A STRUCTURAL ENGINEERING APPROACH TO LONGWALL ROCK MECHANICS IN MASSIVE STRATA

Ian Gray[1], Tim Gibbons[2]

INTRODUCTION

Geotechnically massive strata behave as strong units, rather than the geologically massive description which describes the sedimentary formation. The consequence of such geotechnically massive units to mining is a series of problems associated with these units failing to break up into small blocks.

This paper deals with the determination of what constitutes geotechnically massive strata. This is essentially a function of the geology and the mechanical properties of the geological stratigraphic units that need to be combined into geotechnical units. The mechanical properties depend on the rock itself and on the structure in the form of joints and faults. The importance of anisotropic rock behaviour cannot be underestimated and neither can the variations in geology.

Structural plate and beam analysis is presented as a method to determine where and at what span the initial fall and subsequent goaf falls will take place. While this is elastic analysis, it should be appreciated that the stronger rocks which lead to massive strata conditions will behave more elastically, and therefore this method of analysis is considered justified in most cases. This approach is considered to be far more usable and reliable than complex numerical analysis, which generally lacks the range of real rock behaviour as a fundamental input and is frequently uncheckable. The use of these established analytical techniques lends confidence to the solutions derived from their use. The pre-existing stresses are important as they will control initial goaf formation.

Dynamic support loading is considered as is preconditioning to break massive units down so that they break in smaller blocks.

PROBLEMS ENCOUNTED WITH MASSIVE STRATA

Strong massive strata behaves quite differently from weak rocks which break up readily under stress. The rock mass tends to behave as a series of strong plates which tend to move as units and only break when they reach a substantial size. This brings about a number of problems in mining which include the following:

Weightings – caused by the formation of large blocks which load the powered supports.

Windblasts – caused by the fall of large blocks that displace air and gas from the goaf.

Open fractures – which exist between blocks and may progress to surface. They may lead to water ingress or disrupt the ventilation of the mine.

Uneven subsidence – associated with the large block movement causing steps in the surface topography.

[1] *Principal Engineer, Sigra Pty Ltd. Email: ian@sigra.com.au Mob: +61 419 783 535*
[2] *Principal Mining Engineer, Sigra Pty Ltd. Email: timg@sigra.com.au Mob: +61 439 786 330*

Face spall – large lumps that fall on the longwall face causing risk to workers and stopping the operation of the longwall.

Coalbursts – forms of rib and face spall where coal is ejected at speed.

Tailgate problems – caused by the lateral movement of the roof towards the goaf, which shears the tailgate pillars.

GEOLOGY

The foundation of understanding how strata will behave is the geology of these. The determination of what strata exist, their thickness, continuity and state of jointing is essential. It is normal to conduct such investigations in several stages. If the existence of geotechnically massive units is suspected at the early stages of exploration then the next stages of exploration should be designed to determine if this is the case. The subsequent exploration may need to incorporate angled drilling to find joints, or surface seismic to help determine the thickness variations of various units. Detailed core logging for lithology is essential as is an assessment of structural features which should be achieved using both core examination and acoustic televiewer images so that the orientation of structural features within the rock mass may be determined.

MECHANICAL PROPERTIES OF MASSIVE STRATA

Geotechnically massive strata can be sandstones, siltstones, limestones and conglomerates. On occasions it can also be thick igneous sills. What makes it geotechnically massive is its strength and lack of jointing or weak bedding planes. It is quite possible to have a sandstone and a siltstone behave as single unit provided that the contact has sufficient cohesion. It is the strength of a unit of stratum of several units of strata that determines whether they will behave as a massive unit, or will readily break up.

The strengths that are important are the tensile strength of the rock, both across the bedding and in the direction of the bedding. Also, the shear strength of the rock. Here however the important shear strength is generally that which exists on the bedding planes. This is not usually measured. Specific tensile and shear tests are more useful than those currently adopted in geotechnical testing regimes are described by Gray (2020) and Gray and Wood (2022). They comprise axial and transverse testing of core and core discs to failure and direct shear or a modified GOST shear test to measure the bedding plane shear characteristics.

The elastic properties of rocks are also important as these affect the manner in which stress and strain are distributed. The measurement of the pre-failure, substantially elastic behaviour is covered in some detail by Gray, Zhao and Liu (2018). This paper describes testing by uniaxial, hydrostatic and triaxial processes to obtain the stress-strain characteristics of rocks. These vary from nearly linearly elastic to highly non-linear. In some cases, Young's modulus changes fourfold in the elastic range. It is also quite possible to get anisotropic behaviour with up to 2.5:1 lateral to vertical stiffness ratios. Some of the rocks display large plastic offset behaviour long before the strength of the rock begins to diminish.

Much has been written about the actual strength and modulus of rock in the field as opposed to that measured in laboratory samples. The difference is that the larger the rock mass being considered the more likely it is to contain flaws. These are structural features on which movement may take place.

The increased use of the geophysical strata rating (Hatherly et al, 2009), where geotechnical properties are derived from borehole geophysics, is not considered adequate in determining strata behaviour because the analysis of material properties is based upon isotropic solutions to sonic log behaviour.

The real anisotropy of the rock mass is extremely important in determining how caving behaviour will take place.

Where systematic joint systems exist the rock mass can no longer be regarded as geotechnically massive and jointing will generally control failure. The determination of the existence of such jointing is difficult where exploration drilling is by vertical holes and the dip of the joint systems is also near vertical. Where a structural unit appears joint free and therefore geotechnically massive exploration by angled core holes should be employed to determine whether any joint systems exist.

Where massive strata does exist goaf falls are frequently caused by significant geological discontinuities, typically faults and sandstone channels.

INITIAL STATE OF STRESS

The initial state of stress in horizontal lying sedimentary strata can be complex as it is affected by the depth of burial and erosion, diagenesis, thermal changes and loadings caused by tectonic effects. Where failure occurs in the form of faulting these are invariably areas of stress relief which tend to have high stress areas near fault extremities that do not daylight. Despite this complexity it is useful to start with a simple model of the stress distribution and then to modify it based on measurement and structural information.

This simple model is divided into three components. The first is derived from self weight which imposes the vertical stress. The second is the horizontal component due to self weight in a zero lateral strain environment. The third is the stress component that is due to external strain which is named for convenience ‘tectonic strain’, though it may be made up of a myriad of effects. This tectonic strain is frequently quite even or at least monotonically variable, whereas the stresses vary widely because of the different stiffnesses of the rock layers.

Figure 1 illustrates the concept of monotonically varying tectonic strains and the varying stresses that they induce in strata of differing elastic properties. While this is a theoretical example there are numerous cases where the tectonic strain approach can be shown to provide an adequate explanation of the stresses. There are also cases which are far more complex, especially where there are numerous faults. An example of this at Tahmoor Colliery is described in more detail by Gray, Wood and Shelukhina (2013).

Figure 1. Stresses and tectonic strains in an example of a layered sedimentary sequence

MECHANISMS OF FAILURE

Massive strata may in most instances be thought of as behaving as a series of plates of rock which are subject to initial stresses which change as the goaf is developed. The determination of what constitutes a plate is a function of the geology, the stresses that act to change the plate geometry (thicken or thin it) and the anisotropic strength of the rock.

Figure 2 shows the process of goaf formation in a longwall. (a) shows the start of the first panel of the longwall before the goaf falls. In this case the stresses on the edge of the goaf roof are maintained and the goaf edges are clamped. (b) shows the situation where the goaf has fallen behind the face (c) shows the start of the next panel before the goaf has fallen. (d) shows the situation when the goaf has formed over the second panel.

Case (a) may be viewed from a structural viewpoint as being primarily loaded by gravity within clamped edges. The vertical unloading will probably lead to delamination of the strata at some height above mining. In most cases this will occur when the weight of the hanging strata exceeds the tensile strength on some lamination. This delaminated plate of rock will deflect under its own weight and bend inducing tensile and compressive stresses in the plate. Failure usually occurs when the tensile stress at the upper edges of the roof plate exceeds the tensile stress of the rock. It is less likely to occur due to tensile stress at the base of the central span of the goaf. It is also possible that failure will occur due to compressive stress at the bottom of the goaf edge or at the top of the goaf plate at the central span. Sometimes failure may occur due to shear at the goaf edge. The determination of which of these mechanisms will initiate failure, and at what span, can be determined using structural plate mechanics.

