KEYNOTE ADDRESS
THE PROBLEMS OF STRONG ROOF BEDS AND WATER BEARING STRATA IN THE CONTROL OF LONGWALL FACES

By
Arthur H. Wilson

ABSTRACT

The difference between a hypothetical and a theoretical approach is considered and the advantages in using a hypothesis is discussed.

Virgin horizontal stresses play an important role in stabilizing strata which bridge across an excavation, and reduces the tensile stresses which develop in a beam subjected to bending. Simple formulae are deduced for the bridging of a thick massive stratum under such conditions. At shallow depths, tensile failure on the upper side of the beam is the most likely cause of fracture, but below a certain depth this gives place to compressive failure on the underside, with the possible incidence of rock bumps.

The presence of strong massive beds delay the onset of subsidence and may restrict the width of the subsidence trough. If the material involved in the caving is strong, this will reduce the final subsidence, especially at shallow depths.

The British regulations for the control of aquifers and their dependence on the strata developed at the seabed are discussed. As an alternative, the concept of a disturbance factor is introduced to permit the consideration of aquifers at depth, leading to an explanation of the make of water on the first faces to be mined in the Selby Coalfield.

The paper closes with a brief consideration of the different roles played by longwall face supports in the control of readily caving strata, and the control of strong beds which hold up in the waste.

INTRODUCTION

Much has been published in literature concerning the successful working by longwall of the relatively soft Coal Measures of the United Kingdom, but recent problems encountered in the first faces of the Selby Coalfield in Yorkshire, where extraction is taking place below a hard shale roof with a bridging massive limestone bed some distance above, has prompted thought on the working of longwall faces in hard rock conditions.

To carry out rigorous, analytical solutions in varied strata is difficult. Also the spacing of joints, degree of bed slippage, the properties and behaviour of strata remote from the excavations are all unknown factors. In such circumstances, recourse must be made to the formulation of simple hypotheses in order to explain the phenomena observed.

These hypotheses need not be rigid in their application. Initially they can be based on simple theory, but as more experience is gained they can be modified and extended to suit the additional measured data as it accumulates. To quote Terzaghi (1948), "From a practical point of view, the working hypothesis is as useful as the theory ... If the engineer is fully aware of the uncertainties involved in the fundamental assumptions of his computations, he is able to anticipate the nature and the importance of the differences which may exist between reality and his original concept of the situation."

In any situation, a number of alternative hypotheses are possible. Those chosen in the paper which follow the facts as observed in Britain, but others may be equally valid. It is up to the individual to choose the hypothesis which best fits his conditions.

BRIDGING OF STRONG MASSIVE BEDS

For the purpose of this paper, a massive bed is defined as a thick stratum of homogeneous rock, devoid of any weakness along the bedding plane, and whose behaviour can be
expected to conform to beam theory. Such strata must be treated differently to the more pliable beds so often found in the Coal Measures.

In the softer rocks, the strength exhibited underground may only be a fraction of that obtained from small specimens in the laboratory. For example in well cored rock such as coal, the strength in situ is believed to be only one fifth of that obtained from a laboratory sized specimen (Wilson, 1983, p 92). In the case of strong massive rock, however, this will not apply, and the strength of a small specimen may be taken as representative of the strength of the mass as a whole.

The virgin stress condition in soft pliable rock is generally assumed hydrostatic. This will not necessarily apply to strong massive rock. An analysis of available information by Brown and Hook (1978) suggested that the virgin horizontal stress in strong rock can be up to four times the cover load stress. One possible explanation is the retention of some of the horizontal stress present when the rock mass was consolidated at great depth. Subsequent erosion reducing the vertical stress. The ratio of residual horizontal stress to vertical stress cannot exceed the triaxial stress factor $k = (1 + \sin \theta)/(1 - \sin \theta)$, otherwise failure of the bed will have taken place. The horizontal stress plays an important role in the stability of massive beds.

