ROCK BOLT REINFORCEMENT SYSTEMS FOR COAL MINE ROADWAYS

By
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Thesis submitted to the University of Newcastle upon Tyne
for the Degree of Doctor of Philosophy

November 1987

"En hvatki er missagt er i fraeðum þessum, þa er skylt at hafa þat heldr, er sannara reynist."

Ari Thorgilsson
Islendingabok
c. 1130 AD

AFFIRMATION

The work submitted in this thesis is my own, and has not been previously submitted for any other degree. The following publications have been presented based on this research :

Mosley, J.T.B. & Tully, D.M. 1986. Roof bolting in friable strata at Rufford Colliery. *Colliery Guardian*, V.234, p.305-307.

Tully, D.M. 1987. The application of scale model studies in the design of rock bolting systems for mine roadways. *Proc. 28th US Symp. on Rock Mechanics*, University of Arizona, Tucson, p.781-788.

ABSTRACT

The utilisation of rock bolting for the support of British coal mine roadways can improve roadway strata conditions and, by permitting a reduction in the density, cross-section or total elimination of steel standing support, can produce considerable savings in roadway support costs.

This study reviews worldwide experiences in the use of rock bolt reinforcement techniques to enhance the stability of coal mine roadways. Details of methods of geotechnical design data acquisition and assessment are given as well as a critical study of various empirical, analytical and observational methods of tunnel support design. The use of scale model studies is shown to be particularly effective for the design of rock bolt support systems for coal mine roadways.

With reference to numerous case studies, descriptions are given of rock bolt systems available and their suitability to specific mine roadway conditions is discussed. Installation procedures and equipment are also reviewed.

It is the author's intention that this study should be used as the basis for further detailed investigation of specific aspects of rock bolt support systems. A number of recommendations are made as to the fields in which further research should be undertaken.

CONTENTS

LIST OF FIGURES

LIST OT TABLES

CHAPTER 1
INTRODUCTION

1.1 The Techniques And History Of Rock Bolting

The development of rock bolt reinforcement techniques began in the nineteenth century with the use of simple wooden dowels in slate quarries and tin mines. During the 1930s the St Joseph Lead Company successfully applied rock bolting techniques in their mines in the United States.

Following the 1939-1945 war rock bolting became widespread in US coal mining operations which initiated interest in the techniques by British mining engineers. By 1959 over 100 km of underground roadways were supported by rock bolts alone in British collieries (Adcock 1959). These bolts were point anchored by a mechanical device at the top of the hole and tensioned. Several roof failures occurred and rock bolting was condemned as being an unreliable means of support in British coal mines.

During the 1960s and 1970s improvements in the mechanical shell unit and the development of resinous materials as an anchorage medium led to a gradual increase in the use of rock bolting for supplementary support of British coal mine roadways. This was paralleled by the utilisation of rock bolting as the sole means of support in the majority of other underground mining operations throughout the world.

In 1984 considerable interest in the potential of rock bolts as a primary and supplementary means of support in British coal mines was again initiated. The type of bolt most widely used in the British rock bolting trials of the 1980s is the full column anchored bolt; the annulus between the steel bolt and the hole wall is grouted with resin throughout its length. There are a number of other types of rock bolt reinforcement systems available which are suited to specific mining situations; these include truss bolts, yielding bolts, pre-tensioned full column anchored bolts, full column mechanical bolts (e.g. Swellex and Split Set), inorganically grouted bolts and cable bolts.

1.2 Mechanisms Of Rock Bolting

1.2.1 Background

Immediately following the excavation of an underground opening, stress redistribution occurs so that vertical compressive stresses existing within the rock mass are transferred to the strata adjacent to the excavation. Tensile and shear forces develop in the de-stressed area above and below the excavation due to the bending of strata layers as they delaminate. The resultant deformation of the rock mass leads to closure of the opening.

Rock bolt reinforcement is capable of strengthening the rock mass to sustain greater stresses and increase the integrity of the roof strata. Possible mechanisms of rock bolt action in coal measures strata detailed in the following sub-sections have been identified from in situ observations and model studies.

1.2.2 Suspension From Competent Strata

The mechanism of suspension considers the rock bolt as a suspension device transferring the mass of a finite volume of weak rock from a layer of overlying competent strata (Figure 1.1a). The length of rock bolt should be adequate to traverse through weaker rock zones and provide the necessary anchoring length in the stronger strata for the transfer of load.

Point anchored bolts transfer the load at the bottom of the bolt to the competent rock at the anchor zone at the top of the hole. Full column anchored bolts provide a greater shear surface for the transmission of the load from the rock to the bolt and vice versa. If there is differential movement along the bolt, then there is differential suspension. The load at one point could possibly be reacted just above the loading point, or carried in the bolt until a competent layer or horizon is encountered.

1.2.3 Beam Building

Mine roadways, being much longer than they are wide, are often modelled using elementary beam equations. The use of such techniques for the analysis of bolted roofs is well established (Panek 1956a, 1956b, 1956c, 1964; Obert and Duvall 1967).

The basic concept of this reinforcement mechanism is to bind thin layers of rock together so that they behave as one thick layer (Figure 1.1b).

Thus, through beam building, the mechanism provides a means to carry the horizontal shear produced by bending. This can be accomplished by preventing sliding of the rock layers over each other, that is, producing shear resistance by frictional forces on the bedding planes or shear resistance of the bolts themselves.

Tensioned rock bolts (either point or full column anchored) provide some compressive force to increase the frictional resistance of the bedding planes. Full column anchored bolts (tensioned or untensioned) are capable of resisting horizontal slip by shear resistance.

1.2.4 Keying Of Blocks

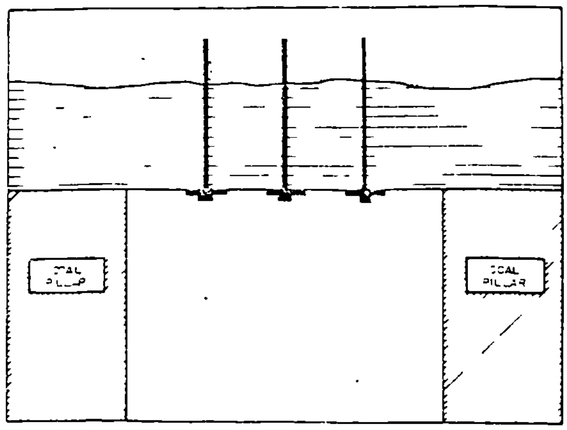
The rock surrounding a coal mine roadway can be intersected by various discontinuity systems with different orientations forming discrete rock blocks around the periphery of the opening. The eventual movement or slip of the blocks along the discontinuities can be prevented or reduced with rock bolts which cross these planes, thus keying the blocks together (Figure 1.1c). The keying action can lead to the formation of competent rock arches across a roadway.

In a similar manner to the beam building mechanism, tensioned bolts rely on the development of frictional interfaces and a locking phenomena, whereas full column anchored bolts resist slip along discontinuities by shear resistance.

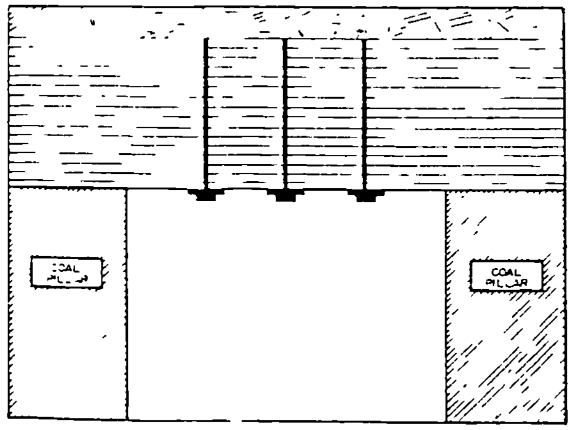
1.3 Maintaining Rock Mass Integrity

Gale (1986) states that "the primary aim of reinforcement design is to enhance confinement and restraint of axial and shear displacements occurring in the rock as it is exposed at the face in order to maintain the structural strength of the roof strata to maximise the self supportive capability to bridge or arch across an opening, and to restrict the height and lateral extent of failure in the roof".

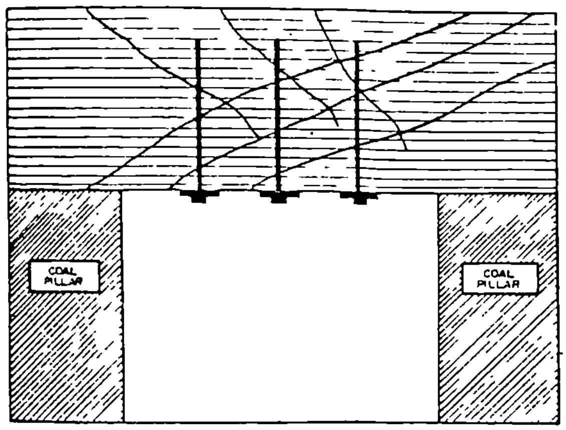
Section 1.2 summarises some of the benefits of fully grouted rock bolts, which are capable of developing both axial and shear restraint. The axial stiffness of fully grouted bolts is nominally 10 to 20 times greater than for mechanical point anchored bolts (McCoy et al 1971; Barnes 1971; Franklin and Woodfield 1971; Pells 1974; Haas et al 1978). Lateral and/or axial movement within the rock mass causes load to be transferred to the bolt via shear stresses in the grout. Additional movement increases the load on the bolt and reduces the rate of movement



a. Suspension from competent strata.



b. Beam building.



c. Keying of blocks

Figure 1.1 Mechanisms of rock bolting.

in the mine rock. This process takes place through transference of developed load to stable rock (Serbousek and Signer 1987).

Radcliffe and Stateham (1980) have observed that a single fully grouted rock bolt can be in both tension and compression at different points along its length.

Under axial load induced by strata separation, a uniform strain distribution will be created either side of a fracture (Figure 1.2a). The length of influence along the bolt being a function of the load magnitude and bolt/rock properties. Creep of resin bolts has been found to increase the load transfer distance with time, particularly in softer rocks and coal (Kwitowski and Wade 1980).

When subjected to transverse shear the bolt will be flexed above and below the shear interface, causing flexural (bending) strain to occur in the bolt core fibres located away from the neutral axis. The form of flexural distribution and the length of influence will be functions of the magnitude of transverse shear activity (the differential lateral displacement) and the bolt/rock properties. The load transfer length under transverse shear loading has been found to be much less than under axial loading (Kwitowski and Wade 1980). Tensile (positive) flexural strain will be at a maximum at a point farthest from the neutral axis and, diametrically opposite, a compressive (negative) flexural strain of equal magnitude will occur. Axial force can develop in a bolt as a secondary effect of transverse shear activity. This axial force might be caused by the bolt core stretching near the shear interface or by shear friction (Farmer 1975). The resulting strain distribution will be the algebraic sum of the flexural and axial strain distributions (Figure 1.2b).

Axial loading and transverse shear occurring simultaneously will produce the strain distribution depicted in Figure 1.2c.

The mechanical interlock between the bolt and the grout, and the grout and the rock are the most important parameters in developing the reinforcing strength of fully grouted roof bolts (Gerdeen et al 1977; Serbousek and Signer 1985). Under loading, mechanical interlock will cause shear forces to be transferred from one media to another until the maximum shear strength is reached. At that point, the weakest material will fail and then friction will control the load transfer.

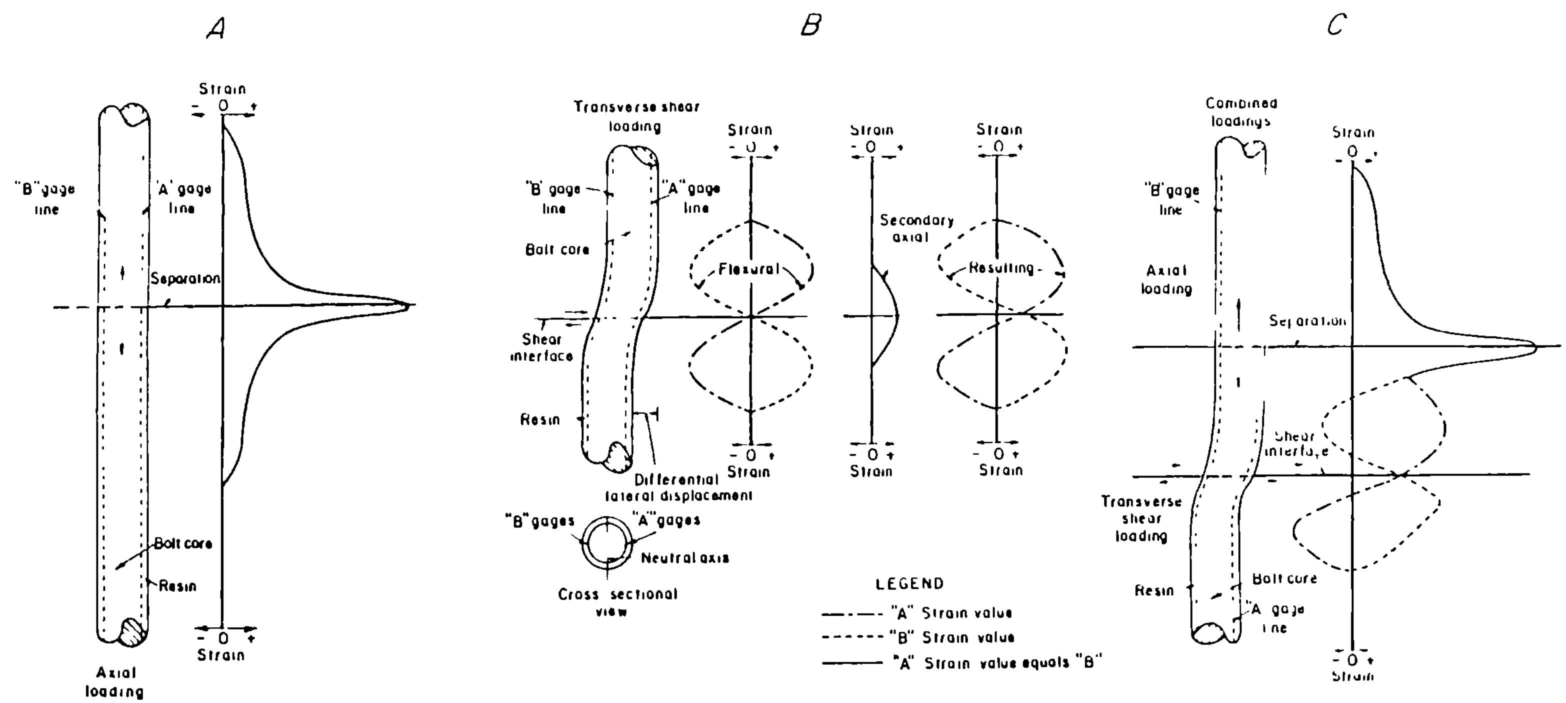


Figure 1.2 Assumed strain distributions for axial and transverse shear loadings (after Kwitowski & Wade 1980).

The mode of failure therefore depends on the characteristics of the reinforcement system and the material properties of the individual elements. Four types of failure are commonly recognised:

(i) failure of the rock mass;

(ii) failure at the grout/rock interface;

(iii) failure at the grout/bolt interface;

(iv) failure of the bolt.

CHAPTER 2

GEOTECHNICAL DESIGN DATA ACQUISITION AND ASSESSMENT

2.1 Background

A primary aim of the initial stages of the design programme of an underground opening is to gain an accurate prediction of strata stability and support requirements. There are no satisfactory quantitative solutions currently available, although approximate solutions can be obtained based on practical experience. The assessment of strata stability and support requirements is an evolutionary process that should begin in the preliminary planning stages and continue with development and mining.

The design of a rock bolting system for a mine roadway must be based on criteria concerning the nature of the strata, the stress field in the rock mass and limitations imposed by the mining method and equipment available. The first stage of the design process is to carry out a geotechnical assessment of the site. This should identify geological structures and other features affecting roadway stability as well as establish parameters for use in empirical and analytical design methods. Information gathered can also be used for the planning and interpretation of a roadway monitoring programme.

Preliminary information may be obtained from a study of available archives. Sources include published and unpublished geological maps, reports, memoirs etc; logs of excavations and boreholes in close proximity to the proposed roadway and reports on projects in similar geological conditions/mining environments. The compilation of a data base of rock bolting sites in British coal mines is currently being undertaken at British Coal HQTD; this should be a valuable source of preliminary information in the future.

Further geotechnical data for engineering design purposes can be acquired from a combination of regional geological and in-mine geotechnical mapping, borehole core logging and laboratory sample testing.

Procedures should be undertaken to detect geological anomalies and potentially hazardous ground conditions prior to roadway development. Mapping of these and other features can provide an effective means of locating areas where strata control design adjustments need to be made. The number and types of maps necessary will vary depending on the nature of the site. Typical maps and sections of a proposed roadway may include:

(i) Isopach maps of the type and thickness of the immediate roof and floor strata.

(ii) Isopach maps of potential parting planes.

(iii) Maps and sections showing the position of adjacent, superjacent and subjacent mining.

(iv) Palaeoenvironmental maps.

(v) Structural maps.

(vi) Maps showing areas of roof falls and excessive roadway closure in adjacent mine workings.

Plotting the data on separate maps of a similar scale will permit superpositioning of these maps to locate areas where a number of potential hazards overlap.

The following sections discuss some of the important features to be identified and evaluated during a geotechnical assessment of a proposed mine roadway.

2.2 Physical Properties Of The Rock Mass

Data concerning the physical properties of rock can be obtained from laboratory testing of samples taken from the site or borehole cores. Commonly determined parameters are mechanical properties such as uniaxial and triaxial compressive strengths, tensile strength, shear strength and internal angle of friction; as well as elastic constants, i.e. Poisson's ratio at a specific strain and the modulus of elasticity. The mechanical properties describe the strength of the rock material, how well the rock will stand and at what level of stress failure will commence. Knowledge of shear and tensile strengths are particularly useful because typical roof failures in a rectangular mine roadway consist of tensile failure at the centre of the opening and shear failures on either side at the intersection of roof line and ribline. The elastic constants describe how the rock material will react to changes in stress in terms of strain and subsequent deformation of the excavation. Details of testing techniques have been described by Szlavin (1971), Davis (1978, 1981), Knight (1979) and ISRM (1981).

Rock strength can be estimated on site with a cone indenter (NCB 1977) or by qualitative judgement using the scheme adopted by Piteau (1970).

Effective rock bolt reinforcement is generally obtainable in rocks with uniaxial compressive strengths greater than 40 MPa and good triaxial characteristics, e.g. a triaxial stress factor (k) greater than 3.5. Weaker rocks, especially those with a uniaxial compressive strength less than 25 MPa and poor triaxial characteristics, e.g. k less than 3.0, may not be able to provide anchorage locations. Due to a larger anchorage length, full column anchored bolts tend to be more effective at supporting weak strata than point anchored bolts. With most mechanical bolts, there is a problem with tension bleed off in soft rock resulting from anchor slippage (Section 7.3.1).

Straps are usually essential with weak roof rock to prevent rock fragments falling from between bolts, thus destroying the reinforced beam effect produced by the bolting system.

Identification of a competent bed above the immediate roof could have a significant influence on the choice of bolt length, so that the mechanism of suspension can be achieved.

2.3 Anchorage Characteristics Of Specific Horizons

Some optimum rock bolt parameters can be determined from pull testing of bolts with a short anchorage length. A typical pull test consists of a force being applied to a bolt via a hydraulic jack with displacement being measured by an extensometer (Figure 2.1). For analysis the load is plotted against displacement. Details of pull test procedures are given by ISRM (1981, 1985).

Franklin and Woodfield (1971) determined design data for a particular resin grout. Figure 2.2 shows the bond length required to achieve a certain pull strength in five different rock types. It is evident that weaker rocks require a greater bond length to achieve the same overall strength. Pull testing by Dunham (1973) concluded that resin bonded lengths of 25 or more diameters appear to be long enough to develop sufficient load to break steel rebar rock bolts (for moderately strong rock).

Pull testing of short grout encapsulation lengths (significantly less than 25 times the bolt diameter) will give data on the bonding capability of specific target horizons in the rock mass. The information can then be used for determination of optimum bolt lengths. Similar tests may be performed for mechanical point anchored bolts or

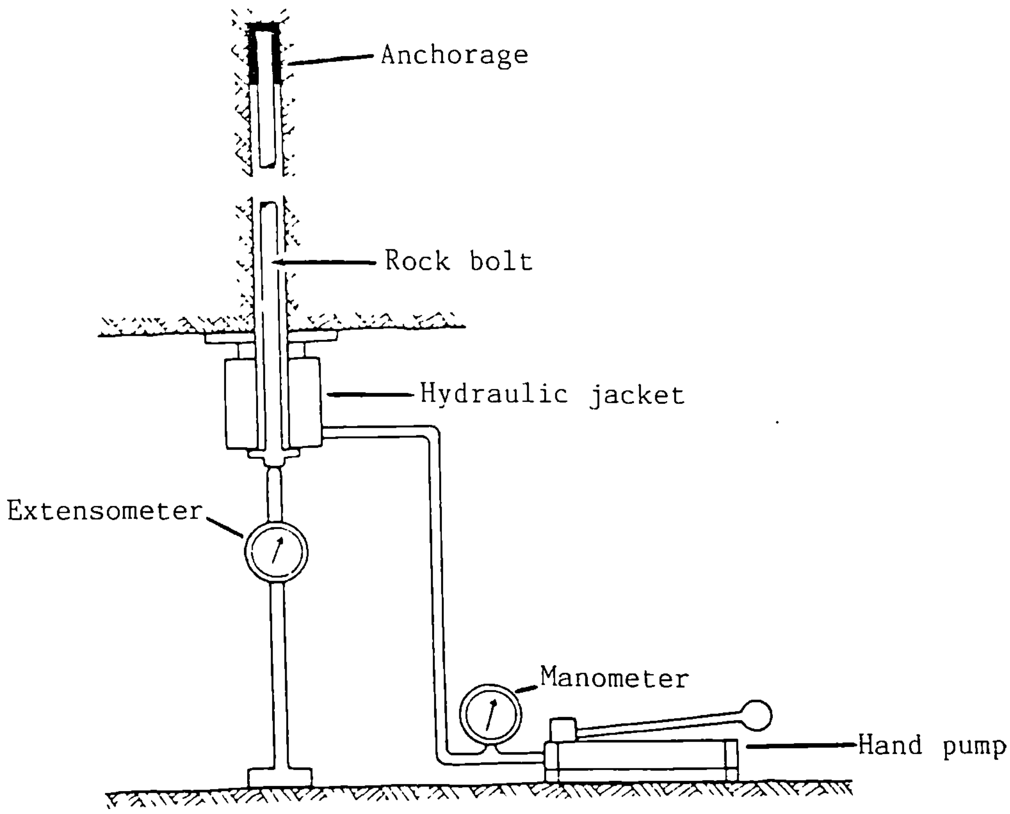


Figure 2.1 Pull test apparatus

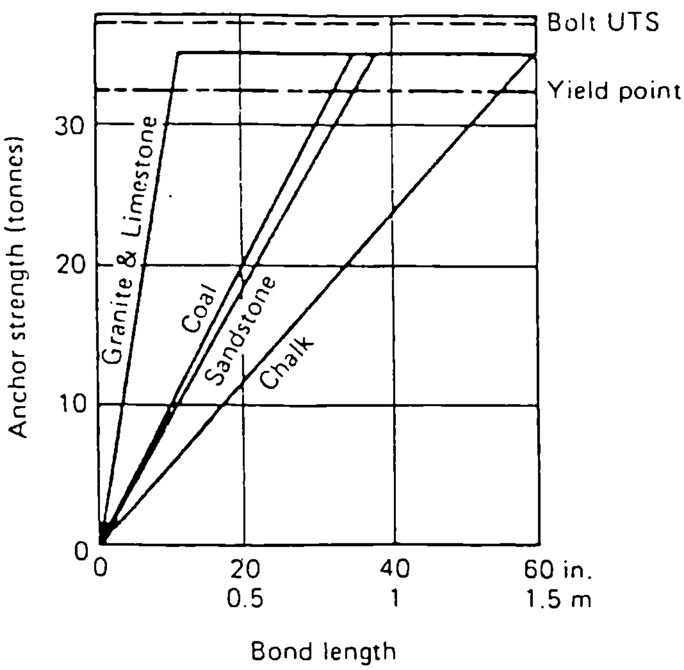


Figure 2.2 Influence of bond length on anchor strength (after Franklin & Woodfield 1971)

Swellex bolts. (Restricting the anchor length of a Swellex is achieved by the use of a steel tube - Section 14.4).

The bonding capability of specific horizons can also be determined from external or single point internal load measuring devices (Section 5.4 .6) fixed to point anchored bolts. The bolt is installed at the head end during roadway drivage. The mean axial bolt load that develops is plotted against face advance. Gale and Fabjanczyk (1985, 1986) consider this approach to be more satisfactory than pull testing as the performance of an anchor in a highly stressed and deforming zone is measured, rather than a static block of effectively destressed rock outbye of the face.

2.4 Parting Planes

The stiffness of a roof stratum determines its ability to support itself and overlying strata. Stiffness is proportional to thickness so that thinly laminated roof rock (with bedding planes parallel to the roadway roof) will tend to separate into thin slabs that are weak and will break easily, whereas thick, massive beds are frequently able to form stable roofs. Delamination of strata can result from deposits of micaceous material along the bedding planes, mineralogical variations, changes in grain size, carbonized debris and non-deposition, causing local induration and erosion surfaces. Roof bolts will help to stiffen the strata and maintain a roof beam.

The choice of roof bolt length should be influenced by the position of major parting planes; for instance it would be undesirable to have the top of the bolts directly below such a plane as this could result in collapse of the bolted slab en masse.

Borehole extensometers are a useful tool for gathering information regarding the position of strata displacements and the height and geometry of any weakened rock above the roadway. If installed in an excavation in the vicinity of a proposed roadway they can give relevant design data.

An extensometer consists of one or more reference anchors positioned at various depths within a borehole. The relative displacements of strata are measured (either mechanically or electrically) as a change in the distance between anchors and the borehole collar. There are many types of extensometers and anchorage points, details are given in Section

5.4.3. Their installation, use and data interpretation are covered by the ISRM (1981) suggested methods.

The position of fracture zones can be identified from shallow slope sections on a plot of anchor distance from excavation surface against anchor displacement. Strains generated in the strata are then determined by calculating the percentage change in length between individual anchors. Points of high strain indicate major parting planes, rock integrity changes and sections where increased loading would be anticipated in full column anchored rock bolts. Rock bolts of sufficient length should be used to extend across dominant rock failure zones (Gale 1986).

Borescope observations assist in the interpretation of results from extensometer installations (Section 5.4.5). The mode of roof dilation and specific intervals and lithologies where fracturing occurs can be determined from the inspection of a borehole adjacent to an extensometer.

2.5 Lateral Thickness Variations

Thinning or thickening of the immediate roof strata will effect the position of significant competent beds or parting planes above the roof line. It is therefore important to map relevant strata thicknesses so that bolting parameters can be altered where necessary.

2.6 Localised Variations In Lithology

2.6.1 Palaeochannels

Palaeochannels are remnants of ancient stream channels that have cut into underlying sediments. Clarke (1963) has made a comprehensive study of roof rolls, washouts, swilleys and other channel related structures found in the Durham coalfield. Depending on the nature of the channel, a variety of lithologies can result (Figure 2.3). Some of these structures and other features associated with a sandstone filled palaeochannel can produce adverse roof conditions.

If a roof sandstone is an aquifer, moisture may be present in roof bolt holes (McCabe and Pascoe 1978). Water dripping from a bolt hole can give an indication of the presence of a channel as the heading advances towards it.

Slickenslides, compactional faults and slumping can result from

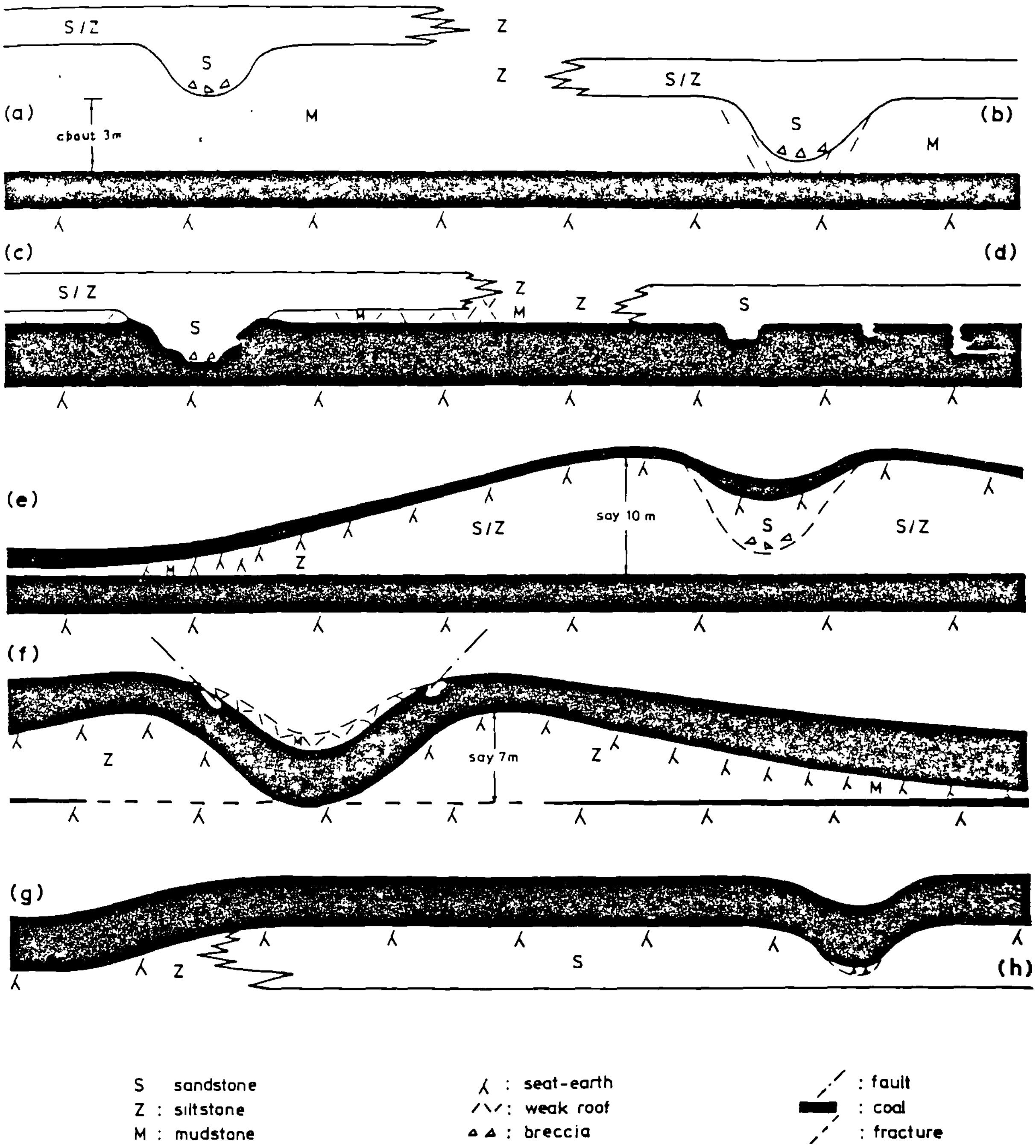


Figure 2.3 Channels at various levels relative to coal seams (after NCB 1984).

differential compaction between the coal or immediate mudstone roof and an overlying sandstone channel (Hylbert 1978).

Geotechnical/palaeoenvironmental mapping can establish the relationships between potential hazards and channels so that an operator will know in advance of mining when drivages are nearing a channel (Ledvina 1986). Where this is not possible, the presence of channels may be inferred in areas where drill core data indicate that thick, lenticular, crossbedded sandstone occurs close to the top of the coal seam. Concretions, clay veins, and thinning and thickening of a coal seam also suggest that a channel is near.

Moebs (1984) recommends the use of angled bolts which anchor into the competent channel filling as a means of supporting mudstone/shale channel margins (Figure 2.4). In severe cases posts and cross bars should be used.

2.6.2 Concretions

The immediate roof of many coal seams contains masses of mineral matter known as concretions. They are usually ellipsoidal in shape and range in size from a few millimetres to several metres in diameter. Concretions that can cause hazardous roof conditions include siderite nodules and coal balls. They are usually accompanied by slickenslided surfaces and are frequently composed of a denser material than the surrounding rock. This facilitates detachment from the roadway roof without warning.

Kettlebottoms are another type of concretion that can produce minor roof falls. They are preserved casts of ancient tree stumps which occur in coal measures strata in Great Britain, the United States, Poland and elsewhere (Raistrict and Marshall 1939; Williamson 1967). Kettlebottom mold and cast surfaces are highly slickenslided. A layer of coalified bark remnants, which varies in thickness from a thin film to 20 mm thick usually separates the kettlebottom mold from its cast. Cohesion between the mold and cast is weak and, when undermined, it is only tensile strength along bedding planes that prevents the structures from detaching. Figure 2.5 illustrates the mode of kettlebottom failure.

Siderite nodules and coal balls tend to be widely distributed in certain roof strata. However, kettlebottoms and similar structures are often small local features of erratic occurrance, which cannot be detected by

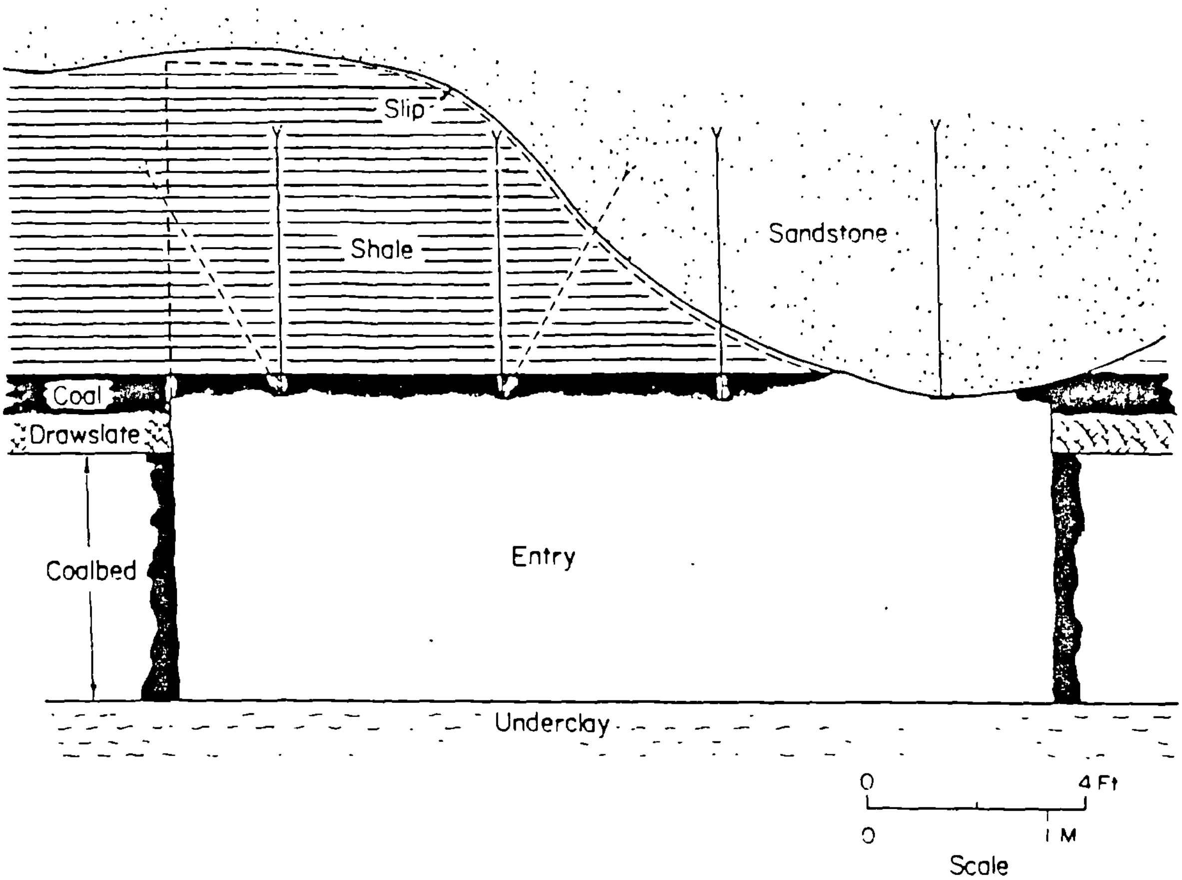


Figure 2.4 Use of angled bolts to support shale strata at channel margins. Dashed line outlines rock not fully supported by vertical bolting (after Moebs 1984).

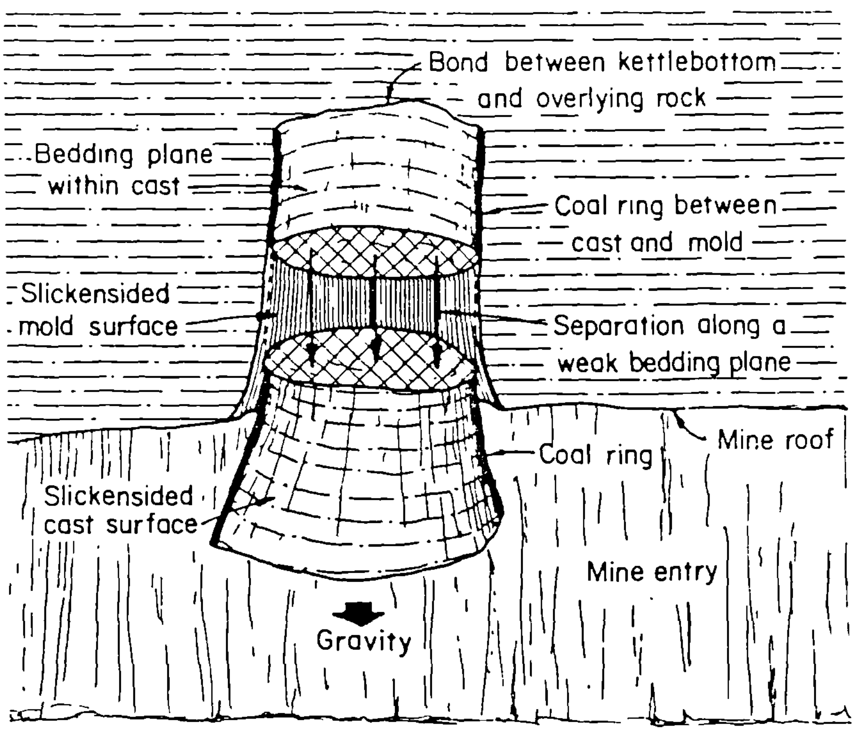


Figure 2.5 Kettlebottom detaching along weak bedding plane (after Chase & Sames 1983).

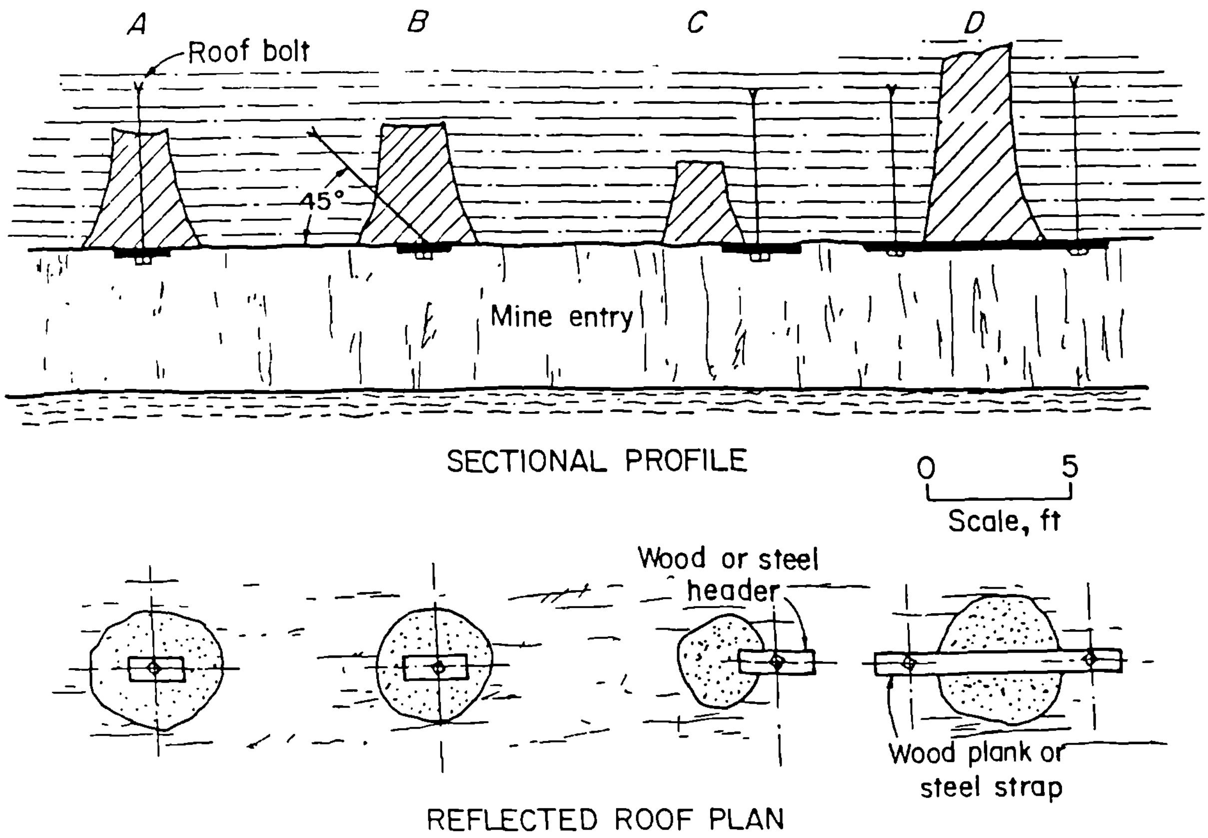


Figure 2.6 Common support techniques for kettlebottoms in US mines (after Chase & Sames 1983).

core drilling.

Where possible small and relatively thin concretions should be barred down from the roof before they fall. Supporting the thicker concretions will prevent the formation of roof voids and minimise the area of roof adversely affected by moist air (Section 2.8). Clusters of small concretions can be supported by bolted straps and/or wire mesh. Larger concretions such as kettlebottoms can be supported by roof bolts. Some techniques used in the USA are depicted in Figure 2.6. Methods A and B can subject the bolter operator to risk as vibration during drilling could be sufficient to dislodge the concretion. In addition, method A assumes that the bolt length available is longer than the structure, this may not always be the case. Method C is more suited to support of concretions less than 1 m in diameter whereas method D should be used for concretions greater than 1 m in diameter.

The practice of leaving a thin layer of top coal is not advisable where kettlebottoms are likely because the coal may not have sufficient strength to support the structure.

2.6.3 Clay Veins

Clay veins are probably the result of clay-filled fissures formed in the seam and surrounding strata before the coal was totally compacted. Slickenslides then develop as a result of differential compaction. These can be orientated either randomly or in parallel sets and contribute to roof instability when the seam is mined. Clay veins range from a few millimetres to two metres or so wide and may persist for a hundred metres in length. Due to their narrow width, clay veins are rarely detected by exploratory drilling. However, they generally have linear to curvilinear strikes and once located can therefore be projected for varying distances in advance of mining (Chase 1985). Preferred orientations can only be determined if clay veins are mapped and analysed.

Ellenberger (1979) recommends the use of full column grouted rock bolts to support the fractured strata in the vicinity of a clay vein. The bolts should be angled towards the centre of the structure so that the slickenslides are bound together and slippage along the planes is prevented (Figure 2.7).

2.7 Fracture Planes

2.7.1 Faults

Faults constitute a structural weakness and wherever they are present rock blocks can detach from the mine roof. Underground excavations can be affected by a variety of fault parameters such as type, inclination, trend, throw and/or horizontal displacement, gouge thickness, vertical extent and the presence of joints, slickenslides and/or anomalous stresses.

A case study carried out in New South Wales, Australia by Shepherd and Fisher (1978) found that normal oblique and strike-slip faults were much more deleterious to drivages than normal dip-slip faults.

The presence of faults in an area of development can be predicted from the study of stratigraphical sections, geological mapping and extrapolation along strike or in the case of multiple seam working, along dip from other areas of the mine.

When a fault zone is encountered in a roadway it is generally necessary to use steel standing supports, although the stability of a fault zone may be enhanced by the correct use of rock bolting techniques. Figure 2.8 shows how extended bolting and angled bolting can improve faulted roof conditions. Jeremic (1980) has demonstrated the danger of mining beneath a low angle fault in the immediate roof (Figure 2.9).

2.7.2 Slickenslides

Predominantly a feature of argillaceous rocks, slickenslides (or "slips") are smooth, polished and sometimes striated surfaces resulting from movement of rock on either side of a plane. Slickenslides in coal mine roof strata are generally curved in a convex fashion towards the coal bed. They have little or no cohesive strength. Slickenslides are found throughout the British coalfields but are particularly common in South Wales and Kent. Where they occur in large concentrations, core drilling may be able to detect these features in advance of mining. They are known to be associated with faults, palaeochannels and clay veins (Sections 2.7.1, 2.6.1 and 2.6.3).

Zones with slickenslides of limited extent can be supported by rock bolts with straps and/or wire mesh. Large slickenslides are often extremely hazardous but may be supported by a dense pattern of angled bolts (Figure 2.10).

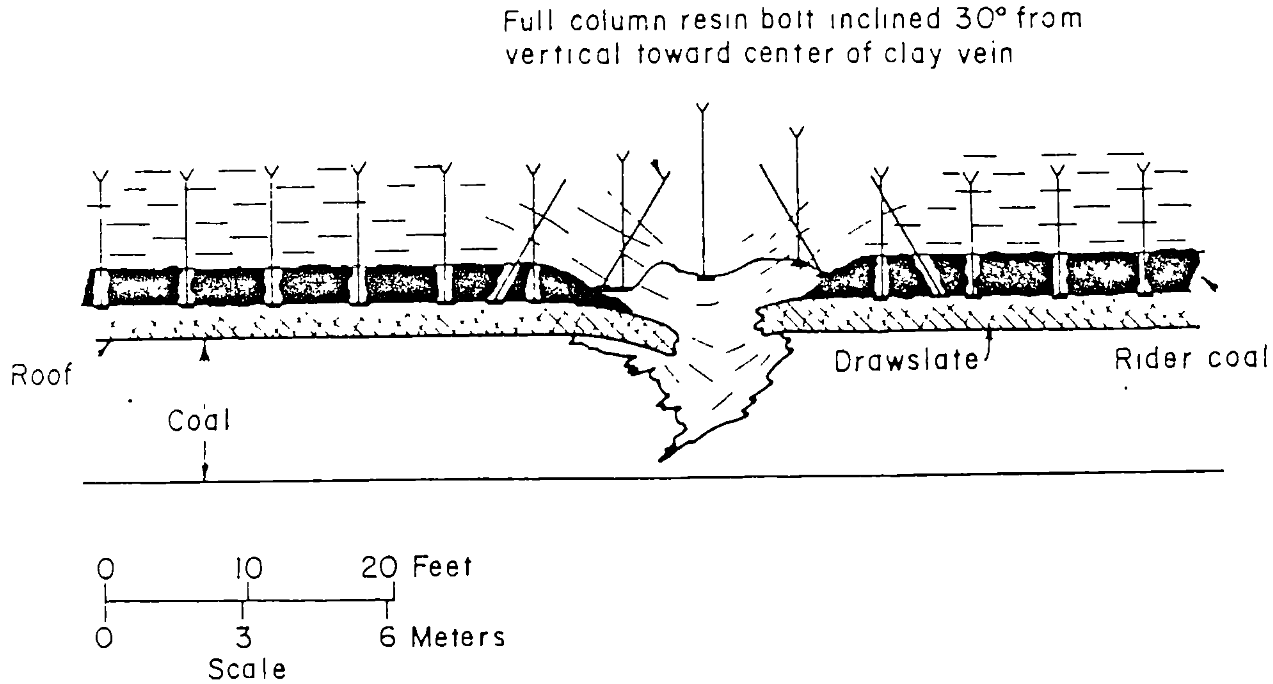


Figure 2.7 Proposed method for support of fractured strata in the vicinity of a clay vein (after Ellenberger 1979).

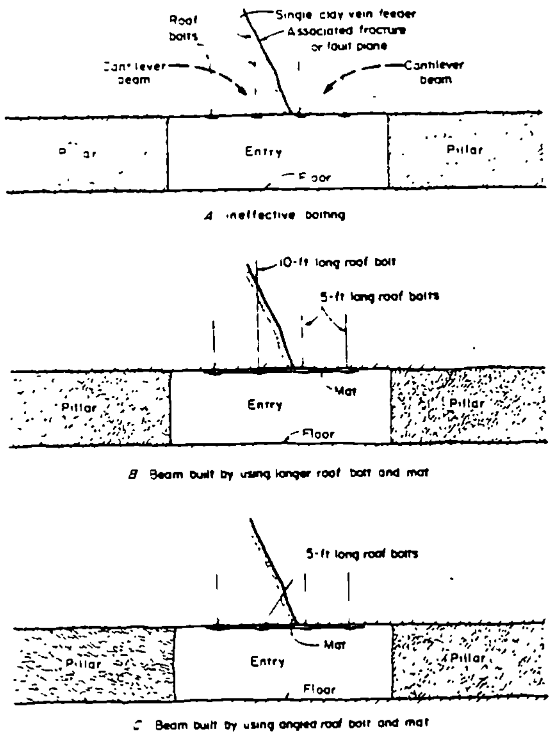


Figure 2.8 Clay vein associated fracture and fault plane bolting diagrams (after Chase 1985).

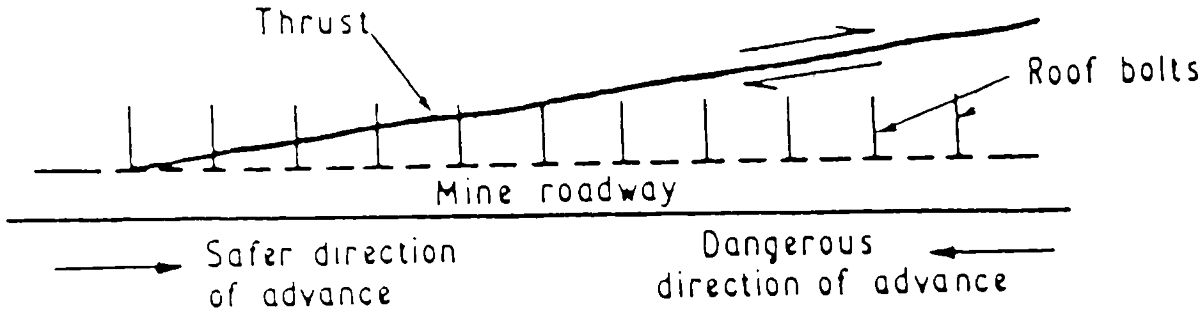


Figure 2.9 Wedge-shape cantilever liable to failure because of a thrust fault (after Jeremic 1980).

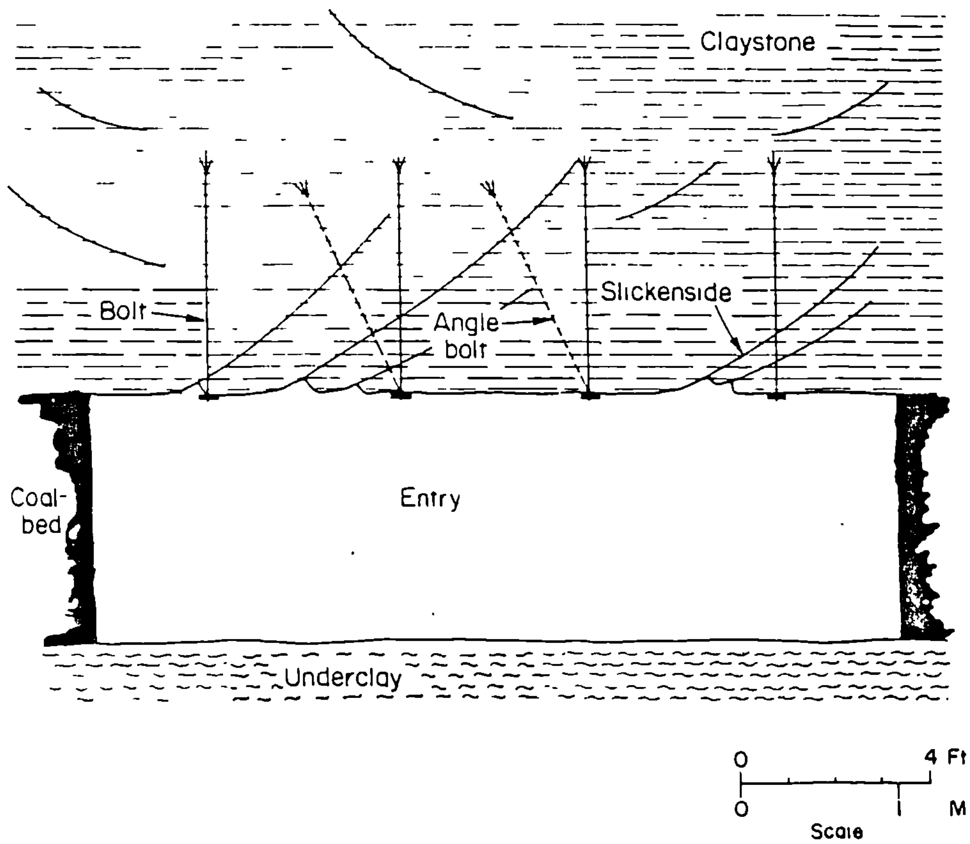


Figure 2.10 Angled bolting of slickenslides (after Moebs 1984).

2.7.3 Joints

The influence of joints on roof stability depends upon:

(a) The nature or type of fracturing.

(b) The length, continuity, direction and attitude of the joint surfaces and their relative orientations to mine openings.

(c) The density or spacing and convergency of joints.

(d) The interaction with other factors of geological weakness.

Methods of describing and measuring joint parameters have been described in detail by the Geological Society Engineering Group (1977) and ISRM (1981). Coates et al (1977) demonstrated how inclined core drilling through near-horizontal beds and sub-vertical joints can give an excellent indication of joint frequency, orientation and filling. Inaccuracies can occur in joint spacing measurements made from borehole core due to rock breakage during the drilling operation and removal from the core barrel. It is possible to measure the orientation of joint sets from borehole cores providing care is taken to orientate the core. Artificial orientation devices operated from the core barrel are available, such as the Craelius core orientator. To obtain joint orientation data from heavily fractured rock masses, a core recovery method known as the integral sampling method can be employed (Rocha and Barroso 1971). Prior to recovery, the core is reinforced with a grouted bar whose azimuth is known from positioning rods. The reinforced bar is coaxially overcored with a large diameter coring crown.

If possible in situ measurements should be taken in existing nearby excavations along scanlines (Priest and Hudson 1976, 1981). The use of orthorhombic sets of scanlines (Anderson et al 1977) will help prevent preferential sampling of joints orientated normal to the scanline (Terzaghi 1965). McCrae and Cook (1985) suggest a combination of scanline mapping and sketching of all the exposed rock will result in a more thorough survey and enable the identification of possible areas of instability. Ewan et al (1983) have reported on the reproducibility of joint spacing and orientation measurements taken on scanlines. They concluded that the variation in the number of joints recorded by different observers can be as high as a factor of four, but with a mean of about two; and for measurement of orientation an average maximum error of $\pm 10^{o}$ for dip direction and $\pm 5^{o}$ for dip angle was recorded.

2.7.4 Cleat

Cleat refers to conjugate joint systems in coal that are composed of closely-spaced, sub-vertical fractures. The best developed set is known as the primary cleat. There is usually a secondary and occasionally a tertiary cleat system. There are a number of theories concerning the formation of cleat sets. McCullock et al (1974) believe that they are the result of tectonic activity (as are joints), whereas Ting (1977) suggests that they are caused by dehydration during the coalification process.

Measurement of cleat orientations from borehole core is particularly difficult due to the friable nature of coal. However, measurement of underground exposures is relatively simple (Section 2.7.3).

The direction of drivage relative to the cleat orientation is often an important factor in determining in-seam roadway stability. Roadways driven parallel to the direction of the primary cleat are particularly susceptible to ribside spalling of the coal, which can increase the roadway width above the critical dimension so that roof and floor instability may ensue.

Ribside bolting patterns can be designed to intersect these planes of weakness and enhance stability, especially when used in conjunction with a liner (Chapter 10).

Coal face spalling may occur at the head end in roadways driven perpendicular to the primary cleat and if top coal is left in the roof it is often more liable to collapse.

2.8 Weatherability Of Strata

Argillaceous roadway roofs are often subjected to delayed deformation due to deterioration as a result of weathering in the mine environment. This problem has been recognised for many years in the United States and considerable research has been carried out by the US Bureau of Mines on the causes and effects of roof deterioration as well as methods of prevention (Hartman and Greenwald 1941; Fish et al 1944; Bobeck and Clifton 1973; Haynes 1975; Aughenbaugh and Bruzewski 1976; Stateham and Radcliffe 1978; Radcliffe and Stateham 1978; Cummings et al 1981; Cummings et al 1983). A study of the breakdown of British coal measure rocks has been undertaken by Taylor and Spears (1970).

Chemical degradation of a mudstone roof caused by humid mine air will result in a decrease in strata strength. In addition, physical weathering due to expansion resulting from moisture absorption will also cause deterioration, especially when subjected to alternating wet and dry spells.

Slake durability tests give an assessment of weatherability (ISRM 1981; Davis 1981). Dejean and Raffoux (1980a) recommend evaluating the rock permeability to give an estimation of its liability to deterioration. Rock with a high permeability should be further analysed by full pressure water filtration. A high concentration of calcium and potassium ions recovered from the first filtrate indicates a particularly susceptible rock.

The presence of pyrite should be noted as crystals readily decompose by hydration and oxidation in mine air; the resulting sulphuric acid reacts with argillaceous minerals. In addition, the dilative recrystallization force from sulphate mineral formation will micro-fracture the surrounding strata.

Full column bonding of rock bolts will prevent strata deterioration at the bolt hole wall. There are a number of methods for protecting the roof surface. Leaving a thin layer of top coal to buffer the roof is one technique; alternatively sealants such as sprayed concrete, tar or polymeric sealants may be applied. Another method favoured in the USA is the incorporation of conditioning chambers into the mine layout for air tempering (Sames 1985).

2.9 Groundwater

High ground water in-flow rates can have a serious effect on the stability of an underground opening. Water will tend to reduce interfacial friction on parting planes and joint surfaces, as well as erode and weather the strata.

Longwall mining induced fractures can disturb roof strata up to 30 to 50 times the mining height. If an aquifer is located in the fractured zone, water may drain down into the workings. Geological mapping will assist in the identification of aquifers likely to influence a roadway. The interstitial pressure is measured using piezometers installed in the strata and the permeability of a rock mass is measured by means of a permeametric test.

The presence of excessive water reduces the anchoring strengths of inorganic grouts (Section 9.27 and Hunter 1986) and will limit the use of point anchored and friction bolts to a temporary support applications due to bolt corrosion. If sprayed concrete is applied it is important to prevent pressure build up behind the lining.

2.10 Ground Stresses

Analysis of the in situ stress field is an important consideration in the design of a roadway rock bolt reinforcement system. Prior to the excavation of an opening, states of stress exist in the rock mass which are functions of gravitational and tectonic forces, thermal stresses, gas pressures, and material and rheologic properties of the strata. In the British Coal Measures, thermal stress and gas pressures are regarded as having a negligible effect.

The gravitational vertical stress is directly related to the depth of overburden. Assuming the average unit weight of strata above coal measures is in the order of 0.025 MN m^{-3}, the vertical component of stress (σ_v) can be taken as approximately:

$$\sigma_v = 0.025H \text{ MPa}$$

where H = depth of overburden (m)

A study by Brown and Hoek (1978) of actual in situ stress measurements taken at many locations throughout the world (mainly in hard rock) has shown the magnitude of horizontal stress to be much more variable than the vertical component (Figure 2.11). The range of the horizontal component of stress (σ_h) was found to be:

$$\sigma_h = 0.5 \text{ to } 4.0 \times \sigma_v$$

Virtually no measurement data on in situ stress fields has been obtained for British coal mines. Wilson (1980) considers that in these relatively soft rocks it is probable that creep over geological time will have caused equalization of horizontal and vertical pressures. Many observations of deformation and failure around British coal mine roadways support this hypothesis, however; it is now becoming apparent that anisotropic stress fields may exist in some collieries (Golder Associates 1986; Gale 1987).

In soft rock at depth, the induced stress magnitudes frequently exceed the rock strength, this results in rock failure and the subsequent development of a yield zone.

Geomorphological features such as streams, valleys, and mountains as well as geological features such as sedimentary structures (e.g. paleochannels) and igneous bodies can affect principal stress magnitudes and directions. High stresses should also be expected in the vicinity of major faults.

Obert (1966) suggests that the measured stress condition near an underground opening may differ from the theoretical prediction due to stress relief in the fractured rock around the opening which shifts the point of high stress further into the rock mass. There are many techniques and devices available for the measurement of ground stresses. Some of these are described by Bauer (1985) and ISRM (1987). Useful information concerning the nature of the stress field surrounding a roadway can be obtained by means of observations (e.g. mapping stress induced fractures) or deformation measurements.

Just as the excavation of a single opening redefines the state of stress in a rock mass, the excavation of adjacent, superjacent or subjacent openings will also result in further redistribution of stresses.

Figure 2.12 illustrates the stress distribution around a longwall face. At some distance from the excavation there is a gradual increase in the vertical stress above cover load, reaching a maximum a short distance from the boundary of the excavation. There is a destressed region on the excavation side of this peak. The front abutment zone generally extends 30 m in front of the face; a significant increase occurs 15 to 5 m from the face line, with a peak 1 to 3 m ahead of the face. The magnitude of the front abutment can vary considerably depending on the nature and structural characteristics of the surrounding strata, the distance from effective support and the extraction height. It is commonly in excess of four times the cover load.

Determination of adequate pillar sizes is of particular importance in roadway design. Formulae derived by Wilson (1980) can give some estimate of the stress distribution in a coal ribside adjacent to a longwall face extraction. The calculations are based on the "stress balance" principle, whereby the stress reduction over the longwall waste must be compensated for by an equivalent stress increase over the ribside and vice-versa.

Interactions between subjacent and superjacent workings tend to be

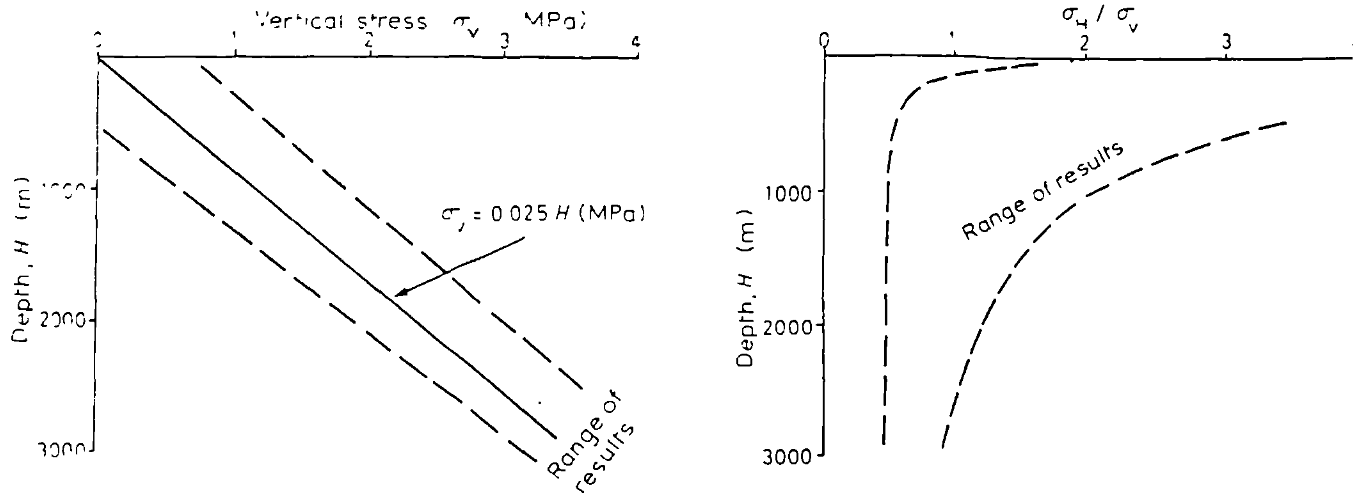


Figure 2.11 Variation of virgin vertical and horizontal stresses with depth (after Brown & Hoek 1978).

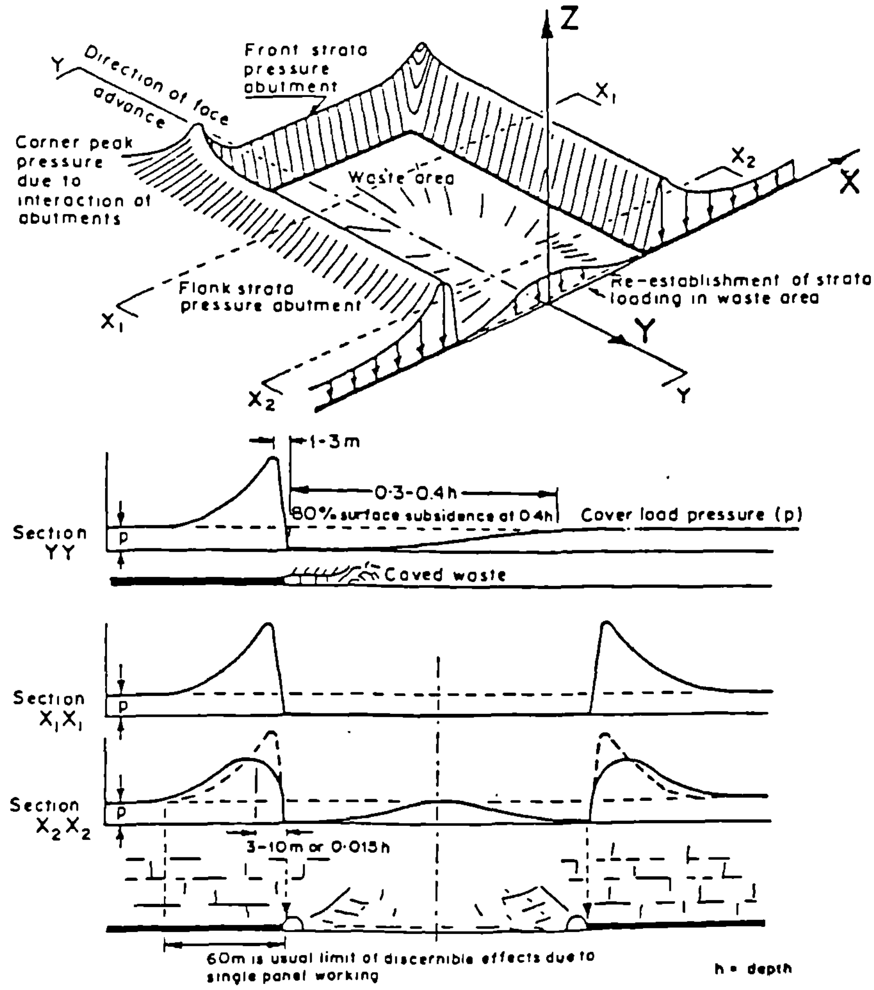


Figure 2.12 Stress distribution around a longwall face.

difficult to predict owing to their complex nature. Redistribution of stress between pillars results in high vertical stress concentrations under pillar edges which are particularly prevalent within 50 m of the extraction.

Anisotropic stress environments, either natural or the result of mining activities, will considerably influence the occurrence and mode of rock failure.

Under high lateral stresses, massive roof and floor rock will fail with low angle shearing whereas laminated strata will fail forming an inverted V-type fracture pattern (Lawrence 1972; Parker 1973; University of Nottingham 1985). Field observations (Blevins and Dopp 1985) and three dimensional stress analysis using the boundary integral equation method (Gale and Blackwood 1987) have shown that in stress fields with dominant lateral stress components, the roadway drivage direction has a considerable effect upon the type and geometry of failure in the surrounding rock mass. In the United States, regionally high horizontal stresses are considered to be one of the causes of cutter roof failures (Moebs and Stateham 1986; Hill 1986; Su and Peng 1987). A cutter is a steeply dipping fracture that initiates at the ribline and propagates upwards into the roof rock. The likelihood of rock shear failure increases as the roadway axis tends towards 90° to the maximium lateral stress component (Figure 2.13). Thus it is very important to consider the magnitude and orientation of in situ stress fields during mine planning.

Under high vertical stress concentrations, vertical or sub-vertical tensile cracks tend to develop above the roadway ribline without the formation of an inverted V-type failure in laminated strata. In extreme cases roof collapse can occur en masse.

Rock bolt reinforcement of roadways subjected to anisotropic stress fields is discussed in Section 4.6.2.

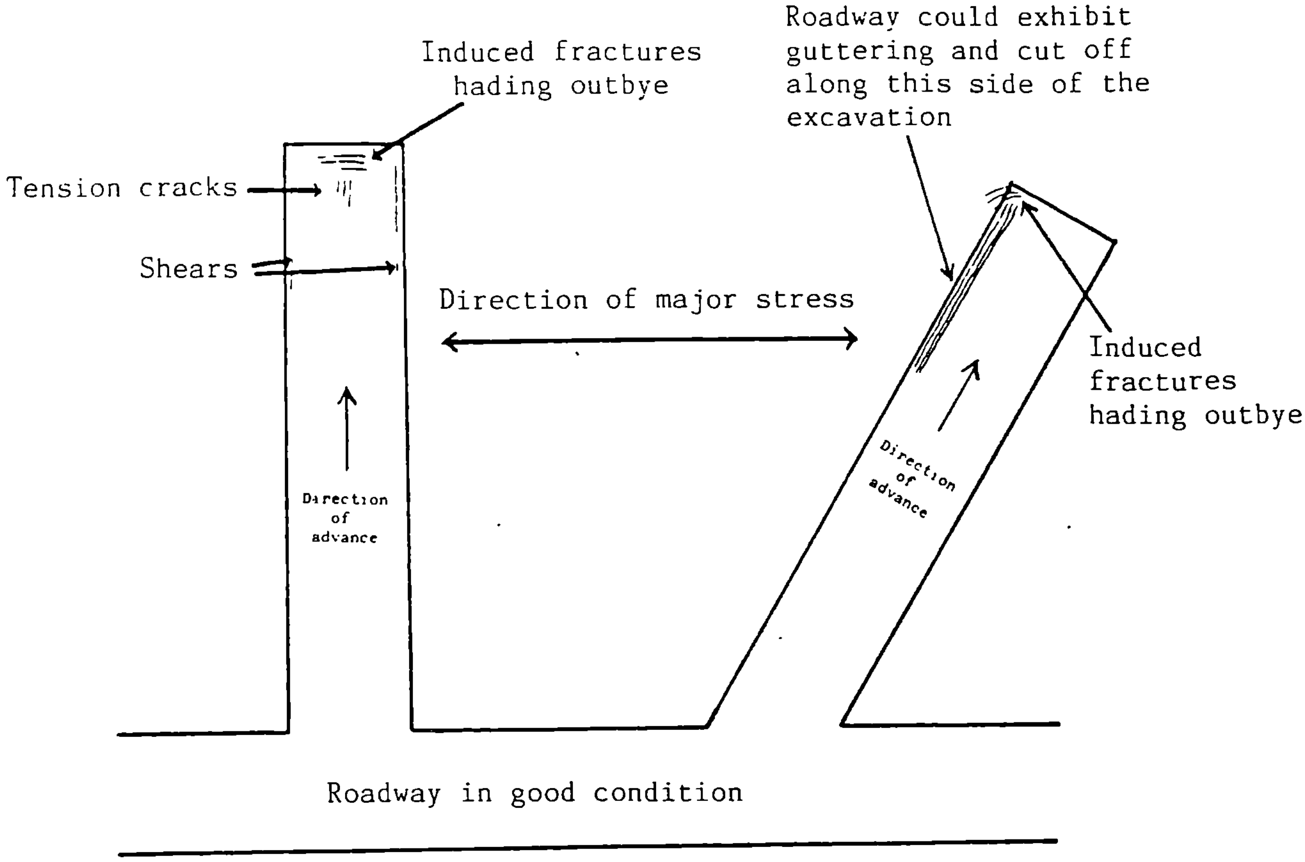


Figure 2.13 Directional stability effect of maximum lateral stress component.

CHAPTER 3
EMPIRICAL DESIGN METHODS

3.1 Background

Empirical methods for the design of rock bolt reinforcement systems are based on statistical treatment of observations made in numerous underground excavations. The statistical data is often used to elaborate some form of rock classification system based on one or more prominent parameters. Guidelines for the selection of support systems are then applied to the identified classes. Empirical methods can be applied during the initial design process (when limited geotechnical data is available) to give an early indication of likely support requirements and later, during roadway excavation, they provide a check on the results of more detailed analysis.

3.2 Rock Quality Designation

Rock Quality Designation (RQD) proposed by Deere (1964) is a quick and simple scheme that has gained worldwide acceptance and become a standard core description method. It is a modified core recovery index based on rock hardness and fracture frequency. The RQD should be determined from core of at least BXM size (42 mm diameter) and is defined as the percentage of core recovered in intact pieces of 100 mm or more in length in the total length of borehole run studied;

$$\text{RQD (\%)} = \frac{\text{Length of core in pieces} > 100\text{ mm}}{\text{Length of run}} \times 100$$

The relationship between the RQD value and classes of rock quality defined by Deere (1964) are given in Table 3.5.

Only considering one parameter, it is obviously very limited as a design when used alone. However, Merritt (1972) has plotted a relationship between excavation width and type of support installed in a number of tunnels driven in a variety of rock types (Figure 3.1). Farmer and Shelton (1980) combined the simple descriptive classification of Terzaghi (1946) with support proposals suggested by Deere et al (1970) based on RQD (Table 3.1).

Some of the sources of error in the determination and application of RQD have been examined by Bikerman and Mahtab (1986) who conclude that it is a useful descriptive device but its application in complex correlative equations may not be justified. They show how different RQD values can

Terzaghi classification	Rock behaviour and possible causes of instability	Approximate stand-up time	Deere classification	Construction method	Deere's support proposals Spacing or pattern, m	Additional requirements and anchorage limitations
1. Hard and intact	Stable excavation unless induced stresses greater than rock strength	Many years	Excellent: RQD 90–100	Boring machine	None to occasional	Rare
2. Hard stratified and schistose	Bed separation with time; surface spalling	1 year		Drill and blast	None to occasional	Rare
3. Massive, moderately jointed	Immediately stable: detachment of blocks under gravity with time	1 month	Good: RQD 75 90	Boring machine	Occasional to 1.5–1.8	Occasional mesh and straps
4. Moderately blocky and seamy	Immediately stable: detachment of blocks, progressively releasing further blocks			Drill and blast	1.5–1.8	Occasional mesh and straps
5. Very blocky and seamy and shattered	Immediately fairly stable: surface dilatation of rock due to rapid block detachment	1 day	Fair RQD 50 75	Boring machine	1.2–1.8	Mesh and straps as required
				Drill and blast	0.9–1.5	Mesh and straps as required
6. Completely crushed	Local roof falls during excavation: Rapid peripheral dilatation	1 hour	Poor: RQD 25–50	Boring machine	0.9–1.5	Anchorage may be hard to obtain, considerable mesh and straps required
				Drill and blast	0.6–1.2	Anchorage may be hard to obtain considerable mesh and straps required
7. Sand and gravel	Immediate collapse	0	Very poor: RQD 0–25	Boring machine	0.6–1.2	Anchorage may be impossible: 100% mesh and straps
				Drill and blast	0.9	Anchorage may be impossible: 100% mesh and straps
8. Squeezing: moderate depth 9. Squeezing: great depth 10. Swelling	Rapid yielding and deformation		Squeezing and swelling ground	Both methods	0.6–0.9	Anchorage may be impossible: 100% mesh and straps

Table 3.1 Rock cl cation for preliminary assessment of rock bolt support requirements (after & Shelton 1980).

arise from different drilling teams. Problems can occur with the misinterpretation of drilling induced fractures and accidental damage. In addition, fracture orientation with respect to drilling must be considered. For example, the RQD obtained from a borehole drilled perpendicular to fractures regularly spaced at 85 mm (±14 mm) intervals would be 0%. Whereas if the hole was drilled at an inclination of 45% a fracture spacing of 113 mm would be recorded giving a RQD of 100%.

Hansagi (1965 and 1974) developed a method for determining the degree of fissuration of rock by analysis of core fragmentation. Five classes of rock strength were introduced based on uniaxial compressive strength determined from rock samples and the fissuration factor (c or Kiruna factor) determined from analysis of borehole core. For particular classes of rock strength Hansagi (1974) recommends the use of rock bolting with different parameters, pointing out that a roadway 5 m wide and 3.6 m high with a roof rock compressive strength greater than 35 MPa will not require support. This method was developed for the Kiruna mine in Sweden and is clearly not directly applicable to British coal mining conditions in its present form.

3.3 Stability Index

Sikora and Kidybinski (1977) have developed a method for obtaining a value of effective strength of mine roof rock using a hydraulic borehole penetrometer (Stears 1965). A rock stability index is then obtained which Sikora and Kidybinski applied to the design of coal mine roadway support in Upper Silesia. The average effective strength (R_{ef}) of the roof is calculated from penetration resistance profiles taken from an 86 mm diameter borehole according to the formula

$$R_{ef} = w.P_{sr}$$

where R_{ef} = effective rock strength (T/m^2)
P_{sr} = average value of critical penetration pressure (from pressure gauge)
w = penetrometer coefficient

The rock stability index (S_g) is determined from the equation

$$S_g = R_{ef}/H.k.a.b$$

where H = depth
k = stress concentration factor, obtained from in situ measurement or approximated from Table 3.2
b = rock failure factor, approximated from Table 3.3
a = exposure factor depending on roadway size, approximated from Figure 3.2

Value	Type and location of roadway
1.5	Main roadways in the intact rock mass from extraction work.
2.0	Main roadways and development workings in extraction panels beyond the zone of abutment pressures.
2.5	Roadways close to the working area driven in coal of low and medium strength (ucs up to 24 MPa).
3.0	Roadways close to the working area driven in coal of high strength (ucs greater than 24 MPa).

Table 3.2 Values of stress concentration factor (k).

Value	Type and location of roadway
1.0	Main roadways in intact rock mass.
1.2	Main roadways in disturbed rock mass.
1.4	Development workings in undisturbed rock mass.
1.6	Development workings in disturbed rock mass.

Table 3.3 Values of rock failure factor (b).

Guidelines for the choice of supports according to the value of the stability index are given in Table 3.4.

3.4 The Q-System

From the study of numerous case histories Barton et al (1974) developed a tunnelling quality index (Q). The Q-system provides a numerical rating of rock quality based on the spacing, orientation and strength characteristics of rock fractures, as well as groundwater and stress conditions. The value of Q can range from a high of 1000 for extremely good rock without fractures to a low of 0.001 for exceptionally poor highly fractured rock. The value of the tunnelling quality index (Q) is defined as

$$Q = (RQD/J_n) \times (J_r/J_a) \times (J_w/SRF)$$

where RQD = rock quality designation
J_n = joint set number
J_r = joint roughness number
J_a = joint alteration number
J_w = joint water reduction factor
SRF = stress reduction factor

The first quotient (RQD/J_n) gives a measure of block or particle size, the second (J_r/J_a) relates to inter block shear strength and the third (J_w/SRF) gives an indication of active stress. Values for each of the above parameters are established by referring to Table 3.5.

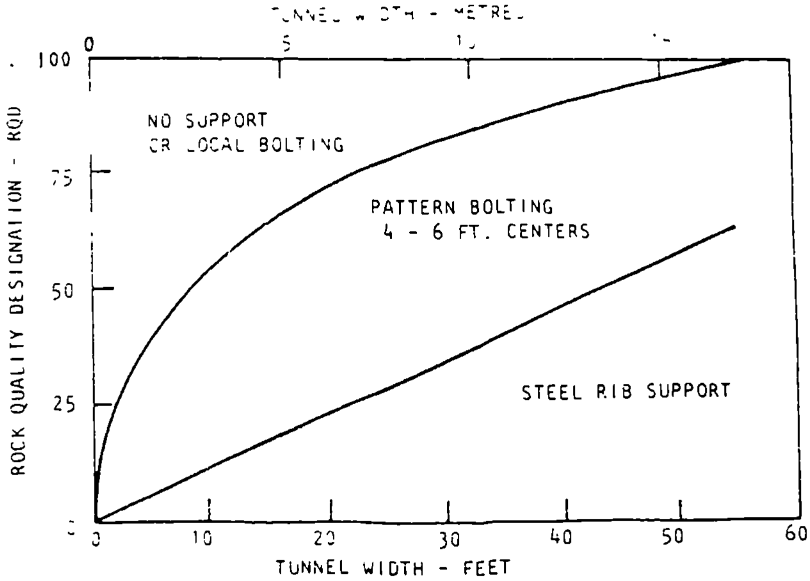


Figure 3.1 Proposed use of RQD for choice of rock support system (after Merritt 1972).

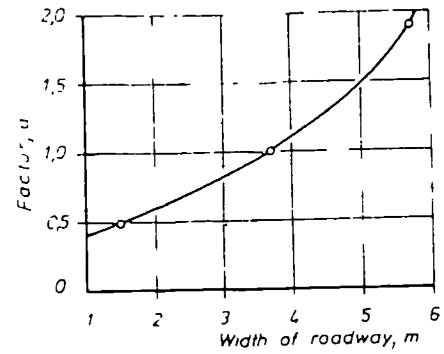


Figure 3.2 Relationship between the exposure factor (a) and roadway width (after Sikora & Kidybinski 1977).