Case (b) shows the situation where goaf had fallen. The face may advance beyond the goaf edge forming a cantilever of rock before failure occurs again. This length is the situation of periodic weighting.

Case (c) shows the situation which exists before the first fall of the second and subsequent longwalls. The adjacent goaf from the first panel relieves stress across the longwall panel but stress in the direction of the panel, and perpendicular to the face, continues to exist. In addition, the remnant pillar between the previous goaf and the rock plate which will fail and form the panel of the new goaf will be less stiff than the other edges to the plate. The tailgate pillar might be thought of as being simply supported (hinged). It may also behave as a sagging support if it has yielded. The main difference in structural behaviour is however the absence of lateral stress from the zone of the adjacent goaf.

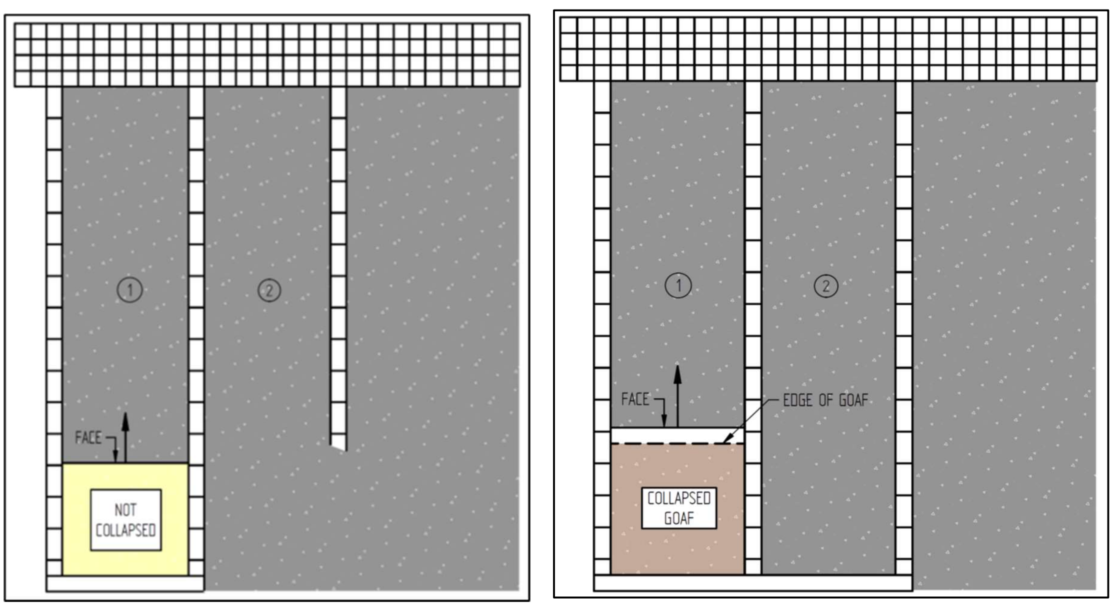
Case (d) shows the situation when the second goaf has formed. In this case minimal lateral stresses are likely to exist on the cantilever of rock sticking out over the supports. Its behaviour is similar to that of case (b).

The models used in analysis are for case (a) of a plate clamped on all edges with stresses across and along the plate. If the advance of the goaf before the roof plate fails is short the plate behaviour may be approximated as that of a beam with fixed edges and the stress along the plate. For cases (b) the case is of a plate with one free edge and three clamped edges. For case (c) the behaviour is of a plate with one hinged boundary over the tailgate pillar and three clamped edges and with stress existing in the direction of the panel but without transverse stress. The hinging makes little difference to the bending behaviour but the lack of lateral stress may make a significant difference to the span at goaf formation. Case (d) may be thought of as having two clamped edges, one at the face and one on the maingate side, a hinged or simply supported edge on the tailgate side and a free edge.

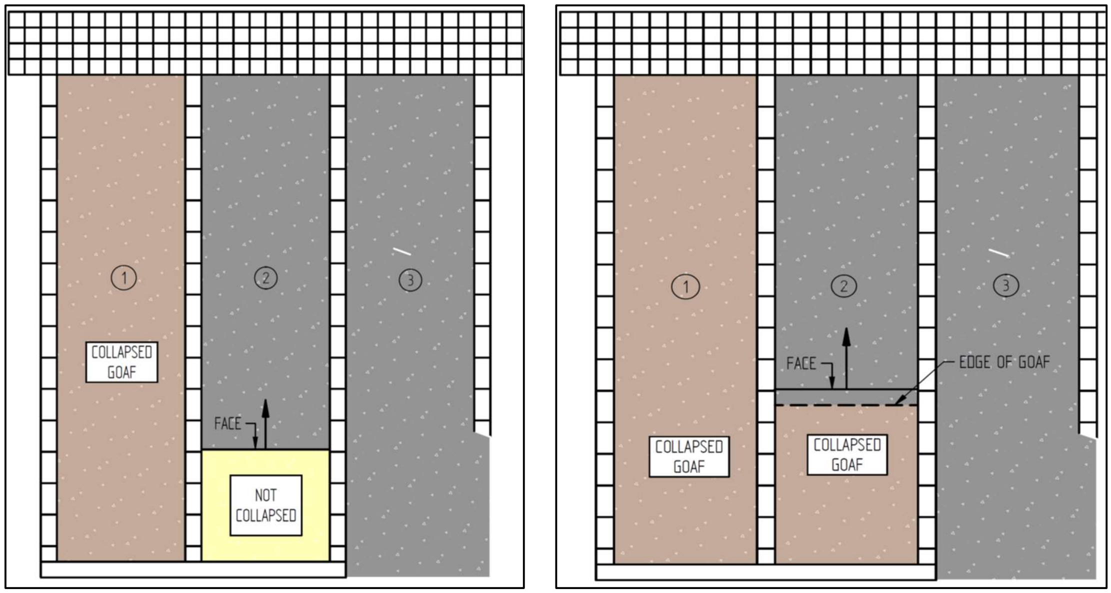
Cases (a) and (b) reduce to that of a beam with fixed edges in many cases where the fall will occur long before the goaf advances to anything approaching half the longwall width.

Cases (b) and (d) reduce to that of a cantilever if the fall occurs at small fractions of the face width.

Four plate cases have been analysed using a finite element structural plate model. Each of these corresponds to the situation described above. The approach used by Young, Budynas and Sadegh (2012) in *Roarke's Formulas for Stress and Strain, 8th Edition*, has been adopted. In each case several critical locations are analysed. The outer fibre bending stresses are described by **Equation 1.**



a b



c d

Figure 2. Longwall panel development, mining and goaf formation

Equation 1
$$\boldsymbol{\sigma_{MAX}} = \frac{\pm\boldsymbol{\beta\rho g b^2}}{\mathbf{t}}$$

The maximum edge shear stress occurs at the mid section of the plate and is described by

Equation 2
$$\boldsymbol{\tau_{max}} = \boldsymbol{\gamma}\frac{\boldsymbol{3\rho g a b}}{\boldsymbol{2L_R}}$$

Where a is the longwall width
b is the length of the plate in direction of the longwall
g is the gravitational acceleration
L_R is the total length of the supported sides of the plate
t is the thickness of the plate
σ_{MAX} is the maximum stress at the outer fibre of the plate at a specific location.
ρ is the density of the plate
β is a value that is dependent on the ratio of b/a
γ is a value that is dependent on the ratio of b/a

The values of β and γ are tabulated for specific critical points on the plates in the Addendum. Where the ratio of span to width (*b/a*), is less than a half, Cases (a) and (c) reduce to that of a fixed end beam as shown in Figure 3 where the outer fibre beam end stresses are given by Equation **3** and the maximum shear stress at the mid depth of the beam is given by Equation **4**. In Figure 3 ω is the weight per unit width of the beam and corresponds to $\rho g t$.