If a massive bed bridging over an excavation can be considered as a simple beam whose ends are rigidly held at the sides by the strata above and below, then the stress component caused by bending along will be a maximum at the ends and equal to

$$
Q_h = \frac{1}{8h} \frac{\ell^2}{I}
$$

where $h$ is the average strata density, $\ell$ the height of the strata being supported, $L$ the span and $I$ the thickness of the massive bed. At the ends of the beam the bending stress will be tensile on the upper side, compressive on the underside. Superimposed on this will be the virgin horizontal stress.

$$
Q_v = \frac{mgL}{L}
$$

where $m$ is the ratio of virgin horizontal stress to vertical stress. Hence the critical stress at the top of the beam, causing compressive strength as positive, will be $Q_b = Q_h + Q_v$.

At the shallower depths, failure will be by tension on the upper side of the beam, and will occur when

$$
Q_v = \frac{1}{2} \left( Q_h + Q_v \right)
$$

where $L$ is the tensile strength of the rock.

If $Q_h$ is small, say because of jointing of the massive bed, then this equation reduces to $L = \frac{1}{8} \sqrt{3m}$, and if $m = 2$ it further reduces to $L = 2L$, i.e. a jointed massive bed will span about twice its thickness, regardless of the depth, subject to failure not occurring first in the compressive mode on the underside.

At deep horizons, failure will be by compression on the underside. The relationship will then be

$$
Q_v = \frac{1}{8h} \frac{\ell^2}{L}
$$

If $H$ represents the depth at which compressive failure replaces tensile failure, then from equations (3) and (4)

$$
\frac{Q_v}{Q_h} = \frac{mgL}{L} - \frac{L}{L}
$$

$$
\frac{Q_v}{Q_h} = \frac{2mgL}{L}
$$

This depth limit is of importance. Above it the relatively quiet tensile failure will occur remote from the excavation. Below it the much more severe compressional failure will occur just above and close to the face line, and may give rise to a severe rock dump. Even if the massive bed is separated from the excavation by weaker beds, the resultant shock waves can cause bursting of otherwise yieldable coal on the face.

The coal extraction below the Parkgate Rock in Yorkshire can be taken as an example. The Parkgate Rock is a massive sandstone of tensile strength about 7 MPa and compressive strength about 70 MPa. Taking $g = 0.025$ MPa/m and accepting a value of 2 for $m$ gives a critical depth $H = 640$ m. It is known from experience that in the area covered by the Parkgate Rock, bumps are only experienced at working depths greater than 2,000 ft (610 m). Even when the Rock and seam are separated by a layer of shale, shock waves can be experienced of sufficient severity to burst the coal on the face, displace supports, and on one occasion to fling the shearer off the conveyor into the waste.

In the Selby Coalfield there is a massive bed of Magnesian Limestone above the area to be worked. It is 65 m thick, of tensile strength about 10 MPa, compressive strength 80 MPa, and over the current workings lies at an average depth of 230 m. At this depth, should failure occur, it will be in the tensile mode, and rock bursts are not expected.

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According to equation (3), this limestone will be capable of bridging a span of 177 m. The initial faces at Selby were 135 m wide separated by stable pillars, and subsidence measurements at the surface confirm that the limestone has not broken.

SUBSIDENCE

Intervening strong beds between seam and surface can influence subsidence in a number of ways. Until the bed fractures, surface lowering is small and is governed by the elastic deflection of the beam. Following fracture, the beds tend to cantilever around the edge of the extraction, and as a consequence the subsidence trough is much narrower than that experienced in the more pliable strata. Local weaknesses in the massive bed may produce uneven cantilevering, giving the trough a lop-sided appearance.

The position of the massive stratum in the sequence also has an effect. By the use of equations (3) above and (9) below, it can be shown that below a depth of about 200 m, the depth of the strong bed has comparatively little effect on the length of the bridge or cantilever. At depths shallower than 200 m, and particularly at depths less than 100 m, the size of the bridge or cantilever increases significantly. This is illustrated in Table 1, where an average massive bed has been considered ($\sigma_c = 70$ MPa, $\sigma_y = 10$ MPa, $m = 2$).