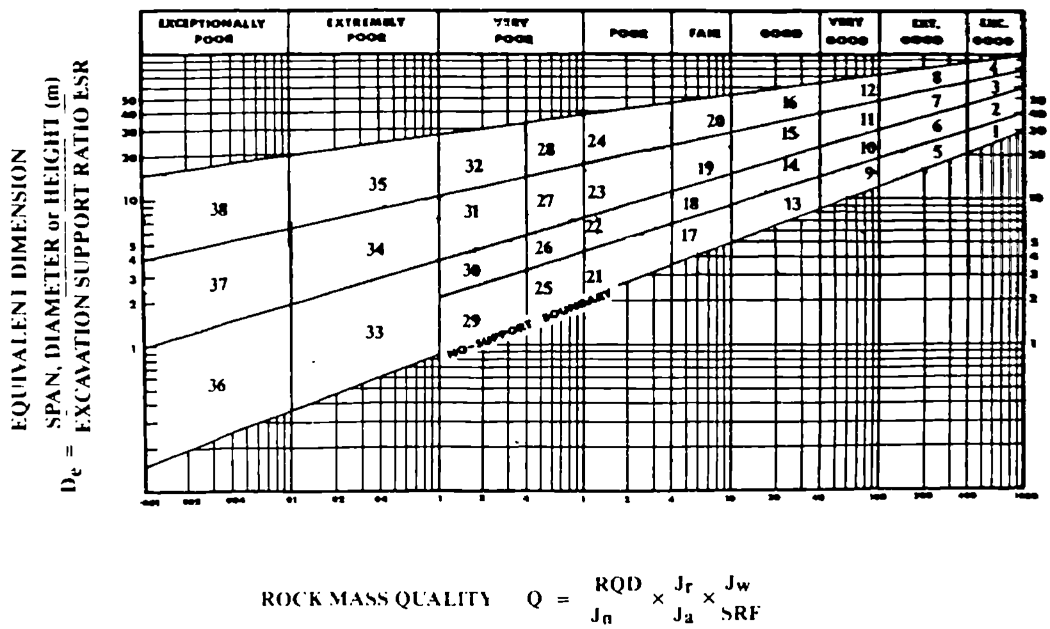


Figure 3.3 Support categories based on rock mass quality and equivalent dimensions (after Barton et al 1974).

	less than 0.60	from 0.61 to 0.80	from 0.81 to 1.00	from 1.01 to 1.20	more than 1.20
Main roadways	Preferred closed supports sets (circular or similar, made of concrete or prefabricated units) made yielding with pads of wood or other materials ensuring about 400 mm radial yield without supports failure or in the case of hard floor by rigid steel arches LS (G-140) with shotcrete 200 mm thick at the roof and 150 mm at the ribs. Required overall carrying capacity of supports not less than 15-30 T/m²	Preferred supports by rigid steel arches LS (G-110) bolted to the rock mass. At the lower limit of the range S_g alternative supports by rigid steel arches LS (G-140) with shotcrete about 100-150 mm thick or supports by steel arches LS (G-140) and resin-bonded bolts about 1/2 S long in the roof and sides. The following types of supports may also be used: 1. supports made of concrete with the increased tensile strength, 2. supports made of prefabricated units with the appropriate strength. Carrying capacity of 10-15 T/m² is necessary. Yielding of the contact of supports with the rock mass by means of thick and tight stone filling.	Preferred supports by yielding steel arches LK spaced every 1.0 m with lapped laggings in the roof and flat in the sides. Tight stone filling necessary. At the lower limit of the range it is necessary to reinforce the supports by resin-bonded bolts. The following types of supports may also be used: 1. supports by rigid steel arches LS (G-110) or equivalent, 2. expanded metal net MM in the roof and sides with resin-bonded bolts 1/3-1/25 long and set every 1.0 m with shotcrete about 100 mm thick in the roof and about 50 mm thick in the sides. Overall required carrying capacity of supports - 5 - 10 T/m².	Preferred supports by resin-bonded bolts 1/4-1/3 S long set every 1.0-1.5 m and metal net to protect the roof and upper part of the side. Advisable shotcrete about 50 mm thick. Overall required carrying capacity of supports 3 5 T/m²	Supports not necessary, opening should be protected against falling of the occasional parted lumps by means of a metal net fastened to the roof of roadway by short bolts
Development workings	Closed circular supports or similar made of rigid steel arches LS (G-110 profile) with overlapped laggings. In the case of hard floor - rigid steel arches LS (G-140) tied to the roof and sides by resin-bonded bolts 1.2 S long at least, 4-6 bolts per arch.	Supports by rigid steel arches LS (G-110) or equivalent spaced every 1.0 m. At the lower limit of the range S_g these supports should be reinforced with resin bolts (length of bolts of 1/3 S at least) or rigid steel arches (G-140) should be used without bolting. Necessary yielding of the contact with the rock mass by means of thick and tight stone filling.	Supports by yielding steel arches LK spaced every 1.0 m and by metal net (type MM) on the sides while laggings are set flat in the roof. Tight stone filling necessary. At the lower limit of the range S_g supports should be reinforced with shotcrete about 100 mm thick in the roof and about 50 mm thick on the sides, or supports by rigid steel arches (G-110) are used.	Resin bolts 1/3 S long every 1.0-1.5 m in the roof and resin bolts (for wooden bolts) 1/2 S long in the sides. Metal net in the roof and sides may be used additionally.	Resin bolts 1/3 S long at every 1.5 m in the roof and wooden bolts in the sides, metal net
Gate headings	Closed rectangular or circular supports sets with the bearing capacity of 20 T/m² at least, and radial yieldability of more than 500 mm.	Preferred rectangular supports by friction props and roof bars with box section or G-110-140 profile. At the lower limit of the range S_g, supports tied to the ribs and roof with bolts. Density of sets of the lower limit of the range - every 0.5 m, at the upper limit - every 1.0 m.	Preferred supports by yielding steel arches LK with metal net on the sides and laggings in the roof. At the lower limit of the range S_g, the supports are reinforced with metal resinbolts or wooden bolts tied to the crowns and legs.		

Table 3.4 Support recommendations based on the rock stability index (after Sikora & Kidybinski 1977).

S - width of roadway measured on the floor

LS - steel rigid arches jointed by screwed fish plates made of 2T section or railway rails

G - 110 - heavy section 2T, 110 mm high

G - 140 - heavy section, 2T, 140 mm high

LK - steel yielding arches made of 19 25 kg/m channels jointed frictionally by means of screwed stirrups

MM - nets expanded from ut steel sheets

Description	Value	Notes
1. Rock Quality Designation	(*RQD*)	
A. Very poor	0— 25	(I) Where *RQD* is reported or measured as ≤10, (including 0) a nominal value of 10 is used to evaluate *Q* in equation (3).
B. Poor	25— 50	
C. Fair	50— 75	
D. Good	75— 90	
E. Excellent	90—100	(II) *RQD* intervals of 5, i.e. 100,95,90, etc. are sufficiently accurate.
2. Joint Set Number	(J_n)	
A. Massive, no or few joints	0.5—1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	(I) For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	(II) For portals use ($2.0 \times J_n$)
H. Four or more joint sets, random, heavily jointed, "sugar cube" etc.	15	
J. Crushed, rock, earthlike	20	
3. Joint Roughness Number	(J_r)	
(a) Rock wall contact and		
(b) Rock wall contact before 10cms shear		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	(I) Descriptions refer to small scale features and intermediate scale features, in that order.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	(II) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.
c) No rock wall contact when sheared		(III) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minium strength
H. Zone containing clay minerals thick enough to prevent rock wall contact	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0	

4. Joint Alteration Number	(J_a)	(**ϕ_r approx.**)
(a) Rock wall contact		
A. Tightly healed, hard non-softening, impermeable filling i.e. quartz or epidote	0.75	(—)
B. Unaltered joint wall, surface staining only	1.0	(25—35°)
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25—30°)
D. Silty-, or sandy-clay coatings, small clay fraction (non-soft.)	3.0	(20—25°)
E. Softening or low friction clay mineral coatings, i.e. kaolinite or mica. Also chlorite, talc, gypsum, graphite etc., and small quantities of swelling clays.	4.0	(8—16°)
(b) Rock wall contact before 10 cms shear		
F. Sandy particles, clay-free disintegrated rock etc.	4.0	(25—30°)
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5 mm thickness).	6.0	(16—24°)
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous but <5 mm thickness).	8.0	(12—16°)
J. Swelling clay fillings, i.e. montmorillonite (continuous, but <5 mm thickness) Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8—12	(6—12°)
(c) No rock wall contact when sheared		
K, L, M. Zones or bands of disintegrated or crushed rock and clay (see G,H,J for description of clay condition).	6,8, or 8—12	(6—24°)
N. Zones or bands of silty-or sandy-clay, small clay fraction (non-softening).	5.0	(—)
O,P, R. Thick, continuous zones or bands of clay (see G,H,J, for description of clay condition).	10,13, or 13—20	(6—24°)

Table 3.5 Classification of individual parameters used in the Q-system (after Barton et al 1977).

	Description	Value		Notes
5.	**Joint Water Reduction Factor**	(J_w)	**Approx. water pres. (kp/cm^2)**	
A.	Dry excavations or minor inflow, i.e. <5 l min. locally.	1.0	<1.0	
B.	Medium inflow or pressure, occasional outwash of joint fillings.	0.66	1—2.5	
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5—10	(I) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5—10	
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2—0.1	>10	
F.	Exceptionally high inflow or water pressure continuing without noticeable decay	0.1—0.05	>10	(II) Special problems caused by ice formation are not considered.

6. Stress Reduction Factor

	(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated	*(SRF)*	Notes
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth).	10	(I) Reduce these values of *SRF* by 25—50 % if the relevant shear zones only influence but do not intersect the excavation.
B.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation <50 m).	5	
C.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m).	2.5	
D.	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth),	7.5	
E.	Single shear zones in competent rock (clay-free) (depth of excavation <50 m).	5.0	
F.	Single shear zones in competent rock (clay-free) (depth of excavation >50 m).	2.5	
G.	Loose open joints, heavily jointed or "sugar cube" etc. (any depth).	5.0	

	(b) Competent rock, rock stress problems	σ_c/σ_1	σ_t/σ_1	*(SRF)*	Notes
H.	Low stress, near surface	>200	>13	2.5	(II) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to 0.8 σ_c and 0.8 σ_t. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to 0.6 σ_c and 0.6 σ_t, where: σ_c = unconfined compression strength, and σ_t = tensile strength (point load), and σ_1 and σ_3 are the major and minor principal stresses.
J.	Medium stress	200—10	13—0.66	1.0	
K.	High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10—5	0.66—.33	0.5—2	
L.	Mild rock burst (massive rock)	5—2.5	0.33—.16	5—10	
M.	Heavy rock burst (massive rock)	2.5	0.16	10—20	(III) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

	(c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure	
N.	Mild squeezing rock pressure	5—10
O.	Heavy squeezing rock pressure	10—20

	(d) Swelling rock: chemical swelling activity depending one presence of water	
P.	Mild swelling rock pressure	5—10
R.	Heavy swelling rock pressure	10—15

Table 3.5 (cont.) Classification of individual parameters used in the Q-system (after Barton et al 1977).

In relating Q to support requirements Barton (1976) defined the equivalent dimension (D_e) of an opening as the ratio of the span, diameter or wall height to a quantity called the excavation support ratio (ESR) ie

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio}}$$

The numerical value for ESR is related to the function of the excavation and the degree of safety required. The suggested ESR values are given in Table 3.6. In addition, support guidelines are given based on 38 categories of support according to different Q and ESR values. Figure 3.3 shows these relationships, with the numbered boxes representing the different support categories defined in Table 3.7.

	Excavation category.	ESR
A	Temporary mine opening.	3-5
B	Permanent mine openings, water tunnels for hydro-power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Table 3.6 Values for ESR suggested by Barton (1976).

3.5 The Geomechanics Classification

Bieniawski (1973, 1974, 1976 and 1979) has developed an engineering classification of jointed rock masses termed the Geomechanics Classification. It is based on five parameters: the strength of the intact rock material; drill core quality (RQD); spacing of joints; condition of joints and groundwater conditions. An importance rating is allocated to a range of the above parameters (Table 3.8a). When an RQD value is not available for a coal mine roof the rating for this parameter is determined from the measured discontinuity spacing using Figure 3.8b (BMC 1986). The sum of the five ratings is adjusted for joint orientations (Tables 3.8c and 3.8d), in situ stress ratio and method of excavation (Bieniawski, 1984) to give a Rock Mass Rating value (RMR) which can be related to rock classes defined in Tables 3.8e and 3.8f. For each rock class Bieniawski has specified rock mass strength parameters and stand-up time that a particular unsupported span takes to failure. The full relationship between unsupported span and the stand- up time is given in Figure 3.4.

Based principally on studies of cavability in asbestos mines and the

Support category	Conditional factors $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{Span}{ESR}$	Type of support	Note
1*	—	—	—	sb (utg)	—
2*	—	—	—	sb (utg)	—
3*	—	—	—	sb (utg)	—
4*	—	—	—	sb (utg)	—
5*	—	—	—	sb (utg)	—
6*	—	—	—	sb (utg)	—
7*	—	—	—	sb (utg)	—
8*	—	—	—	sb (utg)	—
9	≥20	—	—	sb (utg)	—
	<20	—	—	B (utg) 2.5—3 m	—
10	≥30	—	—	B (utg) 2—3 m	—
	<30	—	—	B (utg) 1.5—2 m + clm	—
11*	≥30	—	—	B (tg) 2—3 m	—
	<30	—	—	B (tg) 1.5—2 m + clm	—
12*	≥30	—	—	B (tg) 2—3 m	—
	<30	—	—	B (tg) 1.5—2 m + clm	—
13	≥10	≥1.5	—	sb (utg)	I
	≥10	<1.5	—	B (utg) 1.5—2 m	I
	<10	≥1.5	—	B (utg) 1.5—2 m	I
	<10	<1.5	—	B (utg) 1.5—2 m + S 2—3 cm	I
14	≥10	—	≥15 m	B (tg) 1.5—2 m + clm	I, II
	<10	—	≥15 m	B (tg) 1.5—2 m + S (mr) 5—10 cm	I, II
	—	—	<15 m	B (utg) 1.5—2 m + clm	I, III
15	>10	—	—	B (tg) 1.5—2 m + clm	I, II, IV
	≤10	—	—	B (tg) 1.5—2 m + S(mr) 5—10 cm	I, II, IV
16* See note XII	>15	—	—	B (tg) 1.5—2 m + clm	I, V, VI
	≤15	—	—	B (tg) 1.5—2 m + S (mr) 10—15 cm	I, V, VI

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned,

S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Table 3.7 Suggested support for categories identified by Barton et al (1977).

Support category	Conditional factors RQD/J_n	J_r/J_a	Span/ESR	Type of support	Note
17	>30	—	—	sb (utg)	I
	(≥10, ≤30)	—	—	B (utg) 1—1.5 m	I
	<10	—	≥6 m	B (utg) 1—1.5 m + S 2—3 cm	I
	<10	—	<6 m	S 2—3 cm	I
18	>5	—	≥10 m	B (tg) 1—1.5 m + clm	I, III
	>5	—	<10 m	B (utg) 1—1.5 m + clm	I
	≤5	—	≥10 m	B (tg) 1—1.5 m + S 2—3 cm	I, III
	≤5	—	<10 m	B (utg) 1—1.5 m + S 2—3 cm	I
19	—	—	≥20 m	B (tg) 1—2 m + S (mr) 10—15 cm	I, II, IV
	—	—	<20 m	B (tg) 1—1.5 m + S (mr) 5—10 cm	I, II
20* See note XII	—	—	≥35 m	B (tg) 1—2 m + S (mr) 20—25 cm	I, V, VI
	—	—	<35 m	B (tg) 1—2 m + S (mr) 10—20 cm	I, II, IV
21	≥12.5	≤0.75	—	B (utg) 1 m + S 2—3 cm	I
	<12.5	≤0.75	—	S 2.5—5 cm	I
	—	>0.75	—	B (utg) 1 m	I
22	(>10, <30)	>1.0	—	B (utg) 1 m + clm	I
	≤10	>1.0	—	S 2.5—7.5 cm	I
	<30	≤1.0	—	B (utg) 1 m + S (mr) 2.5—5 cm	I
	≥30	—	—	B (utg) 1 m	I
23	—	—	≥15 m	B (tg) 1—1.5 m + S (mr) 10—15 cm	I, II, IV, VII
	—	—	<15 m	B (utg) 1—1.5 m + S (mr) 5—10 cm	I
24* See note XII	—	—	≥30 m	B (tg) 1—1.5 m + S (mr) 15—30 cm	I, V, VI
	—	—	<30 m	B (tg) 1—1.5 m + S (mr) 10—15 cm	I, II, IV

Table 3.7 (cont.) Suggested support for categories identified by Barton et al (1977).

Support category	Conditional factors RQD/J_n	J_r/J_a	$Span/ESR$	Type of support	Note
25	>10	>0.5	—	B (utg) 1 m + mr or clm	I
	≤10	>0.5	—	B (utg) 1 m + S (mr) 5 cm	I
	—	≤0.5	—	B (tg) 1 m + S (mr) 5 cm	I
26	—	—	—	B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
	—	—	—	B (utg) 1 m + S 2.5—5 cm	I, IX
27	—	—	≥12 m	B (tg) 1 m + S (mr) 7.5—10 cm	I, IX
	—	—	<12 m	B (utg) 1 m + S (mr) 5—7.5 cm	I, IX
	—	—	>12 m	CCA 20—40 cm + B (tg) 1 m	VIII, X, XI
	—	—	<12 m	S (mr) 10—20 cm + B (tg) 1 m	VIII, X, XI
28*	—	—	≥30 m	B (tg) 1 m + S (mr) 30—40 cm	I, IV, V, IX
See note XII	—	—	(≥20 m, <30 m)	B (tg) 1 m + S (mr) 20—30 cm	I, II, IV, IX
	—	—	<20 m	B (tg) 1 m + S (mr) 15—20 cm	I, II, IX
	—	—	—	CCA (sr) 30—100 cm + B (tg) 1 m	IV, VIII, X, XI
29*	>5	>0.25	—	B (utg) 1 m + S 2—3 cm	—
	≤5	>0.25	—	B (utg) 1 m + S (mr) 5 cm	—
	—	≤0.25	—	B (tg) 1 m + S (mr) 5 cm	—
30	≥5	—	—	B (tg) 1 m + S 2.5—5 cm	IX
	<5	—	—	S (mr) 5—7.5 cm	IX
	—	—	—	B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
31	>4	—	—	B (tg) 1 m + S (mr) 5—12.5 cm	IX
	≤4, ≥1.5	—	—	S (mr) 7.5—25 cm	IX
	<1.5	—	—	CCA 20—40 cm + B (tg) 1 m	IX
	—	—	—	CCA (sr) 30—50 cm + B (tg) 1 m	VII, X, XI
32	—	—	≥20 m	B (tg) 1 m + S (mr) 40—60 cm	II, IV, IX
See note XII	—	—	<20 m	B (tg) 1 m + S (mr) 20—40 cm	III, IV, IX
	—	—	—	CCA (sr) 40—120 cm + B (tg) 1 m	IV, VIII, X, XI

Table 3.7 (cont.) Suggested support for categories identified by Barton et al (1977).

Support category	Conditional factors $\frac{RQD}{J_n}$	$\frac{J_r}{J_a}$	$\frac{Span}{ESR}$	Type of support	Note
33*	≥2	—	—	B (tg) 1 m + S (mr) 2.5—5 cm	IX
	<2	—	—	S (mr) 5—10 cm	IX
	—	—	—	S (mr) 7.5—15 cm	VIII, X
34	≥2	≥0.25	—	B (tg) 1 m + S (mr) 5—7.5 cm	IX
	<2	≥0.25	—	S (mr) 7.5—15 cm	IX
	—	<0.25	—	S (mr) 15—25 cm	IX
	—	—	—	CCA (sr) 20—60 cm + B (tg) 1 m	VIII, X XI
35	—	—	≥15 m	B (tg) 1 m + S (mr) 30—100 cm	II, IX
See note XII	—	—	≥15 m	CCA (sr) 60—200 cm + B (tg) 1 m	VIII, X, XI, II
	—	—	<15 m	B (tg) 1 m + S (mr) 20—75 cm	IX, III
	—	—	<15 m	CCA (sr) 40—150 cm + B (tg) 1 m	VIII, X, XI, III
36*	—	—	—	S (mr) 10—20 cm	IX
	—	—	—	S (mr) 10—20 cm + B (tg) 0.5—1.0 m	VIII, X, XI
37	—	—	—	S (mr) 20—60 cm	IX
	—	—	—	S (mr) 20—60 cm + B (tg) 0.5—1.0 m	VIII, X, XI
38	—	—	≥10 m	CCA (sr) 100—300 cm	IX
See note XIII	—	—	≥10 m	CCA (sr) 100—300 cm + B (tg) 1 m	VIII, X, II, XI
	—	—	<10 m	S (mr) 70—200 cm	IX
	—	—	<10 m	S (mr) 70—200 cm + B (tg) 1 m	VIII, X, III, XI

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted
(tg) = tensioned,

S = shotcrete
(mr) = mesh reinforced
clm = chain link mesh
CCA = cast concrete arch
(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Table 3.7 (cont.) Suggested support for categories identified by Barton et al (1977).

Supplementary notes by BARTON, LIEN *and* LUNDE

I. For cases of heavy bursting or "popping", tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.

II. Several bolt lengths often used in same excavation, i.e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i.e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2—4 m.

V. Several bolt lengths often used in same excavation, i.e. 6, 8 and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4—6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25—40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support is generally used as permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i.e. >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i.e. RQD/J_n <1.5, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete, but it may not be effective when RQD/J_n <1.5 or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rock-masses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 (SPAN/ESR >15 m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 (SPAN/ESR > 10 m only).

Supplementary notes by HOEK *and* BROWN (1980)

A. Chainlink mesh is sometimes used to catch small pieces of rock which can become loose with time. It should be attached to the rock at intervals of between 1 and 1.5 m and short grouted pins can be used between bolts. Galvanised chainlink mesh should be used where it is intended to be permanent, e.g. in an underground powerhouse.

B. Weldmesh, consisting of steel wires set on a square pattern and welded at each intersection, should be used for the reinforcement of shotcrete since it allows easy access of the shotcrete to the rock. Chainlink mesh should never be used for this purpose since the shotcrete cannot penetrate all the spaces between the wires and air pockets are formed with consequent rusting of the wire. When choosing weldmesh, it is important that the mesh can be handled by one or two men working from the top of a high-lift vehicle and hence the mesh should not be too heavy. Typically, 4.2 mm wires set at 100 mm intervals (designated 100 x 100 x 4.2 weldmesh) are used for reinforcing shotcrete.

Table 3.7 (cont.) Supplementary notes on suggested support for categories identified by Barton et al (1977).

	Parameter		Ranges of Values				
1	Strength of intact rock material	Point-load strength index	>10 MPa	4—10 MPa	2—4 MPa	1—2 MPa	For this low range — uniaxial compressive test is preferred
		Uniaxial compressive strength	>250 MPa	100—250 MPa	50—100 MPa	25—50 MPa	5—25 MPa | 1—5 MPa | <1 MPa
	Rating		15	12	7	4	2 | 1 | 0
2	Drill core quality *RQD*		90%—100%	75%—90%	50%—75%	25%—50%	<25%
	Rating		20	17	13	8	3
3	Spacing of discontinuities		>2 m	0.6—2 m	200—600 mm	60—200 mm	<60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities		Very rough surface Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1 mm Slightly weathered walls	Slightly rough surfaces Separation <1 mm Highly weathered walls	Slickensided surfaces OR Gouge <5 mm thick OR Separation 1—5 mm Continuous	Soft gouge >5 mm thick OR Separation >5 mm Continuous
	Rating		30	25	20	10	0
5	Ground water	Inflow per 10 m tunnel length	None	<10 litres/min	10—25 litres/min	25—125 litres/min	>125 litres/min
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	OR 0	OR 0.0—0.1	OR 0.1—0.2	OR 0.2—0.5	OR >0.5
		General conditions	OR Completely dry	OR Damp	OR Wet	OR Dripping	OR Flowing
	Rating		15	10	7	4	0

A. Classification parameters and their ratings.

Separation of bedding	None	Hairline	>1 mm	1.5 mm	>5 mm
Roughness of Surfaces	V Rough	Rough	Smooth	Slickensided	Slickensided
Weathering of Surfaces	Fresh, Hard	Slightly Weathered	Highly Weathered	Highly Weathered	Completely Weathered
Infilling (gouge)	None	None	Minor Clay	Stiff Clay gouge	Soft Clay gouge
Continuity	All bedding planes are continuous across entry				
Rating	30	25	20	10	0

B. Assessment of discontinuity conditions in coal mines.

Table 3.8 Geomechanics Classification of jointed rock masses (after Bieniawski 1979).

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0°—20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°—90°	Dip 20°—45°	Dip 45°—90°	Dip 20°—45°	Dip 45°—90°	Dip 20°—45°	
Very favourable	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

C. Effect of discontinuity strike and dip orientations in tunnelling.

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	—2	—5	—10	—12
	Foundations	0	—2	—7	—15	—25
	Slopes	0	—5	—25	—50	—60

D. Rating adjustment for joint orientations.

Rating	100←81	80←61	60←41	40←21	<20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

E. Rock mass classes determined from total ratings.

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2,5 m span	30 minutes for 1 m span
Cohesion of the rock mass	>400 kPa	300—400 kPa	200—300 kPa	100—200 kPa	<100 kPa
Friction angle of the rock mass	>45°	35°—45°	25°—35°	15°—25°	<15°

F. Meaning of rock mass classes.

Table 3.8 (cont.) Geomechanics Classification of jointed rock masses (after Bieniawski 1979).

stability of hardrock mine haulageways, Laubscher and Taylor (1976 and 1984) suggested some modifications to the Geomechanics Classification system whereby a number of adjustments to the RMR can be made. These include consideration of susceptibility to weathering (75-100% adjustment); in situ and mining induced stresses (60-120% adjustment); major faults and fractures (70-100% adjustment) and blasting damage (80-100% adjustment). It is recommended that the total RMR adjustment should not exceed 50%.

Bieniawski et al (1980) suggested that rock weatherability could be taken into account by multiplying the corresponding ratings of the strength of the rock material, RQD and condition of discontinuities by the ISRM slake durability index.

Charts for rock support selection have been derived for tunnelling applications (Table 3.9), hard rock mining (Figure 3.5), coal mining in India (Table 3.10) and main entries in US coal mines (Table 3.11). Unal (1983) developed the coal mine design charts; the following equations were used for calculations:

Mechanical point anchored bolts

(i) Rock-load height (h_t):

$$h_t = [(100 - RMR)/100]\ W$$

where W = roof width

(ii) Bolt length (L_b):

$$L_b = h_t/2$$

(iii) Bolt capacity (C_b):

$$C_b = L_y \text{ or } L_f$$

where L_y = yield load of steel
L_f = anchorage failure load, determined from pull out tests in the field or estimated from the values below

RMR	Anchorage failure load L_f, tonnes
100	12.7
90	10.9
80	10.0
70	9.1
60	8.2
50	7.3
40	6.4
30	5.5
20	4.6

Estimated anchorage failure-loads of mechanical bolts (after Unal 1983).

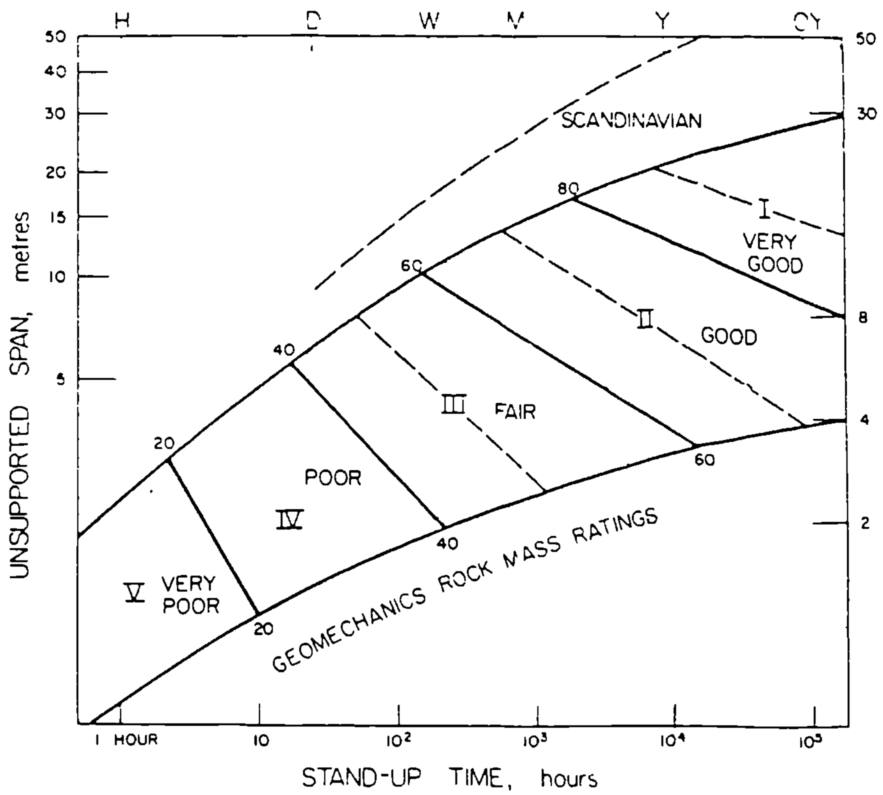


Figure 3.4 Relationship between stand-up time of an unsupported underground excavation and the Geomechanics Classification (after Bieniawski 1979).

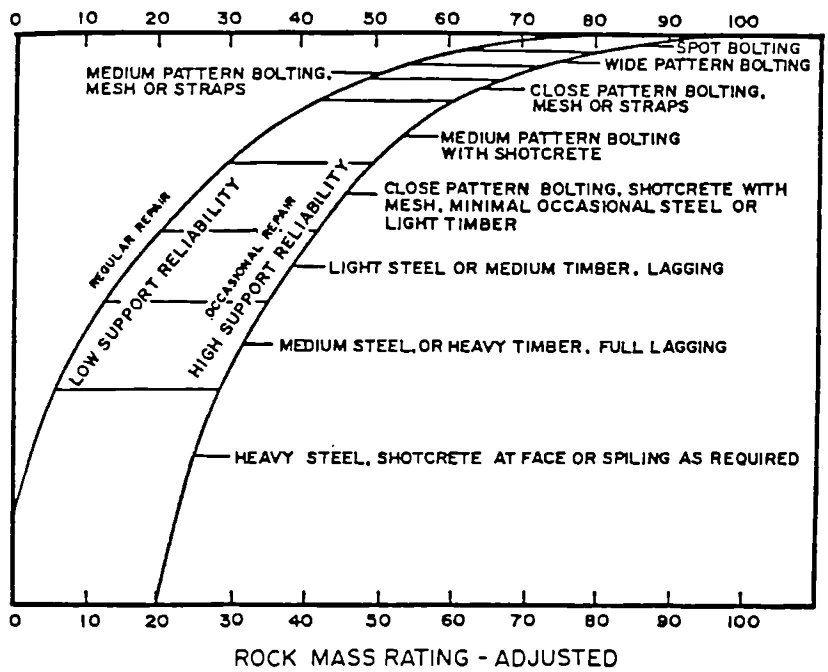


Figure 3.5 Support recommendations for hard rock mines based on the Geomechanics Classification (after Kendorski et al 1983).

SHAPE: HORSESHOE; WIDTH: 10 m; VERTICAL STRESS: BELOW 25 MPa; CONSTRUCTION: DRILLING AND BLASTING				
Rock mass class	Excavation	Support		
		Rockbolts (20 mm, dia., fully bonded)*	Shotcrete	Steel sets
Very good rock I RMR: 81—100	Full face 3 m advance.	Generally no support required except for occasional spot bolting.		
Good rock II RMR: 61—80	Full face 1.0—1.5 m advance. Complete support 20 m from face.	Locally bolts in crown 3 m long, spaced 2,5 m with occasional wire mesh.	50 mm in crown where required.	None
Fair rock III RMR: 41—60	Top heading and bench 1.5—3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 m—2 m in crown and walls with wire mesh in crown.	50—100 mm in crown and 30 mm in sides.	None
Poor rock IV RMR: 21—40	Top heading and bench 1.0—1.5 m advance in top heading. Install support concurrently with excavation 10 m from face.	Systematic bolts 4—5 m long, spaced 1—1.5 m in crown and walls with wire mesh.	100—150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
Very poor rock V RMR: <20	Multiple drifts. 0.5—1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5—6 m long, spaced 1—1.5 m in crown and walls with wire mesh. Bolt invert.	150—200 mm in crown, 150 mm in sides and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert.

Table 3.9 Excavation & support recommendations for rock tunnels based on the Geomechanics Classification (after Bieniawski 1979).

Permanent Galleries (life more than 10 years)

RMR	General supports recommended
0-20	Yielding steel arches of 28 kg/m section
20-30	Full column quick-setting grouted bolts with wire netting, W-straps & props; L - 1.8 m, Sb - Sr - 1.0 m OR Rigid steel arches; spacing 1.2 m.
30-40	Resin bolting with W-strap & steel props (100 mm dia , 5 mm wall thickness); L - 1.8 m, Sb - 1.0 m, Sr - 1.2 m. OR Brick walling (400 mm thick) with steel girders (200 X 100 mm section) at 1.2 m spacing and concrete sleepers.
40-50	Roof stitching supplemented with grouted bolts and wooden sleepers (of treated timber) L - 1.5 m, Sb - 1.0 m, Sr - 1.2 m. OR Rectangular steel supports (110 X 110 mm section) rigidly fixed at the ends with tie rods; timber lagging.
50-60	Full column cement grouted bolts; L - 1.5 m, Sb - Sr - 1.2 m. OR Steel props on either side of gallery at 1.2 m spacing.
60-80	Supports in disturbed zones wherever necessary (roof stitching and bolting).
80-100	Generally supports not required.

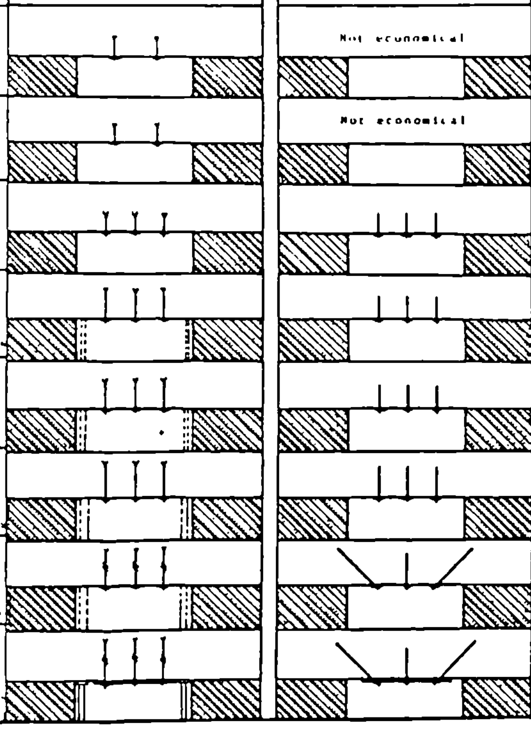
Temporary Galleries (life less than 10 years)

RMR	General supports recommended
0-20	Rigid steel arches; spacing 1.2 m.
20-30	Roof truss using quick-setting grout (spacing 1.0 m) and wooden props (150 mm dia.).
30-40	Rope truss system (spacing 1.2 m) with bolting; L - 1.8 m, Sb - 1.0 m, Sr - 1.2 m.
40-50	Roof stitching supplemented with rope dowelling and timber lagging; L - 1.5 m, Sb - 1.0 m, Sr - 1.2 m.
50-60	Roof stitching with a single rope dowel; L - 1.5 m.
60-80	Roof stitching in disturbed zones wherever necessary.
80-100	Generally no supports.

L - bolt length Sb - bolt spacing Sr - row spacing

Table 3.10 Support recommendations for Indian coal mine roadways (4.2 to 4.5 m wide) based on the Geomechanics Classification (after Singh 1986).

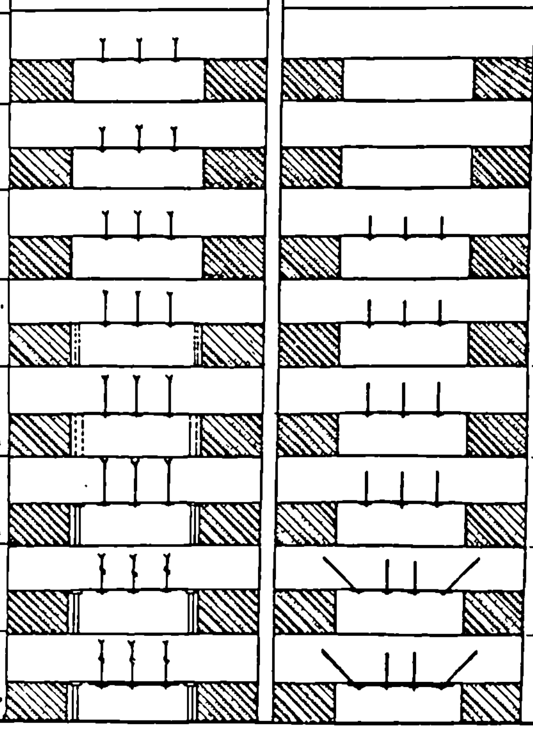
ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT H_T (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS	SUPPORT SPECIFICATIONS: RESIN BOLTS	ALTERNATE SUPPORT PATTERNS: MECHANICAL BOLTS/POSTS	ALTERNATE SUPPORT PATTERNS: RESIN BOLTS/POSTS	SPECIFICATIONS FOR POSTS
I VERY GOOD	90	1.6	L : 2.5' S : 6' x 5' G : 40 ϕ : 5/8" C : 6.2 tons			Not economical	
	80	3.2	L : 2.5' S : 6' x 5' G : 60 (40) ϕ : 3/4" C : 11 tons			Not economical	
II GOOD	70	4.8	L : 3.0' S : 4' x 4.5' G : 60 (40) ϕ : 3/4" C : 10 tons	L : 2.5' S : 4' x 5' G : 60 ϕ : 3/4" C : 12 tons			
	60	6.4	L : 4.0' S : 4' x 5' G : 60 ϕ : 5/8" C : 9 tons	L : 3.0' S : 4' x 5' G : 40 ϕ : 1" C : 15.8 tons			ϕ_p = 4.0" S_p = 10'
III FAIR	50	8.0	L : 4.0' S : 4' x 5' G : 40 ϕ : 3/4" C : 8 tons	L : 3.0' S : 4' x 4.5' G : 60 ϕ : 1" C : 18 tons			ϕ_p = 5.0" S_p = 10'
	40	9.6	L : 5.0' S : 4' x 5' G : 40 ϕ : 3/4" C : 7 tons	L : 4.0' S : 4' x 5' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 5.5" S_p = 10'
IV POOR	30	11.2	L : 6.0' S : 4' x 5' G : 40 ϕ : 5/8" C : 6 tons	L : 4.0' S : 4' x 4.5' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 4.5" S_p = 5'
	20	12.8	L : 7.0' S : 4' x 5' G : 40 ϕ : 5/8" C : 5 tons	L : 4.0' S : 4' x 4' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 5.0" S_p = 5'



A. 4.9 m wide roadways

L = bolt length
S = bolt spacing
G = grade of steel
ϕ = bolt diameter
c = bolt capacity
ϕ_p = post diameter
S_p = post spacing

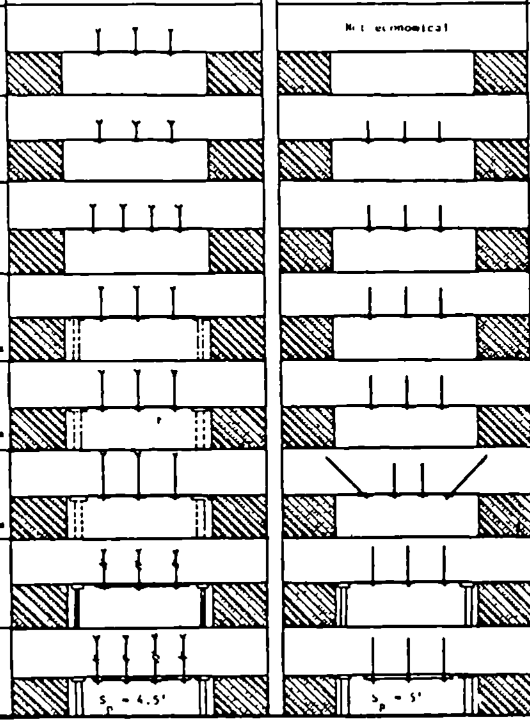
ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT H_T (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS	SUPPORT SPECIFICATIONS: RESIN BOLTS	ENTRY ALTERNATE SUPPORT PATTERNS: MECHANICAL BOLTS/POSTS	ENTRY ALTERNATE SUPPORT PATTERNS: RESIN BOLTS/POSTS	SPECIFICATIONS FOR POSTS
I VERY GOOD	90	1.8	L : 2.5' S : 5' x 5' G : 40 ϕ : 5/8" C : 6.2 tons				
	80	3.6	L : 2.5' S : 5' x 5' G : 60 (40) ϕ : 3/4" (7/8") C : 11 tons				
II GOOD	70	5.4	L : 3.0' S : 4.5' x 4' G : 60 ϕ : 3/4" C : 10 tons	L : 2.5' S : 4.5' x 5' G : 40 ϕ : 1" C : 15 tons			
	60	7.2	L : 4.0' S : 4.5' x 5' G : 60 ϕ : 5/8" C : 9 tons	L : 3.0' S : 4.5' x 4.5' G : 60 ϕ : 1" C : 18 tons			ϕ_p = 4.0" S_p = 7.5'
III FAIR	50	9.0	L : 5.0' S : 4.5' x 5' G : 40 ϕ : 3/4" C : 8 tons	L : 4.0' S : 4.5' x 5' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 5.5" S_p = 10.0'
	40	10.8	L : 6.0' S : 4.5' x 5' G : 40 ϕ : 3/4" C : 7 tons	L : 4.0' S : 4.5' x 4' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 5.0" S_p = 5.0'
IV POOR	30	12.6	L : 7.0' S : 4.5' x 4.5' G : 40 ϕ : 5/8" C : 6 tons	L : 4.0' S : 4' x 4' G : 60 ϕ : 1" C : 23.7 tons			ϕ_p = 5.0" S_p = 4.5'
	20	14.4	L : 8.0' S : 4.5' x 4' G : 40 ϕ : 5/8" C : 5 tons	L : 4.0' S : 4' x 4' G : 60 ϕ : 1-1/4" C : 28.74 tons			ϕ_p = 5.0" S_p = 4.0'



B. 5.5 m wide roadways.

Table 3.11 Support recommendations for US coal mine roadways based n the Geomechanics Classification (after Unal 1983).

ROOF ROCK CLASS	ROCK MASS RATING (RMR)	ROCK LOAD HEIGHT H_T (FT)	SUPPORT SPECIFICATIONS: MECHANICAL BOLTS	SUPPORT SPECIFICATIONS: RESIN BOLTS	ALTERNATE SUPPORT PATTERNS: MECHANICAL BOLTS/POSTS	ALTERNATE SUPPORT PATTERNS: RESIN BOLTS/POSTS	SPECIFICATIONS FOR POSTS
I VERY GOOD	90	2.0	L 2.5' S 5' x 5' G 40 φ . 5/8" C 6.2 tons			Not economical	
	80	4.0	L 2.5" S : 5' x 6.5' G : 60 (40) φ : 3/4" (7/8") C . 11 tons	L : 2.5' S : 5' x 5' G : 60 φ : 3/4" C · 12 tons			
II GOOD	70	5.0	L : 3.0' S : 4' x 4' G : 60 φ : 3/4" C : 10 tons	L : 3.0' S . 5' x 5' G : 60 φ : 3/4" C : 18 tons			
	60	8.0	L : 4.0' S : 5' x 5' G : 60 φ : 5/8" C : 9 tons	L : 4.0' S : 5' x 5' G : 60 φ : 1" C : 23.7 tons			ϕ_p = 5.5" S_p = 10'
III FAIR	50	10.0	L . 5.0' S : 5' x 5' G : 40 φ : 3/4" C : 8 tons	L . 4.0' S : 5' x 4' G : 60 φ : 1" C : 23.7 tons			ϕ_p = 6.5" S_p = 10'
	40	12.0	L : 6.0' S : 5' x 5' G : 40 φ : 3/4" C : 7 tons	L : 4.0' S : 4' x 4' G : 60 φ : 1" C : 23.7 tons			ϕ_p = 6.5" S_p = 7.5'
IV POOR	30	14.0	L : 7.0' S : 5' x 5' G : 40 φ : 5/8" C : 6 tons	L . 5' S : 5' x 5' G : 60 φ : 3/4" C : 12 tons			ϕ_p = 5.5" S_p = 5'
	20	16.0	L : 8.0' S : 4' x 4.5' G : 40 φ : 5/8" C : 5 tons	L : 5' S : 5' x 5' G : 60 φ : 3/4" C : 12 tons	S_p = 4.5'	S_p = 5'	ϕ_p = 6.0"



L = bolt length ϕ = bolt diameter ϕ_p = post diameter
S = bolt spacing c = bolt capacity S_p = post spacing
G = grade of steel

C. 7.0 m wide roadways.

Table 3.11 (cont.) Support recommendations for US coal mine roadways based on the Geomechanics Classification (after Unal 1983).

(iv) Bolt spacing (S_b):

$$S_b = C_b/(1.5\gamma h_t)$$

where γ = unit weight of the rock
This includes a factor of safety of 1.5 corresponding to a reduction of the anchorage capacity by 67%, equivalent to bolt tension $T = C_b/1.5$, to meet US mining regulations.

Full column resin anchored bolts

(i) Rock-load height (h_t): As for mechanical bolts

(ii) Bolt length (L_r):

$$L_r = \sqrt{(\gamma B^2 h_t)/(2\sigma_h)}$$

where σ_h = horizontal stress acting on the roof arch

(iii) Bolt capacity (C_b) and spacing (S_b): As for mechanical bolts

Bolts and posts

(i) Total rock pressure (P_t):

$$P_t = \gamma h_t$$

(ii) Pressure on posts (P_p):

$$P_p = \gamma h_p$$

where h_p = rock load height carried by posts

(iii) Pressure on bolts (P_b):

$$P_b = q h_b$$

where h_b = rock load to be carried by bolts

(iv) Rock load capacity by posts (C_p'):

$$C_p' = C_p/A_p$$

where C_p = load capacity of each post (tonnes)
A_p = area supported by each post

(v) Rock load height carried by bolts (h_b):

$$h_b = (\gamma h_t - C_p')/\gamma$$

3.6 CERCHAR Empirical Design Method

Members of Group Terrains from the Centre d'Etudes et Recherches des Charbonnages de France (CERCHAR) have applied many years of rock bolting experience in French mines, backed by theoretical work, to the development of an empirical method for the determination of bolting parameters for the support of mine roadways (Dejean et al 1976, 1980a and 1983).

Roof bolts are generally not recommended as the sole means of support in French coal mine roadways where:

(a) coal beds in the immediate 5 m of roof exceed a total thickness of 1 m;

(b) bed separation is liable to exceed 50 mm in the immediate 2 m of roof;

(c) the maximium thickness of differing strata in the immediate roof is less than 200 mm;

(d) the roadway is subjected to interaction from other workings.

The principal parameters of bolting patterns used in French mines are selected depending on various geotechnical criteria, stress field characteristics and time dependency. The main parameters can be derived from the matrix shown in Table 3.12.

A computer software has been written so that a more comprehensive design solution can be obtained than that achievable from the matrix approach. The programme, called PC Bolting, is 5000 lines of Fortran code with additional data files occupying several megabytes of hard disc space. The programme is written to run on the IBM XT or a compatible desk top computer. Calculations for a bolting pattern are performed using the following input data:

(a) nature and type of rock to be bolted and details of other strata surrounding the roadway;
(b) physical properties of the strata (uniaxial compressive strength, cohesion, friction angle, Young's Modulus, Poisson's Ratio and density);
(c) fracture pattern;
(d) weatherability;
(e) hydrological pattern, aggressiveness of water and general mining environment;
(f) position and geometry of roadway;
(g) natural and mining induced stresses;
(h) expected life time of roadway.

Owing to the complexity of the input data the programme requires an experienced operator with strata control expertise. A graphic display of the recommended initial bolting pattern is given in the form of a fundamental schematic diagram and dimensional drawings, and a table of bolt characteristics. The programme also gives information on roadway monitoring. An assessment of the applicability of the programme to rock bolt support system design for British coal mining conditions has been carried out at British Coal HQTD (Finch 1987). It appears that the design principles used may have some application in the UK. However the programme would benefit from a number of modifications and enhancements (particlarly concerning the geotechnical input data).

Type of strata	Result of stress field in strata	Bolting Parameters						Comments
		Effect of time	Type of Anchorage	Length	Diameter	Density	Lagging	
HOMOGENEOUS AND HARDLY FRACTURED	Deep and superficial stability	Stabilized Deformation						Support unnecessary
		Delayed Deformation	Light point	Short	Small	Low	Light	When high liability to impairment, the coating of shotcrete may be sufficient
	Deep stability and superficial instability	Stabilized Deformation	Any	Short	Small	Medium	Light	
		Delayed Deformation	Light or strong point	Short	Small	Medium	Light continuous	
	Deep and superficial instability	Stabilized Deformation	Strong point or full-column	Medium to long	Medium	High	Heavy	
		Delayed Deformation	Strong point	Medium to long	Medium	High	Heavy	
STRATIFIED AND HARDLY VERTICALLY FRACTURED	Deep and superficial stability	Stabilized Deformation						Support unnecessary
		Delayed Deformation	Light point	Short	Small	Low	Light	When high liability to impairment, the coating of shotcrete may be sufficient
	Deep stability and superficial instability	Stabilized Deformation	Any	Medium	Small	Medium	Light	
		Delayed Deformation	Light or strong point	Medium	Medium	Medium	Light continuous	
	Deep and superficial instability	Stabilized Deformation	Strong point or full-column	Long	Medium	High	Heavy	
		Delayed Deformation	Strong point	Long	Large	High	Heavy	
IRREGULAR, LENTICULAR OR FRACTURED IN SEVERAL DIRECTIONS	Deep and superficial stability	Stabilized Deformation	Point or full-column	Short	Small	Medium	Light continuous	Precautionary support
		Delayed Deformation	Point or full-column	Short	Small	Medium	Light continuous	
	Deep stability and superficial instability	Stabilized Deformation	Full-column	Medium	Medium	Medium	Light continuous	
		Delayed Deformation	Strong point or full-column	Medium	Medium	Medium	Heavy	
	Deep and superficial instability	Stabilized Deformation	Full-column	Long	Large	High	Heavy	
		Delayed Deformation	Strong point or full-column	Long	Large	High	Heavy	

Table 3.12 Matrix for choosing the main parameters of a rock bolting pattern (after Dejean & Raffoux 1980a).

3.7 US Army Corps Of Engineers Guidelines

The US Army Corps of Engineers (1980) recommend the empirical rules given in Table 3.13 to determine roof bolt parameters.

3.8 German Suitability Criteria

At present rock bolting is only considered to be feasible as the primary support in German coal mines if the criteria detailed below are met (Gotze 1977, 1981; Gotze et al 1982).

(a) If the roadway is to have an arch shape (in order to favour formation of an artificial bolt-reinforced arch surrounding the roadway), the seam thickness should be less than half the roadway height. Where seams are thicker or where they occur high up in the roadway cross section, a rectangular profile should be driven.

(b) There should be no coal seam greater than 0.2 m thick within 5 m above the roadway.

(c) The minimum thickness of individual roof stratum is 0.2 m to permit drilling and bolting operations. In addition the bolt holes should be drillable without caving from the borehole walls.

(d) In advancing longwall mining a high strength, early bearing roadside pack should be installed in order to replace the abutment for the bolted roadway support removed by coal extraction: thus keeping convergence and shearing stress on the bolts from strata displacement to a minimum.

(e) The predicted convergence (K_{ev}) should be less than the critical convergence (K_{crit}). During the 1970s West German rock mechanics engineers developed empirical formulae to predict the amount of convergence in gateroads within the German coalfields (Kammer 1977). The formulae could be used to calculate final convergence in arch-shaped roadways driven ahead of the longwall face and supported by late-bearing yielding arches. They were derived from operation observations and take into account, seam thickness, composition of the surrounding rock and type of roadside packs i.e.

For the Ruhr, Saar and Ibbenburen coalfields:

$$K_{EV} = -78 + 0.066D + 4.3M \times SV + 24.3\sqrt{GL}$$

Parameter	*Empirical rule*
Minimum length and maximum spacing	
Minimum length	Greatest of: (a) 2 x bolt spacing (b) 3 x thickness of critical and potentially unstable rock blocks (Note 1) (c) For elements above the springline: spans <6 m: 0.5 x span spans between 18 and 30 m: 0.25 x span spans between 6 and 18 m: interpolate between 3 and 4.5 m (d) For elements below the springline: height <18 m: as (c) above height >18 m: 0.2 x height
Maximum spacing	Least of: (a) 0.5 x bolt length (b) 1.5 x width of critical and potentially unstable rock blocks (Note 1) (c) 2.0 m (Note 2)
Minimum spacing	0.9 to 1.2 m
Minimum average confining pressure	
Minimum average confining pressure at yield point of elements (Note 3)	Greatest of: (a) Above springline: *either* pressure = vertical rock load of 0.2 x opening width or 40 kN/m^2 (b) Below springline: *either* pressure = vertical rock load of 0.1 x opening height or 40 kN/m^2 (c) At intersections: 2 x confining pressure determined above (Note 4)

Notes:

1. Where joint spacing is close and span relatively large, the superposition of two reinforcement patterns may be appropriate (e.g. long heavy elements on wide centres to support the span, and shorter, lighter bolts on closer centres to stabilise the surface against ravelling).
2. Greater spacing than 2.0 m makes attachment of surface support elements (e.g. weldmesh or chain link mesh) difficult.
3. Assuming the elements behave in a ductile manner.
4. This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections and rock blocks are free to move toward both openings.

Table 3.13 Typical empirical design recommendations (after US Corps of Engineers, 1980 and Douglas & Arthur, 1983).

and for the Aachen coalfield:

$$K_{EV} = -193 + 0.062D + 14.2M \times SV + 80.7\sqrt{GL}$$

where K_{EV} = First use convergence, single unit working, driven ahead of face line, supported by yielding arches
D = Depth (m)
M = Seam thickness (m)
SV = Pack index
GL = Floorstone index

Floorstone index:
1 = sandstone
2 = sandy shale
3 = mudstone
4 = seatearth
5 = coal
6 = mudstone, seatearth and coal alternating

Pack index:
1 = rigid pack
2 = timber chocks
3 = no packs

Corrective factors (Figure 3.6) have been established for the Ruhr, Saar and Ibbenburen coalfields which enable final convergence to be predicted in roadways driven in-line, behind face and for retreat (Kammer 1980). An additional factor can be applied to take into account the convergence reducing effects of early bearing rock bolts compared to late bearing yielding supports (Figure 3.7).

According to Gotze (1981) roadways with rock bolts as the sole support become unstable once the critical convergence is exceeded. This is because excessive marginal dilation of the surrounding rock will overload the bolts. Critical convergence is calculated using the empirical formula:

$$K_{crit} = \frac{d_M.L}{H[1-S/K-(W-L)(p.d_M/100L)]}$$

where K_{crit} = Critical convergence
d_M = Expansion of surrounding bolted rock
L = Embedded length of bolt
H = Roadway height
W = Roadway width
S/K = Proportion of floor heave in total convergence
p = Magnitude which expresses the position of slip planes and adjacent seams in the roof rock

The proportion of floor heave in the total convergence is affected by the floor stone index (GL) and the roofstone index (GH) (Figure 3.8).

Roofstone index:
1 = sandstone
2 = sandy shale
3 = mudstone

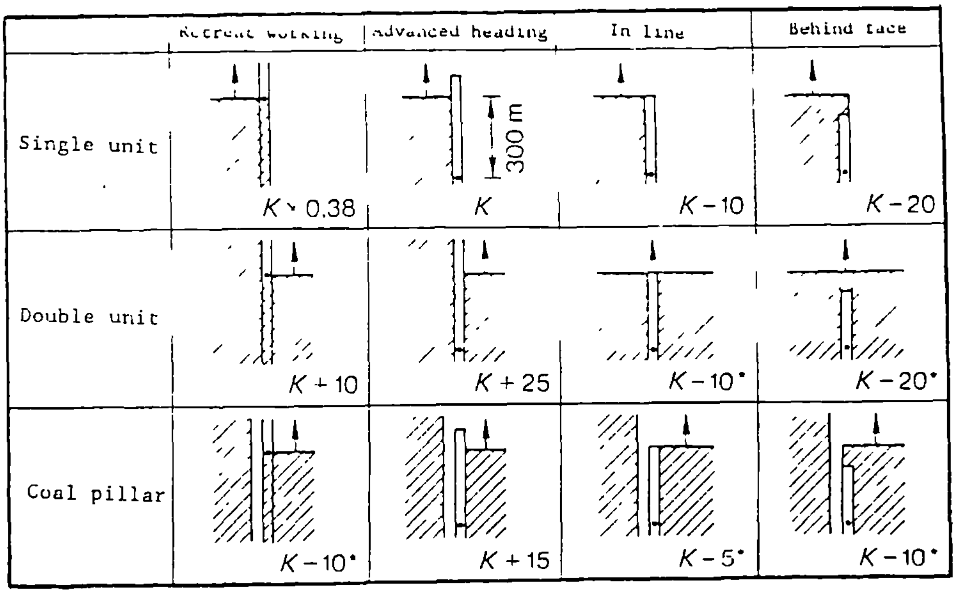


Figure 3.6 Corrective factors to calculate convergence in relation to the type of roadway (after Kammer 1980).

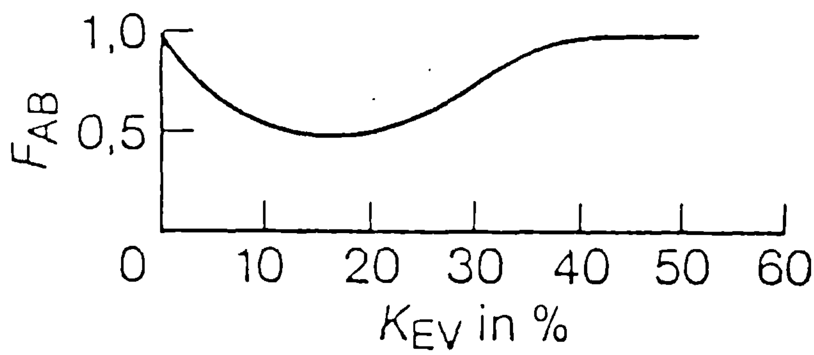


Figure 3.7 Determination of the beneficial effect of bolting F_{AB} in arch shaped gateroads (after Gotze et al 1982).

Magnitude p:

1 = if the surrounding roof rock does not contain any slips or coal beds

.5 = if the roof contains slips or coal beds < 0.2 m thick

0 = if a slip or coal bed < 0.2 m occurs in bolted rock in the crown

-1 = if a seam > 0.2 occurs in bolted surrounding rock at a distance equal to the width of the road

The value of K_{crit} derived generally varies between 8 and 20% of the initial height, depending on the nature of the surrounding strata, the size and shape of the roadway and the length of the rock bolts.

Further operational roadway surveillance (Nyga 1987) has established a relationship between convergence after first use, and convergence at or behind a second face for roadways supported by yielding arches and roadways supported by rock bolts alone.

For yielding arches:

$$K_{ZR} = 1.3K_E + 10\%$$

$$K_{ZV} = 1.3K_E + 30\%$$

where K_E = total convergence after first utilisation (% of initial ht)
K_{ZR} = convergence in road with second retreat working at 10 m behind the face (% of initial height).
K_{ZV} = convergence in road at 300 m behind the second face (%)

NB The roadways have to be kept open up to 10 m behind the face for salvaging purposes.

For rock bolt support:

$$K_{ZR} = 1.2K_E + 5\%$$

A formula to calculate K_{ZV} in bolted roadways has not yet been derived due to insufficient data.

These five criteria are highly restrictive as to the roadways that are suitable for rock bolting as the sole support. It is envisaged that as further practical experience is gained and rock bolting techniques in West Germany are developed, these constraints will probably be relaxed.

CHAPTER 4
ANALYTICAL METHODS OF DESIGN

4.1 Background

Quantitative analysis of mine openings is possible to a high degree, although engineering design cannot be reliably performed by its use. One reason is the variability of basic rock mechanical parameters which cause a large overall inaccuracy in calculations. Another reason is the complicated condition of natural rock structures, which form an obstacle to the application of simple and readily calculable design. Results obtained using the analytical methods discussed below should therefore be used as a guide only and in combination with other design approaches.

4.2 Analysis Of The Suspension Effect

Simple mathematical analysis to obtain various roof bolt parameters can be carried out based on the suspension concept. If the volume of rock to be supported is a well defined beam-like layer, the following simple equation may be applied to solve for either bolt load, and hence required bolt strength, or bolt spacing (Obert and Duvall 1967).

$$W_b = \frac{\gamma t B L}{(n_1 + 1)(n_2 + 1)}$$

where W_b = load per bolt
B = width unstable layer to be supported
L = length of roadway under consideration
n_1 = number of bolts included within length (L)
n_2 = number of bolts included within width (B)
γ = unit weight of rock
t = thickness of unstable roof layer

This equation is only valid if the mass of the loose rock layer is completely suspended by the bolts. Where the unstable layer extends across the entire roadway width (which is generally the case) a portion of the mass of the unstable layer is supported by the roadway ribs. Hence this equation represents the upper limit of the suspension load for each bolt. The effect can be compensated for by an approximation, regarding each rib as contributing the equivalent of one half the load carried by each bolt. The equation also neglects the increase in the load carried by the bolts caused by in situ stresses.

The thickness of the unstable layer (t) should generally be taken as the vertical distance to the highest level of significant bed separation. This can be determined by installing borehole extensometers, examining rock bolt holes with a borescope or by studying the height of roof falls.

4.3 Analysis Of The Beam Building Effect

There is no good analytical design theory for the beam building mechanism and it is difficult to isolate as the sole effect. However, Panek (1956a, 1956b, 1956c, 1964), making several simplifying assumptions, has investigated the action of point anchored tensioned roof bolts in increasing friction between individual roof layers.

A roadway roof can be regarded as a beam clamped above the ribsides. According to classical beam theory, the maximium bending strain (ξ_{max}) in a clamped beam occurs at the clamped ends and is given by:

$$\xi_{max} = \gamma B^2/2Et$$

where γ = unit weight of beam material
B = width of beam
E = Young's Modulus of material
t = lamina thickness

Using regression analysis of data obtained by centrifugal testing of mine roof beam models, Panek (1962) found that the relationship between the decrease in maximum bending strain ($\Delta\xi_f$) due to the friction effect from bolting and the maximum bending strain of the unbolted strata (ξ_{nfs}) can be expressed by

$$\Delta\xi_f/\xi_{nfs} = -0.375\mu(cB)^{-0.5}\,[NP(\iota/t_{avg}-1)/\gamma_{avg}]^{0.33}$$

where ξ_{nfs} = maximum bending strain with no friction or suspension
$\Delta\xi_f = \xi_f - \xi_{nfs}$
= change in maximum bending strain due to friction effect
μ = coefficient of friction between bedding planes
c = spacing between adjacent rows of bolts
B = roof span
N = number of bolts per row
P = bolt tension
ι = bolt length
t_{avg} = average thickness of bolted roof
γ_{avg} = average unit weight of bolted roof rocks

The reinforcement factor (RF), due to the friction effect is then defined as follows

$$RF = \frac{\text{Max. bending strain, unbolted roof}}{\text{Max. bending strain, bolted roof}} = \frac{\xi_{nfs}}{\xi_f} = \frac{1}{1+(\Delta\xi_f/\xi_{nfs})}$$

A nomogram was derived based on these equations from which the reinforcement factor from a point anchored bolted roof can be determined (Figure 4.1). The nomogram is based on $\mu = 0.7$ and $\gamma_{avg} = 2.49$ g/cm³ in

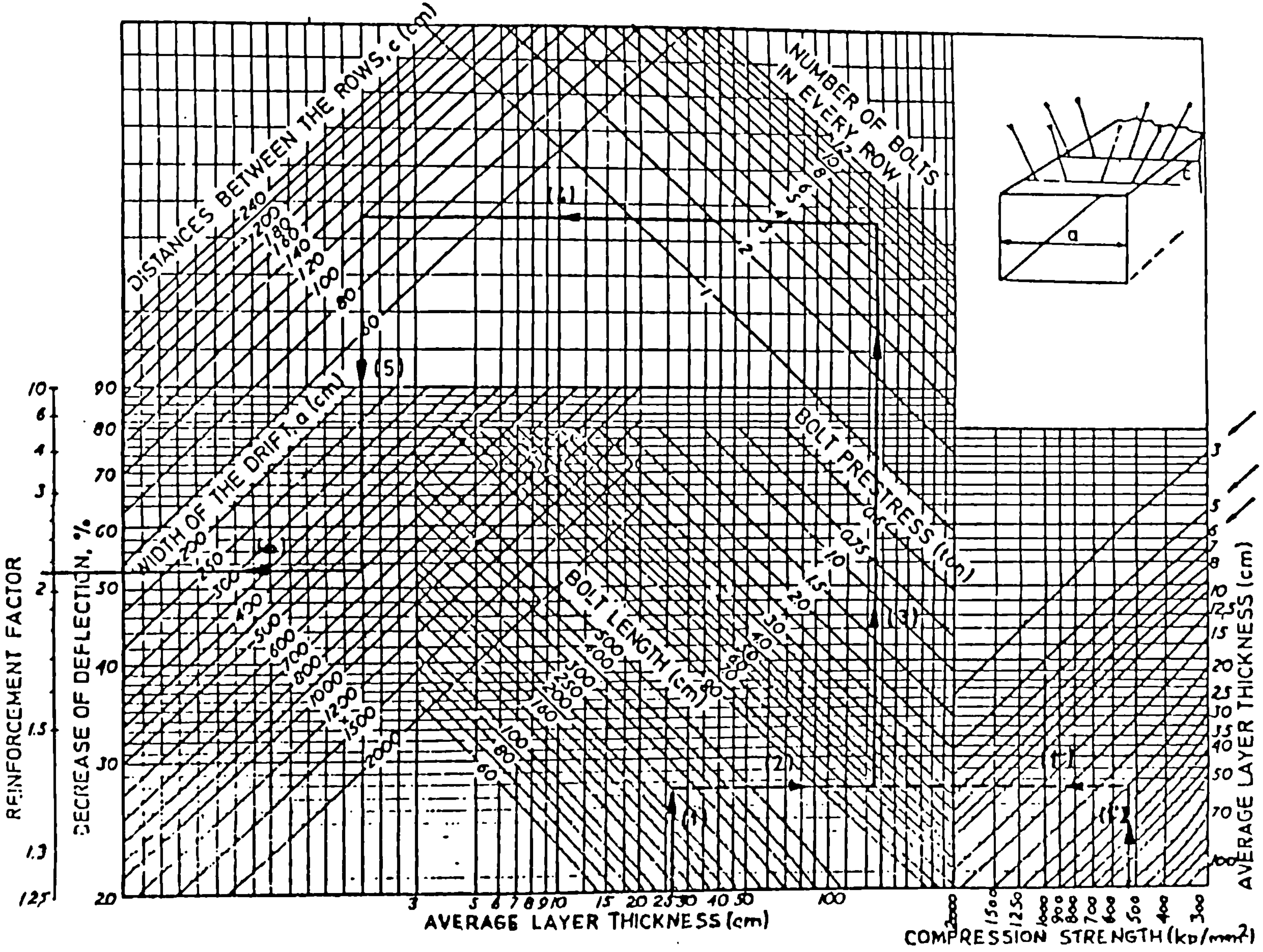


Figure 4.1 Nomogram for rock bolting calculations (with pre-tensioned bolts) in layered roof (after Panek 1964).

addition, the material properties (e.g. Young's Modulus) of each roof layer is taken as being the same.

Using a combined theoretical and experimental (beam model) approach, Panek (1962) investigated the degree of reinforcement produced in laminated roof rock by the combined effects of friction and suspension. A certain degree of support by suspension results if all the strata layers within the bolted zone are not of equal flexural rigidity (i.e. either or both the thickness and Young's Modulus varies between layers). Suspension here applies to the support of more flexible strata by stiffer strata. Panek found that suspension had a multiplicative, though small, effect on reinforcement.

In Panek's proposed design method for a roof bolting system, the maximum bending stress, instead of the total horizontal stress (the in situ horizontal stress and the bending stress combined), was used. This approach is valid only when the horizontal stress is nonexistent or very small (Wright 1974). In practice, this condition is very unlikely to be encountered in a British coal mine.

Fairhurst and Singh (1974) present a theory for the analysis of beam building using a two dimensional plate buckling criterion. The effectiveness is really a measure of the moment of inertia (second moment of area) of the bolted beam. This measure is dependent on several variables, e.g. inter layer friction, bolt density and shear stiffness of bolt-grout-rock combinations. Snyder and Krohn (1982) consider that theory was unsuitable for predicting a value for soft rocks.

4.4 Analysis Of The Stability Of Key Blocks And Arching Action

4.4.1 Key Block Bolting

In shallow excavations where stresses in the rock mass are considerably less than the intact strength of the rock material, failure may occur due to the sliding or falling of blocks of rock bounded by discontinuities. Generally the discontinuity patterns in coal measures strata are too unpredictable for even approximate mathematical analysis, although in certain circumstances, where fracture patterns are well defined, it may be possible to estimate the mass of potentially unstable key blocks and therefore determine bolting parameters.

The necessary design information is obtained from a discontinuity survey carried out using the methods described by the ISRM (1981). Plotting the data on a stereonet, marking the pole to each plane (Phillips 1971) will reveal joint sets. Then by plotting the joint sets as planes and the roadway geometry on a stereonet it is possible to determine which joint sets interact to produce discrete rock blocks which may be capable of falling or sliding into the roadway. A rock bolt system can then be designed to maintain the stability of the blocks (Hoek and Brown 1982; Shelton 1980).

The "Block Theory" approach used by Shi and Goodman (1983) has shown that the required support for key blocks is significantly reduced if there are initial tangential compressive stresses around the opening. The development of Block Theory has highlighted the need to take into account the discrete nature of the bolted rock masses in three dimensions. In its present form this theory appears ideally suited to low stress conditions involving quasi-static loading.

4.4.2 Analysis Of Yield Zones

In the case of roadways where the magnitude of the redistributed geostatic stresses exceeds the strength of the intact rock material, failure of the rock will occur resulting in the formation of a "yield zone" of fractured strata adjacent to the roadway. This process is one of the primary mechanisms contributing to the collapse of rock blocks into an excavation.

Studies by Wilson (1977, 1980) of circular tunnels in unstratified soft rock, have shown that the development of the yield zone can be prevented or its extent restricted by the application of sufficient support pressure. The graphs in Figure 4.2 can be used to predict the extent of the yield zone in a 4 m diameter circular tunnel at a range of depths with various support pressures (p) in different rock types.

The tunnel stability prediction method devised by Wilson (1980) (specifically for circular tunnels) has been shown to give similar orders of anticipated diametric closure at sites with an arch shaped profile (Nottingham University 1983). The width of the yield zone will not be greatly affected by the shape of the roadway providing the width is approximately equal to the height.

Wilson's equation linking roadway closure to the lining strength and rock properties (other than coal) is:

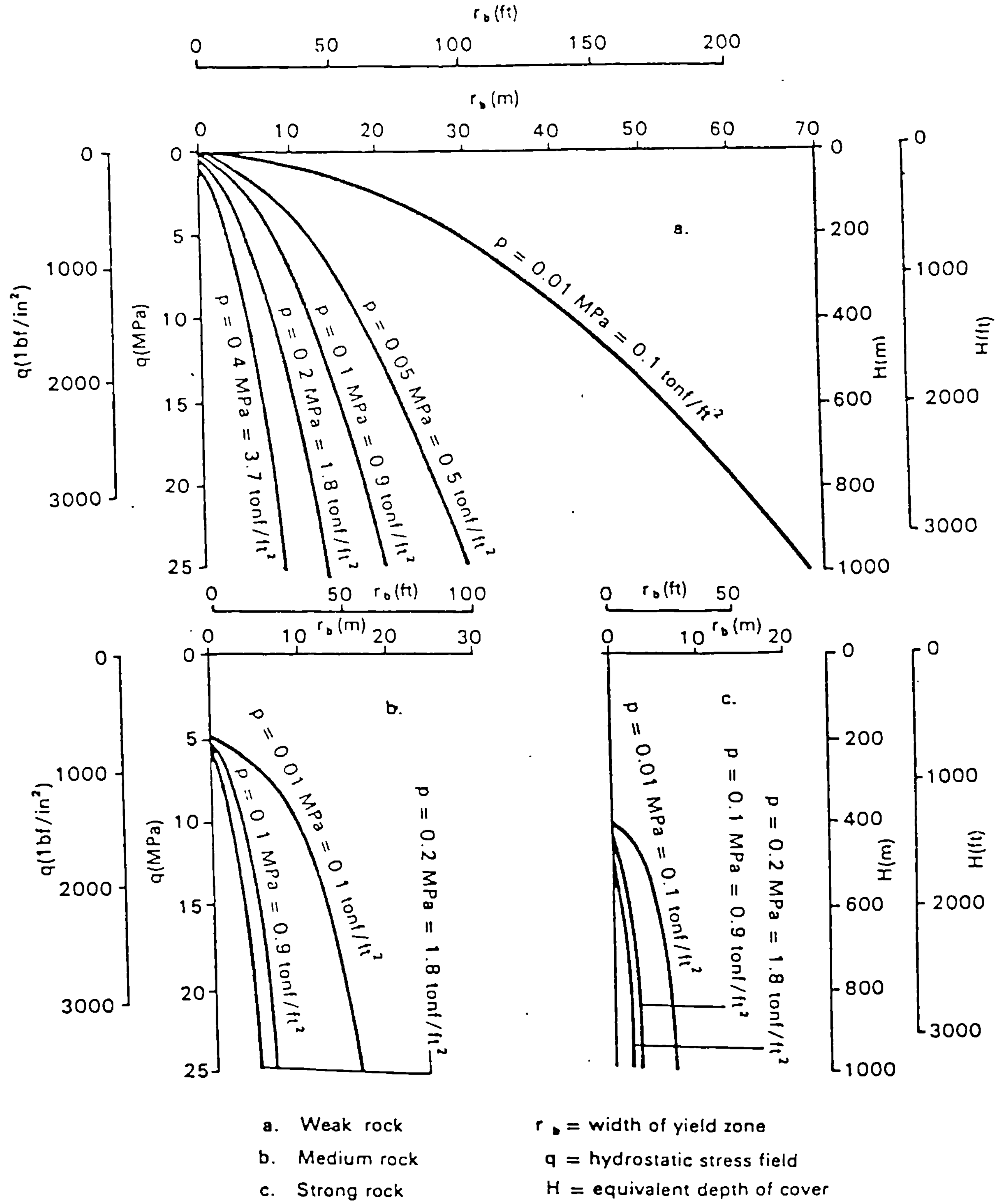


Figure 4.2 Effect of roadway support on yield zone (after Wilson 1977).

$$c = d\,\frac{(1+\nu)}{E}\left[\frac{(k-1)q + \frac{\sigma}{f}}{(k+1)}\right]\left[\frac{2q - \frac{\sigma}{f}}{(p+p')(k+1)}\right]^{(2-\varepsilon)(k\;\;1)}$$

$$\approx d\,\frac{4}{\sigma}\left[\frac{(k-1)q + \frac{\sigma}{f}}{(k+1)}\right]\left[\frac{2q - \frac{\sigma}{f}}{(p+0.1)(k+1)}\right]^{2\cdot 2(k\;\;1)} \times 10^{3}$$

Where c = diametric closure
d = driven diameter
ν = Poisson's ratio
E = modulus of elasticity
k = triaxial stress factor
q = cover load
σ = laboratory unconfined compressive strength of rock
p = lining resistance
f = a factor relating laboratory strength to in situ strength
p' = augmentation of the lining resistance brought about by the cohesion of the yielded rock
ϵ = expansion factor

Wilson's hypothesis for the prediction of yield zone extent and the magnitude of closure can be used in rock bolt system design as a check for unacceptable yielding within a roadway, or for identifying the need to modify the bolt parameters, to ensure that they are located in stable rock and capable of preventing excessive yield.

4.4.3 Theory Of Jointed Bodies

In West Germany, proposed roadways with rock bolting as the sole means of support require a calculated proof of stability before approval can be granted by the German Inspectorate of Mines. The Theory of Jointed Bodies (Kluftkorpertheorie) is used to provide this proof (Gotze 1977). The theory has been developed from underground and physical model tests. It attempts to define the dimensions (in a plane perpendicular to the roadway axis only) of the largest rock fragment in the surrounding strata that is able to be held in position by friction and could collapse into the roadway. Underground investigations have established that roof falls in German coal measures strata generally extend into the roof to a distance no higher than half the roadway width. According to German Inspectorate regulations, the largest rock blocks likely to break away from the roof are trapezoidal in arch shaped roadways and rectangular in rectangular shaped roadways (Figure 4.3).

The bolting pattern is designed on the principle of suspension and nailing of rock fragments so that full column grouted bolts have a bonded length of at least 0.5 m outside the unstable areas and running

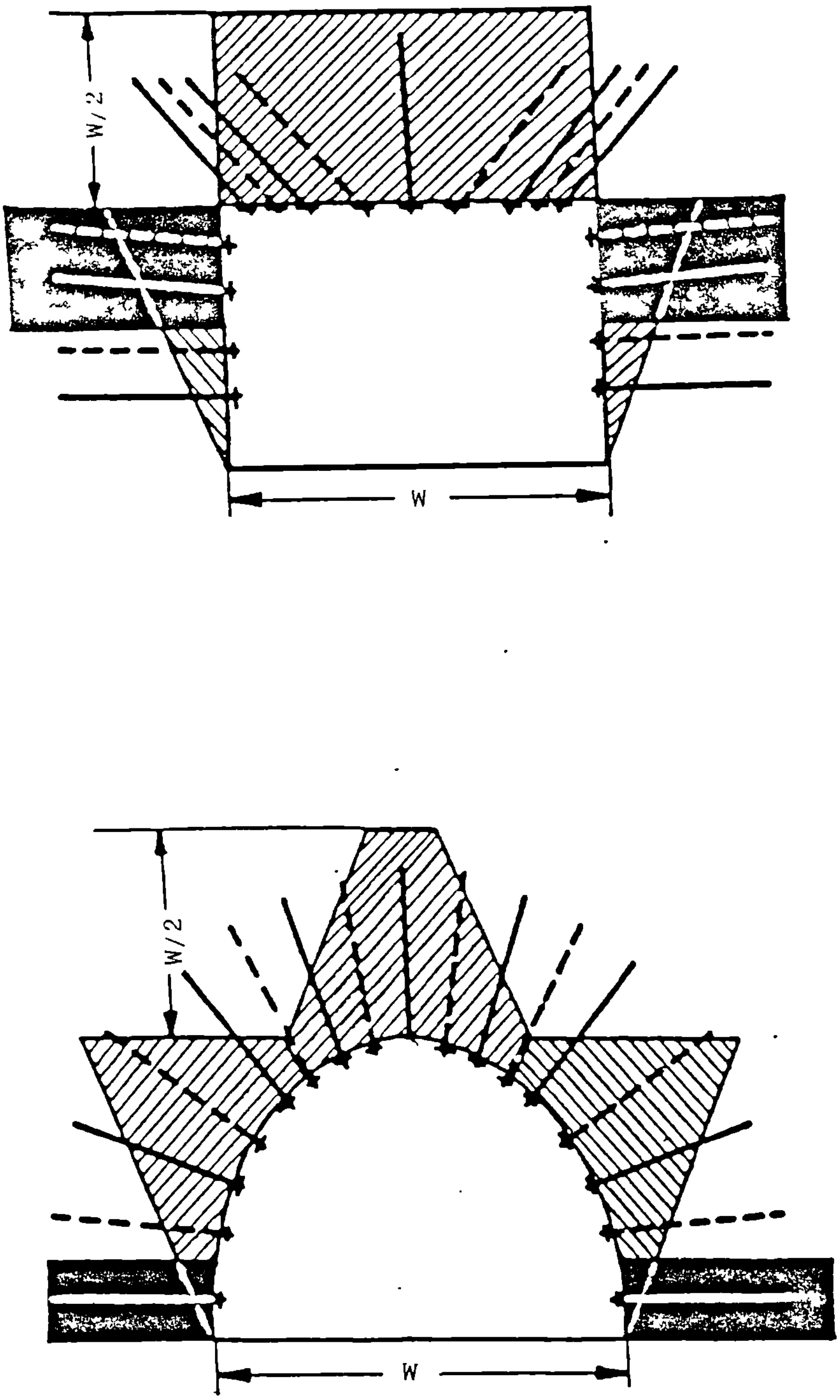


Figure 4.3 Rock bolt pattern design based on the theory of jointed bodies (after Gotze 1981).

at least 0.6 m through it. The bolt density is sufficient to secure smaller fissured blocks and the lagging must be capable of supporting the smallest of these additional fragments.

4.4.4 Voussoir Rock Arch

A Voussoir rock arch is a masonary arch composed of segments and has been used in civil engineering works since Roman times. As the segments or blocks lean against each other they exert a sideways compressive force which prevents blocks sliding out. They have been observed in the field (Gerdeen et al 1977) where lateral compressive stresses are low (eg in shallow workings, or near a hillside or under a valley). If the blocks are not of sufficient thickness to form a stable arch, then roof bolts may supply sufficient reinforcement.

Cox (1974) provides some analysis of the stability of arch for the case of no in situ horizontal stresses. The rock arch is assumed to be made up of jointed rock with little or no tensile strength. As the beam sags, a crack opens at the middle and a horizontal thrust (H) acts over one-quarter of the beam (Figure 4.4a and b). The dashed line in Figure 4.4b indicates the boundary of the rock arch. Below this boundary tensile stresses can develop in the rock and thus the lower rock must be suspended by bolts. It is assumed that the rock arch thickness (t) is equal to the length of the roof bolts used to create and reinforce it. If the roof bolt reinforced arch is to remain stable the roof bolt length (ι) must satisfy the following conditions:

(i) to prevent compressive failure at the ribs

$$\iota > (\gamma h L^2/216 C_p)^{0.5}$$

(ii) to prevent shear failure at the abutments

$$\iota > \gamma h L/72 C_s$$

(iii) to prevent slip along vertical slips at the abutments

$$\iota < \mu L/3$$

where ι = roof bolt length (feet)
γ = unit weight of rock (lb/ft^3)
h = height of rock load (feet)
L = opening width
C_p = compressive strength of rock (psi)
C_s = shear strength of rock (psi)
μ = frictional coefficient along the vertical fracture plane

N.B. It is assumed that the modulus of elasticity of the rock is large enough to prevent significant changes in geometry due to roof sag.