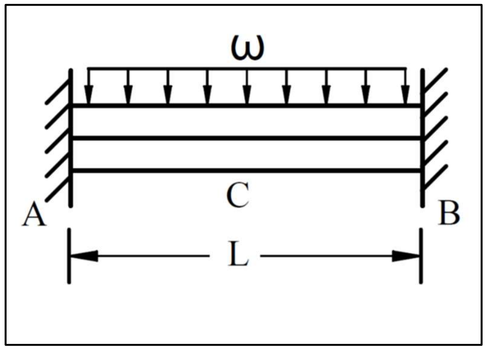


Figure 3. Schematic of beam with fixed ends.

Equation 3 $$\boldsymbol{\sigma_{Amax} = \sigma_{Bmax} = \pm\frac{\rho g L^2}{2t}}$$

Equation 4 $$\boldsymbol{\tau_{Amax} = \tau_{Bmax} = \frac{3\rho g L}{4}}$$

The bending stresses at mid beam are half that at the beam ends.

In cases (b) and (d) where there is an unsupported edge the for values of *b/a* < 0.1 the behaviour approximates that of a cantilever with a uniformly distributed load as shown in Figure 4.

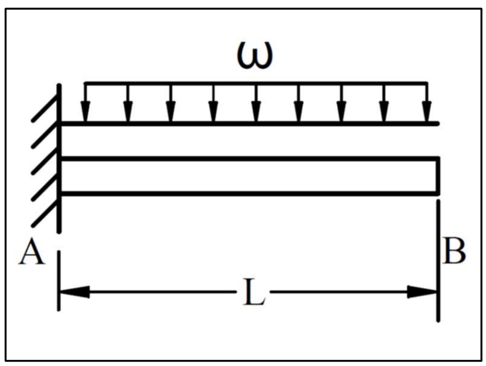


Figure 4. Schematic of cantilever fixed at one end with a uniform load.

The outer fibre bending stresses at point A on the cantilever are given by Equation **5** and the shear maximum shear stress at the mid depth of the cantilever at point A is given by Equation **6**.

Equation 5 $$\boldsymbol{\sigma_{Amax} = \pm\frac{3\rho g L^2}{d}}$$

Equation 6 $$\boldsymbol{\tau_{Amax} = \frac{3\rho g L}{2}}$$

Using Equation **5** it is possible to generate the overhang widths of cantilever blocks into the goaf for different tensile strengths and block thicknesses. These are plotted in Figure 5. For example, for a 10 m thick unit of 5 MPa tensile strength the overhang is 26 m. This length is the theoretical distance between weighting events on the longwall.

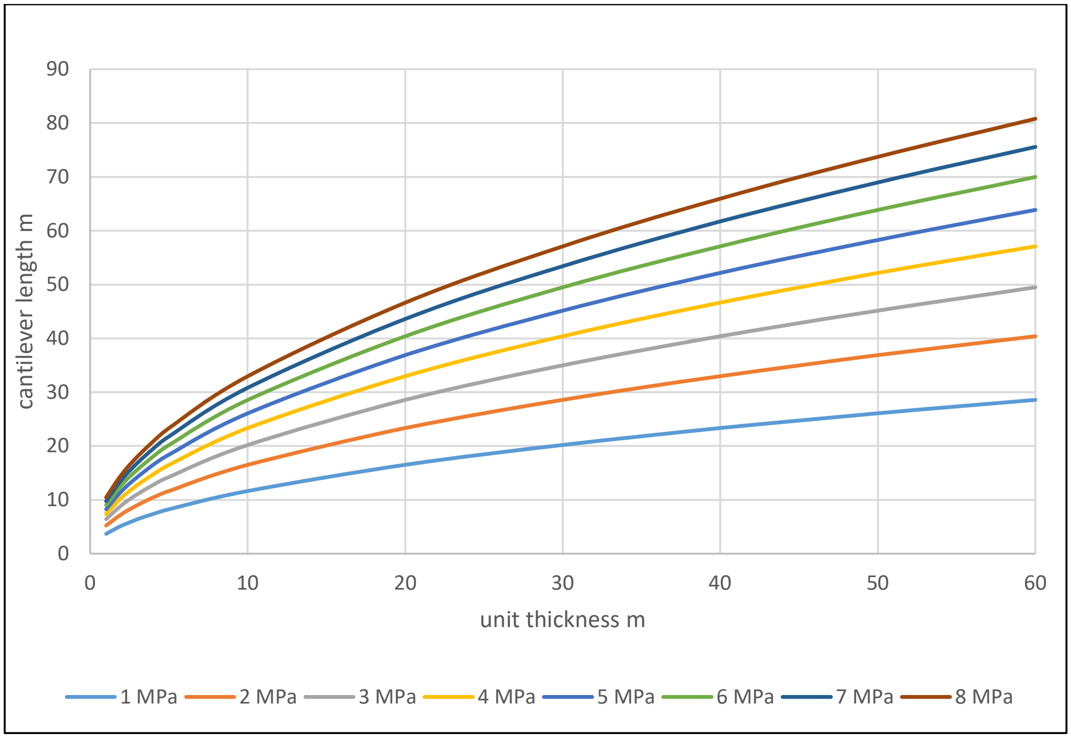


Figure 5: Cantilever length vs thickness of unit. Bending calculation for differing tensile strengths

The combination of block width and thickness enables a calculation of weight per unit width of such a block. These can be very substantial as shown in Figure 6 from which it can be seen that the mass of blocks can become very high.

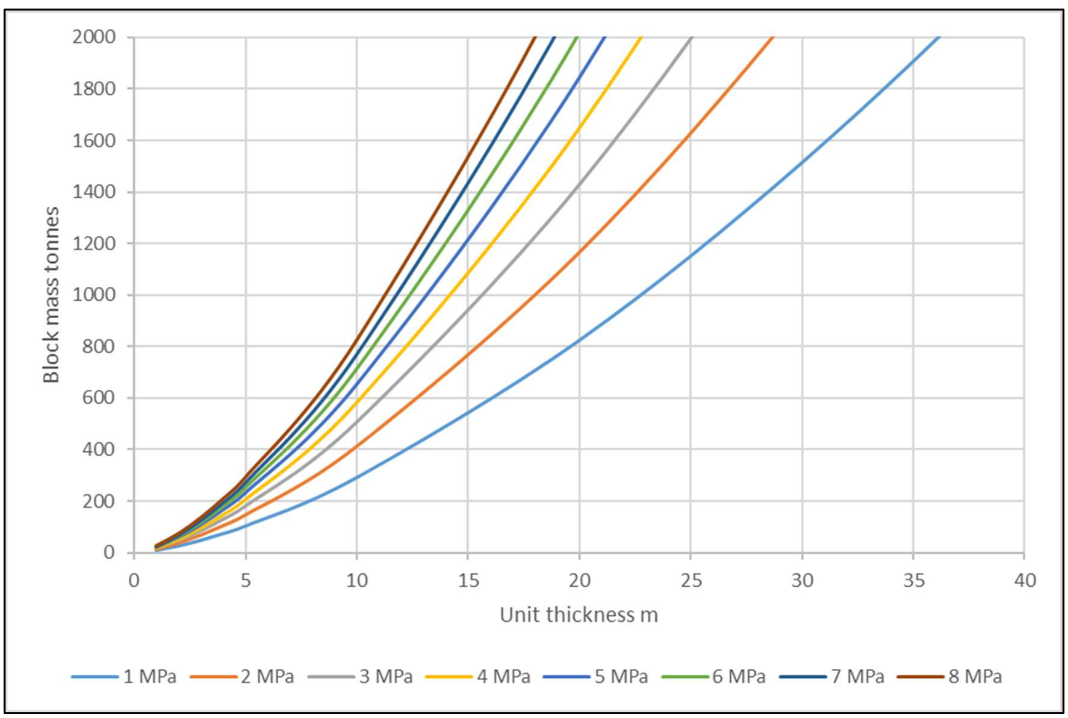


Figure 6: Mass of block per unit width vs tensile strength in bending for goaf cantilevers

The general failure behaviour of a longwall where the goaf is formed behind and to the side is considered in terms of that of cantilevers of rock that protrude into the goaf over the powered supports and then break off causing weightings on the powered supports. Figure 7 is a schematic diagram of the strata directly behind a longwall face and the stresses that act within them. Here the immediate roof is weak and falls directly over the shields. Above it are layers of stronger strata that may behave in a massive manner.