### Table 1
Influence of Depth on Massive Bed Behaviour

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Bridge L/T</th>
<th>Cantilever L/T</th>
<th>Ratio Cant/Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>4.5</td>
<td>1.6</td>
<td>0.4</td>
</tr>
<tr>
<td>100</td>
<td>3.5</td>
<td>1.2</td>
<td>0.3</td>
</tr>
<tr>
<td>200</td>
<td>2.8</td>
<td>0.8</td>
<td>0.3</td>
</tr>
<tr>
<td>300</td>
<td>2.6</td>
<td>0.7</td>
<td>0.3</td>
</tr>
<tr>
<td>400</td>
<td>2.5</td>
<td>0.6</td>
<td>0.2</td>
</tr>
<tr>
<td>500</td>
<td>2.4</td>
<td>0.5</td>
<td>0.2</td>
</tr>
<tr>
<td>600</td>
<td>2.3</td>
<td>0.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The properties of the strata involved in the caving behind the longwall face also has an influence on the degree of subsidence. The compaction of the broken material in the waste can be approximately represented by

$$\varepsilon = \frac{U}{K} (1 - e)$$

Where $\varepsilon$ is the amount of compaction caused by the stress $\sigma$, $U$ is the height of extraction and $K$ is a constant which increases with the strength of the caved material. For a typical British shale, $K = 4.3$ (Wilson, 1983, p 116). The degree of compaction at various depths is given in Table 2, assuming full cover load is restored. This shows that if soft shale is involved in the caving, the compaction at 400 m will be 92% at a depth of 1000 m. However if a caved material is strong, say with a $K$ value of 12.9, the maximum subsidence after extracting a seam at 400 m depth will be only in the order of 50%.

### Table 2
Maximum Subsidence with Different Caved Material

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Degree of compaction at cover load $\varepsilon$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$K = 4.3$</td>
</tr>
<tr>
<td>200</td>
<td>69%</td>
</tr>
<tr>
<td>400</td>
<td>90%</td>
</tr>
<tr>
<td>600</td>
<td>97%</td>
</tr>
<tr>
<td>800</td>
<td>99%</td>
</tr>
<tr>
<td>1000</td>
<td>100%</td>
</tr>
</tbody>
</table>

If a second seam is worked in close proximity to the first, the stress wave caused by the abutment zone ahead of the subsequent face will further compact the initially caved material. This secondary compaction must be limited to less than 10% for weak caved material, but in the case of strong caved material, will produce a substantial increase in the degree of compaction. In addition the rock cantilevers originally formed may fracture, and the subsidence trough may increase considerably in both depth and extent. Hence where beds of strong strata are involved, the compaction experienced in multi-seam extraction will be much greater than the sum of subsidences expected from individual faces.

The variability from site to site of the properties and depths of massive beds and the nature of the caved material makes the prediction of the subsidence profile very difficult. The type of empirical analysis used in Britain and Germany, where the strata are predominately pliable, can no longer be employed. This is a subject which needs much further research.

CONTROL OF AQUIFERS

Another problem which can arise in the laying out of a longwall face is the control of aquifers. Aquifers can be described as permeable strata, filled with water, which if linked to an excavation will produce a flow of water. An influx will endanger lives; an
inflow will interfere with production. In Britain, the working of coal in the
neighbourhood of aquifers is controlled by
two sets of regulations.

The Mines (Precautions Against Inrushes)
Regulations 1979, state that for reasons of
safety, no working shall be closer than 45 m
to any potential source of water, whether it
be sea, water filled strata or merely a
borehole, unless the manager can prove to the
satisfaction of the Inspectorate that no
danger exists. This Regulation is concerned
with the safety of life, and takes no account
of nuisance water which could interfere with
production.

The NCB Mining Department Instruction
P1/1968/8 (Revised 1971) entitled "Working
Under the Sea", in addition to safety of life,
has regard to the prevention of water seeps
or inflows which could interfere with
production. It states that with respect to
longwall workings, the tensile strain at the
seabed as calculated by the NCB Subsidence
Engineers Handbook must not exceed 10 mm/m,
subject to a minimum thickness of cover of
105 m, of which at least 60 m will be
Carboniferous strata.