Cox drew the following conclusions from this investigation:

(a) Compressive failures are unlikely because relatively short bolts are needed even for low compressive stress (this precludes the existence of in situ lateral stresses).

(b) Shear failures of rock are possible if roof bolts are too short.

(c) Slip failures along vertical planes are always a potential for typical roof bolt patterns.

(d) Increasing the bolt length does not always increase roof stability.

(e) Decreasing opening width does not always increase roof stability.

(f) Some roofs with a combination of vertical jointing and weak shear strength cannot be supported by roof bolts alone.

4.4.5 Rock Mass Confinement Approach

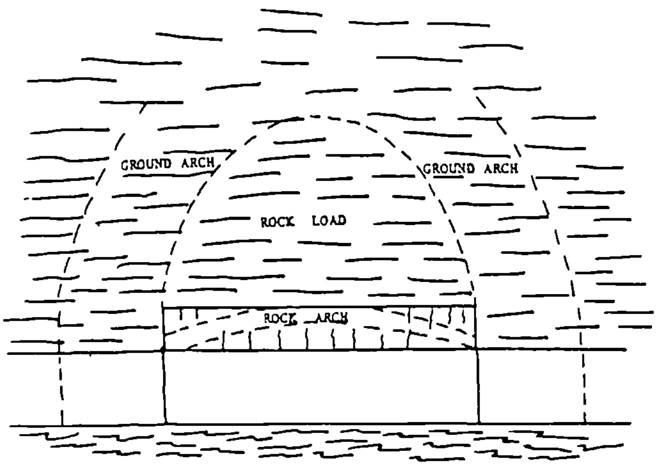
Lang (1958) performed a series of model experiments using fine (< 5 mm) crushed rock, plastic rods, or marbles to simulate fractured roof rock and scale sized tensioned rock bolts. The tests resulted in derivation of the following relationship between the clear space (S) between bolt bearing plates and mean size of the supported fragments (M):

$$F = S/M < 3.0$$

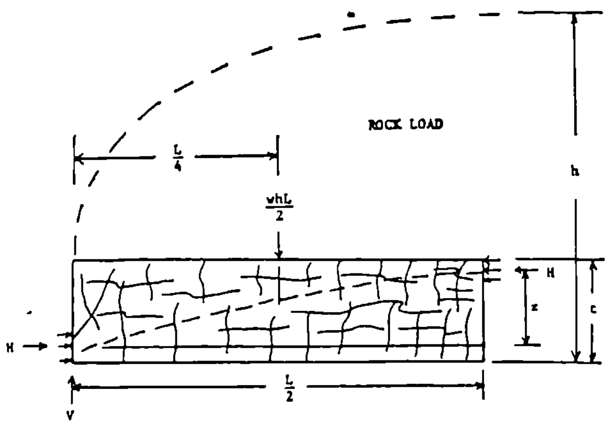
Under these conditions, even a mass of glass marbles could be stabilized. At $F = 4.0$, the glass marble mass would always collapse but crushed rock would often be supported.

Based on results from photoelastic studies as well as the simple physical models Lang (1961, 1972) found that tensioned bolts spaced closely enough produced a zone of uniform compression within the rock mass (Figure 4.5). In order for this bolt zone to develop, the bolt length must be at least twice the bolt spacing and the spacing between the bolts less than seven times the average fragment size. This zone of uniform compression is based on an angle of dispersion equal to 45^0 and has a thickness approximately equal to the bolt length minus the spacing. Thus it is assumed that each tensioned bolt produces a zone of influence defined by a square with the bolt itself defining one of the diagonals. The zones overlap in such a way that each zone finishes at the adjacent bolt. Once this zone is created with induced compressive stresses remaining lower than the permissible compressive stresses of the rock mass, the rock behaves as a stable hidden beam or arch.

Bischoff and Smart (1975) proposed a concept whereby rock bolt reinforcement creates a uniform additional pressure on rock that is equivalent to that taken by steel ribs. Daws (1977, 1983, 1986a, 1986b) has developed and applied this hypothesis in UK coal mine roadway



a.



b.

Figure 4.4 Concept of ground arching and the rock arch (after Cox 1977).

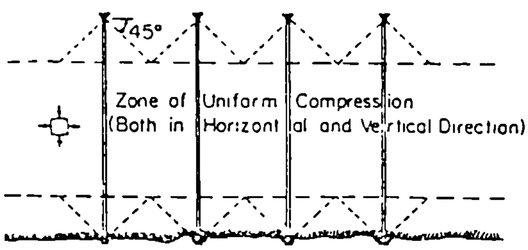


Figure 4.5 Development of uniform compression zone by use of tensioned bolts (after Lang 1961).

design.

The method seeks to create a reinforced rock arch capable of supporting itself and a zone of broken ground above. There are several methods for estimating the height of broken ground that will develop above a roadway; some of the more commonly used expressions are given in Table 4.1. From these formulae the expected dead weight loading can be calculated. The active load (P_a) due to volume expansion of broken material can be considered as

$$P_a = P \left[1 + \frac{1 - \sin\phi}{1 + \sin\phi} \right] \qquad 1.$$

where P_a = active load
P = rock load on support system
ϕ = angle of internal friction

The increase in confinement in the reinforced rock arch produces a triaxial stress state. If rock is confined with a stress σ_3 in the minor axis, then its loading capacity in the major axis, σ_1 is increased, as occurs during triaxial testing of rock specimens. Figure 4.6 is a Mohr diagram which illustrates that an increase in the confining stress by a value (σ_3 - β_3) will result in the failure stress in the major axis increasing by:

$$\sigma_1 - \beta_1 = \tan^2 (45 + \phi/2)\ \sigma_3 - \beta_3 \qquad 2.$$

The increase in rock mass confinement provided by bolting (σ_3 - β_3) may be assumed to be the elastic yield load of the bolt (U) divided by the roof area over which the bolt acts. For a square pattern of bolts this area is taken as the bolt spacing squared (S^2). So that:

$$\sigma_3 - \beta_3 = U/S^2 \qquad 3.$$

The load supported by the bolting system (P_a) can be taken as:

$$P_a = (\sigma_1 - \beta_1)t \qquad 4.$$

where t = effective thickness of rock arch
= L - S
L = bolt length
S = bolt spacing

The effective thickness of the reinforced rock arch can be determined by applying the Lang (1972) approach detailed above (i.e. that the zone of uniform compression is equal to the length of the bolt minus the bolt spacing).

AUTHOR	ASSUMPTIONS	HEIGHT OF ENVELOPE	AREA OF BROKEN GROUND/m LENGTH (m^2)	DEAD LOAD ON SUPPORT Tonnes/m	ACTIVE SUPPORT LOAD Tonnes/m
Airey (1974)	Broken envelope Triangular	$H/2\cot\phi = 5.20$ m	$\frac{H^2}{4}\cot\phi = 14.30$	$\frac{H^2}{4}\cot\phi\gamma = 37.18$	50.60
Dejean & Raffoux (1980a)	Parabolic Envelope	$H/..\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right) = 2.20$m	$\frac{H^2}{6}\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right) = 8.31$	$\frac{H^2}{6}\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)\gamma$ $= 21.62$	29.40
Whittaker & Singh (1980)	Broken envelope is arched shaped. Can approx. to a rectangle.	L 4.3	L.H 23.65	$L.H.\gamma.1$ $= 61.5$	78.7
Daws (1980a)	Triangular envelope	1.5L	$0.75L^2$	$0.75L^2\gamma$ $= 57.8$ tonnes/m	74.0
Terzaghi (1943)	Elliptical envelope	Calculated by Wilson's formula $\frac{W}{2}\left(\frac{(q)}{p+p'}^{\frac{1}{K-1}} - 1\right)$	Vertical Stress $\sigma_v = \frac{XB}{2K_1\tan\phi}\left(\gamma - \frac{2C}{XB}\right) \times (1-e^{-K_1\frac{\tan\phi 2L}{XB}})$ $= 15.38$ tonnes/m^3 or 84.7 tonnes/m	Horizontal Stress $0.3(\sigma_v + \frac{1}{2}\gamma W)$ $= 6.27$ tonnes/m^3 $= 27.09$ tonnes/m	100.60 Vertical 35.00 Horizontal

H = Roadway width
L = Height of broken material surrounding roadway

Table 4.1 Load on support system calculated by different formulae for an arched-shaped roadway, size - 5:5 x 4·3 m (after Zadeh 1982).

Substituting equations 2 and 3 into 4 gives:

$$P_a = [\tan^2(45 + \phi/2)]\ [U/S^2]\ t$$

$$\text{Let } q = \tan^2(45 + \phi/2)$$

$$\text{Then } P_a = \frac{qU}{S^2}(L-S)$$

$$\text{Or } P_aS^2 + qUS - qUL = 0$$

So that for a given bolt length, the spacing may be calculated and vice versa.

i.e. $S = \dfrac{-qU \pm \sqrt{qU(qU + 4P_aL)}}{2P_a}$

$$L = S + \frac{P_aS^2}{qU}$$

4.4.6 Reinforced Rock Units

Lang et al (1979, 1981, 1982, 1984) have proposed the concept of reinforced rock units (RRU) which consist of an individual bolt and the rock immediately surrounding it. Equations have been developed which give the minimum bolt tension required to ensure that units are stable relative to each other and act together as a structural member analogous to a Voussoir arch.

The rock is assumed to be de-stressed at a depth (D) (Figure 4.7a), but variable vertical stresses (σ_v) and horizontal stresses ($K\sigma_v$) are assumed to be induced within the de-stressed zone. Typically, K is taken as 0.5. The units (Figure 4.7b) are rectangular with dimensions S x S x L, where the side boundaries, CF and JE, in the rock mass, are at the point of failure. This failure is defined by the Mohr-Coulomb failure criterion:

$$\begin{aligned} \tau &= \sigma_h \tan\theta + c \\ &= \sigma_h\mu + c \\ &= K\sigma_v\mu + c \end{aligned}$$

where θ = angle of internal friction
c = apparent cohesion of rock
σ_h = horizontal stress
= $K\sigma_v$
σ_v = average vertical stress at distance y from FE
τ = shear strength of the side boundaries
μ = coefficient of friction (tan θ)
K = ratio of horizontal to vertical stress

If an enlarged zone of de-stressed rock such as CJHG is considered to be in limited equilibrium then the relation for stress at rock excavation surface CJ is:

$$\sigma = (\gamma R - c)\ 1/K\mu\ (1 - e^{-K\mu D/R})$$

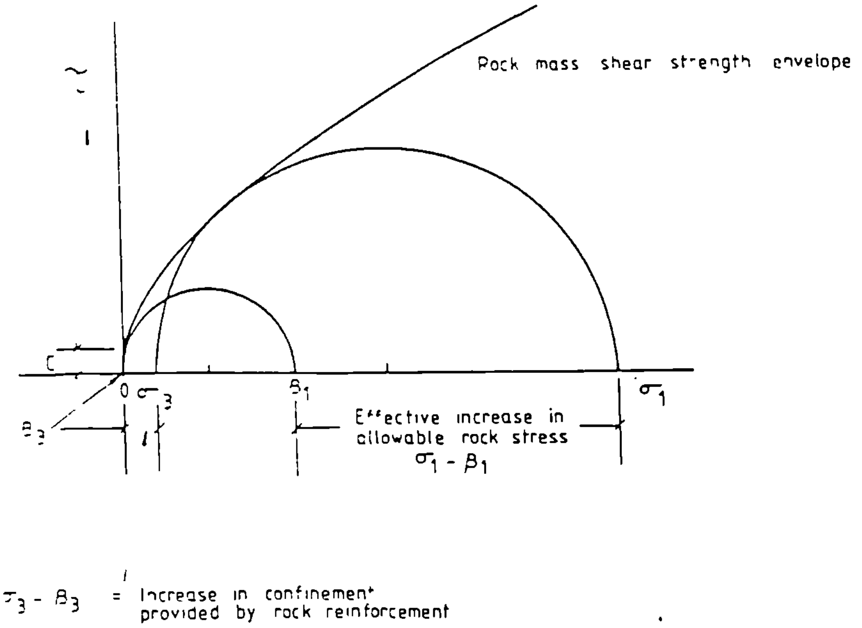
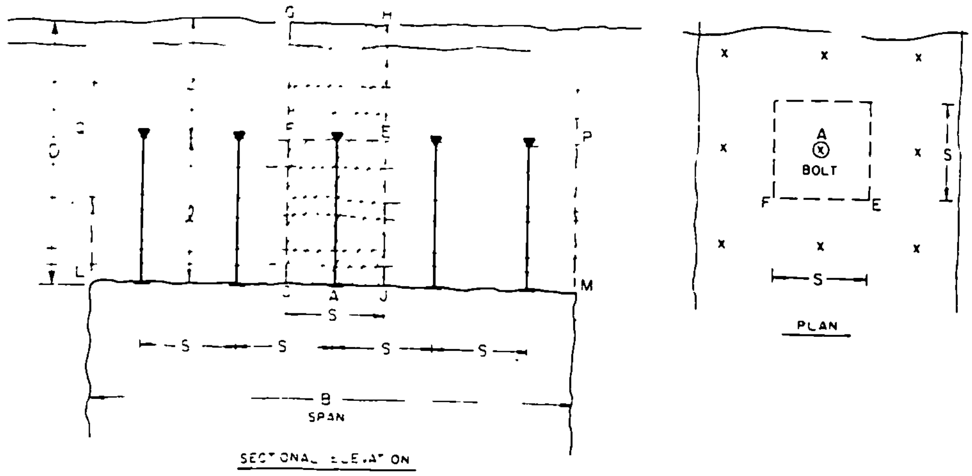


Figure 4.6 Effective increase in allowable rock stress with increased confinement.



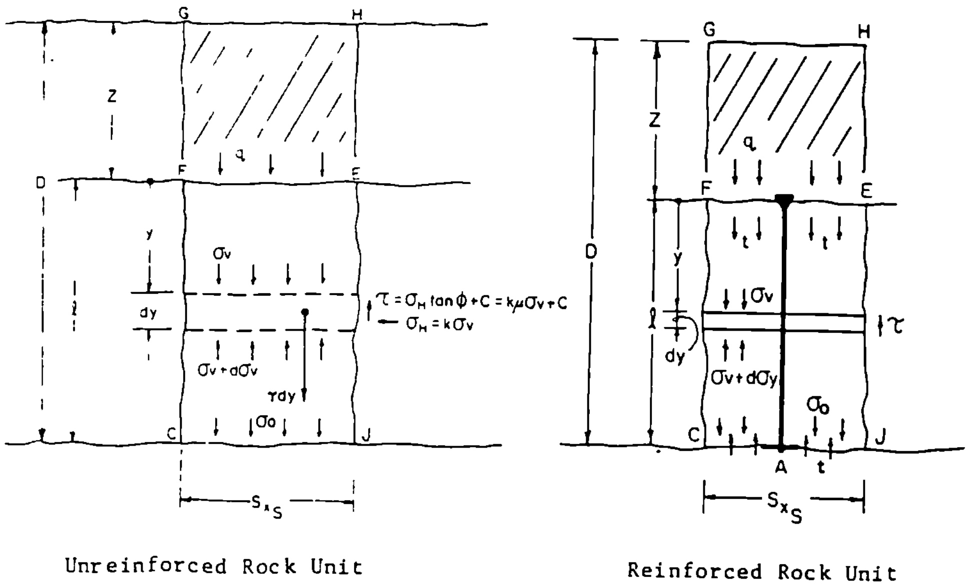


Figure 4.7 Concept of reinforced rock units (after Lang 1982).

where R = shear radius of rock column
= area/perimeter
= $S/4$
σ = average stabilizing pressure of the rock bolt
D = height of de-stressed rock above opening (length of rock bolt, L, plus length of influence)
γ = unit weight of rock

If (γR - c) is positive, support at the rock face is required to prevent fall out. In the case of a blocky beam, the rock bolt is angled across a joint or zone of anticipated tensile failure with a load σS^2. It is generally assumed that the bolting load T is equal to σS^2 and is uniformly distributed over the rock face, so that:

$$T = \alpha \frac{\gamma AR}{K\mu} (1 - c/\alpha R - \sigma h/\alpha R) \frac{1 - e^{-K\mu D/R}}{1 - e^{-K\mu L/R}}$$

where T = minimum bolt tension
α = factor depending on time of bolt effectiveness and installation, equals 0.5 for active reinforcement and 1.0 for passive reinforcement
A = reinforced area (s x s)
P = shear perimeter of rock column (4s)
s = bolt spacing
L = bolt length

Due to the possibility of obtaining misleading information from currently available testing methods, Lang and Bischoff (1982), suggest that, in initial designs and investigations the cohesion should be taken as zero. It is recommended that the bolt length should be greater than the bolt spacing to ensure that a stable arch is maintained. The bolt length/spacing ratio should be between 1.5 and 2.0.

4.5 Numerical Modelling

A considerable amount of research has been carried out in recent years into the application of numerical modelling techniques for the determination of stresses around (and the performance of) underground excavations. The two most commonly used techniques are the finite element and boundary element methods. Coulthard et al (1983) have demonstrated that boundary elements are more economical and easier to use, while finite elements are more versatile. Both techniques have been applied to the modelling of rock masses reinforced with rock bolts (St John and Van Dillen 1983; Guo and Peng 1984).

4.6 Physical Modelling

4.6.1 Design Of Specific Sites

Physical models of underground structures are capable of demonstrating some of the characteristic features of rock mass behaviour which are not always evident in numerical models. It has been shown that it is possible to obtain quantitative or semi-quantitative data from physical model tests, however, the technique is more suited to qualitative analysis. A valuable advantage of scale model studies of mine roadways is the ability to examine changes in roadway deformation caused by altering only one of the many parameters affecting roadway closure.

Workers at a number of research establishments throughout the world have applied physical model techniques to evaluate the performance of underground structures supported by rock bolt systems. Some of this work has been reported by Carr and Silvester (1972), Silvester (1975), Brook (1977), Mullins (1985), Grotowsky (1977), Gotze (1977), Oldengott (1979), Egger and Gindroz (1979), Gotze et al (1982), Gotze (1986), Dhar et al (1983), Panek (1955), Evans (1960), Goodman et al (1972), Roko and Daemen (1983), Stimpson (1983) and Pettibone et al (1985).

The model rig (Figure 4.8) and many of the techniques employed in this study were initially developed by Hobbs (1965). Models tested using the equipment over the last 20 years have provided a large store of information on the effect of different parameters on roadway closure (Hobbs 1966, 1967a, 1967b, 1968a, 1968b, 1968c, 1968d, Lawrence 1972, Silvester 1975, Bloor 1980). Several improvements have been made to the modelling technique over the years, so that specific sites can be simulated and in certain cases semi-quantitative data can be obtained. The model consists of sand/plaster slabs which represent a 30.5 m square of underground strata with a geometrical scale factor of 1/50. Other scale factors, derived by dimensional analysis (Hobbs 1965, Lawrence 1972) are given in Table 4.2.

Physical Parameter	Scale Factor
Linear Dimension	1/50
Applied Pressure	1/35
Strata Strength	1/90
Density	11/20

Table 4.2 Model scale factors.

A model is designed using geotechnical information from 15 m above and below the roadway under investigation. The equivalent rock material

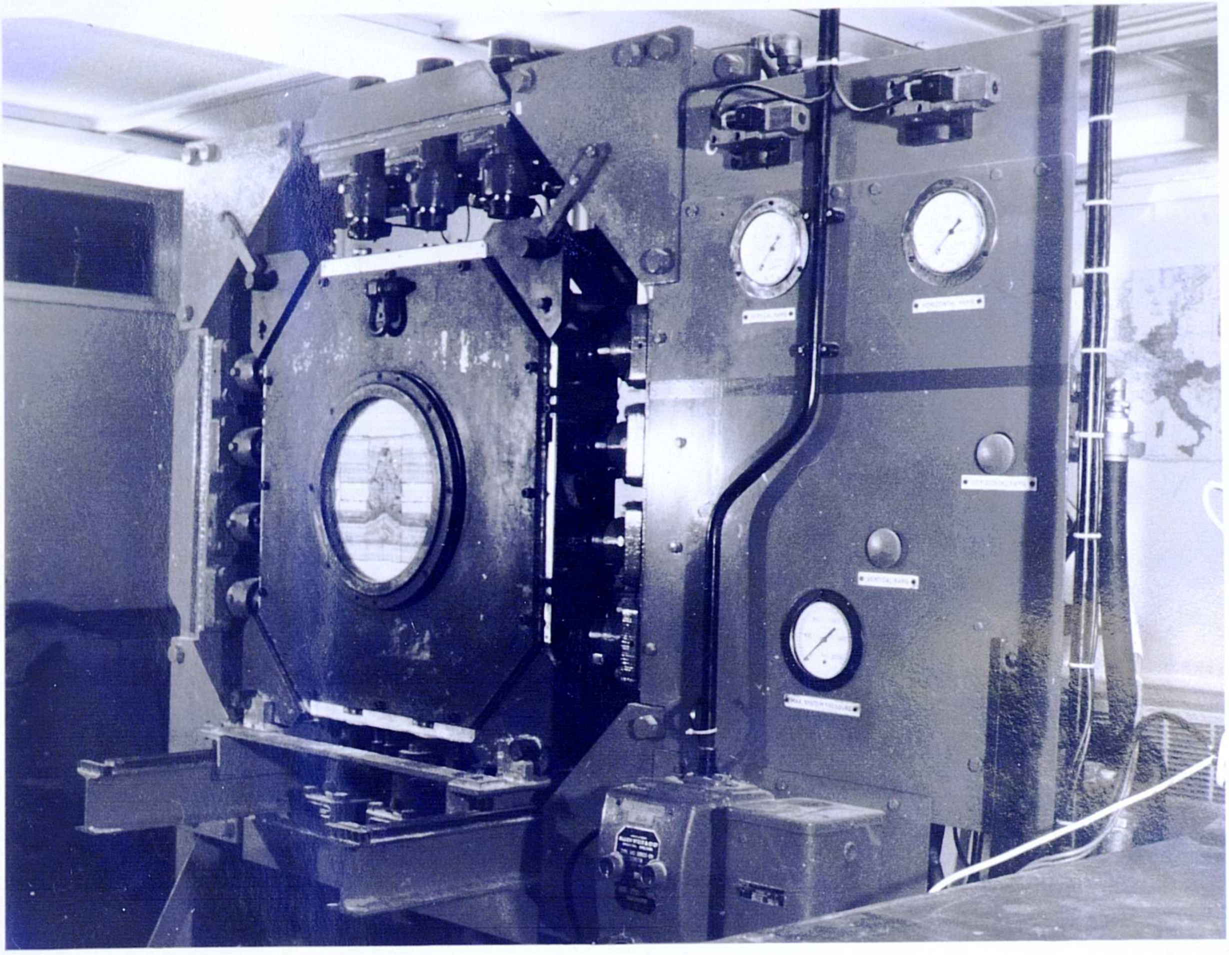


Figure 4.8 British Coal HQTD roadway model rig

consists of slabs made by mixing sand, casting plaster (calcium sulphate hemihydrate) and water. The physical properties of the material can be varied by altering the constituent proportions to enable the simulation of different strata. A colouring pigment is added during mixing to permit easy identification of model strata with differing properties. Figure 4.9 shows the relationship between the proportion of sand in the model strata and the laboratory strength of the rock that they simulate.

The procedure developed by Hobbs (1965) whereby the slabs were dried for up to a week at approximately 90°C in an electrically heated cabinet with no thermostatic control has been discontinued. Tests carried out by Bloor (1980) concluded that the length of time required to ensure decomposition of the calcium sulphate dihydrate of the set plaster to the hemihydrate required for the model strata was strongly dependent on temperature. The degree of temperature control in the existing cabinets was inadequate to ensure good reproducibility of model strata strengths. To overcome this problem, material for each complete model is heat-treated in a force-ventilation, thermostatically controlled oven at 120°C for approximately 48 hours. The model material is left to stand for 24 hours after heat-treatment to achieve equilibrium with laboratory conditions of humidity and temperature, as absorption of atmospheric water was found to be responsible for a further decrease in strength to within the specified region.

A standard slab is 12.7 mm thick, although thicker slabs can be manufactured to simulate massive strata. The slabs can be solid or made of up to five laminations, each with a minimum thickness of 2.54 mm. Formerly lens tissue was used as an interface medium between each lamination, however this had some effect on the strength and altered the frictional properties of the laminated slabs. A thin coating of detergent is now applied to the surface of each lamination to reduce the cohesion between the layers. A complete model is illustrated in Figure 4.10.

Steel arches and girders are modelled using lengths of pure lead soldered together and formed into the appropriate support shape. Lead was adopted for this purpose because the tensile strength ratio of lead to steel is approximately 1:31 and therefore compatible with the stress scale ratio for the model (1:35). Rectangular section lead supports were specifically designed (Bloor 1985) to model the buckling behaviour of a 114 mm RSJ more closely than the original H-section lead supports

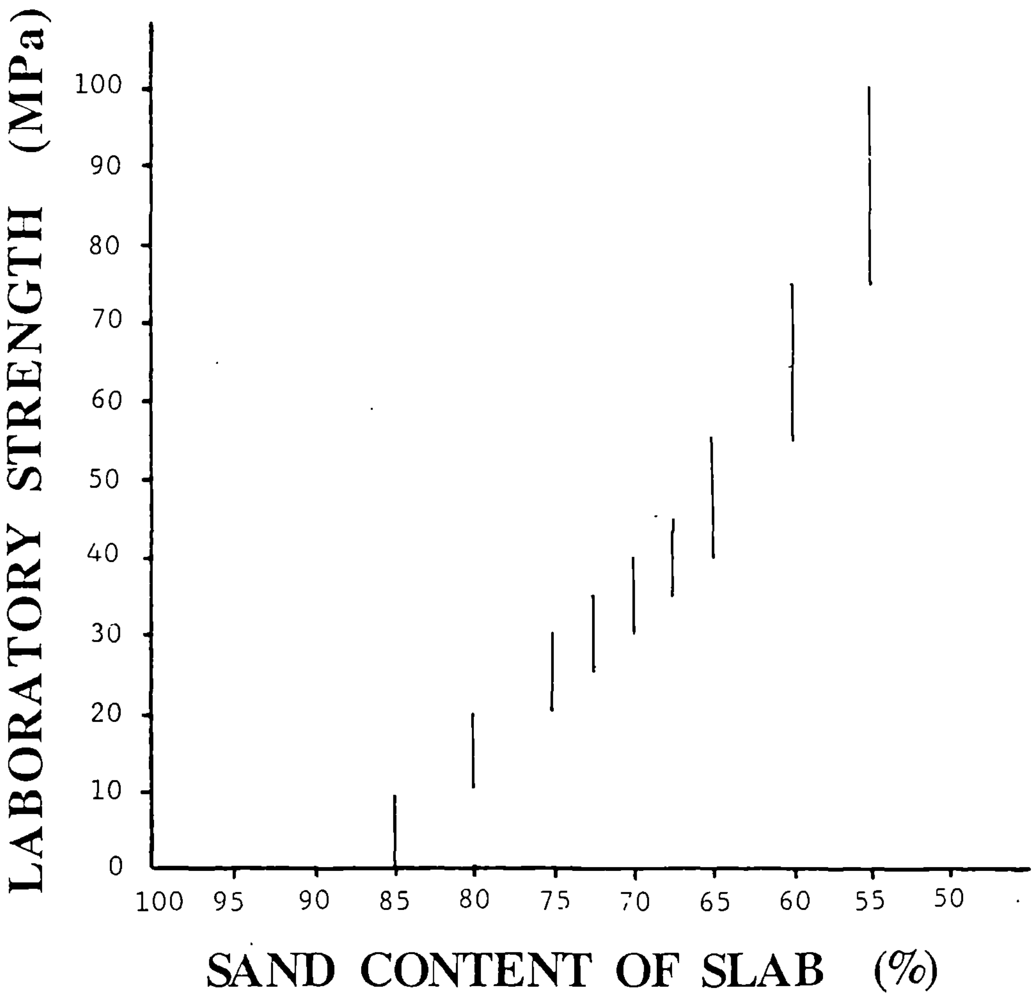


Figure 4.9 Laboratory strength of rock versus sand content of model rock.

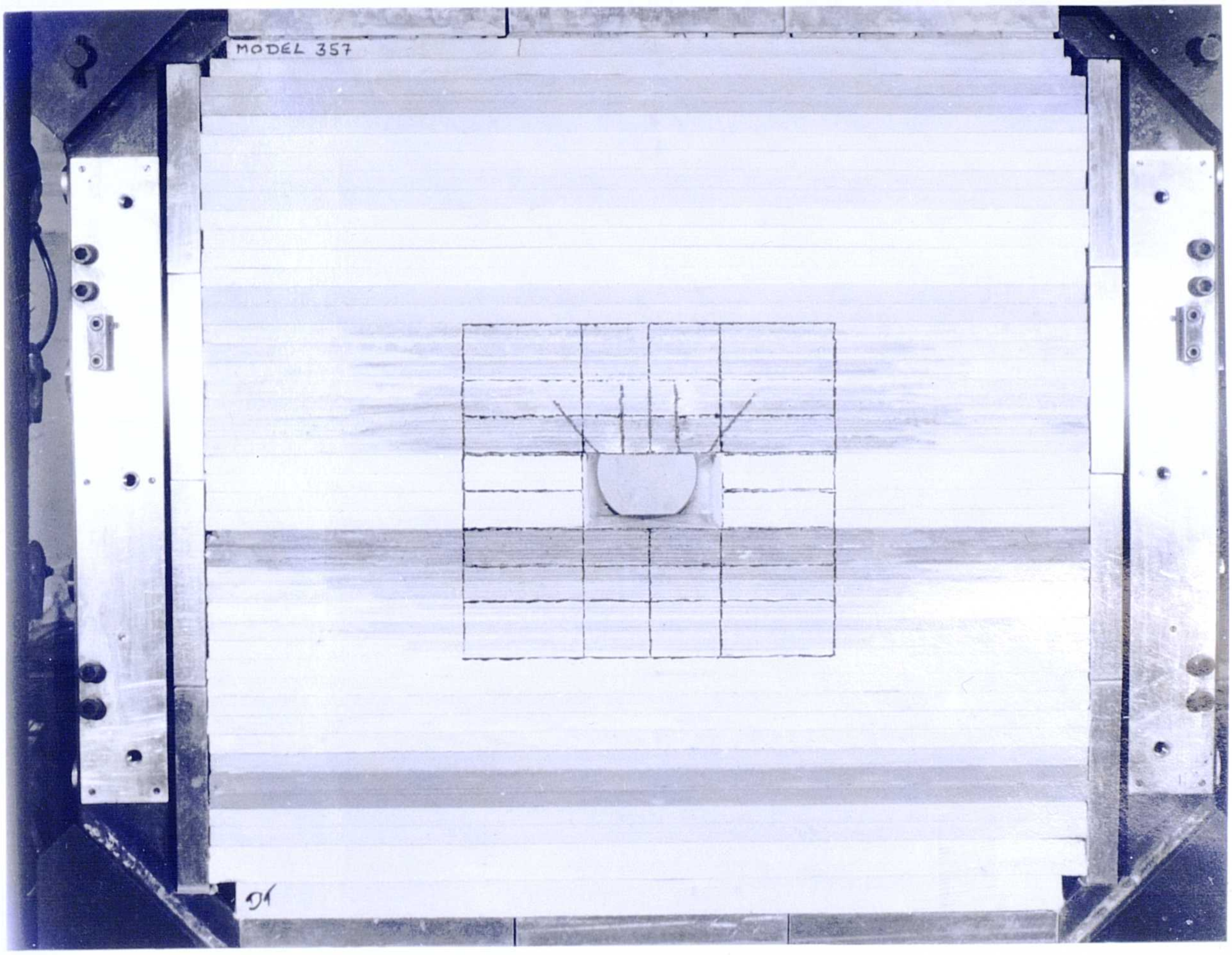


Figure 4.10 Physical model of a mine roadway

used by Hobbs (1965). Discrepancies still exist particularly in the scaling of Young's moduli for lead and steel (ratio 1:13) and shear moduli (ratio 1:15). In order to model bending or shear characteristics the lead cross section would need to be adjusted.

A mechanical type rock bolt can be simulated by the use of steel pins and a spring of a size such that when fully compressed it provides a residual tension, after compaction of the sand/plaster slabs. Full column anchored bolts are represented by tin/lead wire cut to the appropriate length and fixed into the model strata by a plaster grout (Figure 4.11). There is no pre-tension applied to these bolts.

The models are loaded inside a box on four 610 x 127 mm faces by 0.3 MN hydraulic rams acting through 25 mm thick steel platens. The original box had six pairs of rams (three horizontal and three vertical), and was designed such that one pair of horizontal rams acted on both the immediate roof and floor strata of the model roadway. It was found that the strains were equal which meant that the model tended to under estimate floor closure (weaker strata) and over estimate roof closure (stronger strata). Therefore no qualitative comparisons between bolted and un-bolted strata could be made. The model box has been re-designed with four pairs of horizontal rams such that the roof and floor strata in the vicinity of the roadway are compressed by different pairs. This allows roof and floor strata to be subjected to independent horizontal strains.

The theoretical hydrostatic pressure range of the equipment is 0 to 1.2 MPa. At various fixed ram pressures high quality colour photographs are taken. In addition video images are stored on tape and within an image analysis system. A grid painted on the model and markers on the face plate simplify the measurements of roadway closure and allow corrections to be made for errors caused by compression and compaction of the model strata.

After a test the model is off loaded, the cover plates removed and then the strata surrounding the roadway is dissected. This reveals fracture patterns within the model that may be disguised by surface effects and enables detailed inspection of the bolts so that deformation and failure modes can be determined.

Ideally the initial test in a series should represent an existing

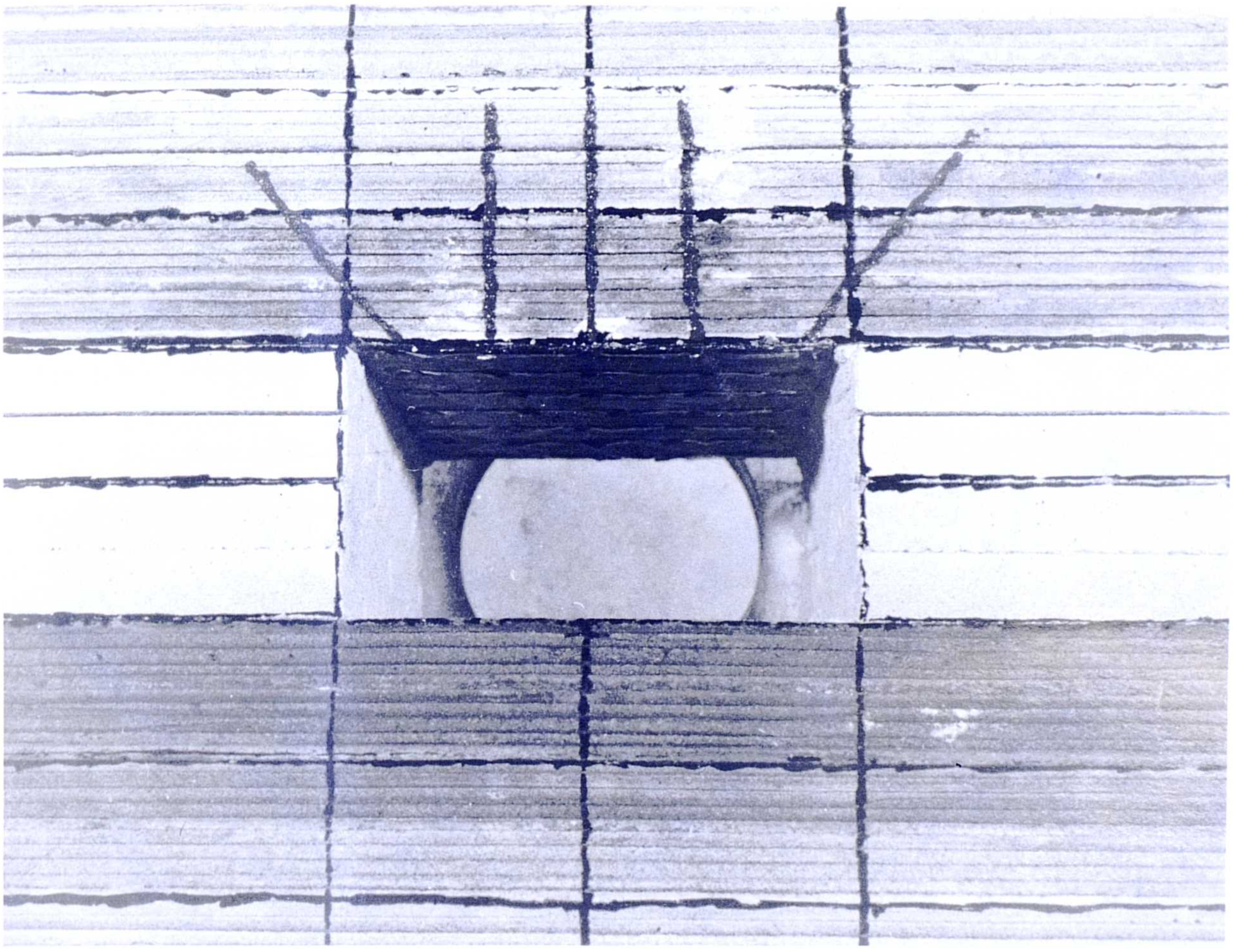


Figure 4.11 Simulated roof bolts in a model roadway

roadway similar to that under investigation, (i.e. in the same seam and stress conditions) so that checks can be made to ensure that the model behaves the same as underground observations indicate.

To be used as an effective design tool the limitations of the modelling technique must be appreciated. The accuracy of any qualitative or quantitative data derived from modelling is dependent on the available information regarding the site under investigation.

The materials used are by no means ideal. There are several inaccuracies in scaling, for example the density of the equivalent rock material is less than half that deduced by dimensional analysis and the Young's modulus of the model supports is over twice that required, consequently bending and buckling strength cannot be accurately scaled with the same section support. The only forms of discontinuity which it is attempted to reproduce in the models are laminations, bedding planes and principal parting planes. In addition, the modelling technique takes no account of strata creep.

The test rig also has limitations. It is a biaxial compression rig, simulating rock stresses and deformation in one plane only and not the three dimensional conditions that occur in practice. The models can be loaded up to maximium applied boundary pressures of 1.2 MPa, corresponding to 42 MPa underground. This is the level of stress found in virgin coal measures strata at an overburden depth of approximately 1700 m. The maximium winning depth in British coal mines is currently 1208 m. Therefore stresses surrounding access roadways and development drivages can be represented. Difficulties arise with gateroads where face abutment pressures can be greater than four times the cover load (Section 2.10). Consequently with the scale factors used at present the simulation of the effect of a longwall on a gateroad is limited to faces at a depth of less than 425 m.

It is therefore clear that the British Coal HQTD roadway model rig is limited in the degree of similitude that can be achieved. However it has been used as an extremely effective tool in the design of underground support systems. Each model is relatively economical to produce, taking approximately 20 man hours and using inexpensive materials. Strata deformation and the formation of fracture patterns can be easily observed; and roadway support systems can be optimised and frequently tested to destruction.

4.6.2 Qualitative Assessment Of Rock Bolting Parameters

In addition to modelling specific sites for design purposes, the physical modelling technique can also be used in a purely qualitative manner to assess the factors affecting roadway closure. A series of tests have been carried out to evaluate various rock bolt and excavation parameters. The tests described in this section only considered full column anchored roof bolts (tests simulating floor bolts, truss bolts and bolted roof straps are described in Sections 11.4 - 11.5, 12.2 and 15.1 respectively).

The model configuration chosen for the tests sought to represent a typical development drivage in British coal measures that might be chosen for a roof bolting trial (i.e. moderately strong roof strata). The roadways were rectangular, and (unless otherwise stated) had equivalent dimensions of 4.75 m wide and 2.54 m and were formed within the model strata configuration shown in Appendix 1a. Details of each support configuration is given in Appendix 2.

Effectiveness Of Roof Bolting In Different Stress Environments

Figures 4.12a, 4.12b and 4.12c show the effect of different stress environments on roadways supported by steel work, five 2.44 m long vertical bolts and five 2.44 m long bolts with the two shoulder bolts angled at 40^{0} to the vertical over the ribsides. Plotted on each graph is the percentage roof lowering (with respect to the original roadway height) occuring at various pressure increments. Three types of stress environment were simulated by:

(i) increasing the applied pressure equally in the vertical and horizontal planes;

(ii) increasing the horizontal pressure at twice the rate of the vertical;

(iii) increasing the vertical pressure at twice the rate of the horizontal.

All three graphs have basically similar pressure-deformation curves. The shape of the curve on each graph is related to the mode of roof fracturing.

The models under hydrostatic loading showed greater stability than those subjected to anisotropic stress conditions. The gradient of the deformation curve increased following the formation of an inverted V-shaped fracture pattern in the roof. This occurred at a critical

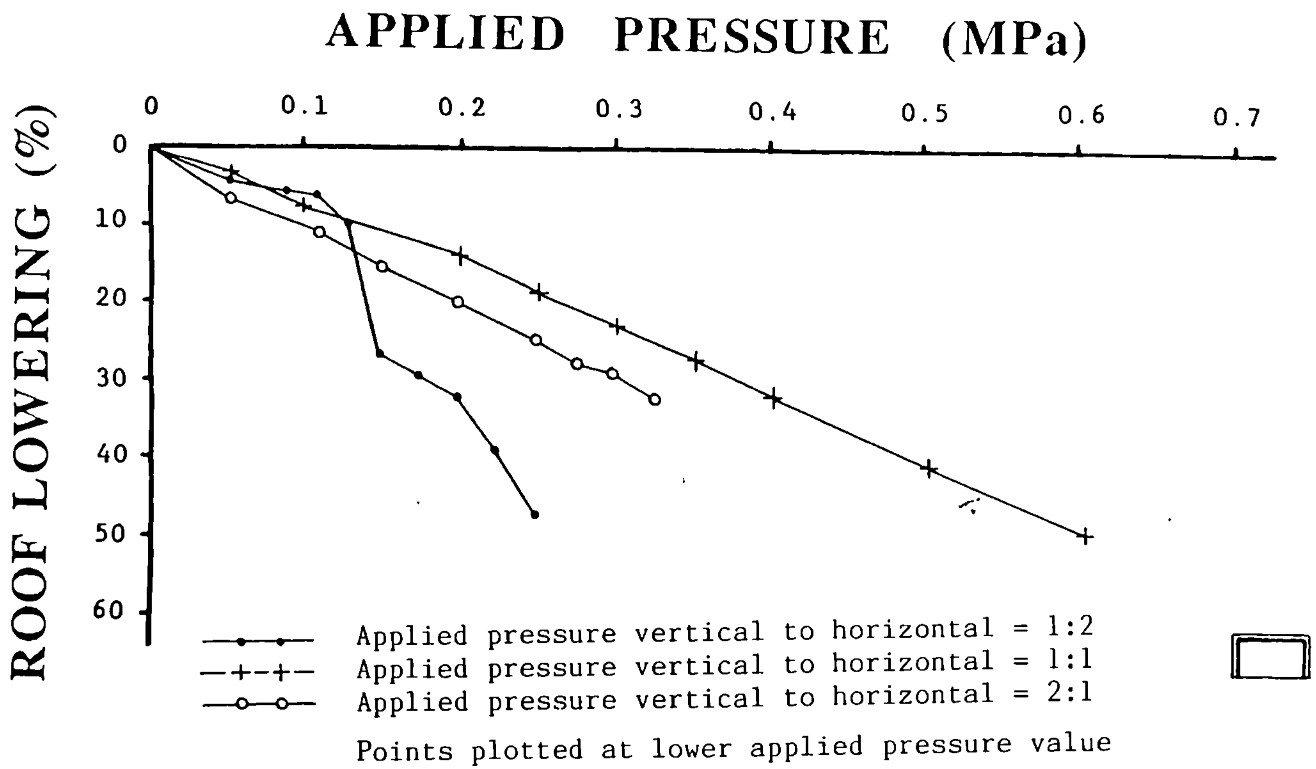


Figure 4.12a Variation in roof lowering as a percentage of initial roadway height with applied pressure - steel standing support.

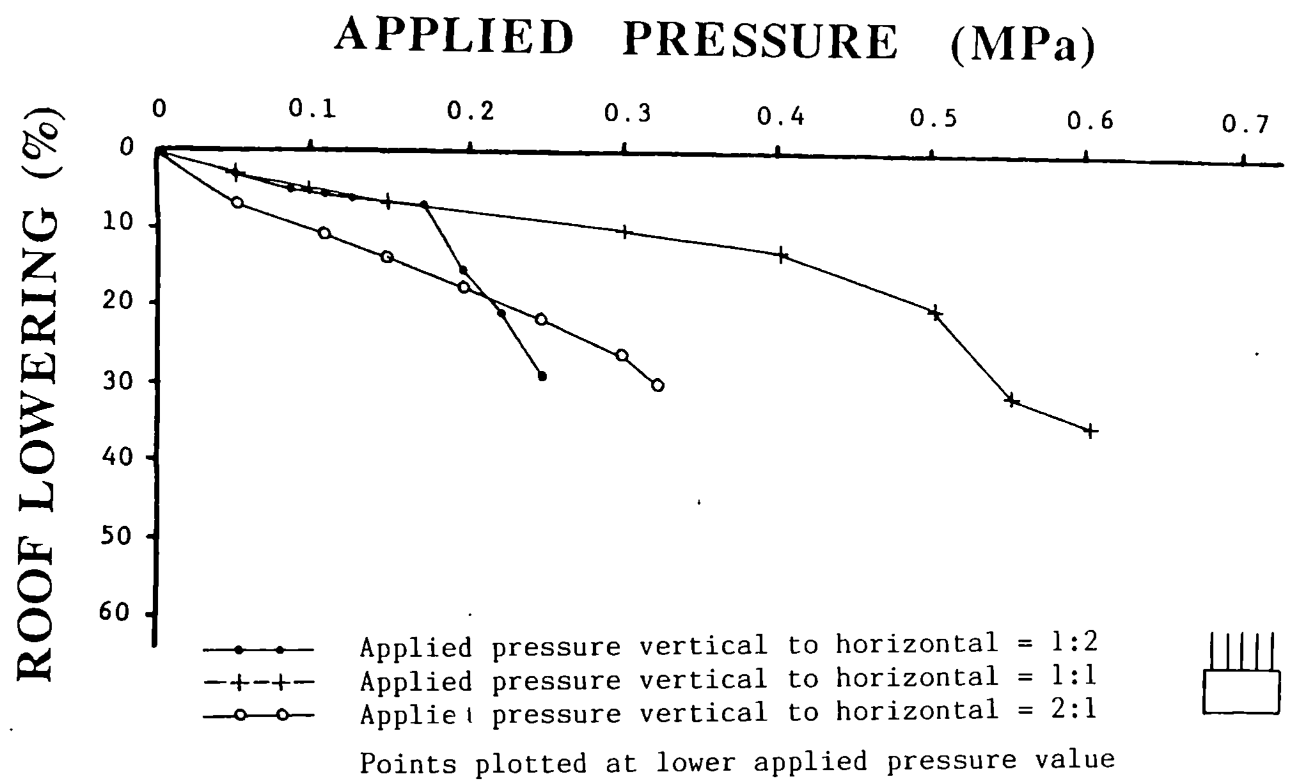


Figure 4.12b Variation in roof lowering as a percentage of initial roadway height with applied pressure – five vertical bolts.

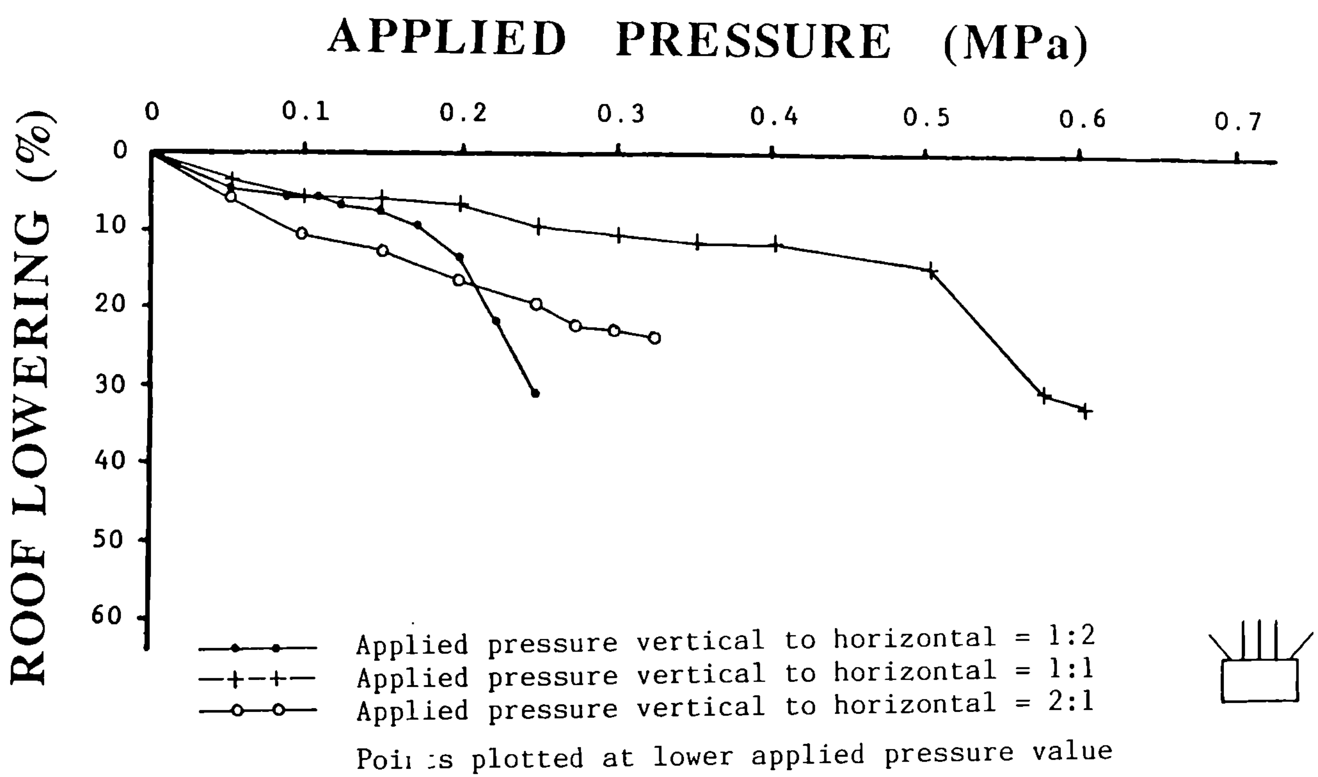
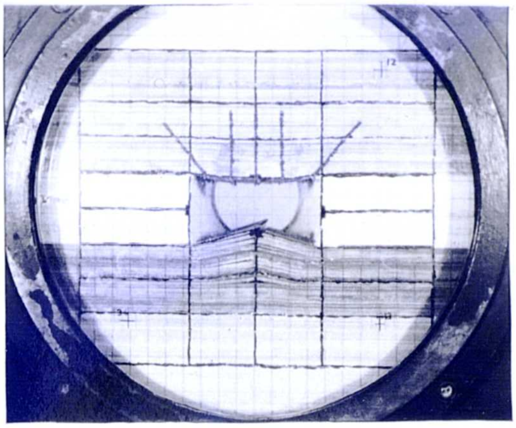
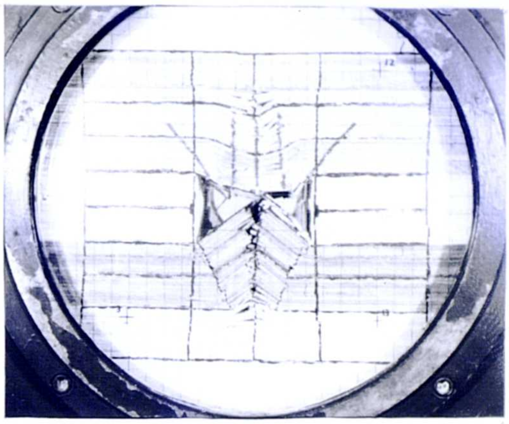


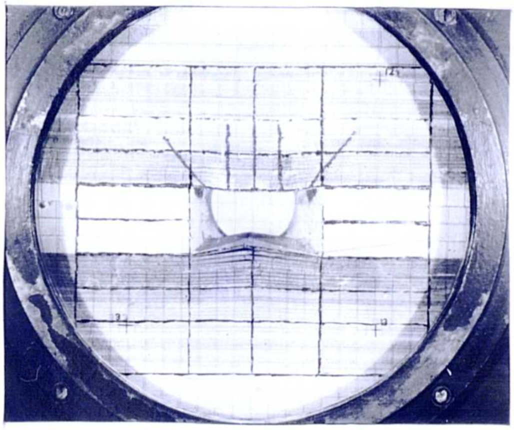
Figure 4.12c Variation in roof lowering as a percentage of initial roadway height with applied pressure – three vertical bolts and two inclined bolts.



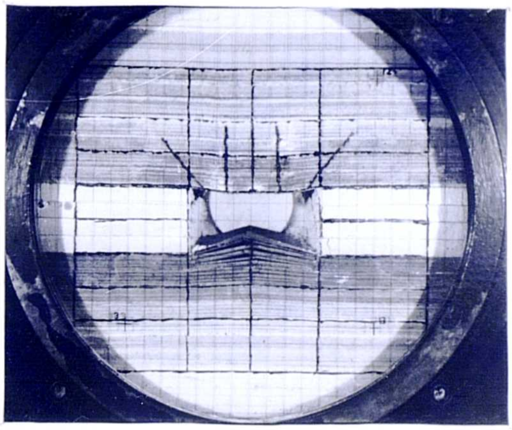
Vertical 0.15 MPa
Horizontal 0.30 MPa



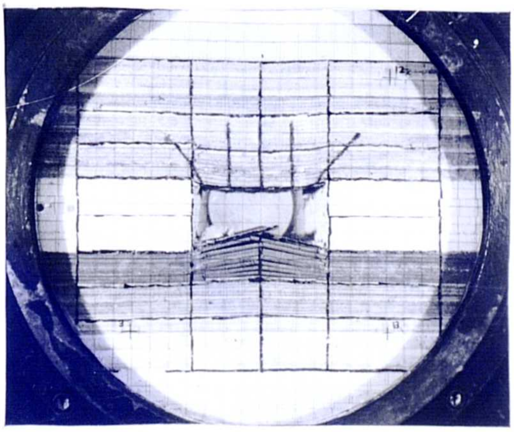
Vertical 0.25 MPa
Horizontal 0.50 MPa



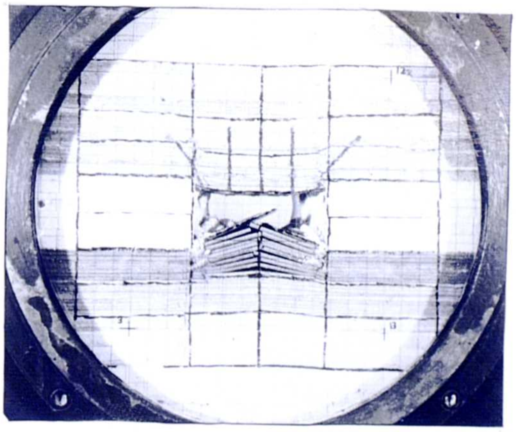
Vertical 0.15 MPa
Horizontal 0.15 MPa



Vertical 0.25 MPa
Horizontal 0.25 MPa



Vertical 0.30 MPa
Horizontal 0.15 MPa



Vertical 0.50 MPa
Horizontal 0.25 MPa

Figure 4.13 Deformation of modelled roadways under various stress conditions applied pressure, the level of which is dependent on the support configuration.

The model roadways subjected to high horizontal stresses deformed in a similar manner to roadways in a hydrostatic stress up to a critical loading; above this level of applied pressure the roof underwent considerable deformation, following the formation of an inverted V-shaped fracture pattern in the roof.

A virtually constant rate of roof lowering with increased pressure occurred in the model roadways subjected to high vertical stress. Roof deformation was marked by the development of vertical fractures above the ribsides.

The results of these tests verify those obtained from similar scale model studies of rectangular roadways with steel work support carried out by Lawrence (1972). These tests compared the effects of uniaxial pressure in the vertical direction, uniaxial pressure in the horizontal direction and hydrostatic pressure. Lawrence concluded that high horizontal stresses were a major cause of roadway failure because the layers of strata, acting as struts, fail by buckling when the horizontal stress exceeds the Euler crippling load but increased vertical pressure lessens the effect due to increased interbed friction.

Figures 4.14a, 4.14b, and 4.14c are graphs showing the same data presented in Figures 4.12a, 4.12b and 4.12c but plotted to illustrate the comparative performances of each support configuration in the three different stress environments.

It is apparent that roof bolting is significantly better at maintaining roof stability than steel work under hydrostatic and high horizontal stress conditions. This illustrates the reinforcing action of full column anchored roof bolts, raising the level of applied pressure that the roof can withstand before the formation of an inverted V-shaped fracture pattern. In a high vertical stress field the roof does not deform in this manner, consequently bolting only produces marginally superior roof conditions than those obtained using standing support.

The practice of inclining shoulder bolts over the ribsides of rectangular roadways varies worldwide. It is common in the mining industries of France (Raffoux et al 1970; Raffoux 1971; Auriol 1972;

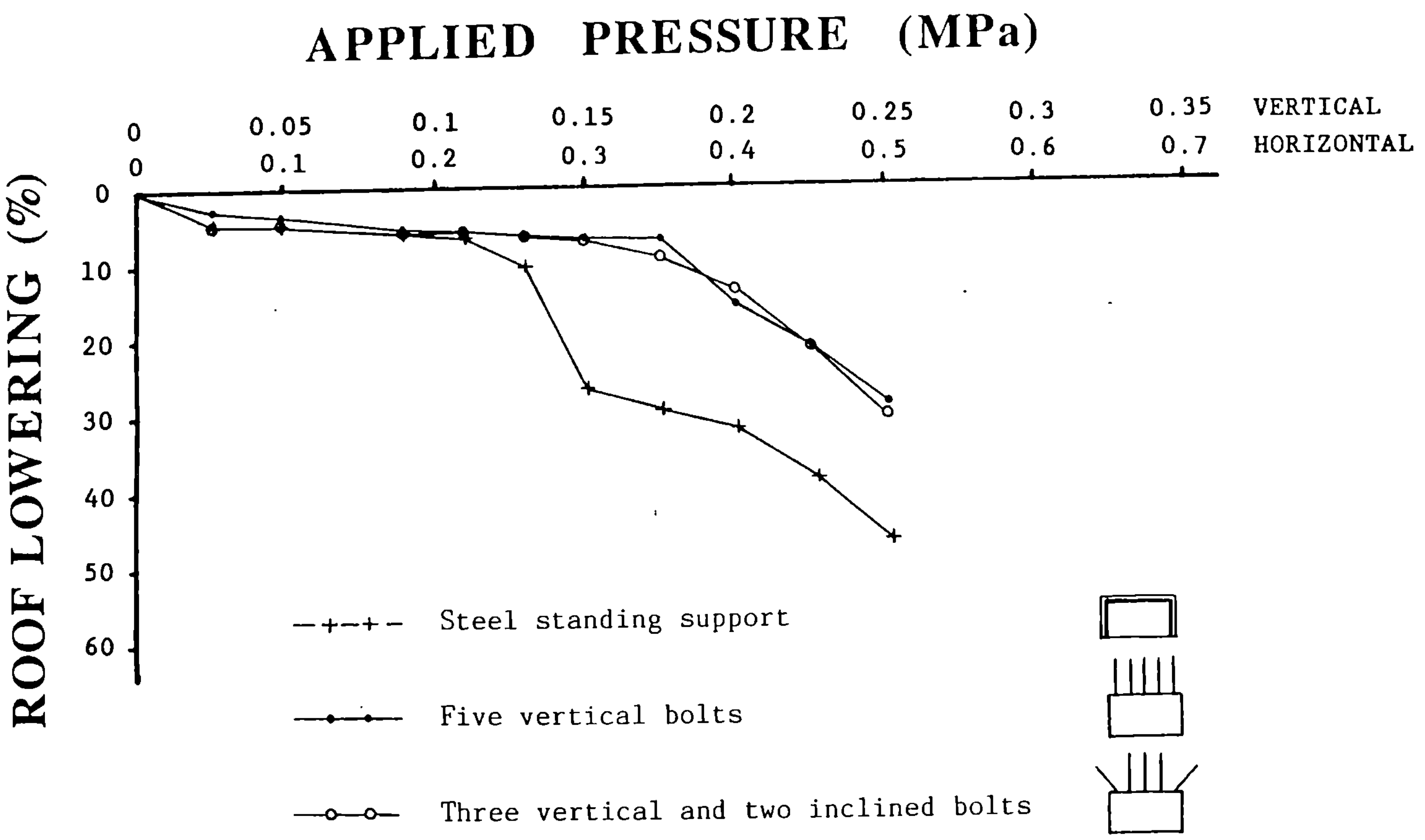


Figure 4.14a Variation in roof lowering as a percentage of initial roadway height with applied pressure - high horizontal stress field (applied pressure vertical to horizontal = 1:2).

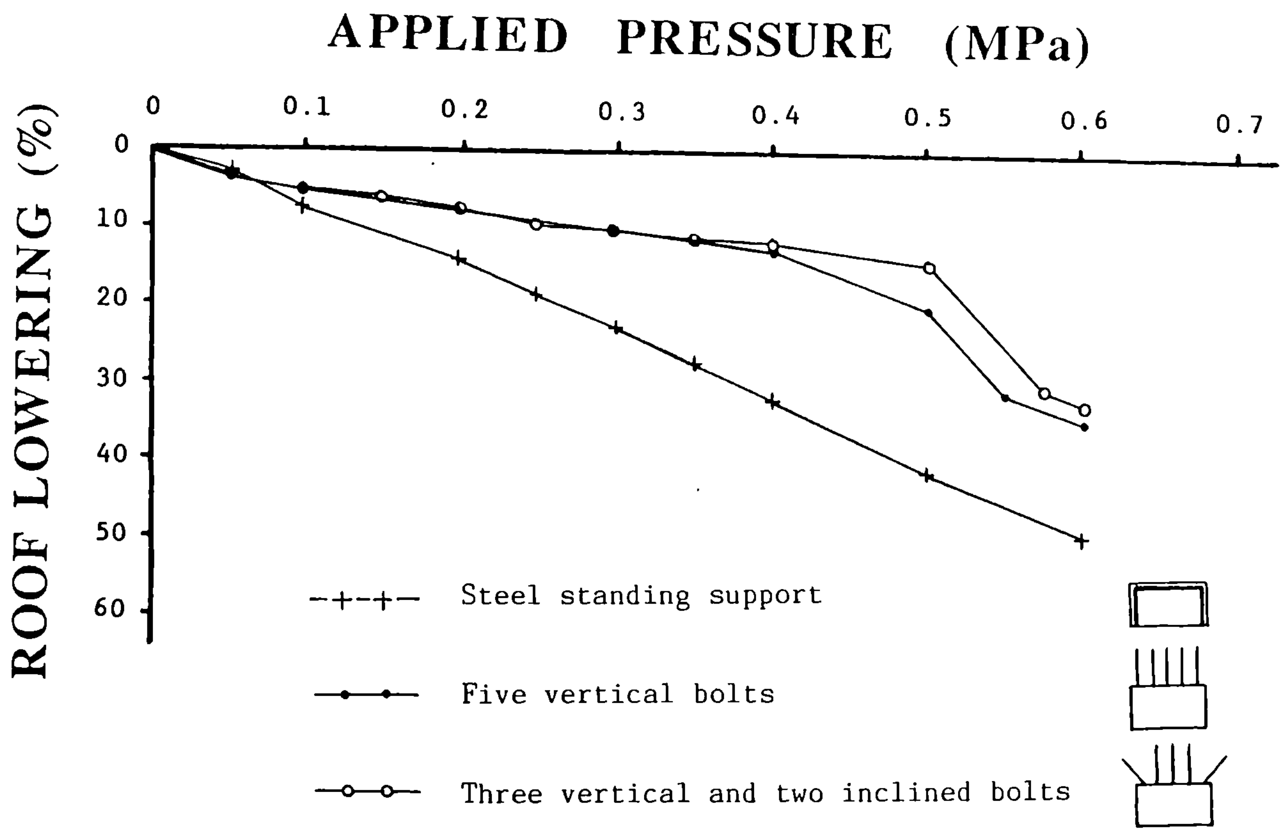


Figure 4.14b Variation in roof lowering as a percentage of initial roadway height with applied pressure - hydrostatic stress field (applied pressure vertical to horizontal = 1:1).

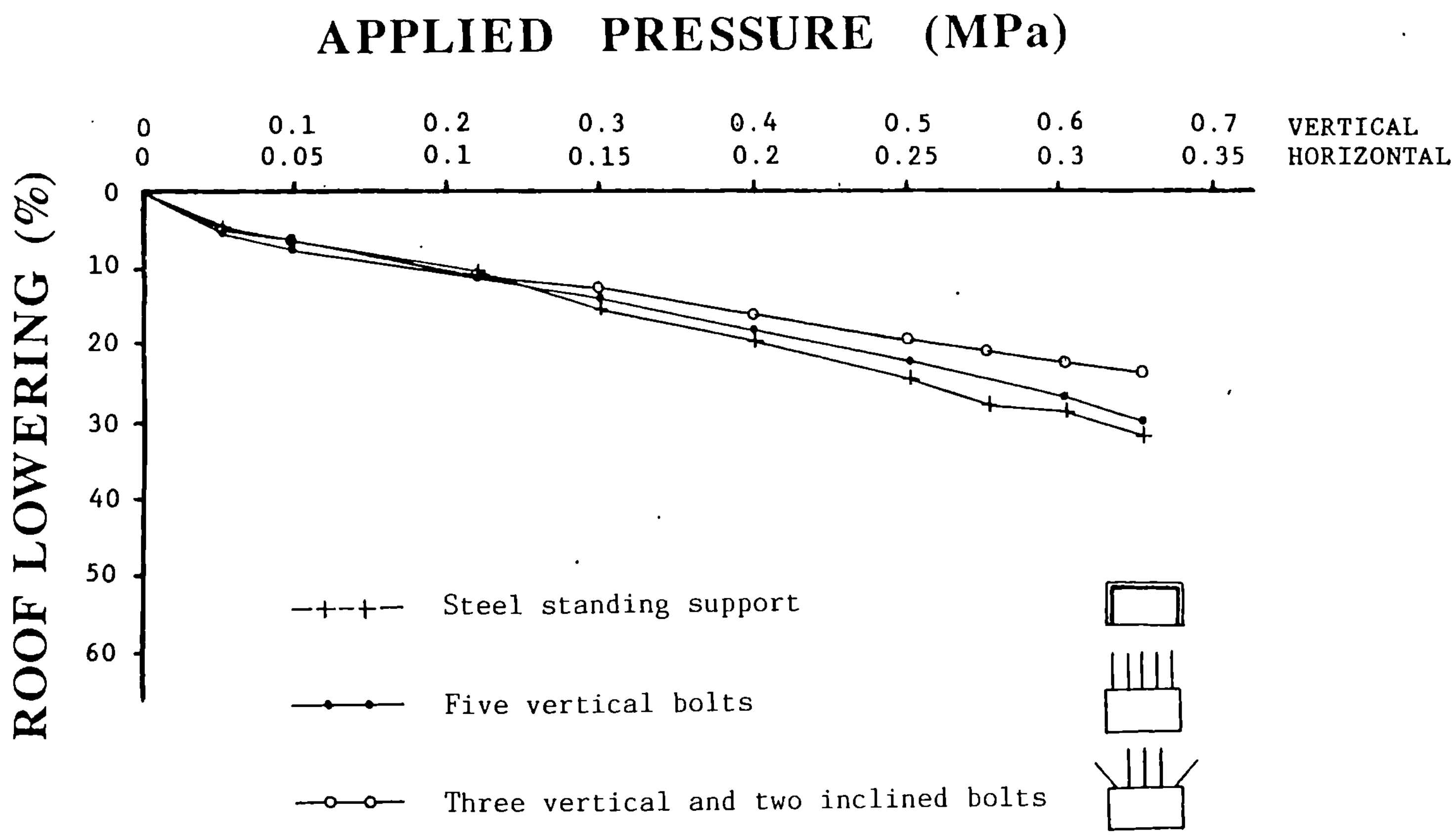


Figure 4.14c Variation in roof lowering as a percentage of initial roadway height with applied pressure - high vertical stress field (applied pressure vertical to horizontal = 2:1).

Gouilloux and Piguet 1977; Tinchon 1980; Dejean and Raffoux 1980a) and West Germany (Nocke et al 1968; Nocke 1970; Maiweg 1981; Bohnlein 1981).

The vast majority of rock bolts in United States coal mines are installed vertically. Inclined bolts were installed during the 1950s and in many cases found to be very effective (Thomas 1962). Owing to practical considerations such as longer installation time and lack of understanding as to how they act, this practice has been discontinued. Field studies in a US room and pillar mine by Singh (1978) could not establish any clear advantages in using inclined rock bolts.

Monitoring of rock bolted roadways in Australia has indicated that high axial loads can develop in the bolted roof over the centre of the heading and very high shear loads can develop over the ribs. For this reason vertical shoulder bolts are installed as it is considered that angled ribside bolts can only be of assistance to reinforce the abutment of a high arch, or "stitch" fractures (Gale 1987). The aim of bolting in this instance is to prevent such an arch forming rather than deal with its effects.

The physical model tests have demonstrated that where there is a high horizontal stress field (such as is known to exist in many Australian and US coal mines), inclining shoulder bolts over the ribsides has a negligible effect on improving roof stability. In a hydrostatic stress field, angled bolts gave a slight reduction in roof lowering at high applied pressures. Where a vertical to horizontal stress ratio of 2:1 was applied, inclining shoulder bolts over the ribsides resulted in a significant improvement in roof conditions. These bolts were able to provide reinforcement across vertical shear fractures which developed above the ribsides. The photographs in Figure 4.13 illustrate the performance of the modelled roadway with angled bolts under different stress conditions.

Studies Of Rock Bolt Pattern, Length And Density

In the preceding section it was established that angled bolts can give a slight improvement in roadway roof conditions compared to vertical bolts in a hydrostatic stress field. Comparative tests were performed using other roof bolt patterns to determine their effect on roadway stability.

Figure 4.15 shows the further reduction in roof lowering that can be achieved at high applied pressures in hydrostatic stress conditions

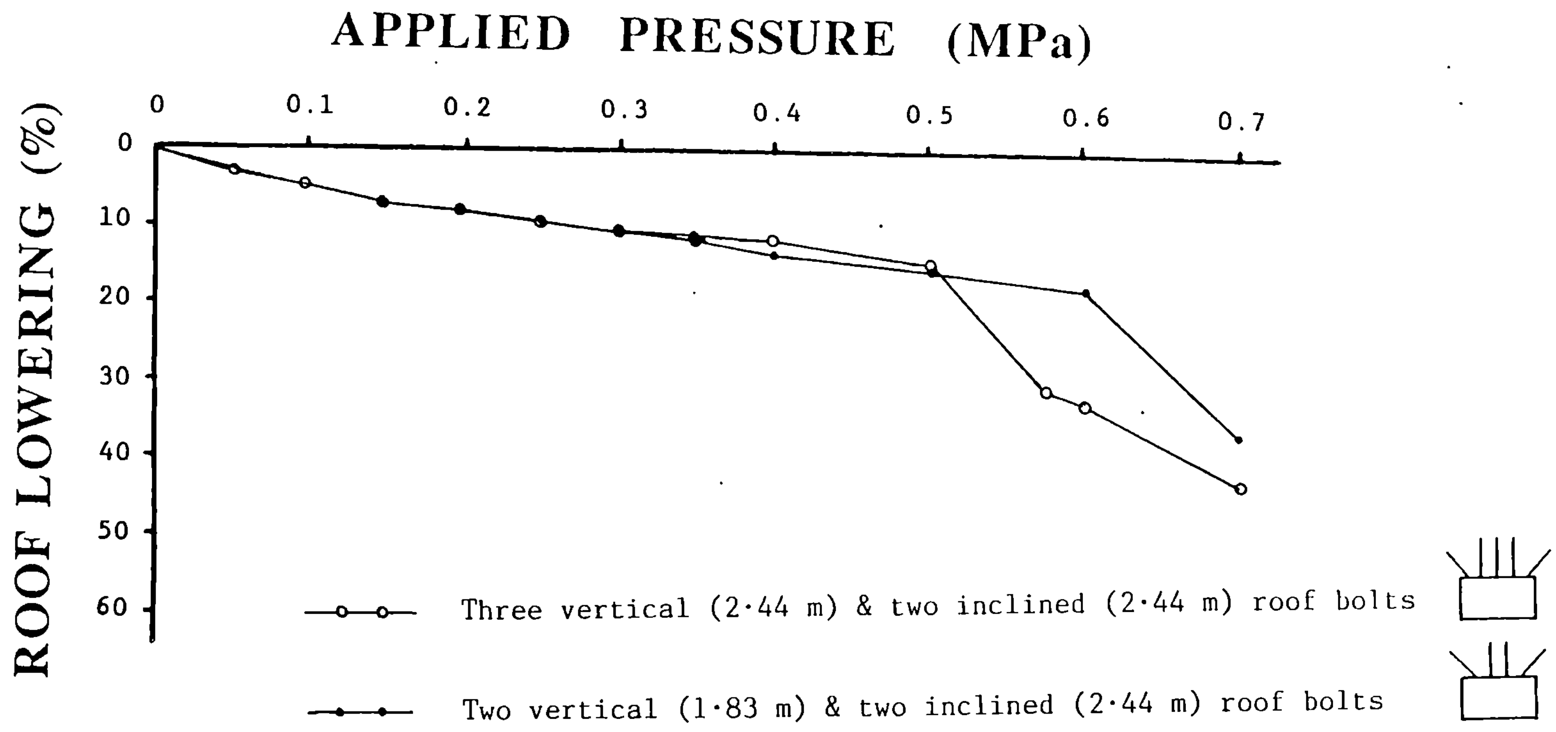


Figure 4.15 Variation in roof lowering as a percentage of initial roadway height with applied pressure - comparison of roof bolt patterns (applied pressure vertical to horizontal = 1:1).

using only a four bolt pattern (2 x 2.44 m shoulder bolts and 2 x 1.83 m vertical central bolts). The shoulder bolts were at a shallower angle (45^{0}) and positioned further from the ribsides (Appendix 2d). Therefore the position and inclination of roof bolts is an important factor influencing the critical load that a roadway roof can withstand before failure. Optimising the location of the shoulder bolts can enable a reduction in the number and length of vertical central bolts to be made. In practice the installation of 2.44 m vertical bolts would prove difficult in the restricted height of a 2.54 m high roadway, these problems would not occur using 1.83 m bolts.

Model tests were carried out to determine the effect of the two components of this improved bolting pattern (i.e. the 2 x 2.44 m shoulder bolts and 2 x 1.83 m vertical central bolts) when acting in isolation (Figure 4.16). Roadways with an equivalent height of 3.18 m were used in these tests (Appendices 2e and 2f). The two vertical bolts provided adequate reinforcement at low applied pressures. As the pressure was increased further, considerable roof lowering occurred followed by the formation of fractures above the ribsides and then catastrophic failure. The two angled bolts performed better, withstanding moderate applied pressures without any increase in roof lowering compared to the fully reinforced roadway. However, when relatively high pressures were applied, considerable roof deformation occurred. These tests not only illustrate the importance of shoulder bolts to retard roof failure but also emphasize the necessity to install a complete pattern of roof bolts systematically across the width of a roadway.

The magnitude of the stress field is an important consideration when determining roof bolt length for the support of a roadway. Plots of roof deformation versus applied pressure for tests using 5 x 1.22 m vertical bolts (a common support configuration in US mines) and 5 x 2.44 m bolts (Appendix 2b) in a hydrostatic stress field are shown in Figure 4.17. At low to moderate applied pressures (simulating shallow and medium depth roadways) the 1.22 m pattern performed in a virtually identical manner to the 2.44 m pattern. Benefits of using longer bolts were only apparent at high applied pressures.

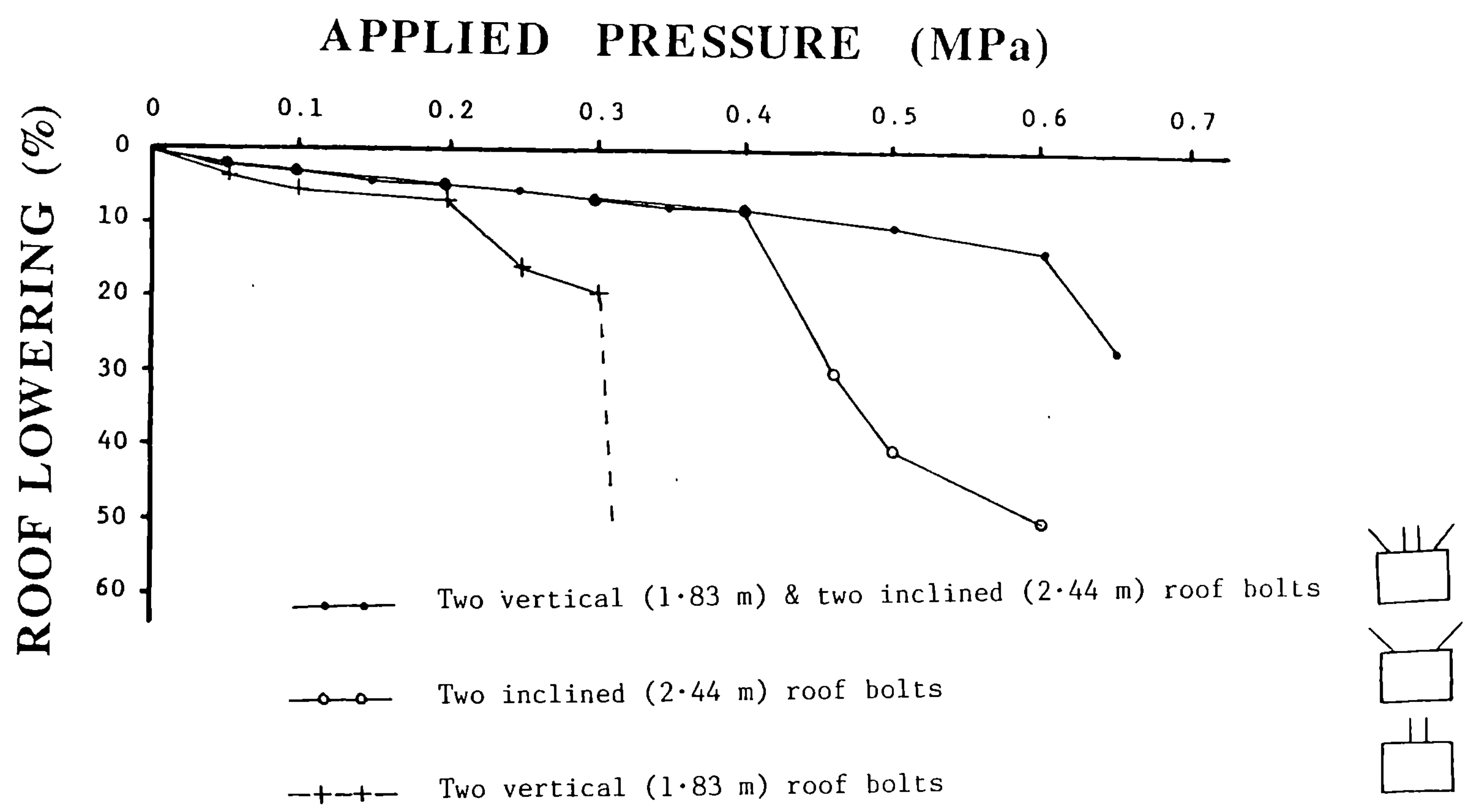


Figure 4.16 Variation in roof lowering as a percentage of initial roadway height with applied pressure - comparison of the effect of individual bolt pattern components (applied pressure vertical to horizontal = 1:1).

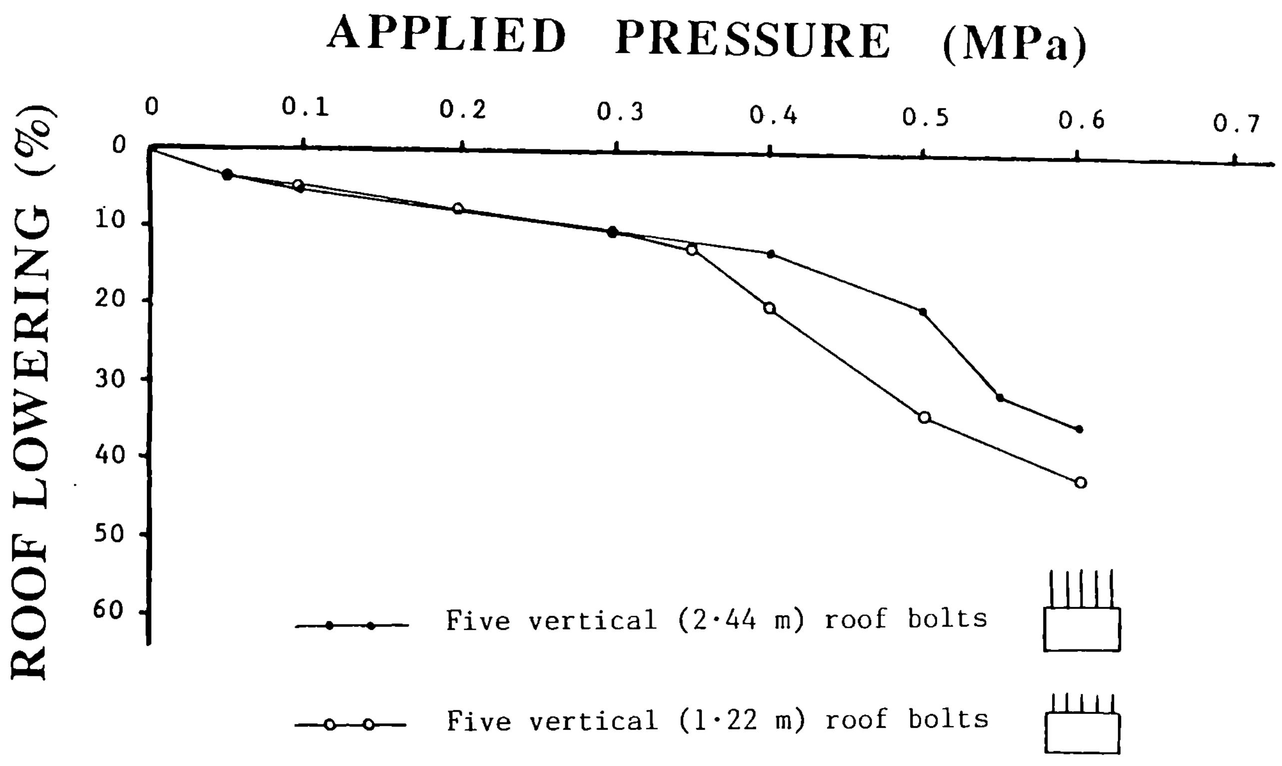


Figure 4.17 Variation in roof lowering as a percentage of initial roadway height with applied pressure - comparison of roof bolt lengths (applied pressure vertical to horizontal = 1:1).

