-

- The purpose is to protect either surface structures or underground mining excavations or to separate ore mine from another.

- The main consideration is to ensure the integrity of structure to be protected.

- No consideration of pillar strength is involved in determination of dimension of these pillars, but

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- attention is given to the stresses and displacement, induced by mining in the rock in the immediate vicinity of structures for which protection is intended.

Control pillars:-

- Control pillars are usually of such geometry that their width/height ratio is well in excess of 10.

- Such pillars usually do not fail provided the rockmass above and below them is able to provide a ~~nec~~ necessary constraint.

Mode of failure of pillar:-

- → gradual/progressive/controlled.
- → sudden/uncontrolled

In case of gradual failure, there is ample warning in the form of pillar spalling and slabbings can be arrested. It is said to be a controlled failure.

Sudden failure is not necessarily preceded by detoriation of the pillars. Once the process has started, it can not be controlled. Hence considered as uncontrolled failure.

Limitation → fails to consider the fact that smaller size pillar yield and this will result in transfer of the load to the bigger size of pillars, thus overestimating the smaller size pillar load.

- In a soft rock bed is lying in the immediate vicinity i.e. roof and floor, the strength of pillar reduces drastically, because it hardly gives any confinement to the pillar.

Assignment – 3.

Q) Explain the influence of higher W/h ratio on the stability and the failure mechanism of the pillar.

Pillar Strength (CMRI formula):-

$$S_p = 0.27\,\sigma_c\, h^{-0.36} + \left(1 + \frac{H}{250}\right)\left(\frac{w}{h} - 1\right) \text{ MPa}$$

σ_c is UCS strength of 1 inch cube coal in MPa

h → height of pillar (m)
H → cover depth (m)
w → solid width of pillar (m)

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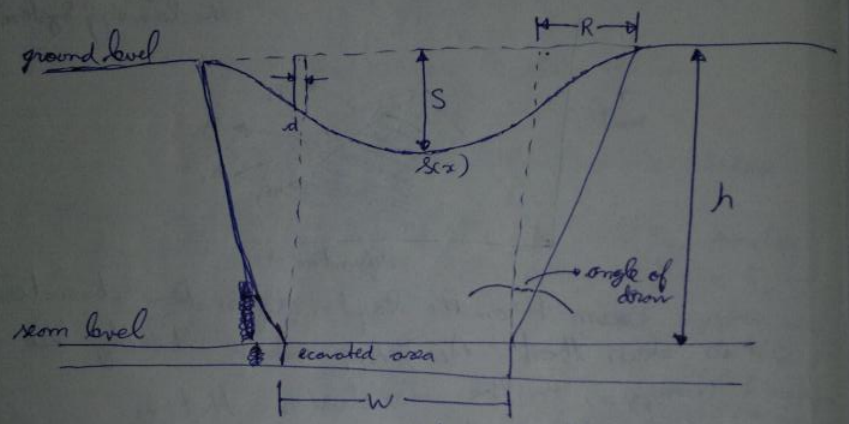
So we can conclude that with the stiffness of the pillar and hence with a higher w/h ratio we can plan for a design of the pillar with gradual or controlled failure with a higher degree of stability.

19/08/2013.

GROUND CONTROL (BKS Sir)

PROFILE FUNCTION METHOD:—



w → span of working

S → max. subsidence on the surface

h → depth of mine

R → horizontal distance between the (rib edge) edge of working to point of negligible subsidence

d → dist between transition (going from compression to tension) point to rib edge (edge of working)

s → subsidence in all other points (varies with x)

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$$s = f(S, x, R)$$

x — horizontal dist from origin

If the extraction is subcritical then another point $\left(\frac{w}{h}\right)$ also has to be taken into account.

$$s = f\left(S, x, R, \frac{w}{h}\right)$$

A profile function simply defines the shape of the curve and it requires a max. value of subsidence and a no. of constants to calculate the subsidence profile. The method is generally applied by adjusting the constants in the equation until a satisfactory shape is obtained which provides an acceptable match to the observed data.

Consequently using this calibrated equation, it should then be possible to predict subsidence curves for that particular locality.

Profile functions has 2 main advantages:-

1.) As it is defined by an equation it can easily be calculated with the help of a calculator / computer.

2.) When very little measured data are available the method can still be applied using limited data to calculate constants

Disadvantages:-

1.) The profile function shape depends on various external parameters that are not taken into account by the function and it may lead to an error in the shape.

2) The profile function may not match exactly the shape ~~given~~ of ~~total~~ that given by measured data and consequently introduces an error. Such an error may not be significant but it may give significant error when tilt or curvature are calculated from the subsidence profile.

ex) Hyperbolic function given by King and ~~Hilton~~ Whetton :-

$$S = \left(\frac{S}{2}\right)\left\{1 - \tanh\left(\frac{2x}{R}\right)\right\}$$

Based on :- Transition point origin

ex) By Kumar, Saxena and Singh (1983) (Indian profile function)

for subcritical

$$s = S e^{-\frac{m x^2}{P^2 - x^2}}$$

for critical.

$$s = S e^{-\frac{m x^4}{P^4 - x^4}}$$

$m \rightarrow$ Constant (1-5)

$x \rightarrow$ distance from extraction center

$R \rightarrow$ radius of critical area of extraction

P = half width of subsidence profile $\left(\frac{W}{2} + R\right)$

This is based on face center origin

Assignment 1:-

Search various profile function methods and explain any one profile function method.

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Design of Pillars (Prediction of width of pillar for a particular extraction height)

- This is a hit and trial method that can be easily applied for square pillars.

If we have a value of K (strength constant)

K = strength of 1 cubic inch sample

then we can easily design the pillar for a particular factor of safety (S.F.) and Extraction height (h).

$$S.F. = \frac{S_p}{\sigma_p}$$

$$S_p = K \frac{w^{\alpha}}{h^{\beta}}$$

$\alpha, \beta \rightarrow$ empirical constants.

$w \rightarrow$ solid width of pillar.

$$\sigma_p = \frac{0.025 H (w + w_b)^2}{(w)^2}$$

$w_b \rightarrow$ width of bord (gallery width)

$$S.F. = \frac{K w^{\alpha} \quad (w)^2}{h^{\beta} \times 0.025 \times H \times (w + w_b)^2}$$

$$\therefore S.F. = \frac{K (w)^{2+\alpha}}{0.025 \times H \times h^{\beta} (w + w_b)^2}$$

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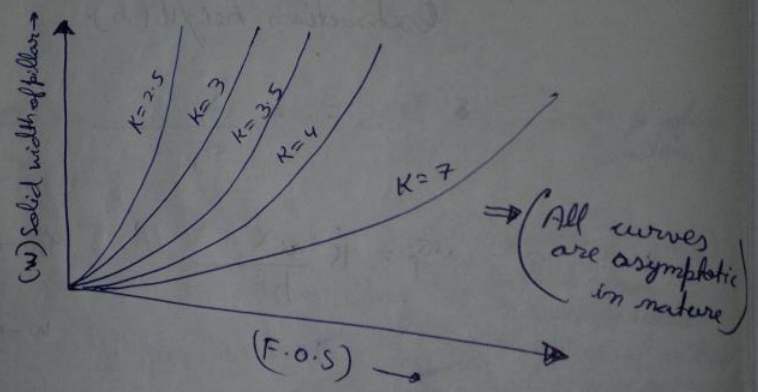
For a particular S.F. and K (strength Constant) and having the values of w_b, h, H, α, β

we would have a non-linear equation in (w) which can be solved by hit and trial values of w.

This is an easy approach to guess the the width of pillar for a fixed K (strength Constant) and S.F.

Now for the same values of K if the pillar is designed for different (F.O.S.) then we see that:



⇒ (All curves are asymptotic in nature)

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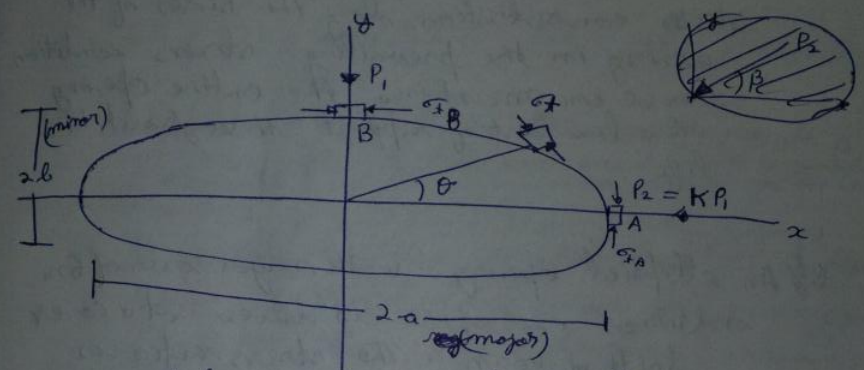
Stress around elliptical shaped tunnel:-

for an ellipse → the diameter along 2 perpendicular axis is different.



The radial stress along roof and sides is zero

P_1, P_2 → Vertical and horizontal principle stress components (if opening is horizontal)
(principle stress)

P_1, P_2 → 2 components of horizontal stress (if opening is vertical)

→ The tangential stresses at point A on the side of the elliptical shaped tunnel and at point B in the roof are:-

Closed form solution for elliptical opening.

$$\sigma_{tA} = P_1\left(1 - K + \frac{2a}{b}\right)$$

$$\sigma_{tB} = P_1\left(K - 1 + \frac{2bK}{a}\right)$$

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⇒ Some areas along the periphery may have a high induced stress concentration and may undergo failure. So such areas have to be highly supported throughout the life of the opening.

But

⇒ If we have an opportunity to ~~do~~ modify the orientation of the opening to relax the stress concentration along the sides of the opening in the prevailing stress condition then we can maintain the entire opening with a low cost of support throughout its life

Q-) An elliptical opening with major axis of 6m and minor axis of 4m is driven at a cover depth of 400m. The stress ratio is (0.4). Suggest a suitable orientation of the opening under the given circumstances.

In case o
and A
larger o

• For P_1

For K

For K=

CONC

⇒ Larger
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In case of uniaxial stress direction (assuming $K=0$) and $A/b = 2$ we have ; P_1 is normal to the longer axis of the ellipsoidal shape of the tunnel:-

$$\sigma_{tA} = 5P_1 \quad ; \quad \sigma_{tB} = -P_1 \text{ (tensile stress)}$$

• For P_1 normal to the shorter axis, they are

$$\sigma_{tA} = 2P_1 \qquad \sigma_{tB} = -P_1$$

For $K = 2$ and $a/b = 2$ $\sigma_{tA} = 3P_1$; $\sigma_{tB} = 3P_1$

For $K = 0.5$ and $a/b = 2$ $\sigma_{tA} = 4.5P_1$; $\sigma_{tB} = 0$

$\boxed{\text{CONCLUSION:-}}$ ∴ due to some confinement in the horizontal direction the stress condition at the roof can be improved.

⟹ Longer axis of the excavation should always be ~~parallel to the~~ oriented parallel to the major principal stress axis for minimizing the high stress concentration around the excavation

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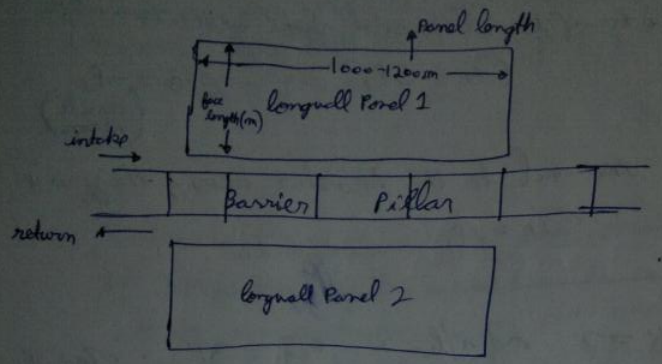
Chain Pillar:-



(90-160m) face length is much smaller compared to the width

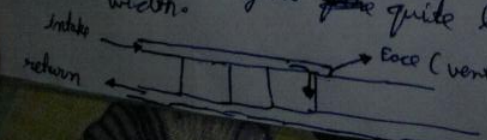
abroad { longest ~~biggest~~ longwall face = 350m
longest panel length = 3500m

Panel is developed by driving gate roads along the boundary of the Panel.

restriction { max. width of gate road = 4.8m
max. height of gate road = 3m

Due to large panel length, such long blind heading cannot be driven without ventilation so there is a requirement for splitting of the barrier pillars but (splitting should be minimum)

These pillars are called chain pillars because their length is quite large compared to their width.



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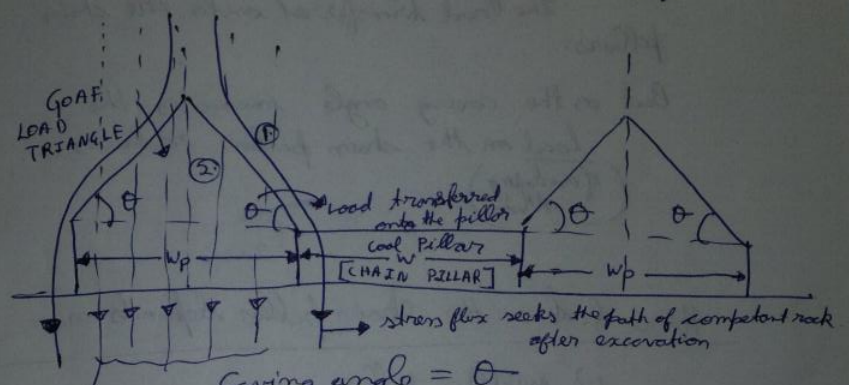
we can have:-

- Single row of chain pillar. → square / → rectangular
- Double line of chain pillar → square / → rectangular



Caving angle = θ

(direction of stress flux in unexcavated rock)

Wp → width of pillar + width of gate road

Rockmass 1 is very very compacted.

Rockmass 2 is very loose rock.

so capability to accomodate stress is more of material 1 than material 2 due to looseness of material 2

So majority of the load on the goaf material is shared by the coal pillar but not entirely.

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⇒ Hence the chain pillar is ~~an~~ more loaded than development pillars in B and P mining due to the large width of the panel.

We've consider the Caving angle → AB

the caving angle reduces the overhang of the goaf roof is more so more is the load transferred onto the chain pillars.

But as the caving angle increases the load (of overlying strata) on the chain pillar reduces.

Hence load on the chain pillar depends on:-

1.) width of pillar
2.) width of Panel
3.) Cover depth of seam
4.) Caving angle of the goaf.

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⇒ All formulations in Ground control taught are analytical formulas because we want to solve a complex problem by a simpler approach. So analytical formulas has been made on some idealizations which cause some limitations in the analytical formulas

⇒ Formulaes which are derived from field observations are called Empirical formulaes

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Topics:-

25/07/2013.

1.) Design of Mine openning
2.) Design of pillars
3.) Rock Support
4.) Caving
5.) Subsidence
6.) Slopes
7.) Rock Burst
8.) Load and Deformation Monitoring

Mine opening:-

Any opening, excavation or structure above or below the ground is a disturbance (from a ground control point of view) to the rock mass. This disturbance is significant.

1.) First type disturbance in rocks is the disturbance of insitu stress flux. in the original orientation

~~Depending on the extent of disturbance w.r.t. the ability of rock mass to resist the disturbance~~

Mine opening
- narrow
- wide

These types behave totally differently. wide is less stable than narrow openning.

The terms have nothing to do with size. They are influenced by structure of rocks in which the openings are made.

The same opening can be narrow in one type of rockmass and wide in another type of rockmass.

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In Case of Single openings:-

narrow opening is that type of opening which is not influenced by the presence of joint structure existing in the rockmass. In contrast to this. In case of wide opening the stability or performance of the opening is essentially affected by the presence of joint structure already existing in the rockmass.

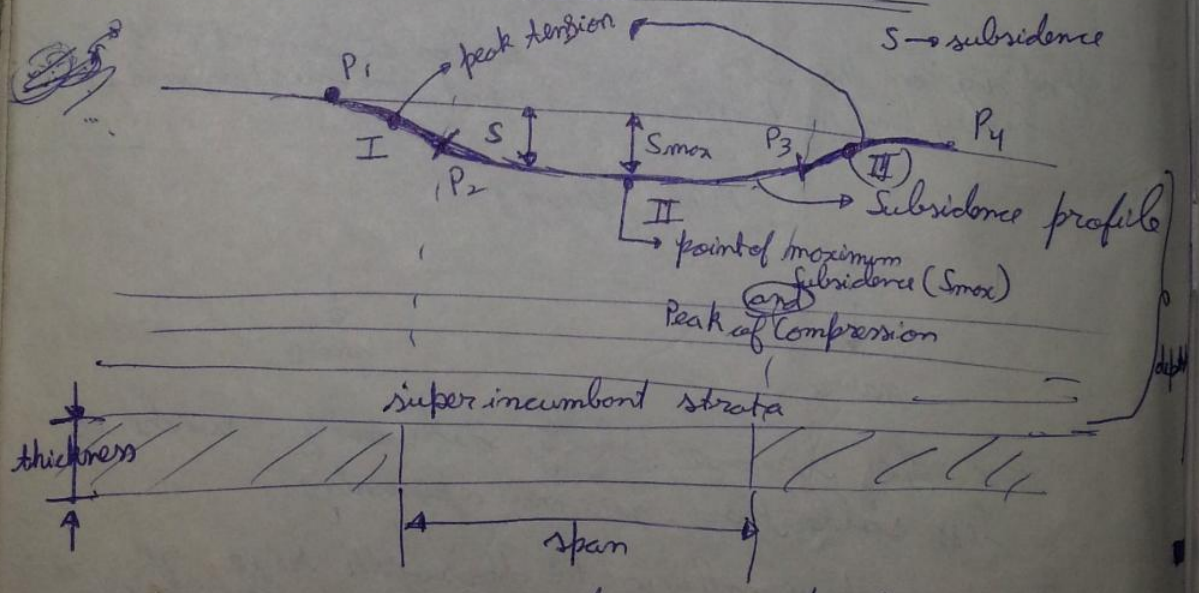
In case of multiple openings:-

an opening is treated as narrow opening only when it is neither affected by the presence of joint structure already existing in the rockmass or by the presence of other structure that has been introduced by mining operations.

If one structure affects the other structure presence then it is wide opening.

29/07/2013

Mine Subsidence (BKS Sir)



The ground movement of the surface (vertical or horizontal) is called mine subsidence

This profile is made up of three curves. (subsidence profile)

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When seen from above:-

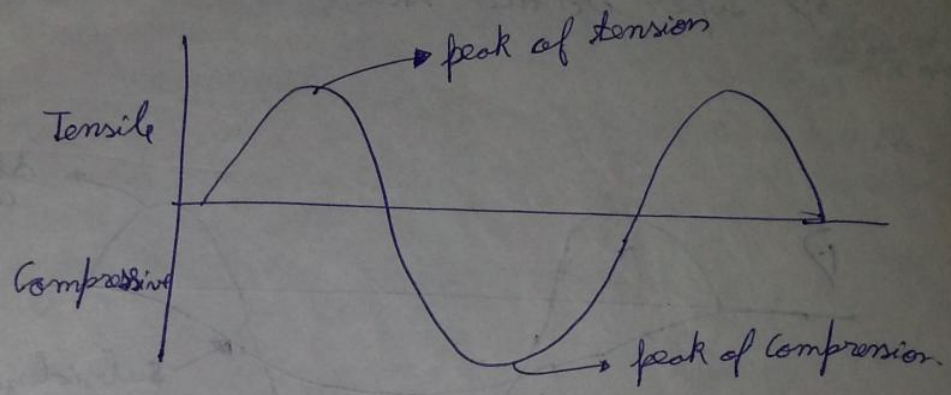
(I) → Convex curve

(II) → Concave curve

(III) → Convex curve

(III), (I) → tensile strain

(II) → Compressive strain



P_1, P_2, P_3, P_4 → Inflexion points of zero strain

Rock is very weak in tension and peak of tension lies outside the panel area so damage is seen more outside the panel area than within the panel area.

The inflexion points P_2, P_3 where there is change from tensile to compressive strain lie directly above panel and slightly inside or slightly outside the compressive strain curve.

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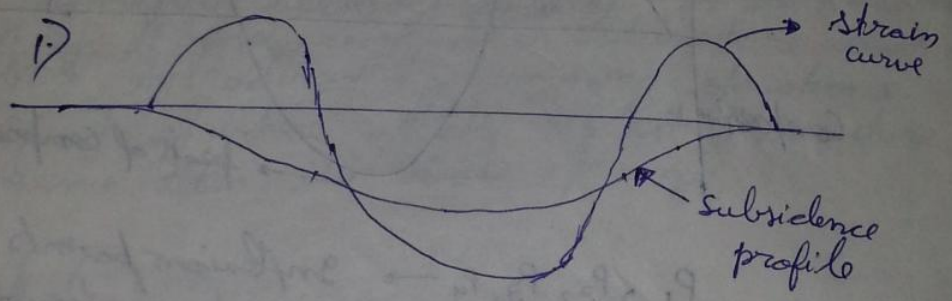
⟹ when span is increased subsidence increases and the limit of subsidence is the seam thickness but it is never the case.

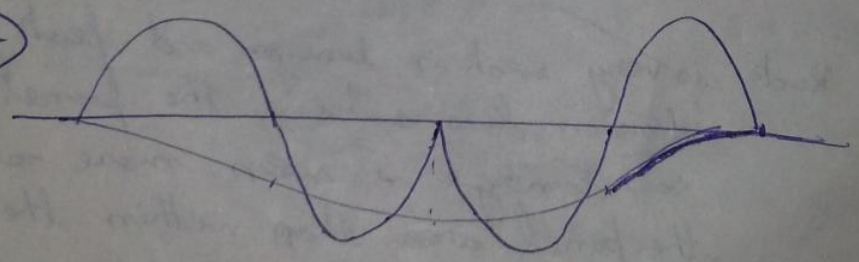
Classification of Subsidence:—

1.) Sub-critical subsidence
2.) Critical subsidence
3.) Super critical subsidence

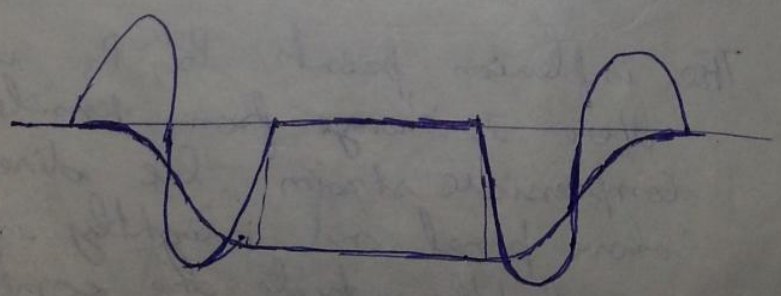
1.)



2.)



3.)



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(GSPS Sir) 01/08/2013

Stresses around opening

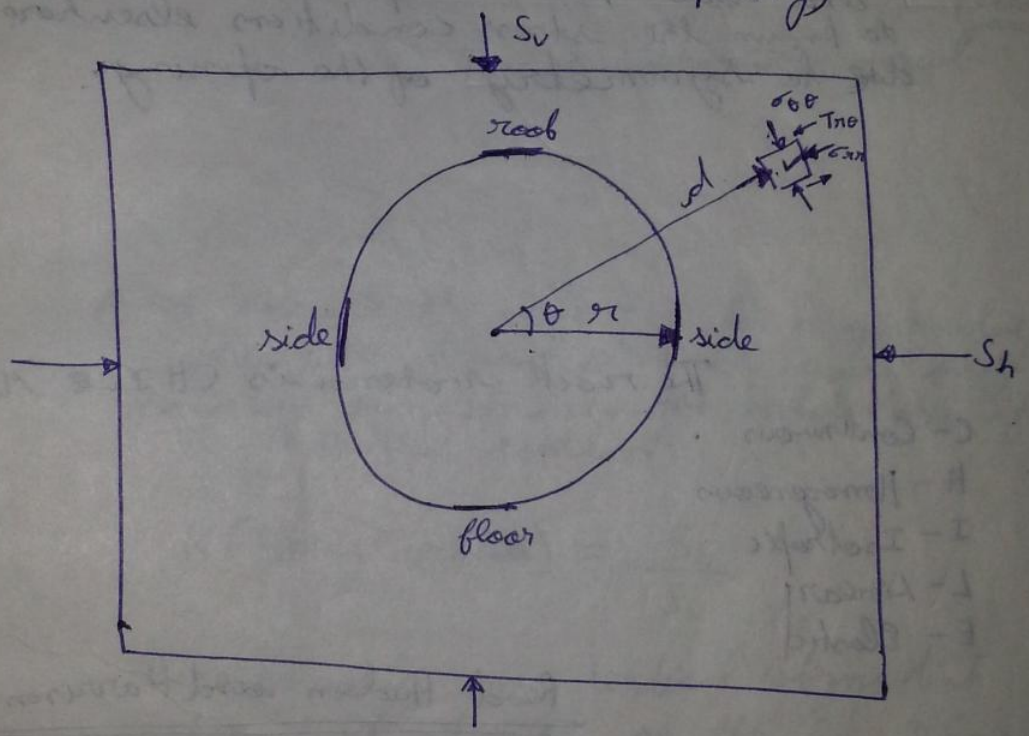
Circular opening

Driven in a rockmass with a vertical stress S_v and horizontal S_h.

S_v and S_h are far field stress (stress are acting at infinite)

Stress → far field stress (is that stress which does not have any influence due to the opening) (disturbance)

→ near field stress (stress which has been influenced by the opening)



Both S_v and S_h are compressive in nature

The radius of the opening which demarcates the size is r

we are trying to evaluate the influence of the opening on a rockmass at a distance d from the center of the opening

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we are calculating that due to the introduction of the opening to what extent the stress have been affected.

We have 3 components of stress:-

$\sigma_{\theta\theta}$ → Tangential stress

σ_{rr} → radial stress

$\tau_{r\theta}$ → shear stress

→ Rather than focussing on the entire periphery of the opening we can focus on roof and side and extrapolate the results to know the stress conditions elsewhere due to symmetry of the opening.

The rock material is CHILE Material.

C - Continuous

H - Homogeneous

I - Isotropic

L - Linear

E - Elastic

Read Hudson and Harrison Vol-I

Kirsh Solution equations

$$\sigma_{rr} = \left(\frac{S_h + S_v}{2}\right)\left(1 - \frac{r^2}{d^2}\right) + \left(\frac{S_h - S_v}{2}\right)\left(1 - \frac{4r^2}{d^2} + \frac{3r^4}{d^4}\right)\cos 2\theta$$

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$$\begin{cases} \sigma_\theta = \left(\frac{S_h + S_v}{2}\right)\left(1 + \frac{r^2}{d^2}\right) - \left(\frac{S_h - S_v}{2}\right)\left(1 + \frac{3r^4}{d^4}\right)\cos 2\theta \\ \tau_{r\theta} = \left(\frac{S_h - S_v}{2}\right)\left(1 + \frac{2r^2}{d^2} - \frac{3r^4}{d^4}\right)\sin 2\theta \end{cases}$$

These set of formulae are known as closed form solution for studying stresses around a circular material.

Minimum value of $d/r = 1$ because for $d < r$ the area is a void/constructed area.

There is not rock material.

As we increase the value of d for a given value of r, the element will be going away from the tunnel into the rockmass if θ is kept constant.

$$K \text{ (Stress ratio)} = \frac{S_h}{S_v}$$

There is only one vertical stress but 2 horizontal stresses so the S_h we are considering here is the mean horizontal stress.

$$\left(S_h = \frac{S_{h\,max} + S_{h\,min}}{2} \right)$$

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$\underbrace{S_v}_{MPa} = \gamma H$ → cover depth (m)

γ → unit weight of rock. MPa/m

$\boxed{Pa/m \equiv N/m^3}$

γ → the amt of the weight the ~~ground~~ rock ~~will have~~ having 1 m^2 surface area of unit thickness

$\gamma = 0.025$ for practical purposes but depends on the density of rock.

we can have value of $\underbrace{K > 1}_{\text{horizontal stress} > \text{Vertical stress}}$ or $\underbrace{< 1}_{\text{horizontal stress} < \text{Vertical stress}}$ or $K = 1$

If this condition is encountered then while designing it has to be taken into consideration. This is a very special case. Otherwise it will create safety and maintainance problems throughout the life of the tunnel.

if $S_v = P$ then $S_h = KP$

$\frac{\sigma_r}{P}$, $\frac{\sigma_\theta}{P}$, $\frac{\sigma_z}{P}$

This gives us the extents of intensification of stress concentrations

if ratio $> 1 \Rightarrow$ intensified

if ration $< 1 \Rightarrow$ reduction in concentration

Lab	K value
1	0.1
2	0.2
3	0.5
4	0.7
5	0.8
6	1
7	1.2
8	1.4
9	2
10	2.5
11	3
12	5

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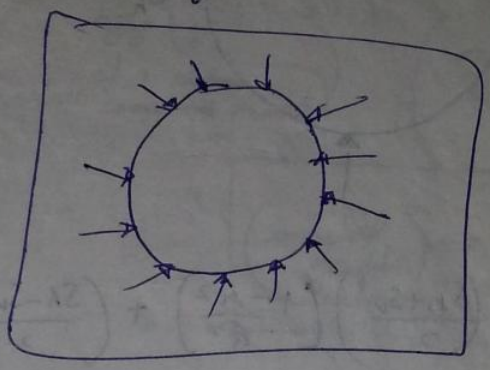
when K = 1 → This type of stress condition is known as hydrostatic stress condition.

if K = 1 then the amount of Far field stress that the opening is subjected to is equal.



Lab	K values
1	0.1
2	0.2
3	0.5
4	0.7
5	0.8
6	1
7	1.2
8	1.4
9	2
10	2.5
11	3
12	5

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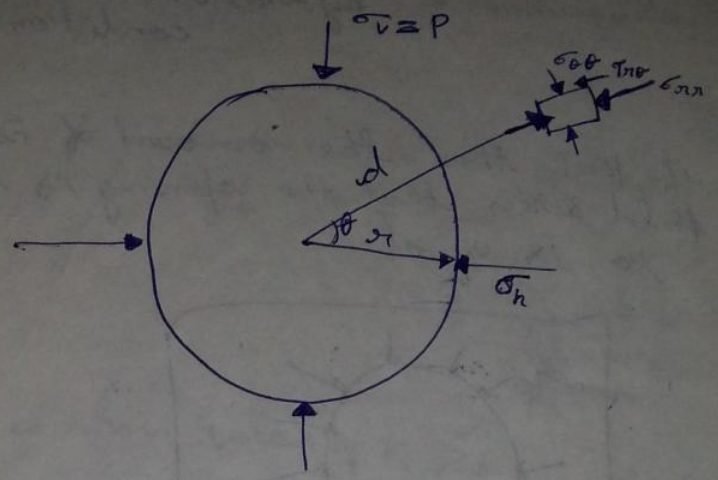
02/08/2013

GSPS Sir



$$\sigma_{rr} = \left(\frac{S_h+S_v}{2}\right)\left(1-\frac{r^2}{d^2}\right) + \left(\frac{S_h-S_v}{2}\right)\left(1-\frac{4r^2}{d^2}+\frac{3r^4}{d^4}\right)\cos 2\theta$$

$$\sigma_{\theta\theta} = \left(\frac{S_h+S_v}{2}\right)\left(1+\frac{r^2}{d^2}\right) - \left(\frac{S_h-S_v}{2}\right)\left(1+\frac{3r^4}{d^4}\right)\cos 2\theta$$

$$\tau_{r\theta} = \left(\frac{S_h-S_v}{2}\right)\left(1+\frac{2r^2}{d^2}-\frac{3r^4}{d^4}\right)\sin 2\theta$$

Case - I :-

If $S_h = S_v$ (Hydrostatic condition)

$$\sigma_{rr} = S_v\left(1-\frac{r^2}{d^2}\right)$$

$$\sigma_{\theta\theta} = S_v\left(1+\frac{r^2}{d^2}\right)$$

$$\tau_{r\theta} = 0$$

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if $r = 2m$ $\quad 2 \leq d \leq 10$

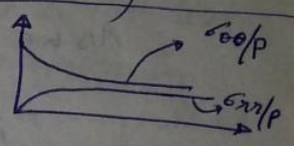
at $\theta = 0$

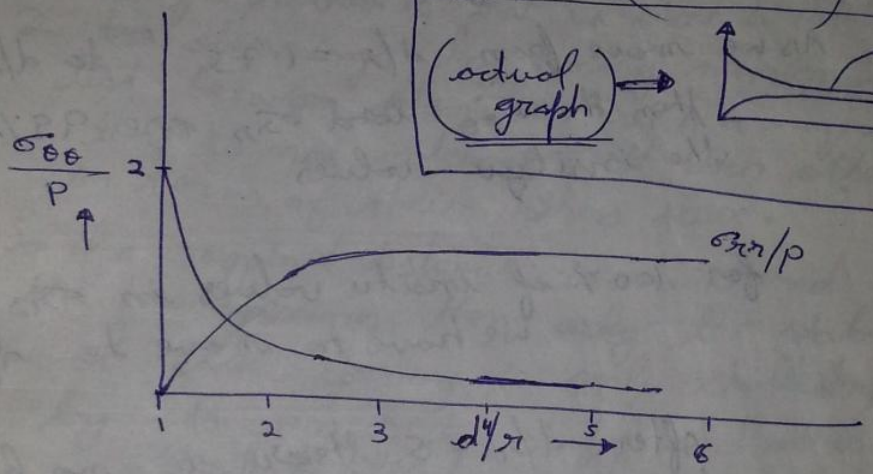
	d=2	d=4	d=6	d=8	d=10
$\sigma_{\theta\theta}$	2P	1.25P	1.111P	1.0625P	1.04P
σ_{rr}	0	0.75P	0.889P	0.9375P	0.96P

($\sigma_{\theta\theta}$ (↓) at a ~~non~~ variable ~~constant~~ rate)

(σ_{rr} (↑) at a variable rate)

(actual graph) ⇒





$\frac{d}{r}$ → indicates how far our rock element is from the periphery of the tunnel.

⇒ The results are independent of the value of θ

At the surface :-

⇒ radial stress is zero at all points on the surface

⇒ tangential stress is 2 times the virgin stress (i.e. the vertical stress)

⇒ In hydrostatic condition only.

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As we go into the rockmass the value of $\sigma_{\theta\theta}$ decreases but with a variable rate and the value of σ_{rr} increases with a variable rate

As we exceed the points of $d/r = 1.75$ the value of $\sigma_{\theta\theta}$ or σ_{rr}, then they are achieving almost the insitu value.

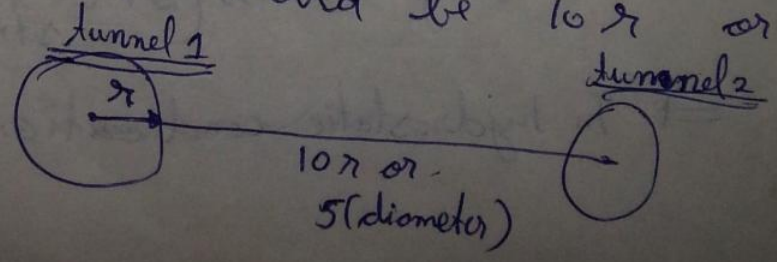
As we move from $d/r = 1.75$ to $d/r = 3$ then the $\sigma_{\theta\theta}$ and σ_{rr} are 99% of the insitu values

for 100% of insitu values in σ_{rr} and $\sigma_{\theta\theta}$ we have to move to $d/r = 5$

After $d/r = 5$ there is no further changes of saturation in the values of σ_{rr} and $\sigma_{\theta\theta}$

In Hydrostatic conditions only:-

If we want to have two companion excavations which remain in proximity of each other without any influence on each other then the minimum distance between the two should be $10r$ or 5 (diameter)



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Zone of influence → the dist away from the surface of the structure upto which the original state of stress is likely to be disturbed.

The Concept of Zone of influence is very important for mine design because it may provide considerable simplification of a design problem.

Our design problem is that we want to establish 2 ~~two~~ openings by having minimum dist. between them in such a way that they do not interact with each other in terms of their stress flux.

So if a required distance is maintained between the two openings then ~~only~~ the stability of each opening is only challenged by its own size, shape and other attributable parameters and not by the size or shape of the nearby opening. Then this design problem will become simple.

➡ Other mining structures are not simple openings because they have influence on each other.

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Zone of influence defines the domain of significant disturbance of the pre-mining stress field by an excavation. It differentiates between the near field and far field stress of an opening. For hydrostatic stress condition it is observed that the state of stress is not significantly disturbed beyond a distance $d = 5r$. Therefore if two openings are kept at a spacing of 10r it is possible to maintain those openings without their mutual interaction. Zone of Influence can be a handy tool for design of simple openings.

05/08/2013.

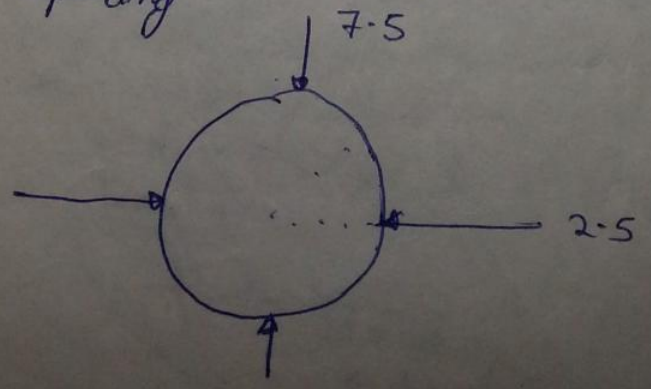
GROUND CONTROL (BKS Sir)

08/08/2013.

GROUND CONTROL (SPS Sir)

Estimation of zone of failure :- (IInd assignment)

Q: A circular tunnel of 4 m diameter is driven in CHILE rockmass under field stress as given in diagram below. The Compressive strength of the rockmass is 16 MPa. Estimate the zone of failure along periphery of the opening



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+ve stress → compressive
-ve stress → tensile } always.

Ist assignment:-

Using Closed form solution available for circular opening to establish the fact that stresses induced along the periphery of the opening does not depend on the size of the opening or the elastic constant of the rockmass.

DESIGN OF PILLARS:-

In U/G working we have to have a form of pillar designed for a purpose.

Depending on purpose of pillar to be designed, pillars can be:-

1.) Support pillar
2.) Protective pillar
3.) Control pillar.

Support pillar → While designing purpose of structure should be kept in mind and should be taken into consideration. The pillar has the objective that while mining in some area the pillar has to take the additional load induced due to excavation till we are working in the area and then they should fail.

eg.) working pillar in a panel.

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Control pillars → To have a control on the damage that is likely to be developed due to mining.

ex.) Suppose we have a railway line passing through the surface. While mining the ore, the overlying strata is going to be damaged. On seeking permission there is a condition that there shall be no adverse effect on the safety of the railway line. We have to understand the engineering parameter that will affect the safety of the railway line. We will come to know that it is the tensile strength that will affect the railway. So we will plan for a design such that the tensile stress induced by subsidence is less that the max. tensile strength of the rock mass and the subsidence is less than the maximum subsidence allowed.

Safety pillar → pillar which is required to separate the area which is potentially dangerous due to the environmental or other condition. We do not want the danger to propogate to the other part where we are working.

ex.) 1) Pillar required to separate old workings from main active working.

2.) Shaft pillar left around the shaft column for its support requirement

3.) To separate areas which are facing problem of water inundation

4.) To separate areas affected by fire in the mine.

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Support pillars:-

→ Includes ~~all~~ situations where the main support to the undermined hanging wall is provided by a system of pillars, usually laid out in a systematic manner.

— Being employed in one or other form to extract a great variety of minerals such as coal, chrome, potash, iron ore etc.

— Aim is to minimize the ore locked up in pillars to a level commensurate with the safety of the mine.

⇒ On developing we are trying to ~~developing~~ commensurate with a minimum size of pillar given by CMR. But recovery of ore is less so we extract more coal from the pillars but later on the ~~rep~~ there can be ~~over~~ premature collapse of the pillars so we have to improve recovery but not at the cost of safety

Protective pillars:-

— The purpose is to protect either surface structures or underground mining excavations or to separate one mine from another.

— The main consideration is to ensure the integrity of structure to be protected.

— No consideration of pillar strength is involved in determination of dimension of these pillars, but

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- attention is given to the stresses and displacement, induced by mining in the rock in the immediate vicinity of structures for which protection is intended.

Control pillars:-

- Control pillars are usually of such geometry that their width/height ratio is well in excess of 10.

- Such pillars usually do not fail provided the rockmass above and below them is able to provide a ~~the~~ necessary constraint.

Mode of failure of pillar:-

- → gradual/progressive/controlled.
- → sudden/uncontrolled

In case of gradual failure, there is ample warning in the form of pillar spalling and slabbings can be arrested. It is said to be a controlled failure.

Sudden failure is not necessarily preceded by detoriation of the pillars. Once the process has started, it can not be controlled. Hence considered as uncontrolled failure.

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- The controlled and uncontrolled pillar collapses are manifestation of more general stable and unstable phenomena.

Sol: ~~to~~ Assignment 1 :-

Along the periphery of a circular opening of the closed form solutions can be formulated as:-

$r/d = 1$ along the periphery.

The induced stresses are:-

$$\sigma_{rr} = \left(\frac{S_h + S_v}{2}\right)(1 - 1^2) + \left(\frac{S_h - S_v}{2}\right)\left(1 - 4(1)^2 + 3(1)^4\right)\cos 2\theta$$

$$= 0$$

$$\tau_{r\theta} = 0$$

$$\sigma_{\theta\theta} = \left(\frac{S_h + S_v}{2}\right)(1 + (1)^2) - \left(\frac{S_h - S_v}{2}\right)(1 + 3(1)^4)\cos 2\theta$$

$$= (S_h + S_v) - 2(S_h - S_v)\cos 2\theta$$

Putting $S_v = \gamma H$ and $S_h = K\gamma H$

$$\sigma_{\theta\theta} = \gamma H\left[(1+K) + 2(1-K)\cos 2\theta\right]$$

(i) Dependence of induced stresses on size of the opening:-

σ_{rr} and $\tau_{r\theta}$ are 0 irrespective of the size of the tunnel and $\sigma_{\theta\theta}$ has no parameters which

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takes into consideration the size of the tunnel. Hence we can conclude that the induced stresses σ_{rr}, $\tau_{r\theta}$ and $\sigma_{\theta\theta}$ are independent of size of the opening.

(ii) Dependence of Induced stresses on elastic property of the rockmass :-

σ_{rr} and $\tau_{r\theta}$ are zero and hence their values does not depend on the property of the rockmass in which the opening has been excavated.

$\sigma_{\theta\theta}$ is a function of γ, H and K. where,

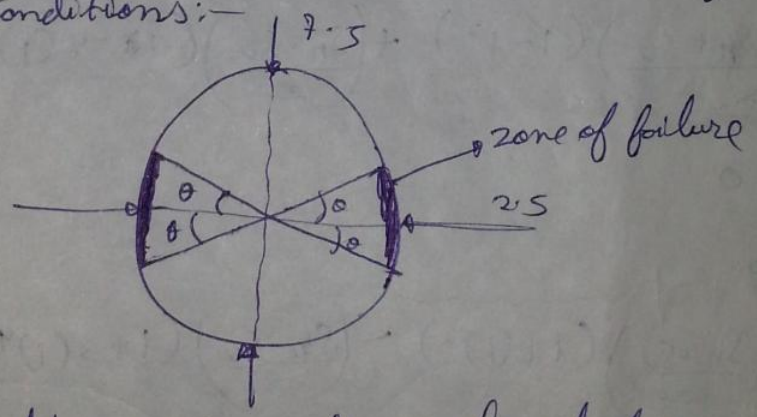
γ = weight of rockmass of surface area $1m^2$ and depth 1m (MPa/m). Hence it is independent of any elastic constants of the rockmass.