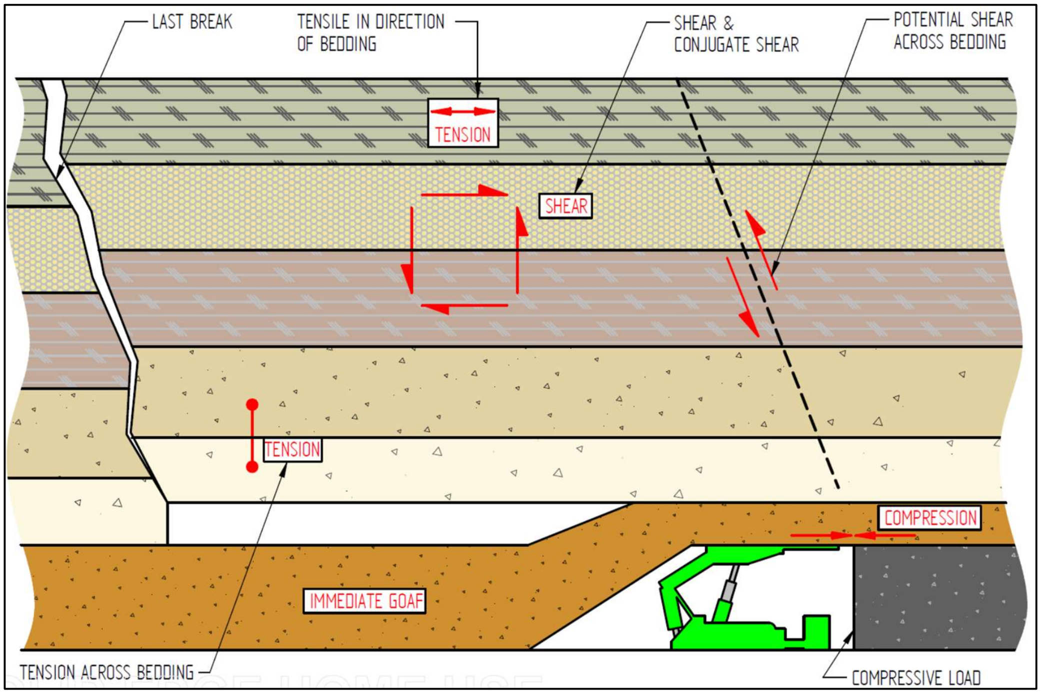


Figure 7. Section through a zone adjacent to a longwall face showing important stresses

In Figure 7 the longwall is advancing to the right and the immediate weak roof is shown collapsing directly behind the powered support with some measure of bulking. Above this the layered rock mass is shown remaining intact in the form of a cantilever until the last break through the strata shown to the left. In this case the last break is shown with the form of a shear across the bedding planes. Another such potential shear is shown directly above the face.

Failure may not, however, be of this form. Shear may also occur along the bedding planes. While it is fairly easily understood that the cantilevered mass has a shear stress acting to support it, it is less obvious that a conjugate shear stress exists that also acts and is perpendicular to it. Shear and conjugate shear are shown acting on an element of rock in Figure 7. The conjugate shear has to exist or any hypothetical element of the rock mass would not be in rotational equilibrium. The consequence of this is that the conjugate shear stress to the supporting shear stress acts in the plane of bedding. As bedding planes frequently have shear strengths which are lower than that existing across them failure often preferentially occurs along these.

To the left of the unbroken goaf in Figure 7 is a label showing tensile stress acting across a bedding plane. This tensile stress may lead to delamination. Thus, the tensile strength of the bedding plane in resisting such delamination is important.

Near the top of Figure 7 and the cantilever section of unbroken goaf tensile stress, tension is shown which is parallel to the bedding. This is generated as a consequence of the bending imposed on the cantilever by its self-weight and any additional loading. Thus, the tensile strength of the strata parallel to the bedding is important. In some sedimentary rocks the tensile strength of the rock in this direction is considerably higher than that perpendicular to the bedding. Examples of this may be brought about by the presence of tuffaceous claystone, mudstone or carbonaceous banding. It may also be caused by mica which lies parallel to the bedding plane.

To the right of Figure 7, directly above the face the cantilevered rock mass is likely to be in compression. Here the compressive strength of the roof rock parallel to the bedding is important. While compressive failure is in the form of shear, the loading at this point is without confinement perpendicular to bedding. It may be thought to be analogous to a uniaxial compressive loading in the direction of the bedding planes, albeit with some lateral confinement parallel to the face.

The coal face is also loaded similarly with vertical and transverse stress but without restraint perpendicular to the face.

The case shown in Figure 7 is that of blocks that may break freely without the influence of horizontal stress from the goaf. This is the case close to the extracted coal seam. Above these the blocks may break but not disengage, with the result that horizontal stress is to some degree maintained in the higher levels of the goaf.

In most cases the extent of the cantilever beyond the face or the powered support is determined by the tensile strength of the rock at the top of the unit in consideration.

In not all cases does the goaf break off as a neat cantilever; it can also break in a series of triangular slabs which impose enormous local loadings.

Dou (2019) presented the observed form of goaf breakage and formation. These are reproduced in Figure 8 and Figure 9. These figures show what Dou refers to as an O-X spatial structure of blocks in the goaf with extending (shear) fractures which form what he refers to as the F structure.

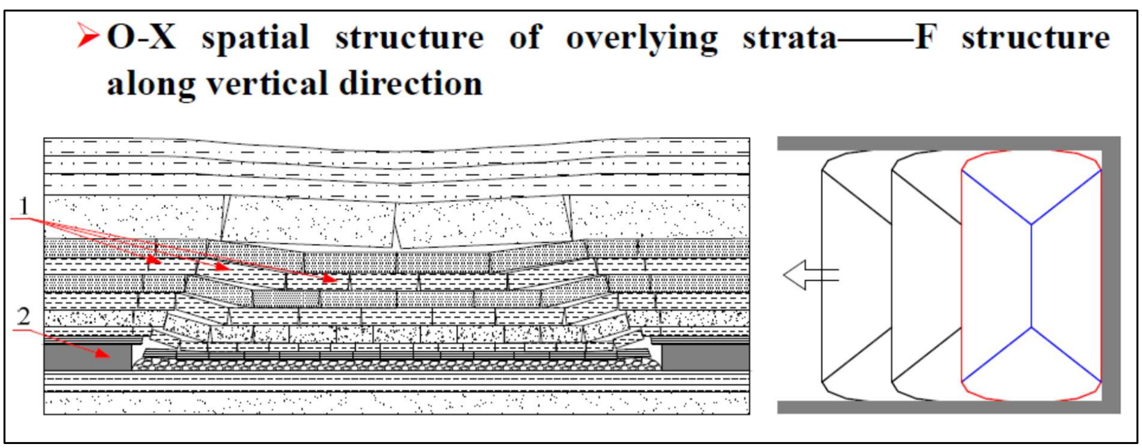


Figure 8. Section of goaf formation and breakage, from Dou (2019)