Roof Bolting In Weak Strata

The qualitative scale model tests described above were all carried out using roof strata with an equivalent uniaxial compressive strength in the range 40-55 MPa. Results of tests comparing the performance of roof bolts (Appendix 2d) and steel work (Appendix 2a), in this model strata and models with a roof strength equivalent to 20-30 MPa are plotted in Figure 4.18. (The model strata configuration was similar to that illustrated in Appendix 1a except the four immediate roof slabs contained 75% as opposed to 65% sand). As was to be expected, comparisons between similar support systems show that the higher strength roof strata was more stable than the weaker model rock. Bolting the weaker roof was more effective at controlling roof lowering than using standing supports. However, the difference between the two systems was not as marked as in roadways with a stronger roof.

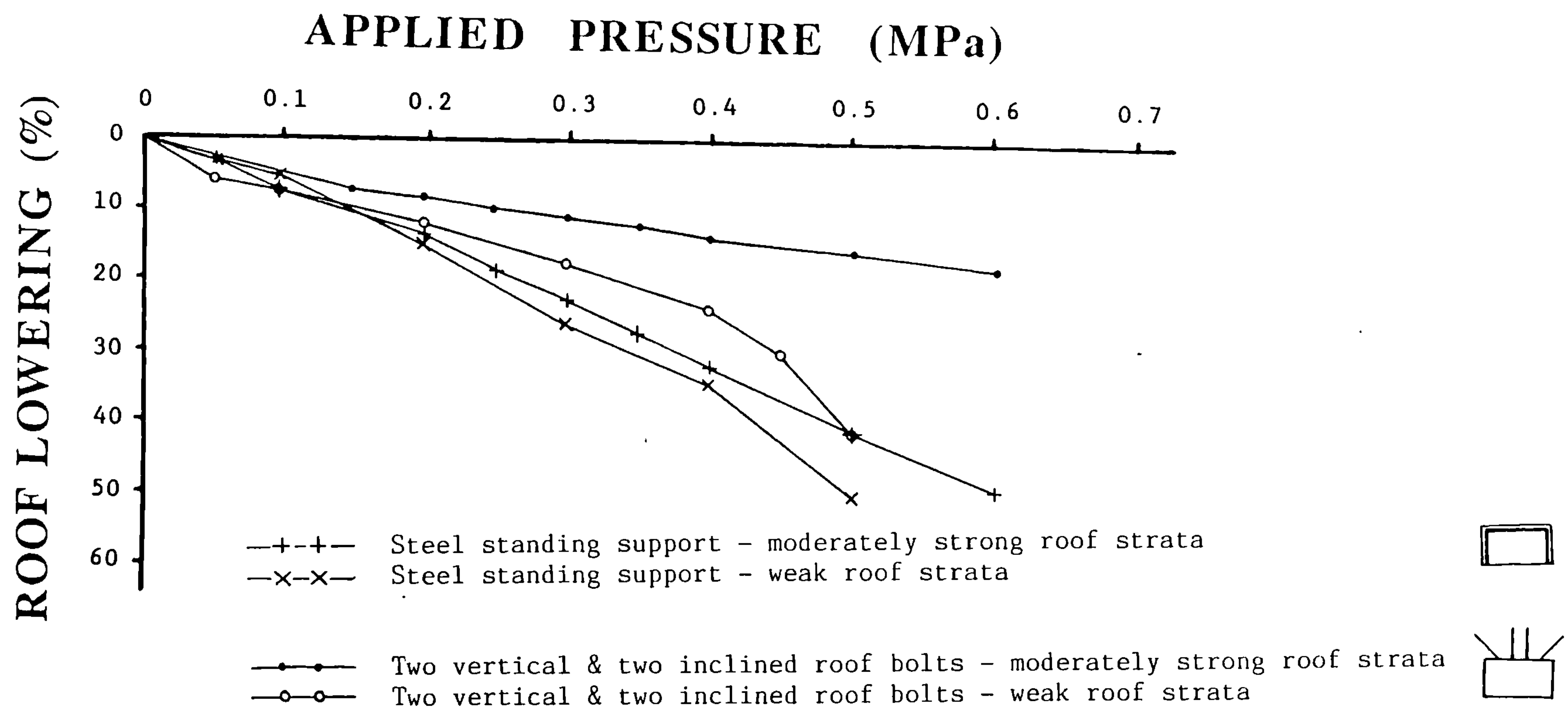


Figure 4.18 Variation in roof lowering as a percentage of initial roadway height with applied pressure - comparison of the effect of strata strength on support performance (applied pressure vertical to horizontal = 1:1).

CHAPTER 5

DESIGN THROUGH IN SITU MEASUREMENT DURING EXCAVATION

5.1 Background

Detailed systematic monitoring of the behaviour of roadway support systems and the surrounding rock mass will provide data to assess the stability of the excavation and design a suitable support system. Instrumentation and monitoring has been carried out in British coal mine roadways for many years to evaluate support performance (Potts 1955, 1957; Thomas 1966). It also forms an important part of the New Austrian Tunnelling Method, now used regularly for civil engineering projects.

5.2 Rock-support Interaction

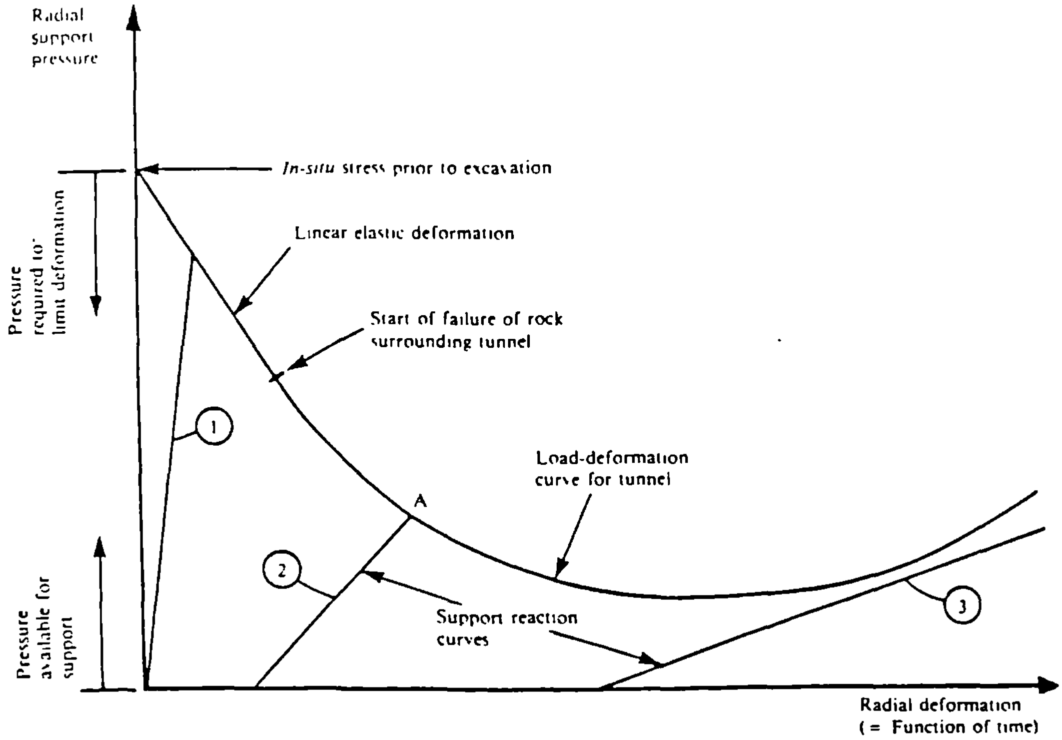
Bieniawski (1987) states that "the behaviour of an opening and the performance of the support system depend on the load-deformation characteristics of the rock and the support, as well as on the manner and timing of installation". This concept can be described by the use of ground support interaction curves as shown in Figure 5.1 (Deere et al 1970; Brown et al 1983). The curve illustrates the load that must be applied to the surface of an excavation to prevent excessive deformation.

5.3 New Austrian Tunnelling Method

Recognition of the gradually increasing deformation of excavated rock masses is the basis of the New Austrian Tunnelling Method (NATM). It was developed in the late 1950s and early 1960s (Rabcewicz 1964). NATM is not actually a construction method but more of an approach or philosophy integrating the principles of the behaviour of rock masses and monitoring the behaviour of underground excavations during construction. The essential elements of this philosophy have been outlined by Muller (1978), Brown (1981) and Bieniawski (1984), and are as follows:

(a) *Mobilization of the strength of the rock mass.*
The method relies on the inherent strength of the surrounding rock mass being conserved as the main component of tunnel support. Primary support is directed to enable the rock to support itself. It follows that the support must have suitable load-deformation characteristics and be placed at the correct time.

(b) *Sprayed concrete protection.*
In order to preserve the load carrying capacity of the rock mass, loosening and excessive rock deformations must be minimized. This is achieved by applying a thin layer of sprayed concrete, sometimes together with a suitable system of rock bolting, immediately after



Support reaction curves
(= Load induced in support by deformation of excavation.)
1. Stiff support installed too early attracts excessive load.
2. Effective support at A, pressure required to limit deformation = pressure available from support; tunnel and support system in equilibrium.
3. Ineffective support not stiff enough and installed too late.

Figure 5.1 Load - deformation curve for rock mass and support system.

face advance. It is essential that the support system used remains in full contact with the rock and deforms with it. While the NATM involves sprayed concrete, it does not mean that the use of sprayed concrete alone constitutes the NATM.

(c) *Measurements.*
The NATM requires the installation of sophisticated instrumentation at the time the initial sprayed concrete lining is placed, to monitor the deformations of the excavation and the build-up of load in the support. This provides information on tunnel stability and enables optimization of the formation of a load bearing ring of rock strata. The timing of the placement of the support is of vital importance.

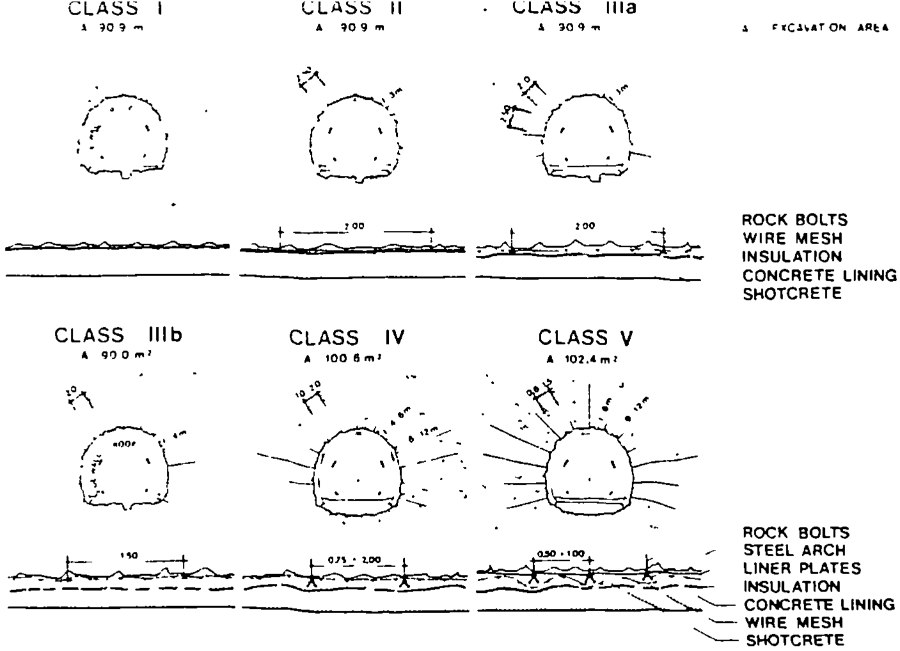
(d) *Flexible Support.*
The NATM is characterized by versatility and adaptability leading to flexible rather than rigid tunnel support. Thus, active rather than passive support is advocated and strengthening is not by a thicker concrete lining but by a flexible combination of rock bolts, wire mesh and steel standing support. The primary support will partly or fully represent the total support required and the dimensioning of the secondary support will depend on the results of the measurements.

(e) *Closing of invert.*
Since a tunnel is a thick-walled tube, the closing of the invert to form a load-bearing ring of the rock mass is essential. This is crucial in soft-ground tunnelling where the invert should be closed quickly and no section of the excavated tunnel surface should be left unsupported even temporarily. However, for tunnels in rock, support should not be installed too early as the load-bearing capability of the rock mass would not be fully mobilized. For rock tunnels the rock mass must be permitted to deform sufficiently before the support takes full effect.

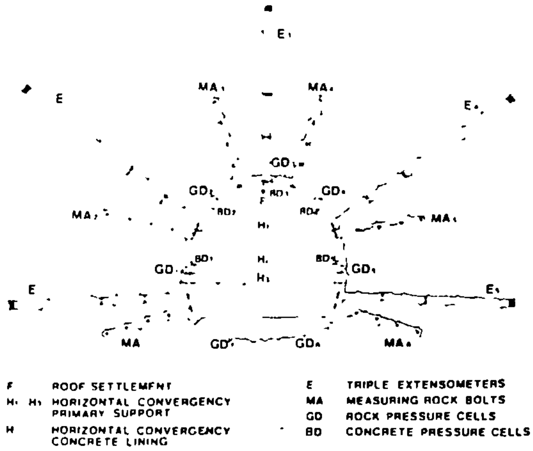
(e) *Contractural arrangements.*
The above main principles of NATM, will only be successful if special contractural arrangements are made. Since the NATM concept is based on monitoring measurements, changes in support and construction methods should be possible.

(f) *Rock mass classification determines support measures.*
Payment for support is based on a rock mass classification after each excavation round (Figure 5.2a)

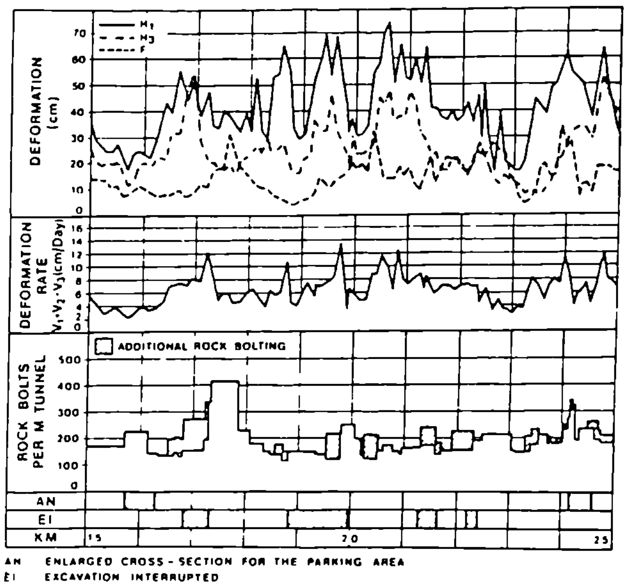
NATM is now applied worldwide in civil engineering projects, some case studies have been described by Atrott (1972), Rabcewicz and Golser (1974), Zillessen (1978), Yagi (1978), John (1980), Babenderde (1980), Daly and Abramson (1986), Wallis (1986, 1987) and Martin (1987). In addition its use in the German and Korean coal mining industries has been reported by Albers et al (1982), Spaun and Jagsch (1983), Maidl (1984) and Lee et al (1987). Some tunnelling engineers consider that the characteristics of Swellex bolts (Section 14.4) make them suitable for use with NATM. The bolts can be quickly installed directly after excavation, providing immediate support and preventing loosening of the



a. Support measures according to rock mass classification.



b. Geotechnical measurements.



c. Correlation of results of geotechnical measurements with support.

Figure 5.2 Construction of the Arlberg Tunnel (after John 1980).

rock during stress redistribution and blasting operations. Case histories using Swellex have been reported by Schmid (1986) and Muller (1987).

5.4 Roadway Instrumentation

5.4.1 Background

The amount and type of instrumentation used to monitor a rock bolt support system will vary depending on the nature of the site. Generally simple, robust instruments based on mechanical principles, that can be quickly installed, are preferable to more sophisticated electronic devices. A thorough geotechnical survey should be made at each monitoring station. Ideally this would involve drill core logging, laboratory testing and petrographic analysis of rock samples. A detailed knowledge of the geology of a monitoring station will assist in the interpretation of measurement data.

Monitoring stations should be established as close to the head end as possible. Generally measurements are taken daily for the first few days after installation, weekly for the next two or three weeks and then at monthly intervals (unless circumstances change such as the influence of abutment pressures from a longwall face). Establishing a monitoring section whenever support or excavation parameters are altered will provide the necessary data to determine the optimum roadway geometry and support system.

5.4.2 Convergence Measurement

Roadway closure can be measured directly with a tape between fixed reference points set in the strata around the roadway. A typical section will have a roof point and a floor point in the centre of the roadway as well as points located half-way up each rib. Measurement between these points and taut lines joining opposite points enables the determination of roof lowering, floor heave, right lateral closure and left lateral closure.

Greater accuracy can be achieved using a tape extensometer or conventional surveying techniques. For continuous recording of vertical closure a convergence recorder may be installed between caps on the reference points. This device is easily disturbed and should only be placed where it will not be subjected to interference (from shot firing, free steered vehicles, passing mine workers etc.).

Relative lateral movement of roof and floor strata is measured by means of a plumb-bob, lateral flow plate (or protractor) and tape. The plumb-bob is suspended from the roof reference point and hung directly over the plate located on the floor point. The postion of the plumb-bob on the plate will indicate any lateral movement by direct measurement from the centre of the plate.

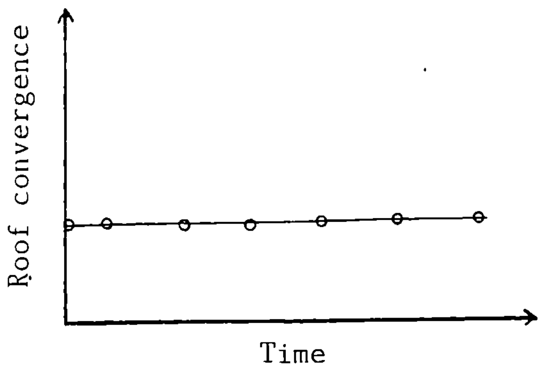
Measurements are recorded in relation to time or distance from the head end/longwall face. Plotting convergence trends will give information on roadway stability (Figure 5.3). Convergence measurements were used for design purposes during the construction of the Arlberg Tunnel in Austria, using the NATM (John 1980). If deformation rates greater than 25 mm per day occurred, additional rock bolts were installed; if deformation declined, rock bolting was reduce during further driving (Figure 5.2). The criterion adopted here are site specific and cannot be used for other rock conditions without study.

5.4.3 Borehole Extensometers

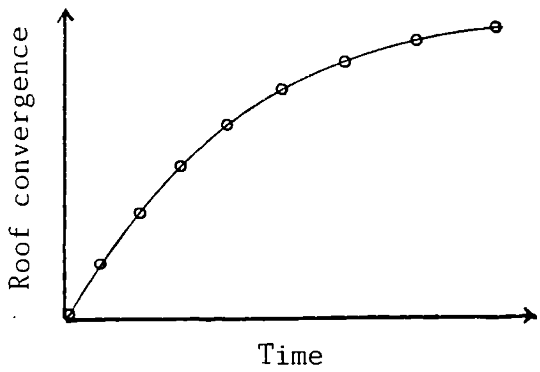
The use of borehole extensometers for pre-excavation determination of rock bolt length is described in Section 2.4. Extensometer measurements in bolted strata can give information on the development of potentially hazardous conditions at an early stage. Separation of the bolted section en masse due to fracturing along a plane just above the top of the bolts will be detected via anchors located above the bolted section. Should this occur alterations in bolt length must be made. Excessive bed separation within the bolted section necessitates increasing the rock bolt density and/or installation of additional standing support.

There are several different extensometer anchors available. The anchor system used is determined by the nature of the strata and type of extensometer.

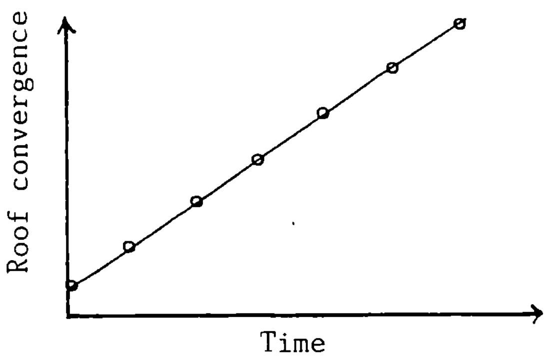
Wooden wedge anchors are only suitable for use as a top anchor. They are simple and inexpensive but can be a problem to install. A short length of wooden dowel is cut diagonally and held together with tape. It is pushed to the top of the borehole using a rod, a sharp tap then breaks the tape and the bottom section is forced up as a wedge to form the anchor.



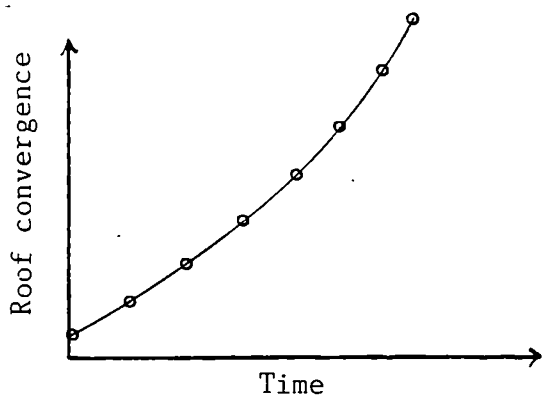
a. Stable situation.
Can measure less often.



b. Becoming stable.
Can measure less often.



c. No decrease in rate.
Must watch closely.



c. Rate increasing.
Potential instability,
additional support required.

Figure 5.3 Convergence trends.

Spring anchors can vary in complexity from simple torsion springs or spring clips to anchors with several trailing leaf springs. They are easily positioned in the borehole using a plastic pipe or length of conduit.

Expansion shell anchors are generally limited to single or double position extensometers. They consist of mechanical expansion shells similar to those used for point anchored rock bolts (Section 7.3). They can be rapidly installed and are particularly suited to rough, uneven boreholes in fractured strata.

Snap-ring anchors are positioned through removal of a locking pin by pulling a cord which allows retaining rings to snap outward and grip the borehole. These anchors are most suited to smooth uniform boreholes.

Grouted anchors require pre-assembly of the extensometer system before insertion, which can be time consuming. Actual installation is accomplished quickly if the proper procedures are followed. No-shrink cement grout or resinous grout can be used. This type of anchor will not slip under tension in blasting areas.

Borehole extensometers can be classified as either single- or multiple-point devices. Some of the more commonly used types are described below.

Single point extensometers simply consist of a length of wire or rod anchored in a borehole (a point anchored rock bolt without an end plate can constitute such an extensometer). Changes in the distance between a reference point on the end of the wire or rod and the borehole collar are indicative of ground movement taking place between the anchor and the rock surface.

Single point Swellex extensometers consist of a modified Swellex bolt (Section 14.4) which acts as a single point rod extensometer (Atlas Copco 1982a). The upper 0.5 m of the bolt is installed as a point anchor by fitting a thin steel tube over the remainder of the bolt to prevent expansion during installation. The lower bushing is not welded to the bolt so that as the strata separates, the bushing is pushed along the bolt. The bushing end is machined to give a surface suitable for accurate measurement using sliding calipers. The ground movement monitoring bolts are either the same length as the standard bolts to measure rock movement within the bolted section or longer to record movement above the bolts.

Multiple-point tensioned wire extensometers consist of lengths of high strength stainless steel wire connected to each anchor and tensioned by either dead weights, coil or leaf springs, cantilevers, or constant-tension clock-type springs at the mouth of the borehole. The simplest type utilizes spring clip anchors in a vertical borehole. Wires from the clips are installed through a reference plug at the borehole collar; a constant tension is maintained by a weight attached to the end of the wire. Movement of a reference point on the free end of the wire relative to the reference plug indicates strata displacement. Measurement of this distance can be made with a vernier caliper or graduated scale. More sophisticated devices are commercially available whereby the wires are tensioned by spring cantilevers in the sensing head at the borehole collar. Anchor movements will either stress or release the wires. Transformers attached to the cantilever beams convert the mechanical movement to changes in electrical quantities; electrical resistance or vibrating wire strain gauges are commomly used for this purpose. Neff (1970) has described the necessary calibration procedures and sources of error incurred with these instruments.

Multiple rod multiple-point extensometers are simple, accurate and generally very reliable. Each measuring rod is freely suspended from an anchor. The rods follow the anchor movements which are sensed by measuring the position of the tip of the rod relative to a reference plate at the mouth of the borehole. This movement can be read mechanically with a depth micrometer, or electrically using a linear voltage displacement transducer, rotary or linear potentiometer. Measurement errors due to rod corrosion are prevented by using a stainless steel tip. If the anchors are secured by a grout, the rods should be encased in a pipe or tube to prevent contact with the grout. The number of anchors within a rod extensometer is limited by the diameter of the borehole. A 43 mm diameter hole is only capable of containing four rods (Whittaker and Scoble 1980); ten point multiple rod extensometers are available for boreholes with a diameter greater than 60 mm.

Single rod multiple-point extensometers have a high resolution and consist of a series of electrical transducers mounted between anchors within a borehole which measure relative displacement between these points (Bourbonnais 1985). The measuring systems are in water tight casings inside the hole and consequently not as susceptable to shock waves created by blasting as borehole collar electrical measuring devices.

Multiple-point magnetic extensometers have been used successfully in many British coal mines over recent years. Strata deformation is determined by monitoring the location of magnetic targets within a borehole. A magnetic ring, cut from magnetized strip, is fitted to a spider anchor consisting of a cylindrical PVC mount with multiple trailing leaf springs. An open reed switch fixed to the end of a series of probe rods closes on entering the magnetic field induced by a ring magnet. This activates an indicator light on an intrinsically safe multimeter connected to the probe. The position of a magnet is measured (by means of a tape attached to the rods) at a reference point on the free end of a pipe which runs the runs through the rings and is fixed at the top of the hole (Figure 5.4). In the majority of field conditions this gives a reading accuracy of ±1 mm. A micrometer system can be used for greater accuracy. Lateral movement of strata may prevent access up the pipe.

5.4.4 Alarm Systems

Single-point extensometers can be modified to act as an alarm system to give a visible warning of impending failure of the roof. Alarm systems are ideally simple, inexpensive and easy to install, but are only capable a specified degree of rock movement. The amount of deformation occurring before rock failure will vary from site to site, consequently for an alarm system to be effective the degree of rock movement that can occur before conditions become hazardous must be determined at each location by experience through detailed monitoring programmes. Some alarm systems that have been used in US mines are described below.

Roof sag bolt: A bolt point anchored in stable strata above the bolted section. Three strips of different coloured reflective tape are attached adjacent to each other at the end of the bolt. As roof lowering occurs each strip is progressively hidden behind a plug at the borehole mouth.

Glowlarm: A flexible translucent plastic tube containing two chemicals in separate glass amphiboles is installed on a metal sling tight against the rock surface, suspended from a wire point anchored in stable strata

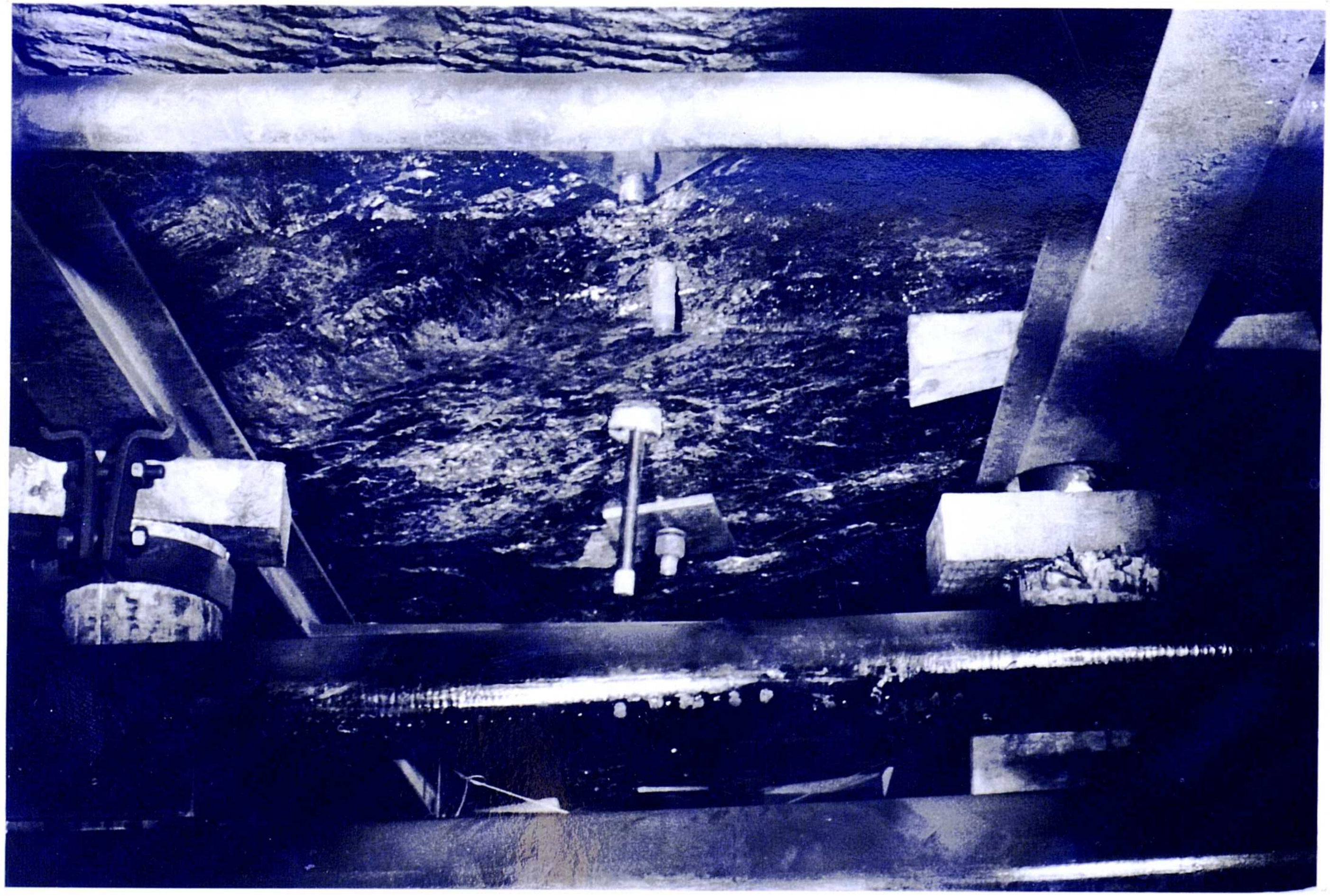


Figure 5.4 Roadway monitoring station (roof)

above the bolted section. Rock deformation bends the device, breaks the glass and mixing of the two chemicals produces a bright yellow light which lasts up to 24 hours.

The Spider and the Guardian Angel: Installed on the end of a bolt point anchored in stable strata at least 0.3 m above the bolted section. A specified amount of roof deformation releases a reflective drum or flag (Guccione 1978).

5.4.5 Borescopes

A borescope is an optical viewing instrument for visual or photographic observations in a borehole. There are a variety of instruments known by names such as, introscope (Thomas 1966), stratascope (Fitzimmons et al 1979), endoscope (Dejean and Raffoux 1980b), petroscope (Adams and Jager 1980), borescope (Mahtab et al 1973) and the ST-6 Arvin Diamond TV camera (Herget 1982). The most versatile of these are the flexible fibre-optical instruments operated using a standard colliery cap lamp and battery as a power supply.

Specific problems that can be tackled using borescopes have been discussed by Shepherd et al (1986) and are as follows:

(a) Mapping lithological variation in a mine roof.

(b) Identifying fracture distribution and type in relation to lithologies and making inferences about roof failure mechanisms.

(c) Assistance with determining support requirements, especially bolt length and checking the efficacy of bolting together with other techniques such as bolt load tests.

(d) Checking and monitoring of roof stability, used in conjuction with extensometer and convergence measurements.

(e) Analysis of spatial and temporal distributions of roof fractures, especially for examining the effects of higher loading adjacent to goaf edges and in longwall gateroads.

5.4.6 Rock Bolt Load Measuring Techniques

Additional support or a reassessment of existing rock bolt parameters may be required at locations where bolts are heavily loaded. Instruments for measuring the load acting on a rock bolt can be classified into one of two general categories; either, external and single point internal devices, or axial and multi-point internal devices. Details of commonly used intruments in both of these categories are given in the following paragraphs.

EXTERNAL AND SINGLE POINT INTERNAL DEVICES

External and single point internal rock load measuring devices can be used to determine:

(i) the load acting on standard tensioned point anchored bolts, where this type of bolt is acting as the primary support;

(ii) the bonding capability of specific target horizons when installed on bolts point anchored in these zones (Section 2.3);

(iii) the load acting on a tensioned bolt point anchored beyond the zone of full column grouted rock bolts;

(iv) the load acting on the bearing plates of full column grouted bolts.

A method for monitoring rock bolt tension using external devices (load cells) has been suggested by the ISRM (1981). Each individual dynamometer must be calibrated before installation, even virtually identical instruments may have slightly different calibration curves. The rock bolt should be tensioned on installation and the load cell positioned so that the load acts along the cell axis; spherical seatings will help to restrict the effects of eccentric loading.

Torque Wrench

Torque gain due to bolt loading or torque loss resulting from improper installation or anchorage slip can be determined for point anchored rock bolts using a simple torque wrench (Barry et al 1953, 1954b, 1956). Friction and other problems have limited the use of this technique (Babcock 1977). Torque levels for pre-tensioned fully grouted rock bolts can also be checked using this technique (Section 8.1.6).

Proving Ring Dynamometer

This is a precision elastic load measuring device. The name is derived from its use for the "proving" of load in a testing machine. It consists of a high quality steel ring loaded diametrically through special loading plates. Deformation of the ring is measured either by a dial gauge (Cyrul 1985) or a pair of strain gauges (Unrug 1986). Measurement of the mean strain of two gauges will eliminate errors due to mis-alignment of the ring during installation. Proving rings with a dial gauge indicator are very low cost instruments. Field and laboratory investigations by Unrug (1986) concluded that their sensitivity is comparable with very sophisticated and expensive dynamometers. However, a dial gauge is not perfectly linear in the measurement of displacement, consequently a micrometer should be used for very high accuracy instrumentation.

Rubber Compression Pad

The roof bolt compression pad is an inexpensive load measuring device developed by the USBM in 1951 (Obert and Barry 1955). It consists of a rubber disc located between two steel plates. Loading of the pad causes changes in the circumference of the rubber membrane which is measured with a calibrated ring gauge. These readings are converted to bolt load values by reference to a calibration chart (Sen 1958). The compression pad is a very low precision instrument; Tadolini and Ulrich (1986) quote an accuracy of ±0.9 kN at low loads, ±45 kN at high loads and a working load limit of 142 kN. The material properties of rubber limit the use of compression pads particularly where cyclical loading may occur. Laboratory tests by Cyrul (1985) have shown classical primary and secondary creep of a rubber used for dynamometers.

Photoelastic Dynamometer

The design and development of a number of types of photoelastic transducer was undertaken by research workers at the University of Sheffield Postgraduate School in Mining and has been reported by Roberts and Hawkes (1963, 1965). A photoelastic rock bolt dynamometer is a robust, self contained device. It consists of a hollow steel cylinder containing a glass disc. When loaded diametrically the disc is strained. The number of photoelastic fringes revealed when observed with an optical viewer can be related to the load on the cylinder. With experience it is possible to read the instruments to an accuracy of 2.5 kN. A maximium of six fringes can be distinguished which limits the load range; one supplier markets a dynamometer pre-set in 70 kN increments within a 0 to 150 kN range (Perard Torque Tension 1985). Remote reading of these instruments is not possible and the necessity to gain close access to the dynamometer for reading creates major limitations.

Hydraulic Load Cell

Hydraulic load cells are generally robust and reliable instruments. The annnular ring cell is filled with de-aired hydraulic fluid. When a compressive load is applied the pressure in the fluid changes. This is measured by a pressure gauge fitted to the cell or a diaphragm transducer; remote monitoring is possible by using an electronic pressure transducer. The flatjack U-cell used by the USBM can measure loads of up to ±90 kN and are accurate to ±2.2 kN (Chekan and Babich 1982).

Annular-ring Vibrating Wire Dynamometer

Vibrating wire load cells are high resolution instruments. A number of vibrating wire transducer elements are positioned longitudinally equidistant around the mid-circumference of a steel cylinder. Changes in the natural frequency of vibration of a stretched wire within the elements occurs when the cell is loaded. A square law exists between strain change and observed frequency change. Readings from vibrating wire strain gauges are unaffected by cable length and are therefore suitable for use in remote monitoring.

Electrical Resistance Strain Gauged Load Cell

Electrical resistance strain gauges are a form of transducer which converts a dimensional change into a resistance change. The most accurate dynamometers are set in a full Wheatstone bridge configuration to compensate for temperature changes and eccentric loading. The use of high resistance strain gauges will minimize cable effects. A simple rock bolt dynamometer was developed by the NCB Mining Research Establishment in the late 1950s to early 1960s. It consists of a collar containing two vertical sensite gauges and two horizontal compensating gauges. Under axial load the resistance of the longitudinal gauges decrease owing to axial shortening, while the resistance of the circumferential gauges increases. To measure this, a voltage is applied and the resistance compared with standard resistances using a transistor dynamometer test set. Thus readings can only be taken by trained personnel with the necessary equipment. A more sophisticated dynamometer was developed (Potts 1957; Smith and Pearson 1961) by the Department of Mining Engineering, King's College, University of Durham (now University of Newcastle Upon Tyne). It used eight 340 Ω gauges, four active and four compensating, connected to a four arm Wheatstone bridge circuit. The development of another full-bridge dynamometer by the USBM, using a titanium diaphragm, has been described by Beus and Phillips (1974). It has a usable range of ±89 kN and is accurate within ±1% for vertical loading and ±5% in angle loading (Langland 1977).

Vibrating Wire Instrumented Rock Bolt

Mounting a dynamometer within the bolt head reduces the risk of instrument damage by mining operations. Dynamometers are available suitable for coupling to a standard rock bolt within a borehole. They contain a miniature vibrating wire strain gauge transducer positioned in a hole along the central axis of a cylinder. This system has the same advantages as the external vibrating wire dynamometers; it works on a similar principle although it is primarily designed to measure tension rather than compression. Bellier and Debreville (1977) describe a 150 kN internal dynamometer that has a stress sensitivity of approximately 0.2 MPa. Another similar type of vibrating wire instrumented bolt which can be read by touching the bolt head with a hand-held probe has been evaluated by Maleki (1985).

Bolt Surface Strain Gauges

A simple internal load measuring device can be constructed by mounting two strain gauges (vibrating wire or resistivity) 180^0 apart on the bolt collar. However, electrical leads from the gauges may be a problem during bolt installation.

AXIAL AND MULTI-POINT INTERNAL DEVICES

Axial and multi-point internal devices are capable of measuring the load acting on a full column anchored rock bolt.

Single Point Rock Bolt Extensometer

The mean load acting on a rock bolt can be determined by measuring its total extension. This may be achieved by using a standard rock bolt with an axial hole along the centre. An unstressed rod is fixed in the hole by a weld at the far end. Measurement of relative movement between the exposed end of the bolt and the free end of the rod gives the total bolt extension (Ward et al 1976). The measurement is taken simply with the end of a caliper or a dial gauge screwed onto the bolt. An electrical transducer can be attached for remote monitoring. Altounyan (1986) has developed a spring loaded 10 kΩ linear potentiometer suitable for single point rock bolt extensometers which are read using an intrinsically safe multimeter, portable data logger or through connection to a MINOS outstation for transmission to the surface.

Multi-point Rock Bolt Extensometer

This is a similar device to the single point rock bolt extensometer except it has several unstressed rods (Bellier and Debreuille 1977) or wires (Farmer and Shelton 1978) fixed at intervals within a hollow bolt. It is capable of determining the position of zones where load is taken up along the bolt and can therefore be used to assess the most favourable bolt lengths. Laboratory calibration of simple mechanically measured wire rock bolt extensometers used by Farmer and Shelton (1978) showed they were sensitive to ±1 kN.

Resistance Strain Gauged Rock Bolts

Electrical resistance strain gauges positioned at a number of points along the length of a full column anchored rock bolt can give information regarding axial strain and force, bending strains, and shear stresses generated along the rock bolt. The strain gauged bolt developed by the US Mining Enforcement and Safety Administration (MESA) has been described by Sawyer and Karabin (1975) and Sawyer and Eakin (1976). Prototype bolts were produced which used quarter Wheatstone bridges. These gauges did not compensate for temperature changes or nullify bending stresses induced during installation. The final design adopted had temperature-compensating, half Wheatstone bridges. Two gauges were placed on each arm of the bridge with each gauge mounted in diametrically opposite positions to nullify the effects of bending. Three sets of these gauges were located along a bolt. Similar designs of strain gauged bolts have been employed worldwide. For example, their use has been described by Wade et al (1977), and Patrick and Haas (1980) in the USA; Bello and Serrano (1974) in Mexico; Walton and Fuller (1980), and Gale and Fabjanczyk (1985) in Australia; Bjornfot and Stephansson (1983) in Sweden and Freeman (1978) in the UK. Resistance strain gauged rock bolts installed in harsh underground environments can prove unreliable, as was found to be the case in the construction of the an inset of North Selby Colliery (Tully 1985). Plots of axial forces and point bending moment developed by a set of bolts, contoured above the roadway, will delineate the geometry of loading (rock failure) and the location of shear reinforcement respectively (Gale 1986). These plots can be used to determine the onset of bolt yield and the need for additional reinforcement or alteration of the installed bolt length/ orientation.

5.5.7 Standing Support Load Measuring Techniques

Reaction loads generated on free standing supports can be determined from load measuring devices placed on or under the support.

Hydraulic and electrical resistance strain gauged load cells are available. These are placed underneath steel supports and normally located in a carrier. Where wooden supports are used under girders in rectangular roadways, the load cells may be placed between the top of the post and the girder (Figure 5.4). This will provide easier access for measurement and recovery of the cell.

Strain gauges such as the surface mounted vibrating wire type, provide an alternative to load cell as a means of measuring loads on steel work. They are generally inexpensive, simple to install and highly accurate. Intrinsically safe readout equipment is available. Interpreting the measured data may be problematical due to complex bending and twisting loads.

CHAPTER 6

PRELIMINARY INVESTIGATIONS FOR ROCK BOLTING SYSTEMS IN THE DEEP HARD/PIPER SEAM IN NOTTINGHAMSHIRE

6.1 Background

The neighbouring collieries of Mansfield and Sherwood are at present working the 2.3 m to 3.0 m thick Deep Hard/Piper Seam at depths of between 500 m and 600 m. Information gathered from preliminary investigations for rock bolting systems at these collieries has been used to evaluate some of the design methods discussed in Chapters 3 and 4 and to make recommendations concerning rock bolt parameters and monitoring methods for futher design by in situ measurement.

As subsidence is a very serious problem in the Mansfield conurbation, methods of mining have been designed to overcome this particular disadvantage. Both collieries have planned a system of single entry retreat workings (Figure 6.1), which entails retreating down a pre-driven roadway, allowing it to collapse behind the face-line. With a one-road system, the stability of that roadway is vitally important. At Mansfield Colliery the use roof bolts is being considered, as a reinforcement supplementary to the existing flat topped roadway supports. This system has been designed to use 45 m long faces and initially 45 m wide pillars which should considerably reduce surface subsidence.

At Sherwood Colliery an additional method of working has also been adopted. It involves driving a series of 5.5 m wide headings by means of a Dosco In Seam Miner 3000 giving a partial extraction system (17% extraction) anticipated to eliminate any surface subsidence (Figure 6.1). If this method of mining is successful, vast areas of coal could be released from sterilization. It is intended to support the wide roadways with bolts and rectangular shaped standing supports as indicated in Figure 6.2.

The preliminary investigations have involved engineering geological mapping of the roof strata and a programme of mine roadway instrumentation and monitoring. The position of the measuring sections are given in Figure 6.1. Convergence stations were established with short point anchored bolts to monitor roof lowering, floor heave and lateral closure of the openings. Multipoint magnetic extensometers were installed in the roadway centre to measure and precisely locate points of differ-

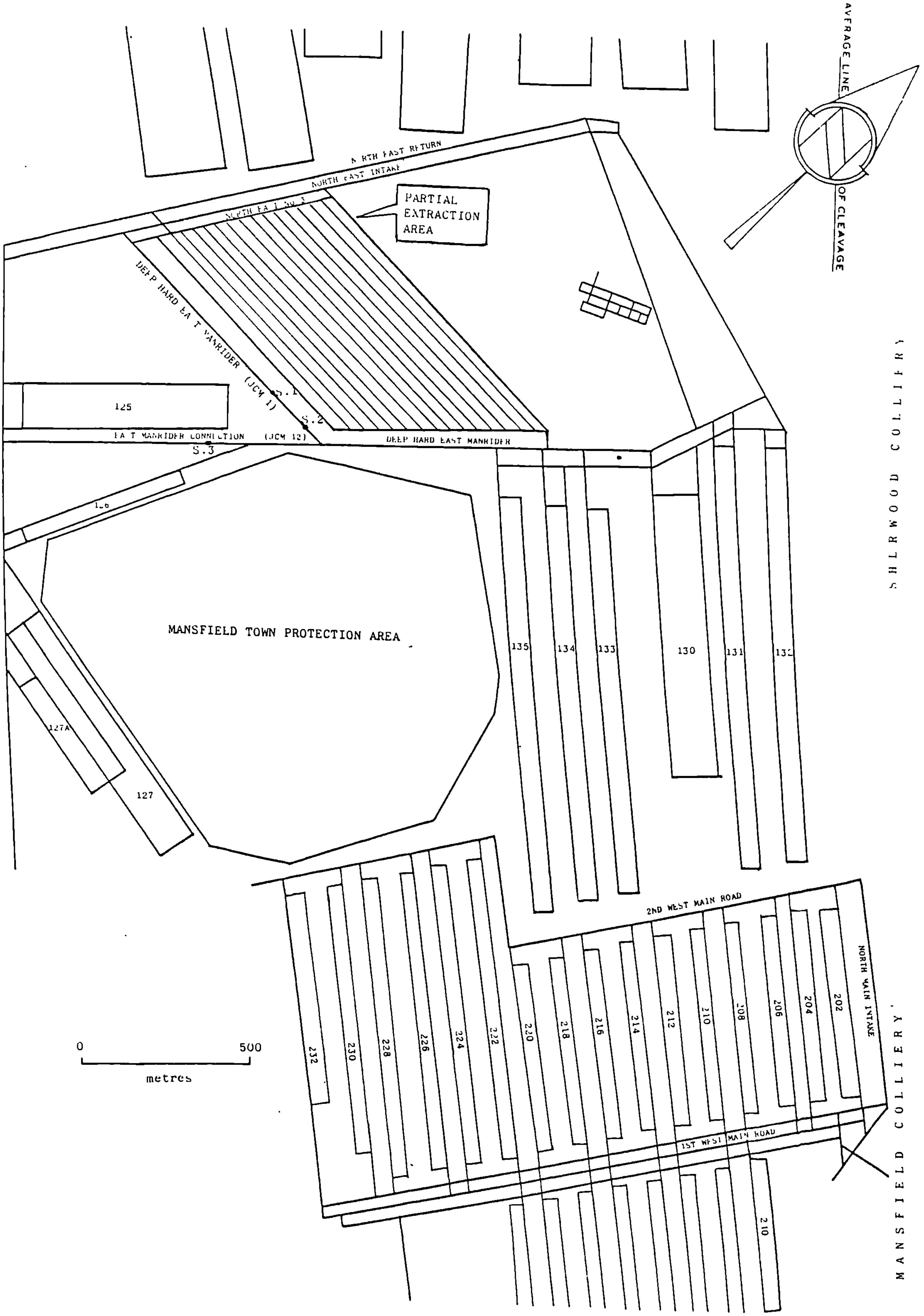


Figure 6.1 Sherwood/Mansfield Colliery Deep Hard/Piper Seam.

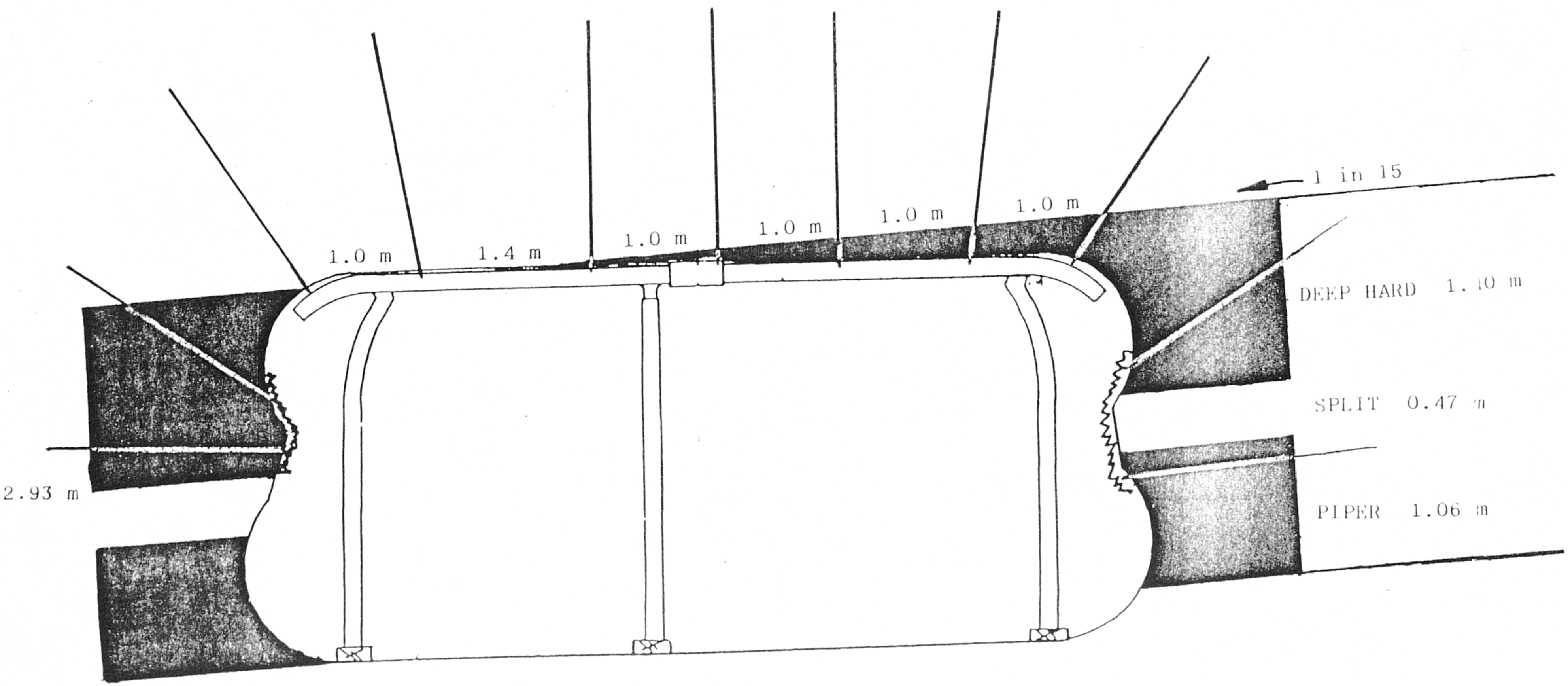


Figure 6.2 Deep Hard East No.2 ISM 3000 Head - scale 1 to 50.

ential strata deformation. In addition, pressure cells were placed along the RSJ girder to determine support loading. The field investigations were backed up by laboratory testing of rock specimens, stress abutment calculations and a series of scale model tests.

Roof conditions in the Deep Hard/Piper seam are generally very sound, however, this study has revealed that there are several factors which could lead to unstable roof conditions within this seam at Mansfield and Sherwood Collieries.

6.2 Geotechnical Evaluation Of Roof Strata

The roof consists of a medium to silty mudstone overlain by a fine siltstone which is covered by a fine sandstone or siltstone with sandstone laminae and layers.

There are well developed parting planes present in the roof. Two principal parting planes occur at lithological boundaries which are generally present throughout the area of the Deep Hard/Piper combined seam at the two collieries. Positions of the two planes relative to the top of the seam are shown in Figure 6.3.

The first parting plane occurs at the mudstone - fine siltstone interface and is sometimes poorly developed. It generally lies between 0.5 m and 0.65 m above the seam, although localised thinning of the immediate mudstone unit can result in it descending to within 0.15 m of the seam. Bed separation at this horizon can lead to collapse of the immediate mudstone unit. However, allowing the roof to break away at this level during drivage can assist in achieving a consistent and regular roadway profile to which flat topped supports can be set. There is often a very well developed second parting plane along an erosion surface at the top of the fine siltstone bed (Figure 6.4). It is usually present at between 2.3 m and 2.5 m above the seam; however, localised thinning or thickening of the underlying mudstone/silty mudstone and fine siltstone units can reduce or increase its position above the seam. Bed separation can occur at this parting resulting in excessive loading on supports.

Laboratory tests have been carried out on roof samples collected from 202's roadway and 131's Main Gate. Rock has also been tested from boreholes drilled into the roof at 130's and 127A's face end lines.

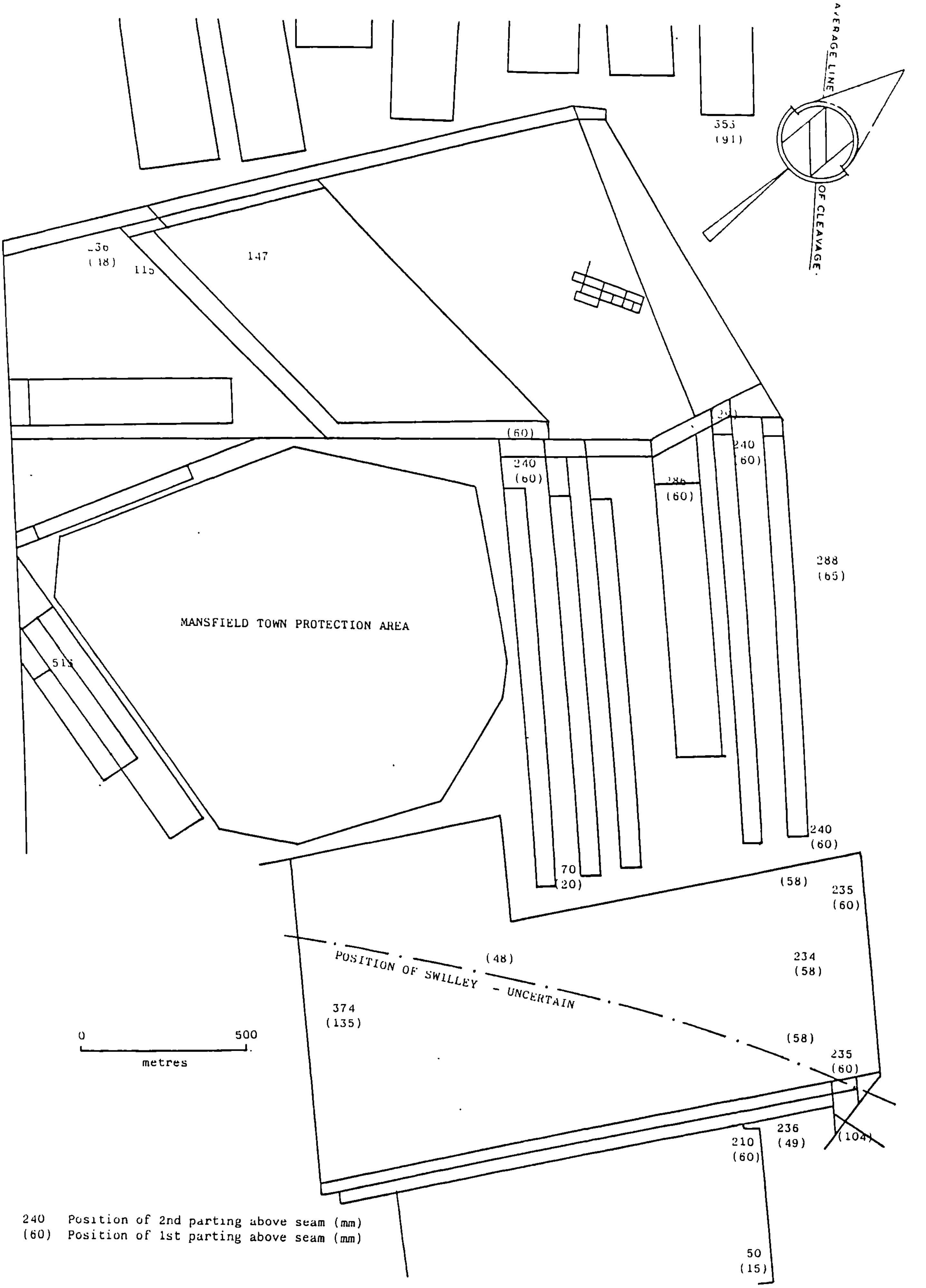


Figure 6.3 Position of parting planes above Deep Hard/Piper Seam.

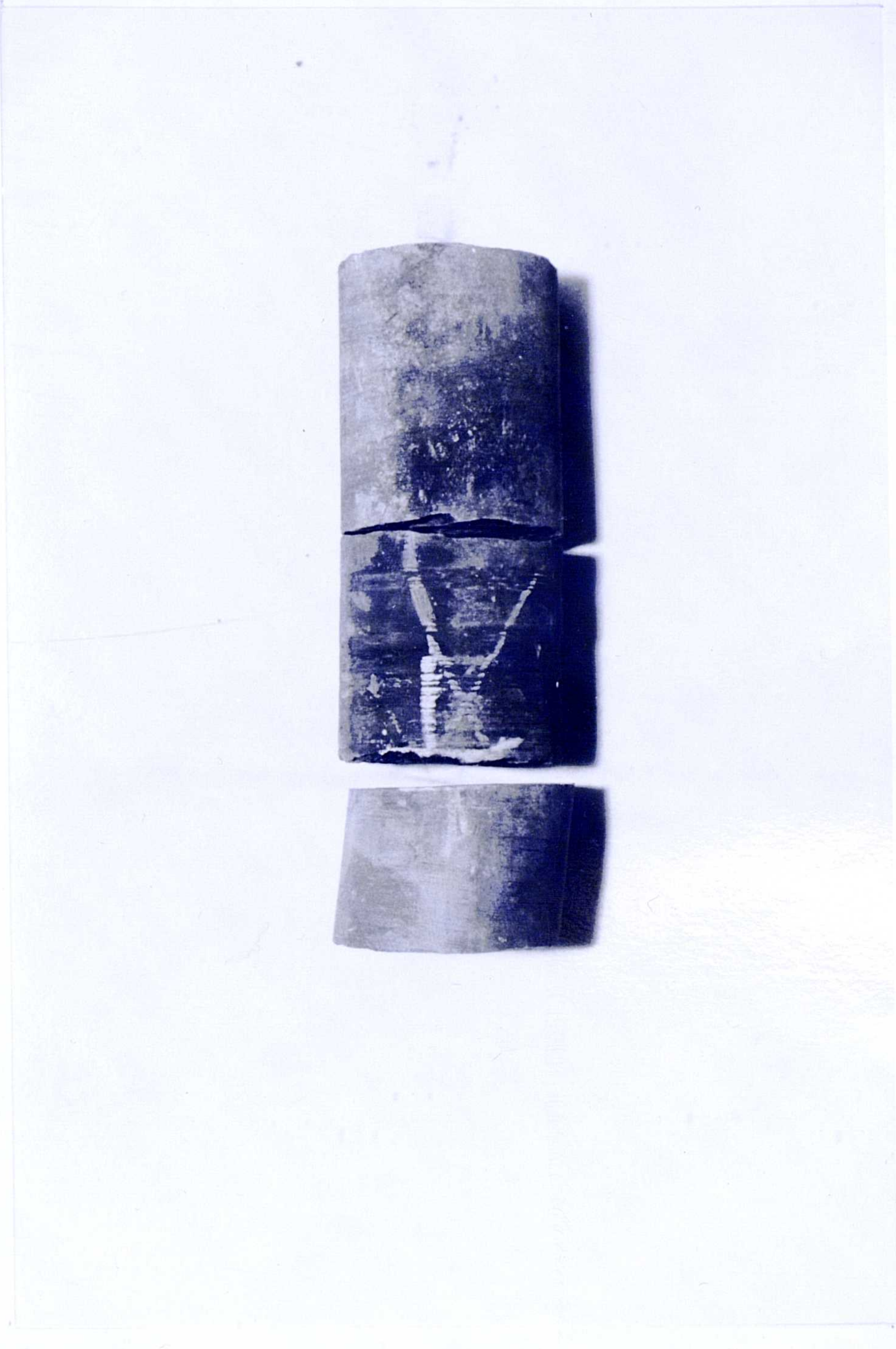


Figure 6.4 Parting plane along erosion surface

Figures 6.5a, 6.5b and 6.5c are geotechnical logs of the three vertical boreholes. All the holes were drilled a considerable time after the roadway or face was excavated. The immediate mudstone was absent, it had probably broken away to the first parting plane, leaving an open hole at the base of the cores. Bed separation had resulted in missing core in the 130's boreholes between 1 m and 2 m above the roadway.

The cores were highly fractured (Figure 6.6). It was impossible to distinguish between natural or mining induced fractures and those caused by man handling of the core during and after drilling. Before arriving at the testing laboratory the cores had been left on the surface in sub-zero temperatures for several days; this may have had an adverse effect on the rock. The values for RQD and fracture spacing obtained from the borehole cores were therefore highly distorted. In addition, corrugated sheet lagging above the standing supports in the in-seam roadways and wooden boards behind the arches in the drifts down to the seam, obscured large exposures of Deep/Hard Piper roof rock, preventing the use of scan line techniques for fracture logging. Realistic values of RQD and fracture spacings of 60% and 150 mm respectively were estimated from the limited exposures of roof strata. The high degree of fracturing, particularly in the lower sections of the core, limited the number of standard sized test pieces that could be obtained. Those that were tested probably came from relatively competent parts of the sample section, such as ferruginous bands. However, it would appear that the roof strata, when unweathered, is of adequate strength in both tension and compression.

Slake durability tests on the fine siltstone from the boreholes suggest that this stratum can deteriorate rapidly in the presence of water. The samples tested underwent partial or completed disintegration after a week of immersion.

Relatively low values from toughness and abrasivity tests obtained for the mudstones and fine siltstone units indicate that bolt hole drilling and excessive drill bit wear should generally not be a problem. However, where drilling into the overlying sandstone/siltstone bed is necessary, rapid bit wear could occur.

Swilley structures have been observed in several seams within the Nottinghamshire coalfield (Elliott, 1965). A major swilley has been proved in the initial Deep Hard/Piper developments at Mansfield and by

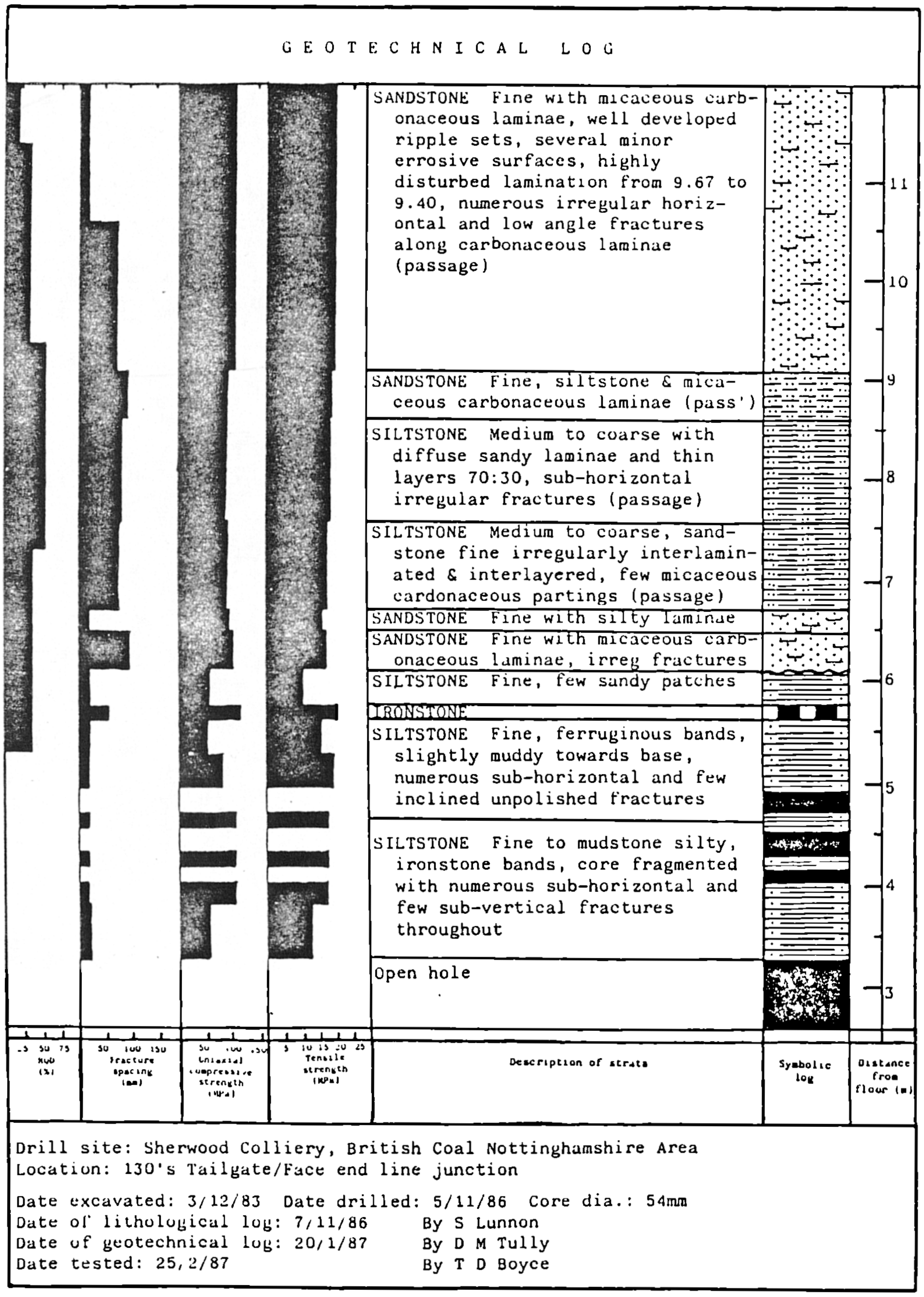


Figure 6.5a Geotechnical borehole log - 130's tailgate/face end line junction.

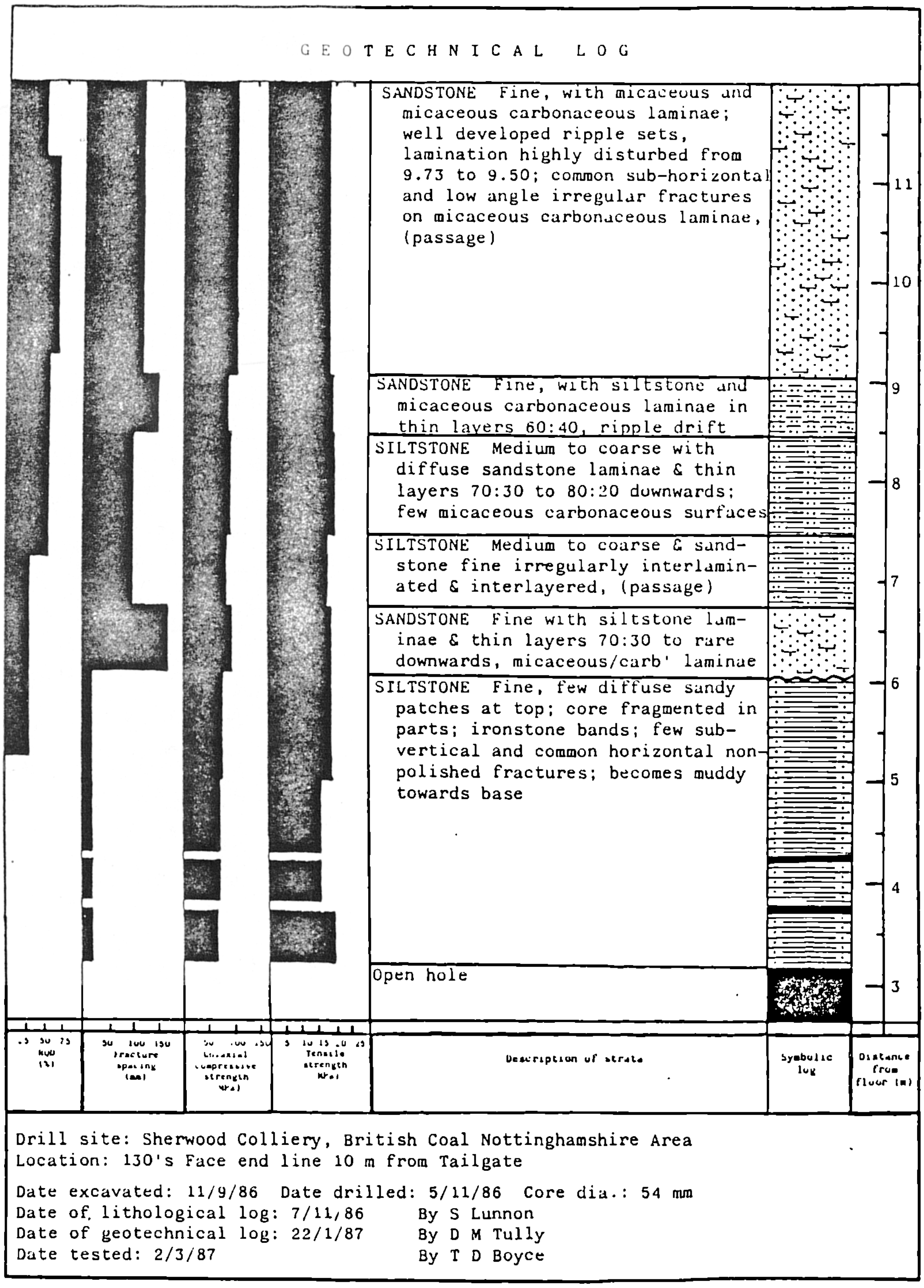


Figure 6.5b Geotechnical borehole log - 130's face end line 10 m from tailgate.

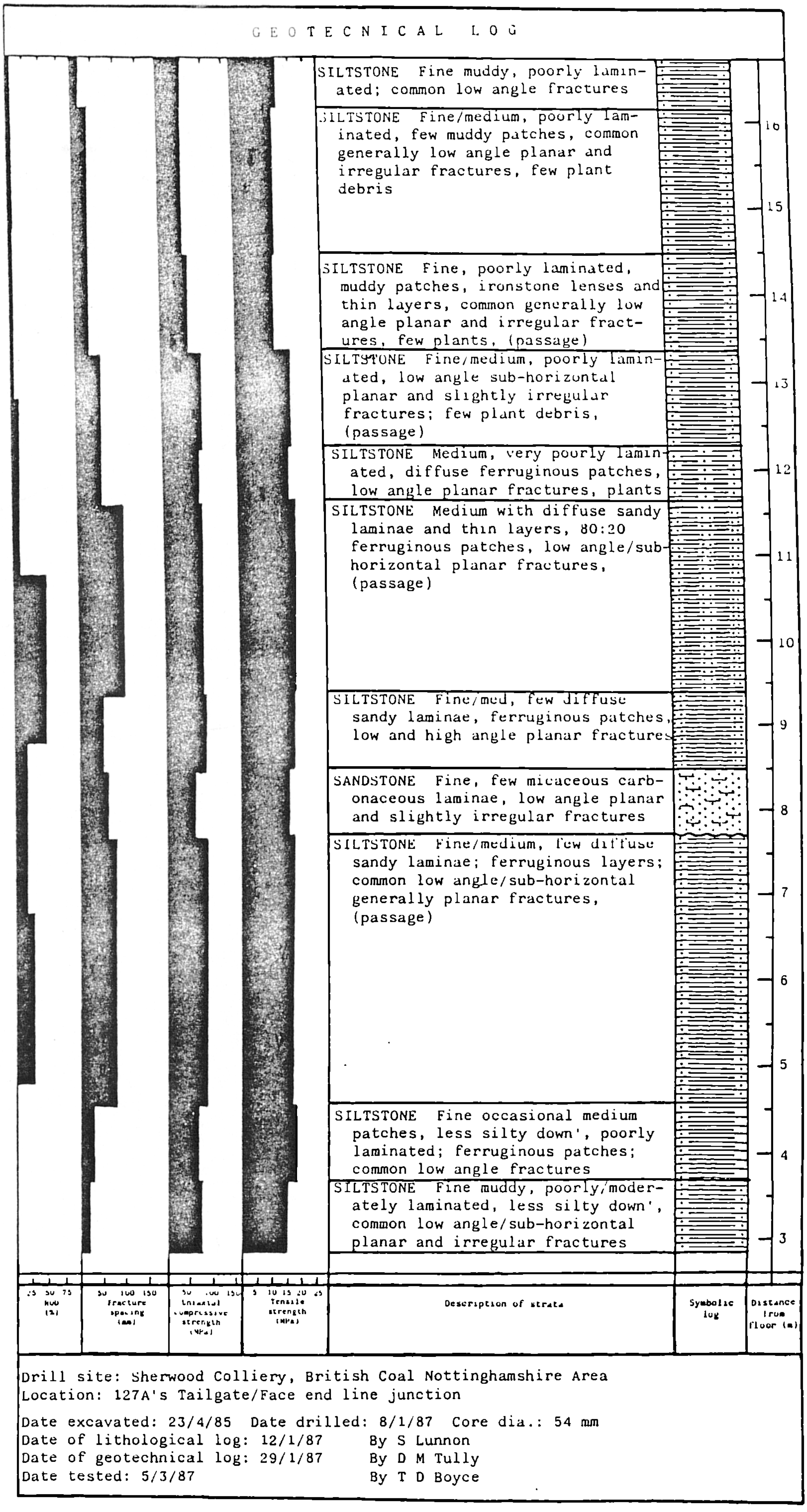


Figure 6.5c Geotechnical borehole log - 127A's tailgate/face end line junction.

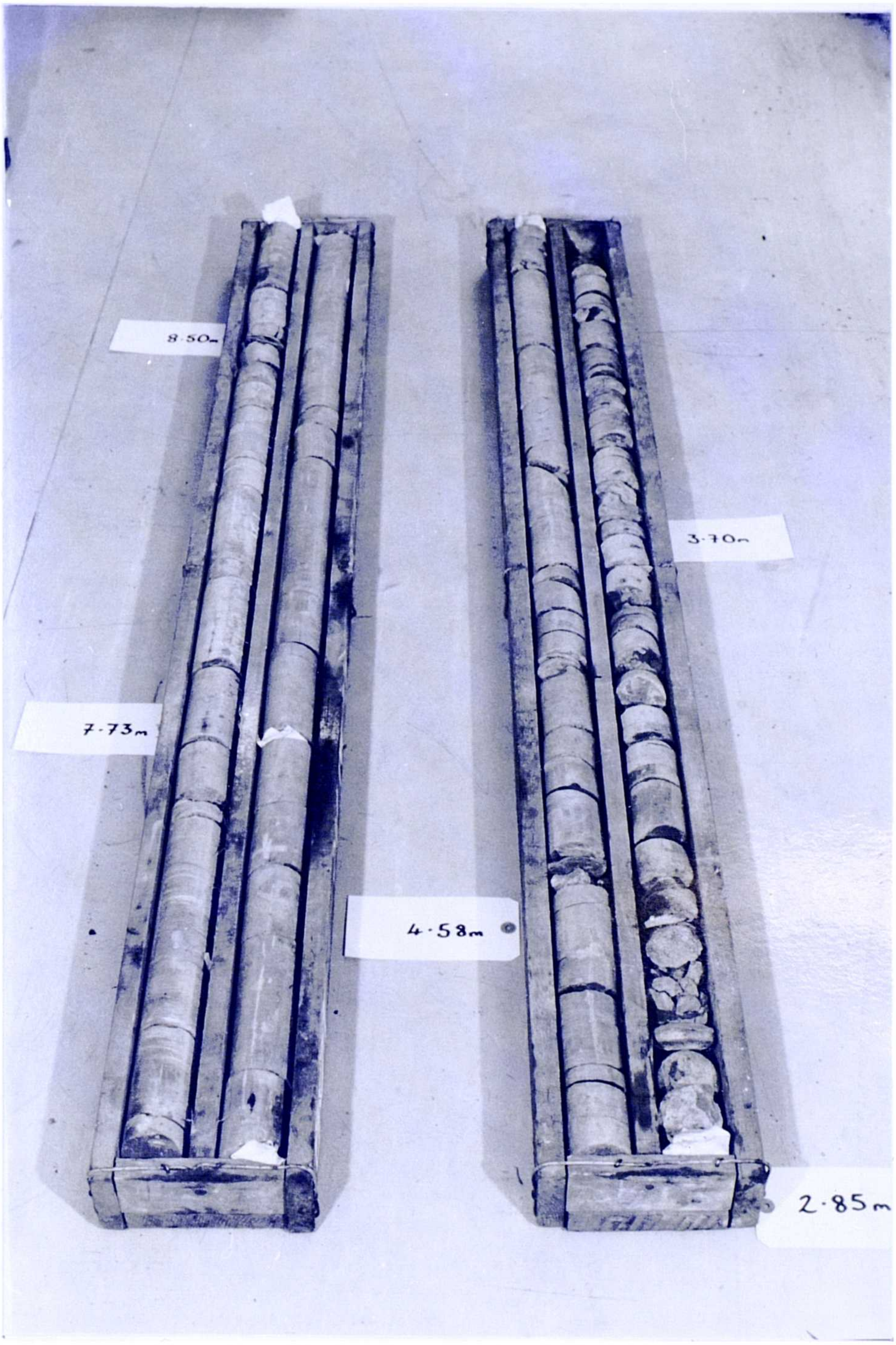


Figure 6.6 Borehole core from above 127A's Tailgate/Face end line junction

202's/204's faces. The swilley is of fairly low amplitude but has resulted in a large increase in seam thickness. Total seam sections of over 3.5 m have been recorded in the swilley centre and sections of 2.15 m close to the swilley brows. It is an asymmetrical structure. Gradients of up to 1 in 7 have been recorded on the inbye swilley bank, the outbye band slope is comparatively shallow. The trend of the swilley is difficult to establish with available information, but it is probably near right angles to the gate line (Figure 6.3). During the deposition of strata above a swilley, differential compaction of the coal filled channel and silt/sand levees of the brows can occur, which may result in minor faulting. Localised areas of poor roof conditions due to compaction faulting associated with this swilley have been observed.

There are occasional minor tectonic faults within the area. A large fall to the second parting plane 2.2 m above the seam occurred during excavation of the Deep Hard East Manrider (JCM12) 7 m outbye of the head end. The strata up to the parting had descended as one large block. The heading had just passed through a minor fault. It is probable that this disturbed and weakened the strata, allowing bed separation to occur at the parting.

6.3 Directional Stability

The line of main cleat strikes approximately North-East to South-West. Roadways driven in a North-East or South-Westerly direction, where cleat is at right angles to the direction of advance have suffered from poor roof conditions. This is contrary to the usual situation encountered in coal mining, where workings running parallel to cleat can suffer from greater roof and floor instability due to ribside spalling producing an increase in roadway width.

Several fractures sub-parallel to cleat are present in the roof. These are probably natural fractures opened by mining which could be responsible for the roof deterioration. Another possible cause could be an anisotropic stress field. Roadways in Australia that are driven perpendicular to the maximum horizontal stress frequently have strata control problems (Pugh et al 1987). Where the major horizontal stress deviates from the perpendicular, tensile fractures and guttering can occur along one side of the roadway, known as the "notch" stress concentration effect (Section 2.10). Asymmetrical support loading has been observed in 2nd West Main Road (driven in the unfavourable direction).

This could be due to a similar effect, resulting from the maximum horizontal principal stress being orientated in a NW-SE direction. If this is the case it would be consistent with the direction of maximum horizontal compression suggested by regional tectonic history and data obtained from in situ measurement recently carried out in Coal Measures Strata in the Vale of Belvoir (Golder Associates 1986).

6.4 Interaction Effects

The Deep Soft seam has been worked at both collieries. Its position varies from 28 m to 35 m above the Deep Hard/Piper. Some roadways developed under Deep Soft rib edges have been known to suffer heavy weightings. A fall occurred in 131's Main Gate beneath one such pillar edge. The roof failed in a 25 m section along vertical shear breaks above the roadway sides to form a 3 m high cavity. The fall was probably an interaction problem associated with the weight of an increased thickness of mudstone separating from the siltstone above. Both outbye and inbye of the fall area there was no deformation or obvious loading of the supports.

This roadway and the surrounding strata were simulated in the HQTD sand/plaster physical model rig (Appendix 1b). A vertical to horizontal pressure ratio of 4:1 was applied to the model to represent the assumed stress condition surrounding this section of the roadway. As the load was increased the roof strata gradually deformed, then vertical fractures developed above the rib-sides which resulted in a sudden roof failure (Figure 6.7). Further models were tested with a dense pattern of roof bolts in addition to the standing supports. Shoulder bolts were angled over the ribsides. It appeared that strata reinforcement would probably have delayed but not prevented this fall.