K = stress ratio is a constant.

Hence $\sigma_{\theta\theta}$ is also independent of the elastic properties of the rockmass in which opening is made.

Sol. to Assignment 2 :-

The following calculation support the above diagramatic representation of the zone of failure of the circular opening under given conditions :-



$d/r = 1$, Hence from closed form solution we have :-

$\sigma_{rr} = 0$ and $\tau_{r\theta} = 0$

and $\sigma_{\theta\theta} = 10 + 10\cos 2\theta$

Zones of failure exist for the following inequality :-

$$10 + 10\cos 2\theta \geq 16$$

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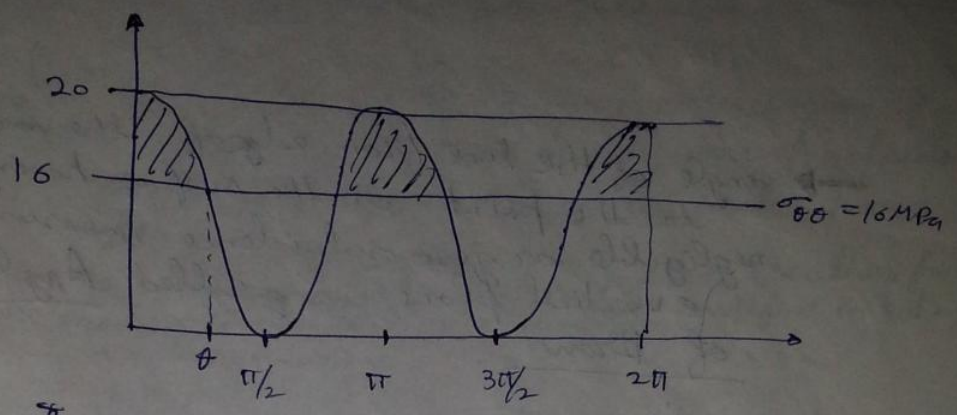
The graphical depiction of the above inequality is shown below:-



The zone of failure of the circular opening lie in the following ranges of θ:-

$$0 \leq \theta \leq 26°33'54''$$

$$(\pi - 26°33'54'') \leq \theta \leq (\pi + 26°33'54'')$$

$$(2\pi - 26°33'54'') \leq \theta \leq 2\pi$$

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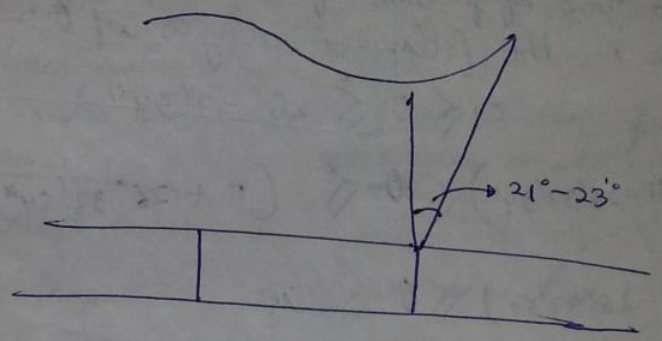
12/08/2013. BKS Sir (GROUND CONTROL)

→ angle of the line from edge of the working to the point on the surface having negligible or zero subsidence measured in a vertical plane is called Angle of Draw



Q.) Draw 2 figures (1 profile, 1 strain) as the span is increasing from sub critical to super critical area.

~~1) Profiles:~~

~~A.) (a.) Subcritical.~~

~~(b.)~~

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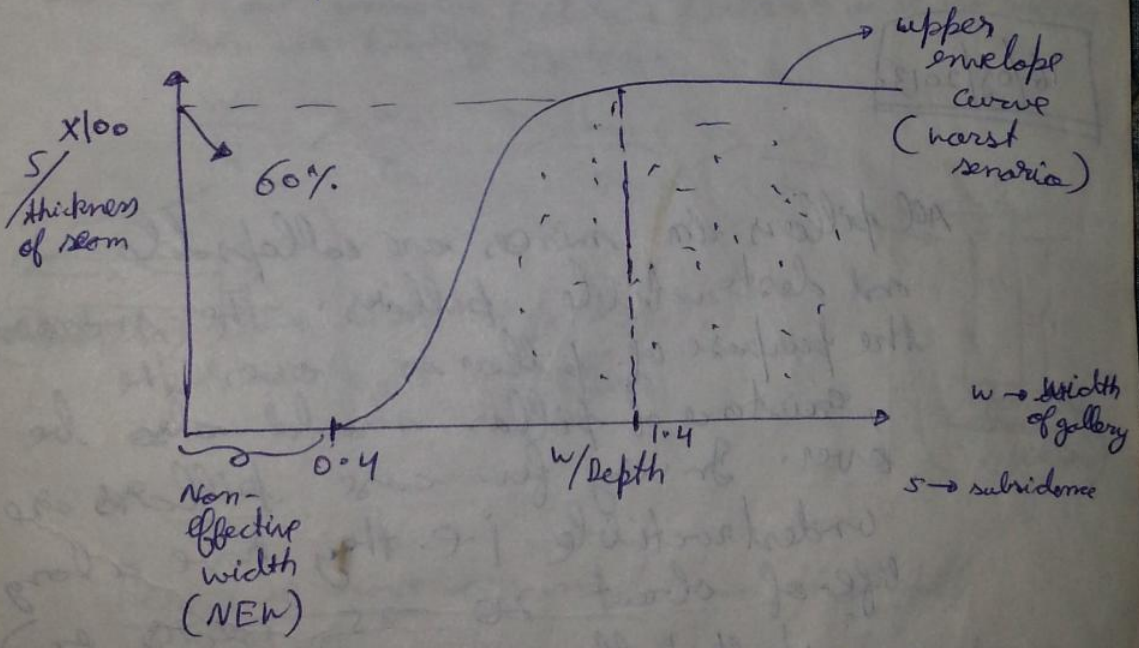
Prediction of Mine Subsidence :-

→ During Mining there is always percussion on the surface. So we have to take care of subsidence so that surface features (roads, rails, houses etc) are protected.

→ Due to subsidence cracks develops and air breathes from these cracks and coal seam gets oxidized and fire is set up.

(i) depth → less then more subsidence
(ii) span → more then more subsidence
(iii) thickness → more thickness of seam then more subsidence

→ subsidence depends on these parameters



if W/d < 0.4 → then ~~subsi~~ subsidence is almost negligible.

if W/d > 1.4 → then supercritical subsidence takes place

~~we can~~ if we know that there is to be super critical subsidence then we can plan accordingly. In supercritical there is no cracks as strain becomes zero.

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This the empirical method based on field data. The upper envelope curve gives the idea of NEW.

At 60% of $\frac{\text{subsidence}}{\text{thickness}}$ ratio we have the maximum subsidence (supercritical)

But by this method no relationship could be developed between subsidence and (depth, span, thickness)

Profile function method:-

Next Class.

16/08/2013.

All pillars in mines are collapsable and destructible pillars. The moment the purpose of pillar is over the existance of pillar should also be over. In very few case pillars are undestructible i.e. they have a long life of about 20 – 25 years ex. shaft pillar. There is complexity in designing a pillar which undergoes a failure after certain time interval.

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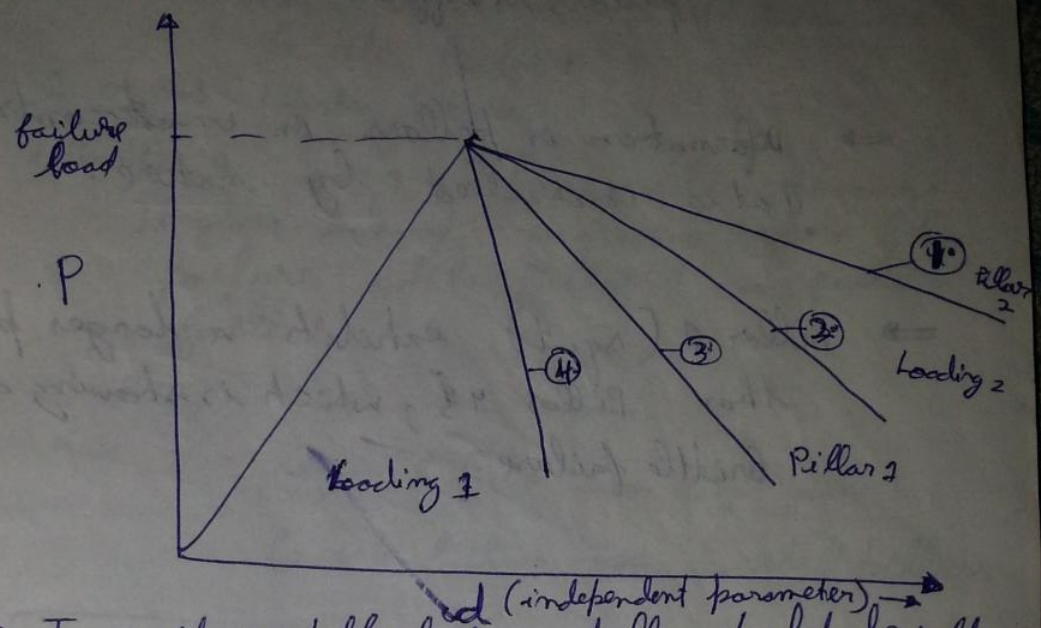
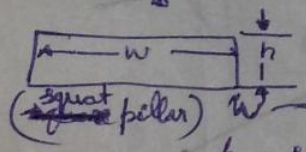
Load deformation behaviour :-



→ To avoid unstable failure, pillar should be softer than the loading system.

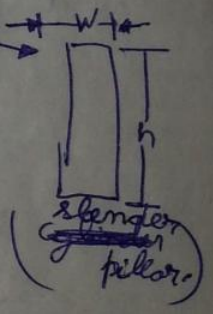
$$\frac{w_1}{h_1} > \frac{w_2}{h_2} > \frac{w_3}{h_3} > \frac{w_4}{h_4}$$



(~~square~~ pillar)

w → width of pillar.

h → height of pillar.



Here we have assumed → that all pillars have approximately ~~have~~ the same failure load.

→ We are studying the post failure behaviour of the pillars.

NOTE:- deformation (d) although an independent parameter is a uncontrollable parameter

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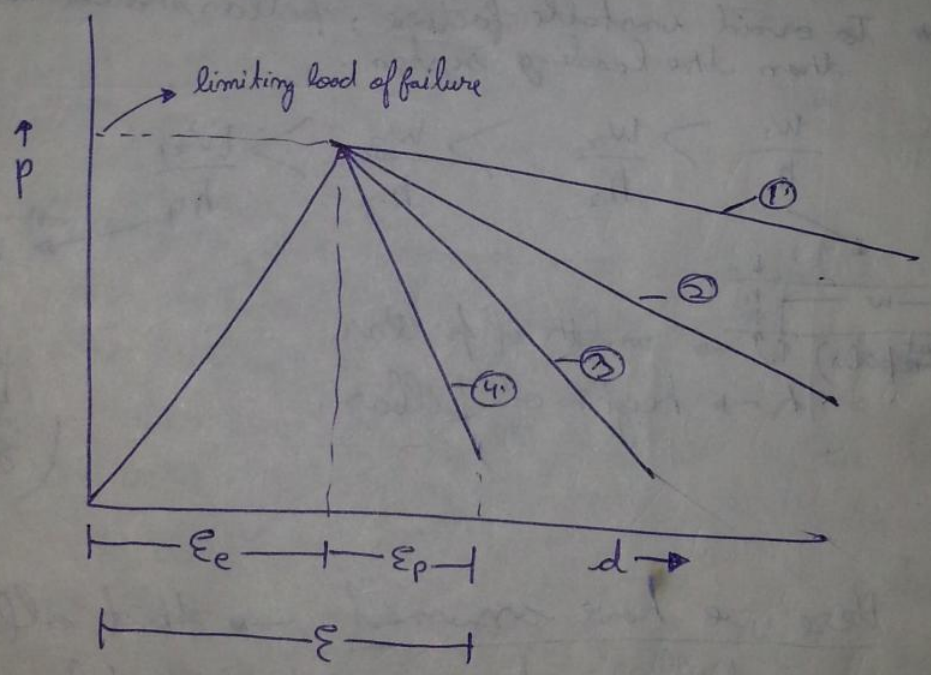
OBSERVATIONS FROM GRAPH :-

⇒ ~~Load~~ For a particular post failure deformation, the load bearing capacity of different pillars is different.

⇒ Deformation in pillars is uncontrolled. That is controlled by nature.

⇒ Pillar 1 (squat) exhibits a larger plasticity than Pillar 4, which is showing a brittle failure.



ε_e → elastic strain

ε_p → plastic strain

ε → total strain

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In pillar 4:-

$\varepsilon_R > \varepsilon_P$ so it shows ~~it~~ has more elastic nature and its failure is brittle failure

→ Pillar 1 shows plastic failure

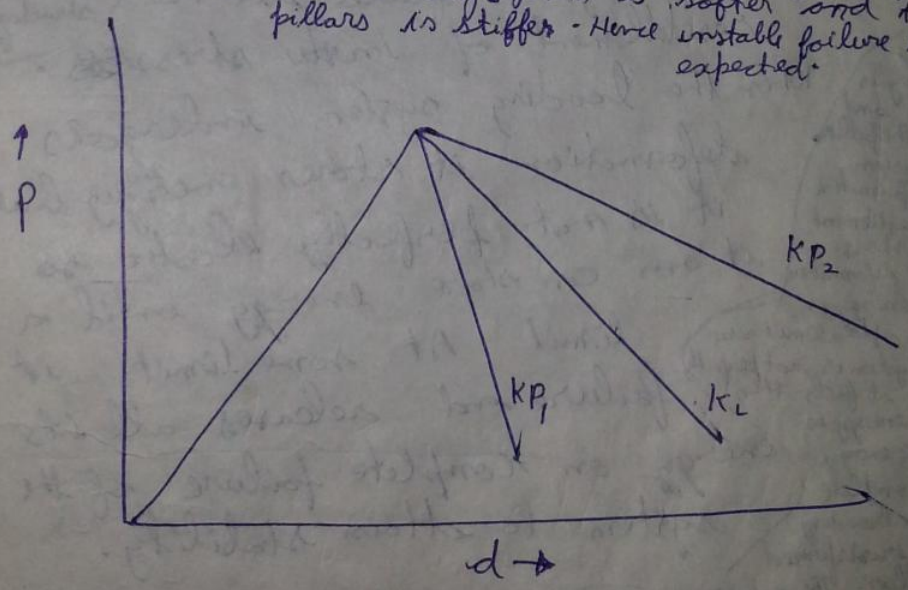
⇒ pillar ~~2~~ 1 is most favourable because it gives enough time for warning ~~before~~ before failure while pillar 4 has sudden failure.

Plastic failure → soft pillars

brittle failure → ~~to~~ stiff pillars

Large ~~Excavation~~ Extraction Cases:-

In case of large excavations the case is just opposite i.e. the loading system is softer and the pillars is stiffer. Hence unstable failure is expected.



K_L → post failure stiffness of loading system (roof and floor)

K_P → post failure ~~to~~ stiffness of pillar

NOTE:- A system is softer in nature than the other if it undergoes a larger deformation ~~for~~ by the application of the same load

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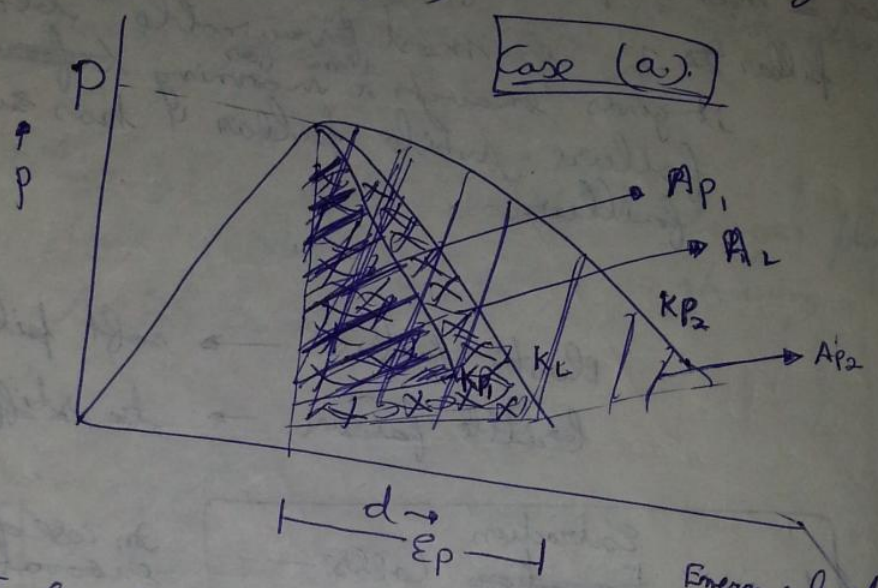
2 Cases:—

(a) $K_{P_1} > K_L$ (Pillar is stiffer than loading system)

(b) $K_{P_2} < K_L$ (Pillar is softer than loading system)

Case (a).



⟹ The loading system fails due to resettlement of insitu stress. When the loading system undergoes deformation it stores energy but it is not perfectly elastic so it can store energy until a limit. At some limit it failure and releases all its energy on complete failure of the system to attain stability.

Energy released by loading system on complete structural failure (A_L) $= \frac{1}{2} P \varepsilon_p$

The loading system undergoes deformation and this loads the pillar. The loading system undergoes deformation due to resettlement of insitu stress. By allowing deformation, it stores energy in it but only upto a limit since the loading system is not perfectly elastic. When it fails then the stored energy is released. The energy released on complete failure of the loading system is transferred to the pillar. The capability of the pillar to store the transmitted energy is A_P.

A_P = energy storing capability of the pillar. (amt. of energy that the pillar can accomodate)

A_L = energy released by the loading system post complete failure.

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$A_{P_1} < A_L$

The potential of the loading system to transmit energy is larger than the capability of the pillar to accomodate energy. So the remaining energy is released in the environment so that the loading system attains a lower energy state and attains stability. But such transition is not smooth. It is sudden and so uncontrolled pillar failure takes place. This is brittle failure.

when the loading system ~~that~~ transfers energy to the pillar and the ~~ea~~ energy storing capability of pillar is less than the loading system then the remaining energy is released and such transition is violent and sudden failure takes place.

In Case (b):-

$A_{P_2} > A_L$

The capability of the pillar to store energy is larger than the energy transferred to the pillar by the loading system. Even after ~~ear~~ the loading system transfers all the energy to the pillar there is still ample scope in the pillar to accomodate more energy. This results in gradual failure or plastic failure.

So such failure is gradual so while designing a pillar we should design a pillar ~~that~~ such that the energy storing capability of the pillar is more than the energy transferred from the loading system. ~~Sud~~ Then the failure is controlled.

Even ~~by~~ after the transfer of energy by the loading system upon failure there is still scope for accomodation ~~for~~ of more energy.

CONCLUSION:- So while design a pillar with the help of the load deformation curve we can plan for a w/h ratio for which there is a gradual failure of the pillars. Depending on the w/h ratio, the same pillar can behave as slender pillar with uncontrolled failure or the same can behave as squat pillar with controlled failure.

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In general Tributary area theory:-

Tributary area theory:-

⇒ The load of overburden is equally shared by adjoining pillars.

Limitations:-

- Estimated pillar load on the higher side

  if pillar load estimation is wrong by the theory then the rational design of pillar is not possible. And rational design is ~~not possible~~ necessary because pillar is just compromising the ore so larger size of pillar ~~than~~ required is a waste of ore.

- Is good when loading system is soft.

  soft system → for a particular load, the post failure deformation is larger for a soft system.

- In case of non uniform array of pillars, small size pillar load is overestimated and large size pillar load is underestimated.

- Hold good when the extraction area is more than the depth of mine, but overestimates the load if pillars are of non-uniform size. (loading system is softer)

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Limitation → fails to consider the fact that smaller size pillar yield and this will result in transfer of the load to the bigger size of pillars, thus overestimating the smaller size pillar load.

- If a soft rock bed is lying in the immediate vicinity i.e. roof and floor, the strength of pillar reduces drastically, because it hardly gives any confinement to the pillar.

Assignment – 3.

Q) Explain the influence of higher W/h ratio on the stability and the failure mechanism of the pillar.

Pillar Strength (CMRI formula):-

$$S_p = 0.27\,\sigma_c\, h^{-0.36} + \underbrace{\left(1 + \frac{H}{250}\right)}_{\text{It is } (1+H/160) \text{ may be.}} \left(\frac{w}{h} - 1\right) \text{ MPa}$$

σ_c is UCS strength of 1 inch cube coal in MPa

h → height of pillar (m)
H → Cover depth (m)
w → solid width of pillar (m)

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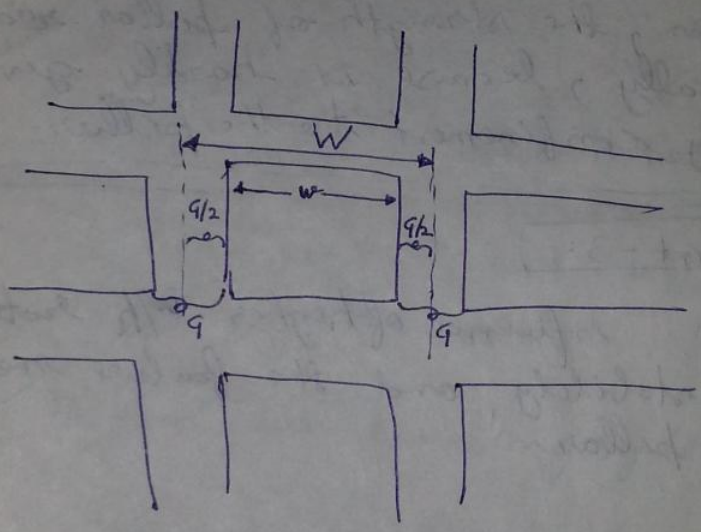
Factor affecting pillar behaviour:-

- → Load on the pillar
- → Strength of the pillar
- → Stiffness of pillar and loading system
- → Extraction ratio

Tributary area theory:-



G = width of gallery

w → solid width of pillar

W → width of pillar (center to center)

$\therefore W = w + G$

Theory → The load of the overburden is equally shared by adjoining pillars.

σ_v → vertical stress ~~on the pillars~~ of the overburden

For square pillars:-

$$\sigma_v \times \underbrace{(W \times W)}_{\text{Area of overburden to be supported by pillar}} = \sigma_p \times \underbrace{(w \times w)}_{\text{Area of solid pillar}}$$

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(B-X)

1" 55" 3' 35'

$\sigma_p \rightarrow$ Pillar load (Pillar stress)

$$\therefore \sigma_p = \sigma_v \times \frac{(W \times W)}{(w \times w)}$$

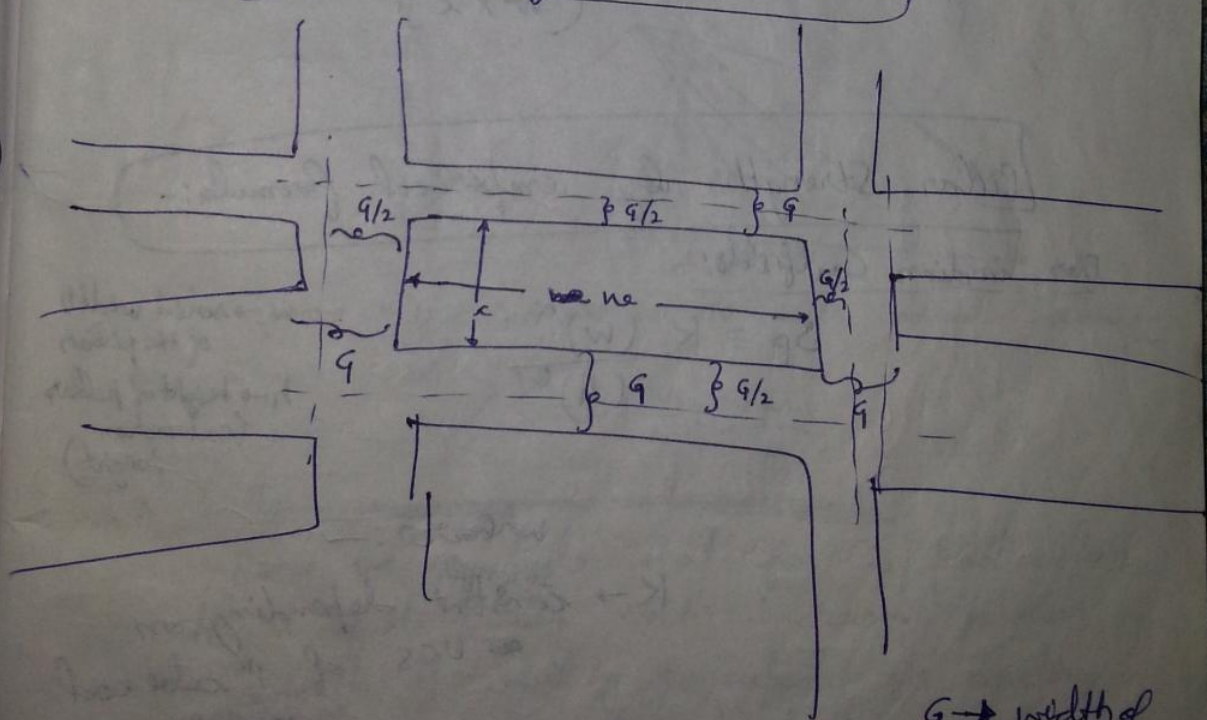
σ_v can be estimated by:-

$$(MPa)\ \sigma_v = \gamma H$$

$H \rightarrow$ Cover depth (m)

$\gamma \rightarrow$ weight density of the overburden rockmass (MPa/m).

For Rectangular pillars:-



$G \rightarrow$ width of gallery.

$w + G = W$

$c + G = C$

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$$\sigma_v \times (W \times C) = \sigma_p \times (w \times c)$$

$$\therefore \sigma_p = \frac{\sigma_v (W \times C)}{(w \times c)}$$

$$= \frac{\gamma H (W \times C)}{(w \times c)}$$

In general $\gamma = 0.025$ MPa/m

$$\therefore \sigma_p = \frac{0.025 H (W \times C)}{(w \times c)}$$

Pillar Strength by empirical formula:-

For indian Coalfields:-

$$S_p = K \frac{(w)^{\alpha}}{(h)^{\beta}}$$

w → solid width of the pillar

h → height of pillar (extraction height)

where:-

K → strength constant depending on UCS of 1" cube coal in MPa

α, β → empirical Constants

w → solid width of pillar

h → height of pillar

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For Jharia Coalfield:-

$K = 1.79$

$\alpha = 0.46$

$\beta = 0.66$

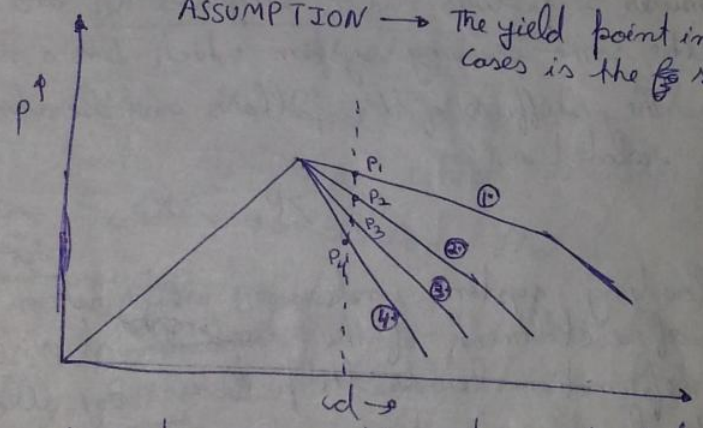
Factor of Safety (S.F.) $= \dfrac{\text{Pillar Strength}}{\text{Pillar stress}}$

$$\therefore S.F. = \frac{S_p}{\sigma_p}$$

Solution to Assignment 3:-

The load deformation curve for various values of w/h is shown below:-

ASSUMPTION → The yield point in all cases is the same.



where w/h ratio are in the order indicated below:-

$$\frac{w_1}{h_1} > \frac{w_2}{h_2} > \frac{w_3}{h_3} > \frac{w_4}{h_4}$$

~~The reason for the above is sta~~

Hence we conclude that for a particular height of the pillar, with increasing pillar width the ability to support the loading system increases for a particular post failure deformation

$$P_1 > P_2 > P_3 > P_4.$$

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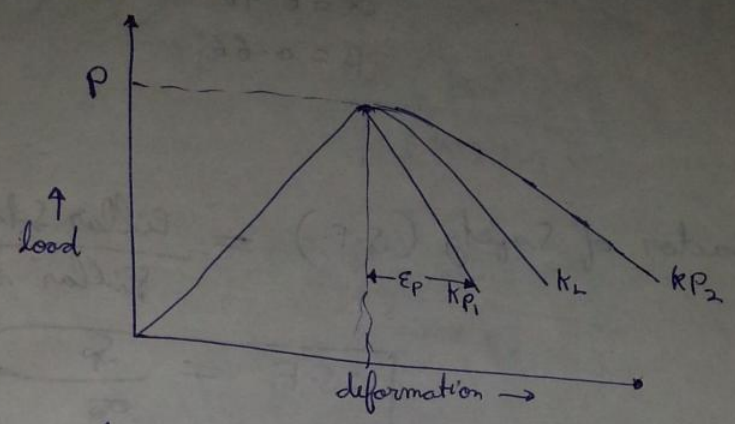
To understand the failure mechanism of the pillar the following cases may be considered.

~~Case I~~



K_L → Post failure stiffness of the LOADING SYSTEM

K_P → Post failure stiffness of the PILLAR

Consider 2 pillars with stiffness K_{P_1} and K_{P_2} under the same loading system which has a stiffness K_L. The stiffness of the pillars and loading system are related as :-

$$K_{P_1} > K_L > K_{P_2}$$

with the w/h ratio of the pillar being in the order $\frac{w_1}{h_1} < \frac{w_2}{h_2}$

The loading system undergoes deformation as result of resettlement of the ~~in situ~~ stresses and this deformation loads the pillars. By allowing deformation the loading systems stores strain energy in it but only upto the yield point (Since it is not perfectly elastic). When it fails the stored energy is released (The energy hence released is in order to attain a stable lower energy state). The energy released on complete failure of the loading system is transmitted to the pillars.

Let

A_P be the capability of the ~~pillar~~ either pillar to accomodate strainenergy.

A_L be the potential of the loading system to transfer energy

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~~Th~~ A_P and A_L can be formulated as:-

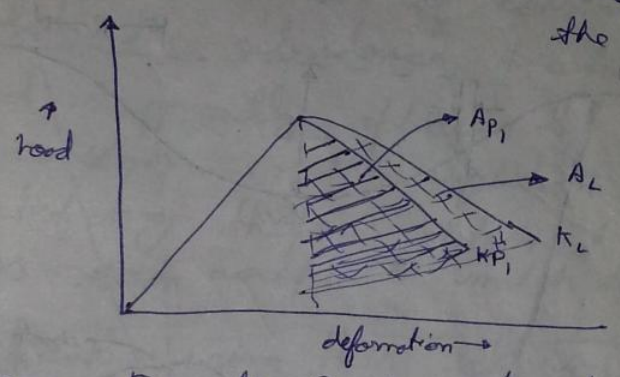
$$A = \frac{1}{2} \times P \times \varepsilon_P$$

where P = yield point load
ε_P = Plastic strain

A is the area under the post failure deformation curve.

Case 1:-

$K_{P_1} > K_L$ (i.e. the pillar is stiffer than the loading system)

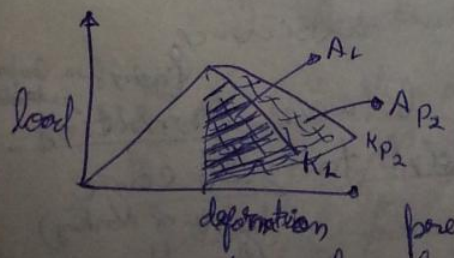


~~Form~~ From the load-deformation characteristic it is clear that $A_L > A_{P_1}$ which thereby ~~two~~ brings us to the conclusion that:-

~~The pillar designed has~~

The loading system has higher potential transfer capacity (A_L) ~~and~~ while the pillar has a lower energy accomodating capability (A_{P_1}). So the excess energy ~~is released~~, when released, will be turbulent thus resulting into a violent or uncontrolled failure of the pillar.

Case 2:- $K_{P_2} < K_L$ (i.e. the pillar is softer than the loading system)



From the curve:- $A_{P_2} > A_L$

The conclusion here is opposite to the previous case and hence even after the loading systems transfers all its energy to the pillar, there is still ample scope in the pillar to accomodate more energy. This can be seen as a gradual failure of the pillar.

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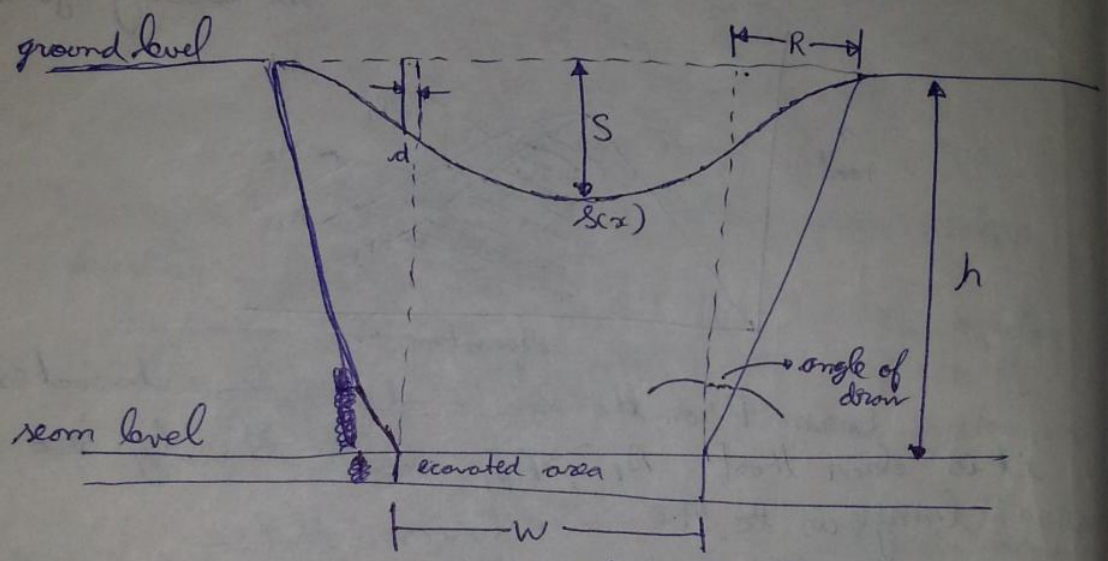
So we can conclude that with the lower stiffness of the pillar and hence with a higher w/h ratio we can plan for a design of the pillar with gradual or controlled failure with a higher degree of stability.

19/08/2013.

GROUND CONTROL (BKS Sir)

PROFILE FUNCTION METHOD:-



w → span of working

S → max. subsidence on the surface

h → depth of mine

R → horizontal distance between the edge (rib edge) of working to point of negligible subsidence

d → dist between transition point (going from Goaf to the) to rib edge (edge of working)

s → subsidence in all other points (varies with x)

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$$S = f(S, x, R)$$

x → horizontal dist from origin

If the extraction is subcritical then another point $\left(\frac{w}{h}\right)$ also has to be taken into account.

$$S = f\left(S, x, R, \frac{w}{h}\right)$$

A profile function simply defines the shape of the curve and it requires a max. value of subsidence and a no. of constants to calculate the subsidence profile. The method is generally applied by adjusting the constants in the equation until a satisfactory shape is obtained which provides an acceptable match to the observed data.

Consequently using this calibrated equation, it should then be possible to predict subsidence curves for that particular locality.

Profile functions has 2 main advantages:-

1.) As it has is defined by an equation it can easily be calculated with the help of a calculator / computer.

2.) When very little measured data are available the method can still be applied using limited data to calculate constants

Disadvantages:-

1.) The profile function shape depends on various external parameters that are not taken into account by the function and it may lead to an error in the shape.

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2) The profile function may not match exactly the shape ~~given~~ of ~~tatg~~ that given by measured data and consequently introduce an error. Such an error may not be significant but it may give significant error when tilt or curvature are calculated from the subsidence profile.

ex) Hyperbolic function given by King and ~~Kitton~~ whetton :-

$$S = \left(\frac{S}{2}\right)\left\{1 - \tanh\left(\frac{2x}{R}\right)\right\}$$

Based on :- Transition point origin

ex) By Kumar, Saxena and Singh (1983) (Indian profile function)

for subcritical

$$s = S e^{-\frac{nx^2}{P^2 - x^2}}$$

for critical

$$s = S e^{-\frac{nx^4}{P^4 - x^4}}$$

$n \rightarrow$ Constant (1-5)

$x \rightarrow$ distance from extraction center

$R \rightarrow$ radius of critical area of extraction

P = half width of subsidence profile $\left(\frac{W}{2} + R\right)$

This is based on face center origin (Center of the subsidence curve)

Assignment 1:-

Search various profile function methods and explain any one profile function method.

NOTE → Strain can be calculated using profile function for a particular location by relating strain to the curvature. And this curvature is the 2nd derivative of the subsidence (s) equation

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Design of Pillars (Prediction of width of pillar for a particular extraction height)

- This is a hit and trial method that can be easily applied for square pillars.

If we have a value of K (strength constant)

K = strength of 1 cubic inch sample

then we can easily design the pillar for a particular factor of safety (S.F.) and Extraction height (h).

$$S.F. = \frac{S_p}{\sigma_p}$$

$$S_p = K \frac{W^{\alpha}}{h^{\beta}}$$

$\alpha, \beta \rightarrow$ empirical constants.

$W \rightarrow$ solid width of pillar.

For indian Coalfields:- $\alpha = 0.46$; $\beta = 0.66$

$$\sigma_p = \frac{0.025 H (W + W_b)^2}{(W)^2}$$

$W_b \rightarrow$ width of bord (gallery width)

$$S.F. = \frac{K W^{\alpha} \quad (W)^2}{h^{\beta} \times 0.025 \times H \times (W + W_b)^2}$$

$$\therefore S.F. = \frac{K (W)^{2+\alpha}}{0.025 \times H \times h^{\beta} (W + W_b)^2}$$

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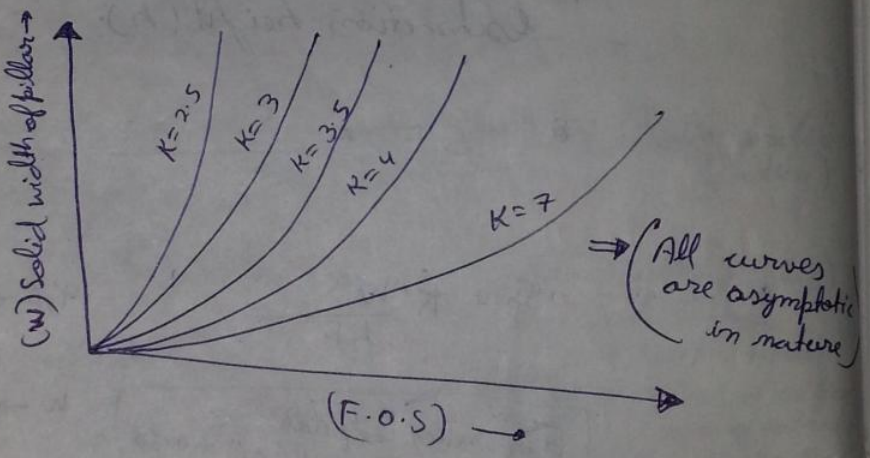
For a particular S.F. and K (strength constant) and having the values of w_b, h, H, α, β we would have a non-linear equation in (w) which can be solved by hit and trial values of w.

This is an easy approach to guess the the width of pillar for a fixed K (strength constant) and S.F.

Now for the same values of K if the pillar is designed for different (F.O.S.) then we see that:



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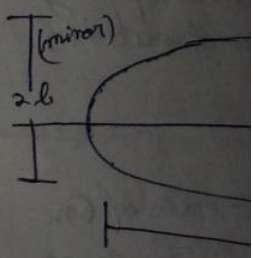

The radial

$P_1, P_2 \rightarrow$

$P_1, P_2 \rightarrow$

The tangential shaped

closed form solution for elliptical opening.

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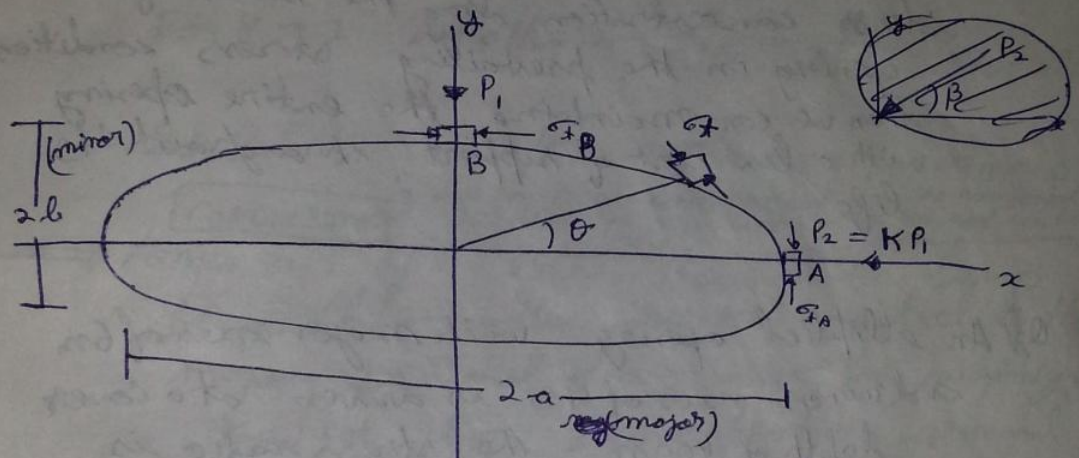
GROUND CONTROL (GSPS)

Stress around elliptical shaped tunnel:-

for an ellipse → the diameter along 2 perpendicular axis is different.



The radial stress along roof and sides is zero

P_1, P_2 → Vertical and horizontal principle stress components (if opening is horizontal)
(principle stress)

P_1, P_2 → 2 components of horizontal stress (if opening is vertical)

The tangential stresses at point A on the side of the elliptical shaped tunnel and at point B in the roof are:-

Closed form solution for elliptical opening.

$$\sigma_{tA} = P_1\left(1 - K + \frac{2a}{b}\right)$$

$$\sigma_{tB} = P_1\left(K - 1 + \frac{2bK}{a}\right)$$

NOTE ⟹ By changing the orientation of the ellipse by 90° the above formulae remain same except that the ratio of a/b get inversed.

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⇒ Some areas along the periphery may have a high induced stress concentration and may undergo failure. So such areas have to be highly supported throughout the life of the opening.