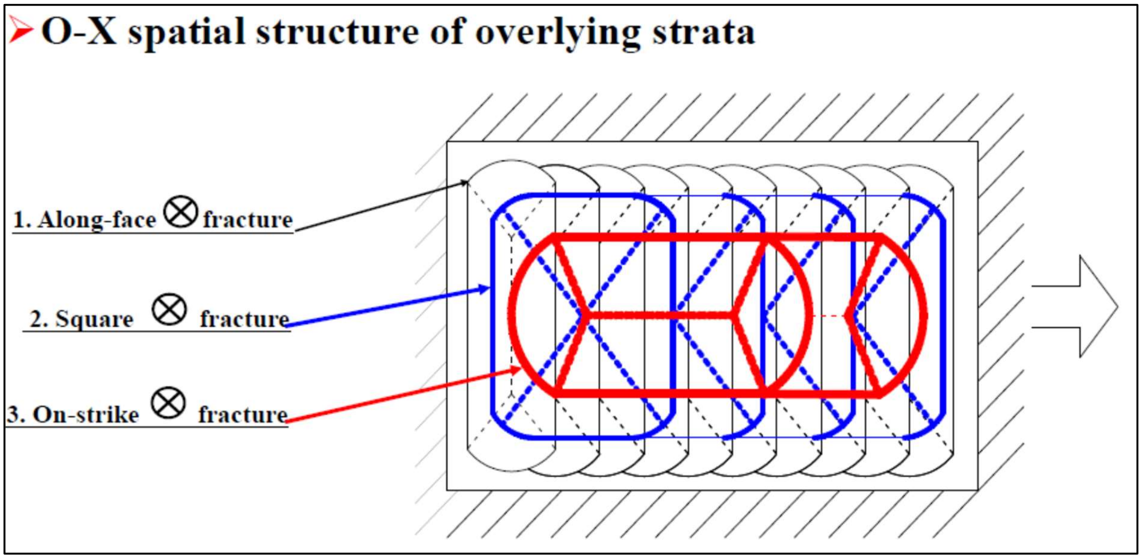


Figure 9. Plan of goaf formation and breakage, from Dou (2019)

The form of the O-X structure of Dou has a remarkable similarity to the forms of failure arrived at by the process of yield line analysis of plate failure used in structural analysis. In this the plate is assumed to yield along particular folding lines which enable it to deform. The analysis is based on plastic yielding and the work done by the load on the plate. While such analysis is not suitable for rock which will not behave in the same ductile manner the shapes that may be generated are similar.

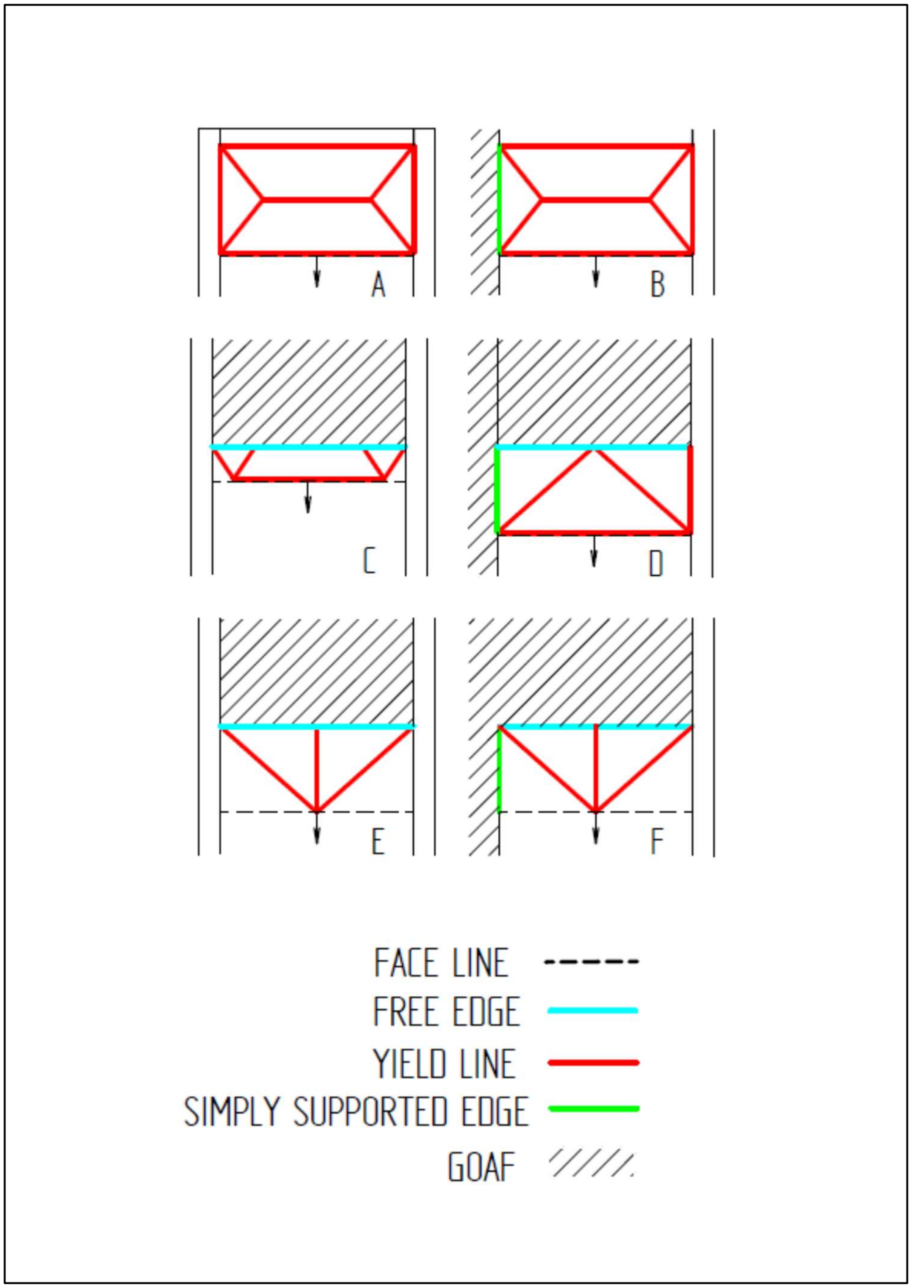


Figure 10. Possible yield line locations applied to a longwall goaf formation.

Not all possible permutations of plate failure (yield) line formation are shown. Cases A and B in Figure 10 show initial goaf formation at the start of the first and subsequent panels. Apart from the nature of support on the goaf side of the right hand panel the shape of deformation is much the same and is very similar to the 'O-X' structure of Dou (2019) shown in Figure 8. Case C of Figure 10. shows the cantilever failure of the goaf along the face. This corresponds to the 'Along Face Fracture' of Dou shown in Figure 9. In Case C what is shown as two close yield lines at each side of the cantilevered plate would in reality be likely to be a combined shear and bending failure that would merge in a zone of broken rock. Case D shows the case of four yield lines and a simply supported tailgate edge that would lead to complete goaf collapse behind the face. Cases E and F show another yield line form which would lead to partial collapse leaving diagonals of goaf roof in the corners.

The bedding plane shear shown by Dou in his F structure is a normal feature of longwall behaviour. If we examine Figure 11 the topmost drawing shows the removal of the coal seam but without any collapse and goaf formation. The relief of stress at the mining boundary may lead to the seam shearing at the seam – roof and the seam – floor boundaries. The middle drawing shows goaf collapse that has occurred in the immediate roof strata. This is accompanied by possible shear of this strata between it and the coal seam and between it and the un-failed rock layer above it. The lower drawing shows an alternative mode of failure where the seam and the first rock layer above it move together to create shear between the seam and the seam floor and between the first stratigraphic layer and the one above it.