Roof fractures have been observed in 2nd West Main Road directly beneath Deep Soft workings. Two fracture sets were present. The most prominent being at right angles to the direction of advance and a less well developed set parallel to advance. Nuisance quantities of water were coming from these roof fractures (Figure 6.8). The water was tested and found to be deep zone strata water. The likely origin of the water is a clastic unit between the Deep Hard and Deep Soft seams. Water probably accumulated in fractures opened by the working of Deep Soft 104's. The fractures were intersected, further opened and drained by 2nd West Main Road.

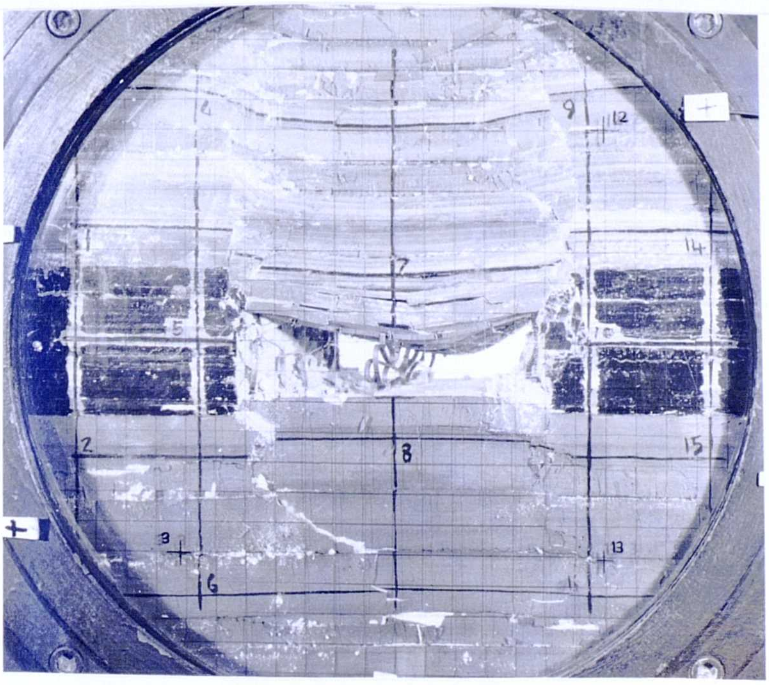


Figure 6.7 Model simulation of roof fall at Sherwood Colliery

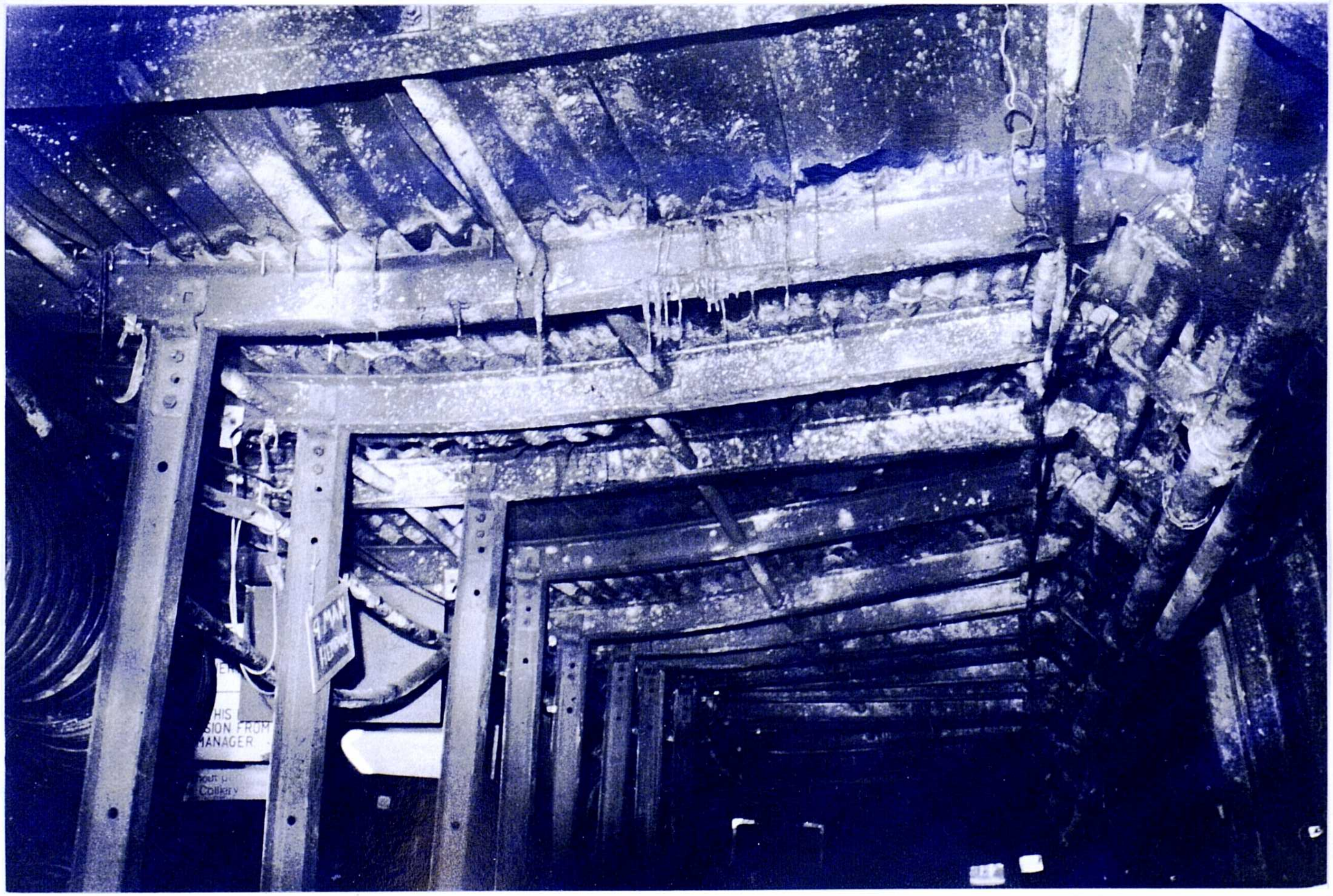


Figure 6.8 Water inflow - 2nd West Main Road

General underground observations and results from monitoring sections at Sherwood Colliery (Figures 6.9a, 6.9b and 6.9c) indicate that the superjacent Dunsil and Top Hard workings have little or no effect on Deep Hard/Piper roadways. The greater floor heave recorded at Station 2 was due to the presence of water at this point.

Stress redistribution due to the extraction of 125's Deep Hard/Piper longwall face resulted in the occurrence of a significantly greater amount of roof lowering at Station 3, in the adjacent East Manrider Connection (JCM 12) (Figure 6.9a). Figure 6.10 is a stress-distance diagram (Wilson 1980) showing the ribside stress distribution adjacent to 125's face. It is apparent that the East Manrider Connection (JCM 12) is being subjected to a vertical stress of approximately one and two-thirds times cover load.

6.5 Extraction Effects

202's panel was the first single entry retreat face worked in the Deep Hard/Piper seam at Mansfield Colliery. It was supported by RSJ flat topped supports 5.18 m x 2.92 m (152 mm x 127 mm section), set at 1.2 m intervals. A 30 m trial section of this roadway was reinforced with roof bolts. The bolts were installed at the head end between each standing support, full column anchored by twin setting speed resin grout and pre-tensioned. Wire mesh was secured against the roof by the bolt bearing plates.

Measuring stations were established in areas of the roadway with and without additional roof reinforcement. As the retreating face approached the stations, the closure patterns were recorded to detect the influence of the forward abutment stress. With this particular panel the roadway was kept open behind the face line for ventilation purposes. This enabled further support performance comparisons to be made. It was noticeable that throughout the bolted area of the roadway there was an improvement in conditions at the face entry, with no evidence of shear breaks appearing at the face side (Figure 6.11a and 6.11b). As a consequence of this the original face-side leg could be replaced behind the face line in the bolted section, whereas normally a shorter leg had to be brought underground and set.

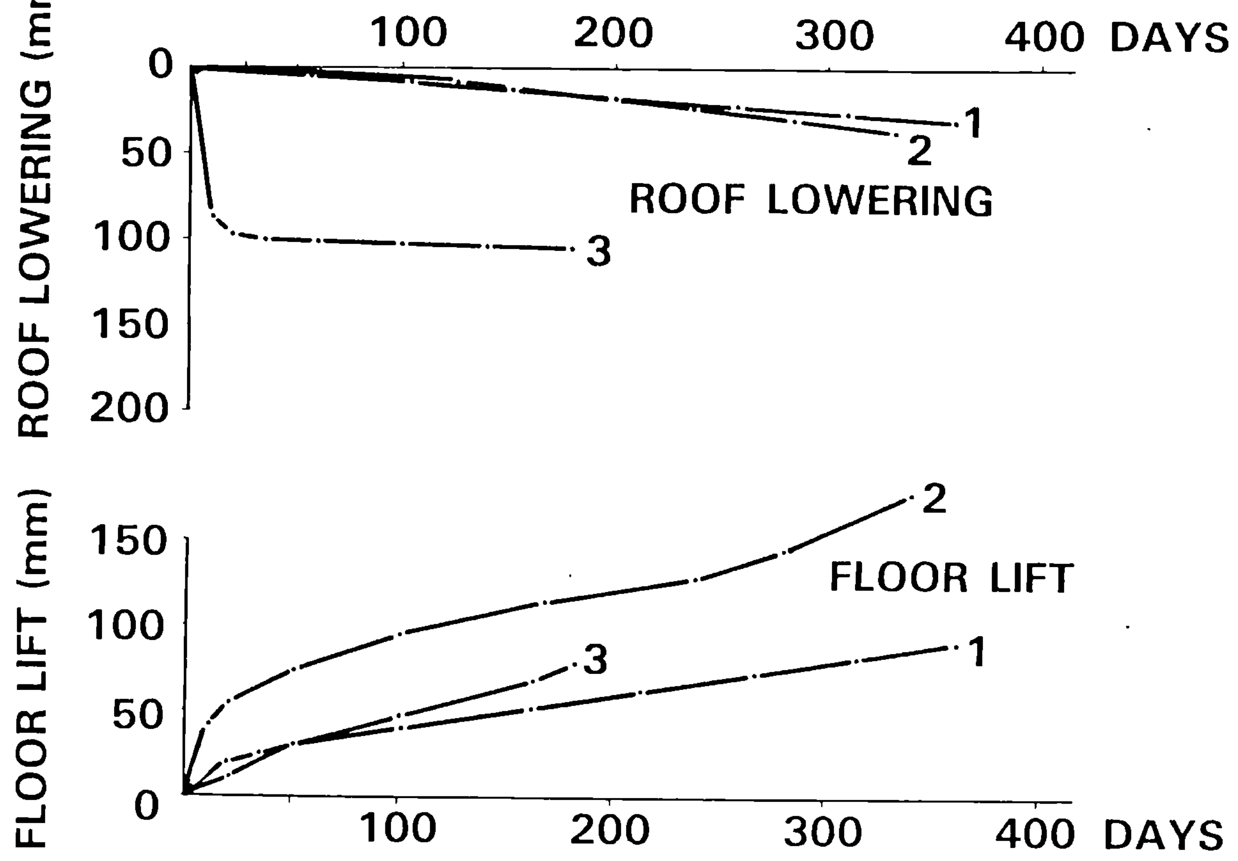
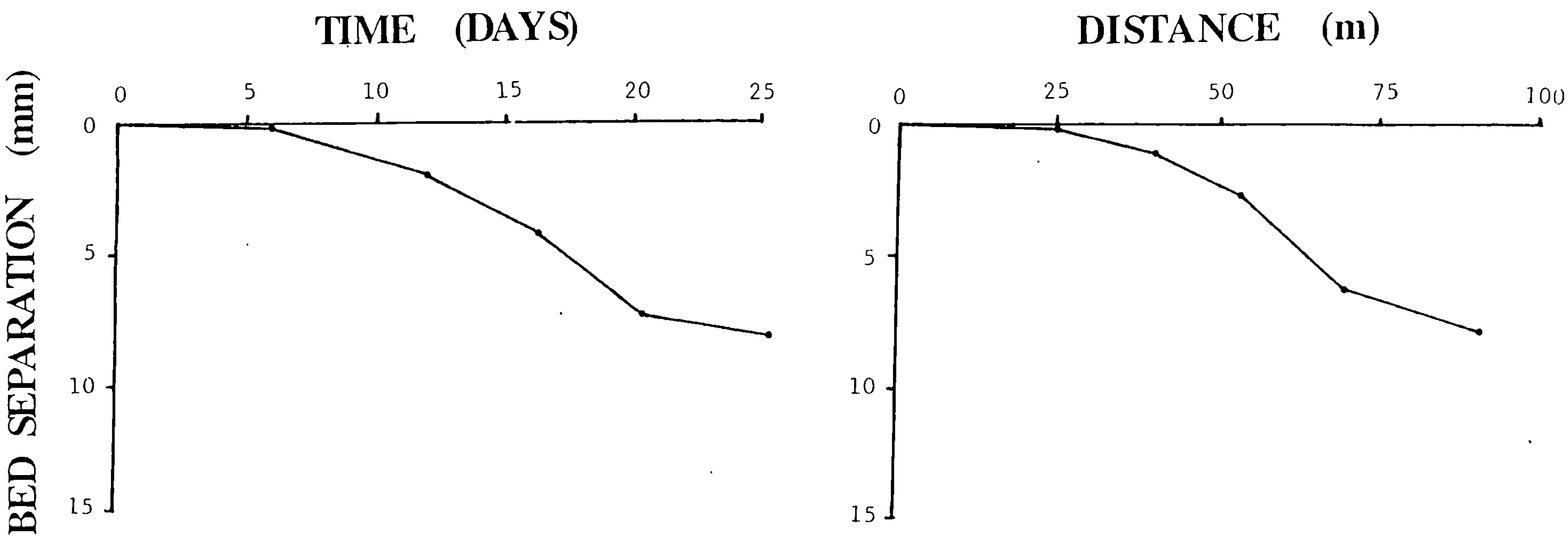


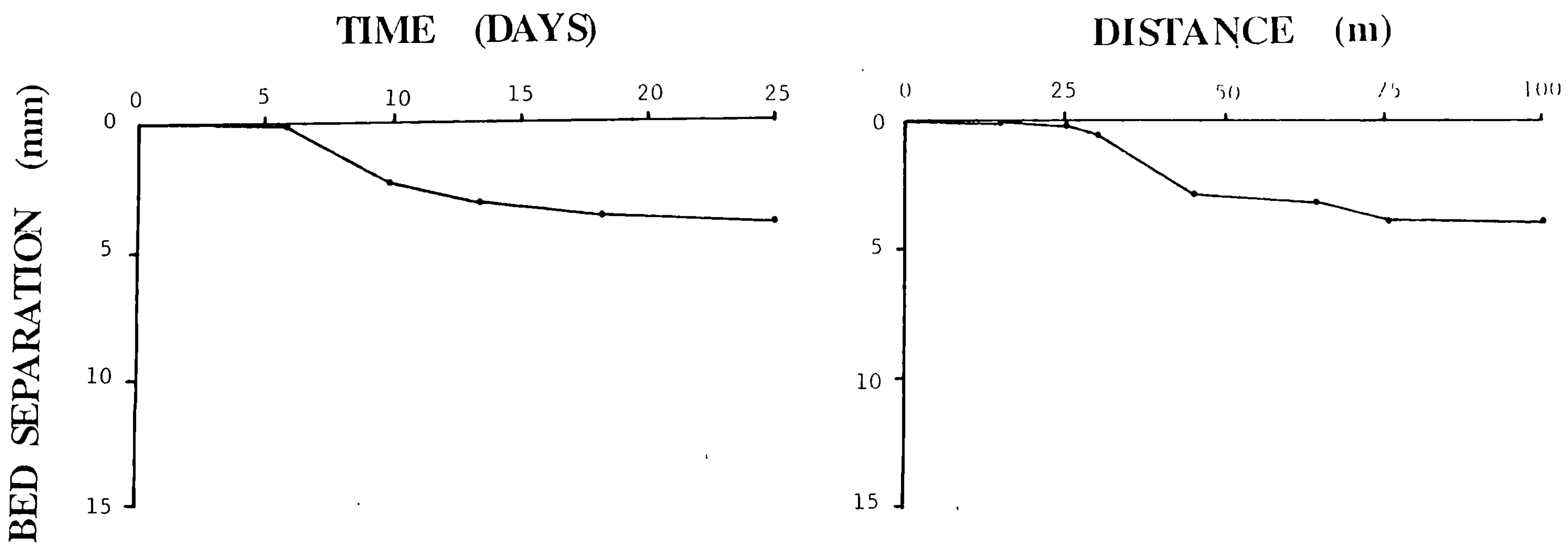
Figure 6.9a Sherwood Colliery monitoring stations - convergence measurements.



No movement was recorded above 1.299 m from the mouth of the extensometer hole.

The station was located beneath a pillar edge in the Dunsil Seam - this seam lies 204 m above the Deep Hard/Piper Seam.

Figure 6.9c Bed separation recorded at monitoring station 2.



No movement was recorded above 0.535 m from the mouth of the extensometer hole.

The station was located beneath a pillar in the Dunsil Seam - this seam lies 204 m above the Deep Hard/ Piper Seam.

Figure 6.9b Bed separation recorded at monitoring station 1.

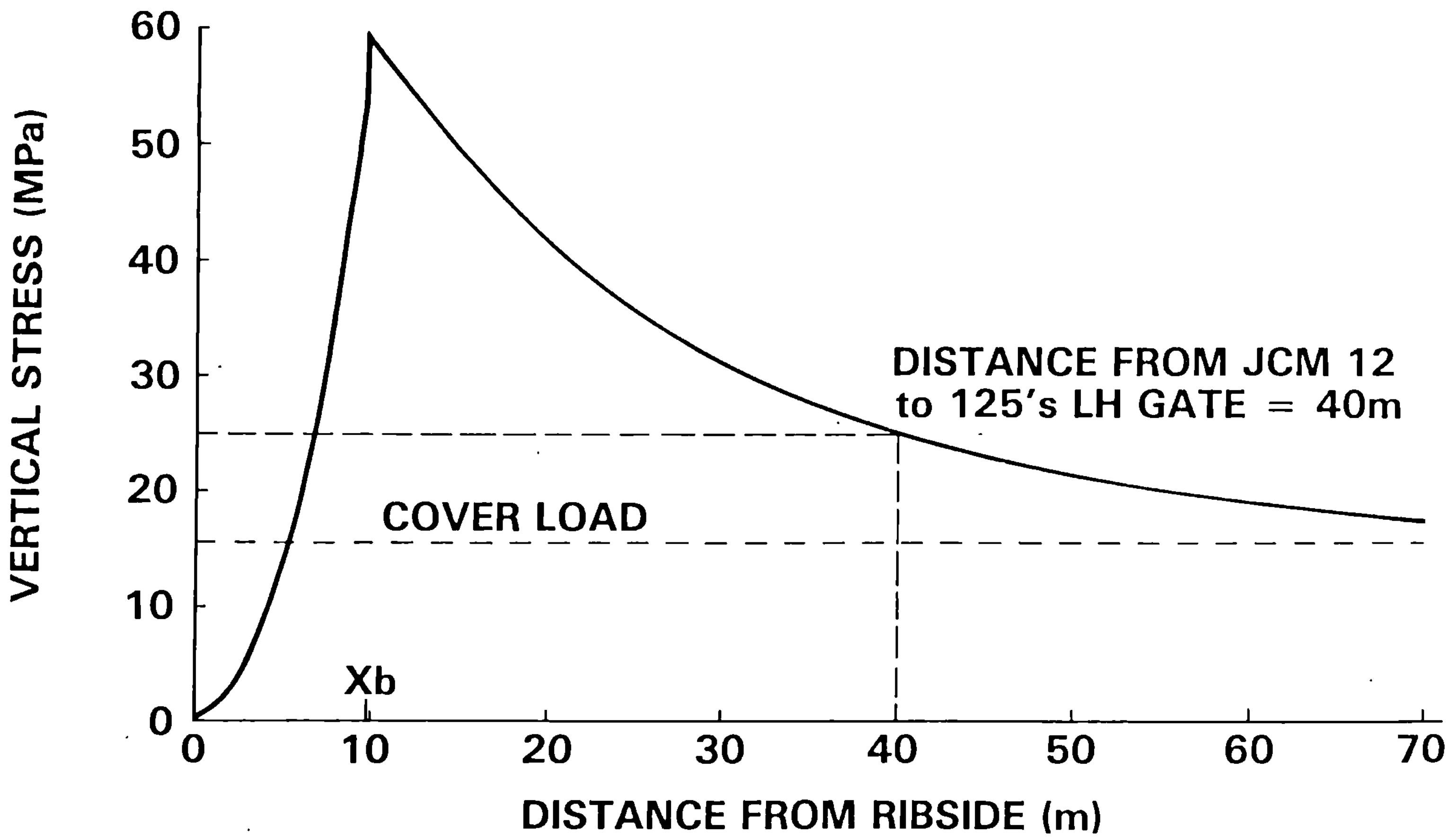
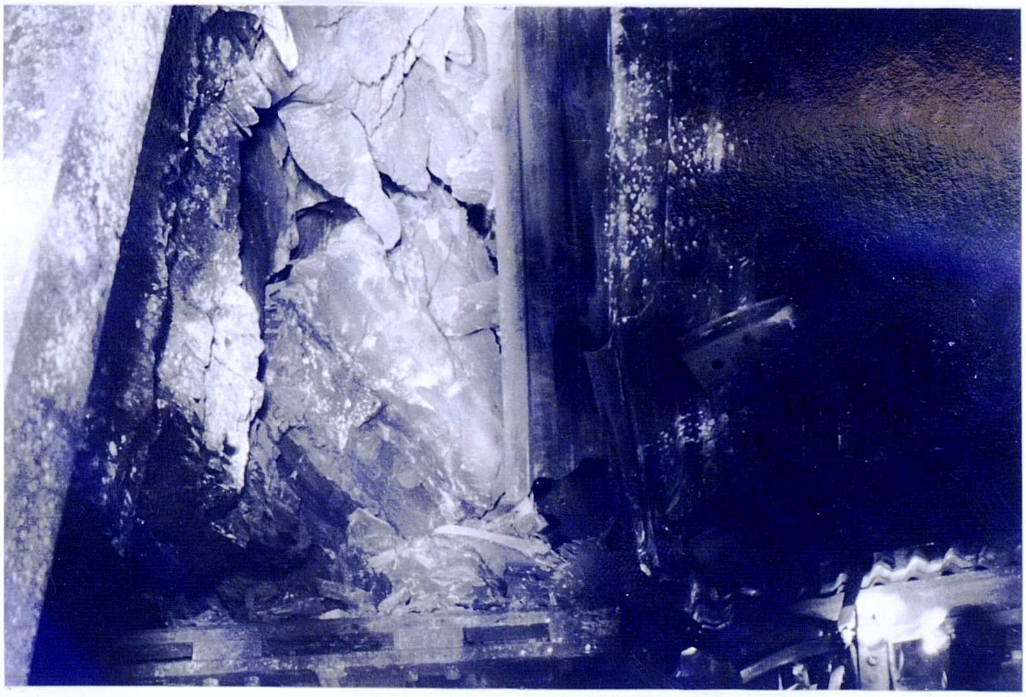


Figure 6.10 Ribside stress distribution around 125's loader gate.

a. Section with roof bolts as additional support



b. Shear break in section without roof bolts

Figure 6.11 202's Face Entry

6.6 Support System Design

6.6.1 Position Of Bolt Installation

Roof bolt systems should be installed before bed separation has had an opportunity to develop. This normally requires installation of the bolt system close to the head end and not behind the heading machine.

Extensometers installed in the roof at Sherwood Colliery indicated that significant bed separation did not occur until the head end had advanced 25 m from the measuring station (Figures 6.9b and 6.9c). This suggests that it may be possible to install roof bolts behind the in-seam miner without reducing their support capacity. However, the information obtained from these stations is obviously limited and may not be representative of the entire area of proposed extraction.

6.6.2 Roof Bolt Length

The lengths of the bolts installed in the roof of the Deep Hard/Piper seam should be governed by the position of the second parting plane relative to the roadway roof. This is dependent on the amount of roof rock extracted and the thickness of the immediate roof strata. A situation whereby the second parting plane lies immediately above the bolted section must be avoided as this could lead to detachment of the bolted block along the parting and a possible collapse of an area of this section en masse.

Roof bolts 1.8 m in length were installed in 2nd West Main Road in an effort to improve roof stability. Initially their effect was limited as the bolts were only just reaching the second parting plane 1.8 to 2.2 m above the seam, where bed separation was occurring.

The 1.8 m long bolts installed in 202's roadway also reached the level of the second parting plane 1.76 m above the roadway roof. Severe lateral movement was recorded by a magnetic extensometer at this horizon immediately after the longwall face had passed.

It is therefore recommended that where possible the end of the roof bolts should lie at least 0.5 m beyond or 0.75 m below this prominent parting plane.

6.6.3 Reducing Support Density

The maximum permissible interval between arch girders without an exemption from Regulation 15 of the Coal and Other Mines (Support) Regulations (1966) is 1.2 m.

A short series of scale models were constructed to give a qualitative assessment of the effect of different support densities and an indication of the degree of strata deformation that will occur when Deep Hard/Piper roadways are subjected to stresses above cover load. These stress increases will occur due to retreat face forward stress abutments or redistributed stresses from adjacent extraction.

Models representing 202's roadway were tested (Appendix 1c), simulating different support systems; i.e. with roof bolts and steel work at 1.2 m intervals, roof bolts at 1.2 m intervals and steel work at 1.5 m centres, rows of roof bolts only at 1.2 m intervals, and steel work only set at 1.2 m intervals. Figure 6.12 is a plot of percentage roof lowering against applied pressure for these tests. The results indicate that below approximately 150% cover load, roof bolts are capable of maintaining a competent roof. At higher applied pressures, failure of the roof beam occurred, initiating gradual roof lowering. The bolted model with the highest standing support density experienced the least roof deformation. The model with standing support only underwent minor roof beam deflection up to cover load. At applied pressures above cover load, roof bed separation occurred followed by the formation of an inverted V-shaped fracture zone and considerable vertical closure.

Models simulating Sherwood wide headings (Appendix 1b) have shown that if both the roof bolt row spacing and standing support spacing are increased to 1.5 m, sudden failure of the roof may take place (Figure 6.13).

Results from the model tests therefore indicate that increasing the roadway standing support spacing to 1.5 m intervals may have only a minor effect on the support capacity of the dual support system. They also illustrate that if the spacing between arches is increased in order to reduce support costs, a relatively dense pattern of roof bolts must be maintained.

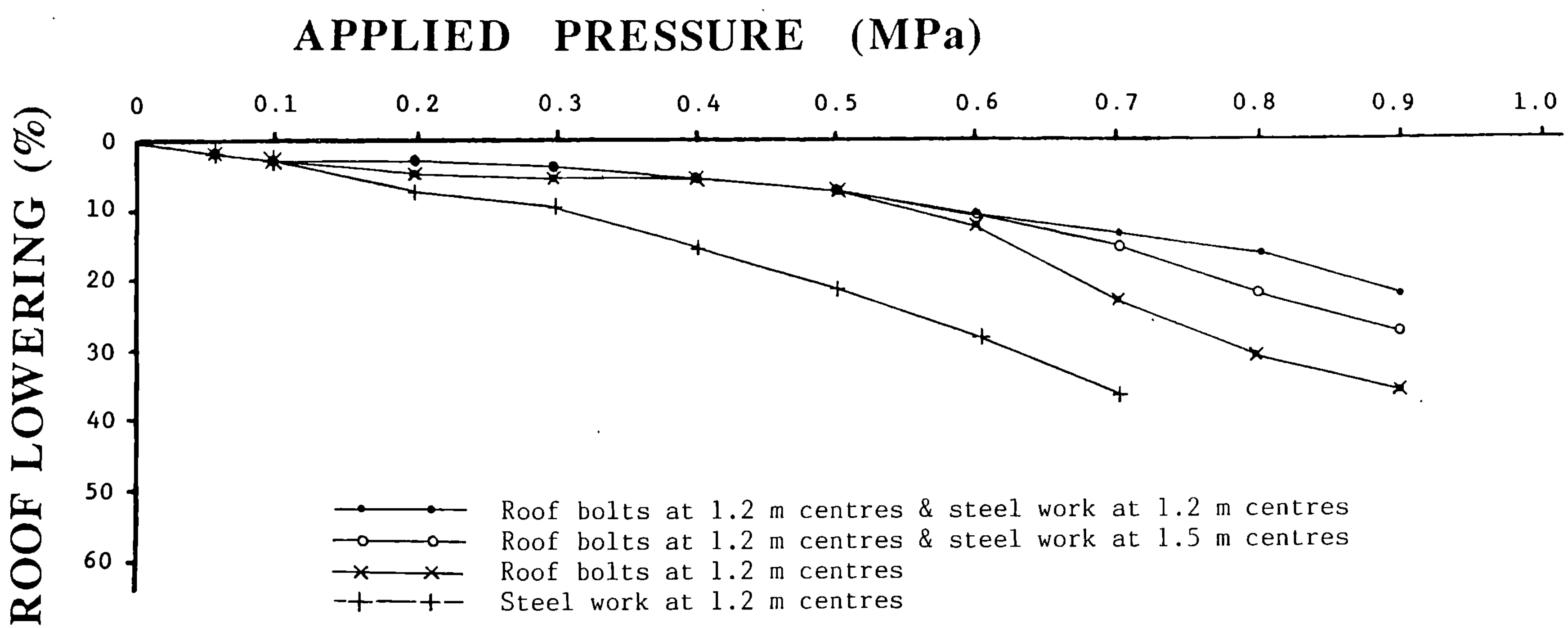


Figure 6.12 Variation in roof lowering as a percentage of initial roadway height with applied pressure - Mansfield Colliery scale models.

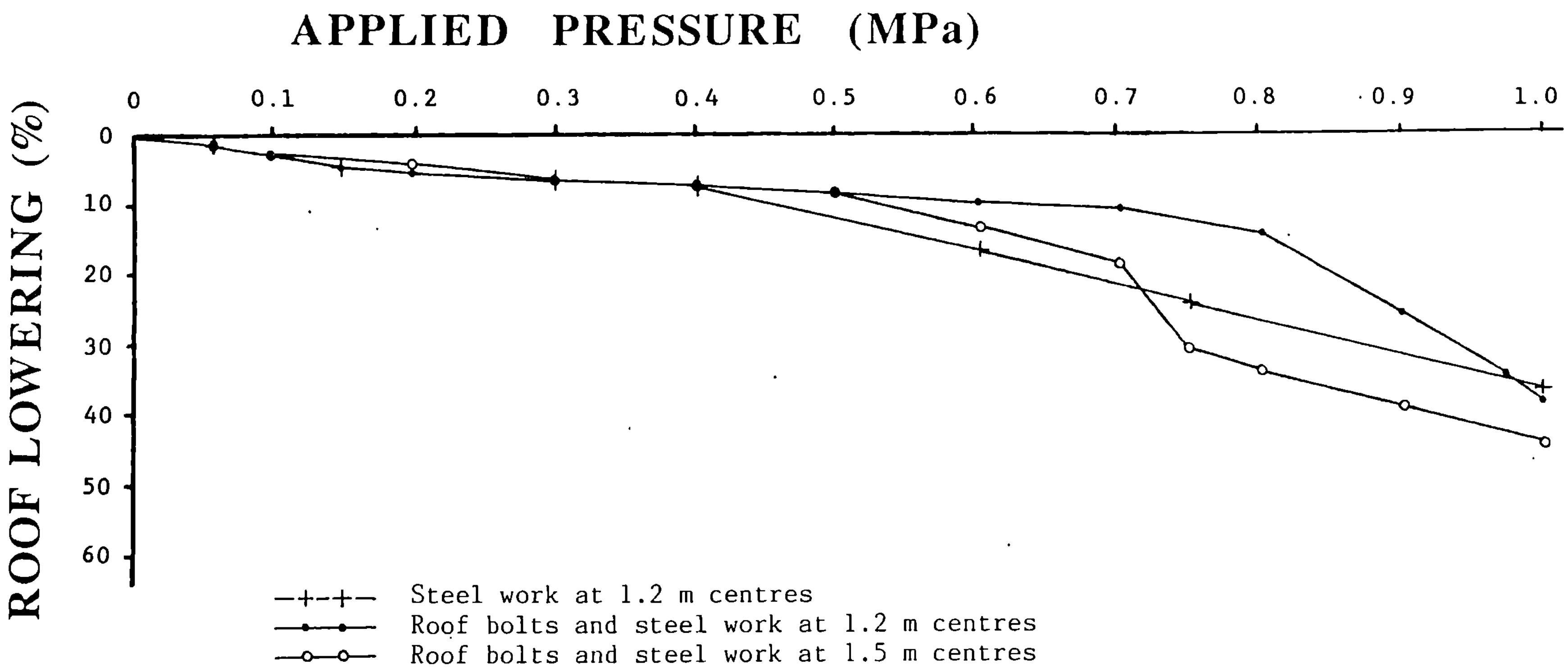


Figure 6.13 Variation in roof lowering as a percentage of initial roadway height with applied pressure - Sherwood Colliery scale models.

6.6.4 Ribside Spalling

Ribside spalling occurred in all the scale model tests and was particularly severe in the models without any steel standing support. Spalling has been observed in existing Sherwood wide headings excavated with a BJD Heliminer 122M. It is expected that the Dosco in-seam miner will give a smoother roadway profile, however, should spalling become excessive it could be reduced and possibly eliminated by meshing and dowelling of the ribsides.

6.6.5 Empirical And Analytical Design Methods

Table 6.1 gives roof bolt lengths and spacings derived from various empirical and analytical design methods that may at first appear to be applicable to the partial extraction workings in the Deep Hard/Piper seam. However, if any of the recommended bolt lengths are strictly adhered to without regard to parting plane location the consequences could be disasterous (i.e. collapse of large blocks of bolted roof).

Bolt spacing recommendations vary from 0.76 m to 1.8 m. A bolt spacing of 1.2 m seems to be reasonable based on scale model studies and roof bolting experience in the seam to date.

	Bolt Parameters		
	Spacing (m)	Length (m)	Diameter (mm)
RQD (Merritt 1972)	1.2-1.8	-	-
RQD (Deere et al 1970)	0.9-1.5	-	-
Q-system (Barton 1976)	-	1.0	-
Geomech. Class. (Singh 1986)	1.2	1.5	-
Geomech. Class. (Unal 1983)	1.4-1.5	1.2	25.0
Cerchar PC Bolting	1.2	1.4	18.0
Rock Mass Conf. (L = 1.83 m)	0.76	1.83	25.0
Rock Mass Conf. (L = 2.13 m)	0.84	2.13	25.0
Rock Mass Conf. (L = 2.44 m)	0.91	2.44	25.0

Table 6.1 Roof bolt parameters derived from empirical and analytical design methods for Deep Hard/Piper Seam partial extraction workings.

CHAPTER 7
POINT ANCHORED ROCK BOLTS

7.1 Background

A slight misnomer exists with point anchored bolts, they should not be regarded as being anchored in strata at one single point but over a small proportion of their total length.

Point anchored bolts consist of three elements: a solid steel bar, an anchoring device at the far end of the bar and a tensioning device at the head of the bar. The bolts are always pre-tensioned at the time of installation; US Federal Regulations require that point anchored bolts should be tensioned to a load level of at least 50% of the yield strength of the bolt (US Government 1977). Although a rock bolt of high yield strength is desirable, the use of a steel bar of very high strength should be avoided where a high strength anchor can be obtained. The reason being that should the bolt fail, the bar could shoot out of the hole at high speed and cause severe injury.

Point anchored rock bolts are basically capable of support in suspension bolting (Snyder et al 1979) or for the formation of laminated rock beams through a friction effect (Panek 1964). They rely on the pre-tension force applied to the bolt creating a compressive force on the strata (Bolstad et al 1983).

Point anchored bolts were once a fairly common means of support in British coal mines (Sen 1959); several kilometres of roadways in the 1950s had them as the sole means of support (Adcock and Wright 1957/58). However, their use severely declined as they were found to be unreliable. Murphy et al (1972) state that this was because the following important factors were not considered:

(i) pre-tensioning was not adequately controlled;
(ii) there were inadequate plating arrangements at the bolt hole mouth;
(iii) the extension characteristics of the bolt anchor did not utilise the full capabilities of the steel rod;
(iv) the ultimate strength of the bolt anchorage was significantly less than that of the yield load of the bolt stem;
(v) the mechanical anchorage lost load substanially with time owing to the high localised stresses where it was in contact with the rock.

It is considered that this type of bolt still has only limited applications in British coal mines.

7.2 Slot And Wedge Bolts

7.2.1 Installation

Slot and wedge rock bolts (Figure 7.1) are currently rarely used in world mining although they were once very popular. The bolts are both simple and inexpensive; consisting of a mild steel rod, commonly 20-22 mm in diameter, of which the top end is split longitudinally for a length of approximately 150 mm. A hardened steel wedge, 130 mm long, is located inside the slot and the whole assembly is inserted in a hole 50 mm less than the bolt length. Anchorage is obtained by hammering the bolt at the head end, against the back of a 28 mm diameter hole so that the wedge is forced further into the slot embedding the mild steel flanges into the sides of the hole. A protective capping is screwed into the end of the rod so that the thread does not become damaged. A bearing plate, washers and nut are placed on the threaded end and the nut is tightened with an impact wrench.

Effective anchorages have been obtained in a variety of strata types (Barry et al 1954a). However, Sinou and Dejean (1980) do not recommend their use in the following instances:

(i) in rock which is too soft, where the wedge may become embedded in the back of the hole instead of wedging itself in the slotted end of the rod;

(ii) in rock which is too hard, where the sides of the slot may wear away instead of embedding themselves in the sides of the hole.

There are some disadvantages associated with drilling the hole and setting the anchor. If the hole is too long, impact cannot be applied to the end of the bolt to set the anchor; conversely, if the hole is too short, the nut on the protruding end of the bolt may become thread-bound before adequate tension can be developed. A source of compressed air is required to operate the pneumatic hammer; this is not always available in underground coal mines.

7.2.2 Slot And Wedge Rock Bolts As A Gateroad Support

Adcock (1955) reported the use of slot and wedge bolts as a support in the loader gate of a double-unit panel in the Piper Seam. This was at an unnamed colliery in what was the NCB East Midlands Division.

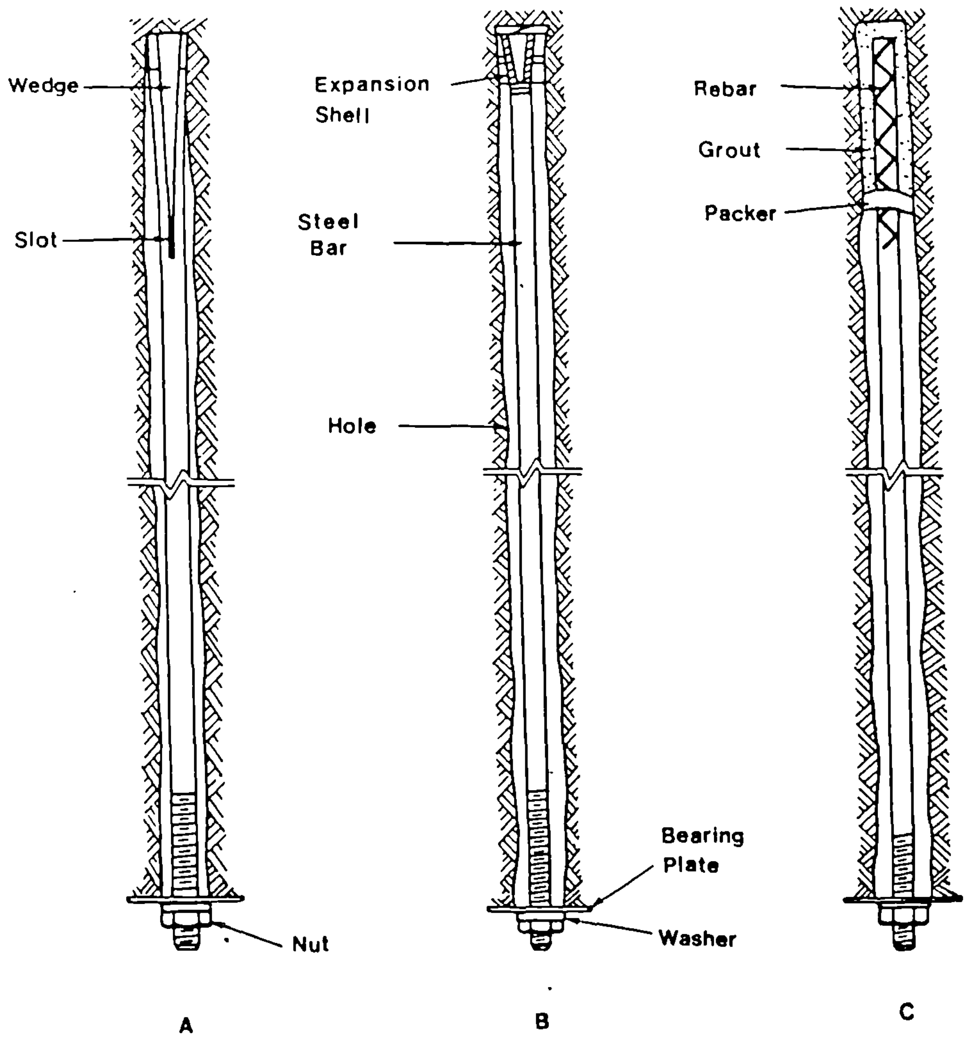


Figure 7.1 Three types of point anchored rock bolt.
A. Slot and wedge bolt
B. Expansion shell bolt
C. Grouted anchorage bolt
(after Lang et al 1979 and Peng 1986).

The immediate roof consisted of 0.3 m of inferior coal overlain by a moderate to weak mudstone with ironstone nodules. Five 25 mm diameter, 1.8 m long bolts were used to secure a 4 m (152 X 76 mm) channel girder to the roof. These girders, drilled to pattern, acted as a template for the bolting pattern (Figure 7.3). The girders were set immediately at the coal face and were temporarily supported on either chocks or props. The roadway was then dinted and the 32 mm diameter bolt holes were drilled in the roof, through the girder. A compressed air drill was used and the bolts were installed with an impact hammer. The girder end legs, which allowed yield, were left in position for a distance approximately 9 m outbye and then removed.

Improvements in roadway stability and economic advantages were noted with this support system which was later adopted in other nearby gateroads.

7.3 Expansion Shell Bolts

7.3.1 Installation

Expansion shell rock bolts superseded slot and wedge bolts and remain a very popular means of support in underground mining, in fact 55% of the roof bolts installed in US coal mines are of this type (Serbousek 1987).

Expansion shell bolts operate by applying a torque to the bolt head which pulls a wedge shaped plug down the bolt, forcing outer serrated shell leaves to grip the strata at the back of the borehole. Expansion shells are used in many different forms and are applicable to a variety of rock conditions. Shell designs differ in the shell lengths, type of serration, angle of plug and number of leaves forming the shell; one principal US supplier currently markets 25 different designs (Frazer and Jones 1987).

The vast majority of expansion shells fall into two categories: the standard type and the bail type (Figure 7.2). The standard type has a limited contact area due to the rigid shell-leaf attachment at the base; consequently high stress concentrations are generated, making this device more suited to hard rock applications. The bail type is capable of making full contact along the length of the shell, so that the high expansion pressure is distributed over all the shell, making it more suited to medium-hard rock.

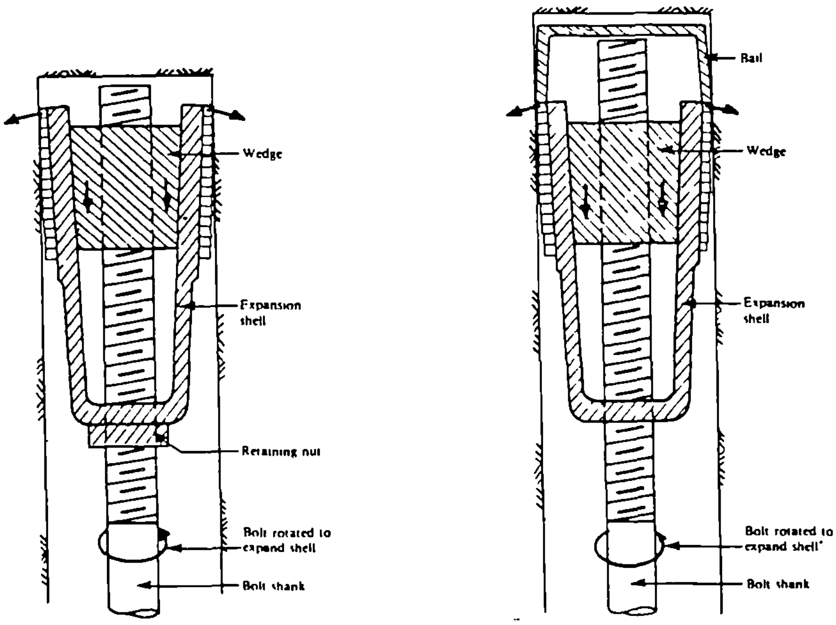


Figure 7.2 Mechanical expansion anchors (after Douglas & Arthur 1983).

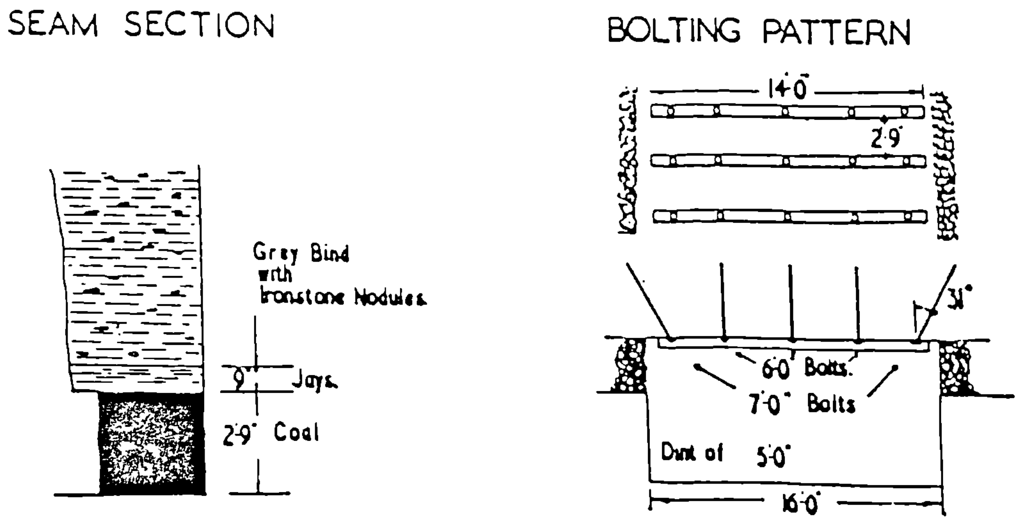


Figure 7.3 Seam section and slot and wedge bolting pattern - Piper seam (after Adcock 1955).

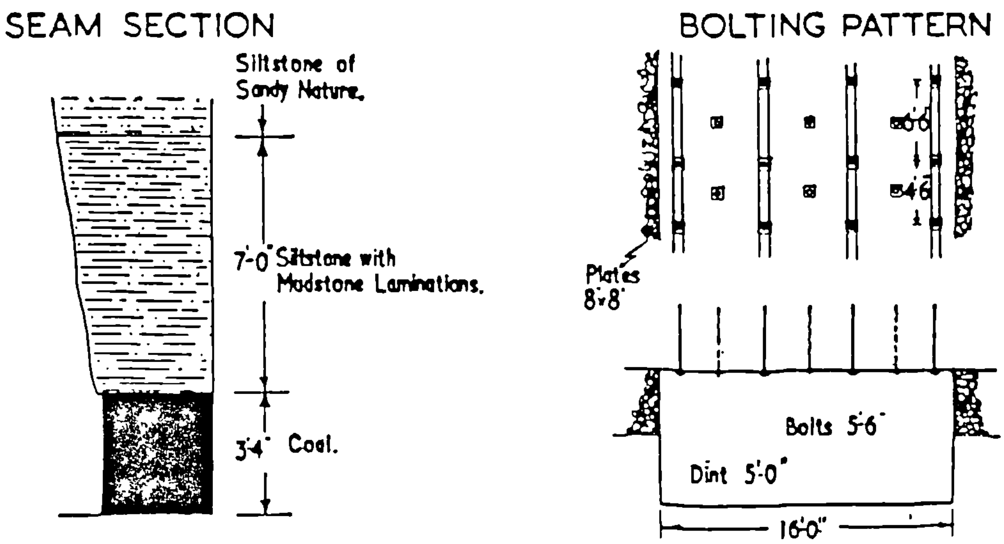


Figure 7.4 Seam section and expansion shell bolting pattern - High Hazels seam (after Adcock 1955).

Both shell types have limited points of load transfer (at the serration tips), creating areas of relatively high stress. In weak strata this can cause rock disintegration adjacent to the anchor, leading to a loss of anchorage and hence bolt tension. The gradual loss of the initial applied pre-tension load is commonly observed. This is due to rock creep (Tamames 1983) or more usually anchor slippage (Parsens and Osen 1969). Anchor slippage allows bed separation, which reduces the frictional shear strength of the bedding planes. Possible causes of loss of expansion shell rock bolt tension are discussed in detail by Thiei (1964).

Undulations on the rock surface can produce point loading of the rock surface at the bearing plate; Parker (1974) has frequently observed that less than 10 % of the surface has been loaded. Failure will ensue in rocks of low strength resulting in a loss of bolt tension. A wooden pad between the rock and the plate to distribute the load can alleviate this problem.

Losses in load bearing capacity can also result from rock spall at the borehole collar due to high rock stresses or from blast vibrations.

Expansion shell bolts have received widespread favour mainly because they are relatively inexpensive and can be rapidly installed. Unlike slot and wedge and capsule grouted anchors it is not critical for the hole to be an exact length. The diameter of the hole drilled for expansion shells is important; in an undersized hole the wedge will not seat inside the shell and in an oversized hole the wedge will pull through the shell. Both situations will cause a poor anchorage to be obtained.

Friction may cause insufficient torque (applied to the bolt head) being converted to tension in the bolt, thus producing a poor installation. Friction can result from insufficient lubrication of threads, deformed bolts pinching threads, dirt, rust, rough castings, hardness of materials and angled bolts gouging the bearing plate.

Misalignment of the bearing plate by as little as five degrees can induce a significant bending moment to the bolt, such that the yield point is drastically reduced. Maleki et al (1985) found that bending combined with torsion can reduce the yield point of the bolt to less than one-quarter of its nominal value.

The process of setting the anchor can induce fractures in the strata at the anchor horizon parallel to the roof, especially if the rock is a fissile mudstone (Culver and Jorstad 1968; Agarwal and Boshkor 1970). Tension in the bolt and rock load added to the initial tension, pulls down on the anchor and opens up the fracture. If all the bolts in the roof are of the same length a crack can develop across the entire opening initiating a roof collapse. This problem can be avoided by using bolts of different lengths, by staggering them and by designing away from concentrated loads on anchors (Parker 1974).

7.3.2 Expansion Shell Bolts As A Gateroad Support

Adcock (1955) described a colliery in the NCB East Midlands Division that adopted the use of 19 mm diameter, 1.5 m long expansion shell bolts as a means of support in the loader gate of a double-unit panel in the High Hazels Seam.

The immediate roof consisted of up to 2.1 m of fine siltstone with mudstone laminations overlain by a sandy siltstone. Holes were drilled to the pattern illustrated in Figure 7.4 using a rotary action electric drill, with the thrust being provided by water pressure. The bolts were tensioned with a hand-operated torque spanner, set to 200 Nm. The bolts held "corregated benk bars" to the roof. These were not across the roadway as is conventional, but parallel to it.

An improvement in roadway conditions was noted using this support technique.

7.3.3 Expansion Shell Bolts As A Support In Room And Pillar Mining

A study and evaluation was made of the use of expansion shell bolts as a support in a room and pillar coal mining operation at Foidel Creek Mine in Colorado, USA.

The Wadge Seam is being worked, which in the immediate mine area ranges from approximately 2.6 - 2.9 m in thickness. The strata overlying the Wadge Seam are very predictable and uniform units of deltaic and marine origin. The immediate overlying lithology is a silt-rich mudstone approximately 0.45 m thick. Deltaic sequences are stacked directly over the silty mudstone. These sequences gradually coarsen upwards, beginning with a mudstone grading into a fine sandstone. At least three deltaic pulses have been identified (Tifft 1987) and combined, form thicknesses in excess of 10 m above the seam. The strata below the seam are variable and thought to be coastal plain deposits with a high degree of fluvial influence. In the mine area the lower 0.3 m of coal is somewhat gradational with the overlying 0.3 - 0.6 m of carbonaceous mudstone. Sandstone with a thickness of up to 1.2 m comprises the floor.

Structurally the seam is very regular with only minor rolls displayed in the floor. The strike of the Wadge Seam ranges from N. 50 - N. 70 E., with a dip of approximately 7° to the northwest. The depth of cover above the room and pillar workings is between 120 m and 200 m. Two joint systems are present. The first, a conjugate shear system striking N. 35 - 70 W., with one set dipping 65 - 85° S.W. and the compliment set dipping 65 - 85° N.E. The second, a N. 40 - 60 E. striking extension system dipping 80 - 90° S.E. In general, the N.W. striking shear system is more frequent, less continuous and less open than the N.E. extension system. The Wadge Seam displays very prominent cleating. The primary cleat is consistently orientated N. 45 - 70 W., corresponding well to the shear joint system. The secondary cleat was found to have a higher range of orientations, N. 15 - 65 E., roughly following the orientation of the extension joint system. Ribside spalling was not a serious problem.

Stress concentrations have been evaluated utilising the stress-relief overcoring technique with a borehole deformation gauge (Tifft 1987). Observation of strain relief in core samples was also used to define the orientation of average principal stress. The maximium horizontal stress ranges from N. 50 - 70 W. and can be up to three times the vertical stress. The entries and cross-cuts are orientated at 45° to the principal stress direction.

The roadways are rectangular in shape, driven totally in-seam with a width of 6.1 m, leaving 15.2 X 15.2 m pillars. Excavation is carried out in two cuts by a continuous miner. The miner operator is not permitted to advance beyond the last row of roof bolts set, the length of each cut is consequently approximately 6 m. If the roof bolting machine is not fitted with an automated temporary support system (ATRS), posts are installed as a temporary support prior to the bolting cycle.

The roof bolt system used in the room and pillar workings consists of 1.2 m long, 16 mm diameter, expansion shell anchored bolts made from high strength steel. The bearing plates are either 152 X 152 X 5 mm embossed or 406 X 127 X 5 mm flat plates with a 20 mm centre hole. The bolts are installed vertically at 1.5 m centres (Figure 7.5) in holes drilled with a 35 mm bit. At intervals of less than 30 m along a development entry a hole is drilled in the roof to a depth at least 300 mm above the anchorage horizon to determine the nature of the strata. The bolts are installed with a torque of between 150 and 350 Nm. A regular check is made on bolt torques after installation; if the bolts are not maintaining at least 135 Nm of torque additional support is installed. This can consist of extra standard bolts, longer bolts, posts or cribs.

Roof conditions in the mine are generally good. The mechanical bolts provide adequate support by maintaining a strata beam across the entries through the friction effect and suspending the immediate mudstone from competent rock. Roof that has been exposed for several months is beginning to weather which results in spalling of small, thin, slabs of the immediate roof.

Minor ribside cutters were observed which are probably the result of the high directional stress field. Mining induced stress due to pillar extraction operations is also affecting roof stability, producing tensional fractures in the centre of the entries and minor roof falls. In these areas wooden posts and cribs are being installed as a supplementary support.

Expansion shell point anchored rock bolts are probably the most suitable support for this particular mining operation which serves to illustrate their potential. However, there are very few sites in British collieries with comparable stress and geological conditions.

7.4 Grouted Point Anchored Bolts

7.4.1 Installation

By using resin (or cementitious) grout to anchor a bolt at the back of a borehole several advantages can be gained over expansion shell and slot and wedge bolts. Grouted bolts are rigid and less susceptible to anchor slippage (Karabin and Hock 1979). The end of the bolt is modified to provide a key to the grout (Figure 7.6). When used in the roof or in upwardly inclined holes, a packer is required to prevent grout loss from the bond length. Grout capsules are generally used, although grout can also be pumped into the void around the bar over the bond length.

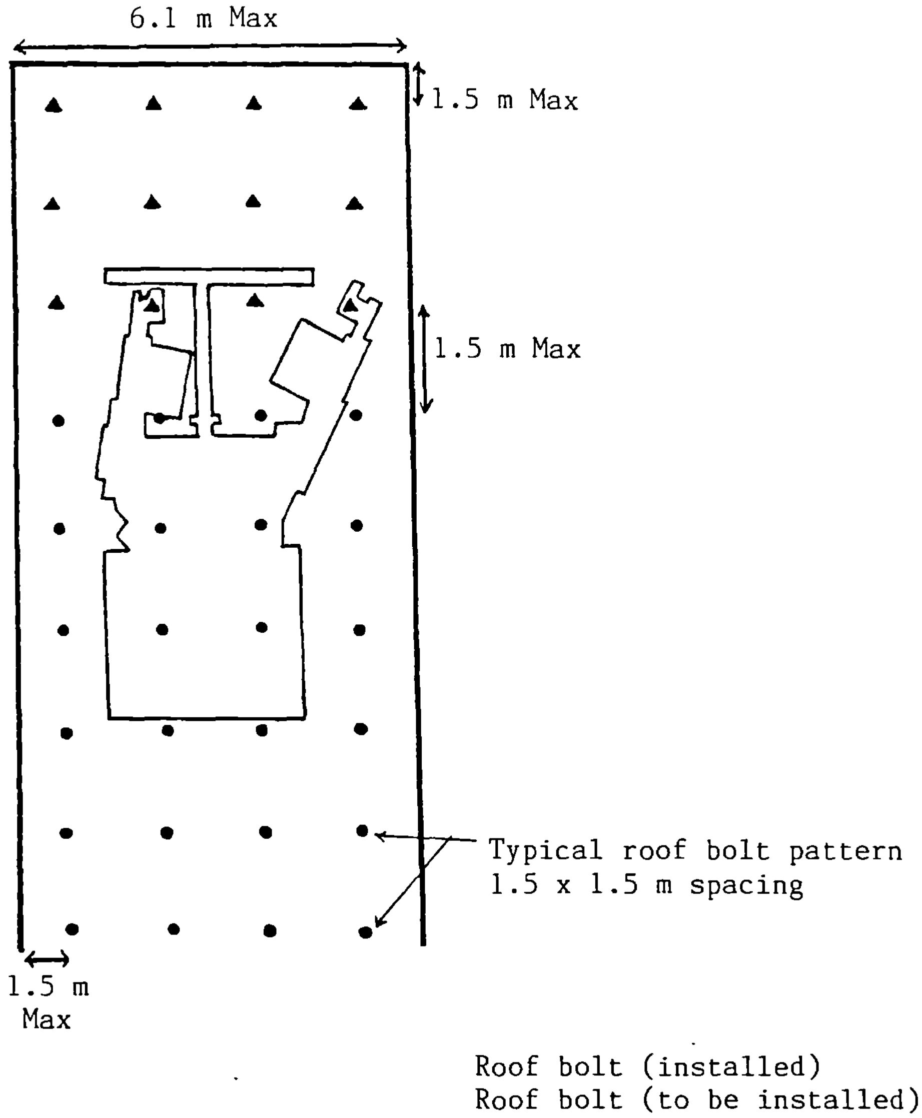


Figure 7.5 Support plan: Foidel Creek Mine.

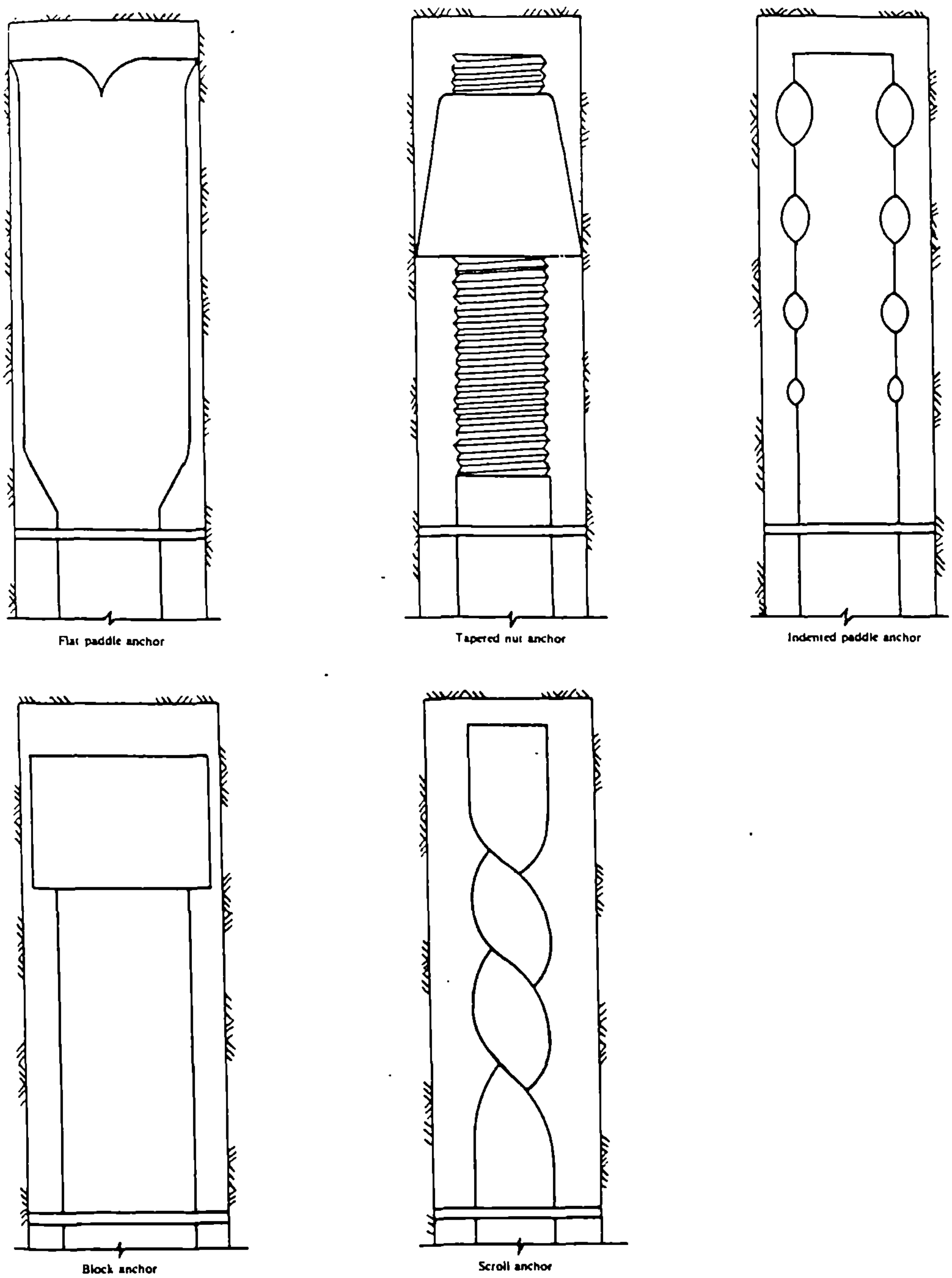


Figure 7.6 Grouted point anchors (after Douglas & Arthur 1983).

A resin point anchored bolt developed by Guy McDowell of Peabody Coal (Roberts 1980; White 1982) and now commonly used in the US coal mining industry (McDowell 1987), features quick setting resin, a flat stopper head in the lower end of the bolt and a counterbored nut (Figure 7.7). The bolts are installed as follows:

(i) A resin capsule and the top of the bolt are inserted into the hole and the nut is placed in the bolter chuck/spanner.

(ii) The nut must be rotated so that it moves down the bolt thread to engage with the flat stopper head. The bolt is pushed into the hole simultaneously.

(iii) Continued rotation causes the bolt to mix the resin.

(iv) After seven seconds, rotation is stopped and the bolt held tightly against the roof.

(v) After seven more seconds, the nut is rotated in the opposite direction to tension the bolt.

Protruding threads indicate that the bolt has not been properly installed; either the resin was not sufficiently mixed or the operation was not stopped for seven seconds before the nut was turned. Grout problems discussed in Chapters 8 and 9 apply equally to grouted point anchored bolts.

7.4.2 Resin Point Anchored Bolts As A Gateroad Support

The use of resin point anchored bolts in the maingate of W1's advancing face in the Clowne Seam at Whitwell Colliery has been reported by Hodgkinson (1971), Whitaker and Hodgkinson (1971) and Murphy et al (1972). The seam was 0.94 m thick. Immediately above was a moderately strong carbonaceous mudstone approximately 2.4 m thick, containing ironstone bands. This was overlain by a weak mudstone which readily collapsed after being undercut. W1's maingate lay at a depth of 164 m.

The gate, formed by the advanced heading method was supported by 4.2 m by 3.0 m three-piece arches, 115 X 115 mm in section, set at 0.91 m centres. A 3.68 m pack, with two hardwood chocks built in, was put on the faceside of the gate.

Severe roof lowering and arch deformation problems occurred outbye of the face which were associated with high water inflow rates. These problems were alleviated by the installation of five 1.8 m long resin point anchored bolts radially around the arch section between each support setting. The bolts were inserted in 43 mm diameter holes and

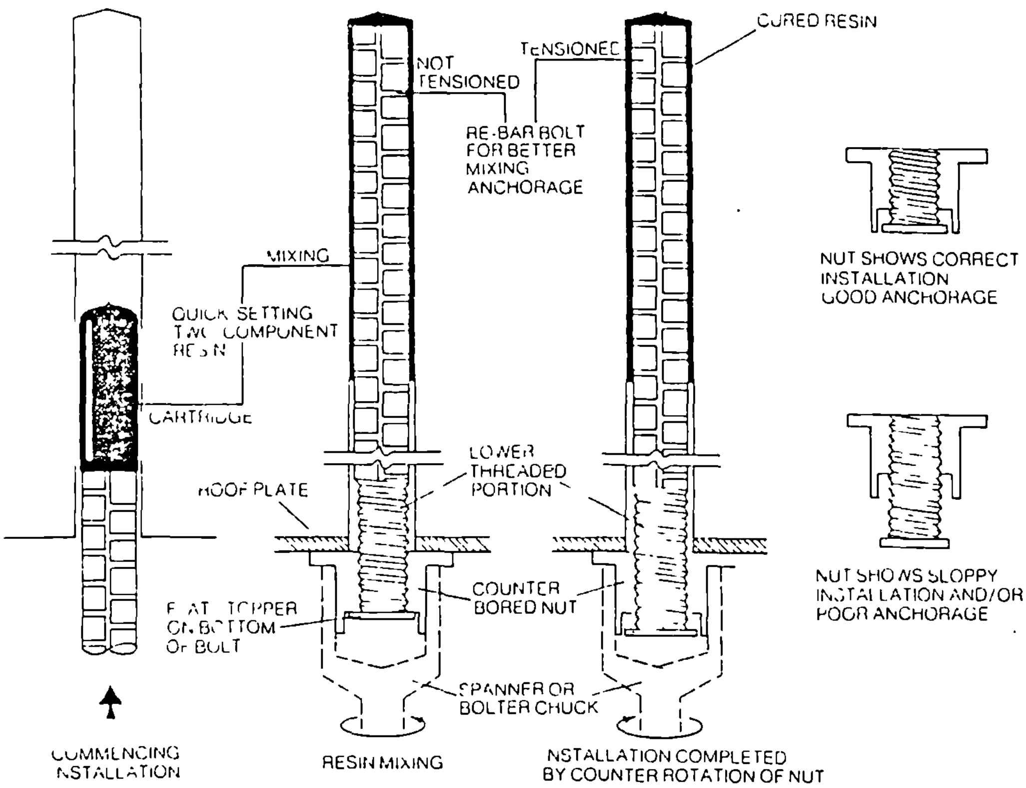


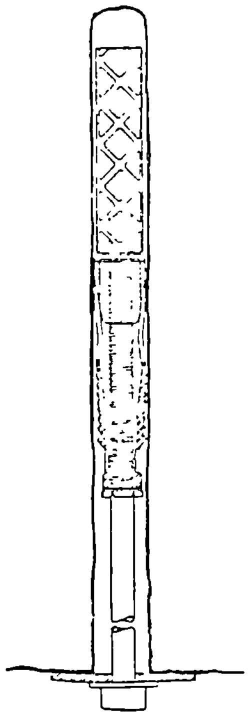
Figure 7.7 Installation of the Peabody-McDowell bolt (after Roberts 1980).

pre-tensioned to a torque of 340 Nm. Roadway monitoring recorded a dramatic improvement in conditions and water inflow was substantially reduced.

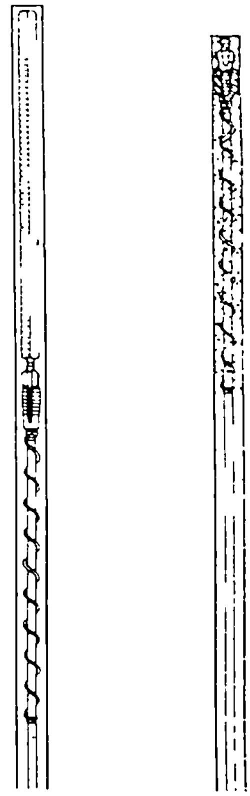
7.5 Point Anchored Grouted Expansion Shell Rock Bolts

Point anchored bolts with a combined expansion shell and resin point anchorage have gained popularity in some US coal mines over recent years. Combining the advantages of the two systems can give a very high capacity anchorage.

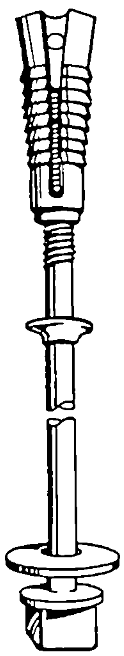
Several different methods of obtaining the anchorage have been developed, some of which are shown in Figure 7.8. Some have a specially designed expansion unit which allows a limited amount of spinning (e.g. about 30 rotations) to mix a resin capsule (Figures 7.8a and 7.8b). Another method uses a wooden shear pin through the expansion anchor wedge which allows the anchor unit to turn with the bolt whilst resin capsule mixing takes place (Figure 7.8c). When mixed, the resin begins to solidify; this creates sufficient friction to shear the wooden pin. The anchor shell opens up and then a pre-tension is applied. An alternative is to use a resin capsule that does not require rotary mixing (Morrow 1981). The resin and hardener are placed one behind the other in the capsule, instead of side by side (Figure 7.8d). The capsule base is designed as a mixer membrane through which the resin and hardener can flow and thereby intermix as soon as the expansion shell is pushed into the capsule at the back of the hole. The expansion unit is tensioned against the borehole wall by rotation of the rock bolt rod.



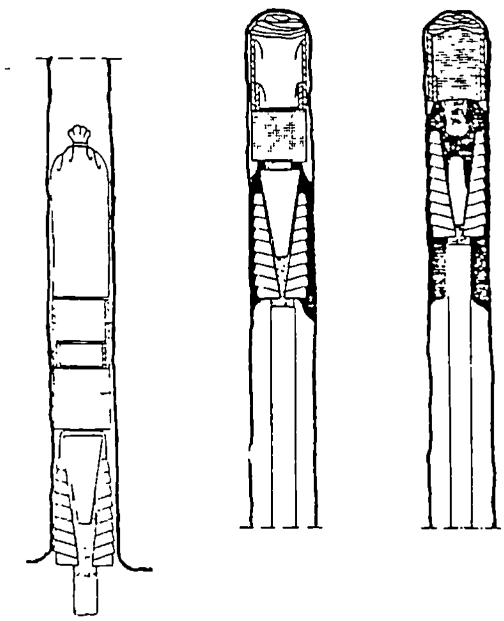
a. Resmech anchor
Birmingham Bolt Co.



b. Spiral bolt
F. M. Locotos Co.



c. Install I bolt
Jennmar Corp.



d. Lock II system
Frazer & Jones - 3M Co.

Figure 7.8 Point anchored grouted expansion shell bolts.

CHAPTER 8

FULL COLUMN ROOF BOLTING WITH ORGANIC GROUTS

8.1 Materials And Installation

8.1.1 Polyester Resin Capsules

Packaged polyester resins for use as rock bolt grouts were initially developed for the West German mining industry by Bergbau-Forschung in 1959 (Schuermann 1960). Further development worldwide improved the system, which was first used in British coal mines in 1965.

Rock bolt resin capsules currently available consist of two compartments containing a polyester resin and a catalyst. An exothermic reaction occurs when these two components are mixed. Initially the liquid undergoes an incremental viscosity increase up to the point where it can no longer be described as a liquid, this is the gel-point. The period of time from catalyst addition to reaching this gel condition is known as the gel-time. Following gelation, the polymerization continues as the gel becomes increasingly rigid, through to a rubber-like condition and finally to a solid formed from cross-linked polymer chains. The stage from gelation to attaining ultimate hardness is known as the cure stage. By variation of resin, inhibitor, catalyst and accelerator, gel-time can be adjusted from seconds to hours. Cure time can be varied correspondingly.

Pure unsaturated polyester resins undergo shrinkage in the region of 8 to 17% when the liquid changes to a solid. Fillers are able to reduce this shrinkage to less than 1% as well as lower the cost of the capsule. Typically the resin component will consist of approximately 75% limestone filler. Mineral fillers reduce the inherent ductility of the plastic resin and make it more brittle. Impurities in the filler can reduce the storage life and stability of the resin system.

Owing to resin inflammibility, changes in the NCB Acceptance Scheme for resin capsules in British coal mines were initiated during the late 1970s. This led to the introduction of water extended polyester capsules. This type of capsule is currently being marketed for use in the US mining industry as a less expensive alternative to the non-water extended polyester. However, it appears that the presence of water can lead to a weakening of the resin properties and increases in gel-time, as well as the negation of any major changes in resin technology. A number of fire retardants are now available that can be added without appreciable changes to the resin properties. Further addition of hydrated salts substituted for fillers could also give the same fire retardant properties as water extended resins (Hirst 1987).

The catalyst usually consists of a derivative of benzyl peroxide which acts as a free radical producer. The rate of formation of free radicals from organic peroxides at ambient temperatures is so slow that an accelerator (e.g. diamethylaniline) must be used. Details of the early development, manufacturing and chemistry of the components for organic grout capsules are given by Peerlkamp and Watt (1971).

The first resin capsules had glass for an outer-casing. High cost and breakage problems during transport led to the development of containers made from flexible materials such as polythene film. The catalyst compartment can either be formed by a separate sheath or tube, or by an interface reaction between the catalyst and the polyester resin compound.

Polyester resin capsules have a limited shelf life. This should be checked on all batches before use. A slow reaction takes place between the polyester and the promoter (which promotes the production of free radicals from the catalyst) within the capsule. This has the effect of shortening the set time as the capsule ages. The rate of this reaction will double with every 10°C increase in temperature, so that the resin will tend to solidify if the capsules are stored at high temperatures for long periods of time.

An inhibitor is added to the resin to limit its reaction with free radicals produced from natural sources such as ultra-violet light, never-the-less the efficiency of the inhibitor reduces with time. It is therefore important to store resin capsules out of direct sunlight.

The polymerization reaction is temperature sensitive. The rate of reaction will double every 10°C, so that a variation of mixing temperature between 10 and 30°C will produce up to fourfold changes in set times. High temperatures will also reduce the strength of resins (Beveridge 1974). It is therefore important to monitor the ambient temperature at rock bolt installations to ensure that over mixing of fast and ultrafast setting capsules does not occur.

8.1.2 Bolt Parameters

The type of rod will govern the manner in which the rock bolt reinforced structure deforms under load. Ribbed bolts are generally capable of attaining a higher anchorage capacity than smooth bars due to the increased frictional resistance caused by mechanical interlock with the grout. Hence, high yield deformed steel bar (rebar) is widely used. NCB Specification 131 (1986) for rock bolts requires that the rebar be manufactured in accordance with the requirements of BS 4449.

In situations where the strata surrounding a roadway is likely to undergo considerable deformation, rods capable of sustaining very high elongation are often used to prevent premature breaking of the bolt. Types of yielding bolt are discussed in Section 14.2. Bolts with a high yield point will create a stiff reinforcement system and so resist deformation.

The most common diameter rebar currently used in British coal mining is 25 mm. However, in most US coal mining applications the strength of a 19 mm has been found to be more than adequate, as shown by the fact that few of these bolts break. A stiff rock bolt system will combat transverse shear, although shear loading will cause the bolt to crush and cut into most coal mine strata. The cure would be to reduce the bearing capacity on the rock, by increasing either the number or diameter of the bolts. According to Dejean and Raffoux (1980a), large diameter bolts should be used when they are subjected to high shearing stress as their shear strength is lower than their direct tensile strength. The shear resistance of fully grouted resin rock bolts has been studied by Bjurstrom (1974), Haas (1976, 1981), Azuar and Panet (1980), Barton and Bakhtor (1983) and Ludvig (1983).

8.1.3 Bolt Hole Parameters

In soft rocks, failure of a correctly installed full column resin grouted rock bolt generally occurs by shearing of the rock at the grout/rock interface. Anchorage capacity is increased with larger hole sizes as the shear stress at the grout/rock interface is decreased due to the larger surface area.

The capsule diameter should be approximately 3 mm less than the hole diameter. If smaller capsules are used, air becomes trapped in the hole and a foam type resin mix occurs, so reducing the anchorage capacity (Carr 1971).

The relationship between bolt hole diameter and rod diameter is critical. If the rod/hole differential is too large, the rod may not pierce and shred the resin capsule or mix it thoroughly enough. The photographs in Figure 8.1 illustrate short lengths of 25 mm diameter rebar installed in holes of various diameters drilled in breeze blocks. Adequate mixing was obtained in the 32 and 33 mm holes (a differential of 7 and 8 mm respectively). In the 38 mm hole, the resin was not sufficiently mixed and consequently a poor anchorage was obtained. The anchorage obtained in the 46 mm hole was completely ineffective as the capsule was not shredded. Conversely if the rod/hole differential is too small, difficulties can be encountered when inserting the rod (stiff drill rods will reduce hole deviation) and during rotation of the rod into the capsules; in addition the resin may not spread evenly within the annulus. The optimum rod/hole differential lies in the range 6 to 9 mm, thus large diameter holes will require large rods to maintain the proper annulus.

In order to ensure that the correct volume of grout is used, it is essential to regularly check the hole diameter, length and alignment at installation sites. A borehole micrometer is a useful tool to determine the diameter within a borehole. Weaker rocks tend to produce a more oversized hole than stronger strata. Thus a bolt hole diameter log may be of assistance in correlation of the position of different strata horizons between cored boreholes. To achieve a consistent hole length, the drill steel should be marked clearly with paint or tape at a point equal to the length of the rod plus 25 mm. An excessive hole length will result in insufficent mixing of the resin at the far end of the hole.

A rough bolt hole wall is usually desirable. Pull tests carried out by Dunham (1974) and Karabin and Debevec (1976) have shown that greater anchorage capacity can be achieved from bolts installed in rough holes compared with smooth holes. Rougher holes show greater shear strength at the grout/rock interface because relative movement along the interface has to first overcome the asperities on the hole walls.

8.1.4 Installation Procedure

A standard procedure for the installation of untensioned rock bolts , fully grouted with capsule resin is illustrated in Figure 8.2.