But

⇒ If we have an opportunity to ~~do~~ modify the orientation of the opening to relax the stress concentration along the sides of the opening in the prevailing stress condition then we can maintain the entire opening with a low cost of support throughout its life

Q.) An elliptical opening with major axis of 6m and minor axis of 4m is driven at a cover depth of 400m. The stress ratio is (0.4). Suggest a suitable orientation of the opening under the given circumstances.

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In case of uniaxial stress direction (assuming $K=0$) and $A/b = 2$ we have ; P_1 is normal to the larger axis of the ellipsoidal shape of the tunnel:-

$$\sigma_{tA} = 5P_1 \quad ; \quad \sigma_{tB} = -P_1 \text{ (tensile stress)}$$

• For P_1 normal to the shorter axis, they are

$$\sigma_{tA} = 2P_1 \qquad \sigma_{tB} = -P_1$$

For $K = 2$ and $a/b = 2$ $\sigma_{tA} = 3P_1$; $\sigma_{tB} = 3P_1$

For $K = 0.5$ and $a/b = 2$ $\sigma_{tA} = 4.5P_1$; $\sigma_{tB} = 0$

CONCLUSION:- due to some confinement in the horizontal direction the stress condition at the roof can be improved.

⟹ Longer axis of the Excavation should always be ~~parallel to the~~ oriented parallel to the major principal stress axis for minimizing the high stress concentration around the excavation.

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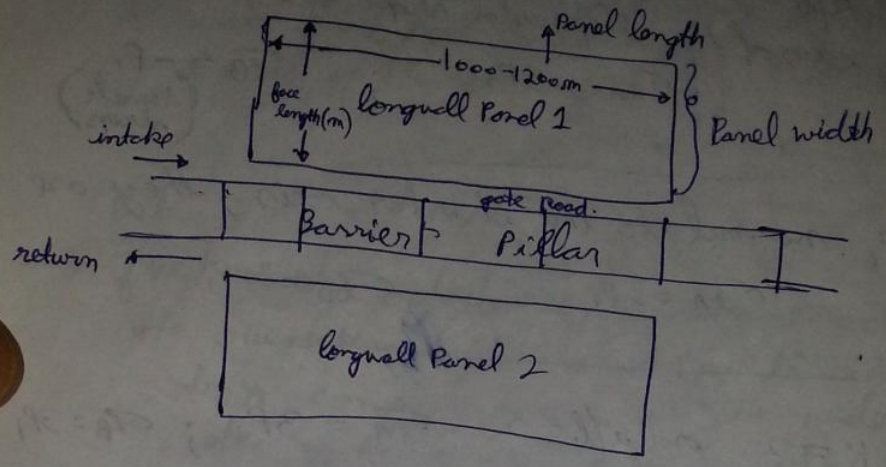
Chain Pillar:-



(90-160m) face length is much smaller compared to the width

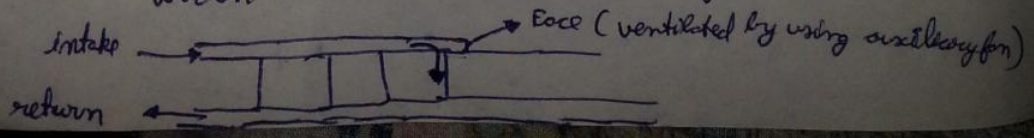
abroad { longest ~~highest~~ longwall face = 350m
longest panel length = 3500m

Panel is developed by driving gate roads along the boundary of the Panel

restriction { max. width of gate road = 4.8m
max. height of gate road = 3m

Due to large panel length, such long blind heading cannot be driven without ventilation so there is a requirement for splitting of the barrier pillars but (splitting should be minimum)

These pillars are called chain pillars because their length is quite large compared to their width.



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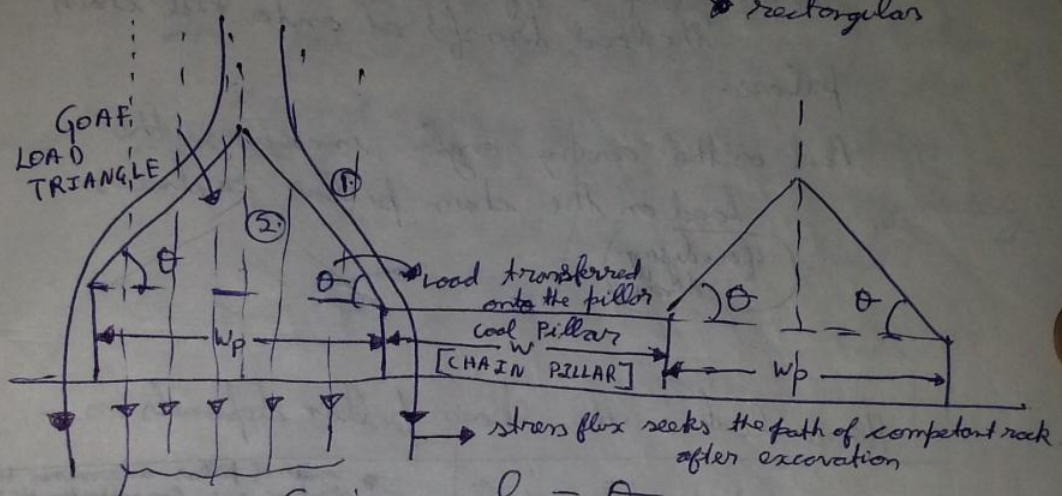
We can have:-

- Single row of chain pillar. → square / → rectangular
- Double line of chain pillar → square / → rectangular



Caving angle = θ

(direction of stress flux in unexcavated rock)

Wp → width of panel + width of gate road

Rockmass 1. is very very compacted.

Rockmass 2. is very loose rock.

Initially soft capability to accomodate stress is more of material 1. than material 2. due to looseness of material 2.

So majority of the load on the goaf material is shared by the coal pillar but not entirely initially

But later on, on compaction of goaf material the load bearing capacity of goaf increases and it too takes a major amount of load of the goaf roof. Rest is shared by the pillars.

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⇒ Hence the chain pillar is ~~the~~ more loaded than development pillars in B and P mining due to the large width of the panel.

If we consider the Coving angle → As the coving angle reduces the overhang of the goaf roof is more so more is the load transferred onto the chain pillars.

But as the coving angle increases the load (of overlying strata) on the chain pillar reduces.

Hence load on the chain pillar depends on:-

fundamental parameters

1.) width of pillar → width of pillar determines the effective area for supporting the roof. As width of pillar ↑ → stress decreases

2.) width of Panel → determines the span of the goaf which is to be supported by the pillar on complete extraction of adjacent ~~pillars~~ panels.

3.) Cover depth of seam → vertical stress increases with cover depth so does the load on the pillar

4.) Coving angle of the goaf → As the coving angle decreases a greater amt of load is transferred onto the chain pillars.

other parameters

5.) Strength of coal seam

6.) Stiffness ratio of coal pillar w.r.t stiffness of loading system

⇒ These affect loading and stability of chain pillars

⇒ Till adjac

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exceptions

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23/08/2013.

GSPS Sir

CHAIN Pillar Design

- Once the second adjacent longwall panel has been extracted, the chain pillars are subjected to extremely high strata loads. In most situations, the pillars will collapse after the second longwall panel has been extracted. This reduces the irregularity of subsidence at the surface.

⇒ Till adjacent panel ~~the~~ is not ~~excavated~~ extracted the load on chain pillar is very near to the in situ value.

exceptions → In 1994 we had started working a longwall at the depth of 180 to 200 m. The extraction hit was 4.2 m. Panel 1 was of a length of 150 m. The width of chain pillar was 34 m.

In the major period of extraction we experienced spalling (failure from side of pillar) and as the face was advanced by 800 m then the spalling was upto 3 m (i.e. $W_{gallery} = 7.8$ m).

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dependence Bending stress of the roof of gallery is $\propto W_{gal}^2$

NOTE:-

Putting inadequate support is more dangerous than putting no support.

Our knowledge about the rational requirement of the support is very important.

But this spalling was observed locally (at some points)

Realizing the problem the next face length (bend) was reduced to 120 m

Fortunately the panel extraction was successful.

This resulted the case of over confidence and the next panel was again at a face length of 150 m.

But when this panel face was advanced by 700 to 800 m, Supports from 1 to 80 suffered mechanical damage (hydraulic system failed), spacing between the support was 1.5 m. Hence a length of $(80 \times 1.5) = 120$ m face length failed.

The major reason of failure was credited to improper design of the chain pillars.

The accident took place in the year 1997

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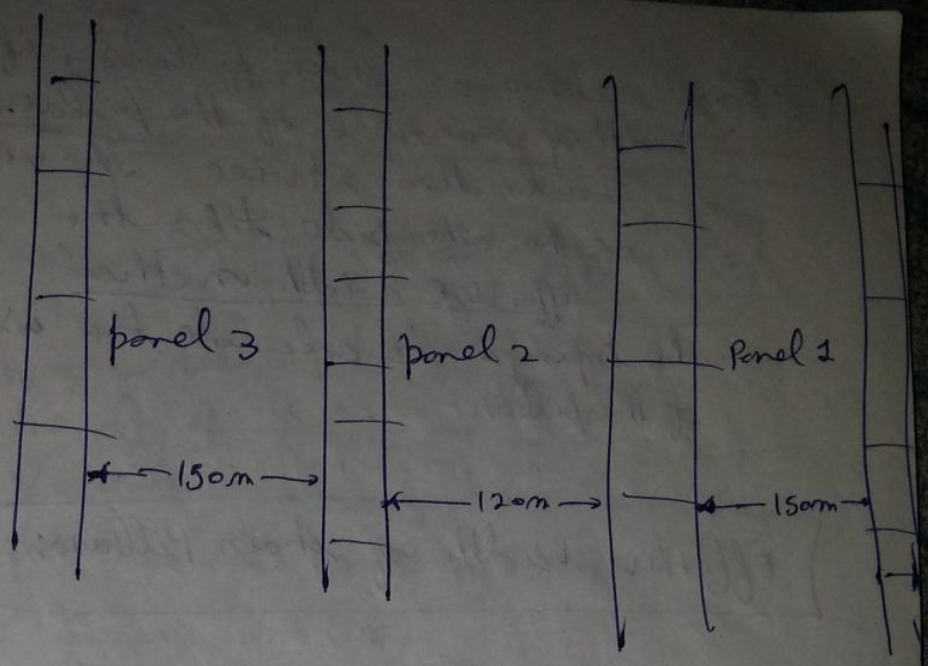

Strength of pillar:-

(Salamon Munro formula)

$$S_p(MPa) = \frac{7.2\, p^{0.46}}{m^{0.66}}$$

7.2 → K (size specific value)

p = pillar width (m) (solid width) m = pillar height (m)

Main pillar are generally rectangular so that the no. of splittings in the Barrier pillar is less.

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- For rectangular chain pillars, if the width of one side of the pillar is greater than twice the width of the other side then the effective width method must be employed to calculate the strength of the pillar.

Effective Width of Chain Pillars:-

The effective width of a large rectangular chain pillar can be calculated by using either of the formulae:-

$$W_{eff} = (W_1 \times W_2)^{0.5}$$

$$W_{eff} = 4A/C$$

W_1 = Width of one side of the chain pillar, smaller dimension (m)

W_2 = width of other side of chain pillar, larger dimension (m)

A = Area of the pillar (m^2)

C = Circumference of the pillar (m)

h = height of the pillar (metres)

Restrictions on the above formulae:
- if $W_2 = 2W_1$, $\frac{W_1}{h} \leq 5$ — $W_1 = W_2$
- if $W_2 > 2W_1$, $\frac{W_1}{h} > 5$ — $W_2 = 2W_1$

here h = height of pillar

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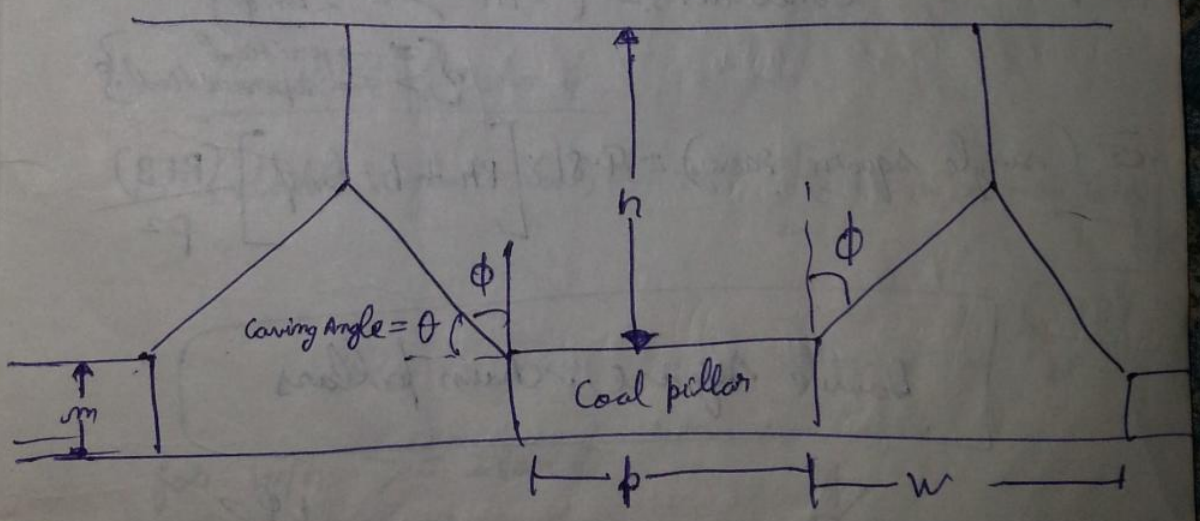
refer to site of

⇒ university of Wollongong

Loading ~~of~~ on the pillar :-

W = width of longwall face (m)
h = depth below ~~the~~ surface (m)
γ = average density of overburden (kg/m³)
ϕ = average angle of shear (°)
P = pillar width (m)
B = Bord (heading) width (m) (width of gate road)



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depending ~~of~~ on width of extraction w.r.t the cover depth, the different types of subsidence are:-

1.) Subcritical

2.) critical

3.) supercritical.

Single Square chain design:-

For $w/h < 2\tan\phi$ (For subcritical subsidence conditions)

$$\bar{\sigma}\,(\text{single square row}) = 9.81\gamma\left[(P+w)h - \frac{w^2\cot\phi}{4}\right]\frac{(P+B)}{P^2}$$

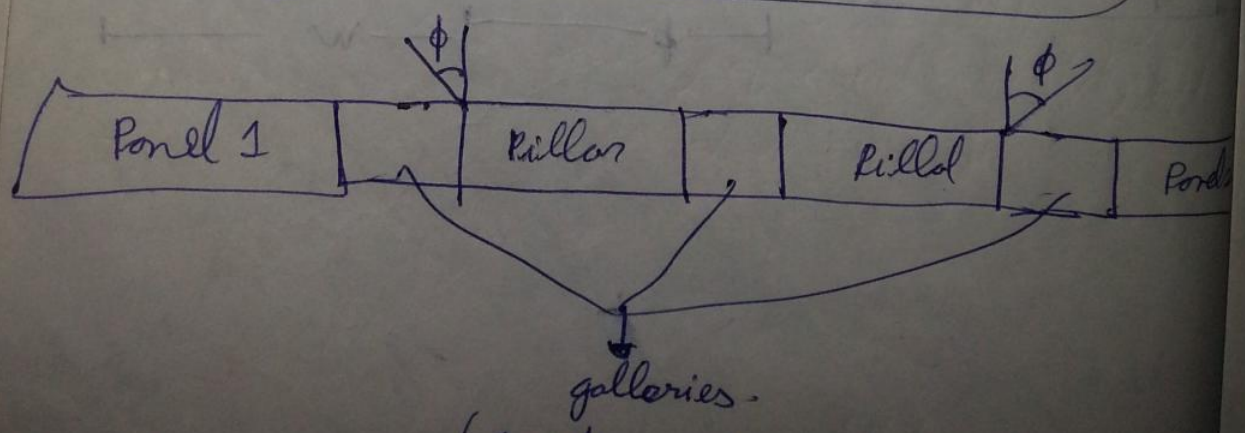
For critical and super critical mining conditions:- ($w/h \geq 2\tan\phi$)

$\geq \rightarrow$ critical

$> \rightarrow$ supercritical.

$$\bar{\sigma}\,(\text{single square row}) = 9.81\gamma\left[Ph + h^2\tan\phi\right]\frac{(P+B)}{P^2}$$

Double square chain pillars



(Roadway Crossection View)

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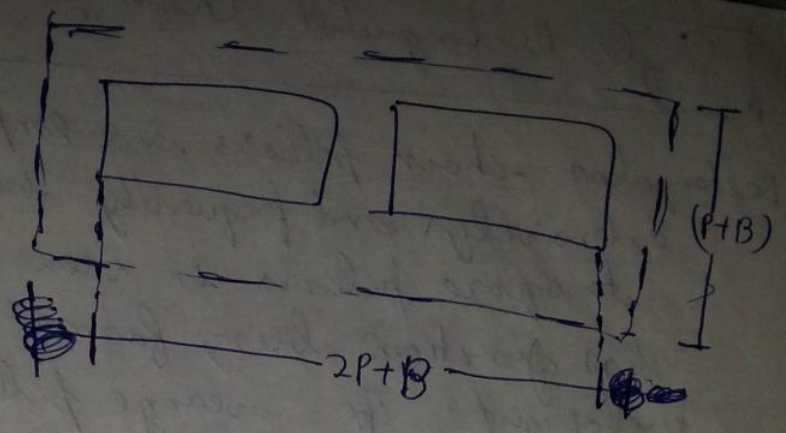

(Roadway Plan view)

- effective width of the chain pillar support area in this case. (2P+B) and load is borne by two pillars of a plan area $2P^2$

The respective average pillar stresses for this situation are:-

<u>for w/h < 2 tan φ</u>

$$\bar{\sigma}\,(\text{double square row}) = 9.81\,\gamma\left[(2P+B+w)h - \frac{w^2 \cot\phi}{4}\right] \times \frac{(P+B)}{2P^2}$$

<u>for w/h >= 2 tan φ</u>

$$\bar{\sigma}\,(\text{double square row}) = 9.81\,\gamma\left[(2P+B)h + h^2 \tan\phi\right] \times \frac{(P+B)}{2P^2}$$

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Single Rectangular Chain pillars:-

- Rectangular chain pillars are employed extensively and frequently in preference to square pillars in view of less junctions being formed during development. The average pillar stress is given by:-

$$\bar{\sigma}\,(\text{single rect-row}) = 9.81\gamma\left[(P+w)h - \frac{w^2 \cot\phi}{4}\right]\frac{(L+B)}{P\cdot L}$$

for $w/h < 2\tan\phi$

$$\bar{\sigma}\,(\text{single rect row}) = 9.81\gamma\left[Ph + h^2\tan\phi\right]\frac{(L+B)}{P\cdot L}$$

for $w/h >= 2\tan\phi$

where:-

w = width of longwall face (m)
h = depth ~~of~~ below surface (m)
γ = average density of overburden (kg/m^3)
ϕ = average angle of shear (°)
P = pillar width (m)
L = pillar Length (m)
B = Bord (heading) width (m)

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pillars:-

employed
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$- \frac{w^2 \cot\phi}{4} \Big] \frac{(L+B)}{P \cdot L}$

$h^2 \tan\phi \Big] \frac{(L+B)}{P \cdot L}$

)

)

$m (Kg/m^3)$

Double Rectangular chain pillars:-

for $w/h < 2\tan\phi$

$$\bar{\sigma}\,(\text{double rec. row}) = 9.81\gamma \left[(2P+B+w)h - \frac{w^2 \cot\phi}{4} \right] \frac{(L+B)}{2P \cdot L}$$

for $w/h \geq 2\tan\phi$

$$\bar{\sigma}\,(\text{double. rect row}) = 9.81\gamma \left[(2P+B)h + h^2 \tan\phi \right] \times \frac{(L+B)}{2P \cdot L}$$

Q.) A longwall mine is planned for extracting 3.2m thick seam for its full thickness under a cover depth of 250m. The planned length of face is 150m. Single row rectangular type chain pillar is proposed with a size of 15m×35m for this purpose. The width of the gate road is 4.6m where as the strength of coal (K) = 7.2 Mpa Estimate the F.O.S of the pillar when both the panels intervening Barrier pillar under consideration has been droped out.

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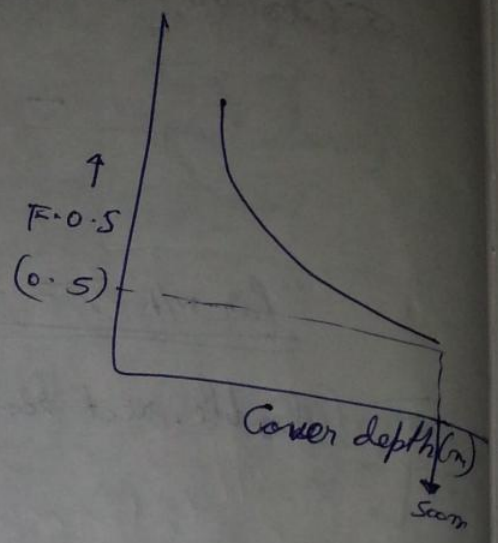
Factors affecting Safety factor of a chain pillar:-

• Cover depth :-

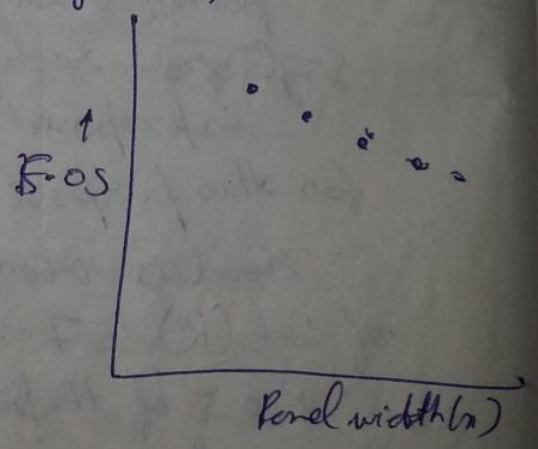
- with increasing depth of cover, the amt. of vertical stress a chain pillar is subjected to, increases.



- A numerical modelling study (Chakhoni, 2013) reveals that with increasing depth of cover, the S.F. of a chain pillar decreases following a power function

• Panel width :-

— The safety factor of chain pillar reduces as a non-linear function with increase in width of panel.



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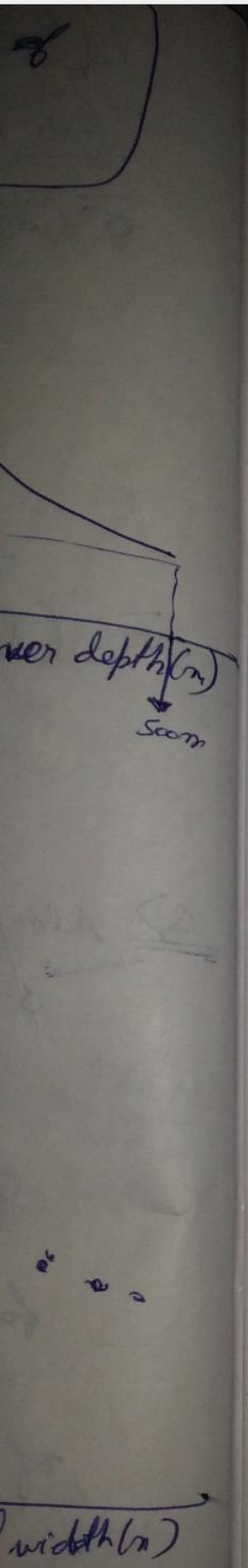
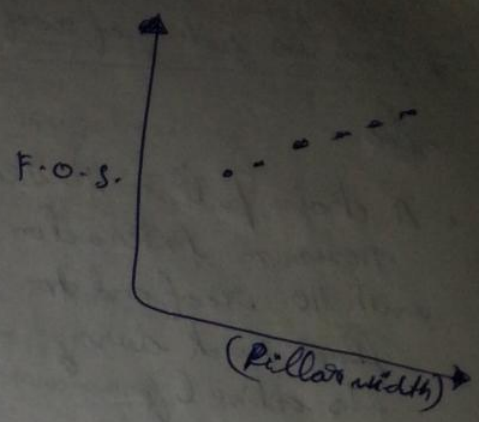


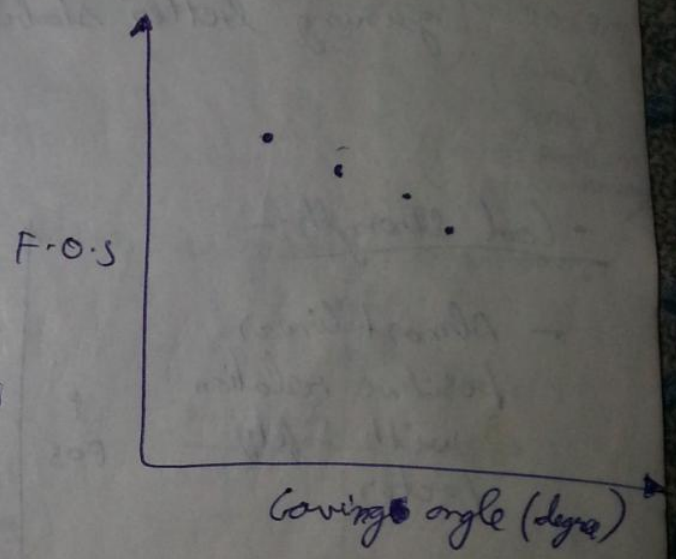
• Pillar width :-

- Numerical modelling study suggests a non-linear positive function of relation between safety factor of the pillar and the width of chain pillar



• Caving angle :-

- Decides the extent of strata involved in induction of abutment loading on the chain pillar



- The F.O.S of chain pillar decreases with increase in caving angle

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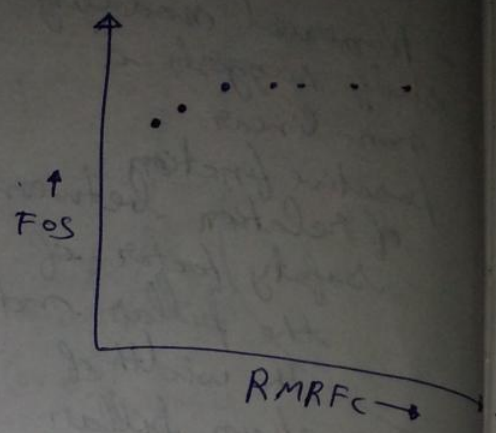
Modulus ratio of roof - floor of coal seam :-

~~flo~~

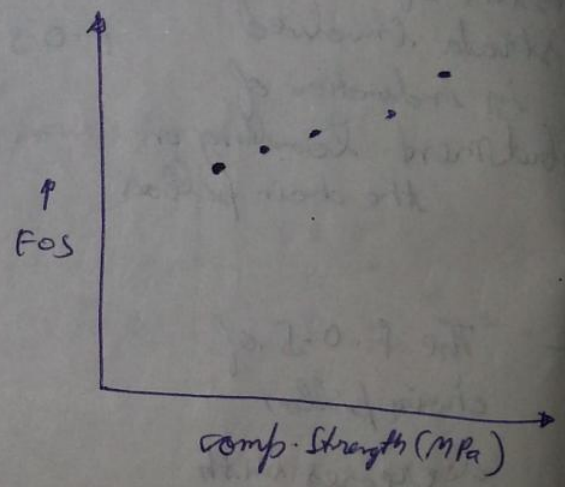
- A chain pillar recieves maximum interaction with the roof and the floor rock during its active life.



- A pillar in stiff loading environment gets better confinement from the roof and floor, gaining better stability

Coal strength :-

- Almost linear positive relation with Safety factor



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Solution of elliptical opening question given on 22/08/2013

given ⇒ $K = 0.4$

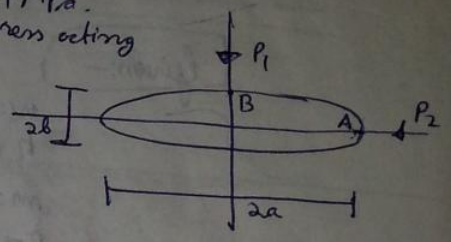
major axis = 6
minor axis = 4
$H = 400m$

$P_1 = \gamma H = 0.025 \times 400 = 10 MPa$

$P_2 = KP_1 = 4 MPa.$

Case 1:- · For major Principle stress acting along minor axis.

Here $a/b = 3/2$



$$\sigma_{tA} = P_1\left(1 - K + 2\frac{a}{b}\right)$$

$$= 10(1 - 0.4 + 3) = 36 MPa$$

$$\sigma_{tB} = P_1\left(K - 1 + \frac{2b}{a}K\right) = 10\left(0.4 - 1 + \frac{4}{3} \times 0.4\right) = -0.67 MPa$$

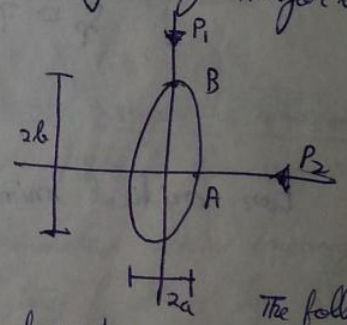
(tensile zone)

(NOTE:- Rocks are weak in tension)

Case 2.

· For major principle stress acting along major axis.

$$\frac{a}{b} = 2/3$$



$$\sigma_{tA} = P_1\left(1 - K + 2\frac{a}{b}\right)$$

$$= 19.33 MPa$$

$$\sigma_{tB} = P_1(0.4 - 1 + 3 \times 0.4) = 6 MPa.$$

is now compressive in nature

The following observations are made:-

1.) The nature of induced stresses is the same at all points around the periphery. The stress at B

2.) The induced stresses at critical points is reduced.

3.) There is relaxation in the stress concentration pattern along the periphery of the opening.

CONCLUSION:-

By orienting the ellipse in a manner that the major axis is parallel to the major principle stress, the induced stresses at the critical points (roof/floor and side) is reduced, thereby facilitating a relaxation in the stress concentration pattern along the periphery of the given elliptical opening.

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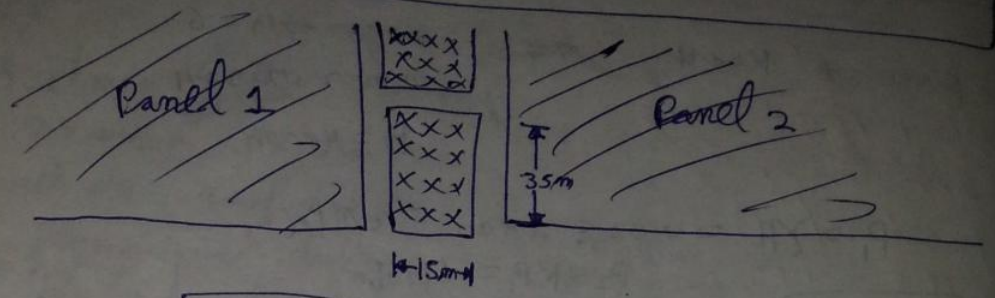
Solution of chain pillar question given on 23/08/2013



Given:-

$w = 150m$
$h = 250m$
$m = 3.2m$
$B = 4.6m$
$P = 15m$
$L = 35m$
$K = 7.2\ MPa$
$\gamma = 0.025\ MPa/m$

Strength of pillar :-

$$P_{eff} = \frac{4 \times (15 \times 35)}{2(15+35)} = \frac{2100}{100} = 21\ m$$

$$S_p = 7.2\ \frac{P^{0.46}}{m^{0.66}} = 13.56\ MPa$$

For critical mining Conditions $\Rightarrow \frac{w}{h} = 2\tan\phi$

$$\Rightarrow \tan\phi = \frac{w}{2h}$$

$$\therefore \phi = \tan^{-1}\left(\frac{w}{2h}\right)$$

$$\therefore \phi = 16.70^\circ$$

(on complete extraction of Panel 1 and 2)

Ultimate stress on pillar :-

$$\bar{\sigma} = 9.81\gamma\left[Ph + h^2\tan\phi\right]\left(\frac{L+B}{P \cdot L}\right) \quad \text{where } \gamma \rightarrow (kg/m^3)$$

$$= \gamma\left[Ph + h^2\tan\phi\right]\left(\frac{L+B}{P \cdot L}\right) \quad \text{where } \gamma \rightarrow MPa/m$$

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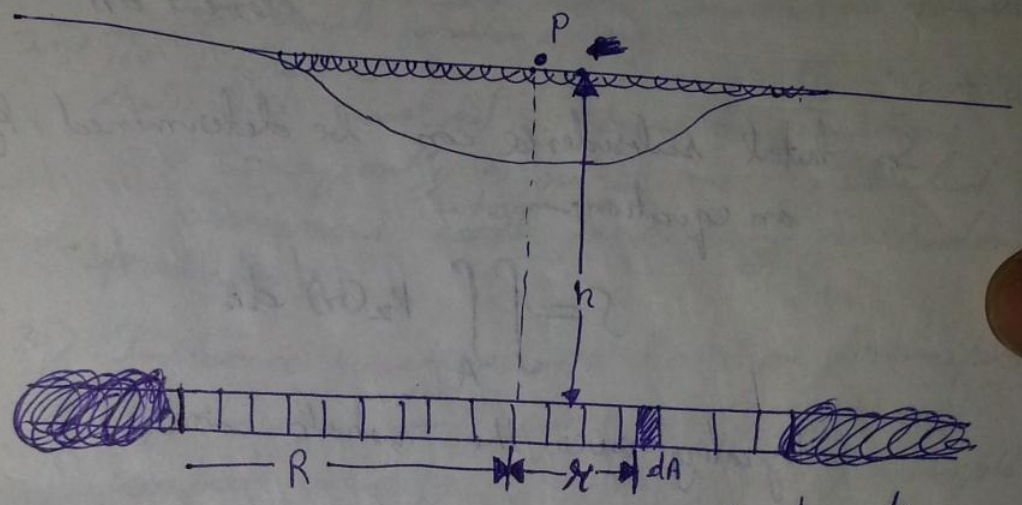
$\therefore \bar{\sigma} = 42.43 MPa$

Factor of Safety $= \frac{S_p}{\sigma_p} = \frac{13.56 MPa}{42.43 MPa} = 0.319$

26/08/2013.

INFLUENCE FUNCTION



R = radius

2R = span of mine

This also on empirical method. We get the subsidence in 3-D. (advantage over profile function method)

The mining area is divided into many small areas (dA) and then by some function we could find out the contribution of that small elemental area on the subsidence at some point P under consideration. Then the function is integrated over the entire area of the mine.

The Basic principle of the influence function method is the application of the law of superposition to determine the overall influence of an extraction area treated in infinitesimal parts. The basic equation of influence function method

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in in terms of:-

$$K_z = f(r)$$

where K_z → magnitude of influence of dA on P

r → is the horizontal distance between position of P and the infinitesimal element dA

So total subsidence can be determined by an equation:-

$$S = \iint_A K_z(r)\, dA$$

integrating over the whole area

H.W. Dr. Shorey ~~Dr. Shorey~~ → Indian influence function method to predict subsidence (Read this paper properly)

Comparison of Influence function and Profile function

Profile function:-

1.) field observations can be easily employed to derive profile functions

2.) Profile function can be readily applied to a wide range of subsidence problems including subcritical situations

3.) The range of subsidence problems to which

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profile functions can be applied is mainly limited to relatively straight forward 2-D mining extraction situations involving essentially rectangular geometrical configurations.

Influence functions:-

1.) Shape of the extraction is not a limiting factor in respect of influence functions, Since they can be applied to any shape of extraction in one or more seams (not applicable for profile function method)

2.) The method is more mathematically rigorous than that involved in profile functions and consequently requires more detailed assessment when performing subsidence predictions.

3.) Influence functions are based on principles of superposition and consequently assume complete validity of applying the principle to any mining situation.

4.) Prediction of displacement with influence functions tend to be more complecated than with profile functions

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2/09/2013 BKS Sir

Numerical Methods:-

Important aspect for subsidence:-

1.) Depth of seam
2.) span of working
3.) Thickness of seam

⇒ Physico-mechanical properties of immediate roof controls subsidence to a large extent. So we have to consider the physico-mechanical properties of the various layers to get the effect of ~~subsi~~ caving on the surface. The present methods do not consider this factor.

⇒ Geometrical features (thickness, etc) of all beds are different so ~~to~~ stresses developed are different. This is also taken into account

⇒ Sequence of arrangment of strata by nature also has a significant impact on subsidence

⇒ Inter bed strength — Bonding between all layers of strata.
On bending ~~the~~ some tensile stresses are developed within beds also. ~~The~~ The inter bed stress should be such as the beds should separate on failure of roof so that proper caving takes place.

⇒ After failure the void of mining is filled by broken material which gradually takes the load. So property or behaviour of the ~~broo~~ broken material is ~~nec~~ necessary

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we establish the whole mining system from surface to seam level in a computer by a mathematical model. And by inputting various parameters of mining we get more and more accurate results of subsidence. After making the model it should be compared with a known case and reliability of the model is to be determined and then results are predicted.

Physical Simulation Method:-

- Prior to numerical modelling methods.
- The dimensions of the mine and all the physico mechanical properties and other properties and parameters were scaled down (to lab scale) and a physical model was made which was simulated to predict results of subsidence.
- But this method is expensive and highly tedious so obsolete.

6/09/2013 — **Support Design** — (GSPS Sir)

Purpose of Support:

- The structure of the design may not be self sustainable for the time it is desired for, so we require to provide proper support to the structure
- The provide safety to the mine personnel and for undisturbed production.

The mining excavations are not permanent, they have limited life, so the amount of support we are putting has to be in accordance with the life of the support and the life of excavation.

- The general support requirements of productive excavations differ significantly from those

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applicable to excavations of a more permanent nature
- The support employed in productive excavations must be simple, robust, effective, easy to install and cheap.

- The actual choice of the support system depends upon the availability of materials, economics of the verture (project) and the method of extraction

availability of material:-
good quality timber is now difficult to find so we shifted to friction / hydraulic supports.

economics of verture:-
The cost of support always adds to the production cost of the mine and affects profit margin.

mining method:-
when we had manual method of mining, the span of gallery could be less so prop support were used but by the use of mechanization we required more span of gallery so props become obstructions so we shifted to roof bolting.

- The decisive considerations regarding the choice of support system is based on either the experience in similar mining condition or sound rock mechanics principles.

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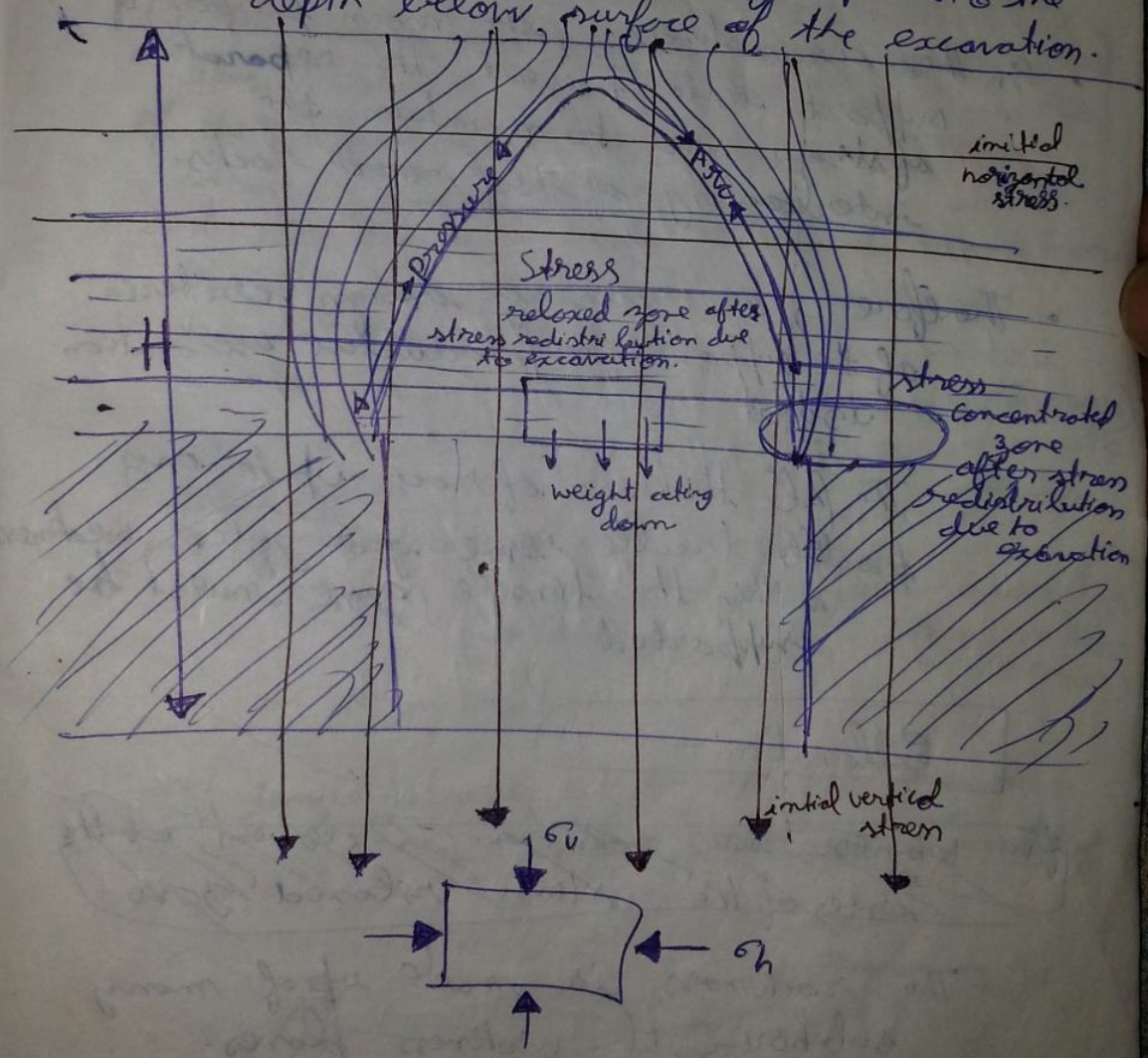
Principle of Roof Control:-

- Mining activities induces vertical and horizontal tensile stresses in the roof strata of tabular excavations. (thickness of the excavation is much less than the length of the excavation)
- The relative magnitude of the tensile stresses and the vertical extent of the tensile zone depend largely on the ratio of the mining span to the depth below surface of the excavation.



On central element $\sigma_v = 0$, σ_h = minimum } So stress relaxed zone because it relaxes both σ_v and σ_h

As we move towards the stress concentrated zone:-

σ_v = increases and at the corner of opening it is very very high, more than initial value.

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σ_h = increases, we can recover σ_H but it is difficult to predict its value, it has complex distribution.

- Where weakness occurs within the tensile zone, strata separation can be expected unless adequate support is provided.

- In these circumstances, the main purpose of support is to prevent separation of strata and to maintain the interlocking of roof rocks.

- Therefore, a necessary design criteria of the support for productive excavation is that
  - The full thickness of rock up to any trouble geological weakness within the tensile zone must be supported.

Explanation:-

~~Here whenever we consider an element at the center of the stress relaxed zone~~

The rockmass is made up of many subhorizontal weakness planes.

An element in the stress relaxed zone tends to displace downward due to its weight acting downward and due to lack of confinement (traction free) to stress relaxation.