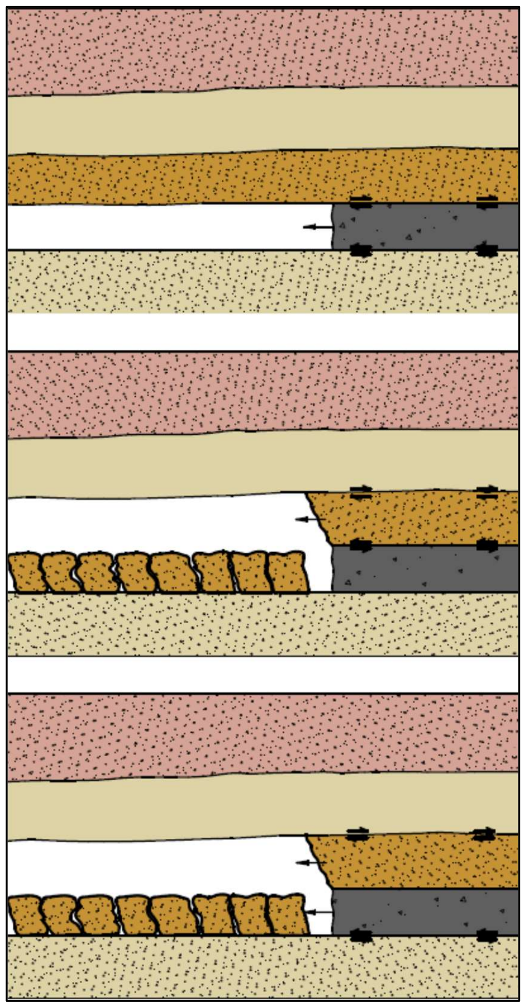


Figure 11. Shear at the goaf edge.

The numerical modelling of these initial shearing cases is relatively straightforward but becomes rapidly more complex when all the combinations of shear behaviour are taken into account with the upwards progression of the goaf. It can be shown readily that the shallower the longwall and the higher the lateral stress then the higher the propensity to shear. Bedding plane shears can be shown to progress several hundred metres laterally into unmined strata. This behaviour is more pronounced when the layer that shears is thick. The reason for this is that the relieved force is the product of the lateral stress and thickness while the restraining force is the shear stress at the top and bottom bedding planes of this potentially moving rock plate. This resistance to shear is dependent on the properties of the bedding planes, the load above them and not on the thickness of the moving block.

POWERED SUPPPORT – ROCK INTERACTION

The breaking of blocks in the goaf will impose dynamic loadings on the powered supports. If the blocks break free and fall suddenly then this loading can be extreme. One of the most severe cases of a failure that destroyed much of a longwall face comes from the Churcha West (Gupta and Ghose, 1992) where mining was being conducted on a 150 m face at 223 m depth. Powered supports 46 to 88 were destroyed in a few seconds after a broad failure on the face. In cases where the failure is more localised the friction between adjacent blocks reduces the dynamic behaviour.

The basic concept behind any support system is that equilibrium of force needs to be reached between the support and the load. Without this the support will continue to collapse. This may be a steady yielding or it may be a sudden accelerating one if the force imbalance is maintained. The problem is that longwall shields cannot continue to deform because at some point they become iron bound. This will then require a difficult and dangerous recovery operation.

It is normally impossible to fabricate a shield that will support all the load that it may be subject to without deformation. The art of design is to make the shield yield enough that the loading reduces to a point where equilibrium between the support that the shield can provide and the load that is imposed upon it is reached. This state of equilibrium may change as loading increases. This then necessitates some more yielding to reach equilibrium. The shields will have a deformation limit at which they become iron bound. To avoid this occurring the face must be advanced at a suitable rate so that new shield heights are re-established. This describes the situation that exists in rock masses where the stress to rock strength ratio is high.

The situation with geotechnically massive strata is that the stress to rock strength ratio is low. This means that the rock fails in bigger blocks. These bigger blocks impose a quite different loading regime on the shields and the face. Instead of crumbling into small fragments which produce a more uniform loading they break suddenly and impose extreme loads associated with the dynamic nature of the failure and heavy loads associated with the geometry of the rock block thereafter.

In the case where a cantilevered block breaks off at the face and falls, the block will be in some manner supported at the face by the shields. The block will begin to rotate and in doing so it will pivot around the face end which is being driven into the roof strata. If the shield cannot support the load of the block it will compress and the block will accelerate downwards until the goaf edge of the block hits the floor. During this short period the shields will compress suddenly. When the goaf edge of the block hits the floor it will stop moving and either the block will break or the movement will be switched from one where the rotation is about the face end to one where the movement is about the goaf end of the block. In either case the block or block fragments have momentum.

For the longwall to operate satisfactorily the shields must be able to arrest the movement of the block. This means they will have to absorb the shock and decelerate the block until it has ceased to move within the operational range of shield movement. This will require that they have adequate energy absorbing capability ($force \times distance$) to slow the moving block within their operating range.

In the upper part of Figure 12 a large block of the roof strata has collapsed leaving an unfailed block cantilevering out beyond the shields. This block has probably sheared on its upper bedding plane. In the lower part of Figure 12 this cantilevering block as broken off compressing the shields severely. It is worth noting that when the block has detached and compressed the shield the shield moves backward from the face exposing more roof with the potential for blocks to fall into the face area of the longwall.

Several other scenarios are possible. One is that the break occurs just in front of the face. In this case the face is subject to the rotation of the block about its fulcrum just inside and above the face. This leads to sudden face compression and potential coal bursting. Another scenario is that after the block has fallen the next one above it may detach and fail so that it falls. This will impose yet another hammer blow to the shields.

These behaviours are why powered supports must be designed to suit the conditions by having adequate capacity and being operated with set pressures, yield pressures and pressure relief systems.

The set pressure is the hydraulic support pressure that the shield is set to after a move. There is good reason to set this to a low level in massive strata. The reason for this is to minimise the support loading so that failure of the major blocks occurs as quickly as the rock will allow.

The yield pressure is that at which the powered support is designed to converge. This yield pressure is reached once movement has occurred. Keeping the yield pressure high means that the acceleration of the broken block is minimised or halted altogether.