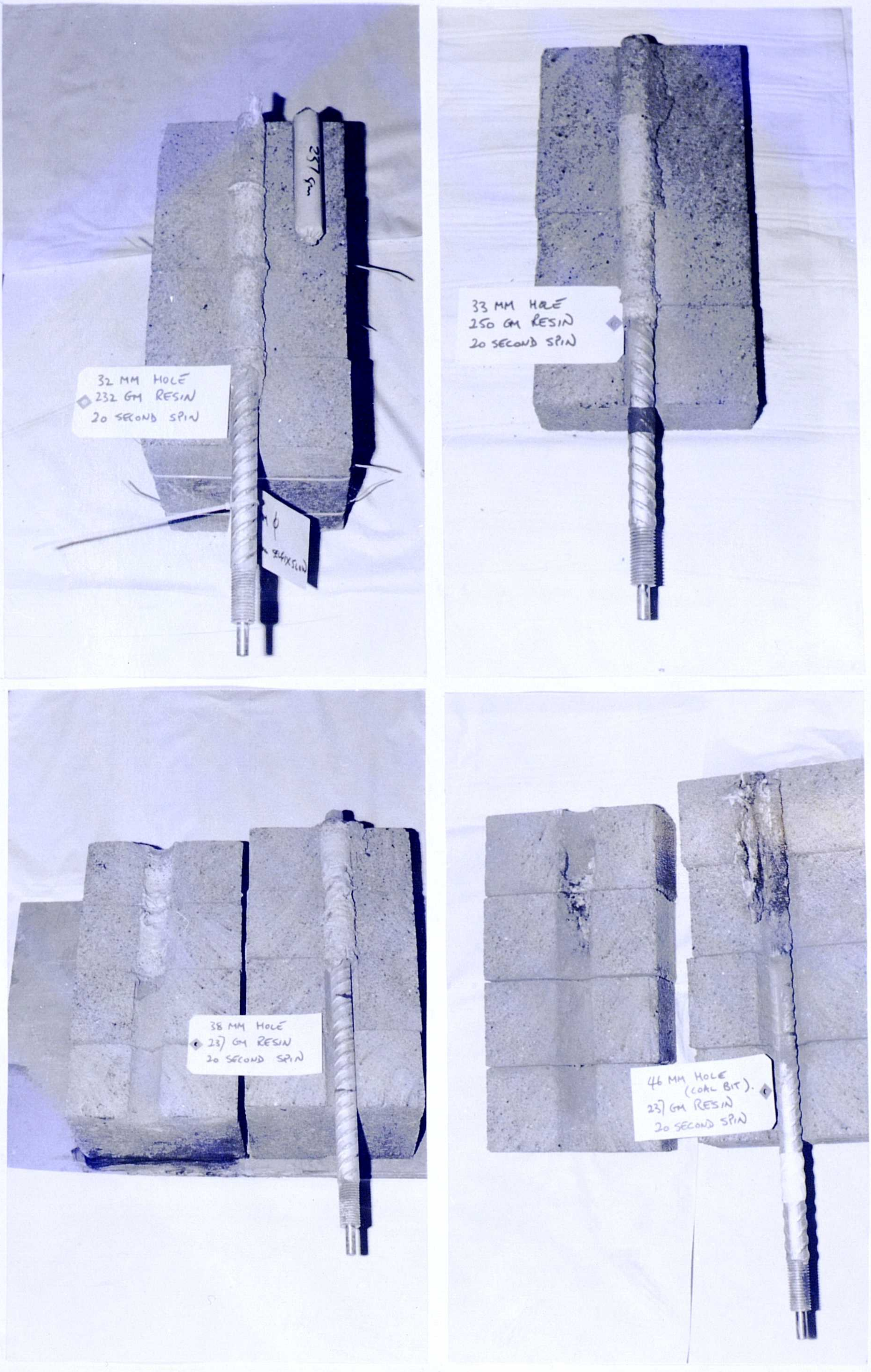


Figure 8.1 Effect of hole diameter on resin mixing characteristics

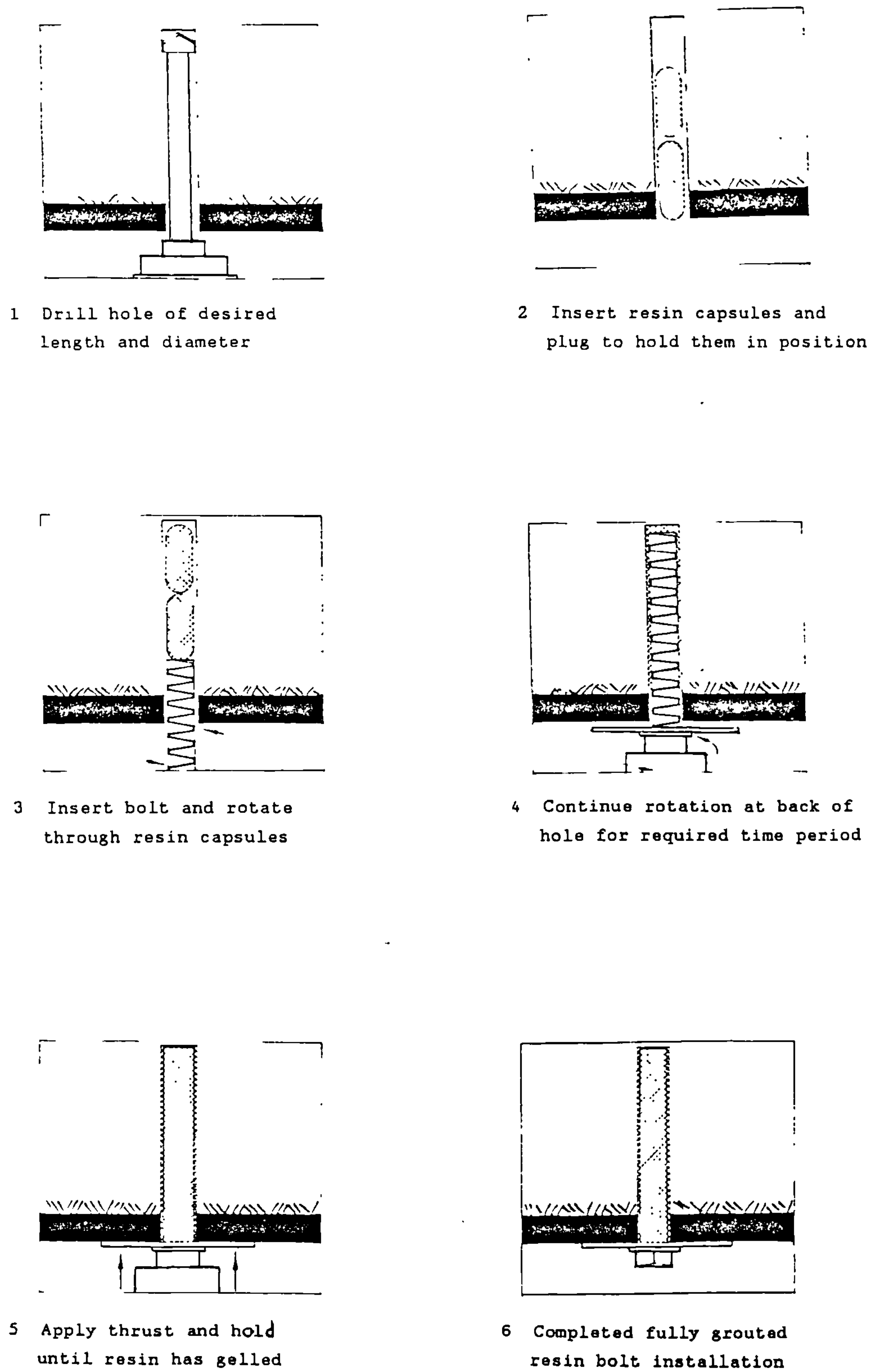


Figure 8.2 Full column resin grouted bolt installation procedure.

Laboratory tests by Singh and Buddery (1983) deduced that the optimum spin speed is in the range 300 to 500 rpm. At high spin speeds the resin may pass back along the bolt before being properly mixed. At low spin speeds the resin is compressed at the far end of the hole.

The spin time must also be tightly controlled. Mixing is normally only necessary for 8 to 10 seconds. Excessive spinning promotes break down of the resin as the spin approaches the gel-time, resulting in a poor anchorage. A very short spin-time will result in insufficient mixing of the resin and catalyst. The anchorage capacity can be severely reduced if the bolt is disturbed before resin curing is complete.

8.1.5 Checking Installation

Overcoring of additional bolts not forming an essential part of the support system, will determine the integrity of the grout at an installation site, however this is an expensive process. Pull out tests (Section 2.3) of full column anchored bolts will only confirm whether or not there is sufficient anchorage to exceed the strength of the bolt and does not assure that full anchorage is being developed throughout the length of the bolt.

Non-destructive rock bolt testing devices have been developed. A Swedish company, Geodynamik AB, are marketing the Boltometer. It is claimed that this instrument can detect invisible faults on fully grouted bolts such as, inadequate grouting, defective contact between bolt and grout, and broken bolts (Thurner 1979, 1983; Bergman et al 1983). A piezo-electric transducer in the Boltometer sensor head transfers compression and flexural waves to the bolt. These waves propagate down the bolt at different velocities, depending on grouting conditions, either part of the way or to the far end where they are reflected back to the sensor. From the time signal history the Boltometer determines bolt length and overall condition of the bolt and grout. Mattila and Boyd (1985) state that the Boltometer is effective under controlled test conditions but too sensitive for production sites. However, successful use of the device has been reported in civil engineering and hard rock mining within Scandanavia (Geodynamik 1986).

A similar device developed by the US Bureau of Mines (Stateham 1982; Moulder et al 1983) functions by sending a known pulse of ultrasonic energy into the bolts and comparing the amount of energy reflected back with the original measured pulse. Problems from overheating and coupling difficulties were encountered with this device. Also for an unknown reason, in some mines the tester performed well but in others it was not satisfactory (Stateham 1987).

The USBM have also investigated measurement of temperature increases at the bolt head due to exothermic reaction during polymerisation (Stateham and Sun 1976). Mixing a partial column of resin was found to generate less heat than mixing a full column.

8.1.6 Pre-tensioned Systems

A pre-tension can be applied to full column grouted bolts by initially forming a point anchor and then tensioning the bolt before the main body of the grout anchorage has reached gel-point. The point anchor can consist of a specially designed expansion shell (Section 7.5) or, as is common in UK mining, resin capsules with two different setting speeds are used. In the latter case a fast-setting resin is inserted to the back of the hole which rapidly forms a strong anchor, permitting tensioning of the bolt in the region of 30 to 45 seconds after mixing. Slower setting resin then grouts the remainder of the bolt.

Shear nuts enable a uniform and consistent level of pre-tension to be achieved. Commonly used shear nuts have a plastic insert which is dislodged at the required torque. Generally an M24 thread will require 45 to 50 Nm installation torque to generate 1 Tonne of pre-tension. A pre-tensioned single point rock bolt extensometer (Section 5.4.6) in S3's main gate at Betws Colliery had an installation torque of 150 Nm but only showed an initial load of 12 kN. If all of the torque was applied to the bolt the pre-tension load would have been in the approximately 30 kN. Clearly some of the torque was absorbed in the threads (Oram 1987).

If pre-tension is applied to a full column grouted bolt it will remove any compression induced into the bolt during installation and encourage interfacial friction along discontinuities within fractured strata. Pre-tensioning is generally not necessary in sound homogenous strata or in circumstances where the strata deforms rapidly, as in this case the bolts will be stressed automatically as a result of that deformation.

Dejean and Raffoux (1980a) regard the application of a pre-tension to fully grouted bolts as unnecessary. However, they consider that complete filling of the annulus along the total length of the bolt is rarely obtainable in French mining conditions and therefore recommend the application of a 15 to 20 kN pre-tension in all circumstances to guarantee minimum bolt effectiveness. It is also the opinion of ACIRL (Gale 1987), that pre-tensioning is unnecessary. It is maintained that if achieved, it tends to put the bolts closer to yield earlier than would otherwise be the case.

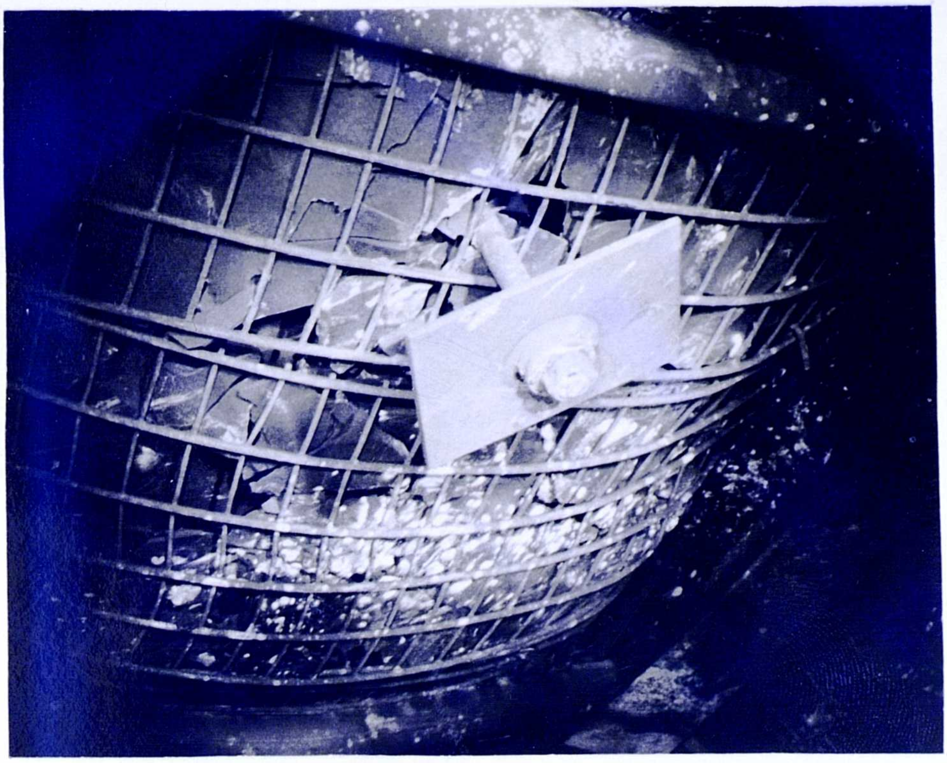
Pre-tensioning increases the complexity of the bolt installation procedure which, if not properly adhered to, can result in ineffective installations. Figure 8.3 shows two bolts in the roadway roof behind 202's retreat face at Mansfield Colliery. Bolt (a) is incorrectly installed, no pre-tension was achieved as the nylon insert in the shear nut is still in place and the plate is not secured tightly to the roof; the strata in the vicinity of this bolt is deformed and highly fractured. In contrast, bolt (b) is correctly installed and helped to maintain the integrity of the roof even after the passage of the longwall face.

Fully grouted rock bolts that are not deliberately pre-tensioned have been observed to develop a small initial tensile loading of about 9 kN (Patrick and Haas 1980) which assists in the reinforcing action.

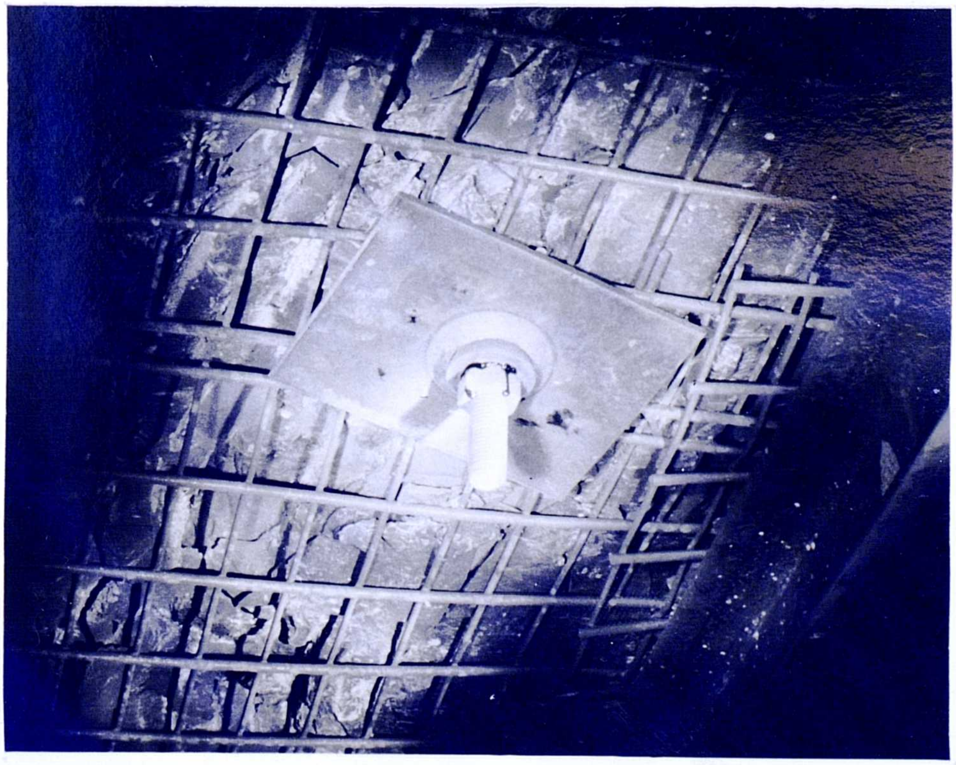
8.2 Reinforcement Of The Face Entry

Collapse of the strata above the face entry is a common problem encountered when working thin seams by retreat mining or advancing longwalls with advanced headings (Stace 1981). Reinforcement of this area using rock bolts installed in advance of the face can improve face end conditions considerably.

Figure 8.4 illustrates the resin grouted rock bolt reinforcement pattern used to stabilize the face entry of N31's advance in the Harvey Seam at Eppleton Colliery (NCB 1979). The seam lies at a depth of 400 m and was overlain by a weak mudstone roof, varying in thickness from 1.78 to 2.26 m, with a sandstone horizon above. The headings were formed by drilling and firing, and supported by steel arches at 1 m centres. This system of reinforcement was systematically installed in each arch bay 10 m in advance of the face and was seen to successfully maintain conditions at the face entry.



a Incorrectly installed



b Correctly installed

Figure 8.3 Resin grouted roof bolts

8.3 Dual Roof Bolt - Steel Standing Support Systems For Gateroads Serving Advancing Faces

Currently the majority of longwall faces in British coal mines are advancing. Although, in recent years there has been a trend towards retreat mining. The benefits and limitations of the two mining methods have recently been discussed by Mills (1985) and Northard (1986).

There are four systems used to form gateroads of advancing faces: advanced headings; half-head rippings; in-line rippings and conventional rippings. Roof convergence is generally much less if the roads are driven in-line with or behind the face, rather than ahead of it.

The effectiveness of a dual full column resin anchored rock bolt reinforcement and steel arch support system for an advanced heading has been evaluated in the Clowne Seam at Whitwell Colliery (Charlesworth and Stokes 1970; Hodgkinson 1971; Murphy et al 1972). The benefits gained by point anchored rock bolting in this heading have been discussed previously (Section 7.4.2). A similar pattern of five 2.14 m full column anchored bolts were later installed in the heading between each set of standing supports. Strata displacement measurements showed a considerable reduction in roadway convergence compared with a section of roadway without rock bolt reinforcement. The full column resin bolting also achieved greater control of the roof beds between the heading face and the T-junction than could be obtained with point anchored bolts.

The application of a dual support system employed in advancing a rip 5.5 m behind 12's Nine Feet face at Baddesley Colliery has been reported by Barratt and Altounyan (1980) and Barratt (1981). At the commencement of the trials, both gates were experiencing problems with roadway closure. The return gate (at a depth of 550 m) was supported by 2.74 x 2.44 m two-piece arches (105 x 105 mm section) at 0.915 m centres. Following the introduction of systematic roof bolting between each steel set (using three 1.8 m long, 20 mm diameter rebar bolts, full column resin anchored, Figure 8.5), the arch centres were able to be increased to 1.22 m centres. At the same time a distinct improvement in roadway conditions was observed.

Following the success at Baddesley, a dual support system trial was initiated at Bullcliffe Wood Colliery (Barratt 1980) in S.10's tailgate in the Lower Fenton Seam (220 m deep). Improvements in roadway conditions were noted (Mallory 1984) following the introduction of a pattern of five 1.675 m long, 25 mm diameter full column resin bonded rebar bolts between each arch set (at 1.2 m centres). The cycle of working at the rip prevented the erection of the Victor Pegasus drilling machine in front of the slusher haulage frame. Consequently, the bolts were installed up to 8 m outbye of the rip; this was probably too late to reduce bed separation and roof dilation effectively.

8.4 Rock Bolting As The Primary Support For Gateroads Of Advancing Faces

Georgel and Raffoux (1968) and Raffoux (1971) reported one of the first field trials using full column resin grouted roof bolts in the Lorraine Basin Coalfield, France. The bottom gate of an advancing face at La Houve Colliery, formed by an advanced heading, contained two trial sections; one being supported by arches over a length of 100 m and the other by roof bolts over a length of 80 m. The roof bolted part of the gate was driven with a trapezoidal section in order to eliminate the need for a bottom stable hole. Nine bolts were installed in the roof and two in the ribs every metre of drivage as depicted in Figure 8.6. The roadway was lagged with galvanized mesh. In the bolted section, additional supports (hydraulic props and roof bars) were set at 5 m intervals ahead of the longwall face and then replaced by timber chocks and wooden bars set on friction props behind the face.

Roof convergence monitoring established that rock bolting reduced convergence in the centre of the roadway by approximately 30%. In the bolted section, roof lowering occurred evenly over most of the roadway width during and after the passage of the face, preventing strata fracturing.

8.5 Dual Roof Bolt - Steel Standing Support Systems For Gateroads Serving Retreating Faces

8.5.1 Post-development Rock Bolt Reinforcement

Retreat roadways where unstable roof conditions have been encountered during drivage or are expected during longwall extraction operations may be reinforced with roof bolts prior to face retreat.

The monitoring of two such roadways was undertaken by Breckels (1978) at Lea Hall Colliery and University College Cardiff (1987)/Oram (1987) at Betws Colliery. Details of these two sites are given in Figure 8.7. In

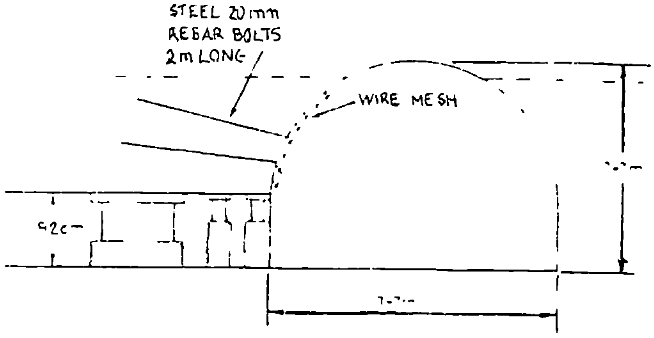


Figure 8.4 Rock bolt reinforcement of N31's face entry, Eppleton Colliery.

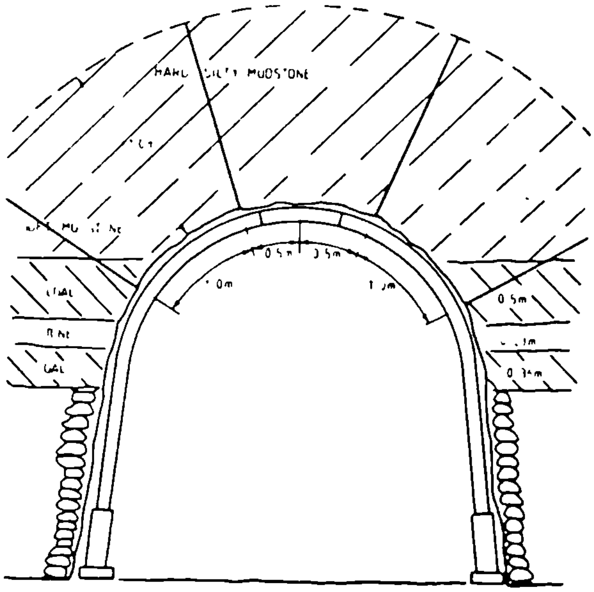


Figure 8.5 Original bolting design for 12's return gate, Baddesley Colliery.

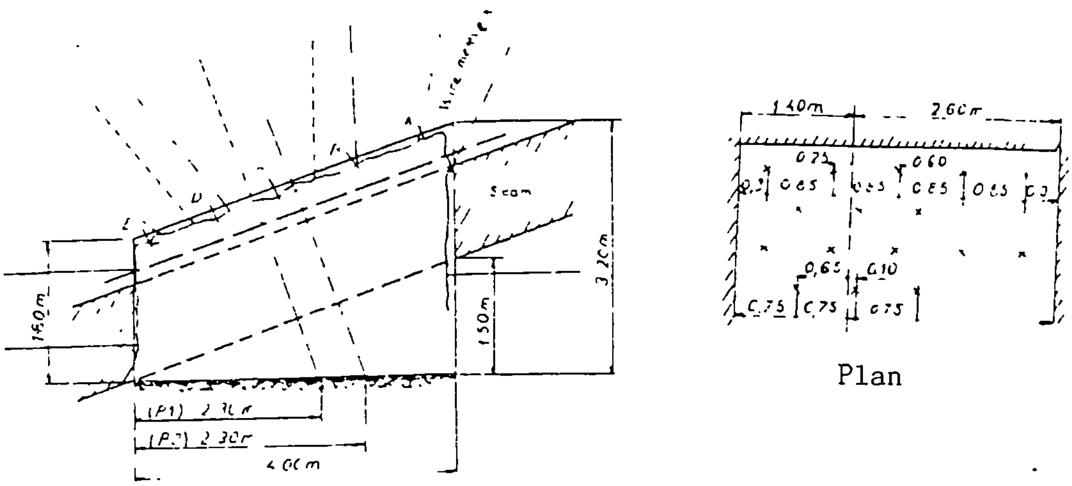
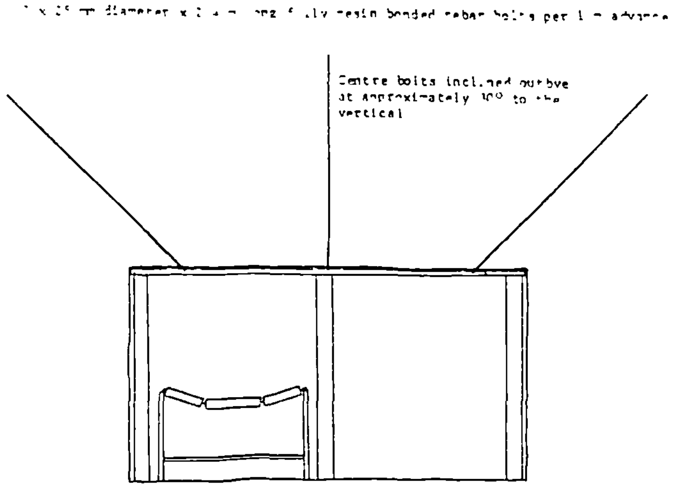
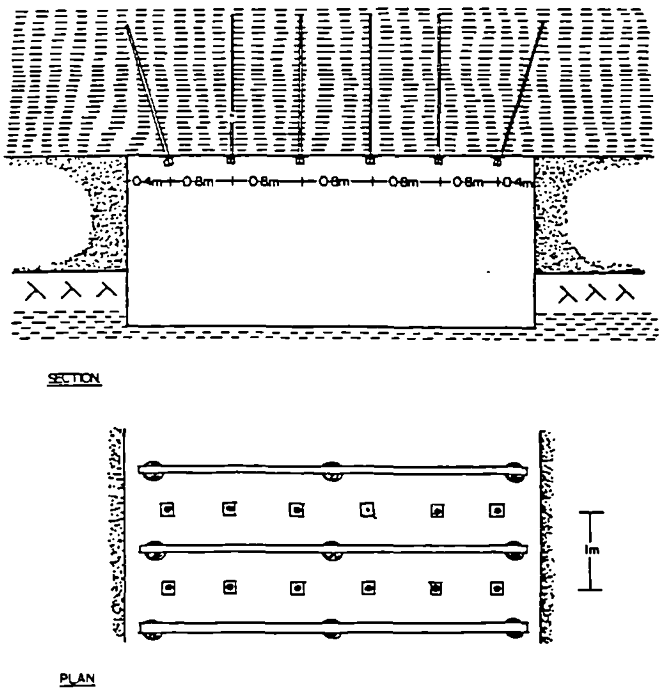


Figure 8.6 Rock bolt reinforcement of an advanced heading, La Houvre Colliery (after Georgel & Raffoux 1968).



Colliery: Lea Hall
Seam: Shallow
Gate: 1062's main gate
Immediate roof: Mudstone
Roadway height: 2.13 m
Roadway width: 3.96 m
Bolt length: 2.44 m
Bolt diameter: 25 mm
Drill bit size: 43 mm
Resin capsule diameter: 40 mm
Reference: Breckels (1978)



Colliery: Betws
Seam: Red Vein
Gate: S3's main gate
Depth: 750-770 m
Immediate roof: Carbonaceous mudstone
Roadway height: 2.2 m
Roadway width: 5.0 m
Standing support: RSJ roof bar and 3 wooden props
Bolt length: 1.8 m
Bolt diameter: 25 mm
Drill bit size: 33 mm
Resin capsule diameter: 30/32 mm
Reference: University College Cardiff (1987)

Figure 8.7 Post development reinforcement of retreat gateroads.

both cases systematic roof bolting was successful in maintaining the integrity of the immediate roof strata and at Betws it appeared that floor heave was also reduced.

It is important to install the bolts well ahead of the front abutment zone. Although improvements in roadway conditions are frequently gained from post-development rock bolt reinforcement, bed separation and strata fracturing may occur during the time delay between excavation and the bolting operations. This can lead to installation difficulties such as resin loss in fractures and rock spalling during drilling. Equipment within the entry (e.g. conveyors etc.) can also cause problems in drill positioning. It is therefore often far more beneficial to install rock bolt reinforcement systems during drivage operations.

8.5.2 Roof Bolt Performance In Differing Lithologies

The development of 1's drivages in the No.7 Seam at Snowdown Colliery provides a fine example of how slight differences in the immediate roof lithology can affect the performance of a rock bolting/steel work combined support system.

The No.7 Seam was accessed at a depth of approximately 920 m by two parallel drifts driven from No.6 Seam level adjacent to the pit bottom. From here these roads were driven forward as gateroads which were intended to serve 1's retreating face; the first workings within the No.7 Seam at the colliery. 1's panel was to have had a face length of 200 m and a run of 630 m.

Flat topped supports, 2.31 x 4.01 m (127 x 114 mm section) were set at 1 m intervals with plated joints and nine tubular struts per setting. These supports were supplied to the colliery as packaged units as there was a restrictive shaft size and a relatively small labour force. In both roadways middle legs were employed but owing to congestion at the immediate roadhead it was not possible to install these nearer than 20 m from the head end. The supports were set on wooden pads and lagged with corrugated sheets. Dosco Dintheaders were used to excavate both drivages.

The tail gate was the first of the drivages to start up. During the initial stages of development the immediate mudstone roof (which was considered to be relatively strong) was breaking up during the cutting operation. Parting planes were present at 0.1 m, 0.25 m, 0.33 m and

0.9 m above the seam. Listricated joints 20 to 30° to the vertical were evident running obliquely into the heading. The combined effect of these discontinuities led to cavitation, generally up to 1 m and occasionally extending to 2.3 m above the roadway. Heavy support loading caused the cross beams to deflect before the middle legs could be set (Coates 1987). To overcome these problems support changes were made in three stages as follows:

(i) Roof bolting with inclined holes of approximately 30 to 45° in advance of the head using 2.4 m long, 25 mm diameter rebar fully bonded with resin grout.

(ii) As soon as the roof was held by the advance bolting, systematic roof bolting was undertaken using four 1.8 m long, 25 mm diameter rebar bolts between each arch setting installed perpendicular to the roof. The shoulder bolts were inclined at 45° over the ribsides and the two remaining bolts were equally spaced across the roof and drilled vertically.

(iii) A heavier section cross beam was used, so that a 152 x 127 mm beam was set on 127 x 114 mm legs.

Improvements in roof conditions were noted as each stage was initiated, so that once a 20 m length had been established there was no further cross beam deflection. Cavitation was not encountered again until an area of faulted ground was excavated during the later stages of development. Measurements by colliery survey staff also noted a reduction in the amount of floor heave following the initiation of systematic roof bolting. Levels of floor lift as high as 1.3 m within 25 m of the head end were noted prior to bolting; this was reduced to an average of 0.6 m, 50 m behind the head once systematic bolting was established.

In the main gate the seam was slightly thicker, approximately 1.47 m as opposed to 1.22 m in the tail gate. The strata surrounding the seam was also considerably weaker, the immediate roof being a seatearth of a Ryder bed of coal. Considerable minor faulting had been recorded while excavating the strata above the seam in the drift. A washout also affected an 11 m section of the main gate. Here the seam was totally washed out and the immediate strata consisted of seatearths and thin coal bands.

Simple roof to floor measurements taken after the main gate had advanced 58 m established that approximately 0.34 m of roof lowering had occurred in the first 20 m outbye of the head end. In addition, over the same distance, 0.66 m of floor lift had taken place giving a total vertical closure of 1.0 m i.e. a loss of 43% of the original height (Mosley 1986). It was found necessary to take a dint in the roadway 30 m outbye, which proved difficult due to the slabby floor. Floor heave continued to occur at a similar rate after the dinting operation.

Scale models of the roadway were constructed in the HQTD roadway model rig. The model strata configuration used in the tests (Appendix 1d) was based on a very limited amount of geotechnical data, principally from a descriptive log of SU 18 borehole which was put down close to the main conveyor drift prior to development (Figure 8.8). Two models were tested, one having standing supports only and the other with rows of five fully bonded roof bolts with an equivalent length of 1.8 m between the standing supports. The two shoulder bolts were angled at 45^0 over the ribsides and the remaining three bolts were vertical, spaced equally across the roadway. Measurements from the test photographs (Figure 8.9) showed that bolting would give a limited improvement by reducing the amount of roof lowering. However, it appeared that floor heave would remain a major problem.

Systematic roof bolting employed in the main gate gave some improvement but conditions were generally very poor along the whole length of the heading. After 349 m of development it was decided to stop the drivage; the equipment was withdrawn and the heading sealed.

8.5.3 Roof Bolting In Delaminating Strata

2DR's tail gate serves the first longwall face to be worked at Riccall Mine. It is a retreat panel in the Barnsley Seam (at a depth of 790 m) with a face length of 150 m and run of approximately 1500 m. The roadway standing support consists of flat topped arches, the 4.26 m cross beam being 152 x 127 mm in section, with 2.89 m legs of 127 x 113 mm section set at 1 m intervals. The seam in this area of the mine is 2.4 m thick, with an immediate roof of friable shaley mudstone which grades upwards into a much stronger silty mudstone/siltstone. Beneath the seam is a 0.8 m thick, weak seatearth mudstone underlain by a thin band of coal. The direction of drivage was north easterly, with the cleat at approximately 45^0 to roadway advance.

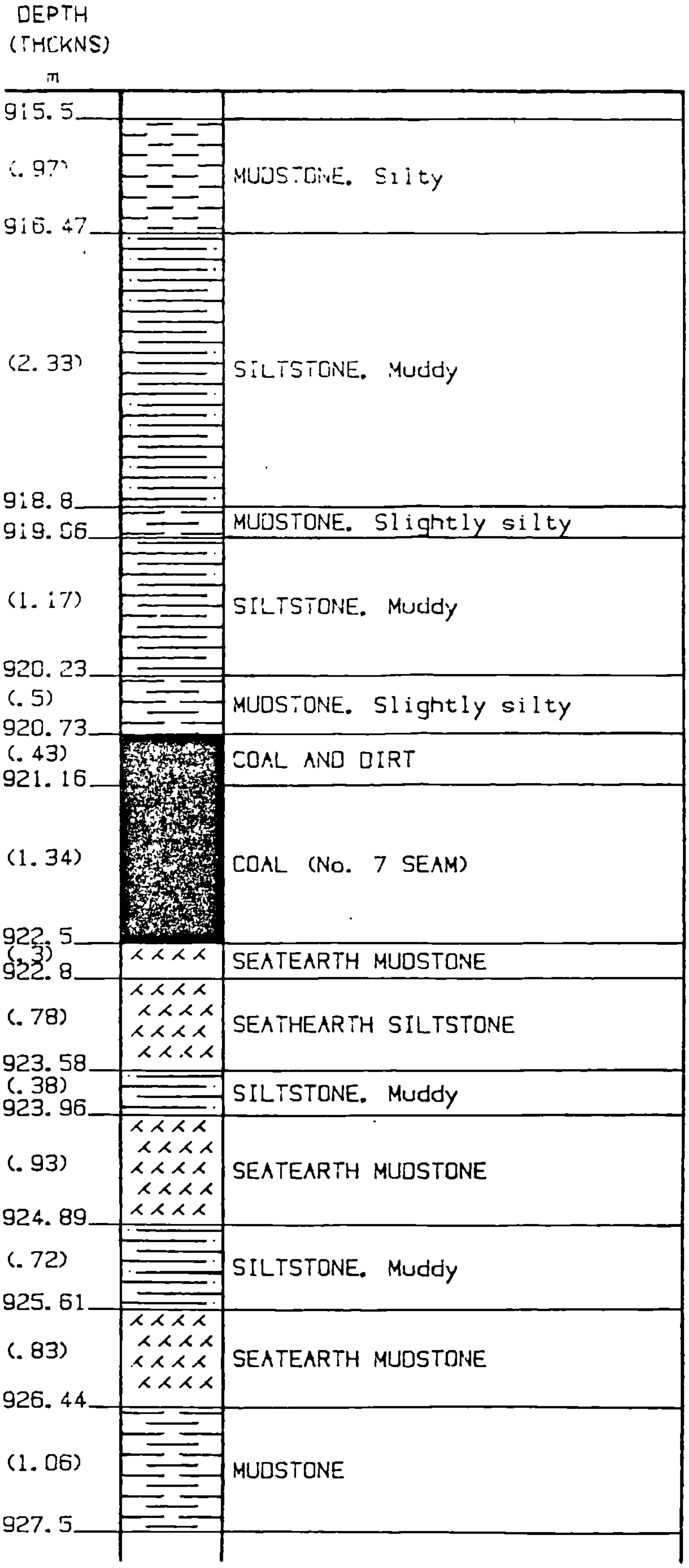


Figure 8.8 Section of strata in the vicinity of the main conveyor drift, Snowdown Colliery.

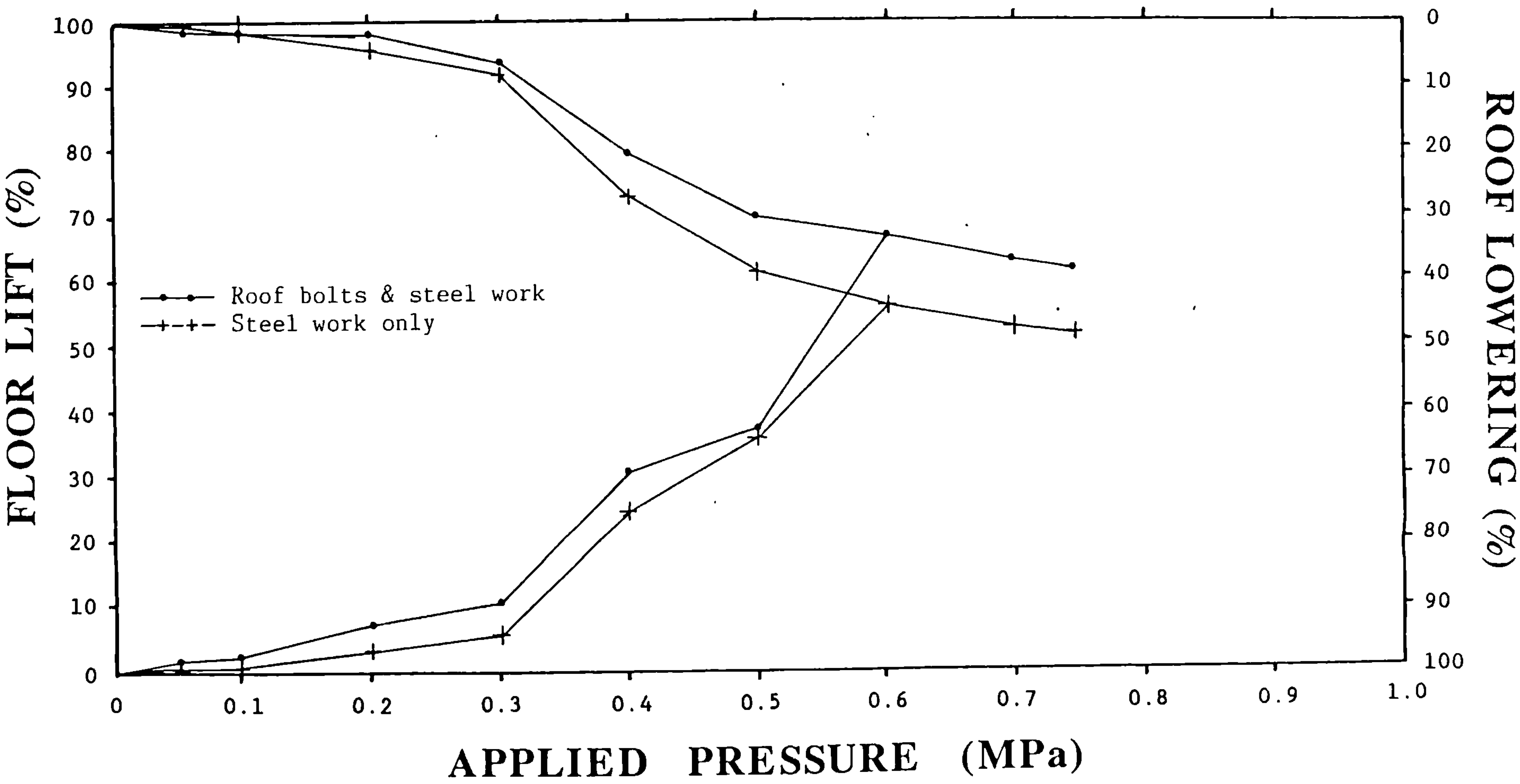


Figure 8.9 Variation in roof lowering and floor lift as a percentage of initial roadway height with applied pressure - Snowdown Colliery scale models.

The supports were set to a parting between the coal and overlying mudstone. After a very short distance of drivage the friable mudstone broke away leaving cavities above the supports. These cavities were filled with chock wood which created point loading on the support beam, causing girder deflection and necessitating the installation of wooden centre legs.

In an effort to prevent delamination of the mudstone, four 25 mm diameter, 2.44 m long fully resin grouted bolts were angled forward into the roof at the head end every 2 m of advance. Once the mudstone roof was held, the roof bolt support system was altered to four 25 mm diameter, 1.70 m long fully resin grouted bolts installed vertically in 35 mm diameter holes.

Closure recorded at a monitoring station installed in this section of the roadway is shown in Figure 8.10. Deflection of the girders took place only a few days after the installation and lateral movement within the bolted section of the roof strata damaged the pipe of a magnetic extensometer 0.8 m into the roof, preventing access to anchors at a higher level.

The roof bolt pattern was altered to six bolts; two 2.44 m long angled over the ribsides and four 1.70 m long bolts vertically across the roadway. Single point wire extensometers installed in this section of the roadway recorded 99 mm of bed separation occuring between the roof line and 3 m above the seam; 23 mm of which was within the bolted strata. Open boreholes indicated lateral movement at 0.5 m and 1.3 m above the roof line.

Improved conditions had resulted from the roof bolting operations. Although in general, bolting had not prevented delamination of the immediate roof or eliminated girder distortion.

8.5.4 Roof Bolting In Friable Strata

Success has recently been achieved using a dual support system in friable ground in the Yard/Blackshale Seam at Rufford Colliery. A series of faces is planned in the next 15 years in this seam and 206's is the first of these. The reserves correspond to 4 km^2 with an extraction height of about 3 m. It is intended to work retreat faces with runs of approximately 1100 m . Details of the strata in the vicinity of 206's are shown in Figure 8.11.

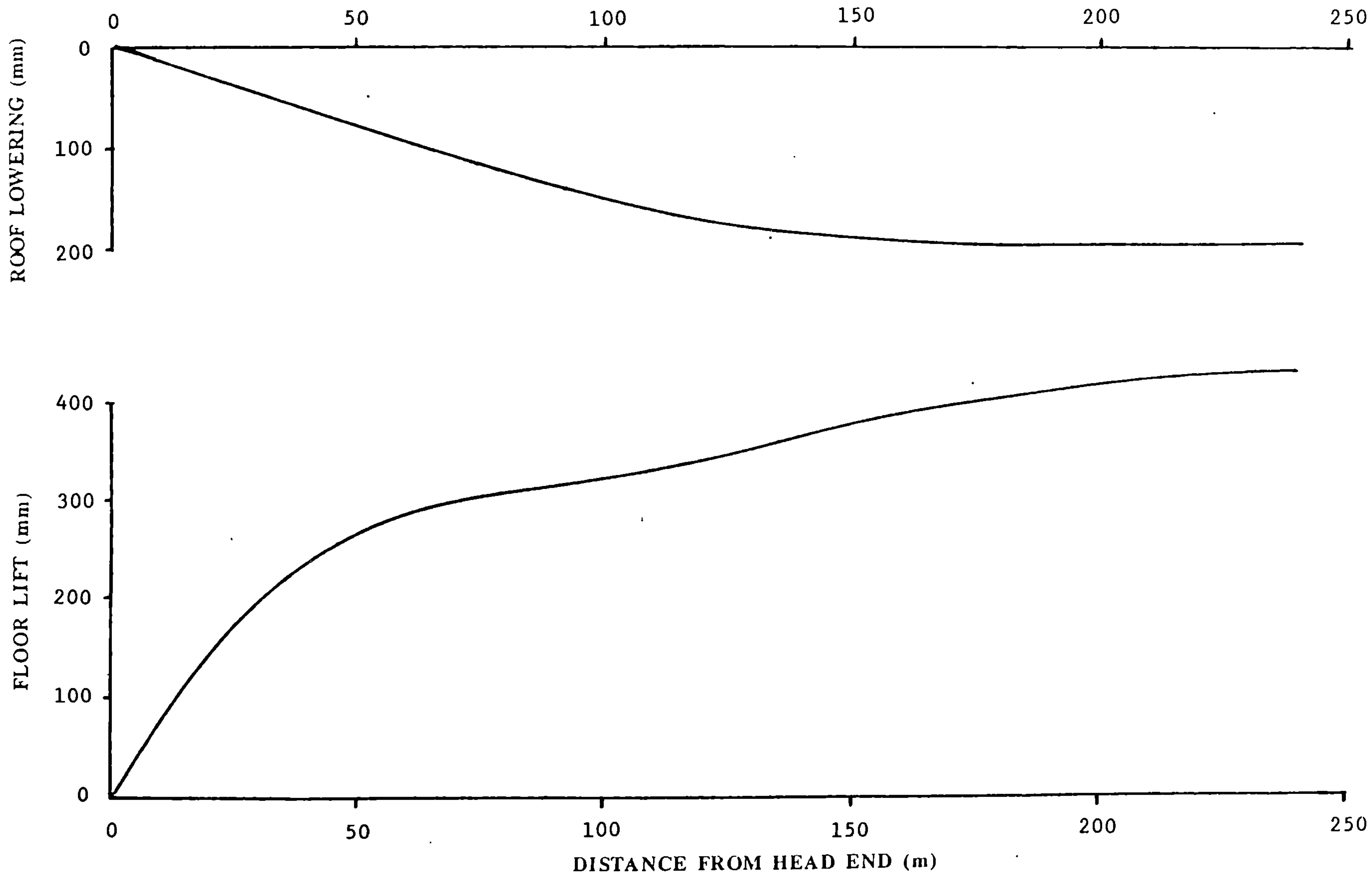


Figure 8.10 Closure recorded at a monitoring station in 2DR's tail gate, Riccall Colliery.

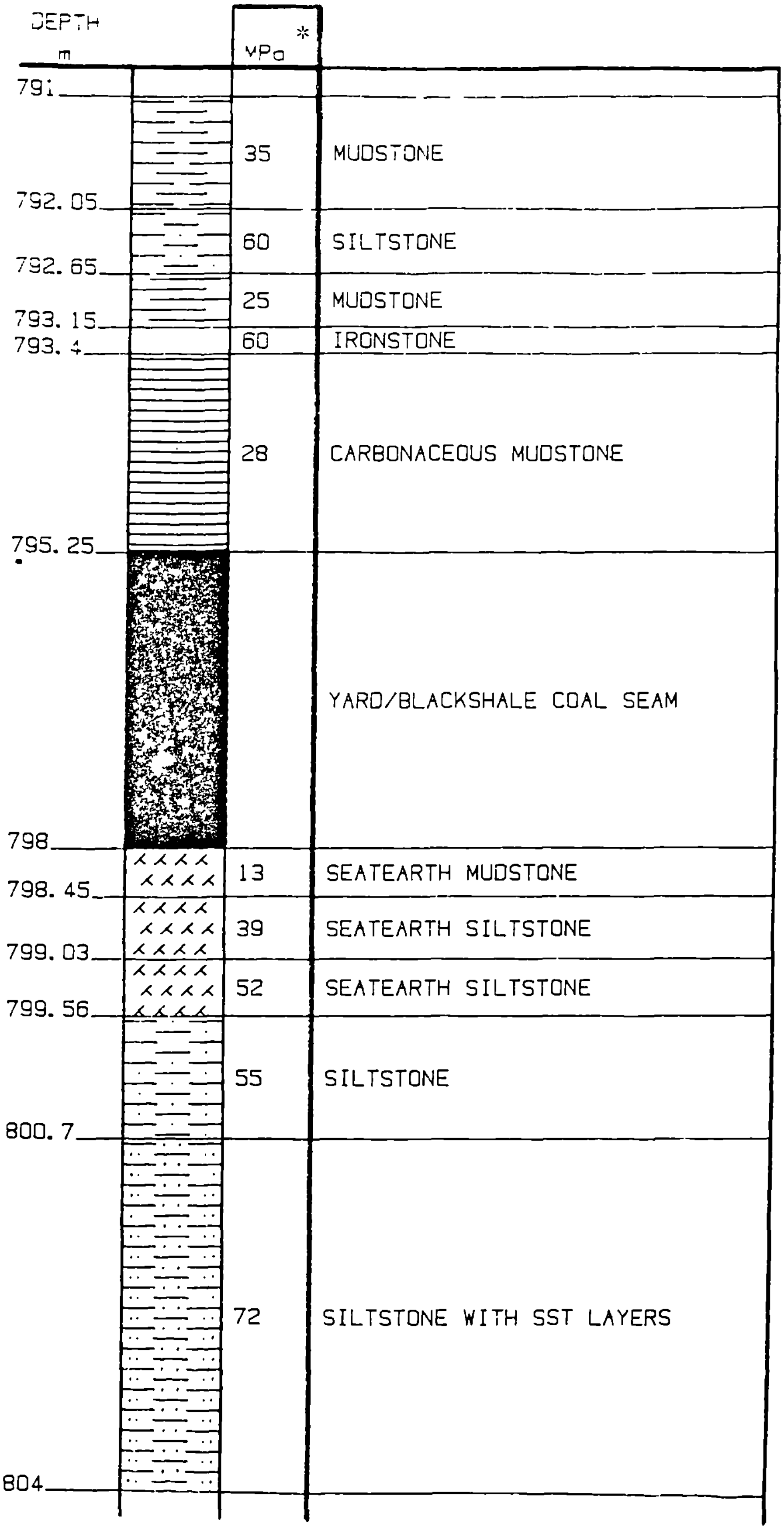


Figure 8.11 Section of strata in the vicinity of 206's panel, Rufford Colliery.

* Uniaxial compressive strength

206's main gate was driven with a Dosco Mk 2A heading machine. Initially the roadway was supported with flat topped supports, 5.23 x 3.45 m, installed at 1 m intervals and lagged with corrugated sheets (Figure 8.12).

The mudstone roof in the main gate is thinly laminated, very friable and contains narrow bands of nodular siderite; minor compactional faults are also present. Consequently the roof proved to be relatively unstable and the immediate 0.65 to 1.0 m of rock frequently broke away. As has been observed previously in the Yard/Blackshale Seam, once this roof is broken it is very difficult to contain. This in turn leads to greater cavities forming with point loading and dead weight acting upon the roadway supports. Soon after the road was driven middle legs were installed in an effort to prevent roof beam distortion, but with very little success (Figure 8.14a). At this stage it was becoming increasingly obvious that this drivage would have great difficulty maintaining its profile for approximately two years to serve a retreating face without very expensive and time consuming repair work being undertaken. It was therefore decided to monitor roadway conditions to establish where bed separation was occurring, the degree of roof lowering, floor lift and dead load acting upon the flat topped supports. Rock samples were gathered to determine physical properties of the strata in the immediate vicinity of the roadway. These results gave the opportunity to simulate underground conditions in the HQTD scale model rig (Figure 8.14b and c).

Simple bending theory shows that by introducing a 4.57 x 3.60 m support (Figure 8.13), together with a pattern of five 1.82 m long rock bolts, the resistance to roof beam buckling could be reduced dramatically. When it was established that the new proposed roadway shape could still accommodate the equipment required for materials and transportation, the change in support design was adopted virtually overnight.

Initially the five fully resin grouted, pre-tensioned, 1.82 m long bolts were installed vertically in the crown of the roadway between every free standing support. This bolting pattern was employed for a distance of 46 m before it was found that slight roof beam deflection was still taking place. Although no middle legs were necessary it was considered that if the two outer roof bolts were to be angled over the solid

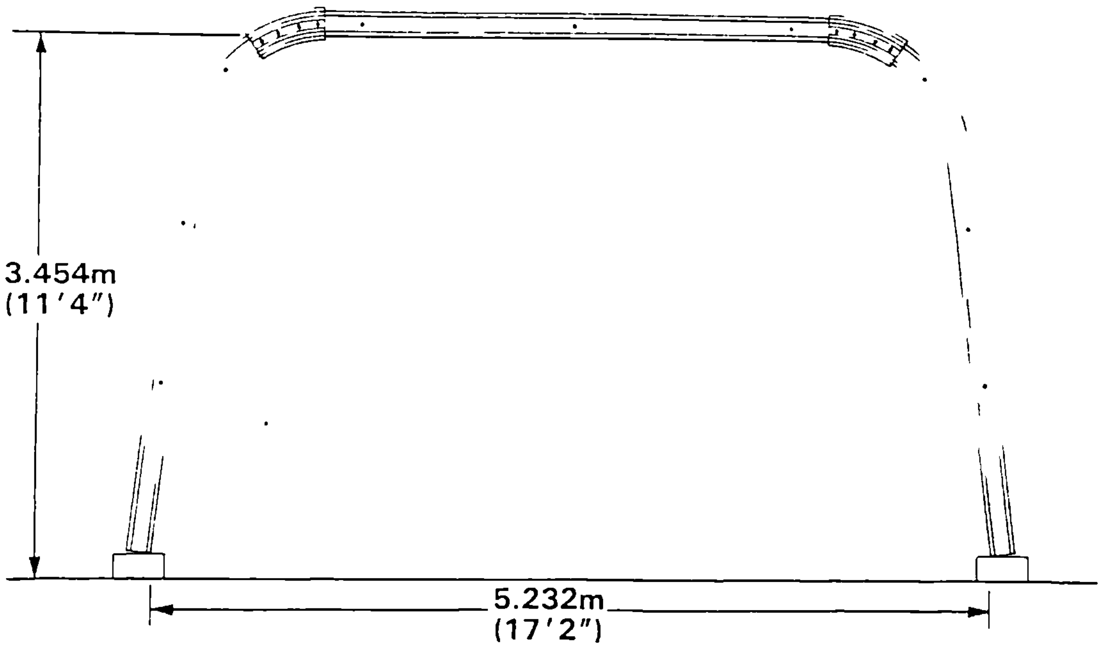


Figure 8.12 Original steel standing support for 206's main gate, Rufford Colliery.

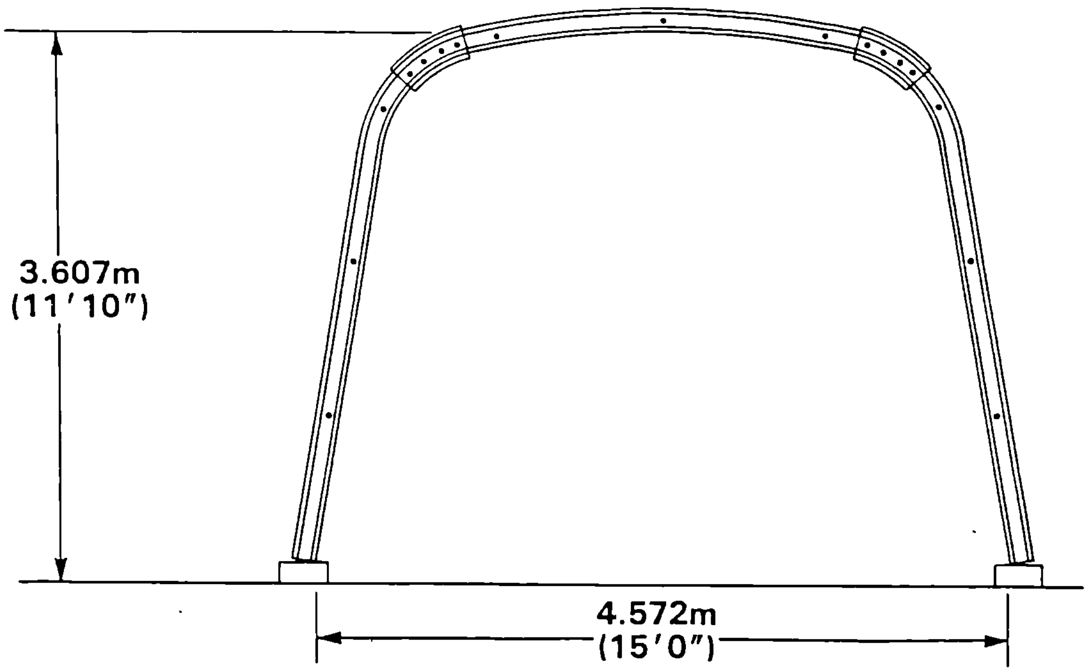


Figure 8.13 Re-designed steel standing support for 206's main gate, Rufford Colliery.

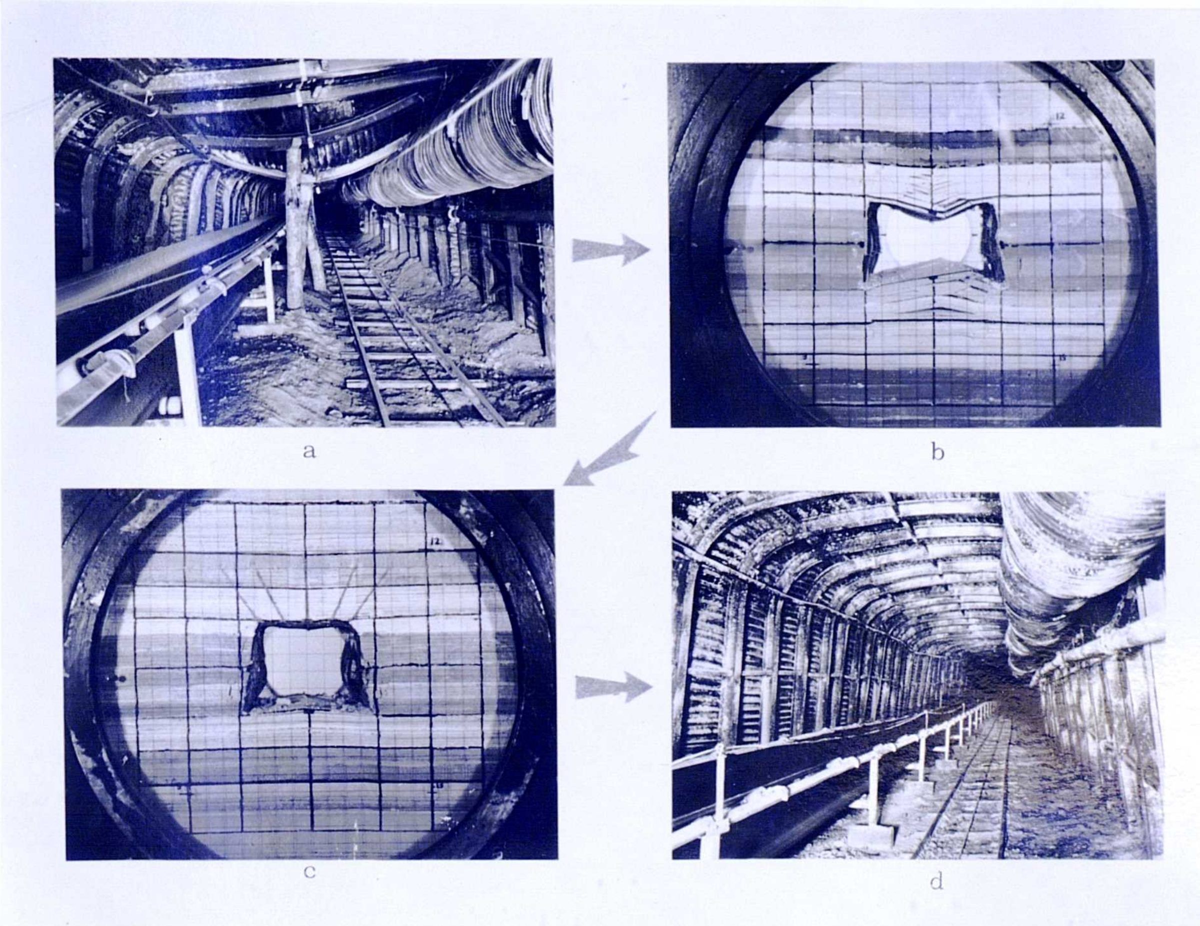


Figure 8.14 Re-design of 206's main gate at Rufford Colliery

roadway sides this would create a longer reinforced strata beam to bridge across the width of the roadway. This gave an immediate improvement on the roadway profile (Figure 8.14d).

On realising the improvements that bolting the roof could produce, colliery management agreed that 206's main gate could be used as an experimental roadway for roof bolting trials. It was hoped that this would enable the determination of optimum bolting patterns for this roadway and future drivages in the Yard/Blackshale Seam. It was decided to change the bolting pattern at intervals along the drivage and set up measuring stations within each section, as this would enable any differing strata movements and loading characteristics to be established.

A variety of bolting patterns have been employed using 1.83 m, 2.13 m and 2.44 m bolts in various configurations. Each pattern consisted of five roof bolts with the outer two being angled at approximately 45^{0} over the roadway shoulders. It is considered that further improvements have been created by the use of longer bolts which are anchored in a relatively competent siltstone horizon overlying the friable mudstone (Figure 8.15).

A scale model was prepared (using the model strata configuration shown in Appendix 1e) representing the original roadway dimensions; when loaded, this deformed in a very similar manner to the underground situation, thus establishing that the conditions in 206's main gate could be satisfactorily simulated in the model rig. Further model tests showed that altering the roadway support size, shape and density would reduce both roof lowering and floor lift (Figure 8.16). However, when a pressure equivalent to the hypothetical cover load was applied, the roof beds began to separate and an inverted V-shaped fracture zone developed above the roadway. Models simulating systematically bolted roof strata indicated that a competent roof beam could be maintained at applied pressures well above cover load. Several bolting patterns were modelled; those simulating five 2.44 m bolts (with the two shoulder bolts inclined over the ribsides) proved to be the most effective pattern for reducing roof convergence.

Due to the friable nature of the immediate roof strata, the extraction horizon was lowered slightly so that the top of the steel arches were set to the parting plane between the coal seam and the overlying carbonaceous mudstone. This meant that the 2.44 m bolts no longer

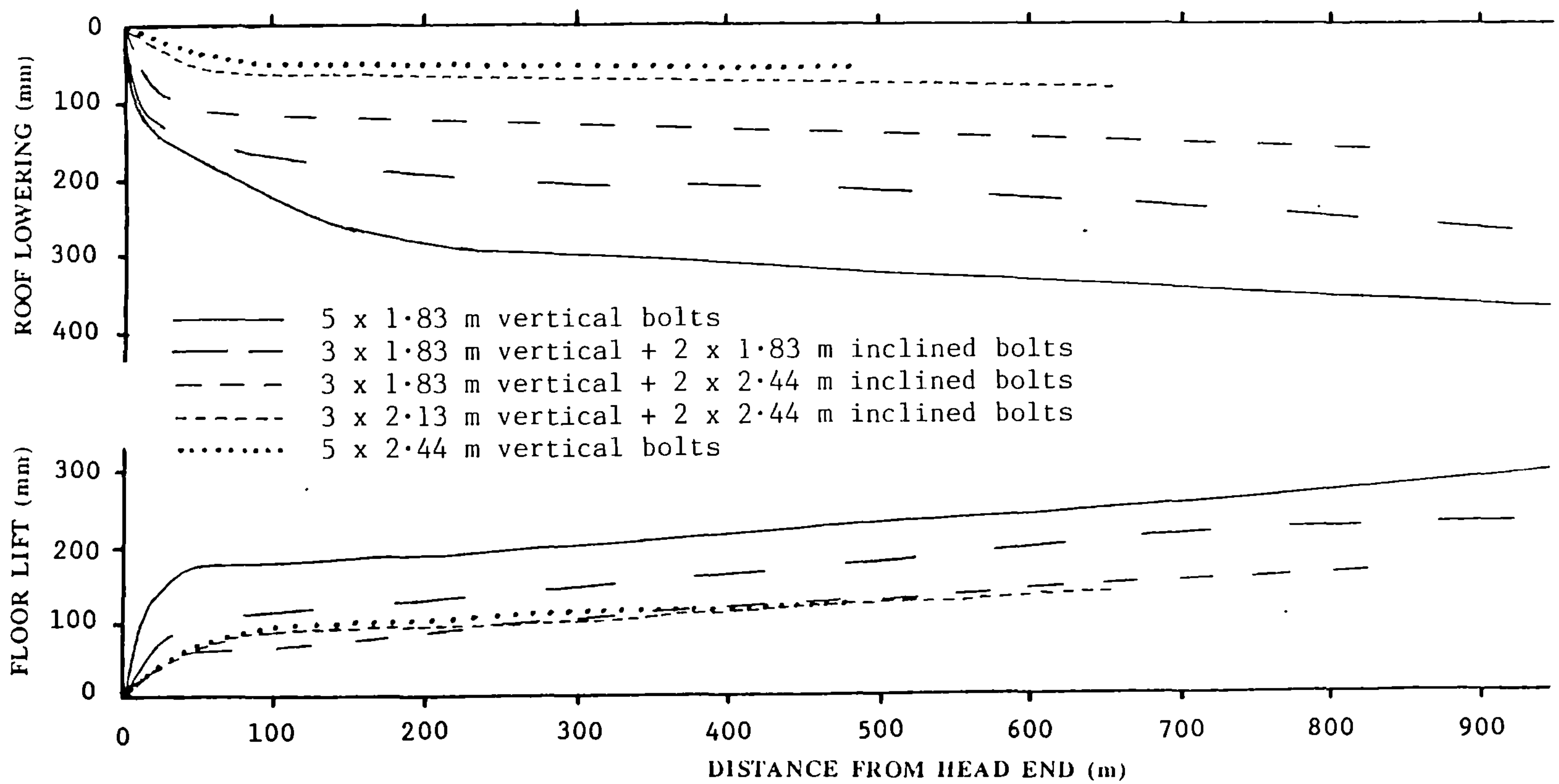


Figure 8.15 Convergence measurements recorded in 206's main gate, Rufford Colliery.

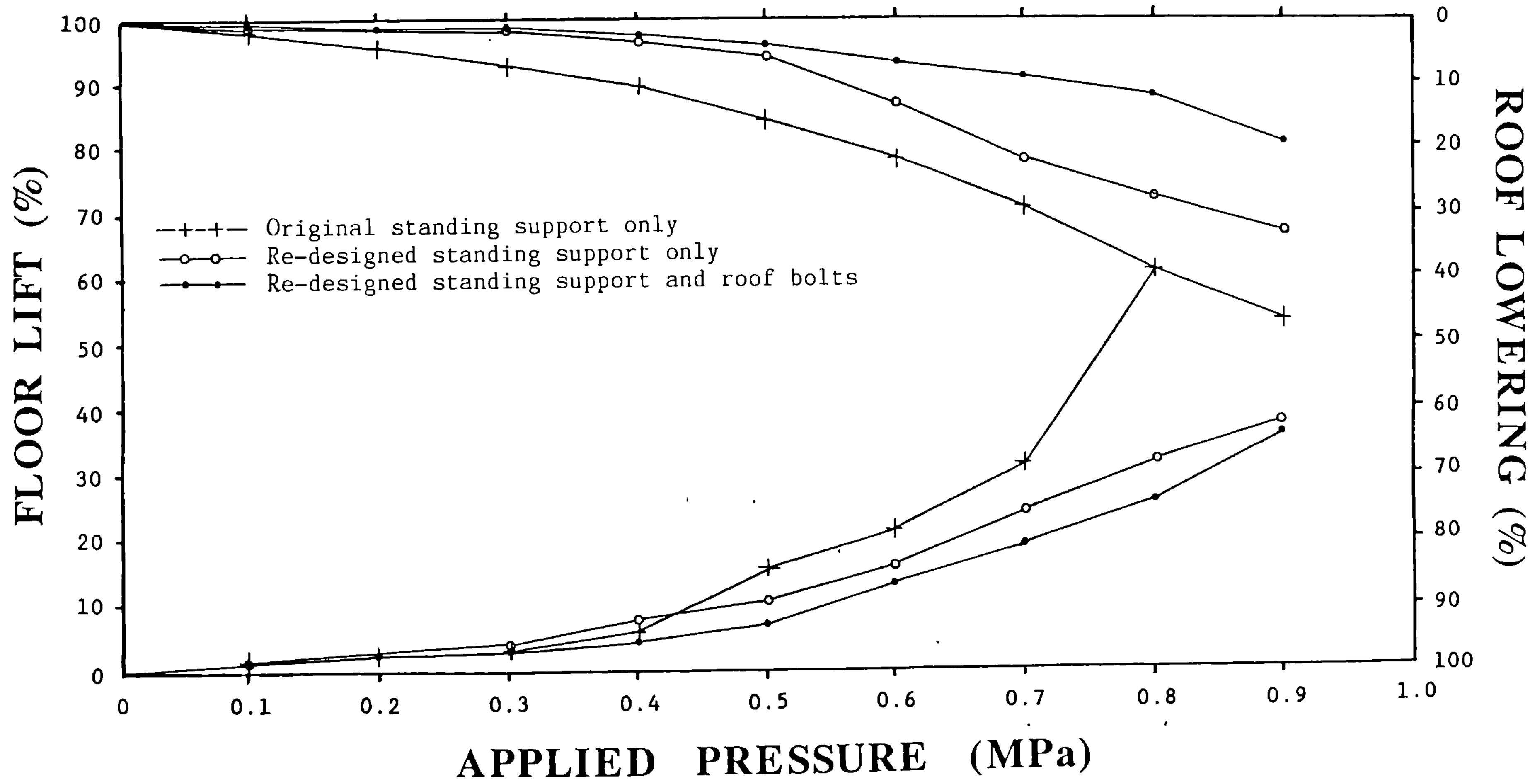


Figure 8.16 Variation in roof lowering and floor lift as a percentage of initial roadway height with applied pressure - Rufford Colliery scale models

reached the siltstone bed. Model tests indicated that no diminution in roof stability was likely when bolt ends were not anchored in the strong siltstone. This suggests that the bolts are probably acting as a reinforcement rather than a means of suspending the weak roof from overlying competent strata, however the model does not adequately simulate dead weight of strata.

The behaviour of the models at high applied pressures showed that when the roadway is subjected to front abutment pressures during face retreat, fracturing and spalling of the coal, particularly in the upper part of the seam, may occur. If unchecked this could result in bowing of the faceside leg and delays at the face entry when this leg is removed, a very vulnerable operation in retreat roadways. The corrugated sheet lagging at the roadway sides was therefore replaced by wire mesh which then enabled the installation of fully grouted wooden dowels to reinforce the faceside in front of the retreating longwall face.

An interesting feature of these trials to date is the apparent reduction in floor lift following the onset of bolting operations and during optimisation of the bolting pattern. There appears to be no variation in the nature of the floor strata which could have resulted in a more stable floor. The reduction of roadway width by 0.66 m will no doubt have had beneficial effects upon conditions. It is considered, however, that this alone is not the sole reason for the marked improvement and that the combined effect of reduced roadway width and roof bolting has created this change in floor heave characteristics.

8.5.5 Roof Bolting To Reduce Steel Work

The support system for 88's Retreat (a 1450 m drivage to serve a rapid retreat longwall face) in the Top Hard Seam at Welbeck Colliery was designed so that the steel standing supports could be erected at 1.12 m intervals. This would enable the shearer to take two cuts of coal per support setting, thus reducing support cost and congestion at the face entry. The Top Hard Seam in this district lies at a depth of approximately 695 m and is surrounded by relatively competent strata (Figure 8.17). The proposed roadway was trapezoidal in shape with a roof width of 3.96 m, floor width of 5.03 m and 2.44 m high.

Scale model studies (using the model strata configuration in Appendix 1f) demonstrated that systematic roof bolting between arches would provide the additional support required to allow this standing support

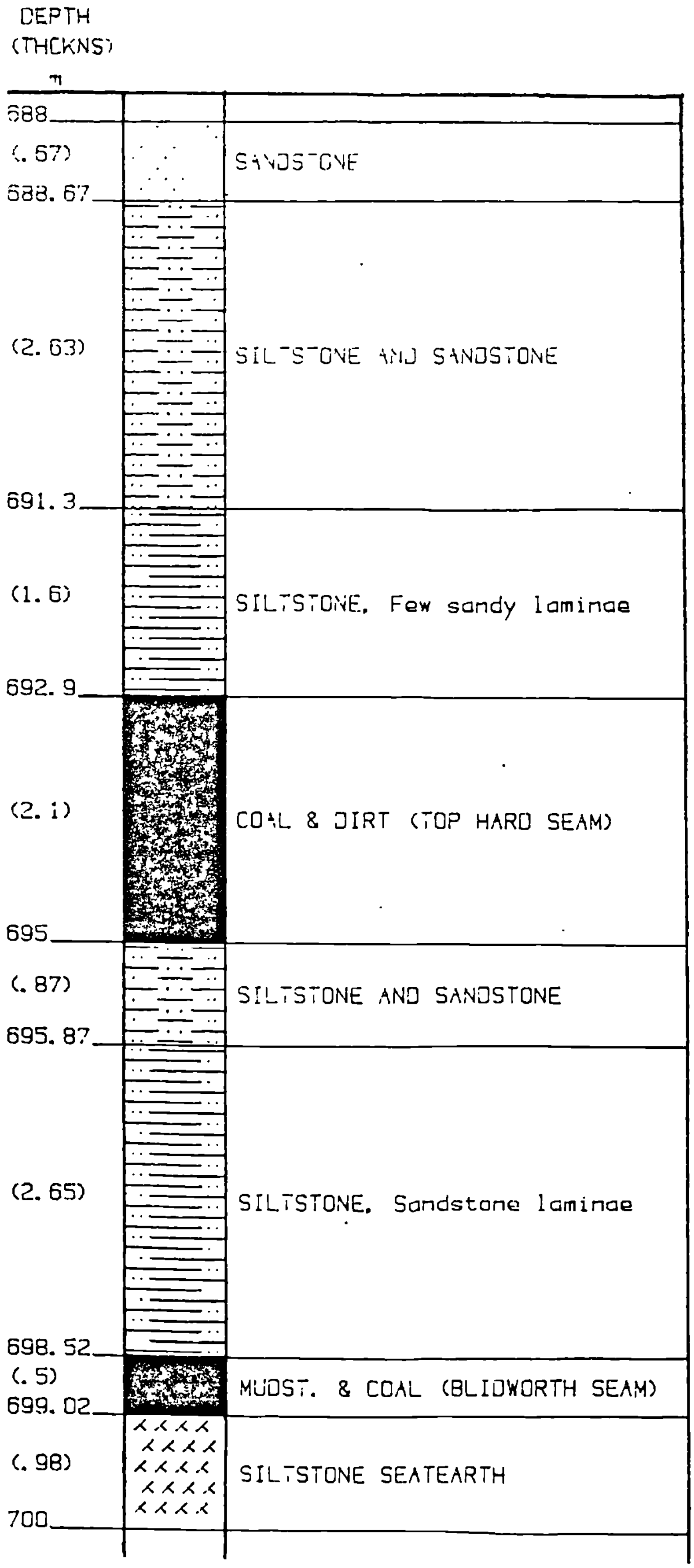


Figure 8.17 Section of strata in the vicinity of 88's panel, Welbeck Colliery.

system to be adopted (Figures 8.18 and 8.19). The photographs in Figure 8.18 illustrate the ability of roof bolting to prevent the development of roof bed separation and cross beam deformation at the hypothetical equivalent cover load (0.5 MPa). The setting of additional supports such as hydraulic props will help to reduce roof beam deformation when the roadway is subjected to stresses above cover load during face retreat. Without roof bolting, cross beam distortion may reach such a degree that the setting of hydraulic props becomes a difficult operation and only has a limited effect at maintaining roadway stability.

8.6 Rock Bolt Reinforcement As The Primary Support For Retreat Drivages

8.6.1 Worldwide Experience

To date no gateroads serving retreating longwall faces have been driven with fully grouted rock bolts as the primary support in any British coal mine. Mining companies throughout the world practising retreat mining in competent rock employ this support method during development and employ additional support, such as chocks, posts and hydraulic props in front of the retreating face.

Multi-entry systems of drivage are popular outside Europe. The US Coal Mine Health and Safety Law (US Government 1977) requires that at all times separate entries should exist for intake and return air and an isolated belt entry.

Roadways at Niederberg Colliery (West Germany) have been used to demonstrate the effectiveness of full column resin anchored rock bolt/wire mesh support systems and to develop operational experience with bolting (Boldt and Fritz 1980; Keck 1981; Lumetzberger 1982; Newson 1986 and Gutberlet 1987). Since the commencement of the research programme in 1978, over 54 km of bolted roadways have been driven. The vast majority were formed with boom headers and the remainder with shotfiring at depths between 400 and 800 m. Retreat mining is practised and where possible the arch-shaped gateroads are used twice.

Seam thicknesses generally vary between 0.7 and 1.4 m. A typical roof consists of 5 m of argillaceous mudstone with an average compressive strength 37 MPa and a tensile strength of 4 MPa. This might be overlain by 9 m of sandy mudstone and then 3 m of sandstone.

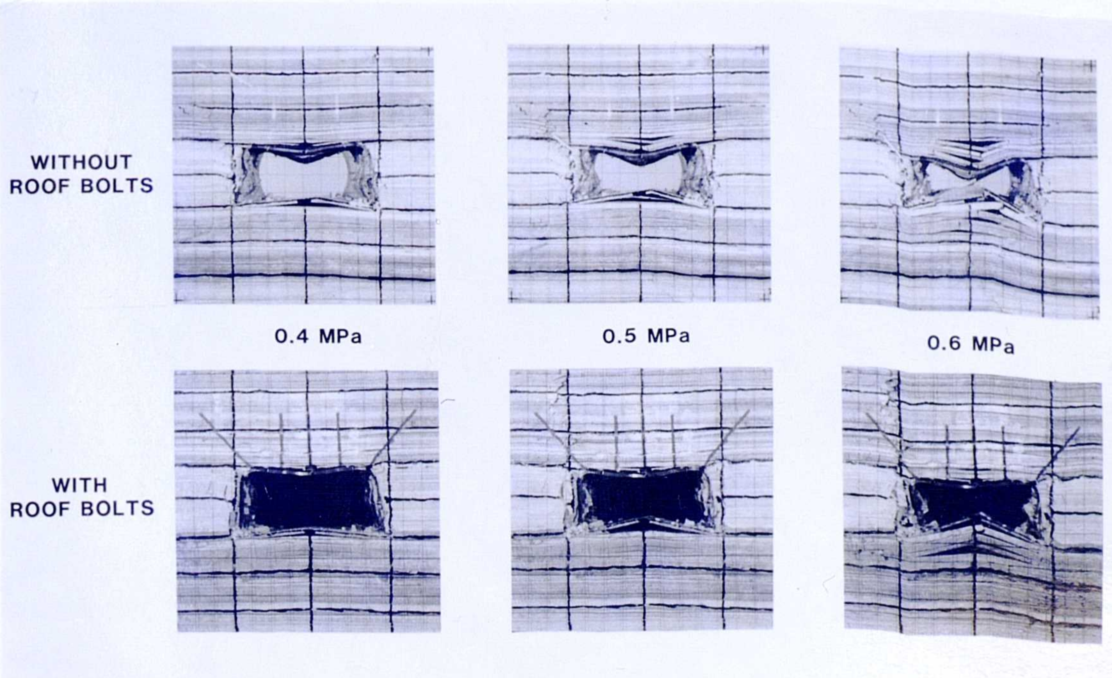


Figure 8.18 Models illustrating the closure of 88's proposed development roadway at Welbeck Colliery, with and without roof bolts

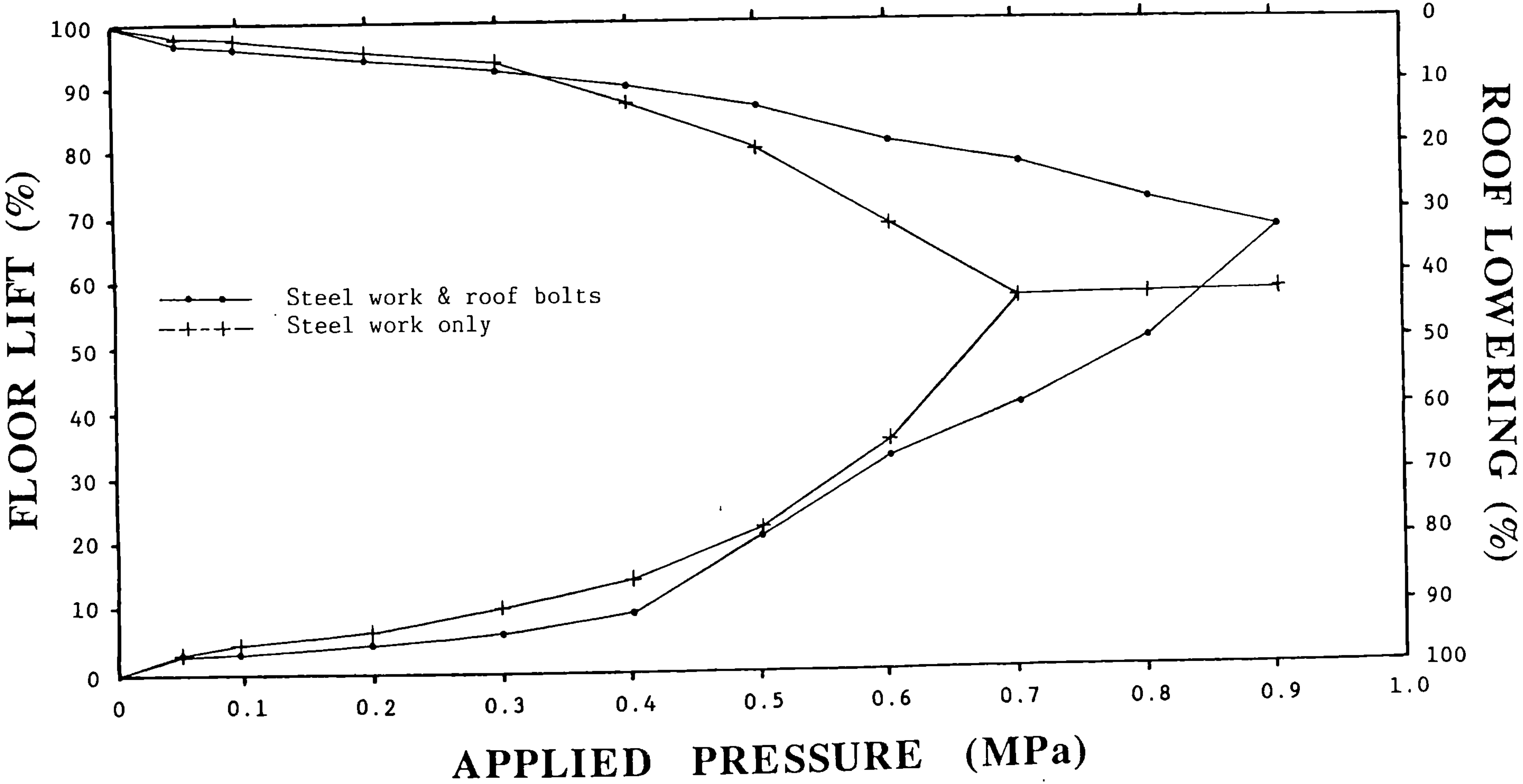


Figure 8.19 Variation in roof lowering and floor lift as a percentage of initial roadway height with applied pressure - Welbeck Colliery scale models.