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So the layers of rockmass tend to bend. But due to differente in the elastic mechanical properties and Bending strength of different layers because they are formed of different material, the Bending extent of different layers is different so a parting develops between the different layers. Bending increases and hence parting increases if the no. of weakness planes in the rockmass increases.

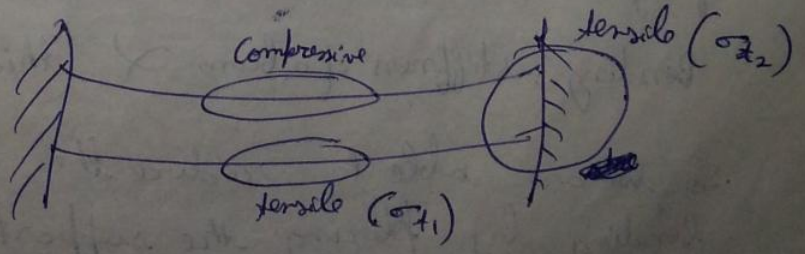
Stresses develop at different points on the roof rock due to bending of the roof (analogous to a beam bending)

At the corners of the opening, (stress concentrated zones), highest tensile stresses develop. At the center of the roof layer on the upper face compressive stresses develop due to compressive strain and on the lower face of the rock layers tensile stresses develop due to tensile strain. (Stress relaxation zone)

On studying the bending of a elastic beam (tensile deformation) with the given boundary condition (i.e. 1.) Confinement at the corners
2.) lack of confinement at the center (i.e. traction free at the center) (to bending)

we know that

tensile stresses at the corner of the beam = 2 X tensile stresses at the center of the beam



$\sigma_{t_2} = 2\sigma_{t_1}$

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So we can conclude that the ~~ce~~ tensile stresses at the center of the roof will be ~~half~~ half of the tensile stresses at the stress concentrated zones (corner of gallery)

But tensile strain will be maximum at the center of the roof compared to the corners and so this point is likely to undergo tensile failure due to maximum bending at the center. The area of ~~sh~~ stress concentrated zones (corners of the opening) are likely to undergo shear failure.

So our point of attack (i.e. the point at which support is required to be installed) is at the center of the roof as it is the position of maximum tensile strain. By supporting at the center we can control the tensile stresses developed at the corners of the roof.

By attacking at the center we are increasing the inherent strength of the rockmass (The rockmass has some strength due to its formation). The support reduces the parting between the rock layers. By reducing the parting ~~to~~ the ~~ten~~ ~~bending~~ rockmass behaves as a massive rock (i.e. a composite block) of higher thickness and we know that

$$\text{Bending stiffness of a beam} \propto (\text{thickness of beam})^2$$

So we are able to reduce the extent of bending by placing the support at the point of maximum strain (stress relaxed zone), i.e. at the center of the roof due to an increase in the bending stiffness of the ~~rock~~ roof rock.

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So the amount of tensile stresses developed at all points reduces so chances of tensile failure of roof reduces.

Q) What should be the ~~vertical~~ vertical extent of support ??

A) The zone of influence of the support should be upto the extent to which tensile stresses are likely to develop. The zone of influence of the support should be able to nullify the zone of tensile stress. The above is true if the support is placed at the point ~~of~~ prone to ~~undergo~~ tensile failure (i.e. at the center of the roof).

If the support is placed as the ~~place~~ ~~point~~ ~~prone~~ point which is likely to undergo shear failure ~~then~~ (stress concentrated zone) then the orientation of the support should be such that it increases the cohesion of the rockmass and the friction of the rockmass. By increasing either of the 2 or both the shear strength of the rockmass increases and its chances of failure (shear) reduces.

⇒ This is the principle of roof bolting and ~~this~~ the above states the reason why the first bolt is placed at the center of the gallery roof.

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9/09/2013.

BKS Sir

Factors affecting Mine Subsidence :-

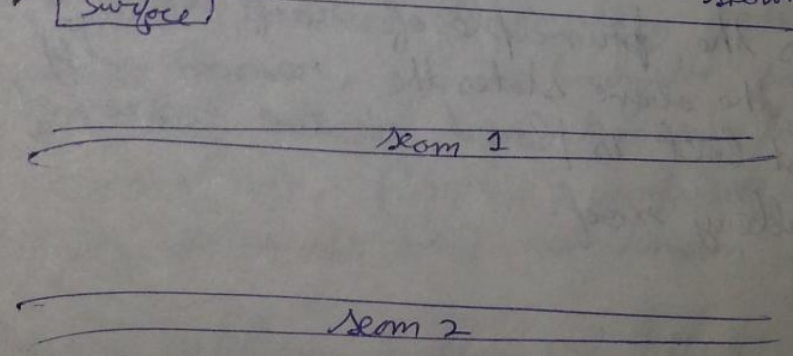
(1.) effective seam thickness
(2.) Multiple seam
(3.) Seam Depth

1.) effective seam thickness → Thickness which is mined out is effective seam thickness

thickness ↑ → subsidence ↑

2.) Multiple seams → we mine in decending order (upper first and then we move down) in Caving. So if we have multiple seam in close proximity then they will have a cumulative effect of subsidence and it will be more than predicted by indivisual seams.

→ we mine in ascending order in stowing



when seam 1 mined → above strata becomes weak and fractured

when seam 2 mined → then parting becomes weak and the entire above strata slips down.

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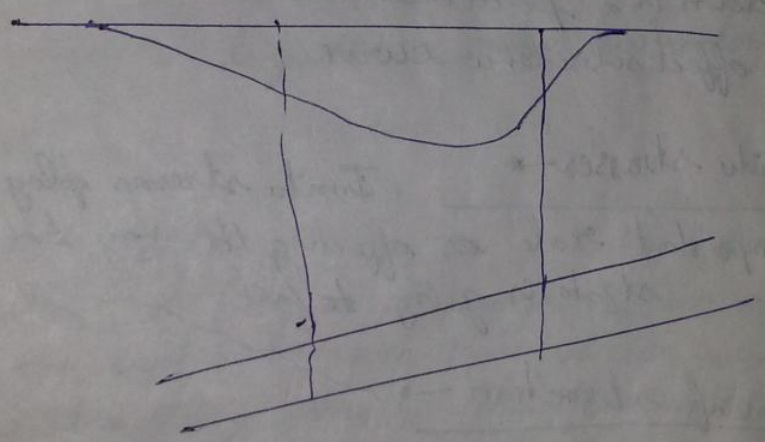
3.) Seam Depth →

more depth → less subsidence

4.) Dip of the seam → when the coal seam being mined is inclined ~~and~~, an asymmetric subsidence trough is formed that is skewed towards the rise. i.e. the limit angle is greater on the dip side of the working, The strains are also smaller towards the dip direction



5.) Competence of Mine roof and floor →

If the ~~at~~ immediate roof is competent and massive, it delays subsidence and we get a different subsidence ~~p~~ curve.

6.) Nature of overburden → The intermediate layers of rocks should be tovable so that we get a conventional subsidence profile. All layers upto the surface should break properly.

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7) Near surface geology →

Surface has very weak material usually (sand, soil, etc). Ultimate result of subsidence is seen on the surface. If surface is made up of hard rock then it will not crack properly thereby affecting subsidence curve.

8.) Geological Discontinuity →

Presence of folds, faults affects subsidence profile by affecting the pattern of caving.

9.) Fractures → Joints allow material to fall down in a particular manner and hence affect subsidence curve.

10) Insitu stresses →

Insitu stresses play a important role affecting the way the strata is going to fail

11.) Degree of extraction →

Degree of extration is less → delays subsidence → ~~affect subsi~~ less subsidence

~~9th~~ Coal is strain softening material so it takes some load even after failure and so if pillars are left then they delay subsidence
• This is the case in B and P mining.

12.) Surface Topography → tensile strain may become more marked on hill tops and decrease in valleys

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13.) Ground water →

Removal of water from underground leads to subsidence in a fashion similar to that caused by removal of material.

But we have a benefit :-

Sometimes water is filled in the mines. after extraction and so water pressure supports the strata and delays subsidence.

14.) Span → (Mined Area)

The span of the mine should be greater than the NEW for subsidence to take place.

15.) Method of working →

Board P → less subsidence
longwall → more subsidence

16.) Rate of face advance → Surface subsidence follows the face as it progresses in a panel. If the coal extraction rate varies significantly, the travelling strain also fluctuates & this results in large differential settlement. A fairly rapid even rate of face advance is best.

17.) Backfilling of the goaf

18.) Time lapse

19.) Structural characteristics

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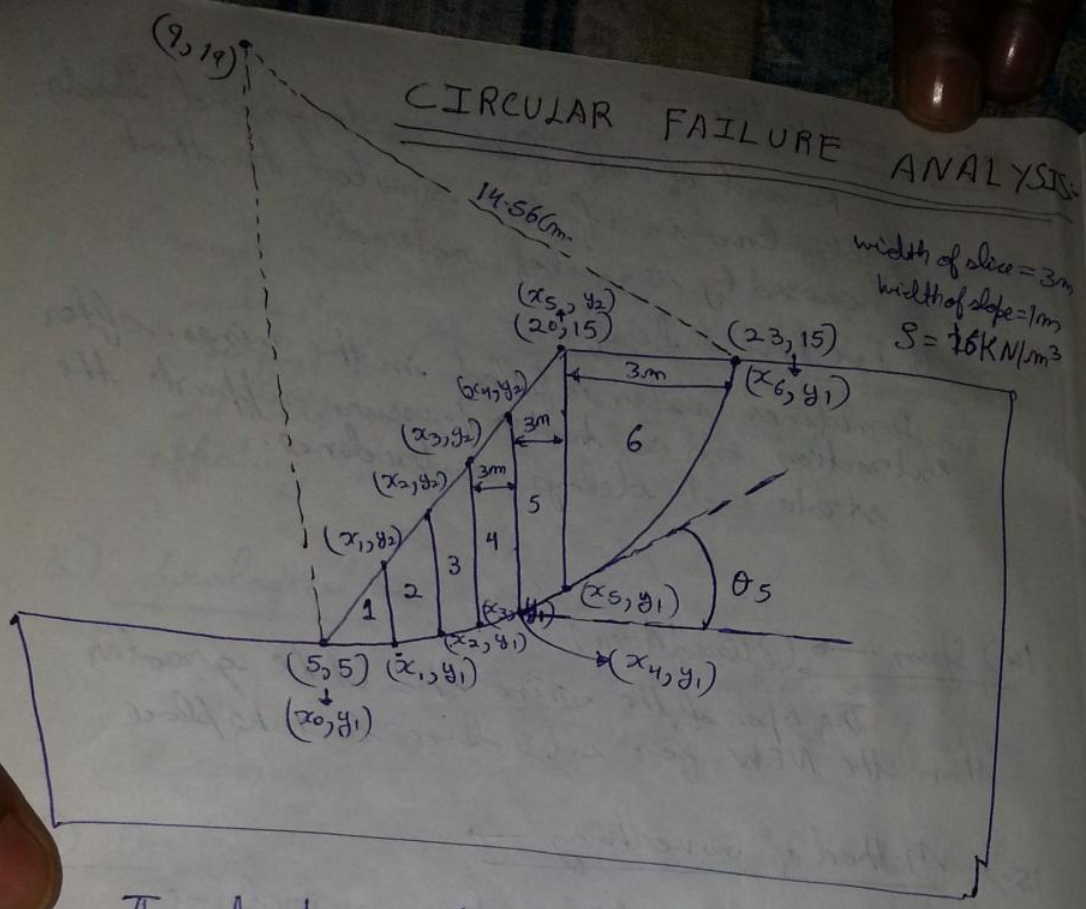
CIRCULAR FAILURE ANALYSIS:-



The dump usually fails in a circular manner. For analysing the failure in a circular mode, we can divide the entire failure block into a number of ~~wedges~~ Blocks. The higher the number of divisions, greater is the ~~no~~ accuracy of our estimation.

The Statement of our problem involves analysis of failure along a circular arc which is part of a larger circle with center at (9, 19). The coordinates (x_0, y_1) and (x_6, y_1) are given as (5, 5) and (23, 15). The $c = 40 kPa$

Angle of friction $\phi = 20°$

The equation of the circle of which the failure arc is a part is:-

$$(x-9)^2 + (y-19)^2 = (14.56)^2$$

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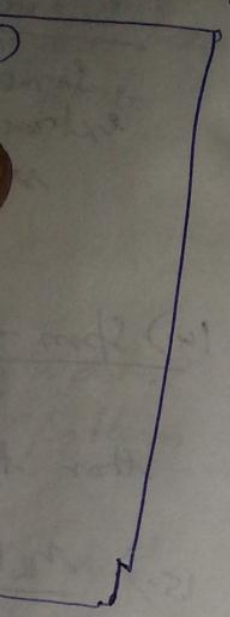
E ANALYSIS:

width of slice = 3m
width of slope = 1m
$\rho = 16 KN/m^3$



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It is used to determine the coordinate y_1 for $x_0 = x_0$ to $x = x_6$

$x_0 = 5$	
$x_1 = 8$	$y_1 = 5$
$x_2 = 11$	$y_1 = 4.47$
$x_3 = 14$	$y_1 = 4.59$
$x_4 = 17$	$y_1 = 5.32$
$x_5 = 20$	$y_1 = 6.83$
$x_6 = 23$	$y_1 = 9.46$
	$y_1 = 15$

The equation of the ~~straight line joint~~ slope face is given by:-

$$(y-5) = \left(\frac{15-5}{20-5}\right)(x-5)$$

which can be used to calculate y_2 for $x = x_1$ to $x = x_5$

$x_1 = 8$	$y_2 = 6.98$
$x_2 = 11$	$y_2 = 8.96$
$x_3 = 14$	$y_2 = 10.94$
$x_4 = 17$	$y_2 = 12.92$
$x_5 = 20$	$y_2 = 15$

The length of the circular arc in each slice is approximately (Base of each slice) :-

$$l_1 = \sqrt{(8-5)^2 + (4.47-5)^2} = 3.046$$

$$l_2 = \sqrt{(11-8)^2 + (4.59-4.47)^2} = 3.002$$

$$l_3 = \sqrt{(14-11)^2 + (5.32-4.59)^2} = 3.087$$

$$l_4 = \sqrt{(17-14)^2 + (6.83-5.32)^2} = 3.358$$

$$l_5 = \sqrt{(20-17)^2 + (9.46-6.83)^2} = 3.989$$

$$l_6 = \sqrt{(23-20)^2 + (15-9.46)^2} = 6.300$$

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$\therefore$ Area of the base of each slice given by:-

$A_1 = l_1 \times 1m = 3.046 m^2$

$A_2 = l_2 \times 1m = 3.002 m^2$

$A_3 = l_3 \times 1m = 3.087 m^2$

$A_4 = l_4 \times 1m = 3.358 m^2$

$A_5 = l_5 \times 1m = 3.989 m^2$

$A_6 = l_6 \times 1m = 6.360 m^2$

Volume of each slice can be found out by using the formula for volume of a trapezoid

$$Vol = \frac{1}{2}(\text{Sum of } \parallel l \text{ Sides}) \times (\text{distance between } \parallel l \text{ sides})$$

$\therefore V_1 = \frac{1}{2}(3)(6.98 - 4.47) \times 1m = 3.765 m^3$

$\therefore V_2 = \frac{1}{2}(3)(6.98 - 4.47 + 8.96 - 4.59) \times 1m = 10.32 m^3$

$\therefore V_3 = \frac{1}{2}(3)(10.94 - 5.32 + 8.96 - 4.59) \times 1m = 14.985 m^3$

$\therefore V_4 = \frac{1}{2}(3)(12.92 - 6.83 + 10.94 - 5.32) \times 1m = 17.565 m^3$

$\therefore V_5 = \frac{1}{2}(3)(15 - 9.46 + 12.92 - 6.83) \times 1m = 17.448 m^3$

$\therefore V_6 = \frac{1}{2}(3)(15 - 9.46) \times 1m = 8.31 m^3$

Slice
1
2
3
4
5
6

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∴ weight of each block/slice is given by $\boxed{W = V \times \rho}$

∴ $W_1 = 60.24 KN$

∴ $W_2 = 165.12 KN$

∴ $W_3 = 239.76 KN$

∴ $W_4 = 281.04 KN$

∴ $W_5 = 279.12 KN$

∴ $W_6 = 132.96 KN$

∴ slope of ~~each~~ r^{th} slice is given by:-

$$\tan\theta_r = \frac{y_1(x_r) - y_1(x_{r-1})}{3}$$

∴ $\theta_1 = -10.02°$

∴ $\theta_2 = 2.29°$

∴ $\theta_3 = 13.68°$

∴ $\theta_4 = 26.71°$

∴ $\theta_5 = 41.24°$

∴ $\theta_6 = 61.56°$

Slice	Cohesive forces (CA) (KN)	frictional forces ($W_r \cos\theta \tan\phi$) (KN)	W sin θ (KN) driving force	Total resisting force (KN) [CA + W cosθ tanφ]
1	121.84	21.579	-10.668	143.419
2	120.08	60.059	-6.02	180.139
3	123.48	84.662	58.125	208.142
4	134.432	91.375	126.32	225.695
5	159.56	76.404	183.963	165.964
6	252	23.047	116.914	275.047
			480.674 KN	1198.406 KN

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$$\therefore FOS = \frac{\text{Total resisting force}}{\text{Total driving force}} = \frac{\text{Shear strength}}{\text{Shear stress}}$$

$$\therefore FOS = \frac{\sum (CA_r + W_r \cos\theta_r \tan\phi)}{\sum W_r \sin\theta_r}$$

where $r = 1, 2, 3, 4, 5, 6$

$$\boxed{\therefore FOS = \frac{1198.406}{480.674} = 2.49}$$

GSPS Sir

12/09/2013.

Support Design → (Continued)

- Therefore the design of support system for tabular excavations should take the following factors into account.

  - Existence of geological weakness in the roof strata
  - Strength properties of rock.
  - Extent of tensile zone
  - Stresses acting on the abutments of productive excavations.

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1) Geological weakness — Joints, Cleavage, Slip

2) Strength — tensile, shear

⇒ whenever we are trying to reinforce the rock we actually increasing its tensile or shear strength depending upon the mechanism of failure the strata is expected to undergo.

Types of Support.
- Local support → The zone of influence of the support is very localized.
- regional support → The zone of influence of the support is to a very large extent. (both lateral and vertical extent)

All artificial supports that are put in mines, by virtue of its properties they are local supports

while a pillar (ex. barrierpillar) they are regional support because their zone of influence is ~~larger~~ to a larger extent

good filling material → regional support

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Local Support.

→ focus is the immediate roof; domain is small

- The sphere of influence of local support is usually restricted to a volume of few cubic metres.

- The need for local support is dictated by the quality of the immediate roof in a productive excavation.

  — often installed as a temporary support and ~~at~~ removed once its main function to protect men and equipment from local roof fall ~~cases~~ ceases

  — To get maximum effectiveness, these supports are required to be installed soon after exposure of the roof.

ex.) Rock bolts, timber props, hyd. props, cogs, chocks, etc.

The contribution of immediate roof in filling the goaf is small so the domain of local support is small but our focus is on supporting immediate roof because by supporting it we can control the failure of the entire strata.

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Regional Support.

- There are circumstances when there is a presence of a competent strata in roof rock in small mining depth and the thickness of the bed is more.

— There could be a situation where this competent layer of rock is required to fail in a stable and controlled manner at a given mining span. If this is not possible, the roof strata is required to be supported by a regular system of pillar (yield pillar). These pillars are designed in such a manner that they do not fail regardless of the extent of mining.

— By the provision of yield pillar the strata will fail but in a controlled manner providing safety against the entity to be addressed (i.e. uncontrolled failure)

— Under such conditions, the pillars so designed perform the function of regional support.

— Similarly filling out the goaf area with waste rock is also a regional support

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Depending on time for which the supports are designed

1.) Temporary support — Their actual life, usefull time is very very small

2.) Permanent support — once installed, they can be left at the place forever.

Temporary Vs Permanent Support:-

- ~~Pe~~ Temporary supports are exclusively local support, ~~designed to be~~ required only for a short period of time ex.) Hyd. Prop, ~~powered~~ Powered supports.

- Regional support are generally permanent support. Permanent supports are generally employed for regional stability reasons.

All ~~sup~~ withdrawable supports, irrespective of the load bearing capacity of the support are Temporary supports ex.) chock, log props.

roof bolts → permanent Support

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Depending on ~~mechanism~~ mode of action, supports can be classified as:-

1.) active supports

2.) Passive supports.

Passive Support

- Any support which is installed unstressed is regarded as Passive support. Such support begin to function only in response to strata movement.

- As the movement of unfractured rock is usually small, passive supports often become effective only when rock fails and undergoes some displacement around the excavation.

- Frequently, there is poor contact between the support and the rock, therefore, the potential benefit of passive support is hardly ~~realised until a~~ realised till a considerable time has elapsed allowing considerable movement of failed rock.

- The delayed in built up of support resistance results in considerable deterioration of mining condition which can be prevented by ~~the~~ use of Active Support.

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Active Support

- A support structure is active if it exerts and maintains a restraint on the rock immediately after installation

- Essential feature of an active support is that the provision of the initial support force does not depend on rock movement.

- Fall of roof or side is one of the major reasons (61%) of accidents in underground mines

- A large number (45%) of these accidents occurs in freshly exposed roof areas and the thickness of fall is generally less than 1 m (in 80% of such cases)

- Fall occurred in all types of roof with a higher rate of incident in coal or shale (weak roof) roof. Geological disturbances have been reported to be the main reason of such accidents in sandstone roof. (competent)

- A proper assessment of support requirement for different part of the working and its proper implementation is very important for better safety and productivity of mine workings.

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While supporting we should remember that 2 factors affect the level of supporting:-

1.) Type of support

2.) Time lag between exposure of fresh roof and installation of support — lesser is the lag better is the performance of the support.

Support design based on CMRI ~~ISM~~ ISM ~~RMR~~ RMR

• For development headings:-

$$P_r = \gamma B F \left(1.7 - 0.037\,RMR + 0.0002\,RMR^2\right) \text{ ton/m}^2$$

• For junctions:-

$$P_r = 5\gamma B^{0.3} F \left(1 - \left(\frac{RMR}{100}\right)^2\right) \text{ ton/m}^2$$

where :-

γ — unit weight of rock (t/m^3)

B — is the Bord width (m)

F — is the factor of safety, generally taken as 1.5

RMR is the average RMR of immediate roof after adjustment

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Q. Why F is used in the formula??

Ans:> γ and B are explicitly obtained and RMR is the subjective parameter (varies from person to person)

So F is used to magnify the ~~stress~~ support requirement in order to address those inherent restrictions developed ~~by~~ due to the subjectivity in determining the RMR value.

Parameters	Rating
Layer thickness	30
Structural features	25
Rock weatherability	20
Intact rock Compressive strength	15
Ground water seepage	10

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Support design in depillaring workings:-

· Slice junction

$$SLD_{sn} = \frac{\gamma H^{0.5} K^{0.64} W^{1.17}}{R^{0.90}} \ t/m^2$$

· Slice

$$SLD_{sl} = \frac{\gamma H^{0.67} K^{0.84} W^{1.74}}{R^{1.42}} \ t/m^2$$

· Split gallery

$$SLD_{sp} = \frac{\gamma H^{0.52} K^{0.59} W^{1.12}}{R^{1.02}} \ t/m^2$$

· Goaf edge

$$SLD_{ge} = \frac{\gamma H^{0.54} K^{0.49} W^{0.89}}{R^{0.79}} \ t/m^2$$

H → depth of working
W → width of gallery
K → stress ratio
R → RMR
γ → (t/m^3)

SLD → Setting load density (t/m^2)

SLD is defined as amt of load density that the support should provide at the point of ~~suppo~~ setting

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Q-) Assignment - III

Estimate the setting load density for a depillaring working under following geotechnical condition.

RMR of the immediate roof = 45

width of gallery = 4.5m

Stress ratio = 1.2

Cover depth = 240m

13/09/2013

GSPS Sir

In 99% of longwall faces, caving is used as goaf control method.

But we can have:-

1.) Caved longwall
2.) Stowed longwall

The challenge of ground control is in caved longwall cases.

Caving Mechanism/Caving Mechanics

Q-) Estimate the span of first local fall, main fall, subsequent local fall and periodic ~~giving~~ caving span in a longwall working with a face length of 150m planned to be operated under set of strata condition as given below:-

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layer-3 Fine grained Sandstone, flex. strength = 5MPa, 3m thick, density = 2500 kg/m³

layer-2 Fine grained sandstone, 8m thick, flex. strength = 5MPa, density = 2500 Kg/m³

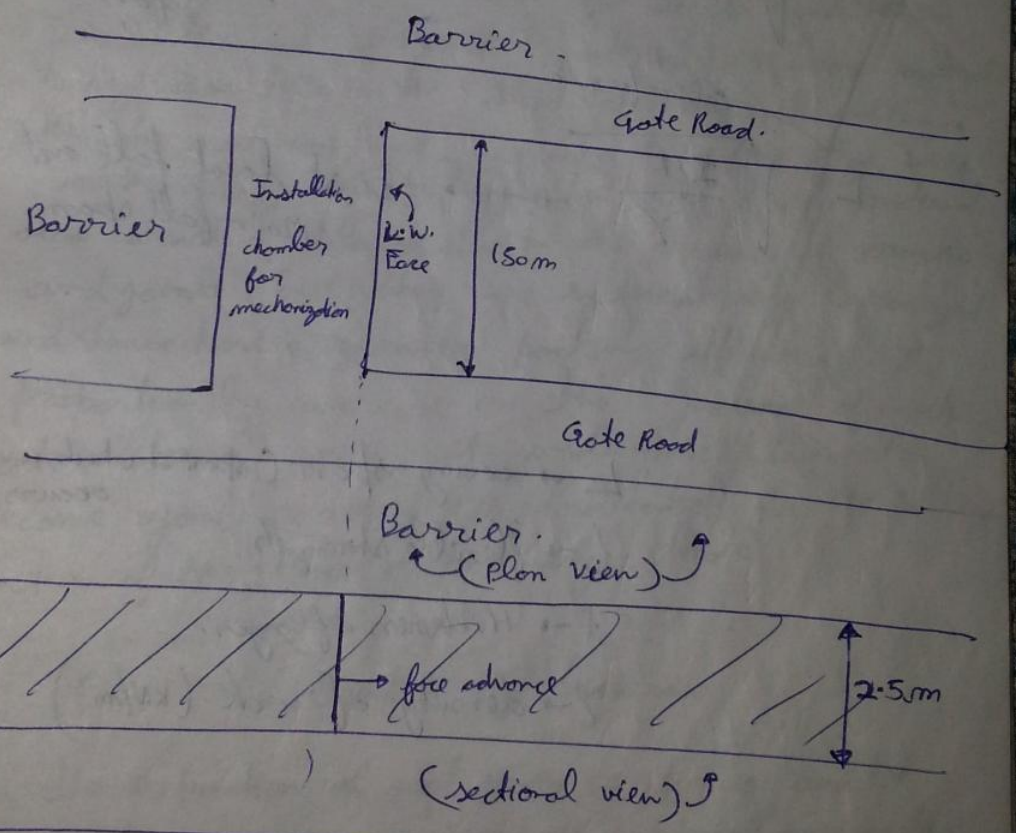
layer 1 Shaly sandstone, 1m thick, flexible strength = 3MPa, density = 2500 kg/m³

Coal seam, 2.5m, full thickness, density = 1400 Kg/m³ extraction

⟹ average cover depth of seam = 200m

(Bulking factor of rock) K = 1.25

sol.



To estimate the ht. of caving zone:-

$$t = \frac{h}{K-1}$$

t → caving height
K → bulking factor of overlying strata
h → extraction ht.

K → bulking factor

GOAF and GOB are same

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~~After extraction of coal seam as the span increases the overlying strata is going to fail and it will become a goaf/gob material.~~

~~The volume of the rock in the goaf will be greater than the the rock in the roof.~~

To estimate spans of roof fall:-

~~First local fall span.~~

cantilever formula

$$L = \sqrt{\frac{ft}{3\gamma}}$$ → subsequent local fall span and periodic caving span

Beam formula

$$L = \sqrt{\frac{2ft}{\gamma}}$$ → First local fall and Main fall span.

L → caving span (span at which caving occurs)

f → flexible strength

t → thickness of layer.

γ → density of rock (KN/m^3)

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The support requirement of longwall with caving is quite complex compared to that required in stowing. The challenge of ground control is in caved longwall cases.

On extraction of the coal seam there will be redistribution of stresses in overlying strata which will cause displacement of the overlying strata and hence as the span increases the overlying strata will undergo failure at some point of time. The strata on failure will fill the void created due to mining. The volume of rock of the goaf material will be larger than the volume of insitu rock. This is because the goaf material is a loose material having voids between the blocks of material and hence they are not compact compared to when they were not fragmented. The insitu material also consists of blocks due to the presence of cracks and joints but they are systematically arranged and hence have a greater packing efficiency. The proportional increase in the volume of insitu rock material when it undergoes failure to become goaf material is indicated by the Bulking factor of the rock of overlying strata.

$$V_{goaf} = K\, V_{insitu\ rock}$$

As the extraction of coal seam continues and the span of mining increases a point comes when the overlying strata fails. This is the time of occurance of the first local fall. This fall involves the failure of the immediate roof. So the first local fall is the process of failure of the immediate layer (layer 1) for the first time. The mining span which is responsible for the first local fall is called the span of first local fall.

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The occurance of first ~~mining span sta~~ local fall takes place at the center of the mining span and the heap of goaf material is formed at the center of the mining span.

~~As the~~ ~~spa~~ On further advancement of the face the occurrance of the next local fall takes place and it is called ~~sequen~~ subsequent local fall. ~~And its Span~~ And the span for subsequent local fall can be given as.

$$\text{span for subsequent fall} = \left(\text{span for immediate fall}\right) - \left(\text{span for previous fall}\right)$$

Ex.) Suppose the first local fall occurs at 15m and subsequent local fall occurs at 20m of span then the span for subsequent fall is $(20-15) = 5m$.

~~But the failure~~ The subsequent local falls also involve failure of the immediate roof rock. But the volume of material dislodged due to fall of the immediate roof is not capable to fill the entire void created by mining. So the second layers (of overlying strata) also fails and starts Caving and as ~~the second layer~~ other layers of overlying strata fails, ~~Completely~~ the void is ~~comp~~ entirely filled by goaf material. The point when the entire goaf is filled up by goaf material / gob material is the point when main fall has occurred. The span of mine (at which there is the occurance of the main fall is called the main fall span. The main fall mainly involves the failure of the second layer which is also called the main roof. The main fall hence involves the failure of more than one layers of the overlying strata.

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The second layer (which is 8m thick) is called the main roof because it has a dominent influence in characterising the caving mechanism.

~~Failure mechanism of Main roof:-~~

Mainfall → It is the occurance of caving upto the point, the goaf is completely filled.

Failure mechanism of Main roof:-

The main roof bends due to its weight and in addition due to loading by the third layer. The third layer is called the loading roof/strata/layer.

→ To understand the above take the example of 2 beams one thin, one thick, restricted at its ends ~~On application of~~ and free to bend. On application of load on the 2 beam system the thin beam bends more compared to the thicker beam. This is because the bending stiffness of thicker beam is more compared to the thinner one. Due to this differential bending of the two beams the thicker beam is additionally loaded.

Hence due to the excessive bending of layer 3 of the overlying strata, this additional bending further induces load on the main roof causing its failure.

Hence the failure of Main roof is due to
1.) Bending by virtue of its weight.
2.) Loading due to additional bending of the 3rd layer (loading roof)

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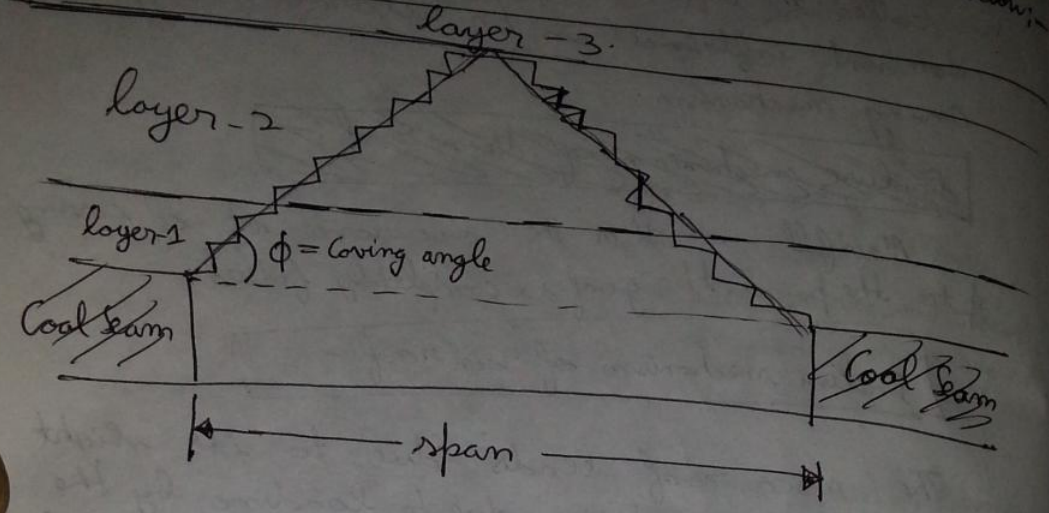
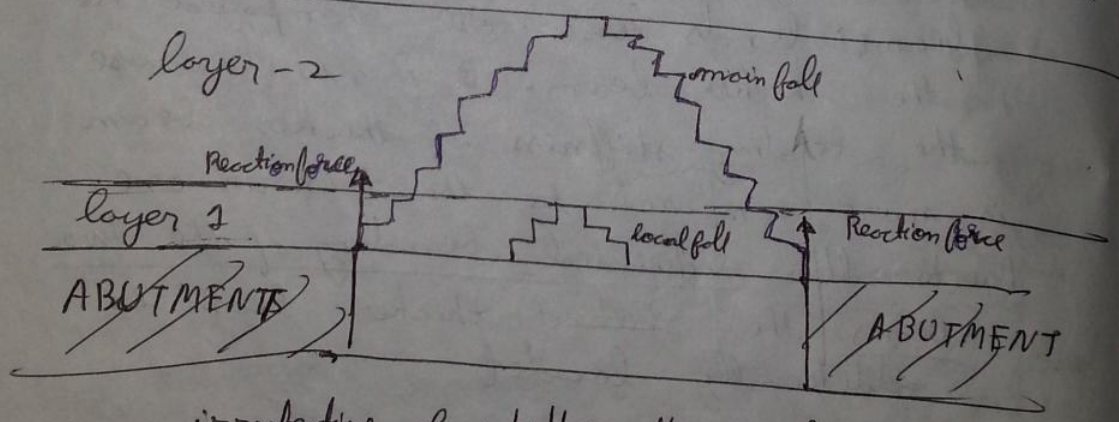
The caving profile of the roof is shown below:-



To estimate the span for first local fall and Main fall:-

$$L = \sqrt{\frac{2ft}{\gamma}}$$

This is called the Beam formula because



irrespective of whether the roof is undergoing first local fall or main fall, the overlying strata behaves as a beam unsupported at the Center (as the underlith material is excavated) and restricted at the corners or boundaries as well as supported at the Corners by abutments (reaction forces). Hence the overlying strata behaves as a restricted as well as simply supported beam.

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To estimate the span for subsequent local fall / periodic caving span :-

$$L = \sqrt{\frac{ft}{3\gamma}}$$

This is called the cantilever formula because the occurance of the first fall creates a free face / end and the overlying strata behaves as a cantelever beam supported and restricted / clamped at one end and free to move at the other end.

A/Q :-

Span for 1st local fall = $\sqrt{\frac{3 \times (3 \times 10^6 \, Pa) \times 1 \, m}{2500 \, kg/m^3 \times 10 \, m/s^2}}$
(involves failure of immediate roof)

Span for subsequent local fall = $\sqrt{\frac{(3 \times 10^6 \, Pa) \times 1 \, m}{3 \times 2500 \, kg/m^3 \times 10 \, m/s^2}}$
(involves failure of immediate roof)

$= 6.32 \, m$
$\approx 6 \, m$

Span for Main fall = $\sqrt{\frac{3 \times (5 \times 10^6 \, Pa) \times 8 \, m}{2500 \, kg/m^3 \times 10 \, m/s^2}}$
(involves failure of Main roof)

$= 69.28 \, m$
$\approx 69 \, m$

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Solution of Question by GSPS Sir given on 12/09/2013

~~SLD_JN =~~

$R = 45$
$W = 4.5\,m$
$K = 1.2$
$H = 240\,m$ (given)
$\gamma = 2.5\,t/m^3$

$$SLD_{JN} = \frac{\gamma H^{0.5} K^{0.64} W^{1.17}}{R^{0.9}} = 8.22\ t/m^2$$

$$SLD_{sp} = \frac{\gamma H^{0.67} K^{0.84} W^{1.74}}{R^{1.42}} = 7.05\ t/m^2$$

$$SLD_{sp} = \frac{\gamma H^{0.52} K^{0.59} W^{1.12}}{R^{1.02}} = 5.34\ t/m^2$$

$$SLD_{ge} = \frac{\gamma H^{0.54} K^{0.49} W^{0.89}}{R^{0.79}} = 9.94\ t/m^2$$

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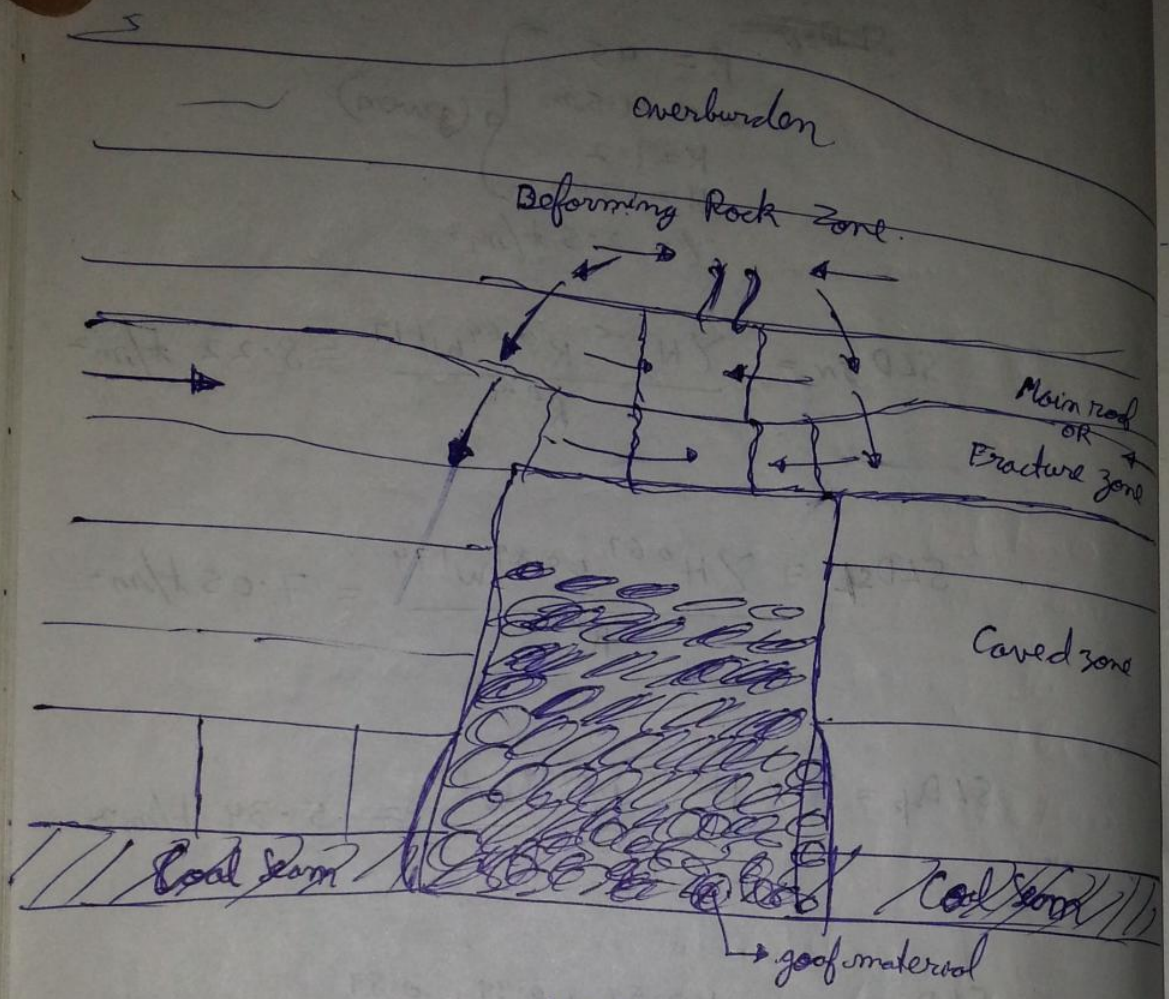
14/09/2013.

GSPS Sir



we have a ~~tabular~~ tabular coal seam and above the deposit we have various layers.

~~The~~

The mineral deposit has a stress domain and by the time, mineral is excavated we have a redistribution of stress and we have a stress relaxed zone. All the layers which are within the stress relaxed zone are free to cave or deform downward under the effect of gravity. The layers undergo bending and there are zones of tensile stresses. when tensile stress > tensile strength

then the immediate roof fails, and as the excavation span is increased more and more tensile stresses develop which propagate the process of failure initiated earlier. Then the failure is increased to such an extent that the overlying strata is no longer able to remain in place and it caves.

Then as the mining span increases the main roof also involves in caving with the same mechanism (bending → development of tensile stresses at critical points → tensile failure followed by caving)

Condition of roof during 1st local fall:-

As the span is increased, at a particular span of mining the roof of the mine behaves as a beam restricted at 4 ends (2 ends across the face and 2 ends along the face) and traction free at the center. The rate of increase of span affects the caving mechanism.

Tensile stresses develop on the lower surface of the immediate roof and (max. tensile stresses develop at) corners across the face and as the span increases, the immediate roof fails. This is the first local fall. As the span further increases the subsequent local falls take place but since after the ocurrence of the first local fall the immediate roof looses its continum

and hence now behaves as a different mechanical structure called Cantilever. Hence the immediate roof behaving as a cantilever is involved in subsequent local falls.

As the span is increased the main roof comes into picture. The main roof is not involved in first local falls and subsequent local falls. The main roof also fails in the same manner as the immediate roof. It behaves as a beam during the first failure of the main roof (i.e. during the main fall) and behaves as a cantilever as the periodic caving process takes place.

The main roof is involved in

1.) main fall

2.) periodic caving process

first local fall (immediate roof) / main fall (main roof)

$$L_B = \sqrt{\frac{2ft}{\gamma}}$$

subsequent local fall (immediate roof) / periodic caving (main roof)

$$L_c = \sqrt{\frac{ft}{3\gamma}}$$

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$f \rightarrow$ flexible strength / bending strength of layer involved in failure

$t \rightarrow$ thickness of layer involved in failure

$\gamma \rightarrow$ unit weight of layer involved in failure

But the main roof is bended by its own weight as well as due to differential loading / bending from the loading roof. So while using formula for main fall and periodic caving span in place of γ we will use

γ_e (effective value) involving the effect of main roof and loading roofs.

$$\gamma_e = \frac{E_1 t_1^2 \left[\gamma_1 t_1 + \gamma_2 t_2 + \cdots \right]}{E_1 t_1^3 + E_2 t_2^3 + \cdots}$$

$E \rightarrow$ Young's modulus

$E_1 \rightarrow$ young's modulus of main roof

$E_2, E_3 \cdots \rightarrow$ young's modulus of loading roofs.