It is a peculiarity of this latter
instruction, that it relates only to working
below the sea, or to aquifers linked to the
sea. It is an unwise manager, however, who
does not give due regard to this Instruction
merely because any surface water in question
is not tidal. Unfortunately the Instruction
does not specify which issue of the NCB
Subsidence Engineers Handbook should be used
in the calculation of seabed strain. There
are differences, and it has been suggested
that if the latest edition (1975) is used,
the allowable strain at the seabed should be
reduced to 6 mm/m.

The limitation of strain was deduced
from under sea workings off the north-east
coast of England, where the shales and other
strata are fairly brittle. Other types of
strata may require other limits. Also the
presence of soft pliable shales (often
referred to as aquicludes), capable of
preventing the passage of water even when
disturbed by mining, can alter conditions
considerably. In the Leicester area of
England, where particularly soft strata
including aquicludes exist, the working of
longwall faces very close below aquifers is
possible.

It is of interest to analyse the
implications of this strain limit at the
seabed. If the extraction is wide compared
to its depth h (i.e. it is greater than its
critical width), then under British

conditions the subsidence S will be approx-
imately 0.5 times the thickness of extraction
M. The maximum tensile strain E at the
surface is given by the NCB Subsidence
Engineers Handbook as

$$E = 0.65 \frac{d}{h}$$

(6)

This is the surface strain, and may not
necessarily apply at any intervening horizon.
The extraction will, however, cause a
disturbance at horizons above it. Referring
to Figure 1, zone A is the caved material, and
near the rib sides will remain unconsolidated.
In zone B, although the strata are continuous,
they will contain induced fractures and bed
separation. The degree of disturbance will
reduce with distance h above the extraction,
until a zone C is reached where the beds
flex without creating a linked fracture
pattern. It is not unreasonable to assume
that the degree of disturbance E required to
allow the passage of water will be directly
proportional to the height of extraction h
and inversely proportional to the distance h
above the extraction.

$$E = \frac{KM}{h}$$

(7)

where K is a constant. This disturbance will
also apply near the surface, therefore
equation (6) fixes the constant K as 0.6. If
E is regarded not as a strain, but as a
"disturbance factor", then it can be used at
horizons other than the surface. The NCB
Subsidence Engineers Handbook can also be
used to estimate E for extractions which are
less than the critical width. It must be
noted, however, that this Handbook should be
used only where the conditions in the Coal
Measures are similar to those found in Britain,
and the limit of 6 to 10 mm/h applies to the
nare brittle types of strata.

![Figure 1](image)

Degree of Disturbance Above Extraction

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The Carboniferous strata in the Selby Coalfield is similar to that found in the North-East Coalfield of England. It consists in the main of fairly strong, brittle shales and siltstones, and is devoid of aquicludes. It is overlain by the massive bridging limestone discussed earlier, but between the limestone and the Coal Measures there is about 5 m of weekly consolidated sand (see Figure 2). The base of the limestone and the sand form a major aquifer, and the water pressure in it is slightly above the hydraulic head to surface. The 135 m wide extraction will produce a calculated disturbance of 17 mm/m at the aquifer horizon, well in excess of the limit of h to 10 mm/m, and it is not surprising therefore that the water broke through into the workings. This occurred following a major first weight when the face had advanced 110 m.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>200 m mixed strata</td>
</tr>
<tr>
<td>200</td>
<td>65 m massive limestone</td>
</tr>
<tr>
<td>205</td>
<td>5 m weak sand (aquifer)</td>
</tr>
<tr>
<td>260</td>
<td>80 m medium strength carboniferous strata (no aquicludes)</td>
</tr>
<tr>
<td>350</td>
<td>135 m wide extraction</td>
</tr>
<tr>
<td>350</td>
<td>2.4 coal seam</td>
</tr>
</tbody>
</table>