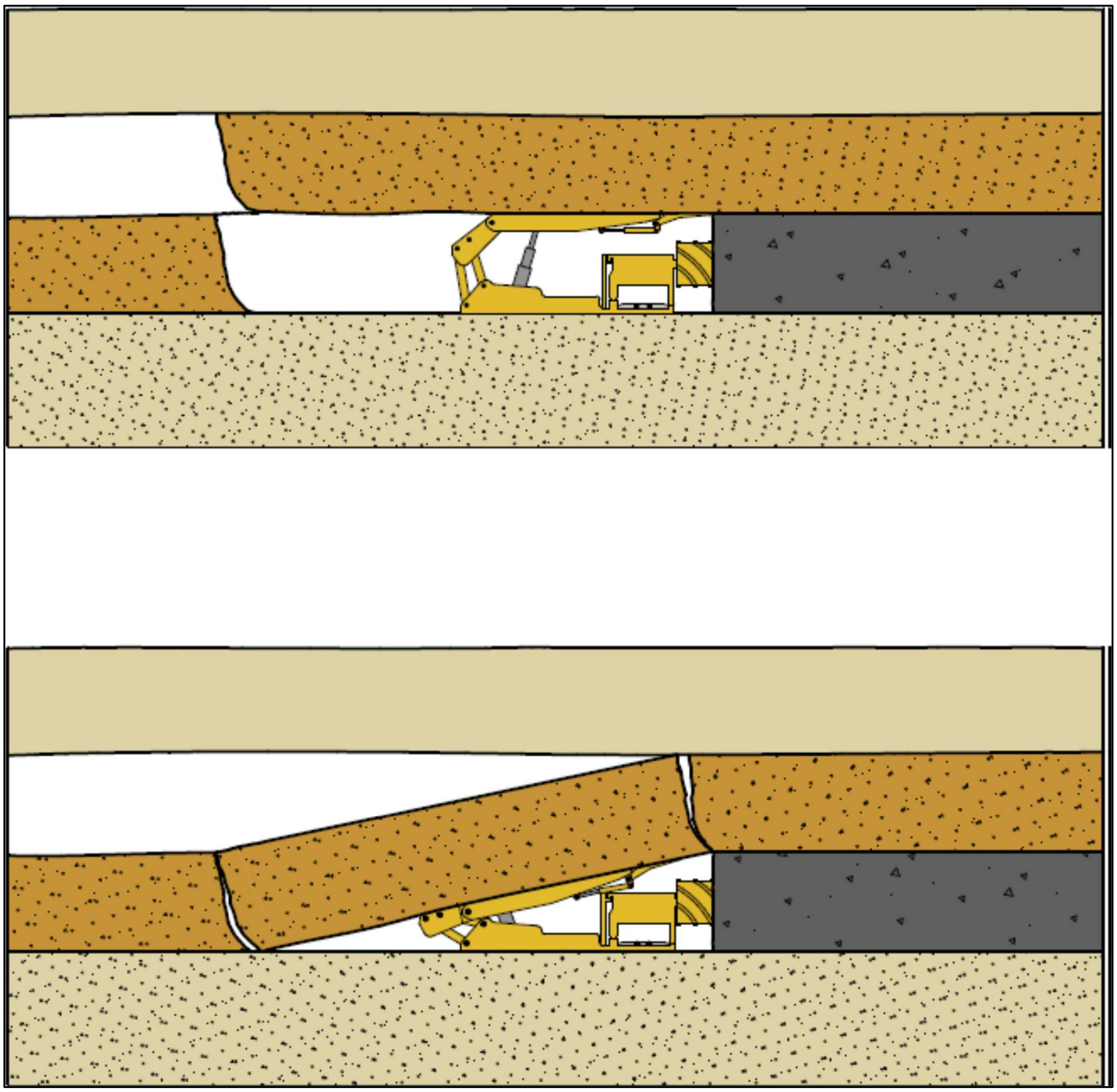


Figure 12. Fall of large block and effect on the shields

In the event of a sudden fall of a rock block the yield pressure control system which dumps hydraulic fluid back to tank is not generally built with adequate flow capacity to prevent pressures rising to the extent that damage to the hydraulic legs of the shields or the shield structures will be avoided. The normal way to deal with these extreme dynamic effects is to incorporate very high flow pressure relief valves that dump hydraulic fluid into the face area. These are designed to stop the hydraulic cylinders blowing apart but may allow an iron bound situation to be reached. If the rest of the powered support survives the impact at least the cylinders then have the potential to provide support again in recovery.

PRECONDITIONING

Preconditioning is the term used to describe the deliberate process of weakening rock so that failure on mining will occur. It involves either breaking the rock, so as to destroy the cohesive and tensile components of the rock strength, or by creating stress concentrations which will lead to failure. While the term 'pre' implies something that must be done before mining the situation may also require measures to be taken during mining. These are 'just in time' measures for which there is little option.

Preconditioning may be divided into hydraulic fracturing, blasting, high energy gas fracturing, the drilling of holes to create stress concentrations, and the use of static fluid pressure in holes to extend fractures.

Generally, the first preconditioning option considered is hydraulic fracturing but it may not be suitable in many cases. The limitation on hydraulic fracturing is that in a uniform rock the fractures will propagate in the direction perpendicular to the least stress. Where the rock is weaker to tension in one direction than another it is possible that the fracture may be captured by this plane of weakness. Bedding planes may provide such planes of weakness. The most favourable hydrofracture orientation for preconditioning operations is in the direction of bedding. If hydrofractures propagate vertically and in the direction of the longwall it is of no benefit in assisting goaf collapse. Hydrofractures may also be captured by a lower stressed horizon. Sometimes the only way to make hydrofracture work is to conduct it just ahead of the face where the stresses are favourable, or when all else has failed, by drilling between the powered supports and fracturing above.

There are simpler and lower cost practices that may do the job. One is the practice of drilling regular holes into the tensile zone of a stratum to induce stress concentrations. Another is to keep water pressure in holes so that it assists fracture propagation once it has commenced. Such water may however leak off where bedding plane shearing has occurred. Blasting is an option used in some countries, but limited by suitable explosives that will not lead to an ignition. Another variant on blasting is the use of high energy gas fracturing. This essentially involves filling a hole with rocket propellant and igniting it so as to create large volumes of gas to fracture the rock mass. Such a technique could only be used well in advance of any mining, and probably from a directional hole drilled from surface.

CONCLUSIONS

The premise of the paper is that massive strata can be treated as layers of plates. These plates separate through tensile vertical loading as the coal is mined beneath them or by bedding plane shear as the end loading is removed by mining or goaf formation. The tensile strength across the bedding is important in the former case and the shear strength along bedding is important in the latter. These need to be measured properly, something that is not normally conducted as part of mine exploration and design. The deformation and stresses within these plates may be calculated using simple elastic plate and beam equations combined with the pre-existing stresses which can be measured.

Failure of these plates most typically occurs when the tensile stress at the top of the plate near the face or goaf edge exceeds the tensile strength of the rock. It may also occur by compressive failure at the bottom of the plate or by shear of the plate. Each of these cases must be considered. In the case of the initial falls of the first longwall panel all stresses exist and may cause the goaf roof to hang up for a dangerous span possibly causing windblasts which may be associated with the expulsion of gas, dust and an ignition.

In the case of established longwall panels the lateral stresses are greatly reduced and the falls are primarily controlled by the tensile strength of the rock in the direction of bedding. As the panel is developed and a goaf is formed the pre-existing stresses have little effect on the collapse of the goaf and the behaviour would approach that described by a cantilever under self-weight. The widths at which the cantilevers of rock break off corresponds to that of periodic weighting that will occur.

In some cases, particularly in narrower longwalls, goaf edge cantilevers may not form, rather the failures of substantial plates of rock is expected to occur and follows to some degree the form of those that would be predicted by the yield line approach used to determine plate behaviour in structural engineering, thought that analysis approach is not considered to be suitable to determine when failure will occur in rock.

The dynamic capability of supports is important as in massive strata they need to have the capacity to absorb sudden loading and shocks.

The use of preconditioning is essential in cases where the blocks that would be created are too large.

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While the report only names two authors many others have contributed to its production. These are notably Jeff Wood, Sigra's Principal Geologist, Lucy Liu who measured rock properties in Sigra's laboratory and Thanh Nguyen who contributed to the analysis of plate behaviour. Ian Ginn is also thanked for producing the drawings.

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ADDENDUM

In this addendum the four cases of plate boundary fixity are considered. β is the factor for outer fibre bending stress shown in Equation **1** while γ is the factor for the mid plate thickness maximum shear shown in Equation **2**. The second letter corresponds to the position on the plate. In the case of bending the third character corresponds to the direction of bending being considered, either in the y direction corresponding to the longwall direction or in the x direction corresponding to the longwall face. The two end letters in the case of shear correspond to the direction of shear with z being perpendicular to the plate. In all cases a Poisson's ratio of 0.2 has been used.

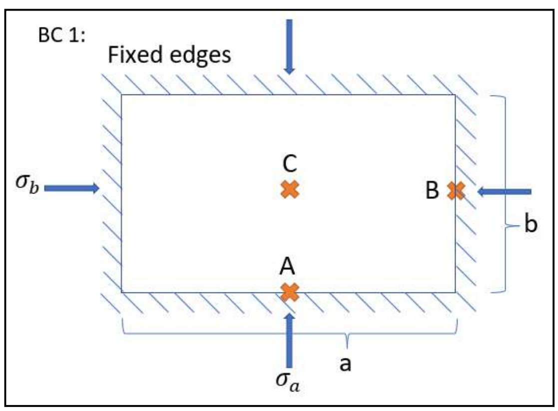

Figure 13. Case 1 with four clamped edges.

Ratio b/a	βAx	βAy	γAyz	βBx	βBy	γBzx	βCx	βCy
0.00	0.1001	0.5000	0.9700	0.3409	0.0682	0.7000	0.0501	0.2506
0.10	0.0999	0.4994	1.0999	0.3409	0.0682	0.8701	0.0501	0.2506
0.20	0.1001	0.5000	1.1999	0.3409	0.0682	1.0822	0.0501	0.2509
0.30	0.1002	0.5011	1.3027	0.3408	0.0682	1.1918	0.0501	0.2513
0.40	0.0998	0.4990	1.4252	0.3411	0.0682	1.2909	0.0590	0.2492
0.50	0.0994	0.4969	1.5478	0.3414	0.0683	1.3879	0.0709	0.2452
0.60	0.0938	0.4756	1.6601	0.3430	0.0684	1.4839	0.0880	0.2259
0.70	0.0881	0.4403	1.7345	0.3409	0.0682	1.5792	0.1068	0.2069

0.80	0.0797	0.3985	1.7749	0.3355	0.0671	1.6598	0.1190	0.1800
0.90	0.0711	0.3498	1.7824	0.3246	0.0652	1.6968	0.1250	0.1571
1.00	0.0616	0.3079	1.7653	0.3079	0.0616	1.7653	0.1270	0.1270

Table 1. The values of γ and β for Case 1.