But in case the main roof is thinner than the loading roof then there is no loading by the layer above the main roof so we will not use γ_e but only γ of the main roof for main fall span and periodic fall span.

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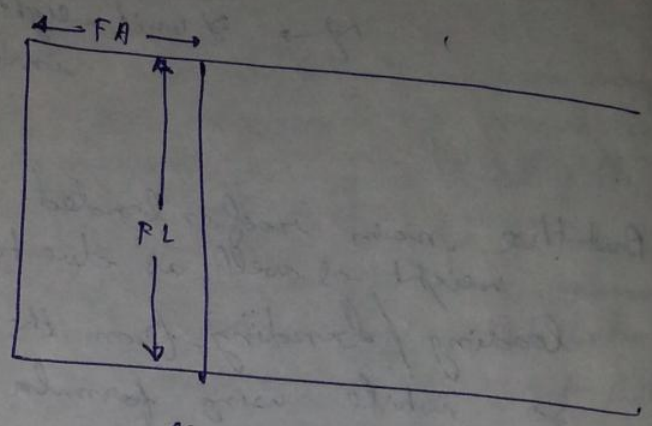
Plate formula:-

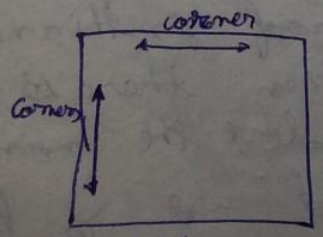
We observe that during the occurence of the first local fall, the span of face advance is lesser than the face length



Taking the smaller length as a and longer length as b

$\frac{b}{a} \geq 2 \rightarrow$ roof behaves as an elastic beam and the classical formula for elastic deformation of beam can be used.

$1 < \frac{b}{a} < 2 \rightarrow$ The roof behaves like a plate and plate formula is used to explain failure of roof



Plate

The plate formula says that the max. stresses developing along the corners of the plate depends on an empirical function β which is a function of b/a

$$\beta = f(b/a)$$

$$\sigma_{max} = \frac{6\beta \gamma_e a^2}{t}$$

γ_e is used in case upper layer is also involved in loading of roof.

When σ_{max} exceeds the flexible strength/bending strength of the layer then the layer becomes unstable and is bound to undergo failure.

b/a	1	1.25	1.50	1.75	2	>2
β	0.0513	0.0665	0.0757	0.0806	0.0829	0.0833

Beam Plate formula:-

Closed form solution based on Beam and Plate ~~formula~~ theory.

$$\sigma_{max} = \frac{6\beta \gamma_e a^2}{t}$$

$$L_B = \sqrt{\frac{2\sigma_t t}{\gamma_e}} = \sqrt{\frac{2ft}{\gamma_e}}$$

$$L_C = \sqrt{\frac{\sigma_t t}{3\gamma_e}} = \sqrt{\frac{ft}{3\gamma_e}}$$

Caving Mechanism ~~of~~ in Stratified rock formations:-

~~Bulking factor controlled caving~~

Bridge Bed → Strata with a high elasticity showing perfectly elastic deformation without fracture of the bed. ex:) Sill

(i) Bulking factor Controlled Caving → The goaf material dislodged from the overlying roof strata fills in the goaf completely thereby proving substantial amount of support to the bridge bed. Subsequently it ~~provides~~ facilitates a controlled caving and ~~the~~ simplifies the support requirement at the longwall face.

??

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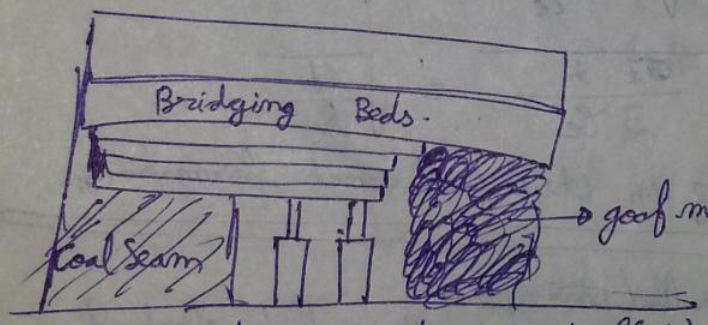
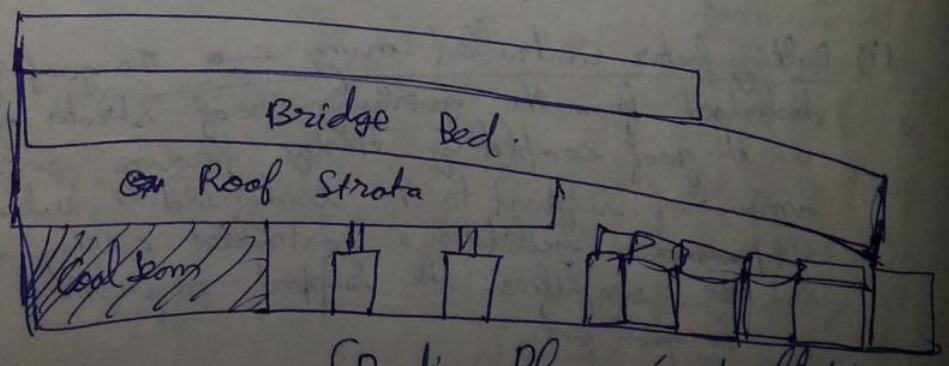
Parting plane Controlled Caving → The quantum of goaf material is insufficient ~~to fill~~ to fill the goaf completely. So There is hardly any contribution from the goaf pile to support the bridging bed. Hence there is significant ~~deformation~~ bending of the bridge bed until it is supported either by the goaf pile or by the floor. This deformation of the bridging bed with the advance of the longwall face is uncontrollable. The rate of deformation of bridging bed with face advance is not uniform. ~~[illegible]~~ So with increasing deformation the load on immediate roof increases (with induction of higher tensile stresses) which further increases the load on the supports at the longwall face. Due to non uniform loading of the supports the role of support ~~at~~ has to be justified at a point of time. This makes the support design ~~at~~ the face very complex depending on the requirement at the face.



(Bulking Factor Controlled Caving.)



(Parting Plane Controlled Caving.)

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Factors affecting Cavability:-

1.) Area of excavation → Area of exposure of goaf is essential for caving of the roof layer.

2.) Thickness of the layers → More is the thickness, difficult it becomes to undergo caving.

3.) Bending strength of layer → more the bending strength, more difficult it becomes to cave.

4.) Depth of Cover → Greater depth results in easier cavability (because vertical stress increases and augments the caving process)

5.) Horizontal stress → with its increase, the intrinsic strength of rockmass increases as it increases confinement of layers, So it impedes the caving process.

Note :- Horizontal stress has an effect on first local fall (increases span of fall) and Mainfall because roof behaves as a beam but does not affect subsequent local fall and periodic caving span because roof behaves as a cantilever.

6.) Presence of Joints and parting planes → ~~more the~~ Higher density of joint planes in overlying strata, the effective strength of rockmass decreases and hence caving ~~pro~~ characteristics increases (it augments caving process)

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Empirical Model:-

$$I = \frac{\sigma L^{m} t^{0.5}}{5}$$

(caving ability index)

(σ → compressive strength (kg/cm²))

(We can estimate the caving ability of the rock bed)

L = average core length of rock bed, cm

t = thickness of the rock bed, m

$1.1 < m < 1.2$ (depends on the RQD of the layer)

For RQD < 80 $m = 1.1$

else $m = 1.2$ for RQD ≥ 80

To understand the overall caving behaviour of strata above the coal seam :-

1.) Evaluate caving index of each and every rock bed lying between the caving zone.

2.) Find the rock bed of max. I value. This is the strongest bed. (I_{max})

3.) Put I_{max} in next formula to obtain span of main fall with accuracy.

$$S_{mw} = 0.72\, I_{max}^{0.51}$$

(S_{mw} → Span for main fall (m))

4.) The periodic caving span can be evaluated by the next formula:-

$$S_{pw} = 3.05 + 0.25\, S_{mw}$$

(S_{pw} → span for periodic fall (m))

(S_{mw} (m) → main fall)

Sl.	Rock Bed	Rock Characteristics	Thickness (m)	RQD (%)	Avg. length of core	σc kg/cm²	I
	1	Cg MgSst	4.4	98	24	210.68	4005
	2	Interdiction	0.4	79	17	411.13	1558
	3	Cg Sst	3.8	93	22	269.17	4284

We are moving upward from

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Q.)

We are moving upward from the Coal Seam

Rock Bed	Rock Characteristics	Thickness (m)	RQD (%)	Avg. length of core	σ_c kg/cm²	I
1	Cg Mg SSt	4.4	98	24	210.68	4005
2	Intercalation	0.4	79	17	411.13	1558
3	Cg fg SSt	3.8	93	22	269.17	4284
4	Intercalation	0.76	97	23	411.13	3084
5	Cg Fg SSt	12	87	20	293.91	$\boxed{7413}$ I_{max}
6	Coal	0.67	not ~~possible~~ Possible to find ~~proper~~ RQD			0
7	laminated bed of shale and intercalation	3.78	80	18	362.29	4520
8	Sandstone	2.4	97	23	439.16	5859
9	Laminated bed of shale and ~~Sandstone~~	3.8	84	19	377.11	5034
10	Sandstone and intercalation	3.9	79	17	396.45	4691

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Roof Category	I_{max}	Expected Caving behaviour
Cat I	≤ 2000	Easily Caving
Cat II	$2000 < I_{max} \leq 5000$	Moderately Caving
Cat III	$5000 < I_{max} \leq 10000$	Cavable with difficulty
Cat IV	$10000 < I_{max} \leq 14000$	Cavable with substantial difficulty
Cat V	$I_{max} > 14000$	Cavable with extreme difficulty

~~The problem of caving is to cavity formation~~

Maximum Convergence (mm/m of face advance) (between roof and floor)	Expected Roof Condition
< 60	Continuity of roof remains intact
60–100	Minor cracks and breaks in the exposed face area
100–160	Prominent fractures for lower range, rock fall occuring at higher range
> 160	Rock fall leading to face collapse

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Ans:)

The roof is of category III as our $I_{max} = 4/3$

Difficulty in Caving → Cavity formation will be there. The roof will try to converge and the support will not be able to bear the convergence of the roof strata at the face and will allow large deformation of the roof at the face without failure. So the roof will develop cracks and fractures at this stage.

16/09/2013.

BKS Sir

H.W.

Methods of Subsidence measurement (Submition 18/09/2013)

Damages due to Subsidence:-

1.) Buildings → Most earth structures are weak in tension. Due to tensile strain the walls are likely to crack. If building is reinforced then damage is less due to a higher tensile strength of iron. If the building are earthen structures then they very easily break up.

Differential ground movement in vertical movement also causes damage of building.

2.) bridges → Horizontal tensile strain leads to the damage of bridges.

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3.) Roads and airport runways → Cracks and potholes develop in roads due to subsidence vertical movement of land.

4.) Rail lines → Generally in Indian condition, mining below rail lines is not allowed. Due to subsidence land gradient changes leading to non feasibility for rail lines. A gradient greater than 1 in 300 in not fit for railways.

Remedial measures for mitigating subsidence:-

I.) Partial mining → Techniques involve:-

1.) Development only

2.) Wide stall method → Pillars extracted in L-shape leading to widening of the gallery and the remaining pillars is left intact. The maximum permissible widening of gallery is upto (8 - 10) m. The roof of the gallery is supported by roof bolting, etc. The amt. of widening of gallery depends on the roof condition.

3.) Splitting → Splitting of pillars into 2, 4 stooks is done but the danger is that the size of pillars is reduced to a large extent so their supporting strength reduces so wide stall is preferred but in case of shallow workings splitting can be done.

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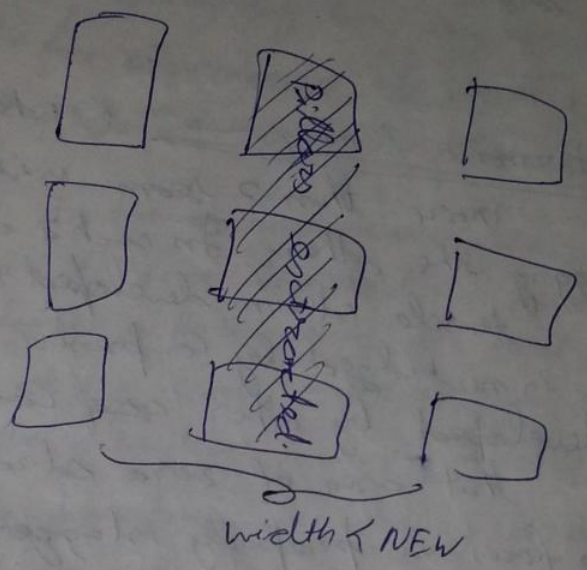
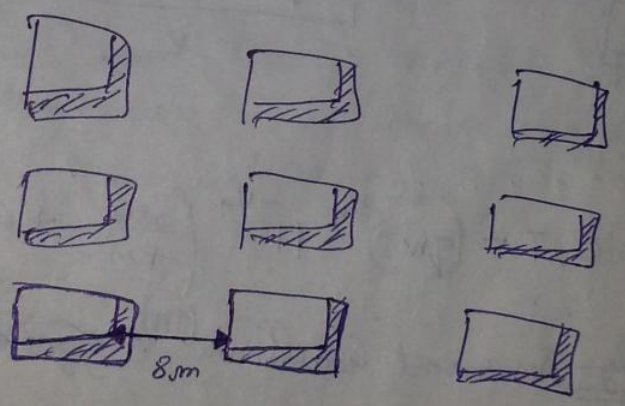
4.) NEW → A row of pillars is extracted such that the width is less than the NEW to reduce chances of subsidence, but it was not an effective method so discarded. Alternate row are mined out.





Wide - Stall Method.

III.) Back filling →

1.) Material taken from underground followed by filling using sand stowing. (common in coal mines)

2.) Many metal mines take the Mill tailings and use it for filling purpose in method of cut and fill mining.

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3.) fly ash → It is also used for backfilling but flyash retains water and so remains muddy and fleshy and so now flyash is made into pellets (by mixing with cement, etc) on the surface and send with water.

III.) Harmonic Mining → when we have more than 2 seam with one face logging the other. In such a case the tensile strain developed by one face is neutralized by compressive strain developed by other ~~coal~~ coal face so that area of zero strain is developed. The faces are properly staggered for this purpose.

19/09/2013.

GSPS Sir

$$C_vT = (qw)^{3/4} H^{-1/4} \left(\frac{6800}{PM} + 66\right)$$

Given by → Josien and Gouillaux (1978) ~~suggested that~~

C_vT → Convergence parameter

PM → function of the support resistance

q → parameter associated with goaf treatment.

w → working thickness of seam

H → ~~Cover~~ Cover depth

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Josien and Gouillaux (1978) suggested that a relation between the desired load bearing capacity P, per meter length of face may be obtained using the following equation, by limiting the face convergence to its threshold value, C_{vo} of 40mm/m of face advance.

where C_vT is the average convergence of the walls in the face expressed in meter per m advance of face; w is the working thickness of the seam expressed in m $(0.8 \leq w \leq 3)$; q is the subsidence factor, 2 for caving and 0.6 for ~~pneu~~ pneumatic stowing and 0.15 for ~~hydrau~~ hydraulic stowing, H is the depth of the mine in m $(100 \leq H \leq 1000)$, PM is the load bearing capacity of the supports in tonnes/linear m of the face $(20 \leq PM \leq 260)$

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Estimation of support requirement in longwall panel:-

$$C_m = \left(\frac{A}{P} + 9.6h\right) + \left(\frac{KI}{K'+1.5} - 23\right)$$

Where :-

- h → extraction height
- C_m is the maximum convergence at face (mm/m)
- P is the mean load density (t/m^2)
- I is the cavability index of stronger bed
- A is a ~~constant~~ ~~depending~~ on rock type is 1440 in Cat I and II, 1700 for Cat III and IV and 1900 for Cat V rocks
- K' is a factor depending on the ratio of thickness of cavable bed in between the strong bed ~~and the roof rock~~ extraction height causing the weighting and the coal seam, and the extraction height; its value is 2 for ratio upto 2, 3 for ratio between 2 to 4 and 5 for ratio between 4 to 8 and 10 for values above 8, ~~K is 0.025 for sandstone~~

K is 0.025 for sandstone (strong beds)

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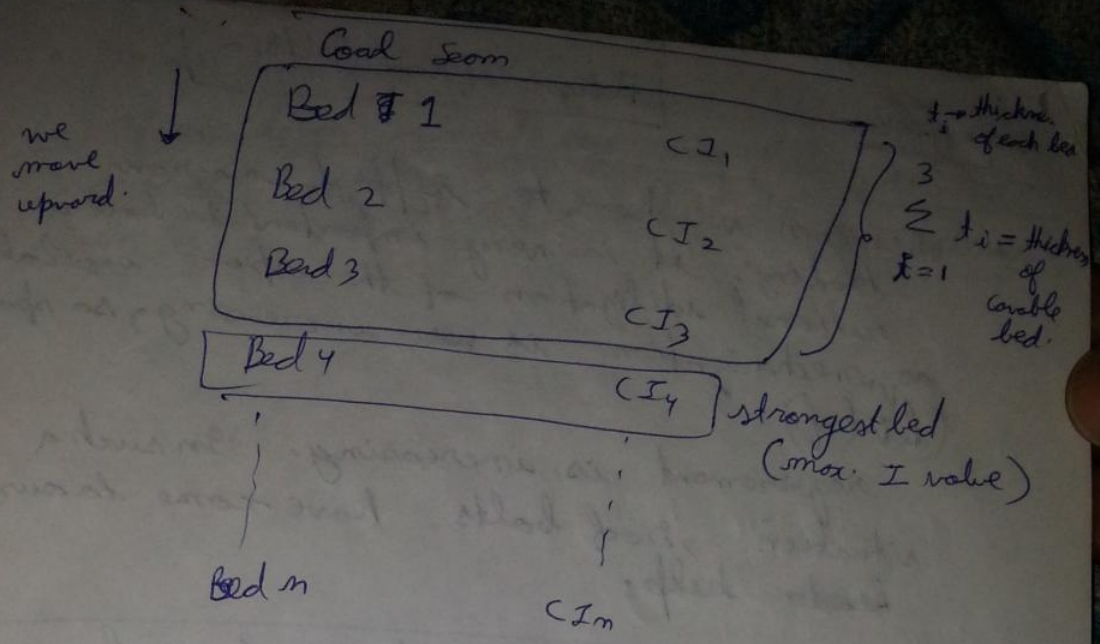

For all practical purposes → the convergence at the face is limited to 60 mm/m (permissible maximum convergence)

⇒ Then we can obtain the support requirement. P

Then the value of P is magnified or enhanced by 30% to look into the leakages in hydraulic circuit of supports.

We observe that the support capacity specified by the ~~Com~~ manufacturer and the capacity at which supports are used in field ~~have~~ has a variation of around 30%.

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20/09/2013-

Design of Roof bolts:-

- Whenever we have to support narrow headings it is very important to have a rational utilization of the space available as mechanization is increasing, so space (level of) requirement is increasing. In such a situation, roof bolts have come to our help.

- If we are able to make use of roof bolting technology effectively then we can control all situations of roof failure.

  So we should have.

  1.) knowledge of roof bolt design

  2.) knowledge to select the correct roof bolt for the purpose

- Roof failure has been the primary concern in all kinds of U/G mines

- However for centuries, all support systems were passive and external. They were roof to floor supports, such as timber props or rib supports.

- These supports require a great amount of timber and constant maintainance, further more they are not effective in controlling roof stability

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- In an attempt to reduce the number of accidents caused by roof fall, USBM advocated the use of roof bolting technique in 1947

Advantages of roof bolting:-

- Reducing storage and material handling requirements
- Decreasing the size of opening that is needed to achieve the same given clearance (bolts provide less hindrance in v/s pathways so size of opening can be reduced so maintaince of opening becomes easier)
- Preventing any appreciable roof deformation by quick installation after opening is excavated.

The maximum intensity of deformation is in the green roof zone so it is necessary to install support as soon as possible, which is the case of roof bolts.

- Improving ventilation by lowering the resistance to the air via elimination of obstruction, such as cribs, posts, girders, etc.
- Providing greater freedom for trackless vehicles without risk of dislodging support.

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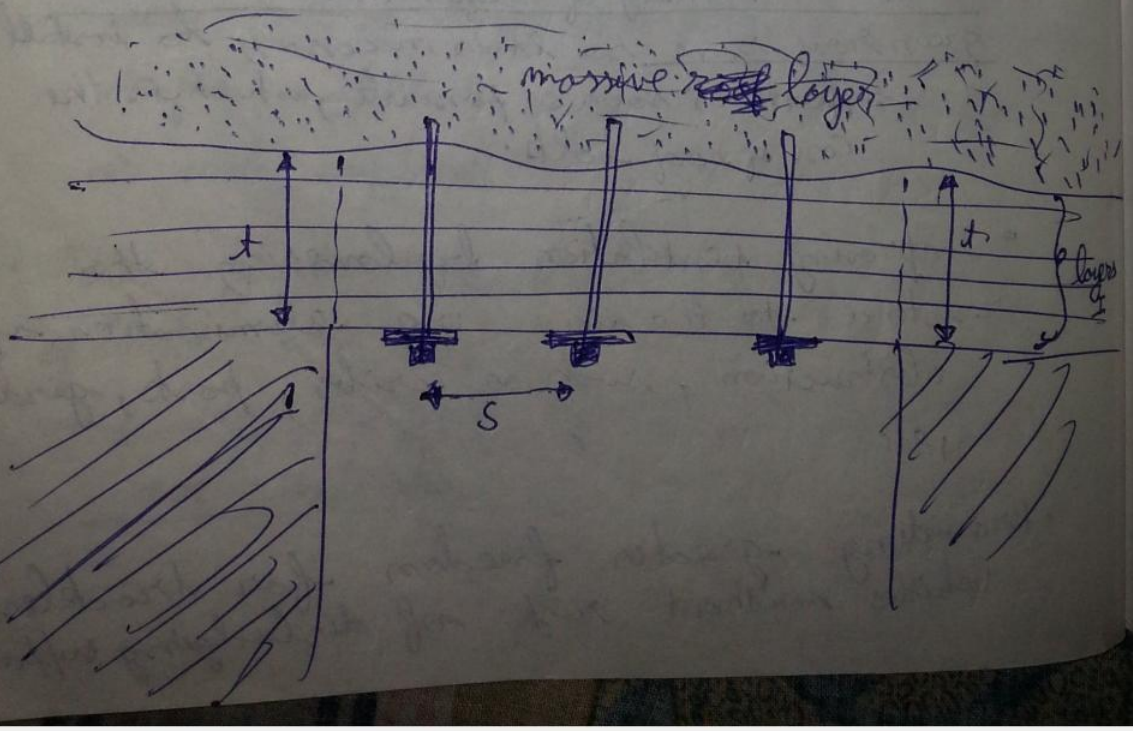
- Providing natural supports for hanging pipes, tubes, electrical cables, etc.

Theories of roof bolting:-

- Suspension theory
- Beam theory
- Keying theory

Suspension theory :-

In a given heading if a roof has a massive roof very close to the roof of the heading then it is possible to bound the layers ~~of~~ between the roof and the massive roof such that these intermediate layers will not fail as they are clamped to the layer which is ~~non fractable~~ massive and will not fail.



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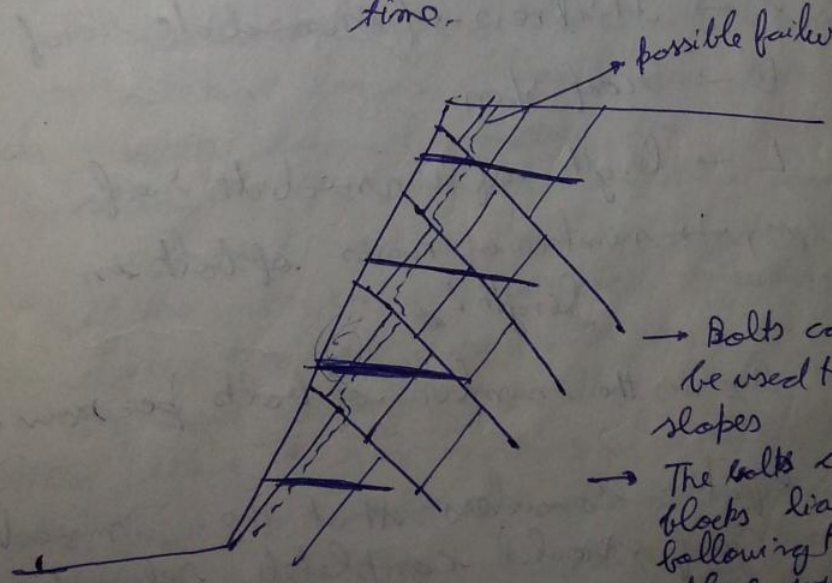
The layers I are liable to fail as their inherent strength is low due to the weakness planes in them. They will not remain intact for a long time after the drivage of the gallery. But their strength can be increased by clamping or suspending them with the massive roof which will not fail easily. The immediate roof will not fail till the massive roof fails.

— when an opening is made, the strata directly overhead tends to sag and if this strata is not supported adequately and properly in time, the laminated immediate roof could separate from main roof and fall out in due course of time

Suspension mechanism of roof bolting :-

- When an underground opening is made, the strata directly overhead tend to sag
- If the strata are not ~~suspended~~ supported adequately and properly in time, the laminated immediate roof could separate from the main roof and fall out in due course of time.



→ Bolts can also be used to stabilize slopes

→ The bolts clamp the blocks liable to fail following the failure plane with the (competent blocks) blocks of rock lying at an inner location so slope stability is improved.

→ The orientation of bolts w.r.t the slope face and the failure blocks is very important.

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- In situation 5, roof bolts anchor the immediate roof to the self supporting main roof by the tension applied to the bolts. In some instances, it appears that the immediate roof is suspended from the main roof by the bolts.

- The bolts have to carry dead weight of the strata between bolt head and the point of anchorage. The load to be carried by each bolt can be calculated as

$$P = \frac{WtBL}{(m_1+1)(m_2+1)}$$

where :-

W → unit weight of immediate roof
t → thickness of immediate roof
B → roof span
L → length of immediate roof
m_1 → number of rows of bolts in length L
m_2 is the number of bolts per row.

- This equation considers that the immediate roof would completely separate from the main roof such that it is suspended entirely by the bolts. A portion of the weight supported by abutment

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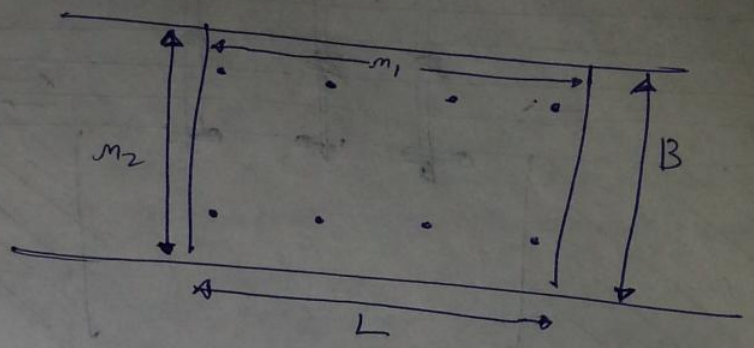
on both sides of the opening is also ignored. Hence this formula always gives an overestimated value of the load on the bolts.



(Plan View)

Beam Building Mechanism:-

If self supporting roof is too away from the roof of the gallery then it is not possible to make use of suspension theory. The roof bolts can be used to support the roof using the Beam theory.

The individual layers between the roof and massive layers ~~then~~ are thin beams which ~~with~~ do not ~~have~~ have the inherent strength to stabilize on their own. They are not self sustaining layers. So if the layers are binded together by bolts then we are able to increase the inherent strength of the beams by making them behave as an individual thick beam/layer. Hence we are able to change the bending behaviour of the immediate thin layers by binding them using roof bolts.

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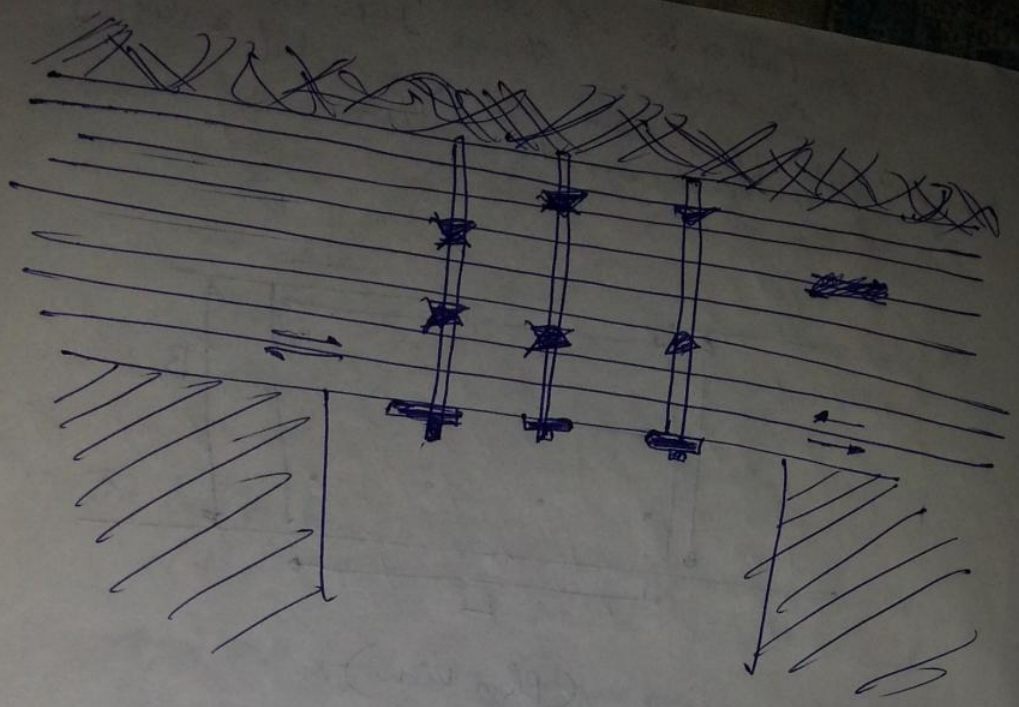

- In most cases the strong and self supporting main roof is beyond the reasonable distance that ordinary roof bolts can reach to anchor for suspension. However roof bolts can be applied in such situation with good success.

- In fact, sagging and separation of roof laminations cause ~~the~~ both vertical movement and horizontal movement along the bedding interfaces.

- Bolts installed through these layers can prevent or greatly reduce horizontal movements

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- Tension applied to the bolts manually on installation or induced by vertical movement of rock, clamps the layers ~~of different~~ together, making them to move with the same magnitude of vertical displacement.

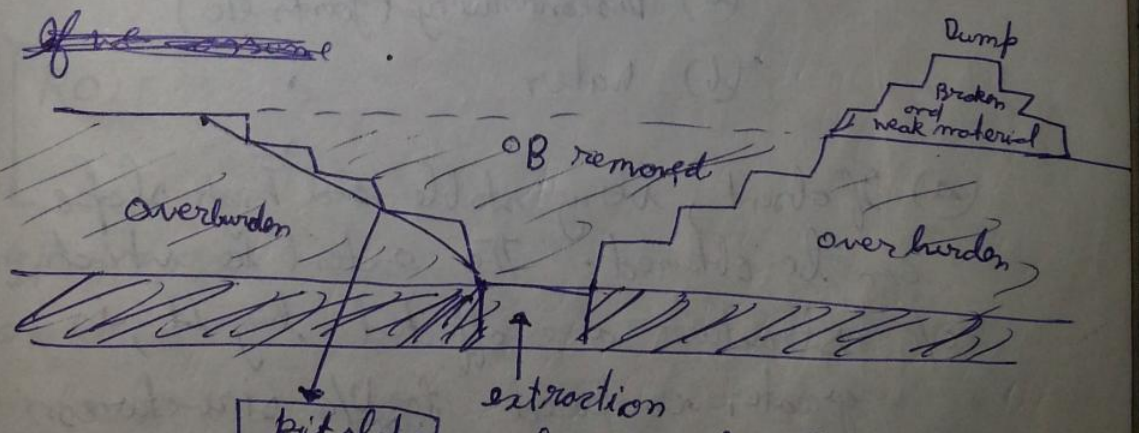
- On the other hand frictional forces (proportional to the bolt tension) are induced along the bedding interface, which also makes horizontal movement difficult.

23/09/2013.

BKS Sir

Slope Stability Analysis:-

It can ~~be~~ broadly be about two things:-
(i) rock mass
(ii) Dump material



pit slope → slope of a line joining the toes of each bench and extending from the surface upto the pit bottom

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* To extract coal, the upper material (overburden) has to be extracted.
Stability of the slope thus formed is very important. This is stability of rock mass

* The extracted material is kept somewhere outside / inside so a dump is created. It contains broken and weak material. The stability issues are called dump stability problems

* If we have a flatter slope the the stability is greater but we have to extract huge amts of OB

But

if we have a steeper slope then it is favourable from the economic point of view but large slope instability problems may arise.

Thus an optimum and stable pit slope is needed.

* The two factors which are most important while considering rock mass stability are
(a) Discontinuity (joints etc)
(b) water.

(a) If absent, very stable and high slope can be obtained. The extent to which O/C structures are affected by it, is greater compared to U/G structures.

(b) If present then worsens the situation further.

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(*) While considering a pit slope 2 aspects are to be looked into from the stability point of view.

stability → Bench stability
stability → overall pit stability

Individual benches contrave steeper slopes but pit slope stability is a concern and is affected by 2 factors:

(i) Height
(ii) slope angle

Height ↑ , slope angle ↓ for stability

Height ↓ , slope angle ↑ for stability.

ROLE OF DISCONTINUITY IN SLOPE FAILURE:-

It has been observed that while many slopes are stable at steep angles and at height of hundreds of meters, many flat slopes fail at heights which is considerably low.

This difference is due to the fact that stability of rock slope varies with inclination of discontinuity surfaces such as faults, joints and bedding planes within the rock mass. When

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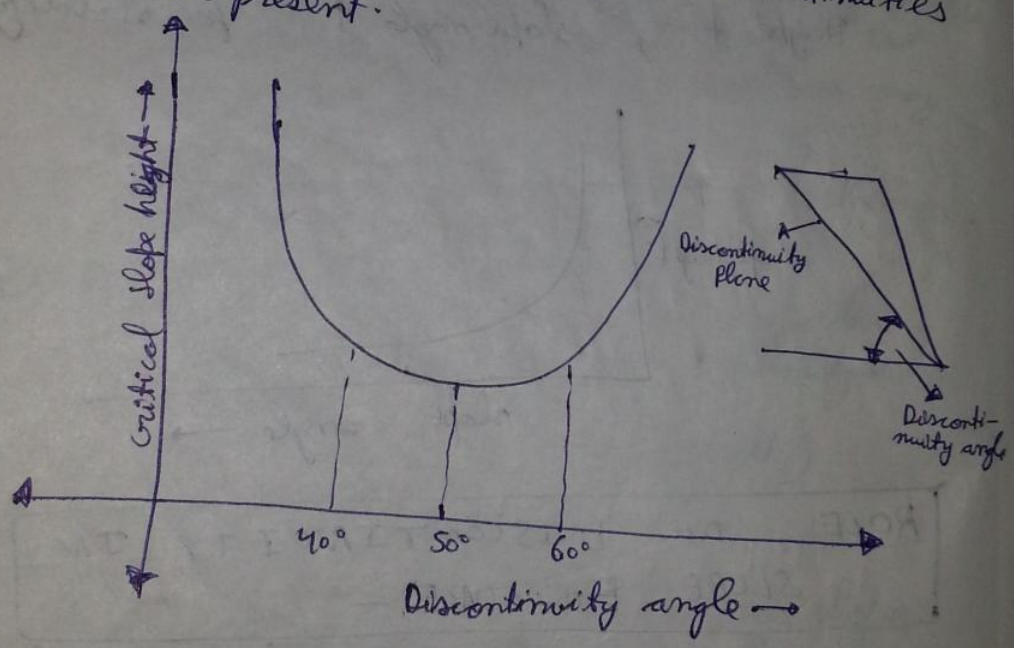
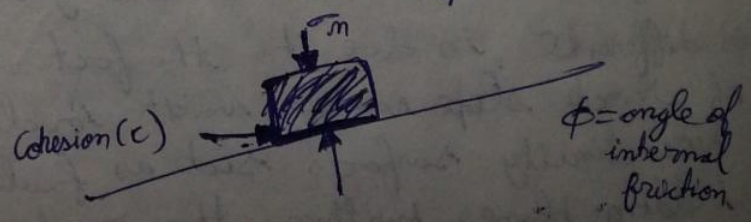
these discontinuity are vertical / horizontal simple sliding cant take place and the slope failure will involve fracture of intact blocks of rock as well as movement along some of the discontinuities.

On the other hand, when the rock mass contains discontinuity surfaces dipping towards the slope face at angles between 30° and 70° simple sliding can occur and the stability of these slopes is significantly lower than those in which only horizontal and vertical discontinuities are present.



Relation between shear and normal stress

$$\tau = c + \sigma_n \tan\phi$$



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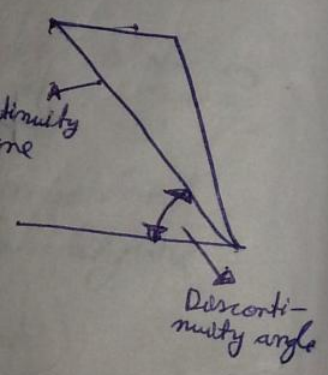


nal stress

= angle of internal friction

26/09/2013.

GSPS Sir

Beam theory (Continued)

- If we assume that all the thin layers are clamped to form a thick layer, the maximum bending strain at the clamped ends of the composite beam is given by:-

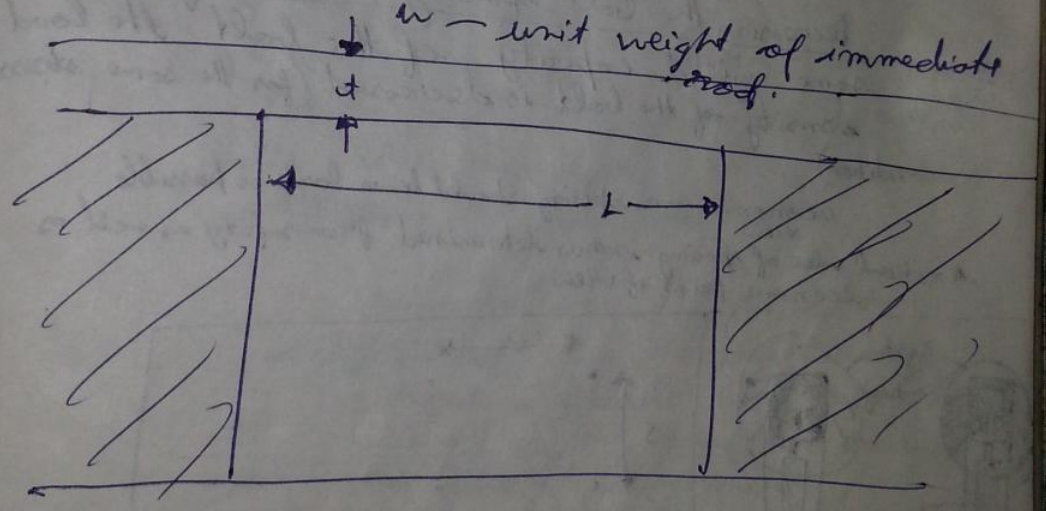
$$\varepsilon_{max} = \frac{WL^2}{2Et}$$ (there is no consideration of insitu stress)

E — Young's modulus

L — length of the immediate roof ~~in the~~ exposed in the gallery

t — thickness of composite beam

w — unit weight of immediate roof.



We are going to have the control on the bending strain by binding the roof layers together (t↑) and hence we are controlling the bending stress and hence we can control the stability of the roof.

Hence by roof bolting we are improving the immediate roof stability.

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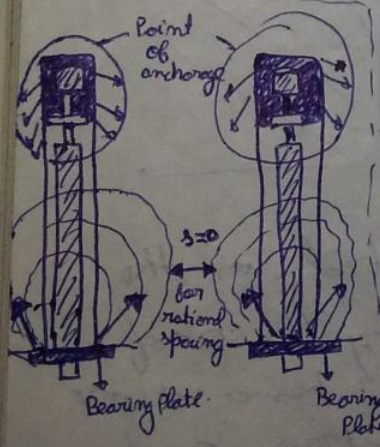
- The above expression indicates that the thicker the beam, the smaller the maximum strain induced at the clamped ends i.e. the clamping action produces a beam building effect.

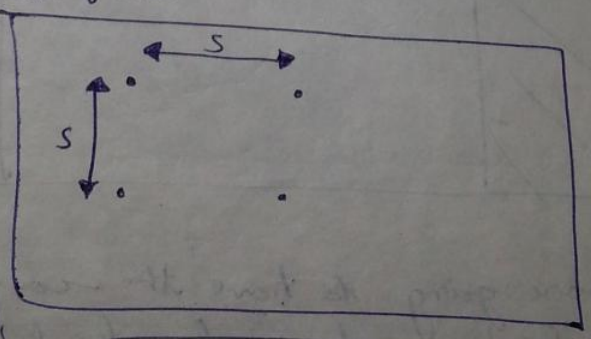
- Findings of some other research indicate that the beam building effect increases with
  - decreasing bolt spacing
  - increasing bolt tension
  - increasing no. of bolted laminal/layers
  - decreasing roof span

Decreasing the bolt spacing → so for the same yield capacity of the bolt the load density of the bolt is decreased (for the same stress condition)

economic view → spacing should be as large as possible

A rational value of spacing is thus determined from safety as well as economic point of view.





Loading of the bolt:-

1) from the dead weight of the immediate roof
2) from the main roof

→ The anchorage will neutralize the tension in the bolt by development of a compression zone at the zone of anchorage.

→ As the roof descends, the bearing plate try to restrict the downward movement of the bolt, hence a compressive zone of larger extent of influence than at the point of anchorage is established.

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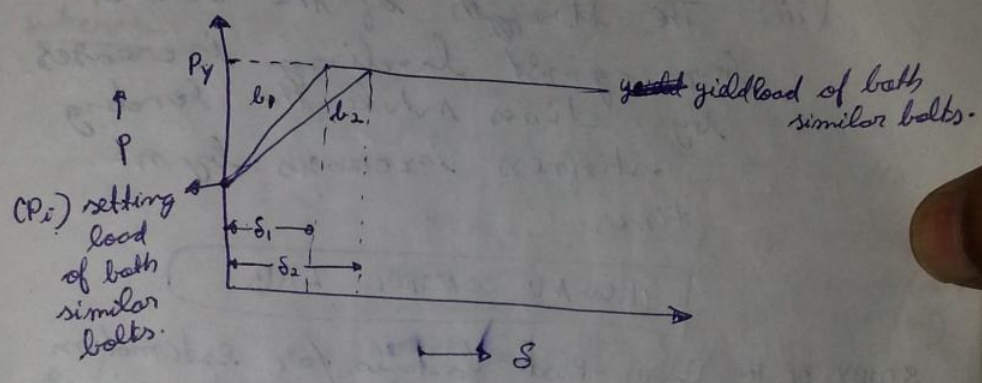
optimum spacing → we should decide for a spacing such that the outer ~~flux of~~ line of the stress flux of two adjacent bolts have a minimum gap between each other (s=0)

<u>increasing the laminae of bolt spacing →</u>

~~thicker~~ thicker layer → stiffer also

A structure which is stiffer in nature induces a larger load for the same amt. of deformation compared to a structure which is softer in nature.