Figure 2: Geological Sequence, Selby

From this value can be calculated the effective thickness of the bridging material within the Coal Measures themselves. On this occasion equation (1) will have to be modified to take into account the water pressure, ie the equation now becomes

$$\sigma_b = \frac{1}{2}(gh + g' h') \frac{L^2}{T^2}$$

where \( g \) is the strata density, \( h \) the distance between the base of the limestone and the neutral axis of the beam in question, \( g' \) the water density, \( h' \) the head of water, \( L \) the span and \( T \) the effective thickness of the bridging beam immediately above the face. The horizontal stress in the beam will remain as

$$\sigma_h = \frac{mgH}{h}$$

where \( m \) is the ratio of virgin horizontal stress to vertical stress and \( H \) the depth of the neutral axis below the surface. At the depth in question failure will be by tension on the upper surface of the beam, therefore

$$\sigma_t = \frac{\sigma}{t} \frac{h}{h}$$

where \( \sigma \) is the tensile limit of the shale, and hence

$$\frac{L^2}{T^2} = \frac{2(\sigma + mgH)}{(gh + g'h')}$$

Taking the values for the shale in question, \( \sigma = 1 \) and \( g' = 5 \) MPa, then by the substitution of these and other relevant values in equation (8), the equivalent thickness of the bridging beam is calculated as

$$T = 44 m$$

Following the first weight, the horizontal constraint will be lost, and the strong strata involved in the bridge will now act as a cantilever and will break periodically when

$$\sigma_t = \frac{3gh \frac{L^2}{T}}{3gh}$$

ie

$$T = L \frac{\sigma}{3gh}$$

For the Selby conditions, \( T = 44 m, \sigma = 5 \) MPa, \( g = 0.025 \) MN/m² and \( h = 80 \), \( 84/2 = 42 m \) hence

$$L = 47 m$$

At Selby, following the first weight, periodic weighting occurred at intervals of about 45 m, giving some substantiation to the use of the above concept.

On the first face at Selby, an initial inflow of about 25000 m³ of water occurred at the time of the first weight, thereafter the flow reduced considerably with occasional
short term increases during periodic weight- 
ings. To reduce the disturbance at the aquifer 
horizon to an acceptable level, short-wall 
mining is now being practiced.

SUPPORT REQUIRED

WEAK ROOF

A method of estimating the degree of 
support required on longwall faces when the 
roof conditions are weak has been well 
documented (Wilson 1975, 1983), and only a 
brief description need be given here. 
Figure 3(a) illustrates the block of roof 
which may fall if the supports are removed. 
By assuming a break along the face line, the 
equilibrium calculation will give the condition 
for zero stress on this plane, thus preventing 
the likelihood of a break developing.

\[ P = \frac{W}{2} \frac{L}{L/2} \]

where \( p \) is the distance of the support thrust 
from the rear of the canopy, \( g \) the strata 
density and \( F \) a factor of safety to account for 
lowering of individual supports when advancing 
and possible deficiencies in the hydraulic 
circuits.

The magnitude of \( P \) is dependant on its 
position relative to \( W \). It will be a minimum 
when it acts in line with \( W \), and will increase 
as \( W \) moves forward or back. If \( W \) is behind \( P \), 
then the reaction \( R \) on the top of the block 
necessary for equilibrium will lie on the 
forward corner of the block. This is illus-
trated in Figure 3(b), where an overhang \( b \) 
beyond the rear of the canopy is assumed to 
occur. By taking moments about the front 
edge of the block, it can be shown that in 
this condition:

\[ P = M(L + b)^2 \frac{GW}{2(L - p)} \]

The magnitude of \( b \) to cause \( P \) to rise 
beyond the value consistent with vertical 
caving immediately behind the supports can be 
found by equating (10) and (11) and solving 
for \( b \), i.e

\[ b = L \sqrt{\frac{L}{p} - 1} - L \]

Substituting values of, say \( L = 4 \) m and \( p = 1 \) m, 
gives the required amount of overhand as

\[ b = 2.93 \, \text{m} \]

Hence calculating \( P \) on the assumption that the 
waist caves vertically at the rear of the 
canopy is the most severe condition until the 
overhang reaches almost 3 m.