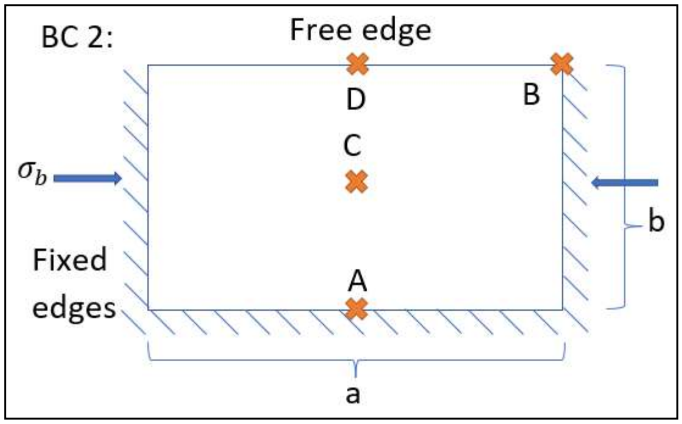

Figure 14. Case 2 – three clamped and one free edge.

Ratio b/a	βAx	βAy	γAyz	βBx	βBy	γBzx	βCx	βCy	βDx
0.00	0.6100	3.2000	1.8500	2.2000	0.0207	2.8000	0.2800	1.0100	-0.3800
0.10	0.5989	2.9945	2.1021	2.1993	0.0207	1.5805	0.1480	0.7455	0.0042
0.20	0.5550	2.7100	2.2631	2.1725	0.0199	0.7024	0.0133	0.5058	0.2761
0.30	0.4617	2.3083	2.3864	2.0371	0.0191	0.8581	-0.1129	0.2689	0.4639
0.40	0.3504	1.7518	2.3026	1.7449	0.0164	2.5712	-0.2160	0.0634	0.5750
0.50	0.2471	1.2355	2.1160	1.3799	0.0137	3.8963	-0.2817	-0.0838	0.6112
0.60	0.1752	0.8756	1.9170	1.0937	0.0111	4.6656	-0.2913	-0.1412	0.5611
0.70	0.1350	0.6749	1.7393	0.8187	0.0085	5.0163	-0.2593	-0.1284	0.4579
0.80	0.1048	0.5240	1.5969	0.5115	0.0099	5.1115	-0.2331	-0.1170	0.3788
0.90	0.0966	0.4050	1.4770	0.3655	0.0078	5.0150	-0.2086	-0.1012	0.2900
1.00	0.0677	0.3386	1.3815	0.3399	0.0056	4.8610	-0.1840	-0.0853	0.2589

Table 2. The values of γ and β for Case 2.

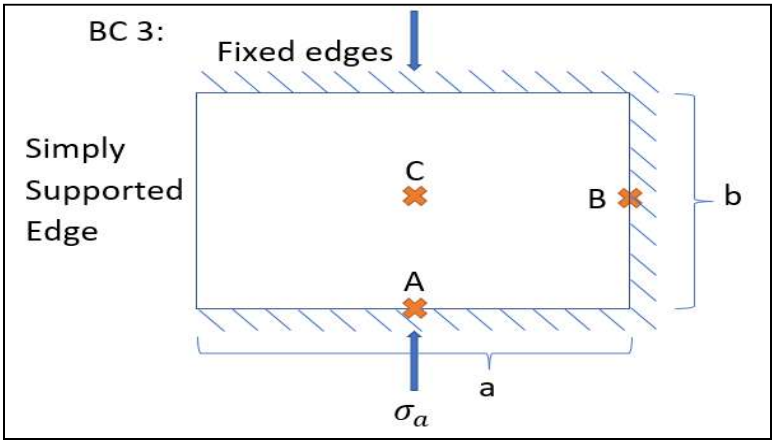

Figure 15. Case 3 – three clamped and one simply supported edge.

Ratio a/b	βAx	βAy	γAyz	βBx	βBy	γBzx	βCx	βCy
0.00	0.1000	0.5000	**0.9700**	0.3409	0.0682	**0.7000**	0.0501	0.2506
0.10	0.0999	0.4994	1.0999	0.3409	0.0682	0.8701	0.0501	0.2506
0.20	0.1000	0.5001	1.1999	0.3415	0.0682	1.0822	0.0481	0.2508
0.30	0.1001	0.5007	1.3027	0.3408	0.0682	1.1918	0.0499	0.2511
0.40	0.1002	0.5009	1.4178	0.3435	0.0682	1.2909	0.0556	0.2498
0.50	0.1002	0.5011	1.5478	0.3412	0.0682	1.3879	0.0655	0.2485
0.60	0.0968	0.4880	1.6601	0.3427	0.0684	1.4839	0.0804	0.2380
0.70	0.0934	0.4672	1.7345	0.3428	0.0686	1.5792	0.0979	0.2218
0.80	0.0867	0.4333	1.7749	0.3421	0.0684	1.6598	0.1117	0.2011
0.90	0.0806	0.4012	1.7824	0.3359	0.0672	1.6968	0.1243	0.1817
1.00	0.0720	0.3599	1.7653	0.3301	0.0660	1.7653	0.1281	0.1565

Table 3. The values of γ and β for Case 3.

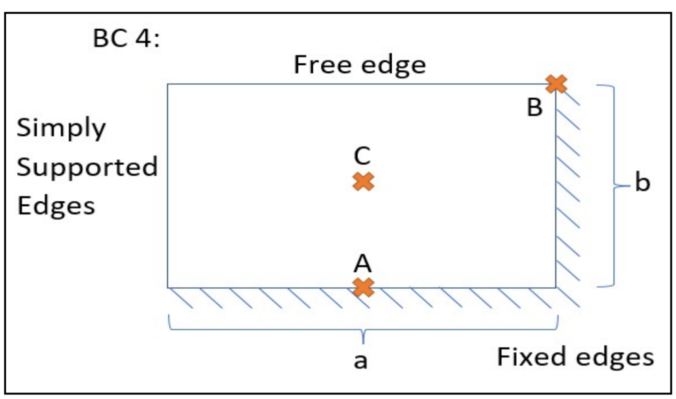

Figure 16. Case 4 – two clamped, one free and one simply supported edges.

Ratio a/b	βAx	βAy	γAyz	βBx	βBy	γBzx	βCx	βCy	βDx

0.00	0.6100	3.0500	**1.8500**	2.2000	0.0207	**2.8000**	0.2500	0.9500	-0.2500
0.10	0.5994	2.9970	2.1021	2.2005	0.0207	1.5805	0.1484	0.7468	0.0033
0.20	0.5695	2.8477	2.2631	2.1803	0.0201	0.7047	0.0486	0.5548	0.2127
0.30	0.4931	2.4656	2.4191	2.0910	0.0195	0.8581	-0.0717	0.3416	0.4051
0.40	0.3961	1.9803	2.4561	1.8945	0.0176	2.7247	-0.1859	0.1322	0.5559
0.50	0.3008	1.5041	2.3665	1.6104	0.0157	4.4346	-0.2708	-0.0372	0.6420
0.60	0.2287	1.1437	2.2265	1.3416	0.0146	5.7280	-0.2900	-0.1152	0.6200
0.70	0.1814	0.9071	2.0723	1.0188	0.0135	6.5377	-0.2905	-0.1311	0.5582
0.80	0.1410	0.7052	1.9252	0.7042	0.0120	6.9770	-0.2738	-0.1321	0.4869
0.90	0.1226	0.6129	1.7965	0.5380	0.0101	7.1446	-0.2500	-0.1206	0.4000
1.00	0.0927	0.4633	1.6854	0.4896	0.0082	7.1316	-0.2300	-0.1091	0.3581

Table 4 The values of γ and β for Case 4.