For the same change in load of the bolts the deformation in the two roof ~~bolts is~~ layers is different.

Here bolt b_1 is installed in a stiffer roof. so the bolt does not get sufficient opportunity to act on the roof to reduce the rate of deformation.

Bolt b_2 is installed in a softer roof which allows a larger deformation for the bolt to yield. It gives enough opportunity to the bolt to act on the roof.

Here we should keep in mind that the length upto which the bolt is binding is not so large that the stiffness of the roof is increased to such a large value that the bolt is not getting any opportunity to act on the rate of deformation.

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- The beam building effect has a dual contribution towards the stability of the immediate roof.
  (i) It increases the bending strength of the composite beam and also
  (ii) increases the bending stiffness of the layer
  (iii) The strength of the bolted beam against bending increases by n times while the bending stiffness increases by n^2 times.

GROUND CONTROL LAB

STUDY of the Beam-Plate formula for estimation of roof failure span in longwall workings.

Q. A longwall face = 150 m length is extracted. The strata condition is as follows:-

Fine grained Sandstone = 3 m f = 5 MPa
density = 2500 kg/m3

Fine grained Sandstone = 12 m , f = 5 MPa
density = 2500 kg/m3

Shaly sandstone = 1 m f = 3 MPa
density = 2500 kg/m3

Coal seam = 2.5 m , density = 1400 kg/m3

Also estimate the effect of reduction of face length on the main fall span if a reduced face length of 80m is implemented.

Ans:- **THEORY:-**

The caving span of roof is estimated using the approach suggested by Obert and Duvall (1967) based on the theory of plate and beams originally developed by Timoshenko et al (1959) for tensile failure for a gravity loaded plate or beam clamped on all edges. The maximum tensile stress likely to develop in the plate under the effect of its bending due to gravity loading is given by the expression below:-

$$\sigma_{max} = \frac{\sigma \beta \gamma_e a^2}{t}$$

where:-

σ_{max} = max. tensile stress (MPa)
β = empirical constant based on the ratio b/a
γ_e = effective unit weight of rock (MPa/m)
b = longer lateral dimension of the plate (m)
a = smaller lateral dimension of the plate (m)
t = thickness of the plate (m)

The effective unit weight of the rock is calculated by:-

$$\gamma_e = \frac{E_1 t_1 (\gamma_1 t_1 + \gamma_2 t_2 + \cdots)}{E_1 t_1^3 + E_2 t_2^3 + E_3 t_3^3 + \cdots}$$

where:-

E_1, E_2 etc are the young's modulus of the rock layers and γ_1, γ_2 -- are the unit weights and t_1, t_2 -- are their thickness values.

The values of β for different b/a values as suggested by Timoshenko is as follows:-

b/a	1.00	1.25	1.50	1.75	2.00
β	0.0513	0.0665	0.0757	0.0806	0.0829

For estimating the failure span of a beam (b/a > 2), the failure span is given by:- (For estimating 1st local fall and main fall)

$$L_B = \sqrt{\frac{2ft}{\gamma_e}}$$

For estimating the subsequent local fall span and the periodic caving span cantilever formula is used:

$$L_c = \sqrt{\frac{ft}{3\gamma_e}}$$

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where:- $f \rightarrow$ flexural strength of the layer involved in specific caving event.

$\gamma_e \rightarrow$ effective weight density.

For estimating local fall span and subsequent local fall span, it is the immediate roof which is involved in the process of caving where as main roof is involved in main fall and periodic caving process.

CALCULATIONS:-

$\rightarrow$ Face length = 150 m

1st **First local fall span:-**

$t = 1m$

$f = 3 MPa$

$\gamma = 0.025 MPa/m$

(γ_e is not required because no additional loading from the main roof.)

$$L_B = \sqrt{\frac{2ft}{\gamma_e}} = \sqrt{\frac{2 \times 3 \times 1}{0.025}}$$

$$= 15.5 m$$

2nd **Subsequent local fall:-**

$t = 1m$

$f = 3MPa$

$\gamma = 0.025$

(γ_e is not required because no additional loading from main roof)

$$L_c = \sqrt{\frac{ft}{3\gamma_e}} = \sqrt{\frac{3 \times 1}{3 \times 0.025}} = 6.32 m$$

E_1 = modulus of main roof

E_2 = modulus of loading roof

Assuming $E_1 = E_2$ (Since main and loading roof are made of same material)

$$\gamma_e = \frac{(12)^2 [(2500)(12) + (2500)(3)]}{[(12)^3 + (3)^3]}$$

$$\therefore \gamma_e = 0.0308 \ MPa/m$$

4th Periodic

$t = 12m$

$f = 5MPa$

$\gamma_e = 0.0308$

3rd Main fall

$t = 12m$

$f = 5MP$

$\gamma_e =$

With the face length of instances so beam accuracy.

when face length = 80m

For local fa since they occur unchanged as th is used to calcula

But with rea For same main roof before

So for the same M

Now the value of will be 0.

$\therefore$ Since

roof so Main Main fall failu flexural stre value of a b/a where

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(2500)(3)]

)3]

/m

4th [Periodic caving span]

$t = 12m$

$f = 5MPa$

$\gamma_e = 0.0308\ MPa/m$

$L_c = \sqrt{\frac{ft}{3\gamma_e}} = 25.48m$

3rd [Main fall span:-]

$t = 12m$

$f = 5MPa$

$\gamma_e = 0.0308\ MPa/m$

$L_B = \sqrt{\frac{2ft}{\gamma_e}} = 62.42m$

With the face length of 150m $b/a > 2$ at all ~~instances~~ instances so beam formula can be used with reasonable accuracy.

when face length = 80m :-

For local fall and subsequent local fall $b/a > 2$ (a/a) since they occur quite early span so they remain unchanged as the same beam and cantilever formula is used to calculate their spans.

But with reduction in face length to 80m, For same main fall span $b/a < 2$ and so the main roof behaves as a plate ~~so it is~~ likely to change. (main fall span)

So for the same Main fall span $b/a = \frac{80}{62.4} \simeq 1.28 < 2$

Now the value of β corresponding to the $b/a = 1.28$ will be 0.0676 (by linear interpolation)

$$\therefore \sigma_{max} = \frac{6\beta\gamma_e a^2}{t} = \frac{6 \times 0.0676 \times 0.0308 \times (62.4)^2}{12}$$

$\therefore \sigma_{max} = 4.05\ MPa < 5\ MPa$ of Main roof so Main fall will not occur.

Main fall failure will occur only when σ_{max} = flexural strength of the Main roof for a certain value of a and corresponding value of β depending on b/a where a is the new smaller dimension.

The value of a estimates as:-

$$\sigma_{max} = 5 = \frac{6 \times \beta \times 0.0308\ a^2}{12}$$

$$\beta a^2 = 324.675$$

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By hit and trial :-

For $a = 80$ m $\quad b/a = 1 \quad \beta = 0.0513$

$$\sigma_{max} = \frac{6 \times 0.0513 \times 0.0308 \times (80)^2}{12}$$

$= 5.05$ MPa) 5 MPa ~~for~~ of the main roof

So the new main fall span = 80 m

when the face length is reduced to 80 m

27/09/2013

Beam building Mechanism (continued)

- For a beam consisting of n identical layers without bolt (all the layers are independent to have their own bending, there is no mutual interaction between the layers.

$$B = n \frac{bh^2}{6}$$

$$T = n \frac{Ebh^3}{12}$$

- For a composite beam consisting of n identical layers with bolts binding them together

$$B_1 = \frac{b(nh)^2}{6} = nB$$ (enhanced by a factor n after installation of bolt)

$$T_1 = \frac{Eb(nh)^3}{12} = n^2 T$$

The proportion by which the bending stiffness (T) is increasing is further n times more than the ~~increase~~ proportional increase in B.

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Where:-

E → Young's modus.

n → no of identical layers

h → thickness of each layer

b → length of the beam (span)

B_D, B → Bending stress of beam (Bending strength of the beam)

T, T → Bending stiffness of the beam.

- The improvement in bending strength is always good for roof stability. But increase in bending stiffness in some cases extra load from the overlying strata acting on the beam.
- Under such ~~a~~ case, the beam may undergo shear failure (in place of commonly expected tensile failure) at the 2 ends once the accumulated shear stresses exceed the shear strength. This type of failure is called Cutter roof failure.
- This kind of failure has the following feature:-
  - The bolted composite beam falls out
  - Failure planes at two ends of the beam are nearly vertical
- The upper failure plane is exactly at the bolted horizon where pretension of the bolts creates a tensile stress area around the anchor of each bolt.
- Sometimes, using longer bolts just creates increased height of roof fall.

Keying mechanism of bolts:-

If we have to support above which have intersecting joint planes. On creation of galleries there are some blocks which are liable to undergo failure. These are critical rock blocks. The exposed rock ~~bolts~~ blocks have a higher potential to undergo failure. To support these

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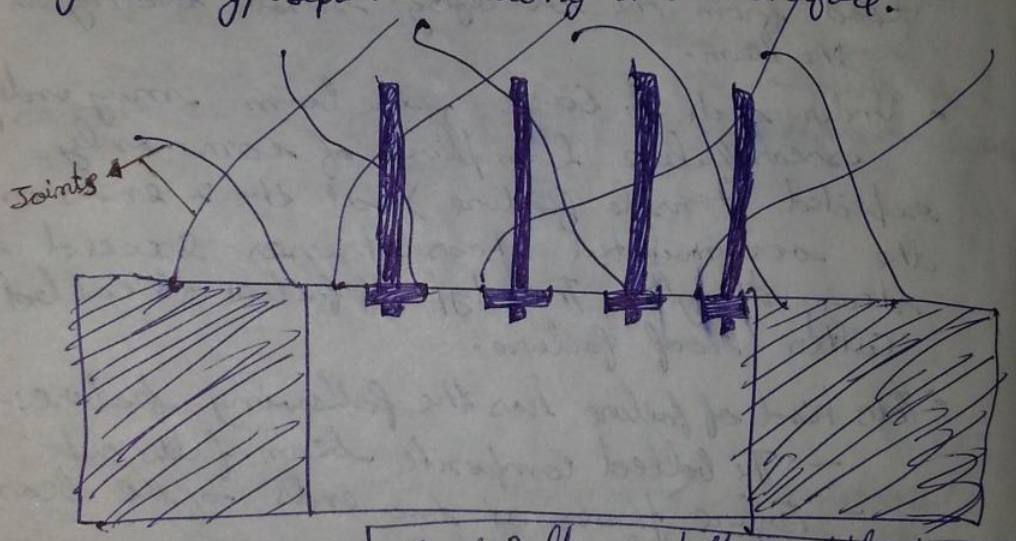
~~bolts~~ - critically unstable blocks we are trying to bind these blocks with other blocks which are stable thus increasing the overall stability of the roof. Otherwise there would be chances of shear failure of these unstable blocks along the joint planes thus resulting in fall of such blocks of rock.

- When the roof strata, are highly fractured and blocky, or the immediate roof contains one or several sets of joints with differed orientation to the roof line,
- roof bolting provides significant frictional forces along fractures, cracks and weak planes, thus preventing sliding/separation along the interface.



~~Types of roof~~ Roof Bolts and their applications:-

Type of bolt	Type of anchor	Suitable strata type	Comments.
Point anchored tensioned bolt	~~slot~~ slot and wedge	Hard rock	used in early stages
	Expansion shell	Medium strength	
	Resin grout	All strata especially weak rock	increasing use recently
Full length grouted (untensioned)	Cement	Most strata	Good anchorage with "no mix" resin
	Perfo		
	Injection		
	Cartridge (quick setting cement in slurry)	(just after 30 min of setting the bolt gives 30-40% of yield load capacity)	

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Type of bolt	Type of anchor	Suitable strata type	Comments
Roof truss	Expansion shell	Adverse roof	To be used at inter-sections and heavy pressure areas

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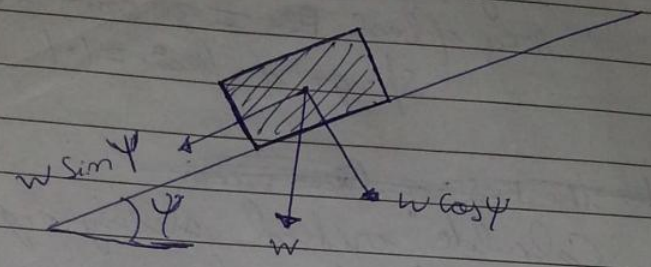
30/09/2013.

GROUND CONTROL

(BKS)

Sliding due to gravitational loading:-



$$\sigma = \frac{W\cos\psi}{A}$$

$$\tau = c + \frac{W\cos\psi}{A}\tan\phi$$

Resisting force $= CA + W\cos\psi \cdot \tan\phi$

Sliding force $= W\sin\psi$

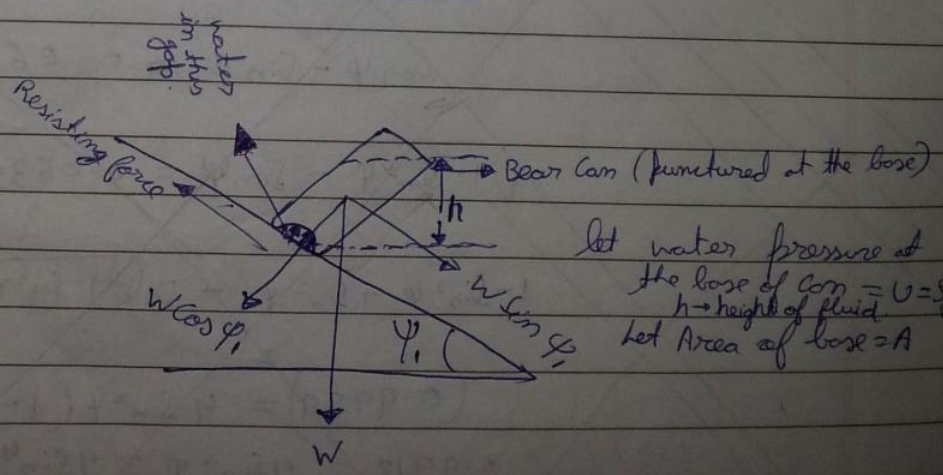
$c \rightarrow$ cohesive forces

$\phi \rightarrow$ angle of internal friction

$\therefore$ FOS $= \dfrac{\text{Resisting force}}{\text{sliding force}}$

FOS = 1 — limiting condition

Influence of water pressure:-



let water pressure at the base of can $= U = \rho g h$

$h \rightarrow$ height of fluid

Let Area of base $= A$

Resisting force $= CA + (W\cos\psi_1 - UA)\tan\phi$

sliding force $= W\sin\psi_1$

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Q1.

weight of Can = 500 gm. (W)

Coefficent of friction between surface and can = 30° (ϕ)

height of bear = 15 cm. = h

Area of base = 30 cm² = A

Sp. gr. of bear = 1.1

~~Calculate the FOS of bear can~~

Calculate critical sliding angle

(i) when bear can is punctured

(ii) when bear can is not punctured.

→ $\boxed{\text{Ignore cohesion}}$:-

Ans:

(i) $450 \cos\psi_1 - \left(1.1 \times 10 \times 1000 \times \frac{15}{100}\right) \times 30 \times \tan 30°$

$= 450 \sin\psi_1$

$\Rightarrow 450 \cos\psi - 3.3 \times 15 \times \frac{1}{\sqrt{3}} = 450 \sin\psi_1$

$\Rightarrow 450 (\cos\psi - \sin\psi) = 28.58$

$\cos\psi - \sin\psi = 0.0635$

$\sqrt{1-\sin^2\psi} - \sin\psi = 0.0635$

$1 - \sin^2\psi + \sin^2\psi - 2\left(\sqrt{1-\sin^2\psi}\right)\sin\psi = 4.032 \times 10^{-3}$

$(0.9959)^2 = 4\sin^2\psi(1-\sin^2\psi)$

$0.9918 = 4\sin^2\psi - 4\sin^4\psi$

$\sin^2\psi = \frac{-4 \pm \sqrt{16 - 4 \times (-4)(-0.9918)}}{2(-4)}$

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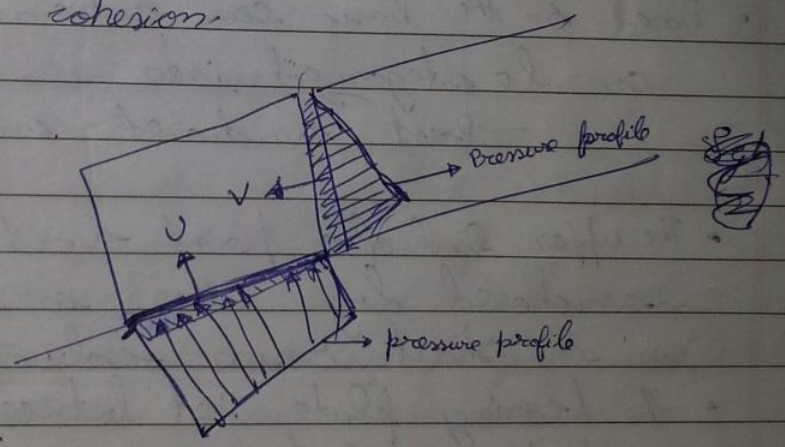
Q.2.) A Block of 30m X 30m X 30m got detached from its parent rock and is lying on the rockmass as shown in the figure. Considering uplift force and horizontal forces. Calculate the angle of the slope at which it is just going to slide.

Sp. gr. of rockmass = 2.7

coefficient of friction between rock and rockmass = tan 30°

Ignore cohesion.



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03/10/2013

GROUND CONTROL (GSPS)

Types of ~~the~~ roof bolts and variations:-

- Bolts reinforce the rock by binding the stratified ~~or~~ broken roof layers (or blocks) together. The binding effect is achieved through friction forces created by the physical interlocking along the anchor and rock interface.

Friction is ~~Bolts and the coverage or anchorage~~ between: (a) Bolt and the grouting material.
(b) grouting material and the rockmass.

- Based on the basic ~~ear~~ ~~actor~~ anchor types, bolts can be categorized into
  - Point anchored and full length anchored.

- The upper end of a point anchored bolt is anchored by either a mechanical device or a short resin column.
- The bearing plate set between the bolt head and the roof line serves as the other anchor.

Use of Bearing plate:-

When lower end of the roof bolt comes under tension due to downward deformation of the roof as the bolt is trying to resist that. The tensile stress acting at the mouth of the bolt is required to be redistributed (significantly throughout the length of the bolt) or else there will be failure at the mouth of the bolt and the utility of the bolt is lost. This is the purpose of the bearing plate.

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- Usually, a certain amount of tension is applied to the bolt at the time of installation. So bolt acts as an active support. The bolts tends to close the existing weakness planes and the inherent strength of the rockmass increases and the time interval for undergoing a certain amount of deformation by the roof strata ~~that~~ increases. This is the benefit of having pre-tension in the bolt.

- A full length grouted bolt is grouted to the borehole by fast setting high strength resin or inorganic cement throughout the full column of the hole. No pretension is applied to the bolt.

Roof Truss

- Introduced as an alternative method of supporting unstable roof
  - when conventional roof bolts alone are not effective, such as when fall out is frequent between roof bolts or where cutter roof failure is encountered.

- Consists of 2 inclined point anchored bolts, a connection bar (provides rigid connection between the bolts), a turn buckle (a spring whose tension can be controlled) to give proper tension to the bar and an adjusting wedge box.

  Supports the roof when there is a tendency of the downward movement of the roof.

- The connection elements utilize and the (incomplete) compressive stresses developed at the point of anchorage to support the ~~downward movement~~ roof and prevent its downward movement by creating compression in the roof.

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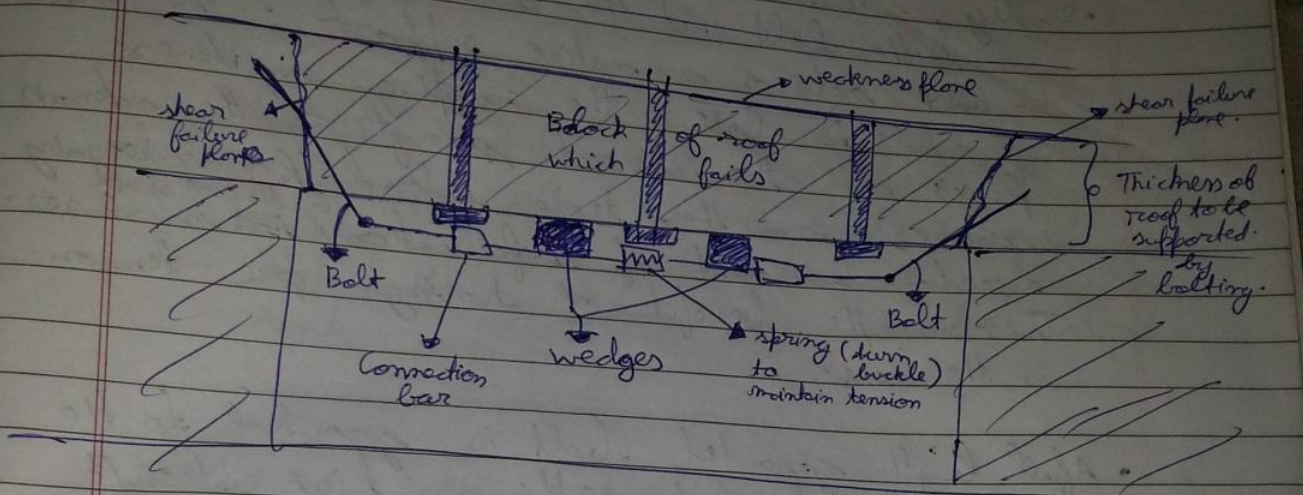
Cutter failure:-



When the roof is so stiff that the roof bolt do not have much opportunity to support the roof (the roof undergoes failure along shear planes (almost vertical) at the corners of the opening) ~~then~~ cracks are developed along the corners of the gallery and these cracks act as shear planes (along which shear failure takes place) along which the entire block of ~~rock~~ roof rock ~~is~~ exposed to the gallery which ~~as~~ is intended to be supported comes down (falls) in one section. The block of rock separated by a weakness plane ~~above~~ from above ~~and~~ ~~undergoes shear failure~~ ~~or and by shear~~ and by shear planes ~~by~~ ~~the competent roof~~ on the corners from the competent roof ~~comes~~ under goes shear failure along these shear planes and falls as an intact block of rock. This type of failure of the roof is called cutter failure. This is not the failure of an individual bolt but the failure of the entire roof of the gallery along shear planes. By putting a vertical bolt or inclined bolt in the block mass ~~undergo~~ liable to undergo cutter failure, we will not be able to reduce the chances of roof fall.

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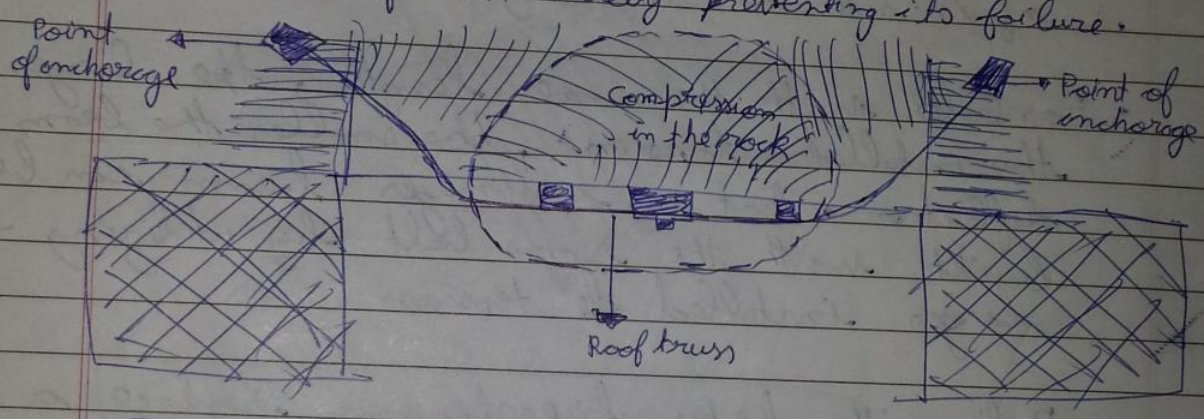
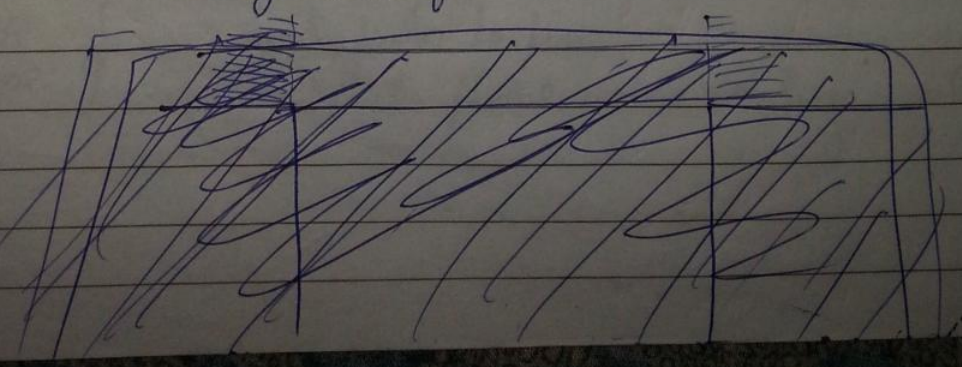
So we install two inclined bolts in such an orientation that the point of anchorage of the bolts still lies in the roof layer but on a ~~block~~ section of roof rock which is much more competent and is not liable to undergo such failure. This competent block lies in the part of the roof strata not exposed by mining. The two inclined bolts are connected by a series of elements which maintain proper tension the bolts. ~~To~~ By the use of connection elements we are able to translate/transfer the compressive ~~f~~ stresses developed at the point of anchorage in the competent roof section to support the block of roof ~~liable to undergo~~ exposed by mining which is liable to undergo utter failure. This entire arrangement is a roof truss. The failure mechanism of the roof of the gallery initially is due to tension but by installation of roof truss we are able to create compression in the roof rock thereby preventing its failure.



- The tensioned connection bar keeps the roof in compression and can also yield downward as the roof displaces.

- In 3 or 4 way intersections where tensile stresses are very high, trusses can produce compressive forces to offset high tensile stresses at mid intersection roof.

- Trusses are also very useful for supporting newly exposed roof.



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Some facts about roof bolts:-

- The longer the bolt, the larger the beam deflection. Alternatively, the shorter the bolt, the smaller the deflection and the stronger the beam.
- The longer the bolt, the larger the load that will be induced in the bolt as the result of in-site or mining related stresses.
- For bolts with the same length, the larger the installed tension, the smaller the beam deflection. That is a stronger beam can be built with the same bolt by utilizing a larger installed ~~to~~ tension.
- Bolts with higher pre-tension induce a smaller deflection within the bolting range, which in turn, results in a smaller deflection higher up in the roof above the bolting range.

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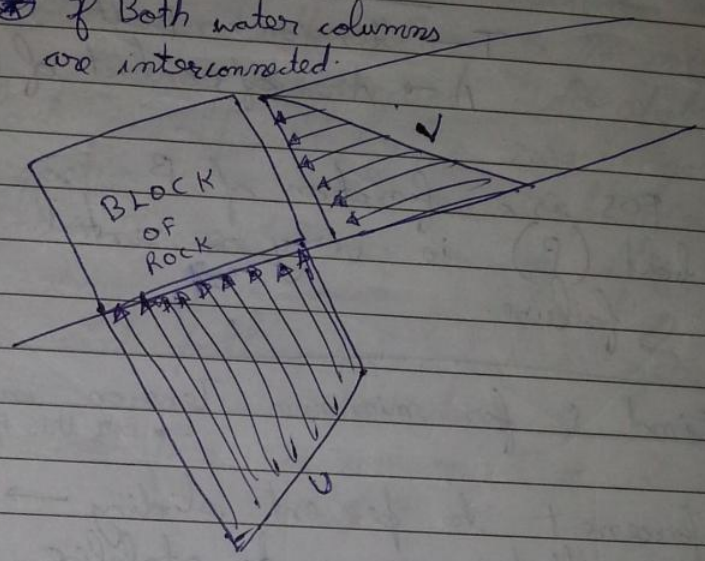
~~solution of questions of BKS Sir given on 20/09/2013~~

21/10/2013.

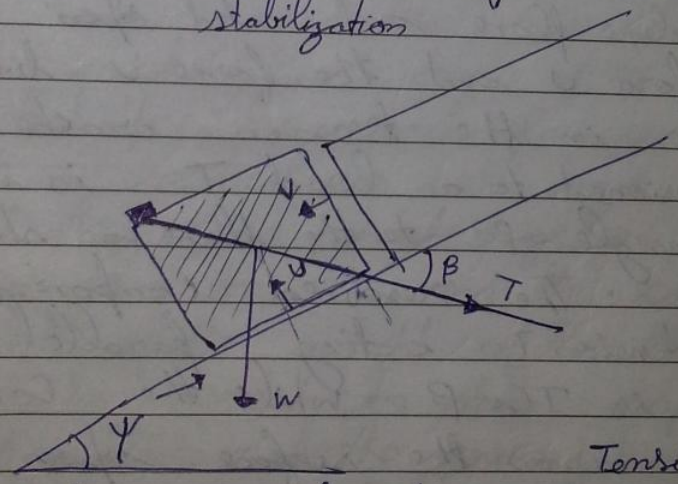
GROUND CONTROL (BKS Sir)

* If Both water columns are interconnected.



Case of rock reinforcement:-

→ Bolt is used for rock slope stabilization



Tension of bolt has 2 Components:-

Both increase FOS.
1) perpendicular to plane → increases normal stress
2) parallel to plane → reduces sliding force

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At equilibrium:-

$\boxed{FOS = 1} \Rightarrow W\sin\psi + V - T\cos\beta = cA + (W\cos\psi - U + T\sin\beta)\tan\phi$

where:-

T → tension in Bolt

A → Area of contact of rock block

FOS is a function of β so if orientation of bolt (β) is very essential to control failure.

Find β for minimum Tension in the bolt:-

→ For this FOS = 1 (Similarly β for minimum T can be found for any value of FOS)

Reinforcement to prevent sliding → One of the most effective means to stabilize slabs of rocks which are likely to slide down inclined discontinuity surfaces is to install tensioned rock bolts. Consider the block resting on the incline plane and acted upon by the uplift force U and the force V due to water pressure in the tension crack. The rock bolt tensioned to a load T is installed at an angle β to the plane as shown in the figure. The resolved component of the bolt tension T acting parallel to the plane is $T\cos\beta$ while the component acting across the surface upon which the block rests is $T\sin\beta$. The condition of limiting equilibrium for this case is defined by:-

$$W\sin\psi + V - T\cos\beta = cA + (W\cos\psi - U + T\sin\beta)\tan\phi$$

It is obvious that the Bolt tension reduces

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the distributed forces acting down the plane and increases the normal force and hence the frictional resistance between the base of the block and the plane. The minimum bolt tension required to stabilize the block is obtained by rearranging the given equation to give an expression for bolt tension T and minimizing this expression w.r.t angle β and differentiating w.r.t β partially and putting it equal to zero

$\boxed{\text{On Calculating:-}}$ ~~So~~ for minimum T:-

$$\beta = \psi$$

Q.) ~~A block of rock~~ A block of rock of size $(30 \times 30 \times 30)\,m^3$ was detached from the parent rock and likely to slide on an inclined surface making an angle of 45°. The water has filled up in the crack between the parent rock both in vertical as well as inclined plane and they are joined together. The coefficient of friction is tan 30° and the cohesive forces are 1/5th of the weight of the block. A bolt has to be inserted to ~~stab~~ improve the FOS of this block and the tension in the bolt is equal to 1/10th of the weight of the block.

Calculate the FOS of the block (Sp. gravity of rock = 2.7)

(a) in dry condition

(b) in wet condition (both joints interconnected)

(c) in wet condition (joints not interconnected)

Calculate (d) minimum angle at which the bolt should be inserted to improve the FOS. (i.e. for FOS to be maximum)

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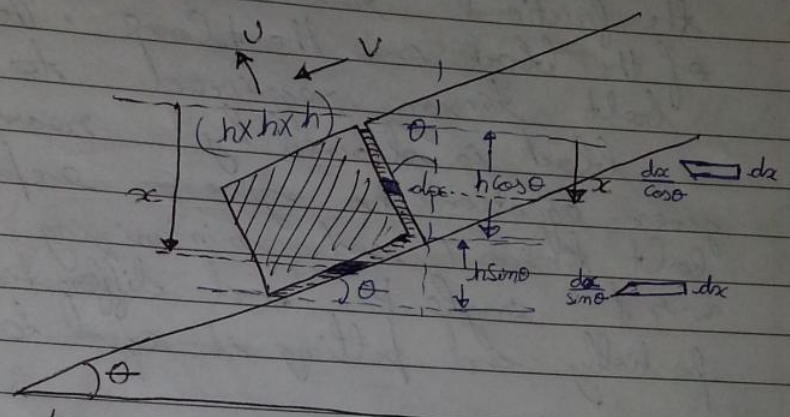
(e) the FOS of the block when the bolt is inserted.

Solution:-



when water columns are interconnected:-

$$dV = \rho g x \frac{dx\,h}{\cos\theta} \Rightarrow V = \int_0^{h\cos\theta} dV$$

$$\therefore V = \frac{\rho g h^3 \cos\theta}{2}$$

$$dU = \rho g x \frac{dx\,h}{\sin\theta} \Rightarrow U = \int_{h\cos\theta}^{h\cos\theta + h\sin\theta} dU$$

$$\therefore U = \frac{\rho g h^3}{2}\left[\sin\theta + 2\cos\theta\right]$$

when water columns are not interconnected:-

$$dV = \rho g x \frac{dx\,h}{\cos\theta} \Rightarrow V = \int_0^{h\cos\theta} dV$$

$$\therefore V = \frac{\rho g h^3 \cos\theta}{2}$$

$$dU = \rho g x \frac{dx\,h}{\sin\theta} \Rightarrow U = \int_0^{h\sin\theta} dU$$

$$\therefore U = \frac{\rho g h^3 \sin\theta}{2}$$

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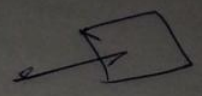
$h = 30\,m$ $\quad \theta = 45^\circ$ $\quad$ sp. gr $= 2.7$ $\quad \rho_w = 1000\ kg/m^3$ $\quad g = 10\,m/s^2$ $\quad \phi = 30^\circ$



(a) $$FOS = \frac{CA + W\cos\theta \tan\phi}{W\sin\theta} = \frac{\frac{W}{5} + W\cos\theta \tan\phi}{W\sin\theta}$$

$$= \frac{(1/5) + \frac{\tan 30^\circ}{\sqrt{2}}}{1/\sqrt{2}} = 0.86$$

(b) $$V = \frac{\rho g h^3 \cos\theta}{2} = \frac{1000 \times 10 \times (30)^3 \times \cos 45}{2}$$

$$\therefore V = 95459415.46\,N$$

$$U = \frac{\rho g h^3}{2}\left[\sin\theta + 2\cos\theta\right]$$

$$\therefore U = \frac{1000 \times 10 \times (30)^3}{2}\left[\sin 45 + 2\cos 45\right]$$

$$= 286378246.4\,N$$

$$\therefore FOS = \frac{\frac{W}{5} + (W\cos\theta - U)\tan\phi}{(W\sin\theta + V)}$$

$$W = 2700 \times (30)^3 \times 10 = 7.29 \times 10^8\,N$$

$$\therefore FOS = \frac{\frac{7.29 \times 10^8}{5} + \left[7.29 \times 10^8 \times \cos 45^\circ - U\right]\tan 30^\circ}{(7.29 \times 10^8 \sin 45^\circ + V)}$$

$$\therefore FOS = 0.455$$

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(c) $V = \frac{\gamma_w h^3}{2} \cos\theta = 95459415.46\,N$

$U = \frac{\gamma_w h^3}{2} \sin\theta = 95459415.46\,N$

$$\therefore FOS = \frac{\left(\frac{7.29\times10^8}{5}\right) + \left[\left(7.29\times10^8 \times \cos45^\circ\right) - U\right]\tan30^\circ}{\left(7.29\times10^8 \times \sin45^\circ + V\right)}$$

$\therefore FOS = 0.636$

(d) Assuming water columns interconnected and wet condition.

$$FOS = \frac{CA + \left(W\cos\theta - U + T\sin\beta\right)\tan\phi}{W\sin\theta + V - T\cos\beta}$$

$$FOS = \frac{\frac{W}{5} + \left(W\cos\theta - U + \frac{W}{10}\sin\beta\right)\tan\phi}{W\sin\theta + V - \frac{W}{10}\cos\beta}$$

$\frac{d(FOS)}{d\beta} = 0$ → for max. FOS

$$\therefore \left(W\sin\theta + V - T\cos\beta\right) T\cos\beta \cdot \tan\phi = \left[\frac{W}{5} + \left(W\cos\theta - U + T\sin\beta\right)\tan\phi\right] T\sin\beta$$

$W = 7.29\times10^8\,N$ $\qquad T = W/10 = 7.29\times10^7\,N$

$U = 286378246.4\,N$

$V = 95459415.46\,N$

$\theta = 45^\circ$

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Substituting above values we get the equation:-

$$2.79\sin\beta - 3.53\cos\beta + 0.421 = 0$$

$$\Rightarrow \beta = 46.37^\circ$$ for max. FOS.

(e) The maximum FOS is:-

$$FOS = \frac{\frac{W}{5} + \left(W\cos\theta - U + \frac{W}{10}\sin\beta\right)\tan\phi}{W\sin\theta + V - \frac{W}{10}\cos\beta}$$

$$= \frac{\left(\frac{7.29\times10^8}{5}\right) + \left[\left(7.29\times10^8\cos45\right) - U + \left(7.29\times10^7\sin46.37\right)\right]\tan30^\circ}{\left(7.29\times10^8\times\sin45^\circ\right) + V - \left(7.29\times10^7\cos46.37^\circ\right)}$$

$U = 286378246.4\,N$

$V = 95459415.46\,N$

$\therefore FOS = 0.55$

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Solution of problems of BKS Sir given on 30/9/2013

A-1) $W\sin\theta = (W\cos\theta - \rho g H A)\tan\phi$

$M = 0.45\,Kg$

$g = 10\,m/s^2$

$A = 30\,cm^2$

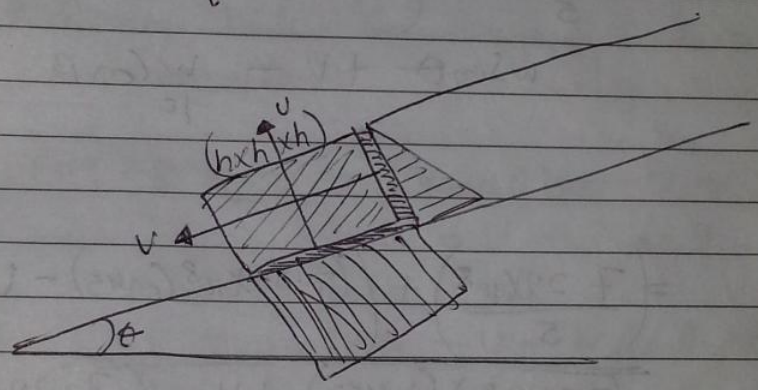
$\rho = 1100\,Kg/m^2$

$H = 0.15\,m$

$\tan\phi = 0.577$

Solve for $\theta = ??$

A-2)



Assuming water columns interconnected:-

$$U = \frac{\rho g h^3}{2}\left[\sin\theta + 2\cos\theta\right]$$

$$V = \frac{\rho g h^3 \cos\theta}{2}$$

$h = 30\,m$ $\quad \rho = 1000\,Kg/m^3$ $\quad g = 10\,m/s^2$

Sp. gravity $= 2.7$ $\quad \phi = 30°$

$W = 7.29 \times 10^8\,N$

$$U = \frac{1000 \times 10 \times (30)^3}{2}\left[\sin\theta + 2\cos\theta\right] = (1.35 \times 10^8)(\sin\theta + 2\cos\theta)$$

$$\therefore V = (1.35 \times 10^8)\cos\theta$$

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$\therefore (W\cos\theta - U)\tan\phi = W\sin\theta + V$

$\therefore 4.21\cos\theta = 0.78(\sin\theta + 2\cos\theta) = 7.29\sin\theta + 1.35\cos\theta$

$\therefore 1.3\cos\theta = 8.07\sin\theta$

$\therefore \tan\theta = \frac{1.3}{8.07}$

$\boxed{\therefore \theta = 9.15^\circ}$

17/10/2013

GSPS Sir

Design of Roof bolting System:-

For complete and appropriate rock bolting system design, the following parameters must be determined properly:-

1.) bolt type 2.) Bolt length 3.) pattern and spacing of bolts

4.) Bolt diameter and anchor capacity

5.) whether pretension should be applied or not. If yes, what is the appropriate magnitude?

1.) Selection of bolt type:-

→ The selection of an appropriate bolt type to meet specific requirements at the lowest cost and max. effectiveness is a confusing task.

→ Selection of type of bolt also depends on strata condition

→ Careful examination of geological condition and through understanding of performance of different types of bolts under different settings are essential for a successful bolt selection type.

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Tensioned bolt → Point anchored Bolt
→ mechanical bolt
untensioned Bolt → Full length grouted

Guidelines for selecting bolt type:-

- Mechanical bolt:-

1. harder rock where the rock properties will not adversely affect the gripping force of the anchor
2. Temporary reinforcement systems
3. Where bolt tension can be checked regularly
4. Rock that will not experience high shear forces
5. Rock that is not highly fractured
6. Areas away from blast sites where bolt tension may be lost

If the rock is hard, the harder rock has high value of frictional angle. The higher the frictional angle higher the frictional force / gripping force

- Grouted Bolt:-

1. conditions where mechanical bolts are not permitted:-
2. permanent reinforcement systems
3. Boreholes without continuous water run off problems
4. Rock without wide fractures and voids in which significant amount of grout could be lost
5. Untensioned bolts are recommented in rock that is highly fractured and deformable as long as adequate bolt installation is feasible. Generally, bolts in more competent strata often requires shorter grout column than that in less Competent strata.
6. Tensioned grouted bolts (point anchored bolt) are recommended for use where additional frictional forces, in combination with a grouted column, may enhance roof stability

→ We cannot ~~put~~ pretension in full length grouted bolt that's why we use point anchored bolt.