Rock bolt installation is carried out in two stages during roadway development. The majority of the bolts are set immediately after excavation by a bolting rig (rotary drilled) mounted on the heading machine. Behind the machine a small mobile bolting rig completes the installation. Details of the roadway configuration and rock bolt pattern commonly used are given in Figure 8.20. After 1.5 m of the gateroad has been cut, weld mesh is fixed to the last row of bolts by the aid of supplementary bearing plates. The holes for the next row of bolts are then drilled through the weld mesh. The heading rate in the gateroads is about 8 m per day.

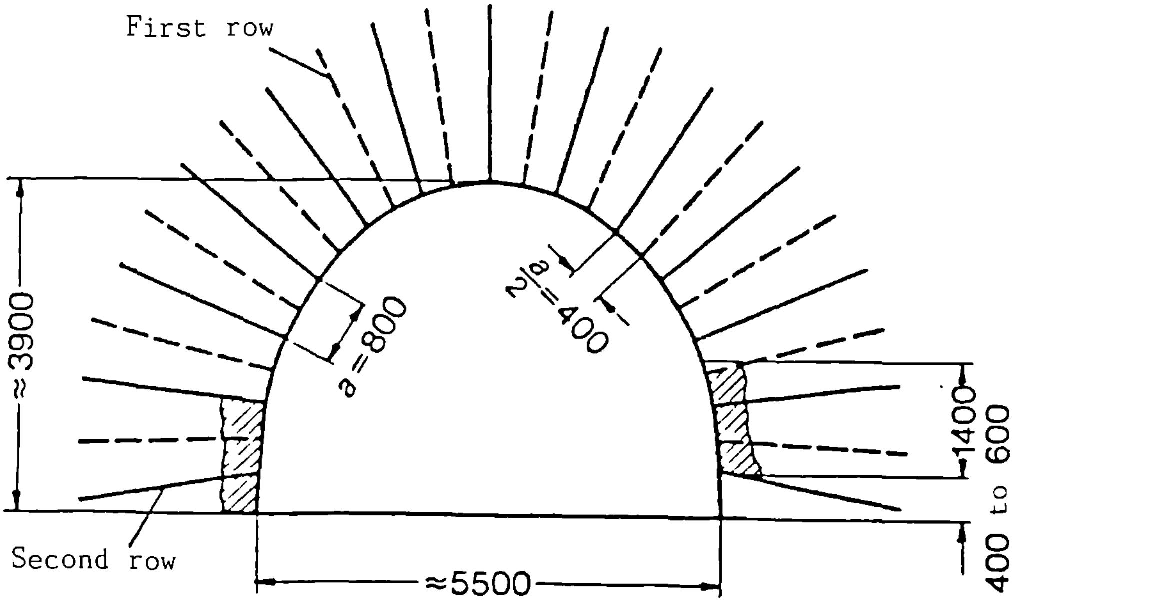
Generally these roadways stand very well and during the passage of the first face no additional support measures are required. A gateside pack made of rapidly setting materials is placed as soon as possible behind the face. The second use of a roadway usually necessitates the setting of one or two rows of articulated roof bars with hydraulic single props fixed at an angle in the T-junction.

Bolted roadways subjected to interaction pressures or influenced by geological discontinuities tend to have stability problems. Beds can become detached and fold themselves into the lagging, which may eventually cause it to rupture, resulting in cavity formation. Under these circumstances additional bolts and/or standing supports are installed.

8.6.2 Scale Model Feasibility Study

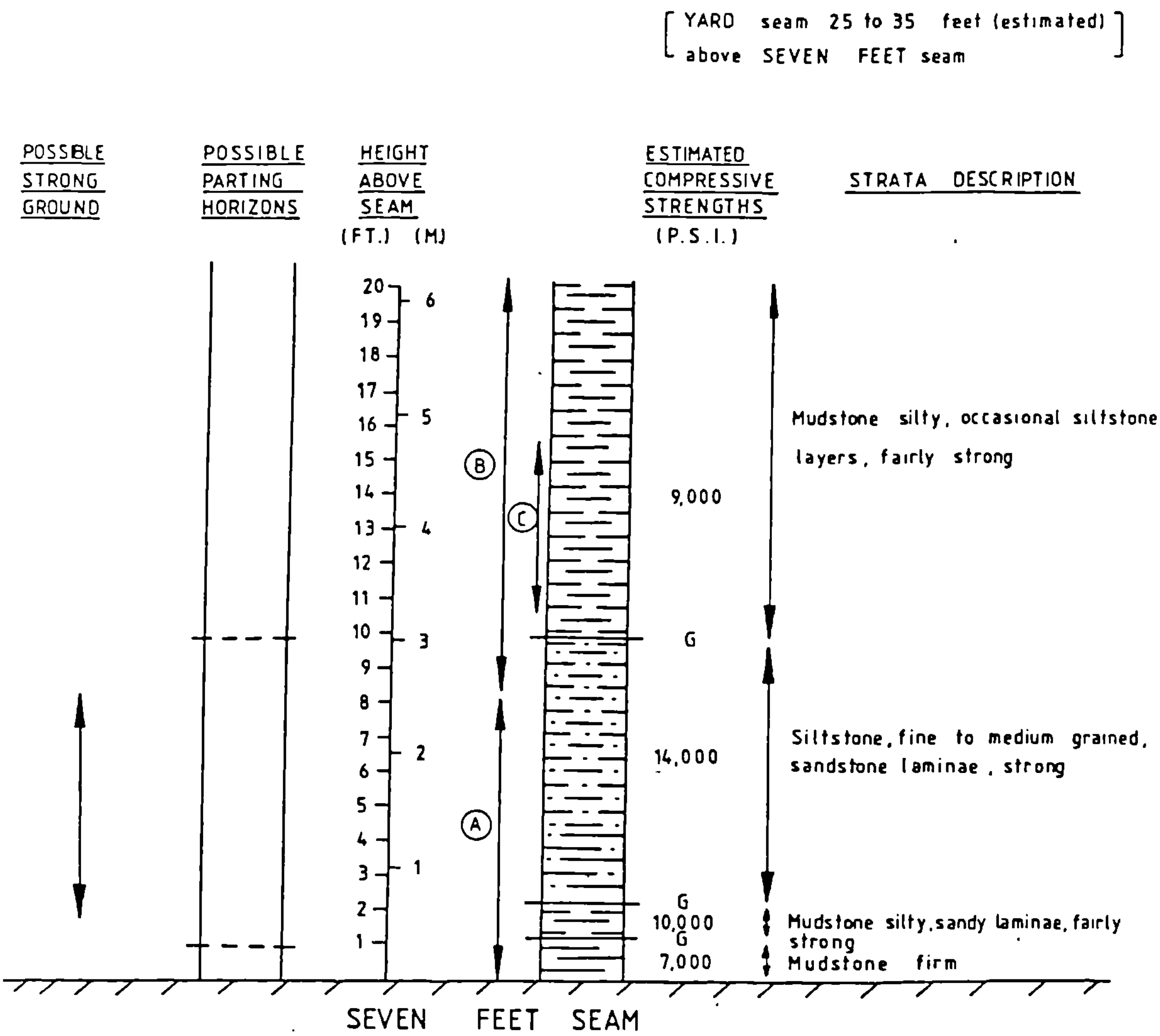
Two scale model tests were carried out simulating possible conditions in the main gate serving M25's proposed face at Penallta Colliery. The objective of these tests was to give an indication whether rock bolts and straps could provide a satisfactory means of roof support. M25's panel will be in the Seven Feet Seam at a depth of approximately 770 m below the surface.

Geotechnical information provided by the HQTD Rock Testing Service and Geological Services, South Wales Area (Figure 8.21) was used to derive the required model strata strengths (Appendix 1g). A structural geological survey was carried out underground to measure the orientation and spacing of all the joints present in the roof (Jeffery 1986). Exposures through the roof strata were limited, but a consistent pattern



Bolt length	2.20 m
Borehole length	2.10 m
Borehole diameter	24 to 28 mm
Number of bolts in first row	13
Number of bolts in second row	12
Distance between bolt rows	0.80 m
Distance to roadheader after cutting	3.0 m
After bolt setting	0.8 m
Bolts per metre road	15.6 bolts/m
Lagging	
Mesh size	40 mm
Wire thickness	3.1 mm
Roll width	1.25 or 2.0 m
Overlapping	200 to 300 mm

Figure 8.20 A typical rock bolted roadway at Niederberg Colliery (after Boldt & Fritz 1980).



(A) — Observed in M 24 development roads

(B) — Observed in core from Penallta underground borehole No 100(300m N E. of M 25 face start)

(C) — Core not recovered in borehole, but thought to be silty mudstone

G — Gradation between rock types

Figure 8.21a Section of roof strata in vicinity of M25's Panel, Penallta Colliery.

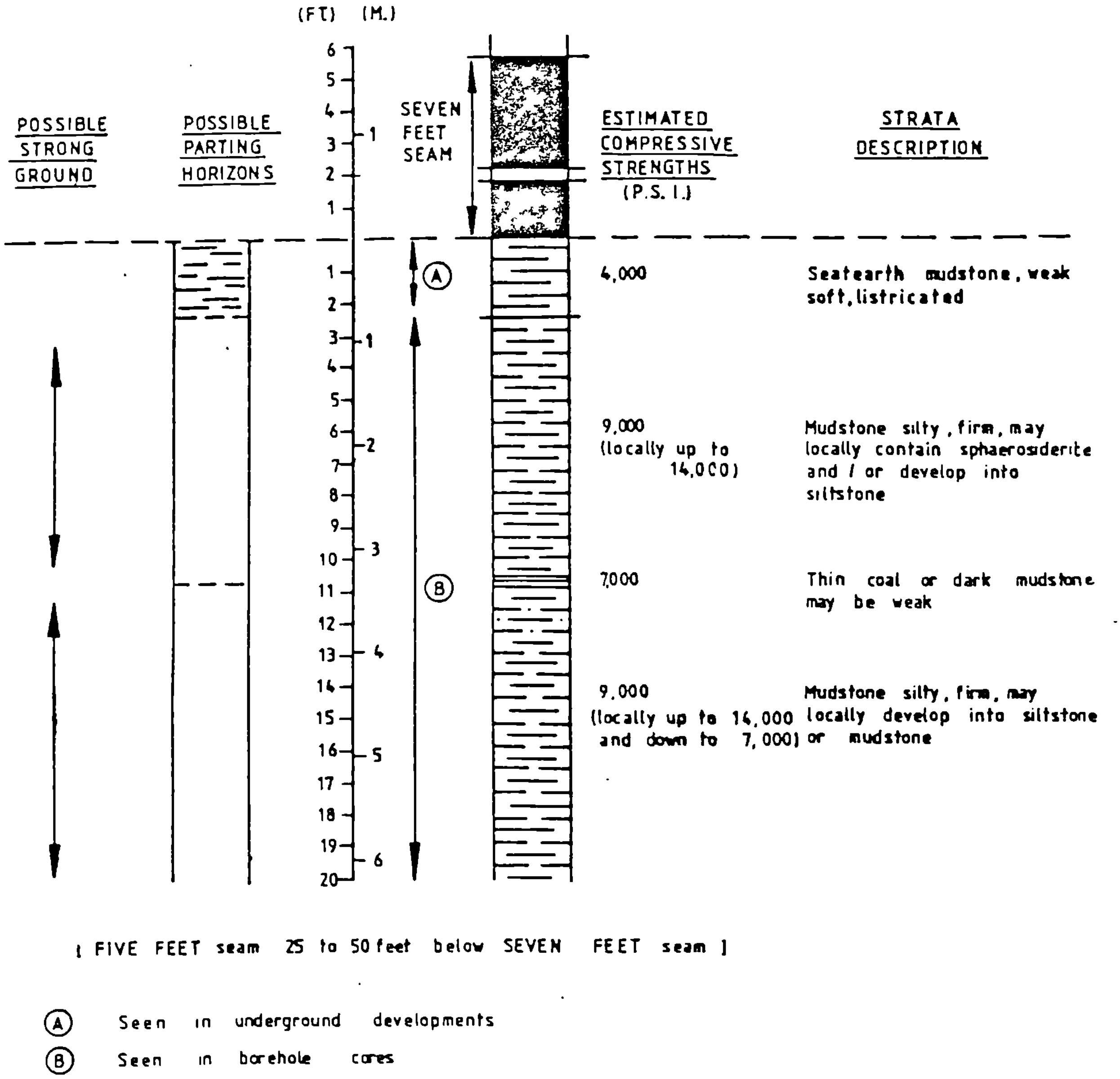


Figure 8.21b Section of floor strata in vicinity of M25's Panel, Penallta Colliery.

of jointing was measured. Two dominant joint sets were recorded, together with a minor less developed set. The directions and dips of the joint systems are given below and plotted in Figure 8.22.

Set 1	Strike 345°	Dip 80°E	Spacing 200 mm to > 1 m
Set 2	Strike 315°	Dip 85-90°E	Spacing 200 mm to 3 m plus
Set 3	Strike 40°	Dip 85-90°E	Random (minor set)

In general the joints were present in the siltstone and did not pass through to the mudstone. A structural analysis of the joint system indicates that as the sets are sub-parallel, any planes of intersection will produce a high angle wedge that would slide rather than fall directly out under gravity. As the spacing of the joint sets are variable and do not fully intersect the immediate roof, a structural induced failure is considered unlikely, and would not warrant the use of specific bolt orientations. No attempt was made to simulate the joint sets in the models.

The modelled roadways were rectangular in section and represented excavations 4.9 and 4.4 m wide. Both were the equivalent of 2.6 m high. Roof bolts and straps alone were used as the means of support. Rows of six bolts with an equivalent length of 1.83 m were installed, each row at an equivalent distance apart of 1 m. The outermost bolts were angled at 30° to the vertical over the ribsides, the two adjacent bolts were inclined towards the ribs at 15° and the remaining two bolts were installed vertically.

Measured roof lowering and floor lift (as percentages of the original roadway height) are plotted against applied pressure in Figure 8.23. Only slight bowing of the floor and roof was observed for pressures up to 75% cover load (wide roadway) and 90% cover load (narrow roadway). For pressures above these values, floor deformation became very evident with the development of bed separation and the formation of a V-type fracture pattern. At estimated equivalent cover load (0.55 MPa) the roof remained intact and the reduction in roadway height of 50% was primarily due to floor heave.

The models were tested to above the equivalent cover load to give an indication of strata behaviour when the roadway is subjected to the front of the retreating face. The roof strata remained as a competent beam throughout the tests although the floor heaved dramatically as the

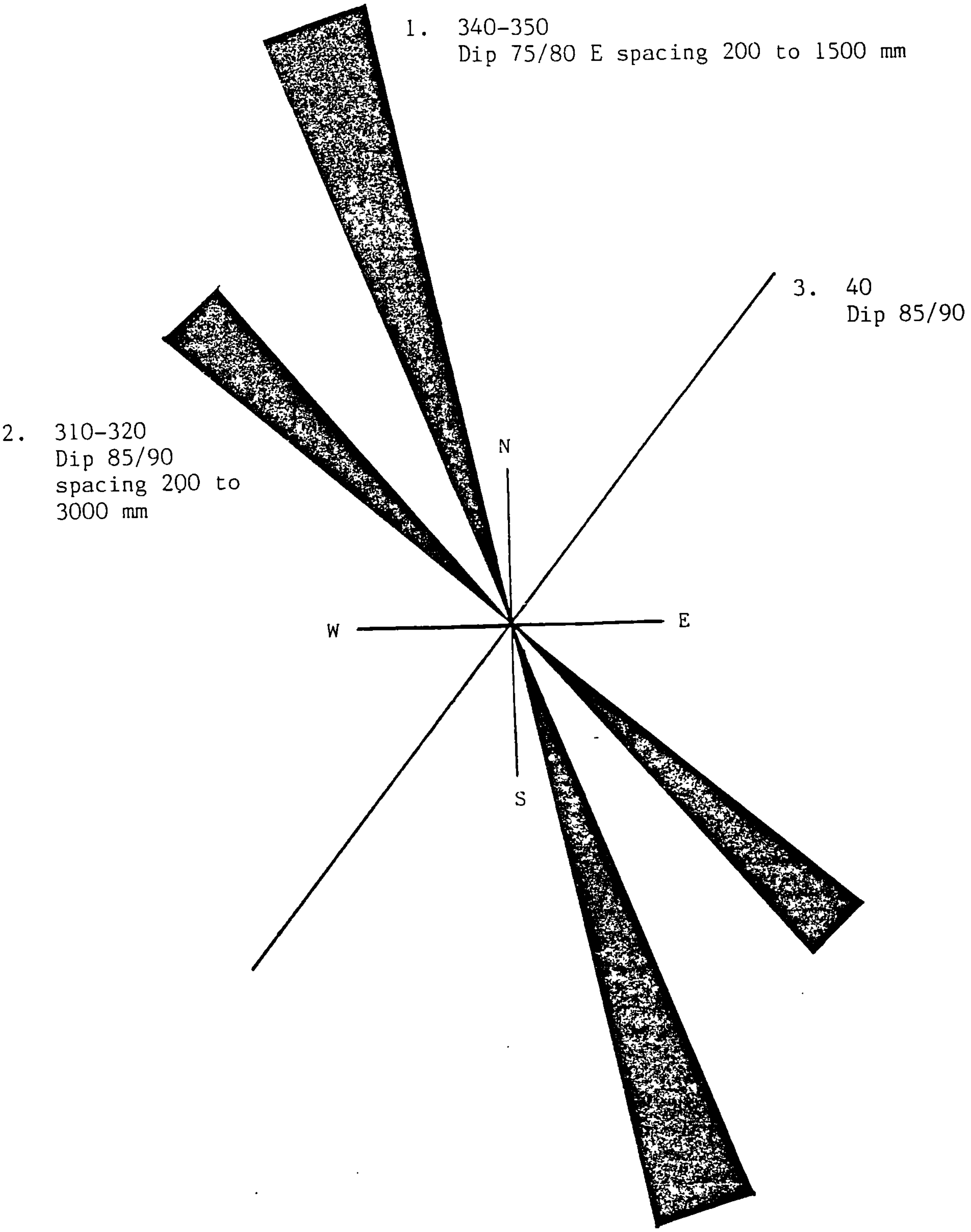


Figure 8.22 Rose diagram showing joint orientations in vicinity of M25's Panel, Penallta Colliery (after Jeffery 1986).

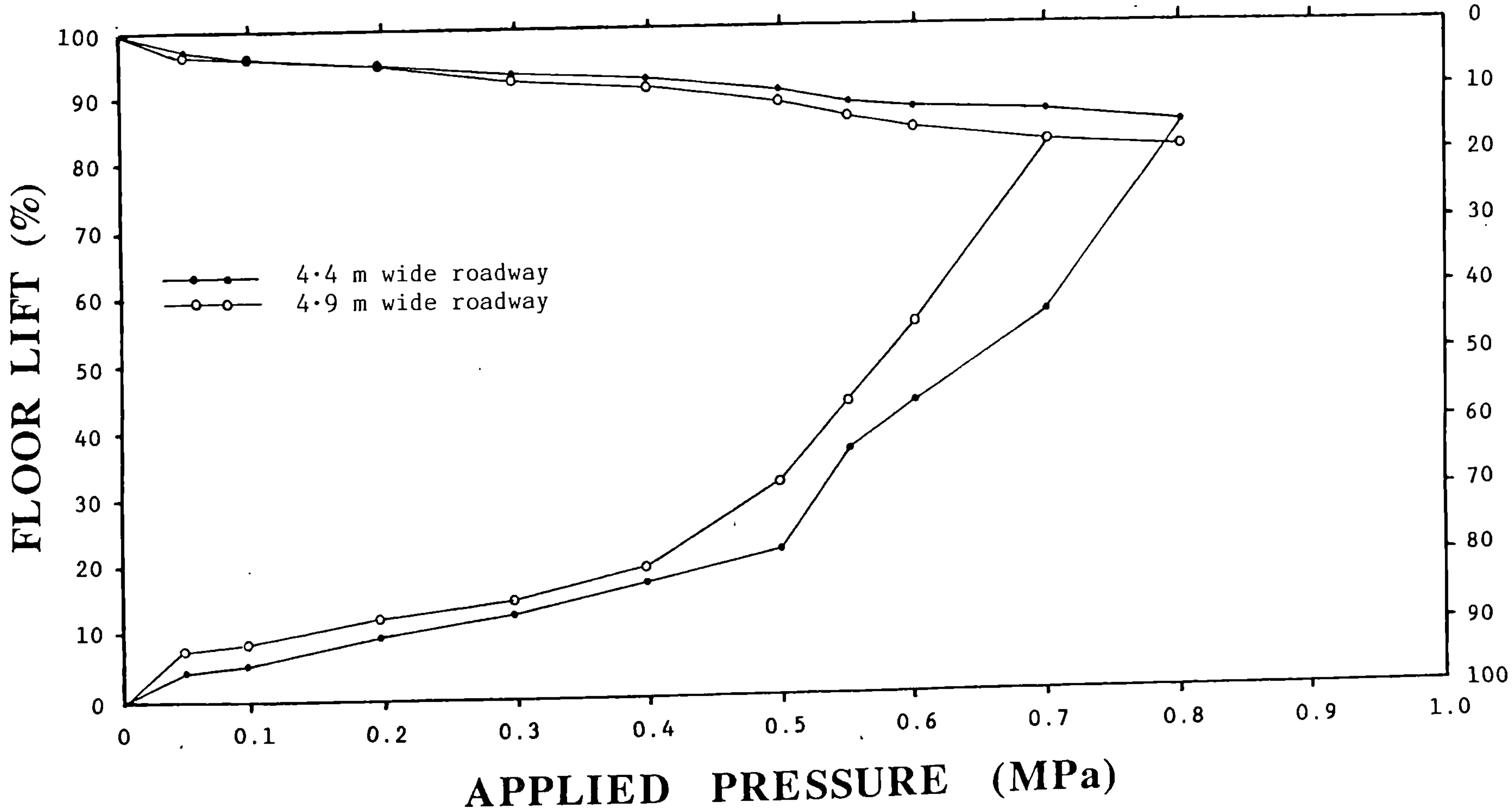


Figure 8.23 Variation in roof lowering and floor lift as a percentage of initial roadway height with applied pressure - Penallta Colliery.

pressure was increased. The narrower roadway underwent slightly less convergence. This was particularly noticable at higher applied pressures.

The model tests indicated that rock bolts and straps could provide a satisfactory means of roof support up to approximately twice cover load. This is probably less than the loading that the roadway will have to withstand when the face retreats. Floor heave could be a serious problem, especially when the roadway is under the front abutment. Reducing the roadway width may delay floor deformation although the narrow (4.4 m) roadway also suffered severe floor heave in these tests.

8.6.3 Scale Model Feasibility Study For A New Prospect

The roadway scale model technique has been used as part of a feasibility study for a large prospect of high quality coking coal (Gellideg Seam) in South Wales. An investigation was carried out to give an indication of the degree that rock bolts (rather than steel arches) could be used as the primary support in roadways within the proposed Margam Drift Mine. This was considered to be an important factor in assessing whether the prospect was economically viable.

The strata in the immediate vicinity of the Gellideg Seam varies within the area of the Margam Prospect. Generally the strata becomes less argillaceous and consequently more competent, from south-west to north-east. The prospect can be broadly divided into two areas with regard to the nature of the roof strata (Figure 8.24). These are as follows:-

(a) *South-West: Roof Dominated By Mudstone*

The immediate roof is formed by mudstone at least 1 m thick and typically 3 m thick. It is silty in places and always described as shaley or with some polished partings. The mudstone is overlain by sandstones and siltstones.

(b) *North-East: Roof Dominated By Sandstone And Siltstone*

The seam is normally overlain by a thin mudstone (up to 0.6 m) that is mostly silty; above this is a thick sequence of sandstones and siltstones. Often the mudstone is absent and sandstone then forms the immediate roof.

Strata sections given in Figure 8.25a show the Gellideg Seam and its immediate roof recorded from boreholes within the prospect area.

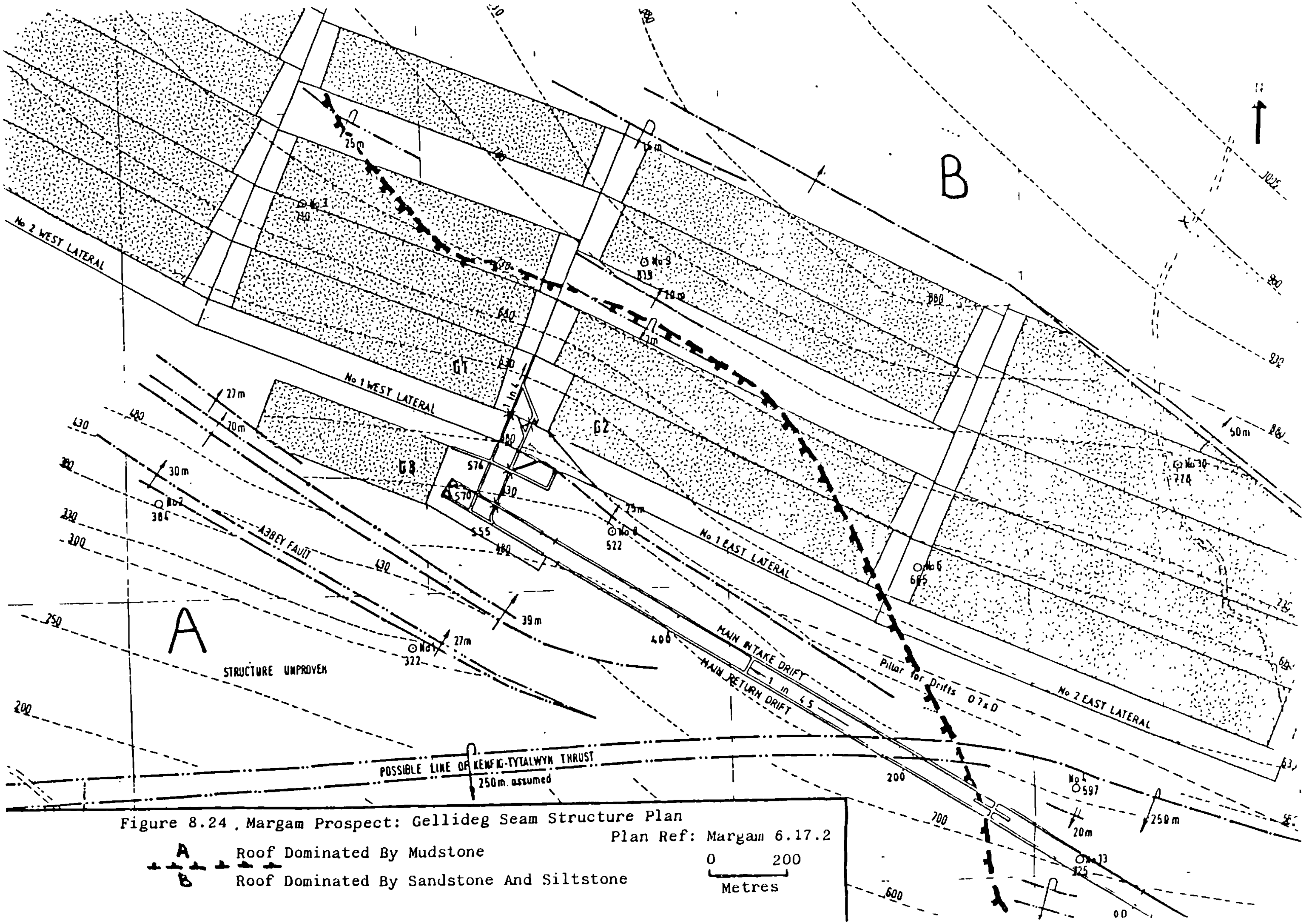


Figure 8.24 . Margam Prospect: Gellideg Seam Structure Plan

Plan Ref: Margam 6.17.2

A Roof Dominated By Mudstone

B Roof Dominated By Sandstone And Siltstone

0 200 Metres

Geological information from Margam Borehole No.8 (Figure 8.25b) and data gathered during inspection of opencast workings of the seam in the Margam vicinity were used to estimate the required model strata strengths (Appendix 1h). No.8 Borehole was chosen as it is in relatively close proximity to the bottom of the drifts and the initial planned faces at the mine. It is also in the area where the roof rocks are predominantly mudstones which are less competent than the roof core recovered from boreholes further to the north-east.

The modelled roadway was quadrangular in section and parallel to the strike of the seam which dips at approximately 14^{o}. The equivalent dimensions of this roadway are shown in Figure 8.26. In this model, roof bolts alone were used as the means of support. Rows of six bolts were installed, each row at an equivalent distance apart of 1 m.

The nearest of the proposed faces to Borehole No.8 is G2's, at a depth of approximately 850 m. As the applied pressures were increased up to the hypothetical equivalent cover load, relatively minor convergence occurred due to slight bowing of the roof and floor. At cover load (Figure 8.27) both the roof and floor model strata were in very good condition; no serious fracturing or bed separation had taken place. Ribside spalling (which was unconstrained) was observed at relatively low applied pressures.

As the applied pressures were increased to 0.64 MPa, corresponding to 22.4 MPa underground (106% equivalent cover load), considerable deformation of the floor strata was observed. Bed separation occurred, quickly followed by the formation of a V-type fracture pattern in the floor. At applied pressures of 0.76 MPa, corresponding to 26.6 MPa underground (126% equivalent cover load), failure of the roof beam occurred and an inverted V-type fracture pattern developed above the roadway. Simultaneously further floor lift took place, resulting in total closure of the roadway. The test was concluded at 133% equivalent cover load.

Post test analysis revealed that the V-type fracture patterns above and below the roadway extended to an approximate underground equivalent distance of 3.8 m in the roof and 4.1 m in the floor. The vertical 1.8 m roof bolts failed along the equivalent grout/rock interface. All the 2.4 m roof bolts that were angled over the ribsides showed no signs of failure.

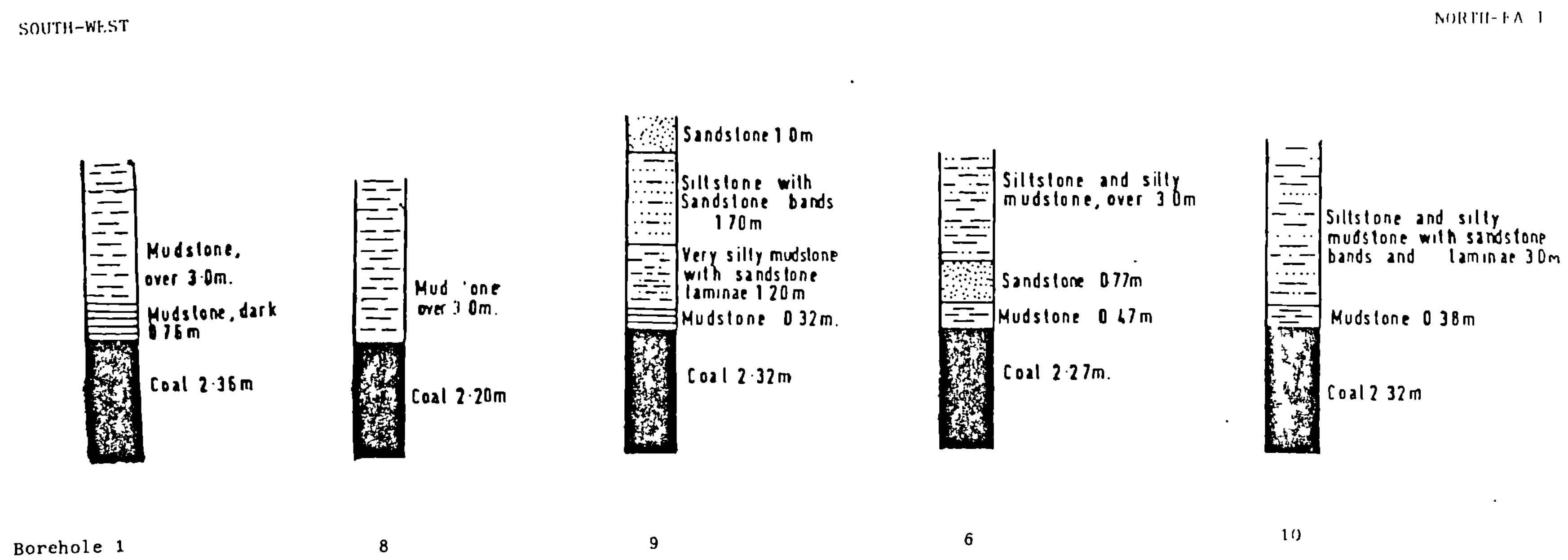


Figure 8.25a Roof variations within the Margam Prospect (Gellideg Seam).

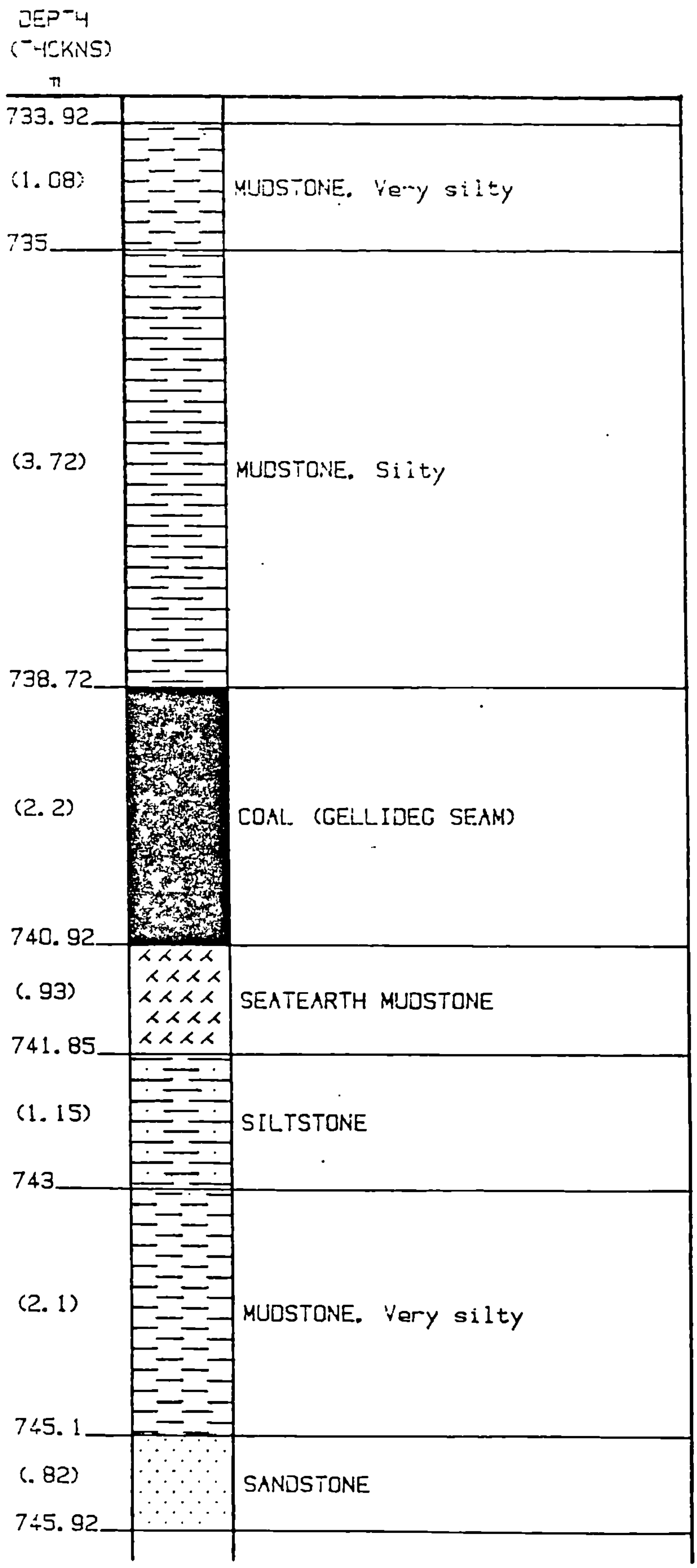


Figure 8.25b Section of strata recorded in No.8 Borehole, Margam Prospect.

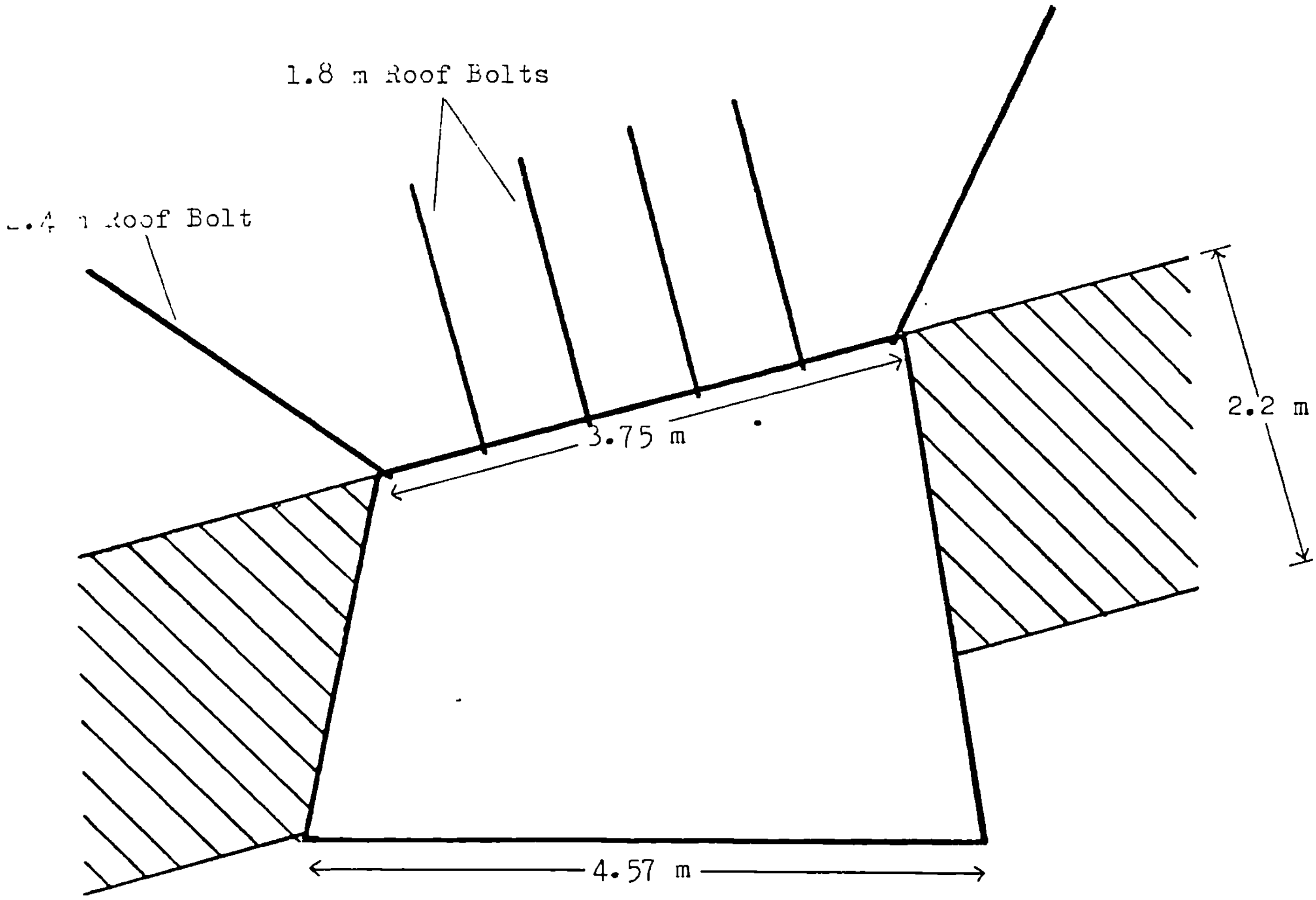


Figure 8.26 Model roadway equivalent dimensions and roof bolt pattern, Margam Prospect.

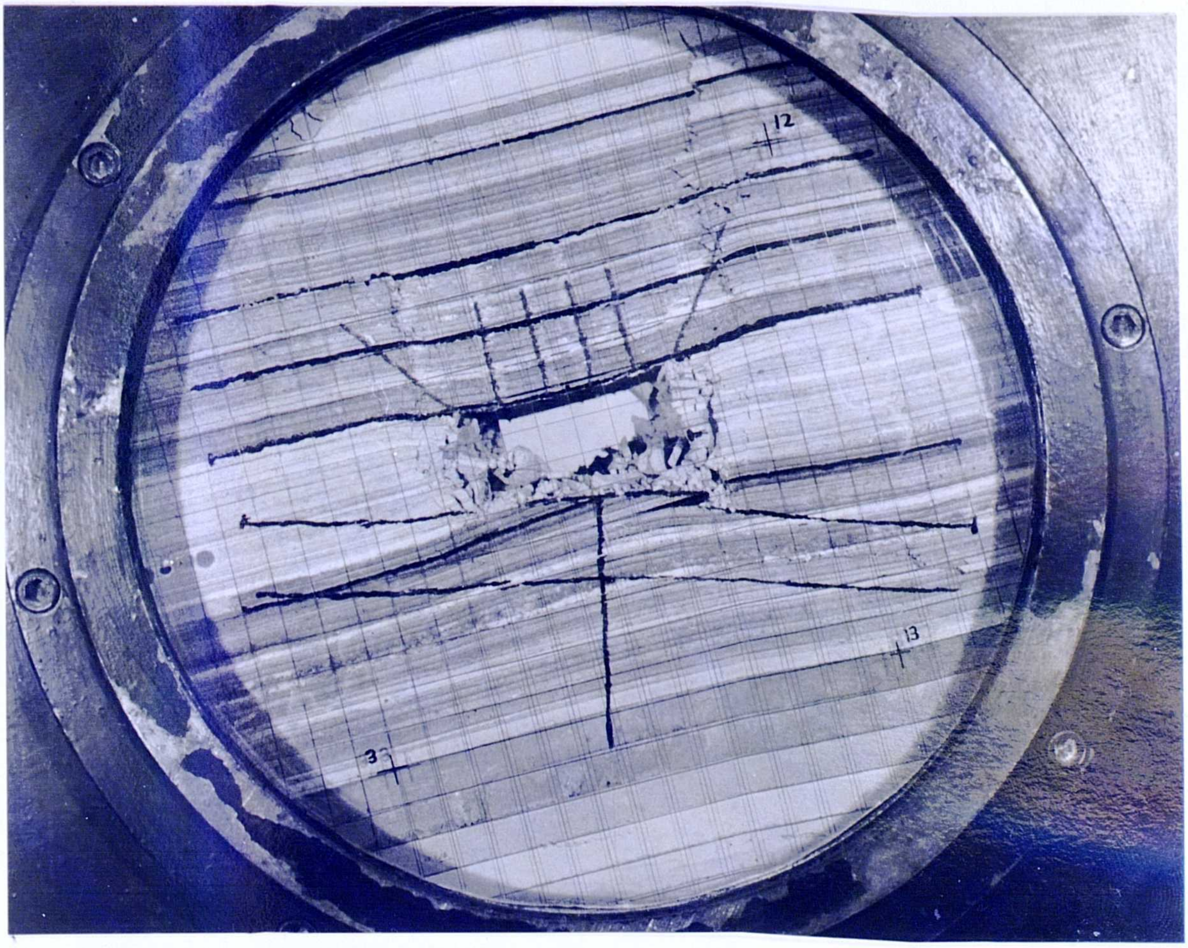


Figure 8.27 Margam model roadway - cover load

The roadway modelled represented an assumed typical situation, from which it appears that, up to estimated cover load, roof bolts could provide a satisfactory means of support. However, above cover load (i.e. when subjected to mining induced stress) roadway closure could proceed rapidly and additional support measures would be necessary. It is considered that a similar failure mechanism might be initiated by local geological weakness at cover load.

Care must be taken in the interpretation of scale model behaviour in such a situation. A model represents a particular section of roadway in a specific area of the prospect. A short series of tests have a very limited application in the prediction of the behaviour of a large tunnel complex, especially when there is no underground closure data for comparative assessment.

8.7 Roof Bolt Support For Partial Extraction Operations

Allerton Bywater Colliery is working the Middleton Little Seam which lies at a depth of 297 m. Much of the seam has already been extracted by longwall. Mining by partial extraction is currently being undertaken in order to remove small blocks of coal left behind by longwall and to be able to extract coal from beneath areas susceptible to subsidence.

56B was a block of coal approximately 100 m by 150 m. The area was worked by finger pillar extraction with 10 m wide pillars. The extraction height was 2.53 m and the 5.3 m wide entries were supported by square work and rock bolts. The square work consisted of a 152 x 127 mm beam with 114 x 127 mm legs at 1.2 m centres. The rock bolts were 25 mm diameter, 1.83 m long full column anchored pre-tensioned bolts at 1.2 m centres radially and 0.6 m longitudinally arranged in a W pattern with an additional angled side wall bolt.

The top 260 mm of coal is normally left up to form a roof. The immediate strata above the seam is a silty mudstone with abundant plant remains providing parting planes which tend to fall away as slabs if left unprotected. The silty mudstone (0.4 to 1.0 m thick) is overlain by approximately 15 m of stronger silty mudstone (uniaxial compressive strength $\approx$ 36 MPa). Beneath the seam is a thin band of seatearth mudstone underlain by a strong silty mudstone. The cleat is parallel to the entries and contributes to a large amount of side spalling. This was observed within a metre of the heading and causes the support legs to be pushed inwards further outbye. The intake roadway for 56's runs at right angles to the finger panel entries and does not suffer the same degree of spalling.

Data gathered from a thorough instrumentation and monitoring programme (Sykes 1987; Altounyan 1987; Bloor 1987) established that:

(a) Bed separation in the first 3 m of the roof is negligible (< 2 mm).

(b) The loads on the supports and roof bolts are low.

(c) Total roof lowering is about 150 mm. This combined with the extensometer results indicates that the roof is behaving as a competent slab but is lowering en masse. This lowering is caused by pillar yielding allowing the entire roof to move down.

Steel supports were salvaged from the entries, observations from the outbye end showed that the roof remained intact. A special exemption from Mines Inspectorate permitted the steel support spacing to be increased to 1.5 m intervals and the bolting pattern was also spaced out to 1.5 m, still using six bolts per steel support installed. Extraction of the adjacent 56A block is currently underway. The experience gained in 56B's may enable permission to be granted for the development of a trial heading supported purely rock bolts and straps.

CHAPTER 9

FULL COLUMN ROOF BOLTING WITH INORGANIC GROUTS

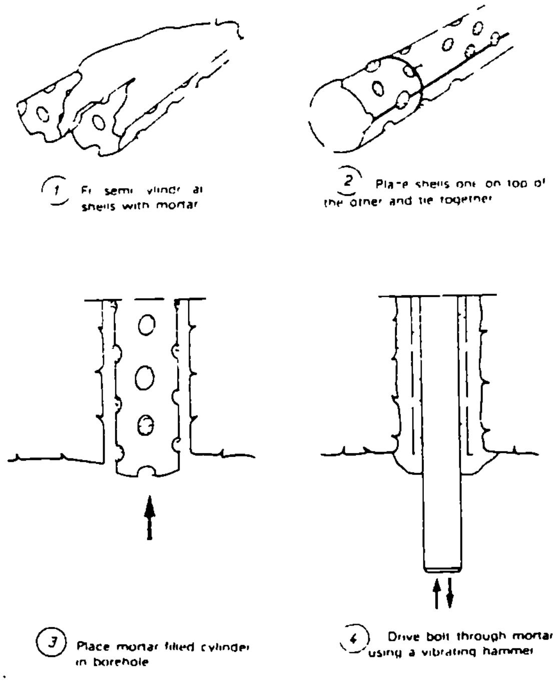
9.1 Cement Grouts

Cement grouting of rock bolts pre-dates resin grouting. Longer setting times limit its use in underground mining today, although in some circumstances it can provide a viable low cost alternative to resin.

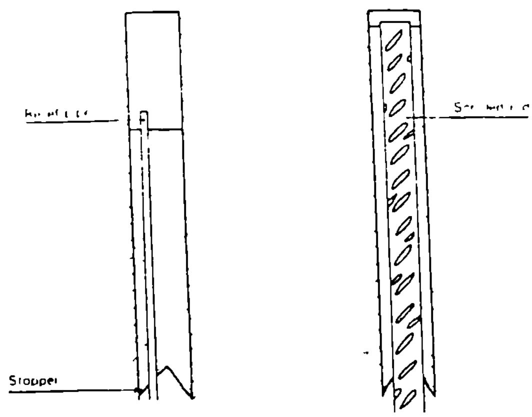
A variety of cement-mortar capsules are available. The most popular types consist of powdered Ordinary Portland Cement and additives encapsulated in a perforated skin. After soaking in water for a few minutes they are put into the bolt hole. The bolt is then inserted through the capsule, spinning is generally not necessary. Successful applications of these capsules have been reported by Lorentzen and Moore (1984), Jones (1986) and Lee et al (1987). Other types of capsule include those where cement and water are contained in separate compartments. There is a critical time limit on the soaking and installation of the above products which is a constraint to their practical applications.

The Perfo technique utilizes a perforated, cylindrical steel tube or sleeve (either as two half-sleeves or a one piece sleeve with a slit), which is filled with mortar and inserted into the bolt hole (Raju et al 1972; Precht 1979). A rock bolt is then driven into this tube forcing the mortar to fill the annulus between the bolt and the rock (Figure 9.1a). Although perfo-bolts are relatively expensive, they are both simple and effective so long as the recommended sizes are strictly adhered to (Hoek and Brown 1982).

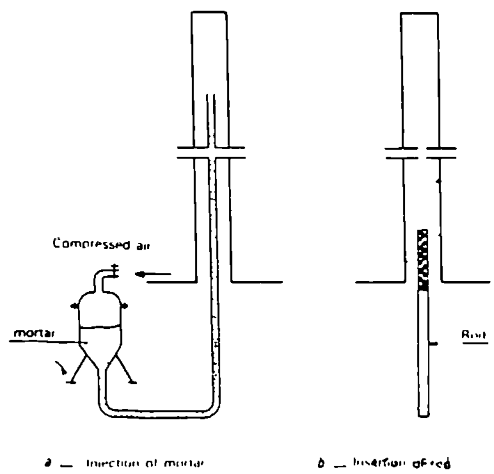
Two methods of cement grout injection have been described by Sinou and Dejean (1980). The Injecto technique simply involves partially filling the bolt hole with cement mortar and sealing the hole with a special stopper (a serrated steel plate). The air is expelled through a relief-pipe. When the hole contains sufficent grout then a bolt is inserted (Figure 9.1b). The Berg-Jet technique utilizes a vessel with a conical base connected to a flexible tube. The vessel is hermetically sealed and compressed air (at pressures of 200 to 400 kN/m^2) is used to force the mortar through the tube which is inserted to the back of the bolt hole. The tube is gradually withdrawn as the hole fills and then a bolt is driven into place (Figure 9.1c).



a. Perfo technique



b. Injecto technique



c. Berg Jet method

Figure 9.1 Cement grout insertion techniques (after Sinou & Dejean 1980).

9.2 Gypsum Grouts

9.2.1 Background

Between 1973 and 1975 the price of petroleum derived resins doubled in the USA. This prompted the initiation of a USBM (United States Bureau of Mines) research programme to find a low cost substitute grout for rock bolts which had similar rapid hardening properties. Numerous commercially available gypsum, portland and calcium aluminate cements with quick setting capabilities were tested. The most suitable of these was an alpha-type gypsum plaster (Wang et al 1976). It is composed of calcium sulphate hemihydrate which hydrates to become gypsum in the presence of water.

Hydrated plasters can take from a few minutes to up to an hour to solidify. The set time is dependent on several factors, including fineness, degree of mixing, and water cement ratio. The demands of roof bolting require rapid solidification, this was achieved by the addition of 1% (by weight of dry plaster) of potassium suphate accelerator. With accelerated hemihydrate, solidification begins within 15 seconds after hydration, the grout is completely solid within 1 minute and significant strength is obtained within 3 minutes.

9.2.2 Packaged Water Plaster Capsules

Two methods of storing packaged water within an accelerated hemihydrate capsule have been developed by the USBM (Simpson et al 1980; Fraley 1984). The water is either contained in a tube along the length of the capsule or in several packages positioned adjacent to hemi-hydrate packages inside the capsule (Figure 9.2).

These techniques proved unsatisfactory because good mixing could not be obtained. The physical separation of the two components by a membrane meant that a considerable amount of bolt rotation was required to achieve even distribution of the water. Bolt insertion was often incomplete due to the presence of sections of dry impenetrable hemi-hydrate within the borehole.

9.2.3 Dry Hemihydrate With Microencapsulated Water

Simpson (1978) developed a capsule combining accelerated hemihydrate with microcapsules of water. The microcapsules were already commercially available and consisted of water that was encapsulated in a thin shell of modified paraffin wax. Typically the spherical microcapsules are 1.8 mm in diameter and have a water content of 64% by weight.

Between 40,000 and 50,000 water microcapsules are uniformly distributed per 300 mm length of plaster capsule. When mixed this produces a gypsum slurry with a water-plaster ratio of 0.325.

Very little bolt rotation is required during installation. The bolt is rapidly inserted into the capsule within the borehole, causing a pressure build up (measured as high as 10 MPa, Serbousek and Bolstad 1981) which is sufficient to rupture the microcapsules. The wetting of the powder and production of a slurry occurs approximately 50 mm ahead of the bolt as it enters the hole (Figure 9.3).

In 1981 the capsules were commercially manufactured in the UK by Commercial Plastics Ltd who named the product Cemicron 2000. The company marketed two types of capsule with different setting speeds. The setting times varied according to thrust of insertion, type of strata, size of annulus and temperature. Table 9.1 gives times quoted by the manufacturer at 20°C.

Capsule Type	Grip Time	Set Time
A	10 Sec	5 Min
B	30 Sec	8 Min

Grip Time: time required for a 1.8 m steel rebar to be held without support in a vertical roof bolting application.
Set Time: time required for the anchorage to have developed strength of adhesion up to 90% of ultimate load bearing capacity.

Table 9.1 Hardening times for microencapsulated water hemihydrate capsules

Type A was manufactured primarily as a grout for steel rebar in roof bolting applications; whereas Type B was suitable for reinforcement of roadway ribsides or longwall faces and used with wooden dowels.

Initial field trials with these capsules in the USA (Simpson et al 1980) and in the UK (Silvester 1982) established that adequate strength could be achieved in a time similar to that for resin bolts. The manufacturer quotes anchorage strengths determined from pull tests of 6 to 8 tonnes per 300 mm with 25 mm rebar (Commercial Plastics 1981). It was later found that the capsule contents tended to suffer from volume reduction in the hole during hydration and there was a potential instability of

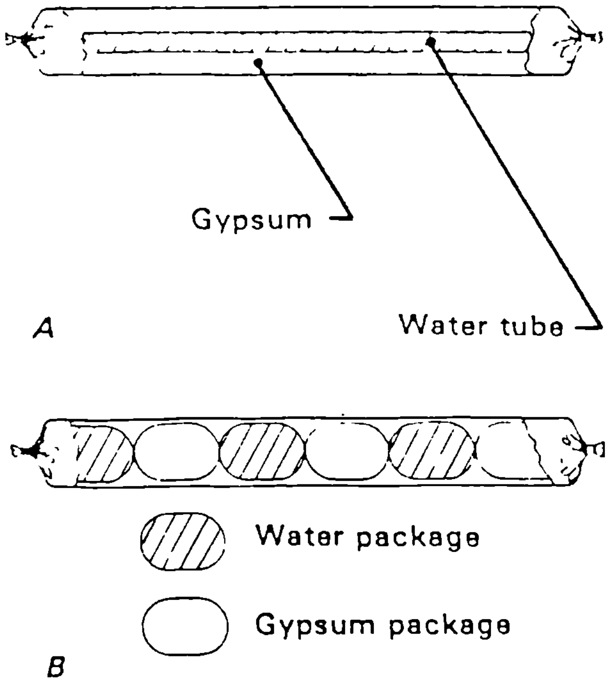


Figure 9.2 Water tube capsules (after Fraley 1984).

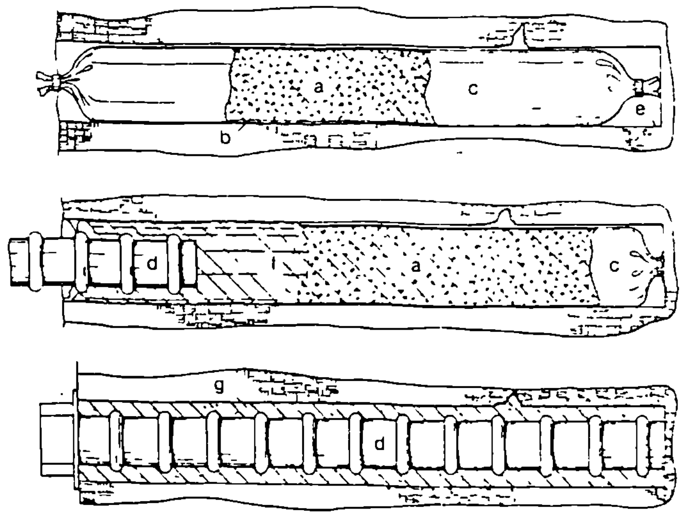


Figure 9.3 Gypsum grout with microencapsulated water
a. Cement
b. Microcapsules
c. Casing
d. Bolt
e. Borehole
f. Flowable paste
g. Solid grout

the water encapsulating wax (hardening and becoming brittle). Restrictions on its use were therefore identified under certain annular gap conditions and on the time/transport/storage cycle.

Ground Control (Sudbury) Ltd are still marketing this capsule in Canada. To the author's knowledge it is not currently being used in any coal mine roadway support system.

9.2.4 Inhibited Gypsum Slurry

A gypsum grout capsule known as Strataset E has been independently developed by Nobel's Explosives Co. Ltd and was introduced into the UK market in 1986. It is a single skin capsule containing a gypsum slurry with an inhibitor to prevent setting. Along side the slurry is a strip of emulsion consisting of droplets of copper sulphate solution ($CuSO_4$ aq) surrounded by oil. There is no physical boundary between the two components. During installation, spinning the bolt through the capsule breaks the surface tension of the oil and and allows the copper sulphate to mix with the slurry. The copper sulphate solution counter acts the inhibitor, permitting the gypsum to set.

The grout from this capsule is slower to harden than the micro-encapsulated water type. It has a grip time of between $1\,^1/_2$ and 2 minutes and a set time of approximately 30 minutes. However, the manufacturer quotes a higher anchorage strength of 10 tonnes per 300 mm (Nobel's Explosives 1986).

Early trials of this capsule with wooden dowels for longwall face reinforcement have found it to be satisfactory for this purpose. A substantial trial as a grout for steel rebar applications in roof bolting has yet to be initiated. The shelf life of this capsule is at least nine months. The plaster eventually drys out from the end clips, consequently efforts are now being made to develop tighter clips (Kennedy-Skipton 1987).

9.2.5 Pellet Injection Device

One of the first attempts at the development of a mechanized plaster grout injection system by the USBM was a device to inject either dry hemihydrate or pelletized particles approximately 3 mm in diameter (Smith 1978).

An air stream propelled the material into the borehole via a rigid tube. A supply of water, atomised by air pressure was fed into the tube. This equipment performed unsatisfactorily and the development project was terminated. The airstream blew back out of the hole bringing the grout material with it causing dust generation problems because the hemi-hydrate was not properly mixed with water.

9.2.6 Slurry Injection Machines

The USBM achieved some success with a laboratory prototype slurry injection device using the alpha-type gypsum plaster (Simpson 1980; Simpson et al 1980). Research contracts were then awarded to Terra Tek, Inc and Foster-Miller Associates, Inc to investigate the design, construction and mine testing of a device for installing inorganically grouted roof bolts.

Twin-Screw Extruder System:

The Foster-Miller grout mix/injection system involves the automatic mixing of dry hemihydrate with water to form a slurry. This is pumped into a delivery hose and injected up the roof bolt hole, without placing a mechanical device in the hole (Ounanian and Cardenas 1986).

The system is based on a twin-screw extruder (Figure 9.4) of the type normally used for processing plastics. The geometry of the screws makes them self-cleaning. Dry hemihydrate, stored in the hopper, is metered through the knife valve into the screw housings. Water is introduced at a point along the screws 70 mm from the centre hopper. The extruder mixes and pumps the grout into a 6 m delivery hose attached to a transfer device. When the grout is in the hose, the transfer device moves into the injection position.

A plastic "rabbit" or plug which forms the interface between the grout and high pressure air, is positioned at the start of the hose. The high pressure air then drives the rabbit and grout through the hose and nozzel into the roof bolt hole.

The device was developed through laboratory testing during which injections were made into simutated bolt holes drilled in concrete. The underground testing and evaluation programme (mounted on a Galis 300 roof bolter) demonstrated the ability of the system to install competent

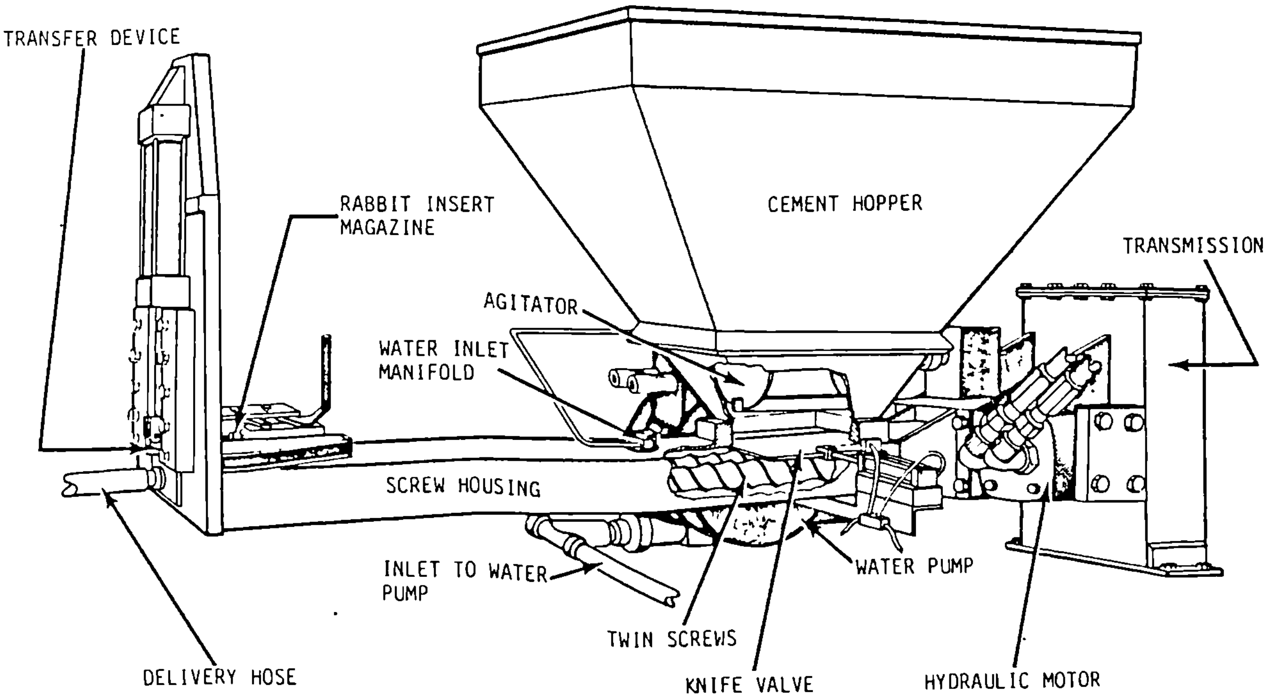


Figure 9.4 Slurry injection: Twin-screw extruder system (after Ounanian & Cardenas 1986).

gypsum grouted bolts in coal mine roof strata. The machine is currently in storage at the USBM Spokane Research Center where further development may take place in the future.

In-The-Hole Mix System

Terra Tek have produced a prototype injection sytem (Figure 9.5) which consists of a standard Fletcher DM-13 single-head bolter, equipped with a grouting system, automatic indexing head and pneumatic control computer (Mahyera and Jones 1985; 1986).

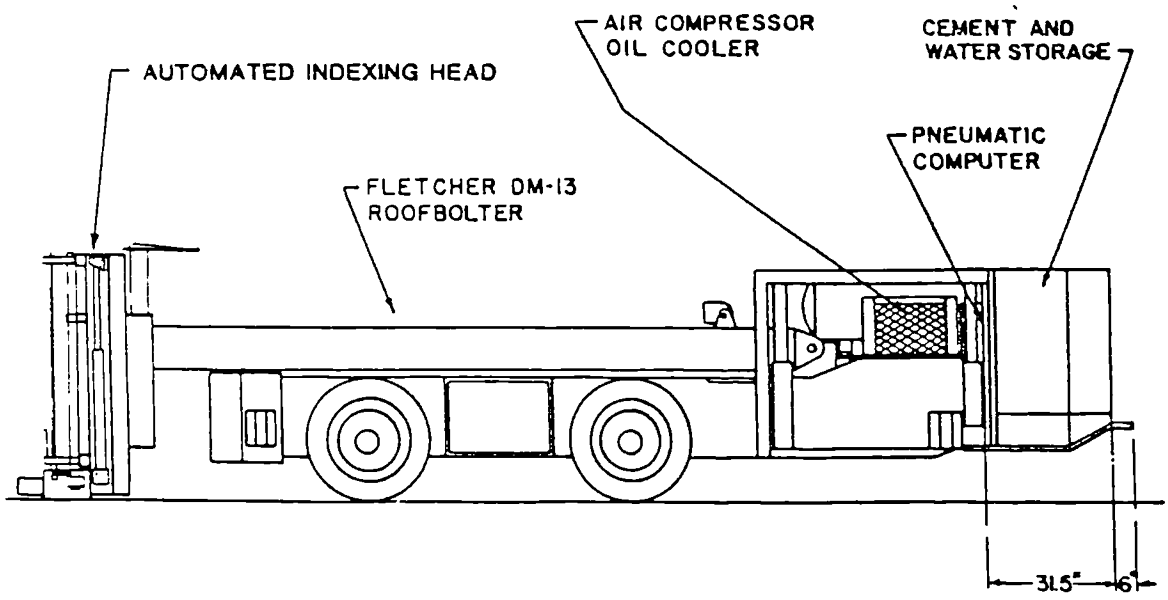
A metering screw allows a measured amount of hemihydrate to be transported via a flexible hose to the mixing/injection module. Special admixtures (including an accelerator) are pre-mixed with water. A flow control valve gives the desired water/cement ratio after mixing in the module (0.22 to 0.28 by weight). The module is inserted into the bolt hole, where the grout is ejected and as the module withdraws the hole is filled. A bolt is then inserted into the hole by the automatic indexing head. There is no pre-mixing, therefore cleaning is generally not required. Should the injector become blocked it can be regarded as a "throw away device" (Mahyera 1987).

Underground testing of this prototype in a Utah coal mine has shown that it has potential. The device is currently in storage on Terra Tek premises as the company is seeking further financial backing to develop the system.

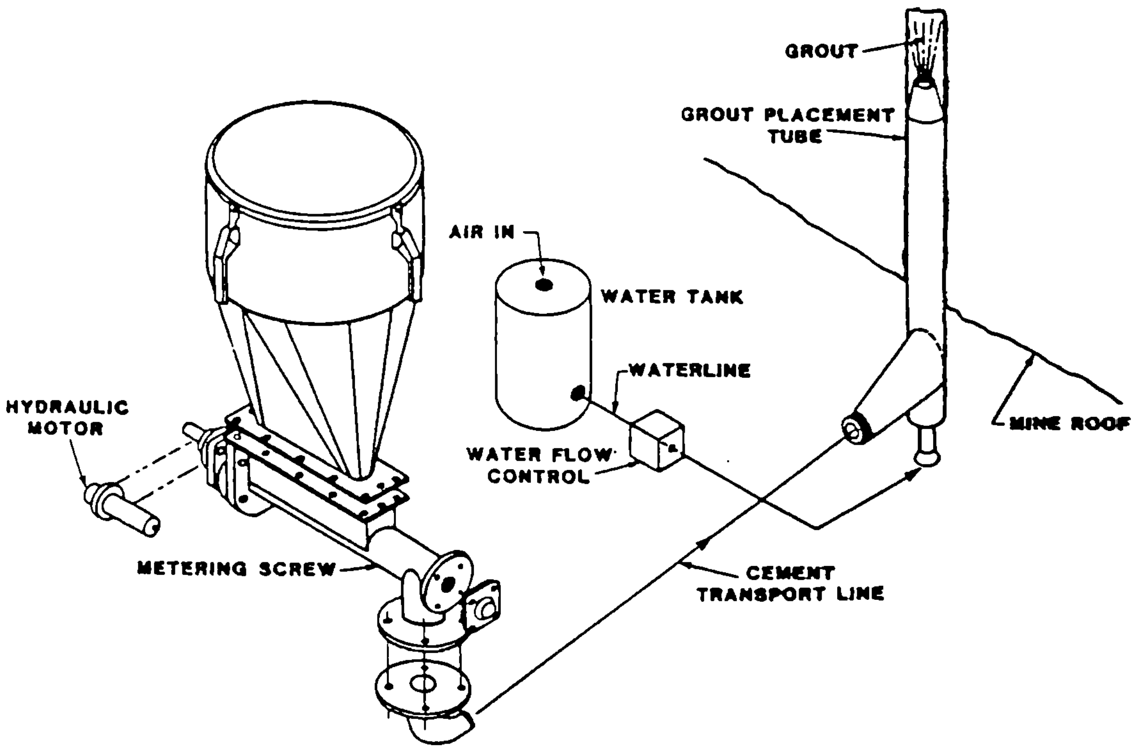
9.2.7 Performance Of Gypsum Anchored Rock Bolts

Gypsum grouts have an advantages over other grout systems as they are nonflammable, nontoxic and nonallergenic. Tests reported by Mahyera and Jones (1985; 1986) comparing gypsum grout (in-the-hole mix injected) with polyester resin found gypsum to have higher strength, stiffness and yielding characteristics. Pull tests performed by Hansen and Gerdemann (1985) and Fraley and Serbousek (1987) have shown that gypsum bonded (microencapsulated water type) bolts can provide adequate anchorage under dry conditions.

Uniaxial compressive strength tests with hand mixed samples of polyester resin and gypsum (Strata Set E) capsules, carried out by the author, gave strengths of 22 MPa and 11 MPa respectively. However, Nobel's



a. View of prototype roof bolter



b. Schematic of the grouting system

Figure 9.5 Slurry injection: In-the-hole mix system
(after Mahyera & Jones 1986).

Explosives Co. have stated that Strata Set E depends on the shearing action of a rapidly rotating bolt to break up the emulsion and that hand mixing can not give a fair indication of grout performance.

Further compressive strength tests concluded that over-mixing gypsum capsules (1 minute mixing) and mixing at high temperatures (36^{0}) will not reduce its strength. Mixing the grout with 10 % additional water, as may occur in wet holes, can halve the compressive strength.

The dissolution of gypsum bonded bolts has been studied by Gerdemann and Hansen (1983). In a coal mine environment, dissolution will occur in static solutions to the solubility of gypsum (approximately 2 g/L) and then cease, resulting in minimal damage. In flows of unsaturated solution as slow as 1 L per month, a loss of up to 10% will occur in two years.

Pugh et al (1987) report that the Australian grout manufacturing company, Cemfix Pty Ltd, consider that rock bolt systems bonded by inorganic capsules may fail after 18 months as the bolt diameter may reduce slightly as it stretches under load and the grout will not expand into the resultant gap, consequently anchorage is lost. Conversely, Mahyera (1987) has the opinion that when kept moist, the crystalline nature of gypsum grout will allow the healing of cracks to occur. Furthermore, Mahyera reports that 300 mm lengths of gypsum grouted bolts pull tested one year after installation (and kept moist), showed an increase in strength compared to similar bolts tested after a few days.

Clearly, further testing of gypsum bonded rock bolt systems is necessary to determine and fully evaluate their properties and performance characteristics.

CHAPTER 10

ROCK BOLT REINFORCEMENT OF ROADWAY RIBSIDES

Spalling of coal ribs can cause serious or fatal injuries and will result in an increase in roadway width which may affect roof or floor stability. Rib instability can also cause congestion and other problems at longwall face entries. The failure of coal ribs is dependent primarily on the orientation and intensity of natural and mining induced fractures but is also influenced by such factors as stress levels, roadway dimensions and shape, drivage direction and rate, coal strength and the presence of dirt bands.

Mapping of ribside fractures will determine the optimum direction of drivage to minimise failure. However, due to the range of cleat/mining induced fracture interactions it is rare that directional mining will completly eliminate rib spall (Figure 10.1). Failure modes shown in Figure 10.1 can be prevented by the instigation of various remedial measures outlined below.

(a) Slab/plate failure. Spot bolting with simple point anchored or fully grouted bolts can reduce toppling and sliding of coal slabs and plates.

(b) Block/column failure. Systematic bolting with some form of strapping to join the bolts can reduce buckling, toppling or sliding of blocks and columns.

(c) Particle failure. Liners such as meshing prevent spalling of small fragments. The liner type is chosen with respect to the size of particles in the ribside. Liners are normally located with simple bolts or dowels.

Non-metallic bolts are preferable, providing they can give adequate constraint, as they can be cut by longwall shearers and roadway heading machines. Dowels manufactured from hardwood such as keruing are widely used in the UK for reinforcement of cutting horizons. Keruing has a straight grain and no knot characteristics. Standard dowels are 36 mm in diameter and either 1.82 m or 2.43 m long, with ends cut at an angle to facilitate penetration of grout capsules. These dowels are commonly installed in 43 mm diameter holes.

Fibreglass bolts provide a high strength alternative to wood. These bolts can be manufactured with a threaded end to enable the fixing of a plate and nut which improves the performance of the bolt and can be used to retain mesh to the ribsides.(N.B. These bolts also eliminate the corrosion drawbacks of the metallic bolt and the low unit weight simplifies handling and transport.) Fibreglass bolts fitted with an expansion anchor unit are also available (Figure 10.2).

Independent laboratory tests by O'Beirne et al (1987) determined maximum values of tensile strength for wooden and fibreglass dowels, available in Australia, of 70 kN and 80 kN respectively. Field testing revealed that fully grouted wooden ribside bolts would accept higher loads when installed in wet drill holes than those installed in dry drill holes. It was considered that this was due to the water removing the layer of fine coal dust on the borehole wall.

Field trials in Australasia by O'Beirne et al (1984) and Shepherd et al (1984) deduced that there were no measurable or discernable differences in rib behaviour using point anchored dowels with a strong face plate over fully grouted dowels.

Weld mesh sheets are a commonly used liner; however, this can be troublesome to mine out. Extruded polyethylene nets have proved very effective for containing fragments on rock slopes (Tully 1984). The successful use of these nets for roadway ribside stabilisation has been reported by O'Beirne et al (1986). They are extremely flexible, light and easily erected. Thorough testing of these nets would have to be carried out to establish their flamability and the nature of any toxic gases produced during combustion before they could be accepted for use underground in British coal mines.

Sprayed concrete can be applied as a roadway liner although this is likely to be too expensive for routine use in gateroads.

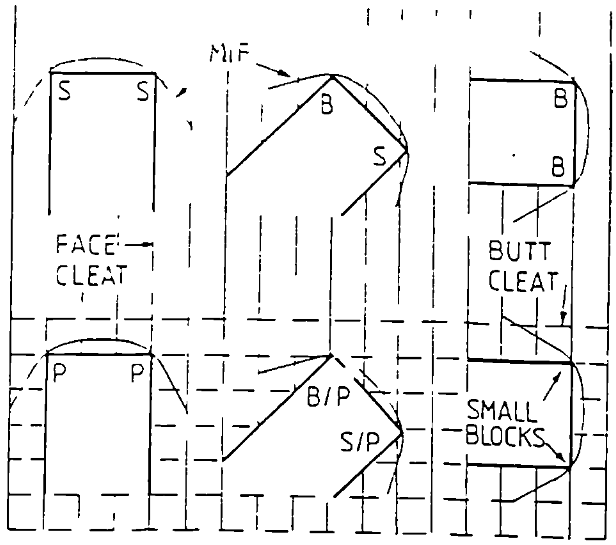


Figure 10.1 Probable mode of rib spall according to drivage direction, cleat and mining induced fractures (MIF) shown schematically in plans (after O'Beirne et al 1986).
S = slabs/plates
B = blocks/columns
P = particles

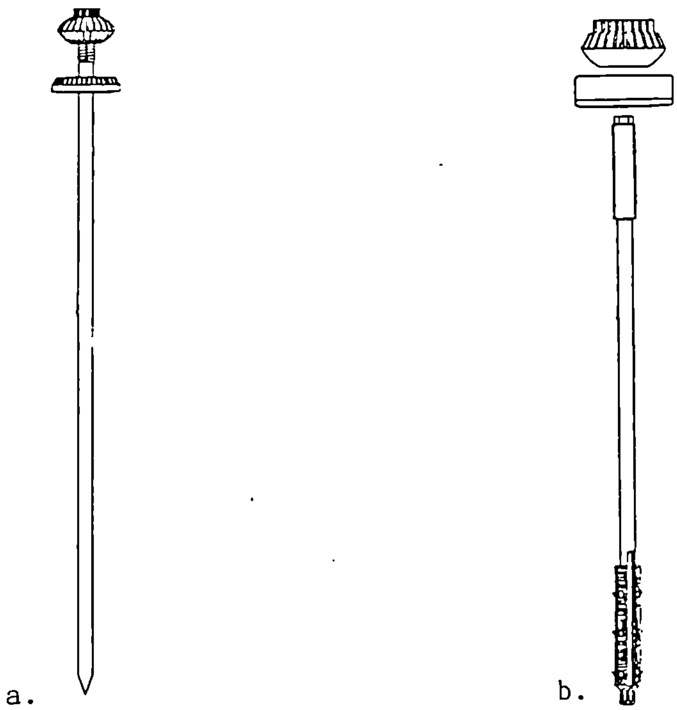


Figure 10.2 Fibreglass rock bolts (after Weinmann 1986).
a. Fibreglass rock bolt
b. Hollow core fibreglass rock bolt with expansion shell.

CHAPTER 11

ROCK BOLT REINFORCEMENT OF GATEROAD FLOORS

11.1 Background

Over the last 30 years numerous attempts have been made to reduce or eliminate floor heave in British collieries by the use of rock bolts installed in the floor. These trials, generally undertaken in gateroads of advancing longwall faces have had varying degrees of success. The initial floor reinforcement investigations utilised mechanical point anchored bolts (Table 11.1). Trials using bolts fully grouted with free flowing resin were carried out in the mid 1960s (Table 11.2). This grouting method was superseded by the development of resin capsule systems (Table 11.3).

11.2 Mechanisms Of Floor Bolt Reinforcement Systems

Floor heave will occur when the lateral thrust on the floor strata exceeds the buckling resistance of the beds; or when the floor is unable to resist the pseudoplastic flow of strata from beneath the pack or ribs. The tendency for pseudoplastic flow has been related to the proportion of illite (Wester 1971).

The action of untensioned bolts in a floor is best considered in two stages:

(i) Before fracture, the bolts bind together a series of laminae. The stress required to cause failure by buckling (Lawrence 1973) is:

$$\sigma f = (\pi^2/3)\ (t^2/W^2)\ E$$

where t = thickness of the bolted floof strata slab
W = width of slab
E = Young's Modulus of slab

Thus the required buckling stress is proportional to the thickness of the slab squared. However, the compressive strength of weak materials such as seatearth floors is not usually very much greater than the buckling stress, therefore fracture by compression takes place.

(ii) After failure, a weak, fractured floor will tend to act as a loose, granular material offering very little resistance to lateral pressure and flowing up into the roadway. The bolts then act by providing confining pressure to the residual broken material. The relationship of the compressive stress in a broken material to the confining pressure (Wilson 1972) is given by:

COLLIERY	DATE	SEAM	ROADWAY	DEPTH (m)	FLOOR GEOLOGY	ARCH SIZE AND SPACING (m)	BOLT LENG. AND DIAMETER (m & mm)	PATTERN AND SPACING (m)	REFERENCES
Bank Hall	C.1959	Union	M/G	-	W	- -	1.5 19	- 1.2	Hind (1960)
Birch Coppice	1966	Top Bench	21's	335	W	3.7 x 2.7	1.8	6/7	Moore (1967)
Granville	1966	Doubles	306's T/G	-	S to I	3.7 x 2.4	-	-	Bullock (1968)

Table 11.1 a Floor bolting trials with mechanical anchored bolts - limited success.

COLLIERY	DATE	SEAM	ROADWAY	DEPTH (m)	FLOOR GEOLOGY	ARCH SIZE AND SPACING (m)	BOLT LENG. AND DIAMETER (m & mm)	PATTERN AND SPACING (m)	REFERENCES
-	C.1960	Brockwell	M/G	-	S	3.7 x 2.4 -	1.37 19	3/2 0.9	Smith & Pearson (1961)
Baddersley	-	Bench	T/G	C.400	W to S	2.7 x 2.4 0.9	1.8 -	4/3 -	Krishna (1974)
Baddersley	-	Seven Feet	49's T/G	-	W to S	2.7 x 2.4 0.9	1.8 19	4/3 0.75	Krishna (1974)
Baddersley	C.1969	Bottom Bench	68's	491	W to S	3.7 x 2.7 0.6	2.0 -	4/3 0.77	Hodgkinson (1971)

Table 11.1b Floor bolting trials with mechanical anchored bolts - successful.

COLLIERY	DATE	SEAM	ROADWAY	DEPTH (m)	FLOOR GEOLOGY	ARCH SIZE AND SPACING (m)	BOLT LENG. AND DIAMETER (m & mm)	PATTERN AND SPACING (m)	REFERENCES
Granville	1966	Doubles	306's T/G	-	S to I	3.7 x 2.4 -	2.0 19	6/5 0.6	Bullock (1968)

Table 11.2a Floor bolting trial with free flowing resin grouted bolts - limited success.

COLLIERY	DATE	SEAM	ROADWAY	DEPTH (m)	FLOOR GEOLOGY	ARCH SIZE AND SPACING (m)	BOLT LENG. AND DIAMETER (m & mm)	PATTERN AND SPACING (m)	REFERENCES
Birch Coppice	1966	Top Bench	21's T/G	335	M	3.7 x 2.7 -	1.8 25	6/7 0.6	Moore (1967)
Birch Coppice	1967	Top Bench	21's T/G	c.335	M	2.7 x 2.4 0.9	1.9 25	5/4 0.6	Gray (1968)

Table 11.2b Floor bolting trials with free flowing resin grouted bolts - successful.