In the case of inclined seams, other 
equilibrium conditions apply. In Figure 3(c), 
the lines of action of the support thrust \( P \), 
the weight of the roof block \( W \) and the reaction 
\( R \) must all meet at the same point, and the 
vector diagram must close. Also the angle \( \theta \)
the angle which the reaction makes with the top of the block, must be less than the angle of friction $\phi$, otherwise the block will slip.

It can be shown from the analysis, that in order to prevent the block slipping,

$$P = W(\sin \phi / \tan \phi + \cos \phi)$$

where $d$ is the angle of inclination of the seam. A survey of available literature suggests a minimum value of 0.4 for tan $\phi$. This minimum value of $P$ must be present at all times, and therefore represents the minimum setting load required. In adverse circumstances, say if a roof fall occurs, slippage may be in almost any direction, therefore $d$ in this equation represents the full dip of the seam. The appropriate safety factor $F$ should be applied to account for the lowering of individual supports, and a guaranteed value of setting load is strongly recommended for all inclined workings.

**STRONG ROOF**

As discussed earlier, mining below a massive bed will cause initial bridging, and the failure of this bridge produces the first and most severe of the weightings. Thereafter the massive strata will form cantilevers, which break periodically. Table 1 above shows that where a single massive bed is involved the periodic weighting will occur at distances equal to about one quarter of that required to cause the first weight. Multiple massive beds may alter this, as illustrated in the case at Selby.

It has been assumed that the cantilever breaks in tension rather than shear. This can be easily verified. If a cantilever of length $L$ and thickness $T$ is subjected to a distributed load $w$ per unit length, then the maximum shear stress developed will be

$$\sigma_s = wL/T$$ per unit breadth

The maximum tensile stress developed will be

$$\sigma_t = 3wL^2/T^2$$ per unit breadth

For rock, the typical shear strength limit is about 1.5 times the tensile stress limit. Hence for the cantilever to fail by shear,

$$\sigma_s > 1.5 \sigma_t$$

$$\text{i.e.} \quad wL/T > 1.5 \times 3wL^2/T^2$$

$$\text{i.e.} \quad T/L > 4.5$$

Although this thickness to length ratio is well beyond the limit for application of beam theory, nevertheless it shows that failure will be in the tensile mode.

It is of interest to calculate the thrust required from the supports to cause the cantilever to fracture along the line of the supports rather than at the face line. The bending moment at the support line $P$ will be

$$M_p = \frac{1}{2} ghb^2$$

where $gh$ is the distributed load on the cantilever, $f$ the distance from face line to supports, and $b$ the overhang. For $M_p$ to be greater than $M$, then

$$\frac{1}{2} ghb^2 > \frac{1}{2} gh(f+b)^2 - Pf$$

$$\text{i.e.} \quad P > gh(b+f/2)$$

Taking a typical case where $g = 2.5 \text{ t/m}^3$, $h = 300 \text{ m}$, $f = 4 \text{ m}$ and $b = 10 \text{ m}$, then the required value of $P$ to break the cantilever along the line of supports will be 9000 t/m run. However, once the cantilever has fractured and been relieved of the weight of the overlying more pliable beds, then by use of equation (11), the thrust required to keep the 14 m block in place will be about 500 t/m run of face for a massive bed, say 6 m thick. Little convergence will take place before the cantilever fractures, then a sudden convergence will occur as the fractured block takes up the angle usually adopted by more pliable beds.

Joints and weakness planes in strata above a coal seam usually have the same orientation as the cleat in the coal. Hence when longwalling is practiced under massive strata, the face should be worked on board in order to exploit any weaknesses present in the massive rock.

**CONCLUSION**

An attempt has been made to consider some of the problems which can arise in the workings of longwall faces below strong strata. Although in parts the approach has been somewhat hypothetical, the conclusions drawn do explain some of the phenomena observed in practice.

A great deal of work is still required to be done to improve our knowledge, and will involve the co-operation of many disciplines.

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including geologists, subsidence engineers, experts in rock mechanics, support designers and mine planners. The value of symposia lies in the getting together of these disciplines for discussion and an exchange of views.

The author of this paper retired from the National Coal Board of the UK some two years ago. The views expressed are his own, and do not necessarily reflect the views currently held by the National Coal Board.

REFERENCES