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Advantages of fully grouted resin bolts compared to mechanical bolts:-

1.) Anchorage is virtually guaranteed and independent of strata type
2.) Bolt is permanently effective throughout its full bonding length
3.) The fully grouted bolt prevents both vertical and horizontal strata movement
4.) The hole length is not critical since the bonding can be adjusted according to the type of strata
5.) Fully grouted bolts seal wet holes and exclude air, thus reducing corrosion of the bolt assembly and weathering of rock.
6.) Damage to the bolt head, bearing plate or rock at the collar of the hole does not cause the resin bolts to become ineffective
7.) The resin grouted bolt can absorb blast vibrations without ~~bl~~ bleed off of the bolt load.
8.) The fully grouted resin bolt shows excellent performance with regard to anchorage creep. When the bolt is subjected to creep loading then the grout bolt or bolt which we have put in the rock should be able to bear that load.
9.) Time and labour for tensioning the bolts are not needed.

A General thumb rule for mechanical bolts:-

- They cannot be reliably installed in critical areas as permanent support, and in places where significant roof deformation is likely to occur. Such bolts may be considered as temporary reinforcement.

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(II) Selection of bolt length:-

- Minimum length of bolt should be greatest than the following three:
  - Twice the bolt spacing
  - Three times the width of critical and potentially unstable rock blocks defined by the average discontinuity spacing in the rockmass
  - For span less than 6 m, bolt length of one half the span, for span between 6-18 m, linearly interpolate between 3-5 m respectively. For excavation higher than 18 m, sidewall bolts are 1/5 of the wall height.

(used mainly for tunnels)

(III) Bolt Spacing:-

(1st)

$$b = \frac{2l}{3} \quad \text{or} \quad b = \frac{2}{9}L$$

} consider minimum of both

where $b \rightarrow$ bolt spacing

$l \rightarrow$ bolt length (~~maximum~~)

$L \rightarrow$ roof span

- Thumb rule maximum bolt length

(2nd)

- least of:-
  - One half the bolt length zone and one half the width of critical and potentially unstable rock ~~bol~~ blocks (1.8 m generally)

(V) Pretention:-

(1) Although application of pretension is very important for mechanically bolts and resin anchored bolts to create frictional forces between layer ~~of~~ and to reduce interlaminal separation, it is not always true that higher pretension will ensure better stability

(2) Maximum pretension should not exceed 60% of the yield strength or the anchorage capacity of the bolt.

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18/10/2013.

Shotcrete Support

(*) Basically it means surface support

Introduction:-

- Shotcrete support has been in use with considerable success in tunnelling projects. In recent years, it has been applied with success for stabilization of vital mining structures life shaft pillars, permanent galleries, and longwall gate roads in coal mines.

Gunite material is used to support

- Basically, shotcrete is a mixture consisting of sand, cement, concrete along with some other additives such as steel fibres and silica fumes.
- The dry mixture is carried through a ~~horse cap~~ hose pipe by means of compressed air and shot over the rock surface through a nozzle.

Shotcrete technology:-

- shotcreting is the generic name given for cement, sand and fine aggregates which are applied pneumatically and compacted dynamically under high velocity.
- Depending on mixing mechanism, shotcrete can be of 2 types: - wet mix
  - Dry mix

Dry mix shotcrete

- Dry mix shotcrete components, which may be slightly pre damped to reduce dust are fed into a hopper with continuous agitation
- Compressed air is introduced through a rotating barrel or feed bowl to convey the materials ~~to~~ in a continuous stream through the delivery hole
- water is added to mix at the nozzle
- Dry mix shotcrete is also known as gunite

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Wet mix shotcrete:-

In this case the shotcrete components and the water are mixed before delivery into a positive displacement pumping unit, which then delivers the mix hydraulically to the nozzle when air is added to project the material onto the rock surface

Difference between Dry and wet mix

In case of dry at the point of preparation of mixture water is not required

- The final product of either the dry or wet shotcrete process is very similar
- The dry mix system tends to be more widely used in mining, because of inaccessibility for large transit mix trucks and because it generally uses smaller and more compact equipment
- This can be moved round relatively easily in an U/G mine environment
- Wet mix system is ideal for high production applications in mining or civil engg. where a deep shaft or long tunnel is being driven and access allows the application equipment and delivery trucks to operate on a more or less continuous basis
- Decisions to use the dry or wet mix shotcrete process are usually made on a site-by site basis.

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Steel fiber, reinforced microsilica shotcrete

- Of the many development in shotcrete technology in recent years, two of the most significant are the introduction of silica fume, used as a cementitious admixture and steel or polypropylene fibre reinforcement.

- Silica fume or microsilica is a by product of ferro silicon metal industry and is an extremely fine pozzolan.

- ~~Pozzolans~~ Pozzolans are cementitious materials which react with the calcium hydroxide produced during cement hydration

- Silica fume, added in quantities of 8 – 13% by weight of cement, can allow shotcrete to achieve compressive strength which is double or triple the value of plane shotcrete mix.

- The result is an extremely strong, impermeable and durable shotcrete. Other benefits include reduced rebound, improved flexural strength, improved bonding with the rockmass and the ability to place layers of upto 200 mm thick in a single pass because of the shotcrete "stickiness".

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→ However when using wet mix shotcretes this "stickiness" decreases the workability of the material and superplasticizers are required to restore the workability.

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24/10/2013.

(Shotcrete Support continued) (GSPS Sir)

- Super plasticizers also known as high range water reducers are chemical admixtures used where well-dispersed particle suspension is required. These polymers are used as dispersants to avoid particle aggregation and to improve the flow characteristics of suspensions such as in concrete applications.

- Steel fibre reinforced shotcrete has gained wide acceptance as a replacement for traditional wire mesh reinforced plain shotcrete.

- As we know rock support is only called upon to carry significant loads once the rock surrounding an underground excavation deforms. This means that unevenly distributed non-elastic deformations of significant magnitude may overload and lead to failure of the

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support system unless that system has sufficient ductility to accomodate these deformations.

- The main role that reinforcement plays in shotcrete is to impart ductility to an otherwise brittle material.

Mechanism Of Shotcrete Working A Support:-

- When the adequately mixed shotcrete is sprayed at the wall to be supported at a high velocity @ 20 - 100 m/s, a layer of cement and sand particles penetrates into pores, cracks, joints in the rock mass and forms a basic foundation for the upper layers to be sprayed.

- As the shotcreting continues, the fines are squeezed into pores, cracks and joints in the rockmass. The shotcrete starts sticking on the initial layer of fines and subsequently, a layer is deposited.

- The shotcrete support stabilises the rockmass by:
  (a) maintaining the existing stability
  (b) acts as a media transferring load from otherwise failure prone weak zone to comparatively stable zone,
  (c) provides additional confinement to the wall against shear / tensile failure.

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(d) a fiber reinforcement provides additional bridging of failure zones apart from incorporating desired ductility to the skin support to cop up the large post-elastic deformation being transferred by the rock surface.

Mesh Reinforced Shotcrete

- While steel fibre reinforced shotcrete has been widely accepted in both civil and mining engineering, mesh reinforced shotcrete is still widely used and is preferred in some applications.

- In very poor quality, loose rock masses where adhesion of the shotcrete to the rock surface is poor, the mesh provides a significant amount of reinforcement, even without shotcrete.

- Therefore when stabilising slopes in very poor quality rock masses or when building bulkheads for underground fill, weldmesh is frequently used to stabilise the surface or to provide reinforcement. In such cases, plain shotcrete is applied later to provide additional support and to protect the mesh against corrosion.

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Recommended Shotcrete applications in U/G mining for different rockmass condition:-

Rock mass description	Rock mass behaviour	Support requirement	Shotcrete application
Massive sedimentary rock, low stress condition	Surfaces of some shales, siltstones or slaystones may slake as a result of moisture content change	Sealing surface to prevent slaking	Apply 25 mm thickness of plain shotcrete to permanent surface as soon as possible after excavation, repair shotcrete damage due to blasting
Massive sedimentary rock, high stress condition	Surface slabbing, spalling and possible squeezing in shales and soft rocks	Retention of broken rock and control of squeezing	Apply 75 mm layer of fibre reinforced shotcrete directly on clean rock, rock bolt are also needed for additional support.
Sedimentary rock with a few widely spaced bedding planes and joints, low stress conditions	Potential for wedges or blocks to fall or slide due to gravity loading, bedding plane exposures may deteriorate in time	Provision of support in addition to that available from rock bolts or cables, seling of weak bedding plane exposures	Apply 50 mm of steel fibre reinforced shotcrete on rock surface on which discontinuity traces are exposed with particular attention to bedding plane traces.

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Bedded and jointed weak sedimentary rock, high stress condition	Slabbing, spalling and possibly squeezing	Control of rockmass failure and squeezing	Apply 75mm of steel fibre reinforced shotcrete to clean rock surfaces as soon as possible, install rock bolts, with face plates through shotcrete, apply second 75mm shotcrete layer
Highly jointed and bedded sedimentary rock, low stress condition	Bed separation in wide span excavations and ravelling of bedding traces in inclined faces	Control of bed separation and ravelling	Rockbolts to control bed separation, apply 75mm of fibre reinforced shotcrete to bedding plane traces before bolting.
Heavily jointed sedimentary rock with clay coated surfaces, high stress condition	Squeezing and plastic flow of rockmass around opening, clay rich rocks may swell	Control of rockmass failure and dilation	Apply 50mm of steel fibre reinforced shotcrete as soon as possible, install girders with invert struts where required and then more fibre reinforced shotcrete to cover girders. Fore poling or splicing may be required ahead of excavation. Gaps may be left in final shotcrete to allow for movement resulting from squeezing or swelling. Gap should be closed once opening is stable.

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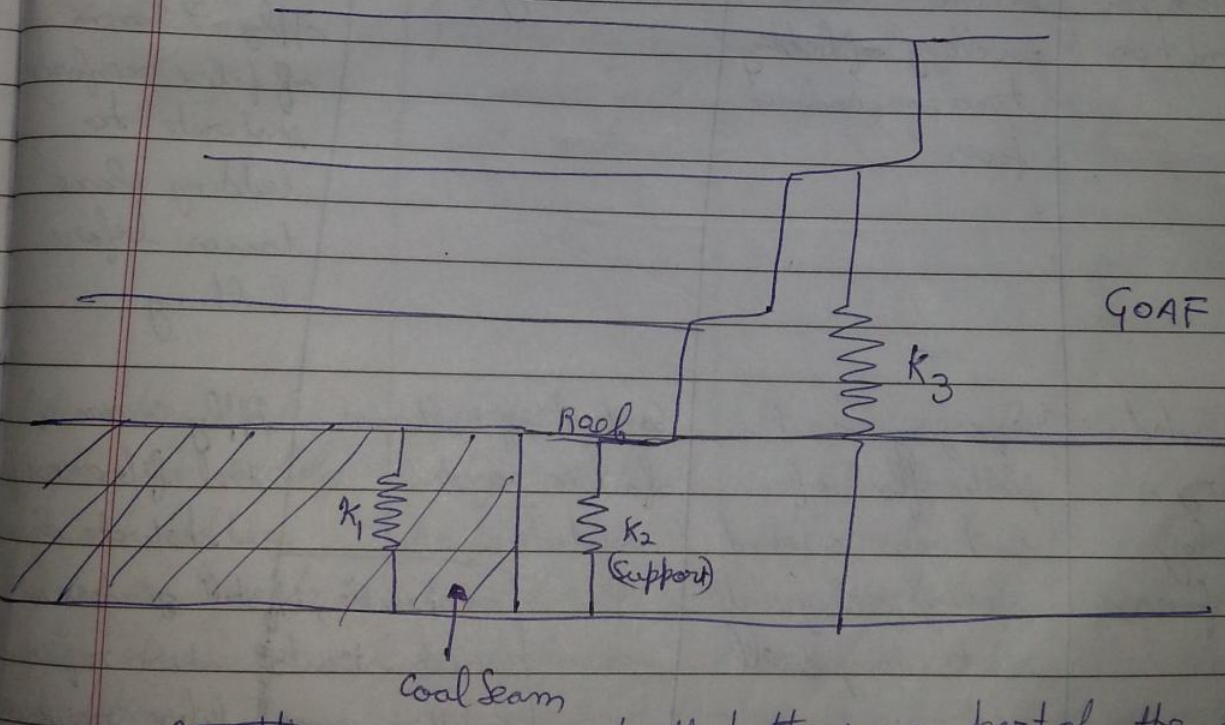
Mild rockburst conditions in massive rock subjected to high stress condition	Spalling, slabbing and mild rock bursts	Retention of broken rock and control of failure propagation	Apply 50 to 100 mm of shotcrete over mesh or cable lacing which is firmly attached to the rock surface by means of yielding rock bolts or cable bolts.

25/10/2013.

[GSPS Sir]

Mechanics of Strata Control by Stowing:-



~~Assumption~~ → we expect that ~~the~~ some part of the load is supported by the remaining ore

⇒ The face supports ~~supp~~ bear the load of the roof at the working space to provide safety in operation and for unhindered production

⇒ We choose a stowing material that can fulfill our requirement to support the goaf

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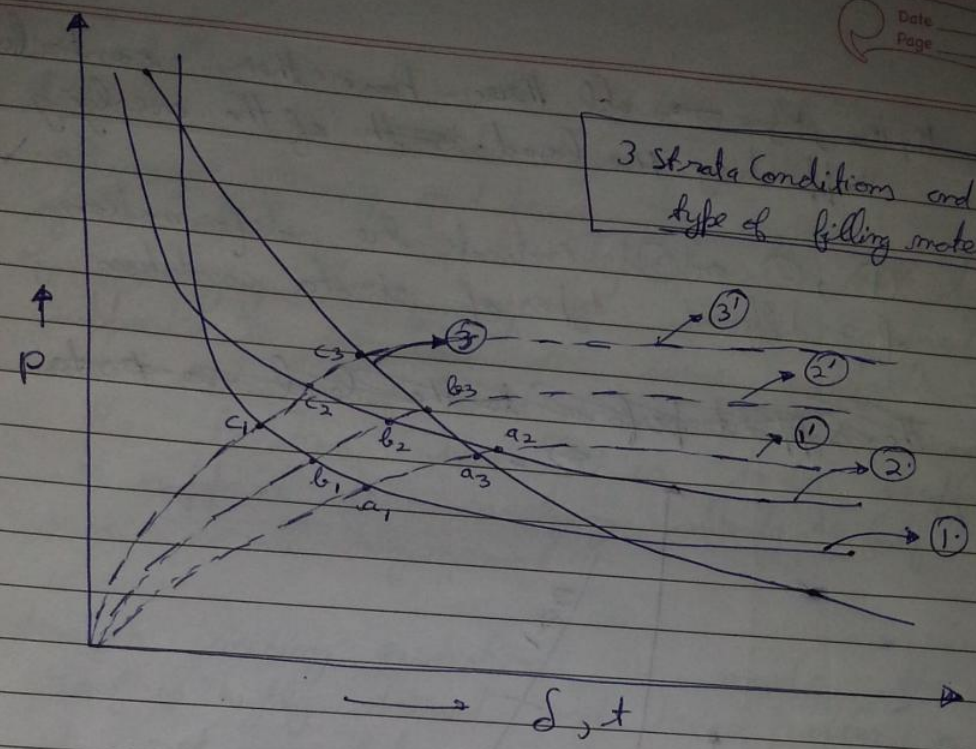

1, 2, 3 → Strata deformation curve
1', 2', 3' → Deformation curve of fill materials

1 – 1' ⇒ a_1
1 – 2' ⇒ b_1
1 – 3' ⇒ c_1

2 – 1' ⇒ a_2
2 – 2' ⇒ b_2
2 – 3' ⇒ c_2

3 – 1' ⇒ a_3
3 – 2' ⇒ b_3
3 – 3' ⇒ c_3

Purpose of filling → to avoid deteriation of the roof so that the ultimate damage is not seen on the surface or on any other mineral deposit lying above so that it can be worked out in future and so that the working area is ~~kept~~ rendered safe.

K_1 → represents the stiffness of the material that we are working

K_2 → stiffness of the material that is representative of the support at the working face

K_3 → stiffness of the material that we intend to fill in the goaf to support the roof.

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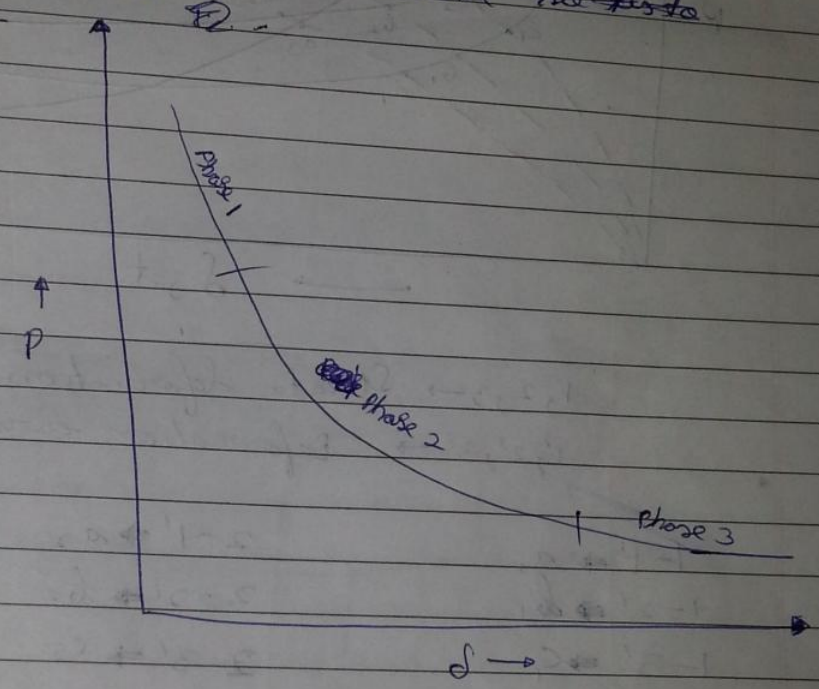
K_1, K_2, K_3 → all three have their contribution in supporting the load. ~~in the~~ of the overlying strata

① , ② and ③ indicate the deformation curves of 3 different types of strata condition

~~The Support perform to the level no ... to~~



Suppose we analyse the deformation curve of strata1 then we note:

In the initial phase (phase 1) → the stress relaxation is significant but the deformation is not as significant.

In the phase 2 → the deformation is significant for a certain amount of stress relaxation

In the phase 3 → the strata continues to deform but there is no significant stress relaxation.

These three phases are different for different strata types and so ~~the~~ hence we can conclude that the

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same filling material will not be able to support different strata ~~conditio~~ types with the same ~~efficacy~~ efficacy and hence the engineering properties of the stowing material depend on the type of overlying strata ~~is~~ ~~as~~ ~~it~~ ~~is~~ required to be supported to ~~produce~~ accomplish the same overall objective. Hence selection of stowing material depends on the location where it is to be employed to support the overlying strata to fulfill the same overall requirement.

Whenever mining excavations are made the roof is highly stressed and hence after a limit it cannot accumulate more stress and hence it redlases its stress (stress relaxation) but we want a gradual transfer of the energy due to stress relaxation of the ~~roof~~ overlying strata. Hence by filling the goaf with a suitable material, the stowing material supports the overlying strata and recieves energy from the strata in a gradual and systematic way by allowing its own deformation. Hence there is no adverse effect ~~on the roof~~ of ~~stress~~ energy ~~release~~ release on the working area and our overall requirement is ~~all~~ also addressed

Our expectation from the filling material is:-

1.) safe operation in the working area

2.) ~~unhindered product~~ preventing damage of overlying roof and hence ultimate damage at the surface

3.) prevent damage of ~~any~~ to any overlying mineral deposit to be extracted in future.

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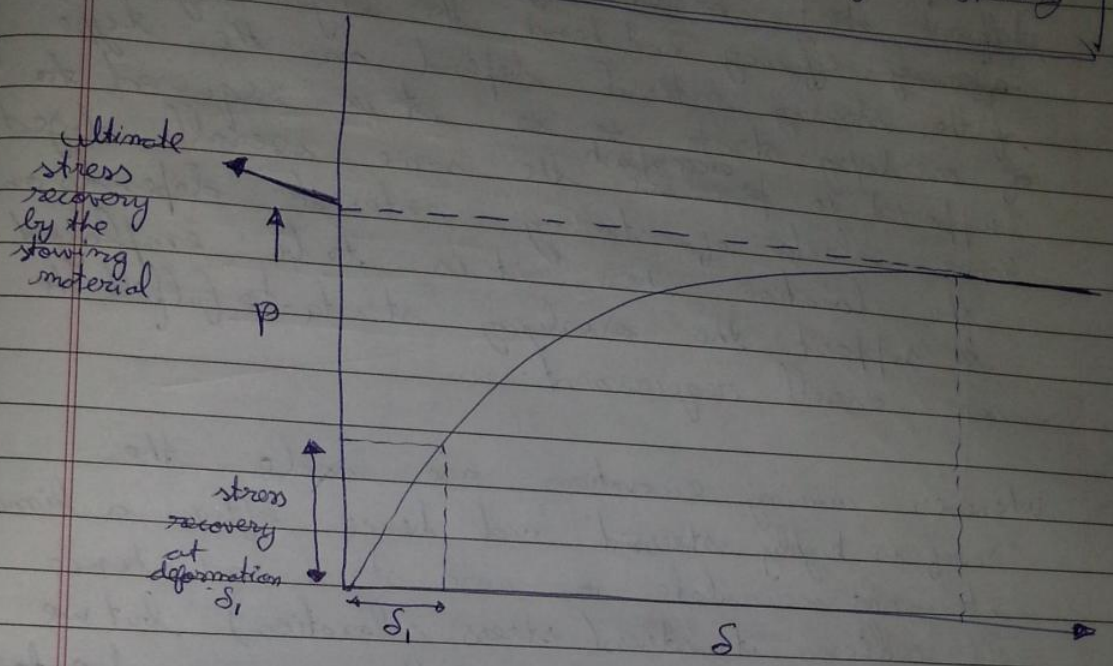
~~Aiot~~ [Nature of deformation curve of the stowing material:-]



Initially when the deformation is 0 then no stress is recovered but as deformation of the material is increases the amount of stress recovered is increased but only upto a limit after with the deformation increases continuously but there is no significant increase in the amt. of stress recovery.

This is the ultimate stress recovered by the use of the filling material.

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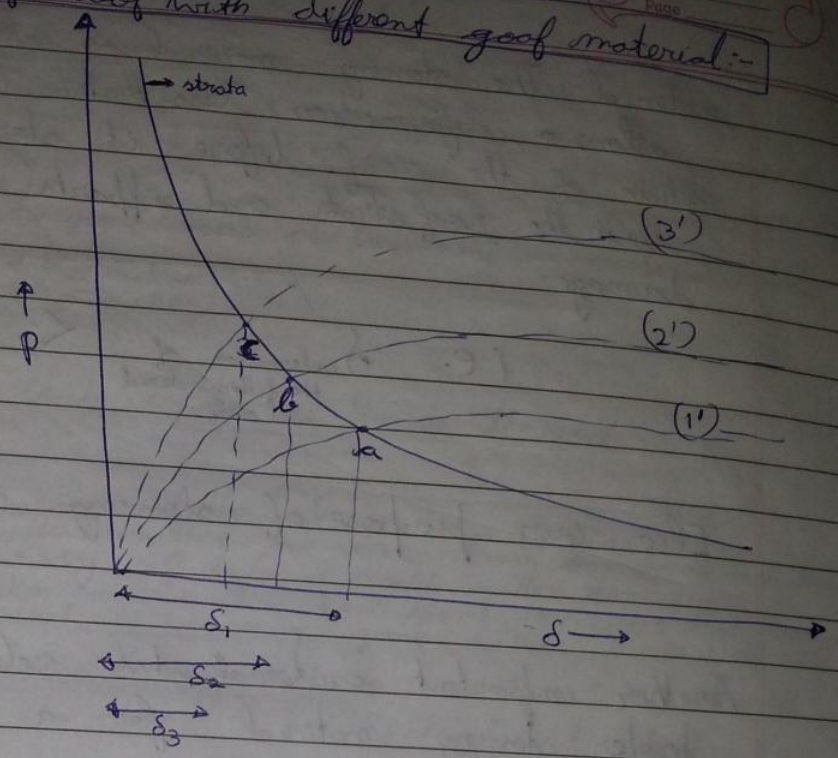
Deformation of roof with different goaf material:-



a, b, c are the points of ~~interaction~~ intersection of the deformation curves of the stowing material and the strata under with they are to be employed. These are the point of interaction between the roof and stowing material

$\delta_1, \delta_2, \delta_3 \rightarrow$ the deformation of the ~~roof~~ filling material at which ~~it~~ the filling material starts interacting with the roof.

We should first quantify the max. allowable deformation of the strata to be supported by the stowing material such that the objective of stowing is not lost. This is the critical deformation of the roof under consideration.

~~Then~~

The selection of the proper stowing material depends on the interaction point of the

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material. The stowing material is selected which allows a deformation less than the critical deformation of the roof before it starts interacting with the roof strata and supports it preventing its further damage.

i.e. $\delta_{\text{interaction of the filling material and roof}} < \delta_{\text{critical of the roof}}$

Else our purpose of stowing is not accomplised.

Another important criterial while selecting the proper stowing material for a given ~~roo~~ strata type:

- It is the ultimate or maximum stress recovered by the use of ~~the~~ a particular stowing material. It should correspond to the max vertical insitu stress level

Hence we can draw a conclusion that selection of stowing material depends on 3 criteria:-

1.) stiffness of ~~of~~ the overlying strata
2.) operation or interation point
3.) ultimate stress recovery.

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GSPS Sir

31/10/2013

Methods for Induction of Caving:-

⇒ If the overlying strata is not caving in the desired manner to have safe and sustainable working.

Introduction:-

- Hard and massive roof rocks are frequently encountered within (indicated by σc) (point function of thickness of layer and RQD)

larger comp. strength (σc) → Harder the rock.

active caving zone of the coal seams in India.

- The sandstone content in this zone varies from one locale to another. A random survey of types of rocks within ten times the height of extraction above the coal seams indicates that average sandstone content in caving zone varies from 11 to 100%.
  - active caving zone → varies from 5 to 15 times extraction ht. depending on characteristic of roof (bulking factor)
- If sandstone concentration in active caving zone is more than 30%. then we experience the problem of caving.
- The overlying hard roof is found to be difficult in caving after the coal is extracted and causes higher lengths of overhang.
- The delayed roof caving leads to higher abutment stress in the face area causing excessive load on supports, side/face spalling; large area roof falls and symptoms of air blast.
- The increased load on supports and cavity formation at the face endangers the safe working environment for men and machinery.
- We have witnessed significant difficulty in smooth maintainence of strata control in a number of B and P and longwall workings arising due to hard and massive roof conditions apart from collapsed and foreclosure of some of the high investment projects.
- In whatever manner does the hard roof manifest itself, it is naturally the most undesirable aspect of mining affecting the safety, productivity and economy in underground coal mines.

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Defination Of Hard Roof:-

- A roof is considered to be hard if it does not cave in the goaf after advancing the supports and stands intact for considerable time with large overhangs and reflects excessive load on the supports causing partial or complete damage of the supports during weightings
- Sometimes, it also leads to occurrence of air blast, pre-mature collapse of remnant pillars and overriding of pillars as observed in bord and pillar workings.
- The uncontrolled caving of strata also gives rise to some other problems like excessive stress condition, spalling of pillars and high convergence due to excessively high abutment loading in the working area.

Methods of Induced Caving:-

It is the ~~[illegible]~~ method to modify the natural caving process such that overlying strata is forced to undergo a systematic and easier caving rather than unsystematic and delayed caving.

- Increasing the width of extraction
- ~~undercutting~~
- Induced caving by blasting (active method)
  - Blasting in advance from underground
  - Blasting in goaf from underground
  - Blasting in goaf from surface
- water infusion technique (Passive method)
- Hydro - fracturing technique (active method)

Increasing the width of extraction:-

- The fundamental knowledge suggests:-

  for extraction of a gallery of width w, a roof of particular thickness t develops bending stress $\propto$ (width of gallery)^2, ~~[illegible]~~ ~~[illegible]~~

⇒ Bending ~~strength~~ stress ↑ with increase in width of gallery irrespective of thickness of strata. If thickness is more stress is ~~less~~ more, if thickness is less stress developed is ~~less~~ ~~[illegible]~~ less

⇒ But strength of roof is restricted by σ_t, σ_c, joint parameter, insitu stress domain, ~~cohesion and angle of friction~~.

⇒ For a given strength of roof we can plan for a width of extraction which can ~~[illegible]~~ allow for a suitable goaf geometry that results into systematic and regular failure of the roof strata.

⇒ But this caving by increasing the width of ~~the gallery~~ extraction is at an expense of:

1.) increased load on the supports at the face
2.) increased abutment load in the working area

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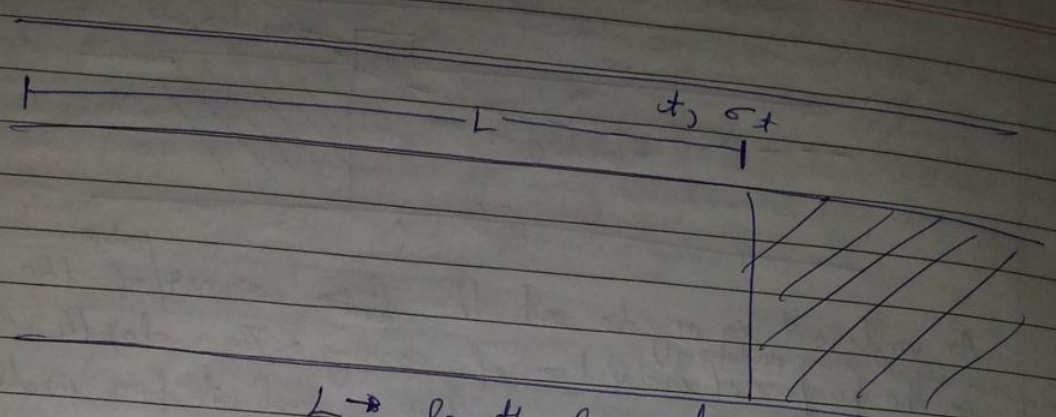
Undercutting:-



L → length of overhang of the overlying strata that has developed in the course of extraction of mineral body.

t → thickness of layer

σt → parameter which indicates the overall strength of overlying rock layer, (not necessarily tensile strength)

⇒ If width of extraction is W, even after such width of extraction roof of a length L is still standing in the goaf so we want to somehow induce caving in it.

⇒ For failure of the layer → Bending stress of the layer > tensile strength of the layer.

⇒ By removing the fraction of the layer by mineral extraction we are allowing the layer to bend but the amount of bending developed in the layer is insufficient to develop requisite amount of Bending stress such that Bending stress > Bending strength, causing failure of roof.

⇒ We have to focus on the parameter which affect the bending stress → width of extraction, thickness of layer but these parameters are fixed

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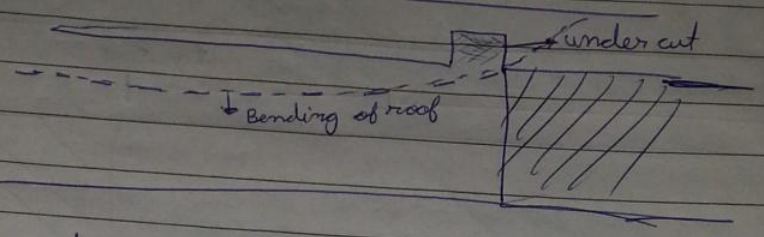
Hence the technique of undercutting is used :-



An undercut is made at the ~~[illegible]~~ corner/edge of the roof (which is the point of max tensile stress development) for the purpose of induction of caving. The depth of the cut is less than 1/2 a metre and it can induce caving in a roof thickness of 8 to 10 m. By introduction of the undercuts we are modifying the geometry of the roof such that there is development of tensile or shear stress and it will go to such a level that it satisfies the failure criteria and hence the ~~[illegible]~~ overlying strata will undergo caving. To trigger the process of caving undercutting is of a very high ~~to~~ potential and its used with great success.

It is effectively utilised in metalliferrous mines stopes having blasting gallery workings where under cuts (called potato cuts) are made at the junctions, along the periphery of the panel ~~where [illegible]~~ by small scale blasting in the roof instead of mechanical cutting.

1/11/2013- $\boxed{\text{PSPS Sir}}$ (continued)

Undercutting → is an established method for inducing caving of host ~~to~~ rock in u/G mining stopes. Recently it has also been practiced to initiate the caving process in blasting gallery workings while mining of thick coal seams. In this method the objective of creating undercut is to reduce the effective bending stiffness of the strata by creating fractures at ~~the~~ critical location such that the strata under consideration is not able to offer significant resistence against its bending thereby ultimately

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facilitating the process of caving. The critical location for introduction of undercut is decided based on the excavation geometry where there is a likelyhood of development of max. bending stress. Generally undercut is introduced in the rock layer by carrying out blasting for a smaller length of the blast hole which in turn creates fracked lines inside the rockmass in addition to significant damage to the already existing joints and the fractures. This type of blasting not only reduces the bending stiffness along the critical plane but also the strength of the rockmass. Field experience shows that it had been possible to reduce the goaf exposure for mainfall from 10000 m^2 to 6000 m^2 by introduction of undercut as a method implemented for induction of caving in a blasting gallery working.

Induced Caving By Blasting:-

Note:- Blasting length > 9m is treated as deep hole blasting.

Induced caving by blasting from surface → Induction of caving by deep hole surface blasting had been successfully implemented in 3 longwall projects of south eastern Coalfields limited (SECL). The method for the first time in our country was experimented for inducing the caving for controlled loading on the roof support in P1 panel of Bolrompur colliery where the earlier strata control experiences revealed that it was not possible to continue safe longwall operation until the length of overhang is effectively controlled to have reasonable load on the support of given capacity which was 4X450 ton for a face length of 150 m. The geomining condition of the longwall panel is given in table below:-

Deep hole surface blasting at Bolrompur P-1 panel:-

Name of the panel → P-1 panel
Name of the seam → Seam - V (Passang Seam)
Thickness of Seam → 2.4m (average)
Length of the face → 150m
Length of the panel → 800m
Height of extraction → Full thickness
gradient of the seam → 1 in 34 to 1 in 51
Minimum depth → 45m (approx)
Maximum depth → 50m (approx)
Thickness of hard cover → Varying from 25 m to 35m (approx)
Face span → Max (6.5m), (Min) 5.8m
Immediate roof → Shale overlain by sandstone
Immediate floor → Sand stone

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• Methodology.

– Step-1 : Identification of strong bed –

• The most important task is to locate the strong bed, which is causing delayed caving and mainly responsible for imposing the load on the supports.

For this it requires:–

• Drilling of fresh boreholes of NX size with double tube core barrel at a suitable interval to know:

– Variation in the RQD of the overlying roof rocks above coal seam

(Double tube core barrel used) ← – Core pieces length as an indicator of strong rock bed

– Testing of rock samples of different characteristics for UCS

– Caviability assessment

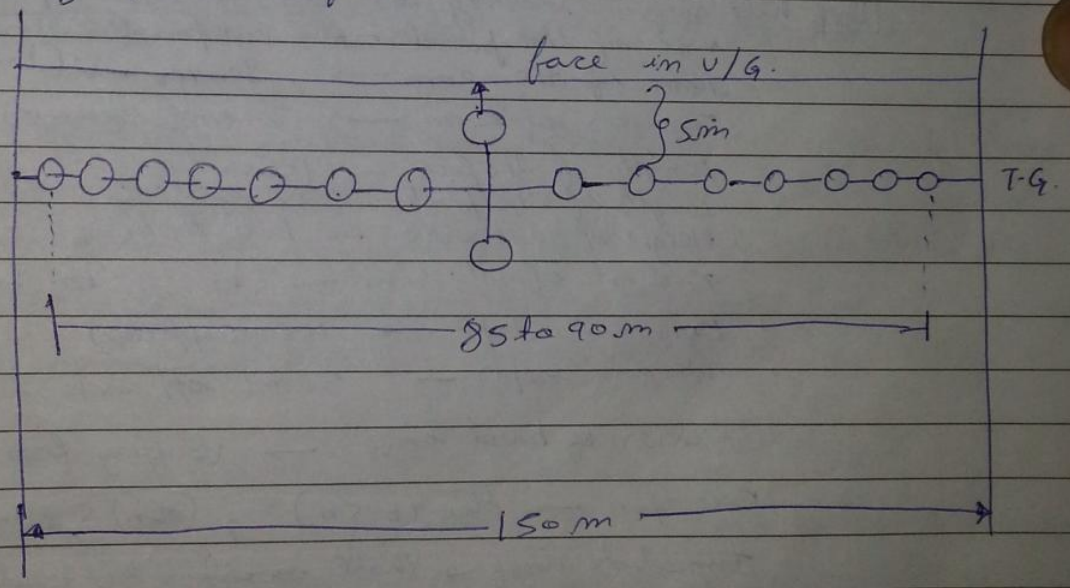
– Location of strong bed on the basis of:–

(a) Physico-mechanical properties as well as caviility assessment

(b) Strata movement observations by Borehole extensometer installed on the surface in center of the face.

• Design of blasting pattern–

– Single row pattern → In this pattern blasting holes are drilled in midzone of the face in a single row at spacing of 6 to 8 m depending upon rock mass massiveness and caving index value. The blasting is carried out when face advances by 10 to 12 m ahead of the blasting row. This is the most general blasting pattern used to induce caving from surface.



⇒ These blast holes are drilled on the surface at a distance of 5 m behind the face. and by the time the face is at a distance of 15 m from the row of blast holes (i.e. advanced by 10 m) then these holes are to be blasted.

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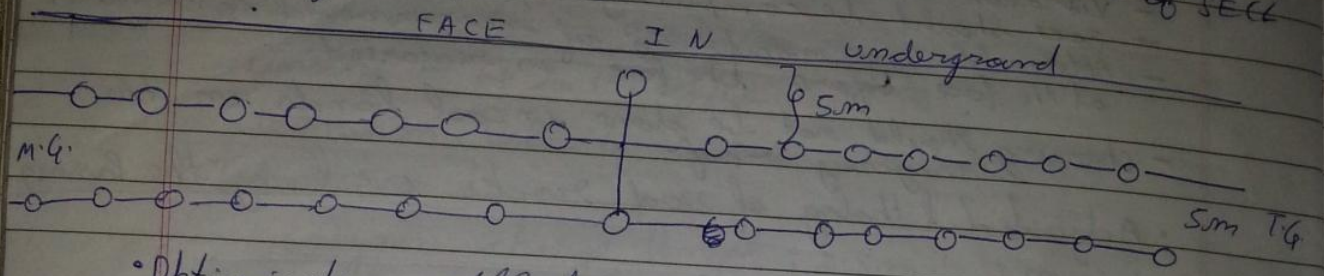
• Staggered holes pattern in 2 rows →
In this pattern blasting holes are drilled in mid zone of the face in a staggered pattern in 2 rows which are at an interval of 5 m as shown in the figure below. This pattern is used when there is a large overhang of strong strata behind the face. This pattern was carried out and at Rajendra mines of SECL in longwall panel-1.



• Optimisation of blasting parameters –
- The optimisation has to be done in following terms:-
(a) depth of blast hole and its diameter
(b) Decking position – single/double
(c) Quantity of explosive per deck or per hole
(d) Span of blasting in a single row
(e) Design of holes in center of the blasting row
(f) Spacing between the two holes in a row
(g) interval between two rows of blasting.

In the blasting hole → the explosive is not placed in the entire length of the hole but only in locations of strong beds. By targetting only the intended locations for blasting we have 2 advantages:
(a) reduced vibration
(b) reduced consumption of blasting explosive
This process of placing the explosive in desired location in the hole with waste in the remaining length of the hole is called decking.

• Correlation of effectiveness of blasting with face observations –
- weighting severity – leg closure, number of bleeding leg circuits
- weighting interval
- Duration of weighting
- The optimum blasting pattern with the existing support is one which makes the support to cope up with the strata load during the weighting without too much leg closure and avoiding structural damage i.e. making inadequate support without blasting to adequate one with blasting.

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07/1/2013.

GSPS Sir (continued)

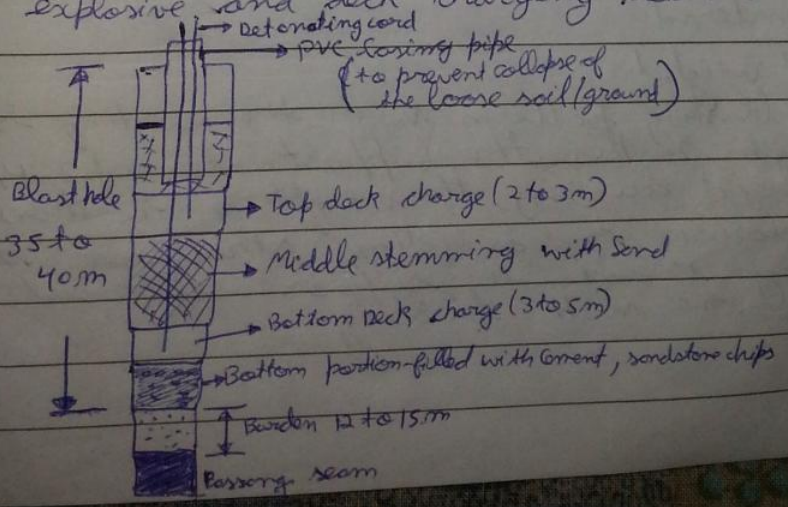
- Code of Practice (Safety code of practice)
  - Blasting should be conducted 10 to 15 m behind the face towards goaf
  - At least 5m burden should be there from bottom of the hole to the roof of the coal seam
  - Vibration measurement
  - Face should be completely stopped
  - After blasting monitoring of environmental conditions at the face before the face is restarted.
  - Charging should not be done in coal bands in the overlying roof.
- Adjusted litholog of roof rocks overlying the Passang Seam.

Bed	Normal phase (within 160m) Thickness (m)	RQD (%)	Blasting Phase Thickness (m)	RQD (%)
Bed 1	5.325	45	3.9	34
Bed 2	6.145	80	7.2	76
Bed 3	3.39	50	6.1	35
Bed 4	13.00	75	12.9	74
Bed 5	11.02	40	10.8	40
Bed 6	10.62	0	9.1	0

These act as competent rock beds and blasting should be focused on these beds.

- The study of strata formation indicates that Bed II and Bed IV are strong and create weightings at the face.
- However the borehole extensometer observation showed that Bed II is responsible for minor weightings only. It is the breakage of bed IV which causes major weightings and associated difficulties
- Therefore blasting was proposed to create fracture in Bed IV at regular intervals.

- Blast pattern
  - 16 to 18 holes of 100 mm diameter are drilled in a single row pattern with a spacing of 3 to 8 m from surface in midzone of the face with DTH drill machine.
  - A typical pattern of charging of blast hole along with explosive and deck charging details is shown in Figure.



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Details of Explosive:-

Explosive name — Apradyne or shakti prime
Cartridge length — 40cm
Cartridge diameter — 83mm
charge per cartridge — 2.78kg
charge density = 9.2kg/m
Density = 1.1 to 1.2 g/cc
Detonation Velocity = 3400 to 4300 m/s

Top Deck:-

column charge length : 2.5m
No. of Cartridge : 08
Total charge in deck : 22.20 kg

Bottom Deck:-

column charge length : 3.0m
No. of Cartridge : 10
Total charge in deck = 27.8 Kg

Initiation System:-

In hole delays of 200 ms, 250 ms and NDT of 17 ms and 42 ms

- The optimum parameters for surface blasting have been found out after observing the weighting severity at the face corresponding to blast design pattern and charging density per hole.

The established parameters for ~~effective~~ effective induced blasting from surface at Balrampur mine are as follows:-

Diameter of the hole : 100 mm
Depth of the hole : 30-38m depending upon the location of strong bed above the coal seam

Number of holes in a row : 17
Spacing between holes in a row : 8m
optimum spacing between the rows : 15m
charge per hole : 50kg in two decks

- Effect of Surface blasting on strata control:-

- The intend and intensity of weightings has been reduced and working conditions have been improved after blasting. In case of more than 60% of total weighting during blasting phase, periodic weighting took place after an intend within 22m.
- No. of bleeding log circuits reduced to 25 to 84 during weighting after blasting which were nearly 120 log circuits before adopting induced surface blasting.

These — act as competent rock bed and should focus on these beds

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- Supports never get solid during weighting at the face. (complete closure of supports)

Advantage of deep hole surface blasting:-

1.) Design of blast holes and its execution for successful implementation of induced caving method is comparatively easier.

2.) There is ample space available on the surface for drilling the blast holes in advance and carrying out induced blasting, maintaining a desired level of phase difference with the face advance

3.) Blasting hazards and corresponding safety measures are not so stringent in case of blasting from surface. It is easier to obtain statutory approval for deep hole surface blasting.

4.) It is possible to induce fracture in a large no. of target rock beds, consuming a very large amount of explosive as compared to the maximum permissible charge that can be allowed for underground blasting application.

5.) The dangers associated with post blasting fumes pull out ~~shot~~ shots, difficulty in handling of misfired shots, etc is not there in this case

Limitation of deep hole surface blasting:-

1.) This method can be applied for induction of caving only at shallow depth of cover

2.) The surface should be free of any constant.

3.) Although the blasting hazards are very small and insignificant in terms of their effect on environment but the roof supports deployed in underground face is subjected to extreme dynamic loading caused by high vibration level. The ultimate effect of this vibration on the support structure is realised in the form of highly prevelant fatigue failure which is not adequately addressed in the design of the structure.

BLASTING FROM UNDER GROUND:-

Applicability:-
1.) When target roof is very ~~close~~ close to the coal seam
2.) Cover depth is very high so surface blasting is not possible.

- The method is frequently used at number of mines to induced caving of the overhanging immediate roof in the goaf.
- In this method, the holes are drilled in the roof from goaf edge or in between in the powered supports with ordinary coal drill (length of hole is 1.5m only)

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(least workers at the face) The holes are charged and blasted during shift changing hours. Drilling the holes and charging for a longer length is somewhat difficult and the height of blasting depends on the drilling length.

Limitation— The method is applicable to bring down the strong bed only when it is very near to the coal seam. The induced blasting from underground has been carried out at blasting gallery panel at GDK-10 incline of SCCL and chora colliery of ECL by drilling the holes by jumbo drilling machine. (14m holes drilled)

Limitation of blasting:-

Induced caving by blasting has many inherent limitations, which are enumerated below:-

- Drilling of blast holes from underground in the goaf is difficult and surface blasting is feasible only at shallow depths. Surface blasting has lot of constraints like depth of cover, topography of the surface, rainy season
- Blasting has natural hazards associated in the case of bulk transportation, charging, blasting etc.
- Blasting causes ventilation problems in underground mines due to toxic gases released after blasting and also increases temperature of the mine atmosphere.
- Blasting cannot be permitted in highly gassy mines.
- Use of non-permitted high explosive and detonating fuse is not permitted in underground coal mines.

Water Infusion technique:-

- In order to meet the challenges caused by hard strata, a new concept to improve cavability is being tried. The technique seeks to achieve strata control under hard and massive strata by altering its mechanical properties by water injection under high pressure

- This softening method of water injected is expected to have direct impact on caveability

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of overlying roof rock in longwall or bord and pillar mining operation.

- It is not limitations as such w.r.t mining method or gassiness of seam but the rock bed we want to target should have sufficient clay content and significant porosity.
- The entire process of water infusion technique can be enumerated under two broad headings:-
  (a) Laboratory test
  (b) field implementation

Laboratory Test:- (to testify the rock mass, to what extent it is suitable for application of this method)

- It has been found that all rocks in general have the property of weakening under water immersion but the degree of weakening varies from rock to rock and depends on grain size, binding materials of grains, porosity and permeability, frequency of bedding planes, fractures, joints and other discontinuities of the rock.
- The softening method can be applied effectively on rocks in which the softening effect reaches certain extent. The same is achieved by laboratory testing of rock samples and studying the possibility of efficient application of the method. There are three important tests to be conducted prior to implementation of the technique in the field.
  - Petrographic test
  - Water immersion test
  - Permeability test

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Petrographic Test:- (to know the mineralogy of the rock)

- Aim of the petrographic tests is to determine, mineralogical constituents of rock matrix, grain size and cementing material, etc. A number of rock samples should be collected from different locales and analysed in the laboratory.
- Thorough investigation of petrographic features of the most commonly encountered sandstone in coal measure rocks, reveals that it is grayish white in colour and contains quartz, feldspar, mica and other minerals in varying proportion. The cementing material are generally clay, silica calcite. The grain size of sandstone varies from fine grain to coarse grain and even to very coarse grain.
- The framework of sandstone has great variation in their shape, size and composition. The grains may be round or ~~granular~~ angular in outline, loosely packed or dense in arrangements. The shape, size, and packing of the component grains are responsible for porosity and permeability of rocks.

(important because we have to inject water into the rock mass)

08/11/2013

GSPS Sir (Continued)

Water Immersion Test:-

- The objective of the water immersion test is to quantify the reduction of strength of the rock upon absorption of water
- The experiment indicates that the degree to which rock weakened depends primarily on the type of cementing material and grain size of the rock matrix. Clay cemented sandstone is more susceptible to weakening on water infusion.

Observation on the basis of Binding material
- clay (test can be done)
- calcium silicate (method is not successful)

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- The study showed that loss of uniaxial compressive strength upon immersion in water varied from 18.1 percent of 79.2 percent of its original dry strength.

- The reduction was higher in case of coarse grained sandstone and interclation of shale and sandstone
- 80% of the loss of strength out of the total reduction in all the rock samples took place within first three days of immersion
- The total strength reduction was observed in 7 days of immersion, there after the loss of strength is negligible.

Permeability test:-

- Permeability depends on the porosity of rock and presence of fissures, fractures and other discontinuities. Porosity of rock is defined as the percentage of pore spaces in the total volume of the rock. The pore system constitutes channel ways for the passage of fluids.

- Permeability is the property of the rock, which allows the passage of fluids. A rock is said to be permeable if it permits an appreciable quantity of fluids to pass through it in a given time and impermeable if rate of passage is negligible. Permeability is a function of porosity and hydraulic head or pressure. Porosity of weak rock mass is a function of stress field imposed on the rock mass.

- Basically there are 2 types of permeability:-
  - Primary permeability
  - Secondary permeability

Primary permeability depends on micro fissures and micro fractures hence it is related to rock matrix. Secondary permeability depends on micro factures and macro fractures hence the secondary permeability is related to rockmass.

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Secondary permeability is responsible for flow of fluid in the rockmass in the field. It is always higher than primary permeability.
Primary permeability can be found in the laboratory, but the latter can not be found in the laboratory and can only be estimated from tests in-situ.

permeability → $K = \frac{QL}{PA}$

Q → discharge of percolation through specimen
L → length of the specimen
A → Cross-sectional area of specimen
P → Pressure difference

Secondary permeability of the rockmass can be found from field in-situ. From the primary permeability test conducted on sandstone rocks in laboratory, the permeability is found to be 2.188 micro Darcy. About 1007 PSi pressure is required to have a flow of 2.5×10^{-5} l/min for a length of 58 mm of rock sample.

- In actual field observation at Parascole, it has been observed that 68 l/min of water was flowing in the rockmass at a pressure of 100 PSi only.
- This shows that primary permeability is insignificant and contributes negligible flow of water in the rockmass. The secondary permeability is the major component, which is responsible for the flow of water.

· PRINCIPLE OF WATER INFUSION:-

- The mechanism of water infusion under higher pressure on the rock can be summarized as follows:
  - Rock consists of micro fissures, cracks, joints, bedding planes and other discontinuities.

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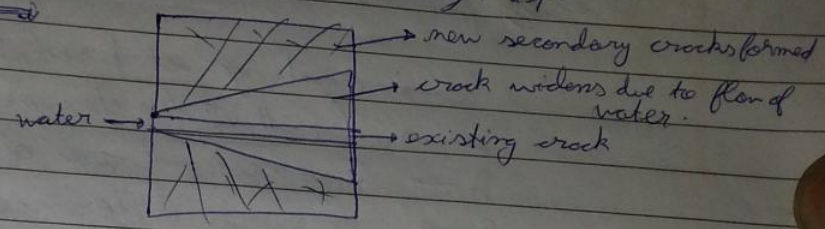
Due to the fact that water by way of joints and discontinuities destroy the integrity of the rockmass. High pressure water has wedge effect on the rock, widens the existing discontinuities and also creates new cracks and gradually breaks the rock. Hence high pressure water reduces the effective resistance the rockmass provides to the flow of water through it.

~~When rock~~



- When rock immersed in water, an intensive bond occurs between the layers of the binding material up on absorption of water, forming crystallization water, free water and hydration water. Surface of clay particle is negative charged thus forming electrified hydrated molecule. These particles repel each other, which destroy binding force and greatly reduce the strength of rock.
- In some cases where pyrites are associated with sandstone they react with water to form sulphuric acid, which ~~associates~~ accelerates the dissolving process of the carbonates of calcium and magnesium.

Methodology of drilling holes in Longwall:-

- In India longwall faces of length 80 to 150m are common, in some extreme cases 50 m long faces can also be seen.

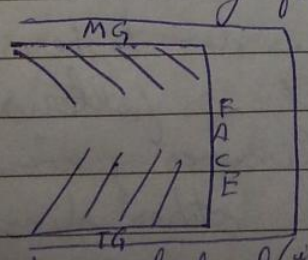
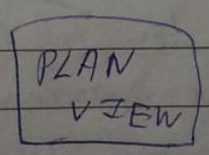


FIG 1:- Method in wider panel (injection holes towards the face)



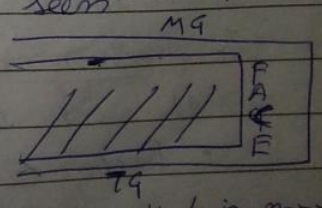


FIG 2:- Method in narrow panel (injection holes towards the face from only one gate road)

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- The holes should be slightly oriented (15 to 20°) towards goaf (against direction of extraction) so that the injected portion gradually exposes in the goaf

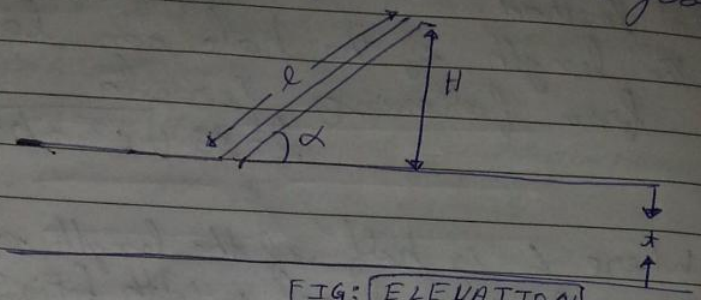

FIG: ELEVATION

- Major parameters of water injection are:
  - Length of the hole
  - Angle of the hole
  - Spacing of the holes
  - water ~~g~~ quantity per hole
  - Injection pressure

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GSPS Sir (continued)

- The above parameters depend on geo-mining parameters and physico-mechanical properties of overlying ~~rock~~ rocks.
- The length of the hole (l) and its angle (α) depends on the caving height (h). The caving height can be calculated from a simple formula:-

  $h = t/(K-1)$, where, t is the height of extraction and K is bulking factor.

  K depends on
  1.) type of rock
  2.) Compressive strength
  3.) Joint properties
  4.) Height of fall

- Bulking factor may be 1.05 to 1.10 for hard and massive sandstone and laminated rock of high compressive strength which caves in big blocks, 1.2 for laminated rocks low or moderate strength.

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~~Higher~~ • Lower is the bulking factor higher is caving height so larger is the hole required to be drilled for water injection method.

• The length of the hole can be calculated from following formula

$$l = \sqrt{L^2 + h^2}$$

• Where L is half of the length of the face in case of wider panel width (Fig 3.a) and equal ~~to~~ length of the face in narrow panel width (Fig 3 b). In fact the length of the hole is shorter than the calculated length l, about 1–1.5 times of the wetted radius (area of zone of influence)

• Inclination of the hole (α) for horizontal can be calculated as follows :-

$$\alpha = \tan^{-1}(h/L)$$

• Hole spacing depends on the characteristics of overlying roof rocks like porosity, presence of fractures, discontinuities and permeability.

• The spacing of the holes is equal to wetted radius of the hole which varies from 15 m to 30 m.

• Quantity of water per hole is an important factor, it should be fixed judiciously. If water is injected below a certain quantity, it will have minimum or no effect on weakening

• The quantity per hole depends on the area to be covered by the hole and saturation capacity of the rock.

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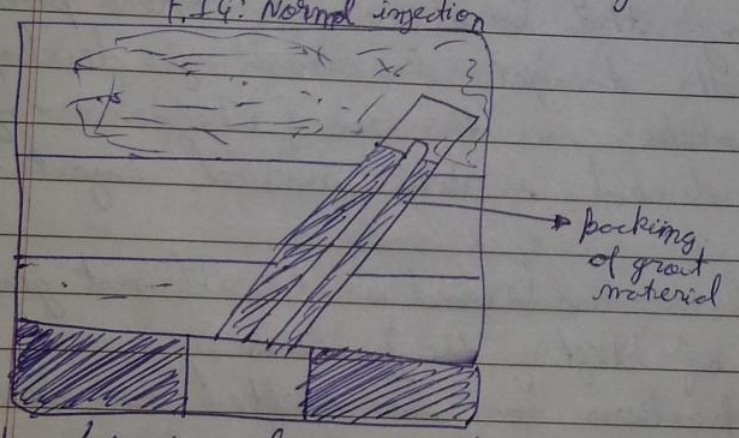
- Saturation capacity of a rock means the quantity of water absorbed by the rock for each a point where further reduction of strength is not possible.
- Generally the water quantity requirement varies from 30000 to 56000 liters.
- The maximum ~~length~~ injection pressure depends on the resistance of roof rocks to passage of water, which in turn is a factor of porosity and permeability and area of opening of joints, fissures, cracks etc. It generally varies between 8-15 MPa.

Methods of water injection →

1) Normal injection
2) Controlled injection

F.IG: Normal injection



~~Strength~~

- grouting / packing length should be such that water does not seep into the effective width of the immediate roof having an adverse effect on the immediate roof thereby requiring fresh support due to its deteriation.
- But the location of propagation of fractures is not fixed. In the roof where caving is to be induced, cracks develop in areas of low pressure in any direction and location

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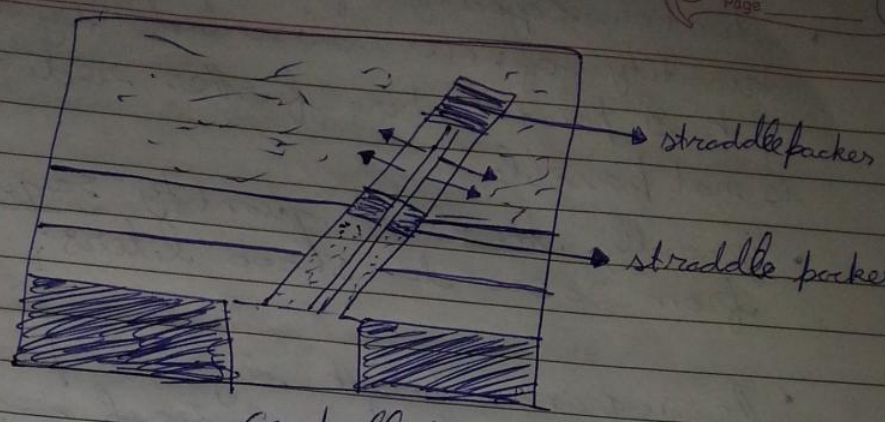

Controlled injection (hydrofracturing)

- ~~Normal inj~~ • In controlled injection method straddle packers are used ~~such~~ such that no water seeps into the immediate roof layer ~~and well as~~ (b) ~~they~~ ~~at~~ and at the blind end of the hole
- These straddle packers are such that on reducing the water pressure they can be withdrawn
- But changing the location of these packers we can fix the target location of development of cracks ~~unlike~~ in the roof where caving is to be ~~to~~ induced unlike normal injection technique.
- Grouting material cannot be used as it will not give us the flexibility ~~to~~ of shifting the packers.
- The pair of packers can be shifted to decide the zone of ~~in~~ development of cracks.

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- Normal injection system is adopted where the overlying beds having similar physico-mechanical properties
- In this method, an injection pipe is grouted in the hole with cement grouting pump or manually as shown in figure. The grouting length varies from 6m to 12m.
- The length of the grouting should be such that the injected water should enter into the beds, which are above the caved immediate bed.

- Controlled injection System is used where the overlying beds having different physico-mechanical properties.
- In this method strong and massive strata (Bridging strata) is identified and the water is injected in that particular strata with help of packer as shown in figure

Conclusion:-

- Safe operation of underground mines particularly high extraction workings like longwall under hard roof is a difficult task. Problems like face spalling, cavity formation, air blasts and collapse of faces are common and demand high capacity special feature supports, which affect the economy of mining operation

- Further, using high capacity support only can not guarantee for extraction of

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Coal safety and economically under such conditions and adoption of a suitable roof management technique becomes inevitable.

- Controlling the extraction width by means of shorter face length has some demerits in terms of frequent face transfers and low production face. Induced caving by blasting is a popular hard roof management technique, but the method has inherent limitations for its viable application in all geo-mining conditions.

- Water infusion technique has got rich potential to work as a technique for improving the rock cavability while introducing longwall technology under massive and hard roof conditions. In future, the technique will play a dominant role in hard roof management in Indian longwall mining.

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ROCK BURST

Rock burst is a term generally used in metal mines. The equivalent term for coal mines is coal bumps. The phenomena is same in both cases.

⇒ Rock burst is a term that is used to explain violent release of (elastic) strain energy in metalliferous mine workings.

⇒ Rock Burst is the term applied to the case of rock failure in underground that combines suddenness and violence on a sufficiently large scale to endanger openings, equipment and/or personnel.

⇒ It is manifested by violent fracturing of portion of rock around an underground opening by audiable noise usually with fragments of slabs projected from walls, roof or floor. Bursts are the sudden release of elastic strain energy stored in the pillar/rock.

⇒ Rock burst is a phenomena that occurs in U/G metalliferous mines whereas bump is a phenomena of sudden violent bursting of coal from a pillar or pillars in coal mines

⇒ Bump is characterised by low magnitude compares to rock burst because the capability of coal measure rocks to store strain energy is much lower compared to the amt that can be stored in the host rock in metal mines.

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- Rock burst occur when static stress of the underground rock exceeds the static strength.

CHARACTERISTICS OF ROCK BURST:-

- Sudden release of strain energy in a violent manner in a rock mass during the process, the volume of the rock involved in this process must be sufficiently large to endanger the integrity of the mining structure or the safety of the persons or the machines deployed there at.

- The process is violent in the sense that the amount of strain energy released should be of sufficient magnitude to cause danger and difficulties in mining.

- The process should be associated with failure and expulsion of rock mass in a mine of sufficient magnitude that may create danger and/or difficulties in mining.

Classification of Rock Burst:-

- Rock bursts are classified on the basis of

— Amount of energy released (ft-lb)

- Very small (10^3) ~~small~~ small (10^4),

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Medium (10^5), large (10^6), very large (10^8), very-very large (10^9)

- Mechanism of energy release
  - Pressure burst, shock burst

- Locality of occurrence
  - Stope burst, pillar burst, area burst, shaft burst.

- Shock burst occurs when a strong massive strata laying immediately above the excavated area ruptures as a beam or flat arch and set up shock wave that is transmitted downward to the pillars on the sides of development headings / stopes. If the underlying pillars / area is already highly stressed, the transmitted shock wave may cause rock burst / pillar burst.

- Pressure burst occurs when a strong and brittle pillar / rock is stressed beyond its strength leading to its sudden and violent failure.

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Mechanism of Rock Burst

- Rock bursts are sudden manifestation of energy. They have their origin in some form of elastic strain energy. This energy is regarded as energy stored in an elastic rock body by virtue of the work done on it during the development of elastic strain by application of external forces.

- Rock burst occurs more in deeper mines because the elastic relaxation is more in deeper mines so the counter parts which accomodates this energy is overlooded and as a result the phenomena of ~~&~~ rock burst occurs. ex.) Gold mines.

- The work done in producing an elastic strain, being stored as strain energy in the strained material, can reappear on removal of this load in the form of recovery of material towards its original form or if the forces are applied until the material is fractured partly by the work done in the process of fracturing and partly by recovery from strain of the fractured portion.

- The storage of elastic strain energy in the rock is similar to the action of a spring. If a spring is compressed, it stores energy.

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- If the load on the spring is released slowly, the energy is dissipated slowly. However, if the spring is compressed beyond its strength, it fractures and the energy stored in the spring is released suddenly.

14/11/2013.

- Rock bursts in mines are considered to be those produced by strain energy induced by application of external compressive forces on the ore/coal and the ~~associa~~ associated strata.

- The application of external pressure is explained by elastic rebound theory, pressure dome theory, cantilever theory and stiffness theory

- The most widely accepted theory is the pressure dome theory.
- Pressure dome theory considers that fractures progresses outwards from an excavation and a dome like area of fractured and destressed rock forms the excavation surrounded by a high stress zone which exist in the unfractured rock.
- Large bursts occur when the load supported by dome is concentrated in pillars and when two or more dome meet resulting in furthur stress concentration.

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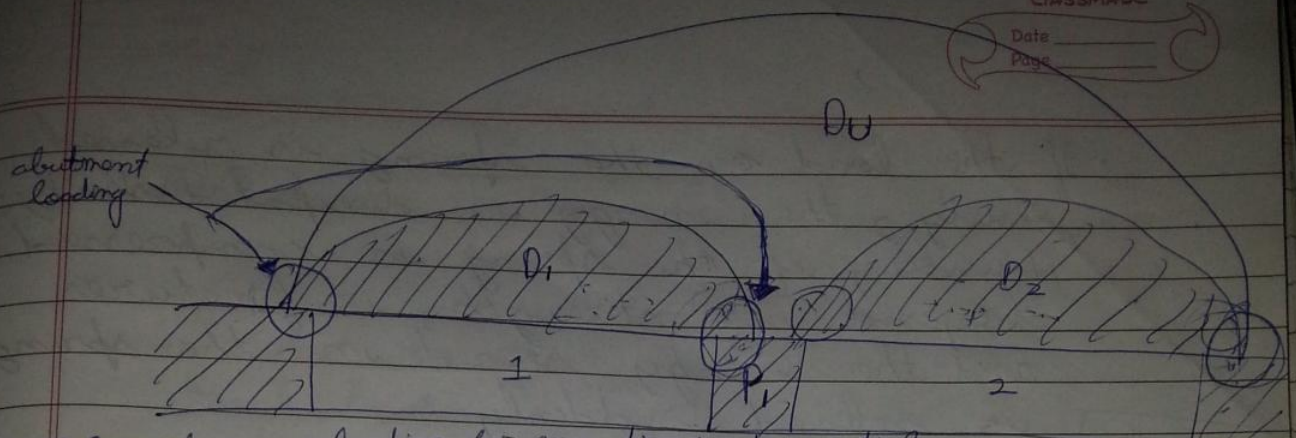
PRESSURE DOME THEORY



D_1, D_2 → Domes of stress relaxation for excavations 1 and 2 respectively

If we start working out the pillar P_1 more and more then the domes of pressure D_1 and D_2 may overlap and form an ultimate dome D_U. The volume of rock in the ultimate dome is much much larger than the algebraic sum of the volume of rock of individual domes D_1 and D_2. So the abutment loading due to dome D_U is very magnified compared to the abutment loading due to individual ~~domes~~ pressure domes. Hence the strain energy transferred to the pillars now due to dome D_U abutment stress is much higher than the ability of the pillars to accomodate elastic strain energy so there is a sudden release of energy and the phenomena of Rock burst takes place.

Triggering agents of rock burst

- Depth
- Strong overlying rock stratum
- Strong floor
- Compressive strength of rock/coal
- Brittleness of rock/coal
- Geological factors
- Environmental factors
- Operational factors
- Improper mining method

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Depth → with increased depth of working, the elastic deformation is very high so ~~amt. of energy~~ elastic component of stress released is very high so the counter part which accomodates the strain energy will not be able to handle such high energy. so rock burst occurs.

- Vertical pressure exerted around an opening is proportional to the height of the cover, depth. The deeper the working, the higher the vertical pressure imposed on a pillar. If the pressure exceeds the failure strength, it could cause bursting of the pillar.

- The amount of pressure imposed on the pillars is proportional to the depth of cover and the percentage of extraction. Large extraction ratio induce higher average pressure on the pillars. Further, with the increase in depth, there is an increased tendency of burst.

Compressive strength of the rock/coal →

Higher the compressive strength of rock that forms pillars, higher is the capability of rock to store strain energy and therefore, higher is the magnitude of bump.

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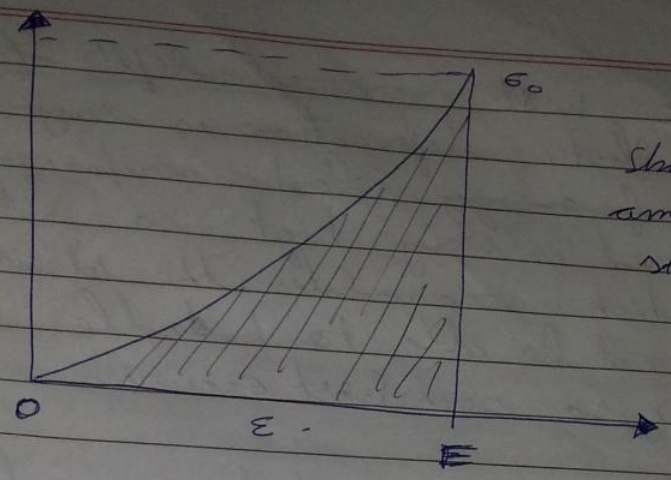
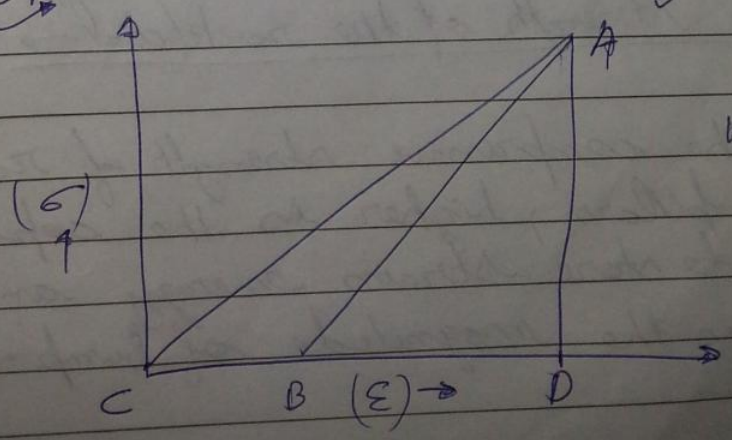


Shaded area = amount of energy stored = $\sigma_0 \varepsilon / 2$

- <u>Brittleness of the rock/coal:</u> →
  Coal/rock burst is related to its mechanical properties. Since burst is a sudden manifestation of the release of stored elastic strain energy, the tendency of rock/coal to burst depend on its ability to accumulate elastic strain energy.

- Rock liability to burst can be expressed by strain energy storage index (SESI) which is the ratio of elastic strain energy, E_e accumulated to the specimen to the (plastic) permanent strain energy E_p



$$W_{ET} = \frac{E_e}{E_p} = \frac{\text{area } ABD}{\text{area } ABC}$$

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- The larger the value of WET, the greater the liability to burst. Using this index, the liability of rock and coal to undergo burst can be classified as follows:

	For Coal	For rock
Severely liable	75	715
Slightly liable	2-5	5-15
Not liable	<2	<5

- Higher the brittleness, lower is the permanent strain energy and higher the energy index, higher is the liability ~~of~~ to burst.

- <u>Geological factors:-</u>
Geological discontinuities like folds, faults and dykes are known to affect the occurrence of burst.
Most of the shock burst are observed along the fault plane which may be due to high residual stress or sharp change in stress distribution near these planes.

When dykes are approached, the frequency of burst is increased.

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• Environmental factors:-

- It has been observed that frequency of rock bursts is higher immediately after blasting
- Further analyses reveals that blasting activities in workings in close proximity to fault or dyke triggers to initial burst that would normally have occured at later period.

- Sudden variation in temperature and humidity can also act as triggering agent to rock burst
- Moisture has also been mentioned as having the triggering effect. Critical evidence in KGF has shown positive deviation of rock burst incidence during those months of the year which are characterised by high rainfall.

• Operational factors:-

- Some of the operational factors causing excessive stress concentration are also responsible for aggravating bursts. They are
  - Narrow drives in abutment zone liable to burst
  - Blasting effect often induce bumps.

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- Remnant pillars in old workings may transfer load from neighbouring beds even at shallow depth causing stress concentration which may result into burst. If the remnant pillars fail suddenly, part of all the load is instantly transferred to the neighbouring pillar producing high impact stress resulting in occurence of rock burst. Similar features occur with safety pillars which may occur at large distance apart from the workings.

• Strong overlying rock stratum.

- It has been found that burst prone rock/ore are invariably overlain by a strong massive rock stratum such as sandstone or conglomerate.
- However, the strong massive rock will not be effective in causing bumps unless it lies directly above the coal seam/ore body and there is no structurally weak material of considerable thickness sandwiched between it and the ore.

- When massive layer is very close to the support structure then it fails after a large overhang and transmits a large amt. of energy to the resisting pillar causing rockburst while a weak material sandwithed between the massive layer and support element screens out the strain energy and prevents occurance of rock burst.

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· A strong floor

- A strong immediate floor is also necessary for a bump to occur. A pillar is likely to punch into a floor that is softer than the pillar itself and ~~dos~~ dissipate energy stored in the pillar.

Prediction of rock bursts / coal bumps

· Several methods can be used to predict the location and timing of an eminent rock burst.

· Drilling yield testing and micro seismic techniques are the two most frequently used methods for this purpose.

Drilling yield testing →

- The drilling yield testing ~~method~~ technique involves drilling a hole of 2-3" diameter in the rock or coal ahead of the face being worked.
- As drilling proceeds from the rib towards the centre of the blocks of ore/coal, the quantity of cutting chips ejected and any dynamic effects such as audible knocking, grating and jamming of the drill in the hole is observed.

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Micro Seismic detection →

It has been established that rock fracturing process under compression consists of three distinct three distinct stages – crack initiation, crack propagation and ultimate failure. The ultimate failure of the rock is the result of numerous cracks rather than a single crack.

- Each stage of crack development generates a stress wave that radiates and propagates away. This stress wave is called rock noise or acoustic emission, a seismic activity the magnitude of which is proportional to the size of the crack and the amount of displacement.

- The frequency spectrum of rock noise is very broad and ranges from 50 – 100000 hz. Both compression (P) and shear (S) waves are present but most of the energy in a rock noise wave train is associated with shear wave.

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Methods of fighting/mitigating rock bursts:-

- De-stressing
  - Drilling holes and light charges blasted
- Water infusion

  $W_{ET} = E_e / E_p$

  As a result of water infusion, E_p increases, thus W_{ET} decreases. Hence, proneness to rock burst also decreases.
- Operational command and control

  Modify → system of mining
  → system of deployment of manpower
  to minimize adverse effect of rock burst. → system of deployment of machinery

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Rock Instrumentation and Monitoring (GSPS Sir)

Need of instrumentation:-

→ whenever we have to validate a design, then this validation requires data (scientific)

→ If you want to assess the performance of any structure so that we can have an allowance to improve the design of the structure. So to check the performance instrumentation is required. (Performance validation).

- Rock Instrumentation encompasses instrumentation for site investigation and site characterisation, laboratory testing, and field monitoring to man made excavations on surface and below ground.

• Broad objectives include:-

- Characterization of rock mass behaviour
- Validation, performance evaluation and design optimisation
- Monitoring the behaviour of rock structure
- Development of site specific database.

Methods of Monitoring

- Visual
  - Laboratory based
  - field
- Instrumental
  - Laboratory based
  - field

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- Visual monitoring in field for ground control:- (requires lot of experience)
  - Crack development
  - Spalling
  - Cavity formation / fall of roof
  - Bending of support.
  - Precipitation of water through cracks
  - Movement of rats
  - Shearing of support wedges.

- Instrumental monitoring :—
  - Load Cell
  - Convergence indicator
  - Tell tale extensometer
  - Tape extensometer
  - Convergence meter

Load cell → used to measure induced load on the supports

Convergence indicator → used to measure the induced deformation taking place between roof and floor of any structure (indicates convergence)

Convergence meter → quantifies the convergence ~~and records it~~

extensometer → generally used to measure the deformation either within the roof as well as floor. In most cases it is used to locate the active parting planes which in turn helps us to design the supports.

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~~Tell tale~~ tape extensometer → generally used to measure spalling (sealing of rock mass from sides of the galleries)

- stress meter → that is used to measure induced stress in pillar or stook
- Vibration level recorder → to examine the amount of vibration induced due to blasting
- strain gauge → to measure surface deformations / strain that ~~safe~~ structure will suffer in course of time.
- Convergence recorder

- Micro - seismic monitoring system

- Data acquisition and recording

- Piezometer → used for measuring pore pressure

- inclinometers

- spiralarms → (point monitoring system). If CH_4 concentration is more than 0.75% it gives a beep and a visual red blinking signal.

- Magnetic type multipoint borehole extensometer → To monitor large boreholes with multiple anchors

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Planning for Rock Instrumentation:-

- Objective
- Type
- Specification
- Layout
- Monitoring schedule
- Format of observation
- Analysis and evaluation

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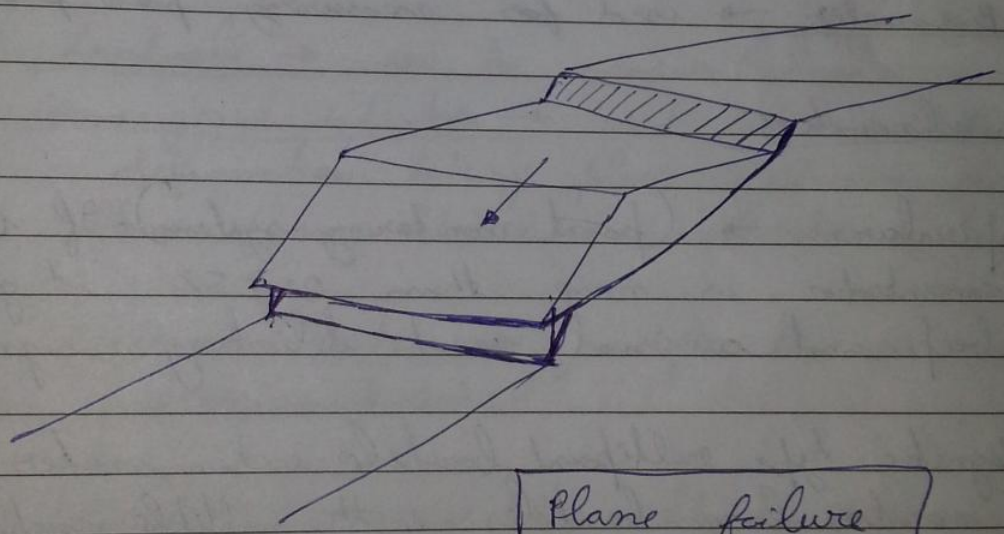
GROUND CONTROL

(BKS Sir)



Plane failure

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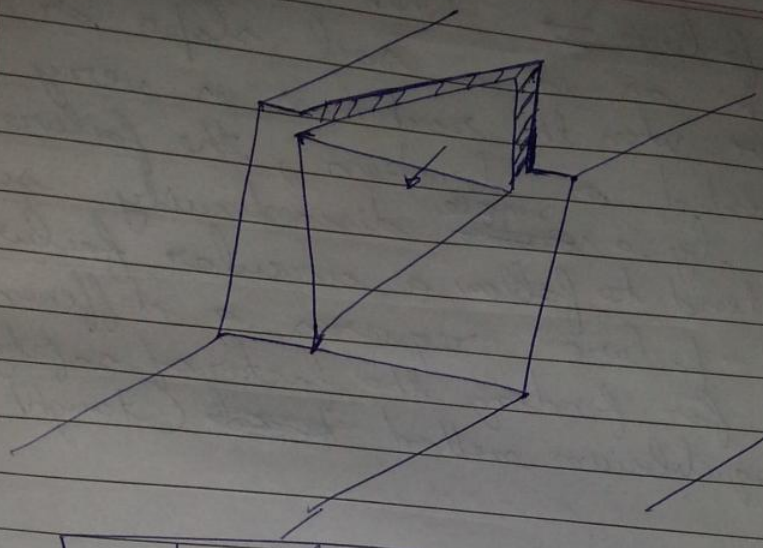
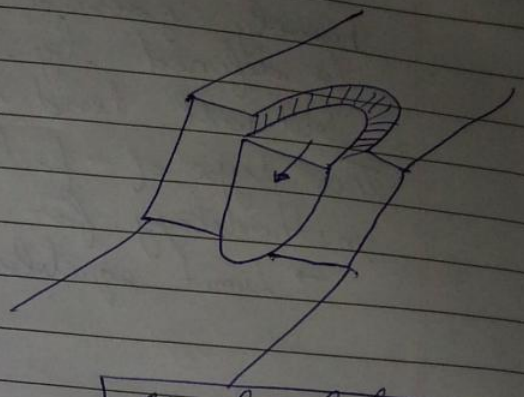


wedge failure



Circular failure

<u>Plane failure</u> as shown in the sketch occurs when a geological discontinuity such as a bedding plane or joint or fault strikes parallel to the slope face and dips into the excavation at an angle greater than the angle of friction. The calculation ~~for~~ of the factor of safety follows precisely the same pattern as we have discussed for a block resting on a rock surface.

wedge failure → when ~~the~~ two discontinuities strike obliquely across the slope face and their line of intersection daylights (dips) in the slope face. The wedge of rock resting on these discontinuities will slide down the line of intersection provided that inclination of this line is significantly greater than the angle of friction.

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Circular failure → when the material is very weak as in a soil slope or rock dump or when the rockmass is very heavily jointed or broken, the failure will be defined by a single discontinuity surface but will tend to follow a circular failure path. This type of failure requires a different methodology for finding the factor of safety.

→ Limit equilibrium method (taught in Lab)

To find FOS of wedge failure:-

1.) Calculate volume of wedge
2.) Calculate weight of wedge
3.) Calculate the reaction on 2 sides of the wedge depending on the obliqueness / inclination of sides of wedge
4.) Calculate friction
5.) Consider cohesion if required.

strength reduction technique to find FOS → we reduce both C and ϕ by a common factor and calculate the FOS for each instance and check the limiting condition when the slope fails.

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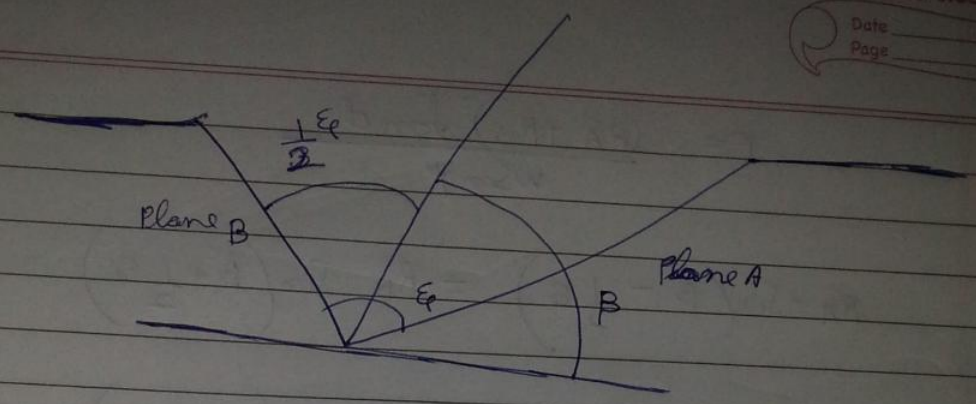

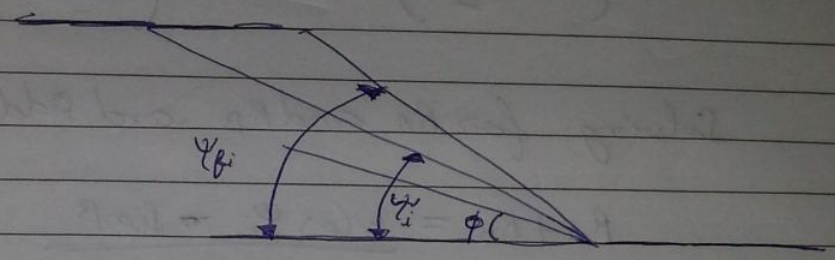

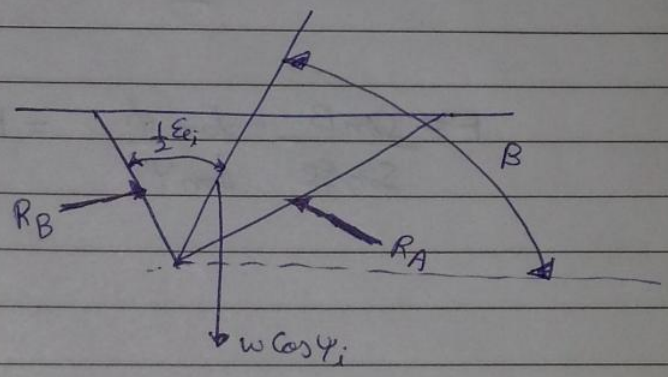

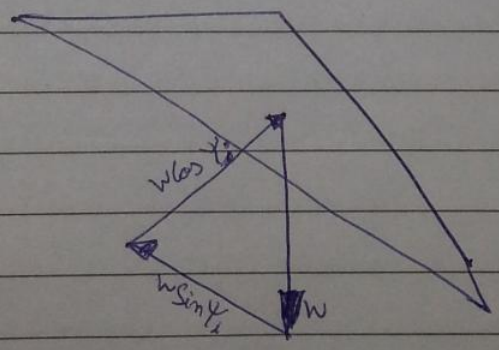

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$$F = \frac{(R_A + R_B)\tan\phi}{W\sin\psi}$$

$$R_A \cdot \sin\left(\beta - \frac{1}{2}\varepsilon\right) = R_B \cdot \sin\left(\beta + \frac{1}{2}\varepsilon\right)$$

$$R_A \cos\left(\beta - \frac{1}{2}\varepsilon\right) - R_B \cos\left(\beta + \frac{1}{2}\varepsilon\right) = W\cos\psi_i$$

Solving for R_A and R_B and adding:-

$$R_A + R_B = \frac{W\cos\psi_i \cdot \sin\beta}{\sin\frac{\varepsilon}{2}}$$

$$F = \frac{\sin\beta}{\sin\frac{\varepsilon}{2}} \cdot \frac{\tan\phi}{\tan\psi_i} = K \cdot F \text{ (intact rock)}$$

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