COLLIERY	DATE	SEAM	ROADWAY	DEPTH (m)	FLOOR GEOLOGY	ARCH SIZE AND SPACING (m)	BOLT LENG. AND DIAMETER (m & mm)	PATTERN AND SPACING (m)	REFERENCES
Granville	1966	Doubles	306's T/G	-	S to I	3.7 x 2.4 -	1.8 16	6/5 0.6	Bullock (1968)
Britannia	1972	Lower Four Feet	L3's M/G	-	S to W	- -	- 36	- 1.0	Issac & Livesey (1975)
Silverhill	1972	Low Main	-	850	W to I	-	- 36	-	Daws (1975)
Manton	1973	Parkgate	30's T/G	914	S to W	3.7 x 2.7 1.1	1.8 36	4/3 1.1	Johnson (1973) Mosley (1974)
Birch Coppice	1977	Bench	46's T/G	420	I	2.7 x 2.4 0.9	1.8 36	8/8 0.9	Mallory (1981b)
Calverton	1977	L. Bright/ Brinsley	C3's T/G	500	S	- -	- -	- -	ECSC (1980) Mosley (1986)

Table 11.3a Floor bolting trials with capsule resin anchored bolts - limited success.

COLLIERY	DATE	SEAM	ROADWAY	DEPTH (m)	FLOOR GEOLOGY	ARCH SIZE AND SPACING (m)	BOLT LENG. AND DIAMETER (m & mm)	PATTERN AND SPACING (m)	REFERENCES
Calverton	C.1970	High Main	M61's M/G	C.500	-	- -	- 36	6/5 -	Murphy et al (1972)
Sutton	1972	Low Main	45's T/G	850	S to I	- -	1.8 36	6/5 1.0	Krishna (1974) Daws (1975)
Birch Coppice	1973	Bench	28's T/G	384	I	2.7 x 2.4 0.9	1.8 36	10/9 0.9	Bains (1978) Mallory (1981a)

Table 11.3b Floor bolting trials with capsule resin anchored bolts - successful.

$$\sigma_f \simeq 4\sigma_c$$

where σ_f = compressive stress to cause continuing failure
σ_c = confining pressure (provided by bolts)
Should a bolt break or slip owing to failure around the grout, its ability to continue to provide confining pressure is obviously considerably reduced.

The most successful of the trials using mechanical point anchored bolts were generally in floors with a competent anchorage horizon. One such site in the main gate of a double unit, advancing face was investigated by Smith and Pearson (1961). Slot and wedge bolts, 1.37 m long, were installed through a sandy mudstone and thin coal band, to be anchored into a strong sandstone bed. All the bolts were tensioned with a torque wrench. Tension measurement of selected bolts was carried out using resistance strain gauge rock bolt dynamometers. As the coal face advanced there was a gradual increase in the recorded bolt tension. Then from approximately 27 m outbye of the face, bolt tension increased substantially as advance continued. In sections of the roadway without rock bolts, floor heave began to become a problem at this distance from the face; with the mudstone separating from the underlying coal band. This sudden initiation of heave was probably due to consolidation of the gateside packs and the subsequent transfer of load to the roadway floor. The floor heave was completly eliminated in the bolted part of the roadway.

Previous workers have used the British Coal HQTD model rig to study full column anchored floor reinforcement techniques (Lawrence and Silvester 1972; Armstrong 1976). This unpublished work has been reassessed in the light of further field investigations and model studies. Model simulations of arched roadways with moderately weak floor strata showed floor bolting to be effective at floor lift control up to a certain applied pressure. Then rapid failure occurred and subsequent floor heave reached or even exceeded the extent to which it would have developed without floor bolting. This phenomenon has also been observed underground in 28's tail gate at Birch Coppice Colliery (Bains 1978; Mallory 1981a). The delay in floor heave initiation produced by bolting was such that a much larger face advance was obtained in 28's (1100 m) than had been experienced on adjacent panels.

Model tests simulating relatively competent floor strata established that bolting had a far greater effect in reducing floor deformation. Floor bolted physical model tests using the Bergbau-Forschung rig reported by Jacobi (1976, 1981) indicated that floor bolting was only suitable for strata with a certain inherent strength. These model results were confirmed by subsequent field observations.

11.3 Installation Position

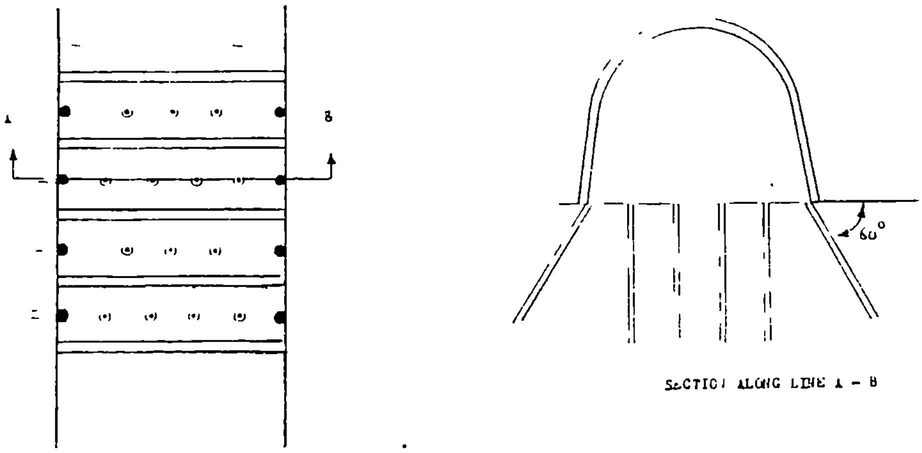
The necessity to set floor bolts as close to the face as possible has been established at a number of trial sites. In 30's tail gate at Manton Colliery it was planned to bolt through successive layers of strong siltstone, weak mudstone and strong sandstone to form a composite slab; the combined strength of which, it was hoped, would be sufficient to resist lateral buckling forces (Johnson 1973; Mosley 1974). However, owing partly to the difficulty in drilling the hard beds, unreliability of the drill rig in these conditions and the presence of a ripping platform and methane drainage rig (which had to be kept up to the rip), the bolts were installed so far behind the face that floor cracking and heave were already evident. This resulted in only a very small reduction in floor lift rates being achieved.

Smith and Pearson (1961) found that the initial tension from mechanical point anchored bolts was considerably greater for bolts installed at the face (42 kN) than those set behind the conveyor driving motor and other equipment 11 m from the face line (13 kN). It was considered that this was probably due to bed separation and strata fracturing occurring behind the face.

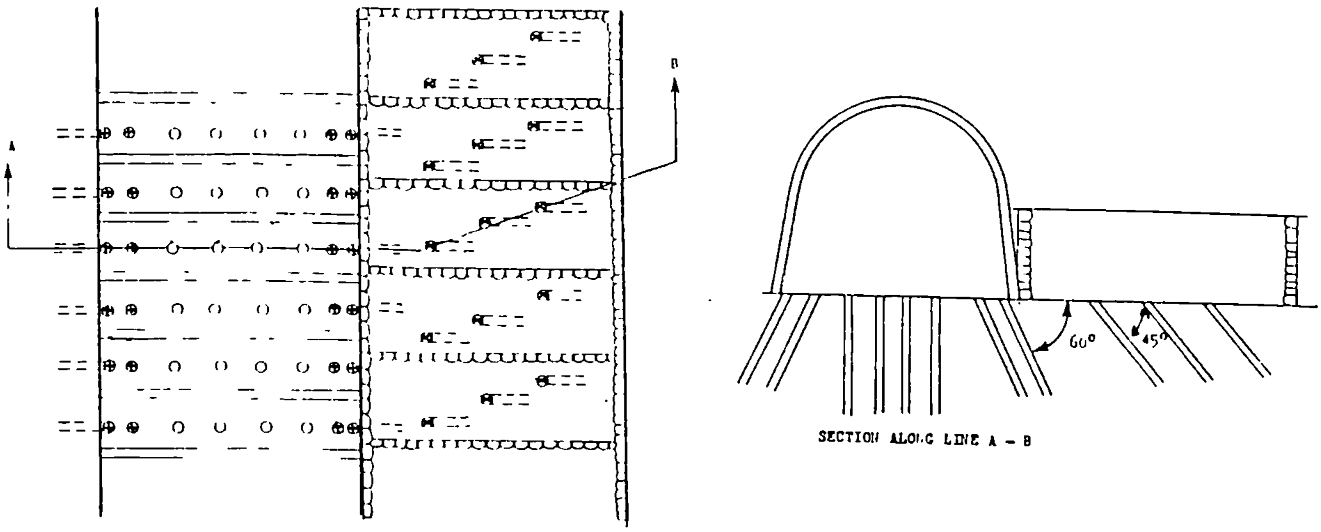
11.4 Floor Bolt Reinforcement Patterns

A variety of floor bolt patterns have been employed (Figure 11.1), with the majority of sites using 1.8 m long bolts.

During the 1970s, 36 mm diameter full column resin grouted wooden dowels became the standard reinforcement system for roadway floors. The use of wood rather than steel made dinting operations simpler in situations where floor heave became a problem. Where a greater shear strength was required wooden dowels with a fibreglass core have been used. These bolts were installed in 43 mm diameter boreholes. The bolts angled under the roadway sides were frequently composite bolts consisting of an inner steel core surrounded by a hardwood sleeve to give an overall



a. Floor bolting of 30's tail gate, Manton Colliery (after Mosley 1974).



b. Floor bolting of 28's tail gate, Birch Coppice Colliery (after Mallory 1981a).

- ● 1·83 m long steel rebar
- ⊙ 1·83 m long wood dowel with fibre glass core
- ⊕ 1·83 m long composite wood/steel bolt
- ⊛ 1·34 m long composite wood/steel bolt
- ○ 1·83 m long wood dowel

Figure 11.1 Examples of floor bolting patterns used in British coal mines.

outside diameter of 36 mm. Thus the benefits of steel bolts could be achieved, without changing the drill bit to a smaller size and without the use of excessive amounts of expensive resin grout.

Scale model studies (Armstrong 1976) and field investigations (Mallory 1981a) have shown that additional dowel reinforcement of gateside pack floors does not produce significantly better floor control and may have been responsible for increase roadway damage.

11.5 Extended Floor Bolting

Attempts have been made in France and Germany to control floor lift by creating a thick reinforced floor slab through extended strata bolting, (the use of long rock bolt reinforcement systems). Coupled steel and wooden bolts or steel cables up to 6 m long have been used. The bolts were full column anchored, using resin capsules or injection of a quick-setting cement suspension.

At Rossenray Colliery in the Rheinland, Oldengott (1979) established that the introduction of coupled bolts, 6 m long, in a relatively hard floor (a sandy mudstone with a compressive strength of 70 MPa) could reduce roadway convergence by 20%. Other successful extended floor bolt installations have been reported by Schuermann (1978) and Pelissier (1980).

Scale model tests were carried out to establish the effectiveness of extended floor bolting in comparison with a typical "British pattern" and an unreinforced roadway. The model strata configuration and the support patterns used are given in Appendices 2h, 2g and 2a respectively. The total length of reinforcement installed per equivalent metre of roadway was exactly the same for both reinforcement patterns (10.0 m/m equivalent).

Closure measurements from the tests (Figure 11.2) showed that, in this roadway situation, floor bolting using 1.8 m bolts will prevent substantial floor lift up to applied pressures of 0.5 MPa (corresponding to 17.5 MPa underground). Extended strata bolting gave a significantly greater improvement; the test indicated that this reinforcement pattern would be capable of keeping this roadway open at an equivalent depth of at least 1100 m. Photographs comparing the three support configurations are shown in Figure 11.3.

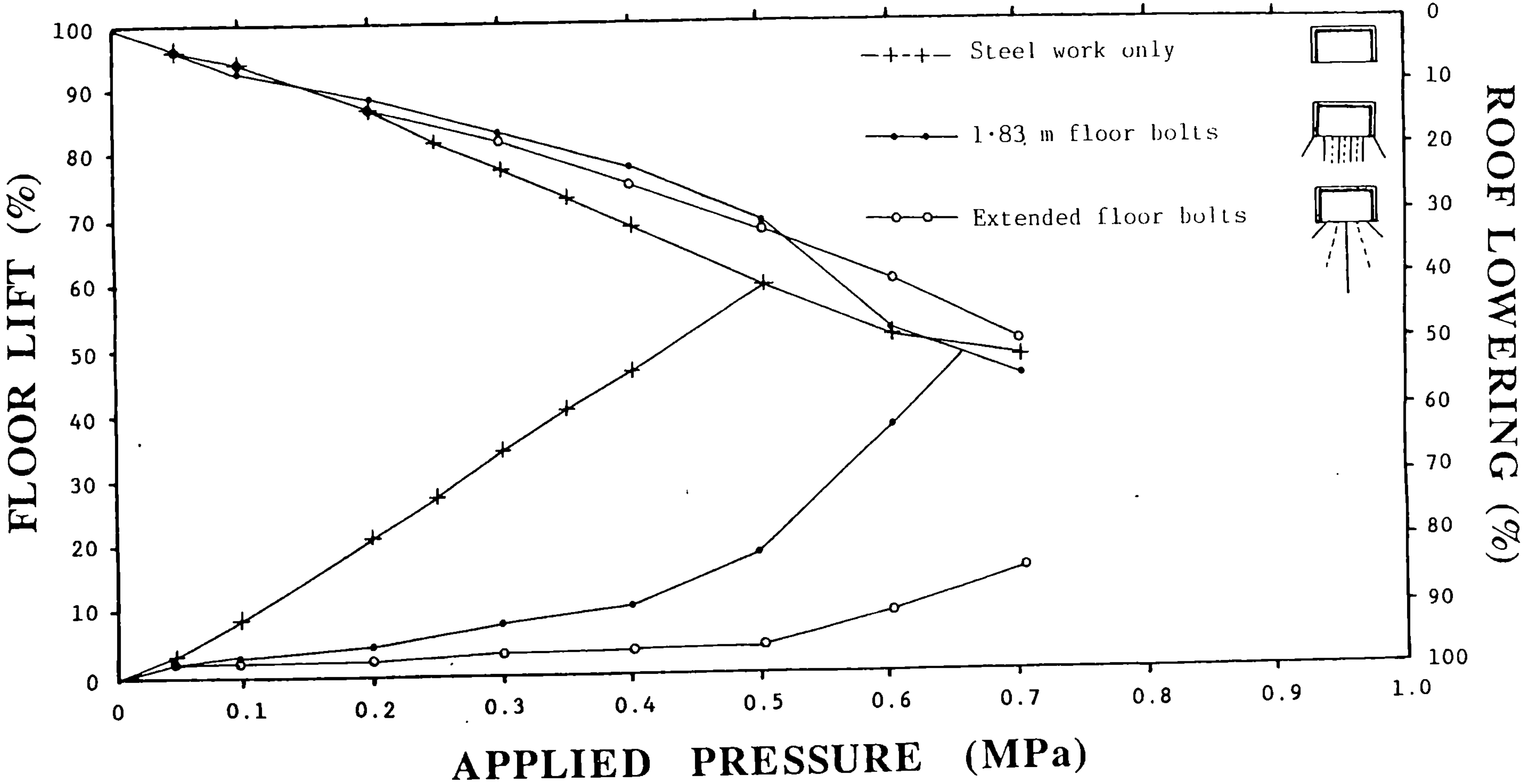


Figure 11.2 Variation in roof lowering and floor heave as a percentage of initial roadway height with applied pressure - floor bolted scale model studies.

11.6 Failure Mode Of Bolted Floors

At some sites where floor reinforcement had not been totally effective a more uniform floor heave was experienced across the width of the roadway in the bolted sections (Hind 1960; Isaacs and Livesey 1975). As a consequence of this there was a reduced effect on distortion of conveyor and haulage systems. This phenomenon has also been observed in scale model studies (Figure 11.3).

11.7 Effect Of Floor Bolting On Roof Deformation

Floor reinforcement has been known to have had both a beneficial and detrimental effect on roof deformation at different locations.

A reduction in roof lowering in a floor bolted section of a roadway has been observed by Moore (1967), Hodgkinson (1971), Oldengott (1979) and Mallory (1981b). Results from comparative physical model studies indicate that more stable roof conditions exist at applied pressures where floor reinforcement is restricting floor heave (Figure 11.2). It is also noticeable that when floor failure commences in the model reinforced with 1.8 m (equivalent length) floor bolts, there is a simultaneous increase in the rate of roof lowering. It is possible that the improved roof conditions are produced by a favourable stress redistribution resulting from floor reinforcement.

Conversely, the increase in roof lowering reported by Bullock (1968) and the greater arch distortion on the ribside observed by Isaac and Livesey (1975), in floor bolted roadway sections, may be the result of unfavourable stress redistribution.

11.8 Floor Drilling

In the UK, the majority of floor bolt holes have been drilled by rotary drills. With this technique it is essential that debris is cleared from the collar of the hole as it is produced.

Dry drilling can be hazardous where there are inflows of methane into the hole. The floor bolting trial in C3's tail gate at Calverton Colliery had to be terminated due to gas emissions from the floor (ECSC 1980). The draw off filters of drill systems using an air flushing system can be blocked by strata water penetrating into the drill hole along rock fissures.

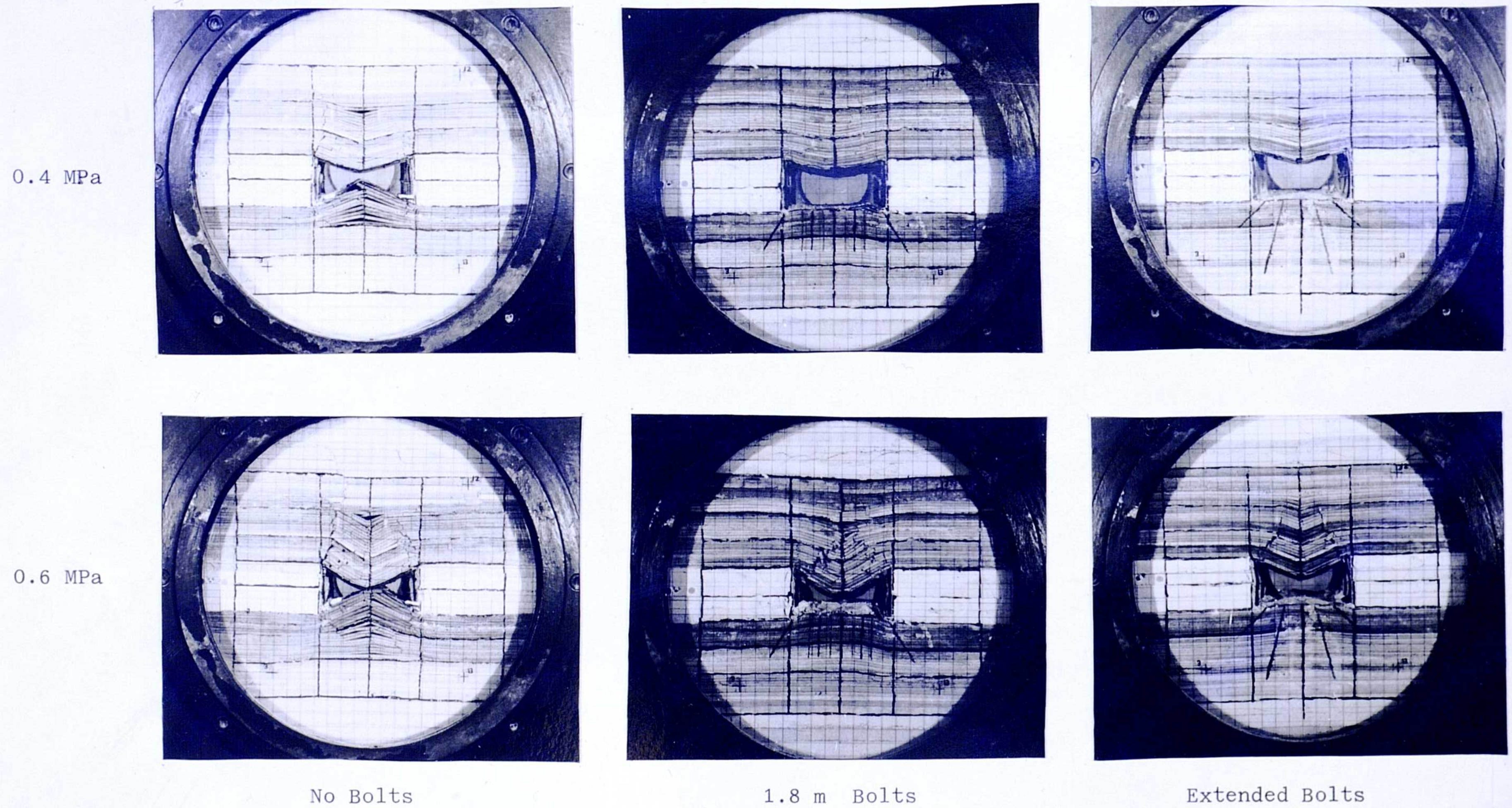
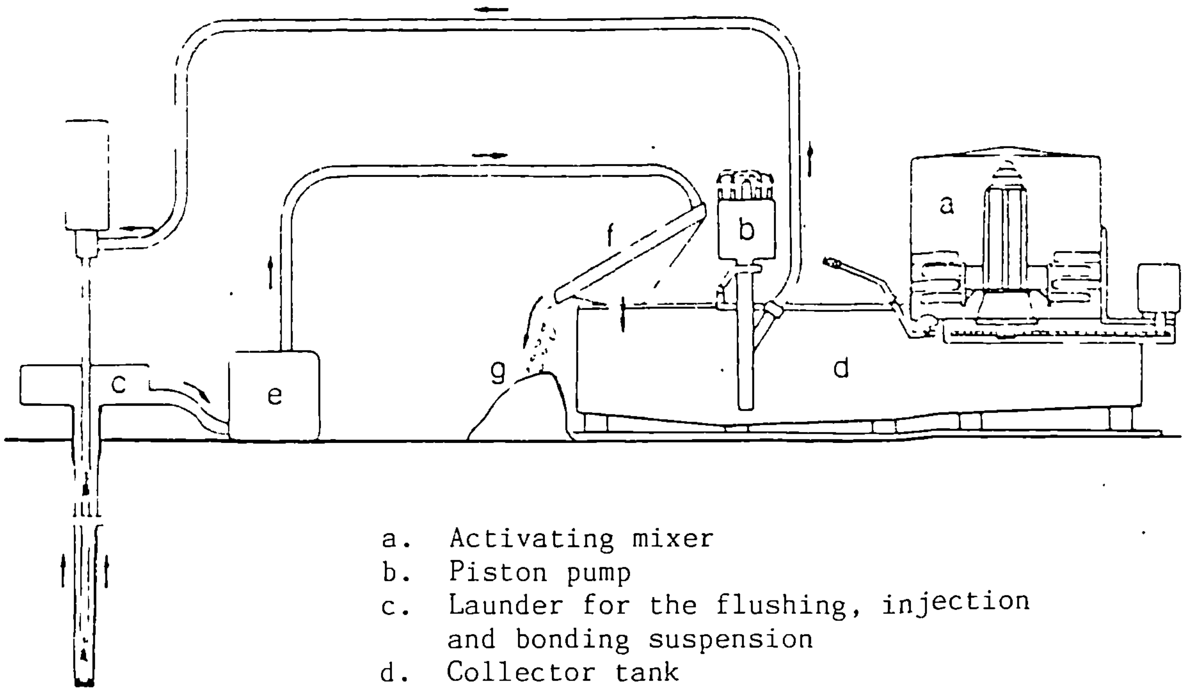


Figure 11.3 Floor bolting: scale model comparisons

Wet drilling in coal seam floors also has its problems. The presence of water will accelerate floor heave due to an effective strength reduction of the strata. The degree of strength reduction will depend on the mineralogical composition of the rock. Therefore in some circumstances floor bolting using this drilling method may reduce, rather than enhance, the cohesive strength of the strata. In addition, where the strata is fractured, flushing water may escape from the borehole along fractures, consequently the debris would not be removed and the drill steel could jam.

Gotze et al (1982) have proposed a method of drilling floor holes with a flushing suspension which flows on a closed circuit (Figure 11.4). The suspension is also intended to act as a slow setting grout for the bolts.

In order to carry out a field trial using the extended floor bolting pattern tested in the scale model rig (Figure 11.3), specialised drilling equipment would be required. A drill rig similar to that used at the Rossenray Colliery installation (Oldengott 1979) might be suitable. It consisted of two drilling carriages fitted to the upper cross beam of a drilling portal so that they could be manoeuvered to drill the hole pattern illustrated in Figure 11.5. Wet drilling was applied using cruciform flushed bits with a 42 mm diameter.



a. Activating mixer
b. Piston pump
c. Launder for the flushing, injection and bonding suspension
d. Collector tank
e. Diaphragm pump
f. Rocker screen
g. Drillings

Figure 11.4 Method for the drilling and injection of bolt holes in mine floors (after Gotze 1982).

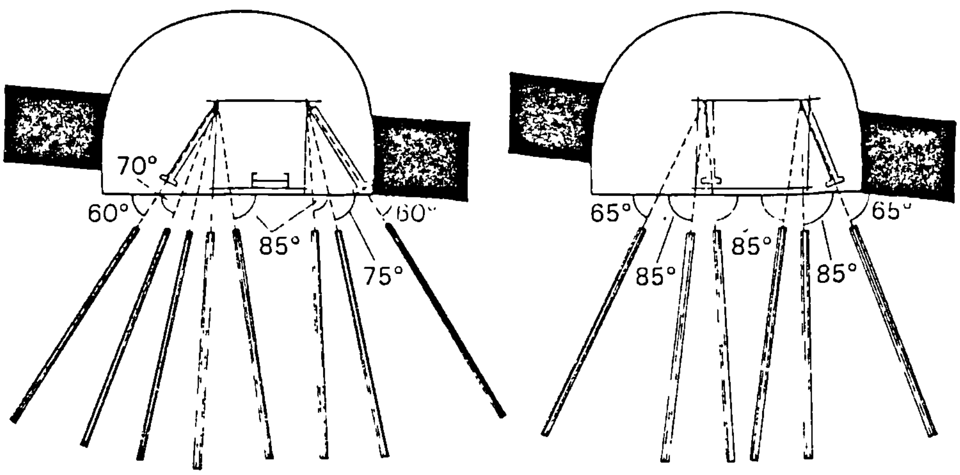


Figure 11.5 Extended floor bolting at Rossenray Colliery (after Oldengott 1979).

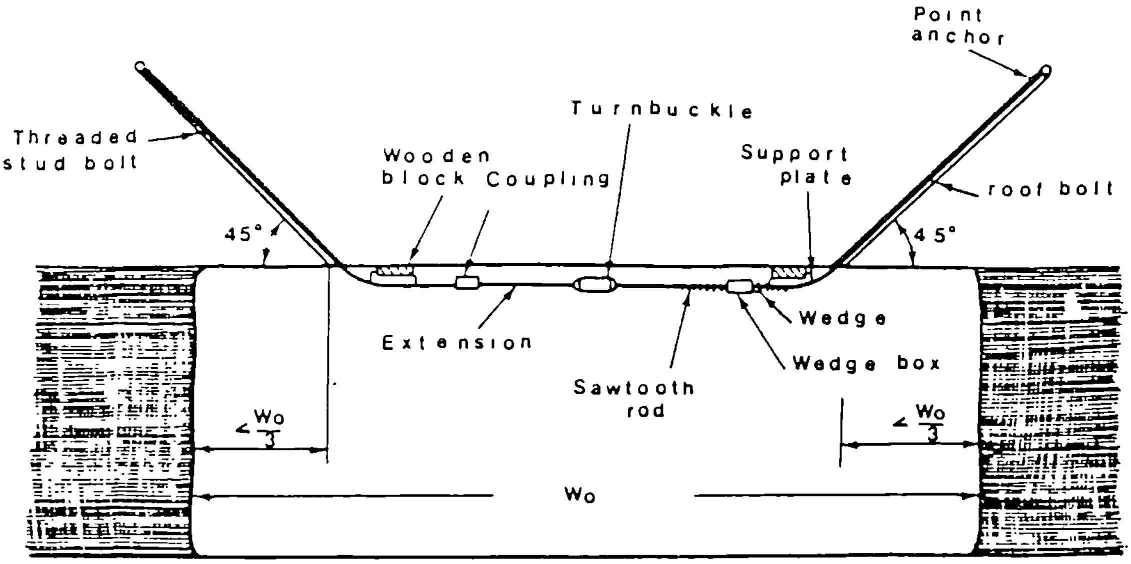
CHAPTER 12

TRUSS BOLTING TECHNIQUES

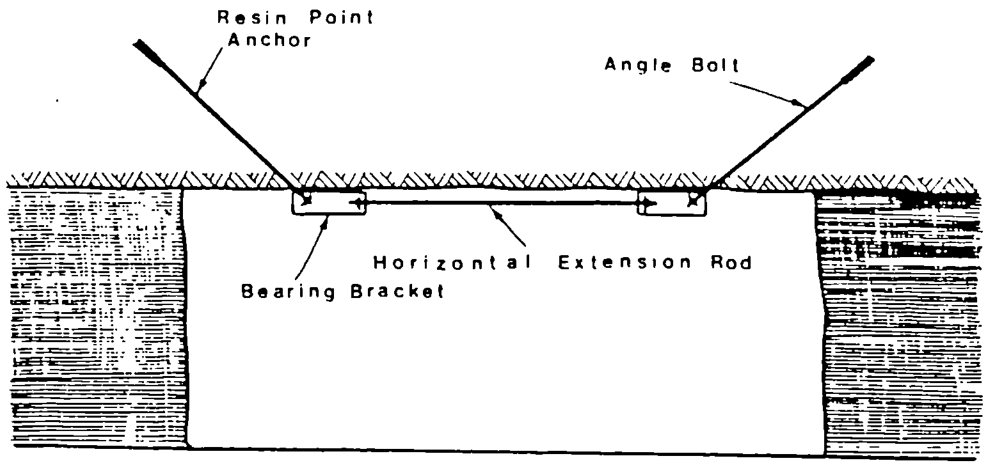
12.1 Background

During the late 1960s White (1969, 1970) developed a support system that reinforces the roof in the same way that a "Queen's Post Truss" reinforces a beam; i.e. by stiffening and strengthening bending members by the attachment of an underslung, sometimes pretensioned, metal strap, rod or cable. This device known as a sling truss or White truss, consists of two point anchored bolts inclined over the roadway ribsides, which are bent over the collars of their holes and joined to form a horizontal chord (Figure 12.1a). A turnbuckle is used to apply an initial tension and a "wedge box" gives some adjustability to the standard lengths of rod components in order to compensate for variations in hole lengths and locations. Bearing blocks (either wood or dimpled steel plates) separate the horizontal chord from the roof to facilitate tightening. Installation of the sling truss is relatively complex. There are a number of different pieces of hardware which must be handled manually and completely assembled before it can become operative. It is therefore difficult to install at the head end during roadway development and has traditionally been viewed as a supplemental form of roof support.

During the early 1980s a different type of truss known as the angle bolt truss came onto the market. A number of designs are available (developed independently by several manufacturers), which are basically similar. These trusses consist of two angled bolts, the heads of which are connected to one or two horizontal chords, tightened to some predetermined tension (Figure 12.1b). Installation is a relatively simple process; they can therefore be used as primary support installed at the head end with other conventional bolts. The horizontal chord may be attached at the time of angled bolt setting or sometime later. McDowell (1987) recommends that if such a delay is necessary, it should be kept to a minimum and under no circumstances should the horizontal chord not be installed. McDowell has found that tensioned point anchored bolts angled over the ribsides can be detrimental to roof stability as they may induce tensile failure in the immediate roof at the ribsides.



a. Sling type



b. Angle bolt type

Figure 12.1 Truss bolt systems.

12.2 Scale Model Studies

An attempt was made to simulate a sling type truss in a scale model roadway. The model strata configuration used is given in Appendix 1a. The chords were simulated by steel wire which was point anchored in a section of electrical terminal block at the top of the angled holes. The truss was tensioned by means of an extension spring (Figure 12.2) stretched to simulate an applied tension of 50 kN (a force scale factor of 1.14×10^{-5} determined by dimensional analysis was used). The spring was then covered with cyanoacrylate adhesive to prevent further extension or relaxation. Two additional full column anchored bolts with a 1.8 m equivalent length were installed in the centre of the roadway. The rows were set at an equivalent of 1 m centres.

The model was loaded hydrostatically. Figure 12.3 is a plot of percentage roof lowering against applied pressure, comparing the truss bolted model with a model roadway supported by full column anchored bolts with an identical configuration (Appendix 2d). The truss bolted model suffered slightly less roof lowering at low applied pressures. At a pressure of 0.56 MPa the roof began to deform and fracture due to failure of the cyanoacrylate surrounding the spring. Therefore to assist in establishing the effect of truss bolting under high stress conditions the model truss will require some modification or redesign.

12.3 Limits Of Behaviour

Mangelsdorf (1986) has summarized his analysis of the support mechanisms of the two truss bolt types. This has been reproduced below:

> "The contribution of the truss to roof stability is a pair of uplift forces at intermediate points between the ribs (Figure 12.4). These forces (R) have vertical components equal to the vertical components of the inclined chord tensions. The horizontal components of the uplift forces depend on the relative magnitudes of the horizontal chord tension and the horizontal components of the inclined chord tensions and on the extent of friction and/or bearing present. In the sling truss the horizontal component of uplift is governed entirely by friction (Mangelsdorf 1980a) and at normal friction levels is always towards the centre of the entry. In the angle-bolt truss the horizontal chord tension at installation can be controlled to produce a near vertical uplift, although some inclination towards the centre of the entry may be desirable as it may be offset later by increased inclined chord tension if the roof begins to work (Mangelsdorf 1982). Except in the case of bed separation forming a very shallow roof beam, the horizontal components of the uplift forces probably have a negligible net direct influence on the support of the roof. It is the vertical uplift that is of primary importance.
>
> Mangelsdorf (1980a) concluded that the transfer of tension from the horizontal chord to the inclined ones, or vice-versa, is mainly due to frictional slippage of the bearing blocks. The contribution

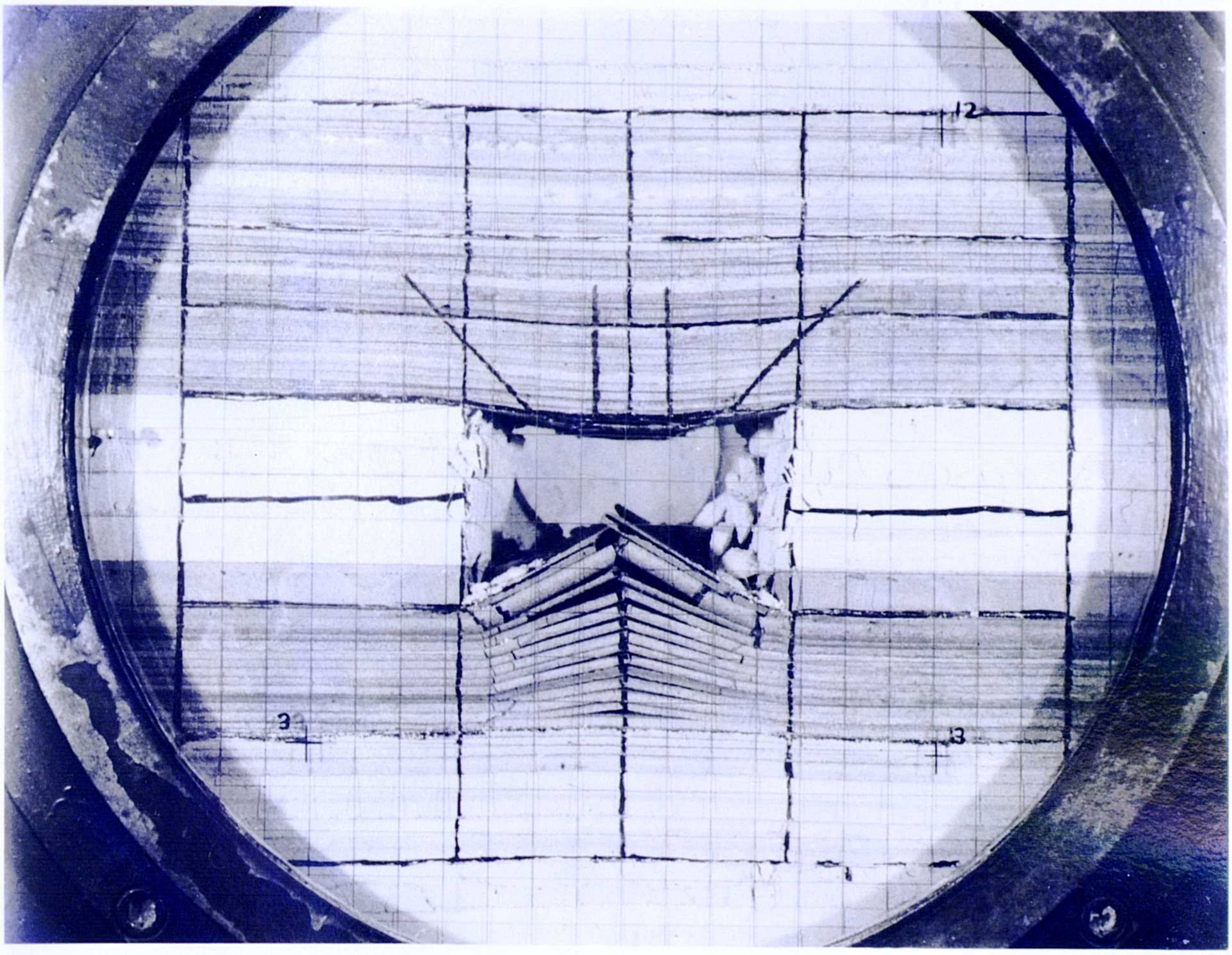


Figure 12.2 Scale model of sling type truss bolt system

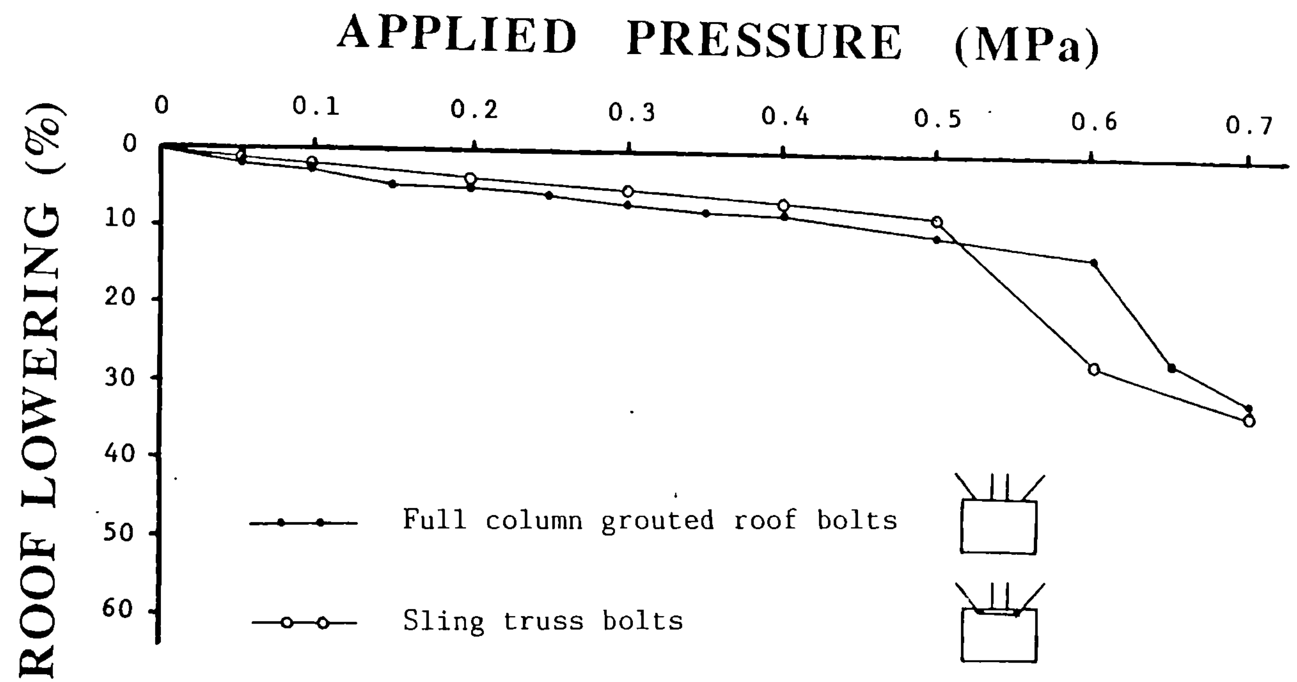


Figure 12.3 Variation in roof lowering as a percentage of initial roadway height with applied pressure - comparison of simulated sling type truss bolts with full column anchored roof bolts.

of friction to the installation and working life of trusses is demonstrated in Figures 12.4. The lower and upper bounds of T/H ratio occur when the bearing block is slipping away and toward the rib, respectively. Installation of the sling truss (Figure 12.4a) follows the path 0-1 with slipping of the bearing block or chord material away from the rib as the horizontal chord is tightened. If, for some reason, H were then reduced without a corresponding decrease in T, the truss would follow the path 1-2 until slipping began in the opposite direction, 2-3. If, on the other hand, the roof began to work causing an increase in T, the truss would follow path 1-4 until slipping began, 4-5.

In angle bolt trusses (Figure 12.4b) a similar though not identical postion prevails. Installation begins with tensioning of the angle bolt, path 0-1. The neck of the bolt bears against the collar of the hole on the side nearest the rib. When the horizontal chord is attached and tightened, the truss follows path 1-2. If it stops at 2 and then the roof begins to work, it will follow the path 2-3, and no slipping of the bearing bracket in either direction takes place. If tightening follows the path 1-2-4-5, then some slipping occurs between 4 and 5. Subsequent loading will then follow 5-6-7-8.

In both systems tightening of the horizontal chord has a direct effect on the tension of the inclined chord only if slipping takes place. The ratio T/H remains essentially constant as does the direction of the resultant uplift force R. Without slipping, such tightening only changes the horizontal component of R. Thus, during tightening of the horizontal chord, either the magnitude or direction of R will be changed, but not both.

On the other hand, during loading, when the initial effect is an increase in the inclined chord tension, R changes in both magnitude and direction if slipping does not occur. Changes in H during this increase in R are generally quite small and are due to minor local deformations in and around the bearing bracket or block and to bending strain in the roof surface which is also quite small except in very shallow roof beams. If slipping does occur, only the magnitude of R changes."

12.4 Optimum Design Of Roof Truss Installations

White (1967) assumed that compression stresses in the rock radiated from the anchors and bearing blocks along lines corresponding to the members of a conventional truss (Figure 12.5). A later analysis (Cox and White 1977) suggested that either suspension or reinforced rock arch action occuring below a pressure or ground arch, may account for the behaviour of trusses.

Scientists at the Central Mining Research Station in India have produced a statical analysis of the sling truss (Raju et al 1972, Sheorey et al 1973). The notation used in this analysis is given in Figure 12.6. Resolving the forces along and perpendicular to the direction of applied tension (T) and taking moments about the point where the truss touches the borehole edge (B), gives:

$$T - \mu R_2 - R_1\sin\alpha - P\cos\alpha = 0$$
$$R_2 + R_1\cos\alpha - P\sin\alpha = 0$$
$$R_2(a + L) + \mu R_2 b - Tb = 0$$

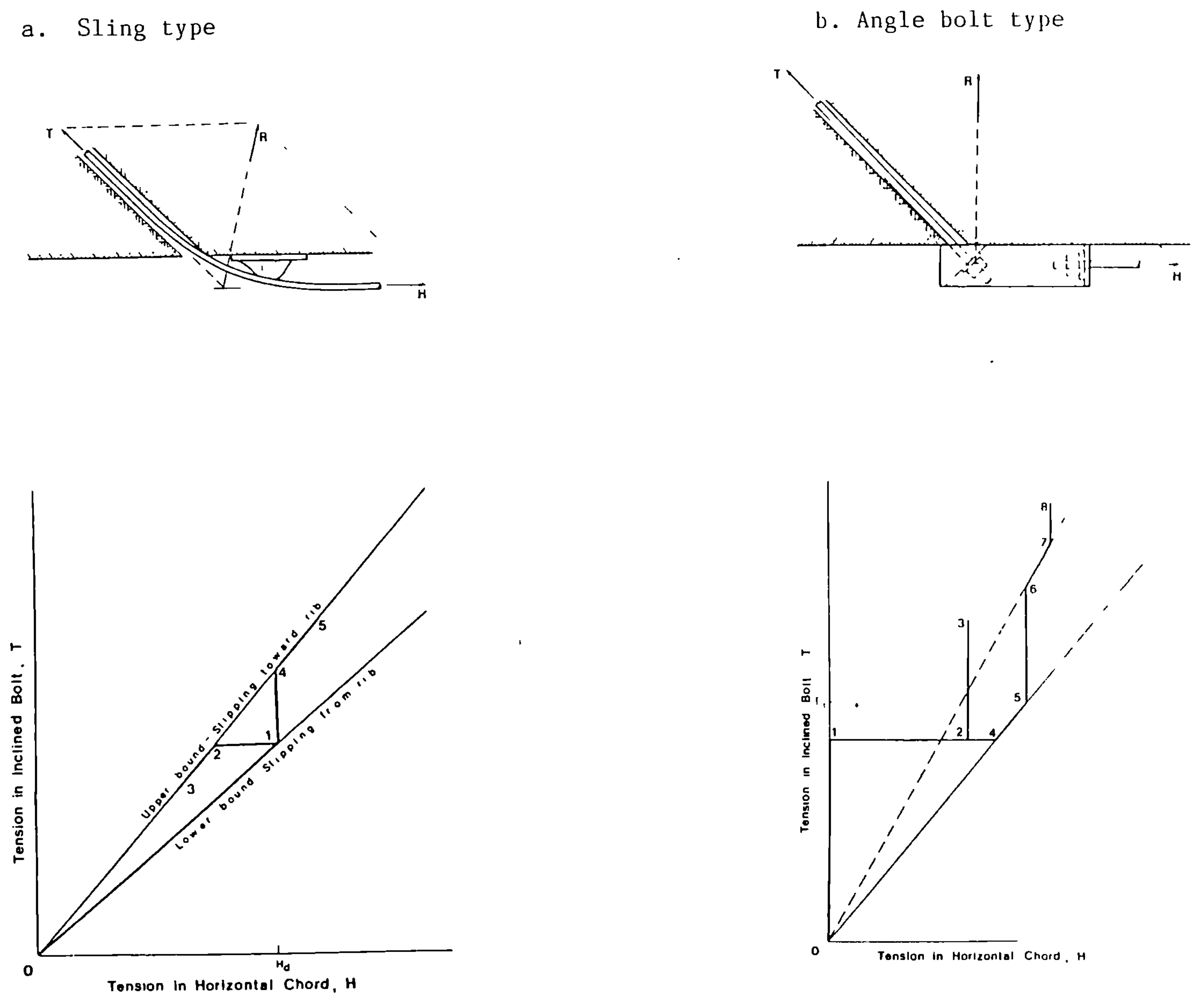


Figure 12.4 Force diagrams and relative chord tensions of roof trusses (after Mangelsdorf 1986).

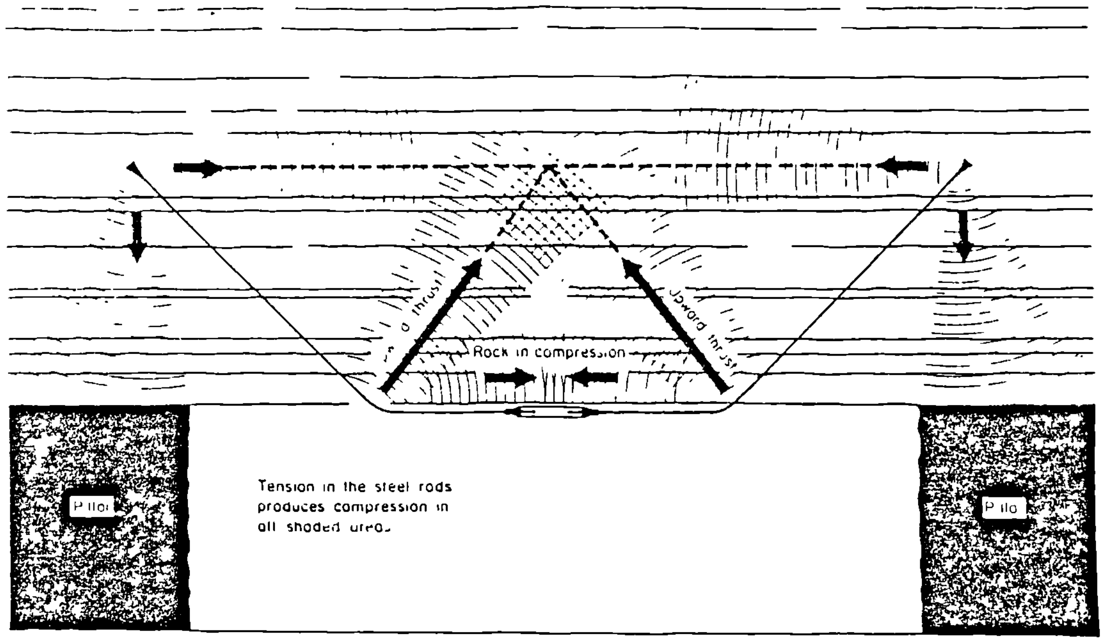


Figure 12.5 Assumed pressure distribution for a fully active sling truss.

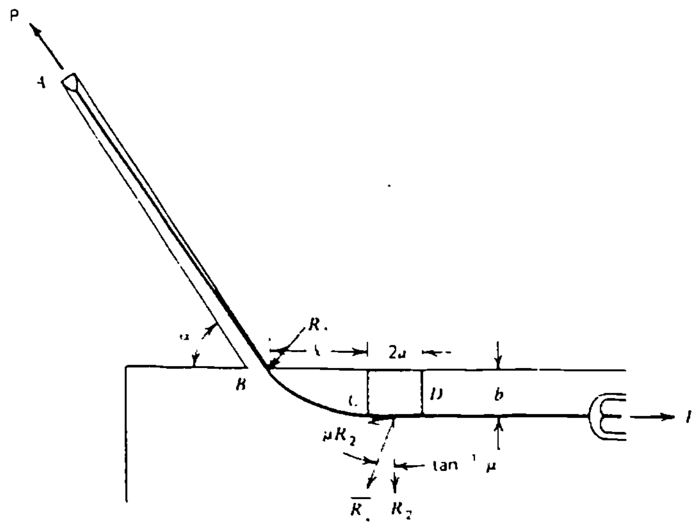


Figure 12.6 Statics of a roof truss (after Sheorey et al 1973).

Solving these equations simultaneously with respect to T, gives:

$$P = \frac{T}{\mu b + a + L}\left[(a + L)\cos\alpha + b\sin\alpha\right]$$

$$R_1 = \frac{T}{\mu b + a + L}\left[(a + L)\sin\alpha + b\cos\alpha\right]$$

$$R_2 = \frac{Tb}{\mu b + a + L}$$

If the bearing block is positioned close to the borehole mouth so that the truss does not touch the roof rock, eliminating a reaction at the mouth of the hole ($R_1 = 0$), then:

$$T - \mu R_2 = P\cos\alpha$$

$$R_2 = P\sin\alpha$$

$$P = \frac{T}{\cos\alpha + \mu\sin\alpha}$$

$$R_2 = \frac{T}{\mu + \cos\alpha}$$

$$\alpha = \tan^{-1}\frac{b}{a + L}$$

Where P = load on the anchorage of the bolt
T = tensioning load on the truss
R_1 = reaction at the mouth of the hole
R_2 = reaction at the block
L = distance from the block to the hole
$2a$ = width of block
b = thickness of the block
μ = coefficient of friction between the block and the roof rock
α = angle of inclination of the hole

Several assumptions have been made in this analysis, namely that:

(i) the truss chord follows an arch in the vicinity of the borehole mouth, in practice angular kinks occur where the truss contacts the rock and the bearing block;

(ii) friction occurs at the block but not at the borehole mouth and only one magnitude of friction coefficient was considered;

(iii) no displacement occurs at the borehole mouth or above the bearing blocks.

The analysis also takes no account of:

(i) any reaction perpendicular to the rod at the anchorage point;

(ii) any movement about this point;

(iii) the rigidity of the steel used.

Hodkin (1975) has produced a more detailed analysis taking these factors into account but continuing to make the same assumptions. The improvement in accuracy obtained is relatively small, so for practical purposes the Indian analysis is generally sufficient.

Neall et al (1976, 1978, 1981) have carried out photoelastic model studies with scaled trusses based on the beam reinforcement concept. The study concluded that the optimum angle for inclined chord installation is between 45° and 60°. This was confirmed by Khair (1983). Neall et al also found that more effective support could be achieved by moving the holes closer towards one another; but only at the expense of longer holes if the anchorage over the rib is to be maintained. These tests were carried out using only one roof span and only one truss span; there was also considerable scatter of the data.

According to Seegmiller (1980), in a rectangular roadway with a width to height ratio of 2 and roof rock with a Poisson's ratio of 0.25, the tensile-compressive equivalence points occur approximately at a distance of 3/10 of the roof span from the centre of the span toward the rib (Figure 12.7). Seegmiller recommends that a truss bolt system should be placed to maximize the support force in the tensile zone while still maintaining effective anchorage in the roof. The optimum position of the angled bolt would be at the point where the stresses change from tensile to compressive. This translation point will be a function of the roof span and opening height. In roadways with width to height ratios between 2 and 3, the preferred angled bolt location would be at the edge of the central 60 to 65% of the opening.

Locotos (1987) recommends that the distance from the rib to the angled bolt hole collar should be 1/5 the entry width, and should not exceed 1.2 m. Locotos uses the following formula to determine the angled bolt length (L_{rb}):

$$L_{rb} = \sqrt{2[(r - s) + 1.5]^2}$$

where r = distance from centre line to hole location

s = half maximium entry width

This ensures that the bolt is anchored 0.45 m into the ribside pillar.

Mangelsdorf (1985a, 1987a) has shown that by minimizing bending strain energy it is possible to optimize inclined chord location and tensions for roof trusses.

To Determine Bolt Angle (θ):

(i) Calculate angle bolt tension (β)

$$\beta = 2T/wL$$

where T = installed chord tension

L = width of roadway

w = load acting on roof truss system

(ii) Calculate aspect ratio (β/λ)

where λ = ratio of free anchor length to roadway width

$= \iota/L$

ι = free anchor length of angled bolt

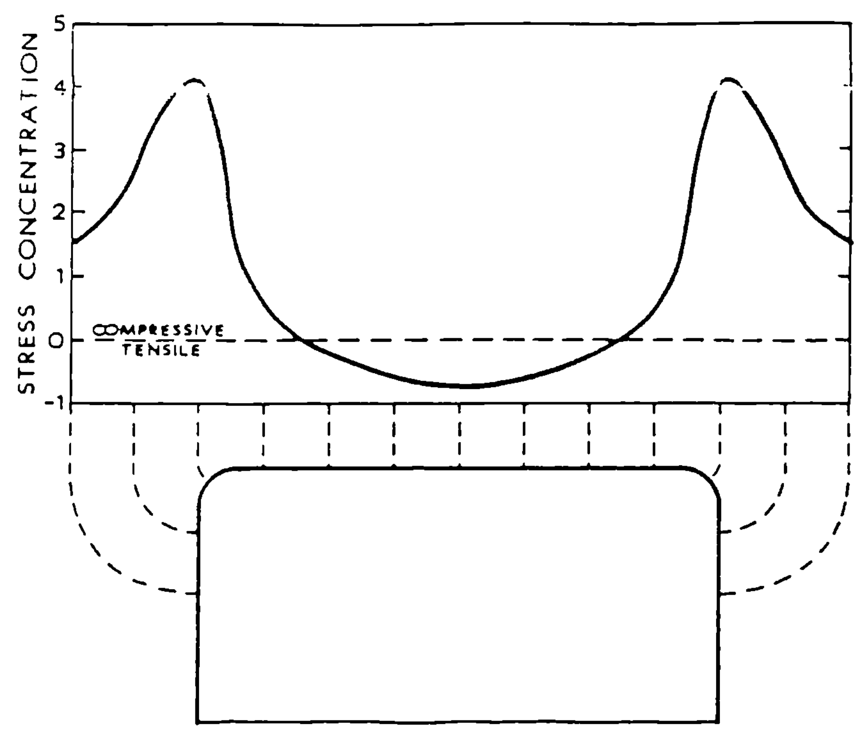


Figure 12.7 Approximate boundary stress conditions in a rectangular opening (after Seegmiller 1986).

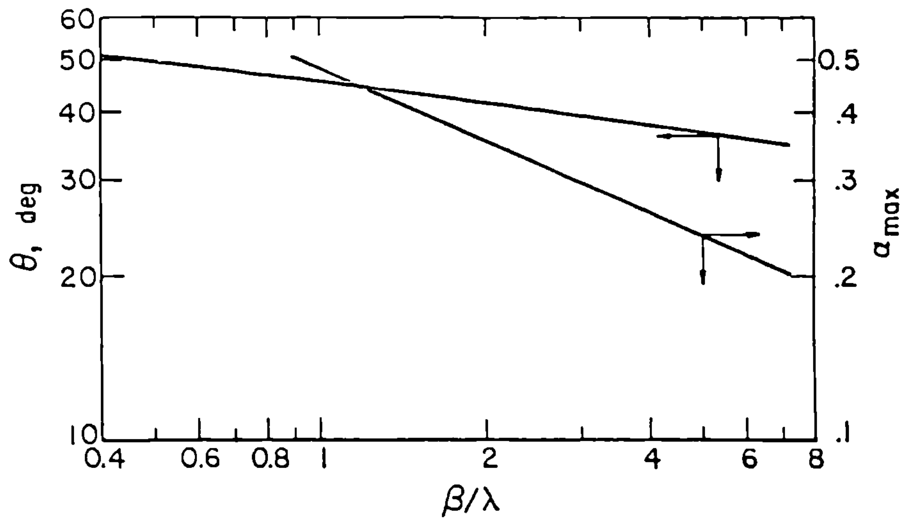


Figure 12.8 Optimum slope for angle bolts based on bending strain energy (after Mangelsdorf 1987a).

(iii) Consult Figure 12.8 to find optimum slope for angled bolts (θ).

To Determine Position Of Mouth Of Angled Bolt Hole From Rib (a):

(i) Calculate the ratio (α) of the distance from the rib to the position of the angled hole (a) to the roadway width (L)

$\alpha = a/L$

Therefore

$a = \alpha L$

12.5 Monitoring

Electrical strain gauges attached to the chords are probably the most direct means of measuring chord tensions, although this will become expensive if a large number of trusses are to be observed.

Mangelsdorf (1980b, 1983, 1985b) has developed a portable, battery operated instrument which measures the natural frequency of vibration of the horizontal chord. It is then possible to calculate the chord tension from a calibration formula. Further evaluation and development of this technique is currently being undertaken by staff at the USBM Spokane Research Center.

12.6 Application Of Sling Type Trusses

A number of articles were published during the late 1960s and through the 1970s reporting successes of sling type truss installations (Anon 1969; Kegel 1969; Kmetz 1970; Raju et al 1972; Mallicoat 1978; Beadnall 1978; NCB 1978; NCB 1980; Round 1979; Mangelsdorf 1979a). Tensioning of sling trusses has been known to lift the roof strata clear off wooden standing support installed in roadways (Kmetz 1969; Raju et al 1972).

A decline in the use of these trusses occurred towards the end of the 1970s due to a lack of proper installation equipment, the relative expense of the hardware, a few explained and unexplained failures, and the emergence of resin rock bolting as a cheaper and faster alternative support for difficult roof conditions.

The most recent sling truss installation in the UK was at Thoresby Colliery in 1985 for 103's retreat face which worked the Parkgate Seam at a depth of 750 m. The main gate was supported by flat topped supports 4.90 x 2.61 m (152 x 127 mm section) set at 1 m intervals with nine heavy duty tubular struts per setting. The roadway was driven some 6 m away from the disused 101's loader gate. In the vicinity of 103's, the

Parkgate Seam is overlain by approximately 1 m of weak laminated planty mudstone which grades into a stronger fine poorly laminated siltstone, containing ironstone nodules, lenses and bands.

Although standards of work were very good it was found necessary to install wooden middle legs along the length of the roadway. Before production had commenced some of the roof beams became deformed. This was probably the result of an inadequate size of pillar between the roadway and 101's workings.

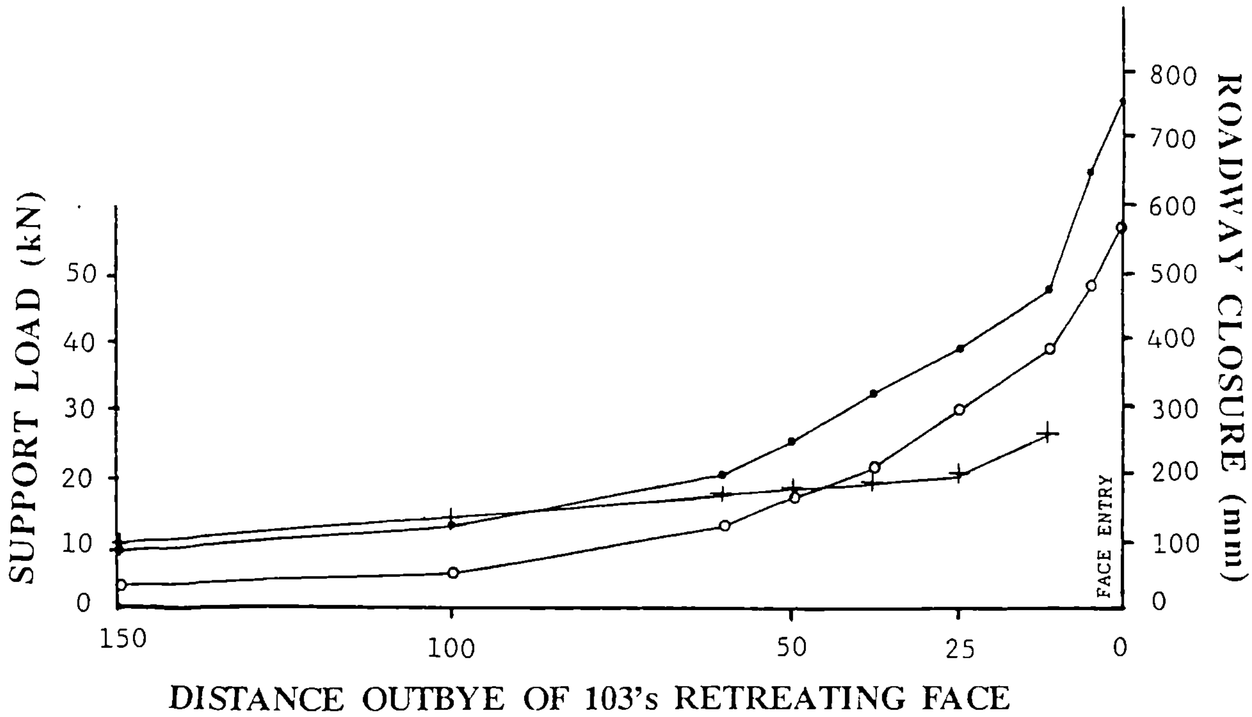
Before face retreat had commenced, sling type truss bolts were installed in two sections of the main gate in addition to the existing steel standing supports. The inclined chords were 2.44 m long, angled at 45° over the ribsides and anchored with resin. Fourteen trusses were set in each section between the steel work. The installation was problematical as the roof was completely sheeted over with corregated lagging.

Monitoring stations were located in a truss bolted section and a section with standing support only, in order to establish the effect of truss bolting on roadway stability as the retreating face approached. The closures and support loading measured (Figure 12.9a and b) showed that no significant benefits were gained from truss bolting; considerable roadway closure and support loading occurred in both sections. The truss bolts were not able to prevent roof rock beam buckling and displacement due to mining induced stress from the front abutment of 103's and the ribside abutment of 101's faces.

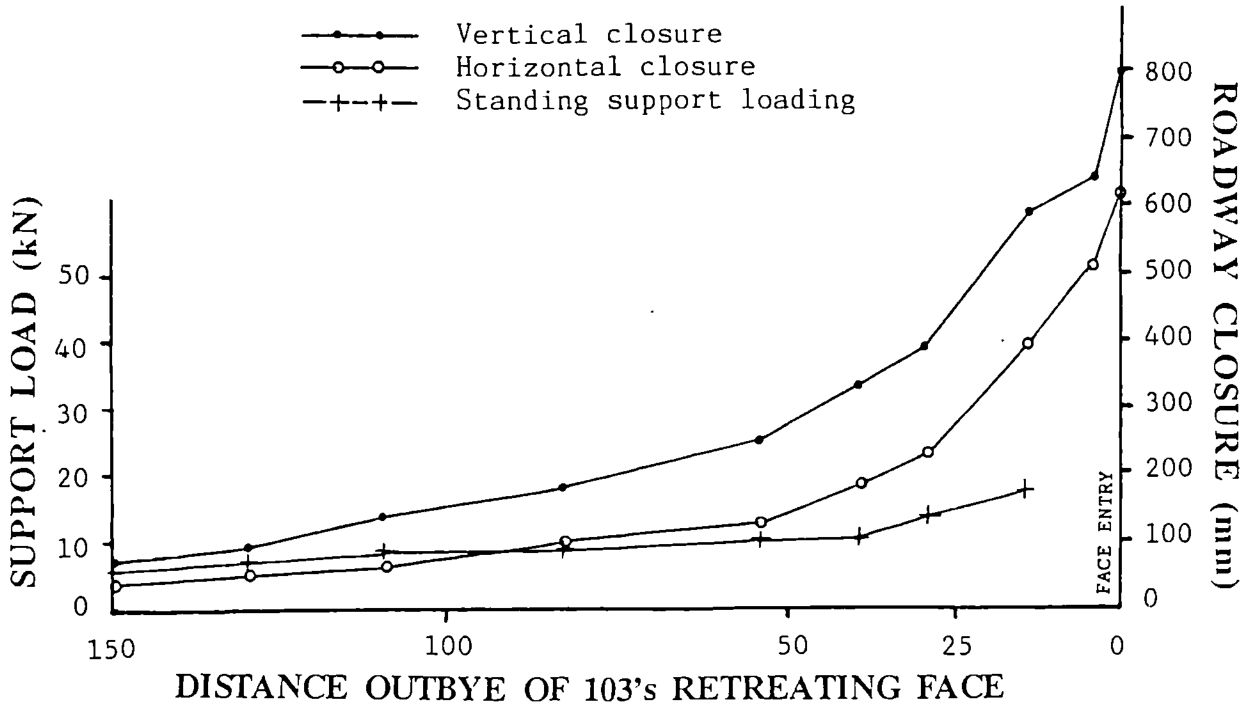
12.7 Application Of Single Bar Angle Bolt Trusses: Tensioned At Blocks

It was stated earlier that a number of different designs of angled bar trusses have been produced. The most successful of these are single bar trusses with a low profile against the roof, a strong bearing block and flexibility/tolerence on placement of the angled holes. The induced and horizontal chords must be in the same plane, otherwise rotation of the block will occur during tensioning and loading, which will lead to tearing of the bracket and bar threads.

Two popular designs were observed in use at the Jane Mine in Pennsylvania, USA. The mine is working the Lower Freeport Seam by room and pillar mining at a depth of 70 to 140 m and with a mineable section of 1.75 m. Overlying the seam is a dark grey mudstone with sandstone laminae and layers which vary from a few millimeters to 350 mm in



a. Section without truss bolts



b. Section with truss bolts as an additional support

Figure 12.9 Convergence and support loading recorded in 103's main gate, Thoresby Colliery.

thickness. Areas of the mine with a high proportion of these laminae and layers tend to suffer roof stability problems.

Limited success was achieved in controlling unstable roof by the use of timber, 152 mm section I-beams, 3 m long coupled resin grouted bolts and point anchored resin grouted expansion shell bolts used in combinations. In addition the main drivages were altered to a more favourable direction. However, mining progress was slow and major roof falls usually 6 to 7.5 m high were still occurring.

The first truss system to be installed in the mine was produced by Pattin Manufacturing Co. It consists of a coupled bar which is connected to the inclined chords at a three-hole block and U-bolt (Figure 12.10). The use of this system in the mine has been described by Barish (1985a, 1985b). The 152 x 152 x 8 mm bearing plates tended to distort and were being pulled into the roof. The plate size was increased to a 203 x 203 x 19 mm plate; the additional bearing surface against the roof alleviated this problem.

Due to the large number of moveable parts inherent in the U-bolt system, a simpler truss marketed by Jennmar Corp. is now being employed at the mine. It has cast ductile iron blocks with a bearing surface of 0.042 m^2 (Figure 12.11) which are quick to install and can withstand loads in excess of 200 kN. The smaller number of moveable parts means that less friction is encountered during the tensioning operation. Consequently the Jennmar system only requires 237 Nm of torque to achieve a tension of 63 kN whereas the Pattin system requires 271 Nm of torque to achieve 62 kN tension.

The trusses are installed in-cycle at the head end with two 1.8 m vertical point anchored grouted expansion shell bolts at 0.9 to 1.5 m centres depending on ground conditions. The 2.1 m long, 19 mm diameter angled bolts are also fixed by a dual resin/expansion shell anchor with a minimum grout length of 0.6 m. The angled bolt hole is drilled at 45^{o}, 0.6 to 0.9 m from the ribside.

Truss bolting has made a tremendous difference to the mine. Management maintain that the mine would not have been able to remain competitive without them. The use of beams, wooden cross bars and support legs has been eliminated. In addition to improved strata control, ventilation and transportation expenses, fewer accidents are now occurring as the mine

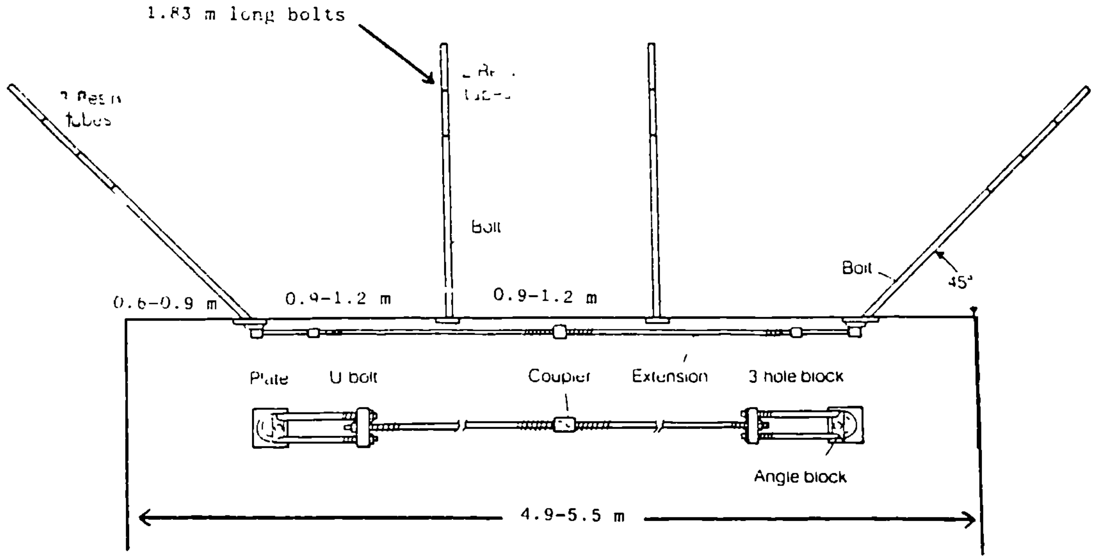


Figure 12.10 Pattin truss installed at Jane Mine.

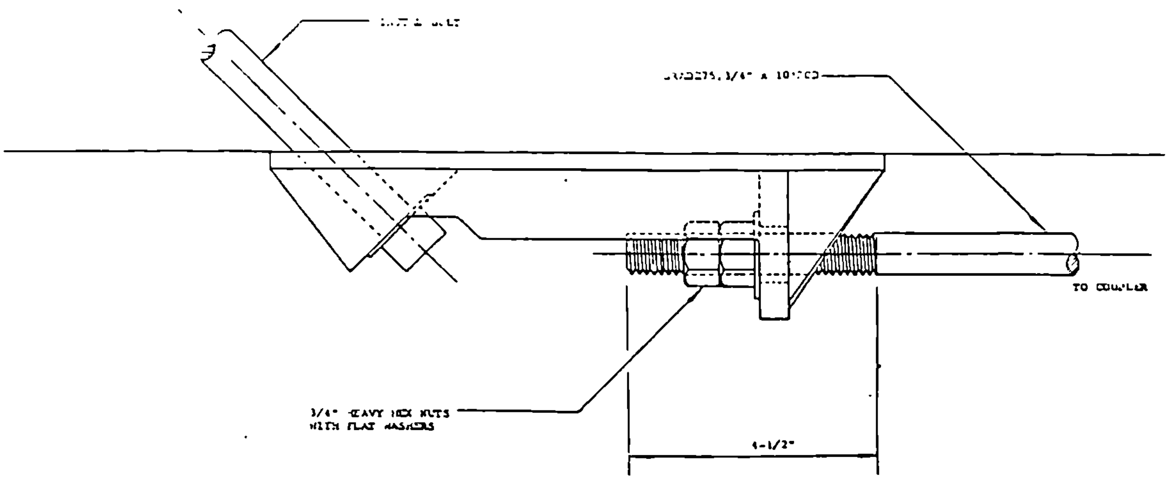


Figure 12.11 Jennmar truss installed at Jane Mine.

workers no longer handle large amounts of extremely heavy, bulky material.

12.8 Application Of Single Bar Angle Bolt Trusses: Wedge-Box Tensioned

The Birmingham Bolt Co., in association with Peabody Coal Co. have developed an angle bolt truss system. The stages of development from the sling truss to the hydraulically tensioned angled bolt truss currently in use was observed at Peabody's Camp No.2 Mine in Kentucky, USA. The mine works the Kentucky No.9 Seam by room and pillar mining. The entries are 6.1 m wide and 1.7 m high, totally in-seam. The workings lie at a depth of between 120 and 140 m. The pillars are at 21 to 24 m centres, this varies depending on the overburden.

The immediate roof consists of a dark grey mudstone which gradually weathers and becomes unstable. The installation of truss bolts has reduced this instability problem in the belt and track entries. Two vertical 1.8 m long resin point anchored bolts are installed across the roadway with each truss setting (i.e. at 1.5 m intervals). The inclined chords are 2.44 m long, bendable and point anchored with a 0.9 m grout length. The inclined hole collars are positioned 1.2 m from the ribside.

The turnbuckle/pipe wrench tensioned sling type truss initially used was replaced by a hydraulically tensioned sling truss with a modified wedge box (Figure 12.12). The development of the hydraulic torque tensioning wrench for truss bolting has been described by Bollier (1982). The tool is capable of applying a uniform tension in every truss, reducing installation time, reducing difficulties in tensioning a truss and providing a method of testing the installed load. The sling has a total length of 8.2 m. If one side of this truss is heavily loaded, the bearing block tends to slip and transfer load to the other side; this is an advantage over the angled bolt truss. However, bearing block slip during tensioning became a problem, consequently a modified plate was used with a notch to grip the roof and prevent slip.

The first angled bolt truss to be installed in the mine had a rigid angle of 45° only, the bolt being fixed to the bearing block by a cotter pin. The rigid angle gives obvious limitations to bolt installation; strain induced in the angled bolts caused a few to break just above the flange.

Thus a flexible angled bolt truss was developed. The bolts consist of a 19 mm diameter rod made from high strength Grade 75 steel. The holes are drilled in two stages; initially with a 38.1 mm or 41.3 mm bit, then a 34.5 mm bit. This type of truss can be installed extremely rapidly. When the angled bolts have been anchored, one of the two sections of the cross bar is inserted into each bearing block (the bars have a T-shaped end to hold it in place, i.e. no nuts are required to secure the cross bar to the blocks). A screw coupler is fixed to the cross bar at one end. Both sections of the cross bar are then inserted into a wedge-box. A nut is screwed onto the threaded rod and then 200 to 230 Nm of torque is applied with a hydraulic torque wrench. The wrench is equipped with an adjustable automatic pressure relief valve, allowing the device to be pre-set to any desired torque or tension range. The wedge-box gives 450 mm of flexibility in angle hole location. The only failure of this type of truss to-date is a fracture at the T-head of one bar during installation.

12.9 Application Of Double Bar Angle Bolt Trusses

Angle bolt trusses with two cross bars which are tensioned at the blocks have recently come onto the market (Figure 12.13). These bolts are generally used under high loading conditions.

A post-development installation of a double tie angle bolt truss system has been successful in preventing excessive roof lowering in a gateroad serving a retreating longwall face at Bailey Mine, Pennsylvania, USA (Locotos 1987). The mine is working the 1.5 m thick Pittsburgh Seam with 150 m long panels. Three or four entries serve each end of the faces. The entries are 5.5 m wide and between 1.7 and 2.0 m high. The seam is overlain by 0.3 to 0.45 m of draw slate, 0.3 m of roof coal and 1.8 to 2.1 m of slickenslided laminated mudstone. Above these beds there is a massive sandstone.

The primary support for the entries consists of rows of four 19 mm diameter, 2.44 m long fully resin grouted rock bolts installed in a 25 mm diameter hole through a 5.2 m long (76 x 203 mm section) wooden plank at 1.2 m centres.

Truss bolts were installed at least 20 m in front of a face line which resulted in a considerable improvement in the face end conditions. Initially some of the angled bolt brackets failed. Recent modifications have strengthened the plate and solved this problem.

Mangelsdorf (1987b) has expressed doubts concerning the necessity for

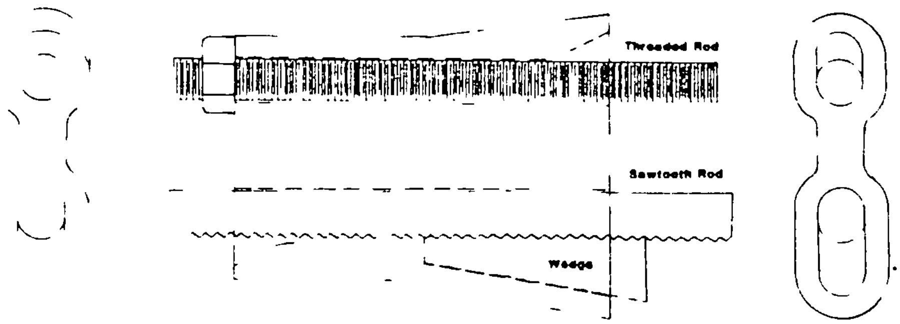


Figure 12.12 Truss wedge tensioner (after Bollier 1982).

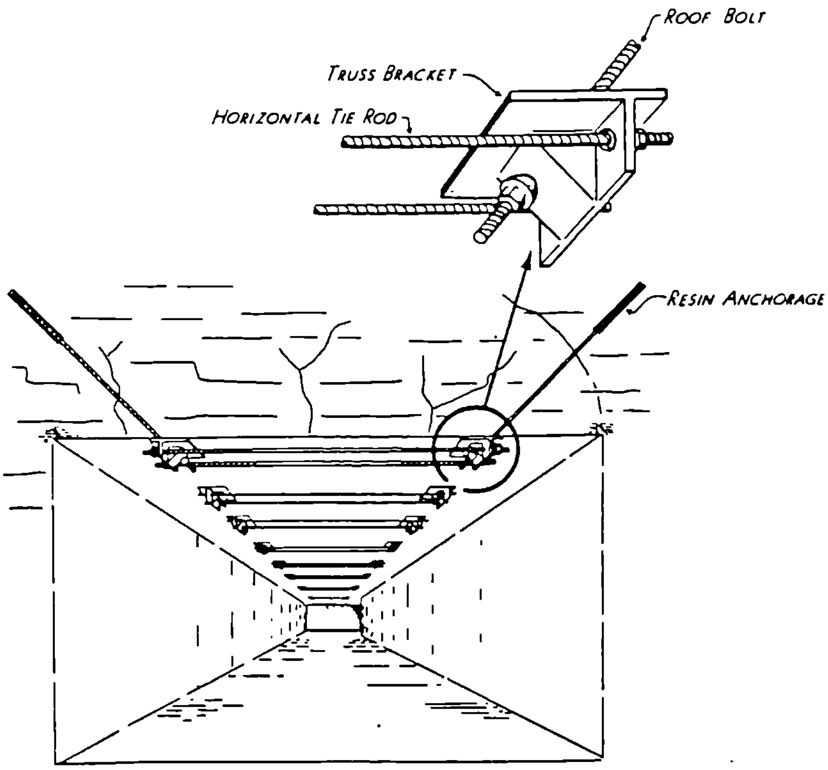


Figure 12.13 Double bar angle bolt truss.

double cross bars. He maintains that the tension in the angled bolts is always greater than in the horizontal chord, therefore unless the individual ties are significantly weaker than the angled bolts, there is no need for double ties. In addition, considerable care must be taken when installing double tie bar trusses; in order to avoid uneven tensioning and rotation of the brackets, each bar must be tightened and then retightened during the installation process. Also the additional steel, hardware and installation time increases the cost of these systems.

12.10 Recent Developments

A four-way truss bracket for use in the centre of double bar angle bolt trusses has been devised by Seegmiller (1987). The trusses are designed for support of mine intersections (Figure 12.14) or to provide additional support along the length of an entry (Figure 12.15).

The successful use of this truss system in three-way and four-way junctions at Deserado Mine in Colorado, USA, has been described by Adams (1987). The central bracket is installed initially and the adjoining tie bars are then tensioned in rotation to avoid excessive side loading on the central vertical bolt.

A trial has recently been initiated involving the post development truss bolt reinforcement of the tail gate of K14's retreat face in the Brass Thill Seam at Ellington Colliery. This is the first use of angle bolt trusses in the UK. Trusses (both single and double tie bar types) have been installed in the 5.5 to 6.1 m wide roadway between existing standing support (consisting of wooden legs and steel RSJs 127 x 114 mm cross members at 1 m intervals). It is intended to monitor the performance of these trusses during face retreat.

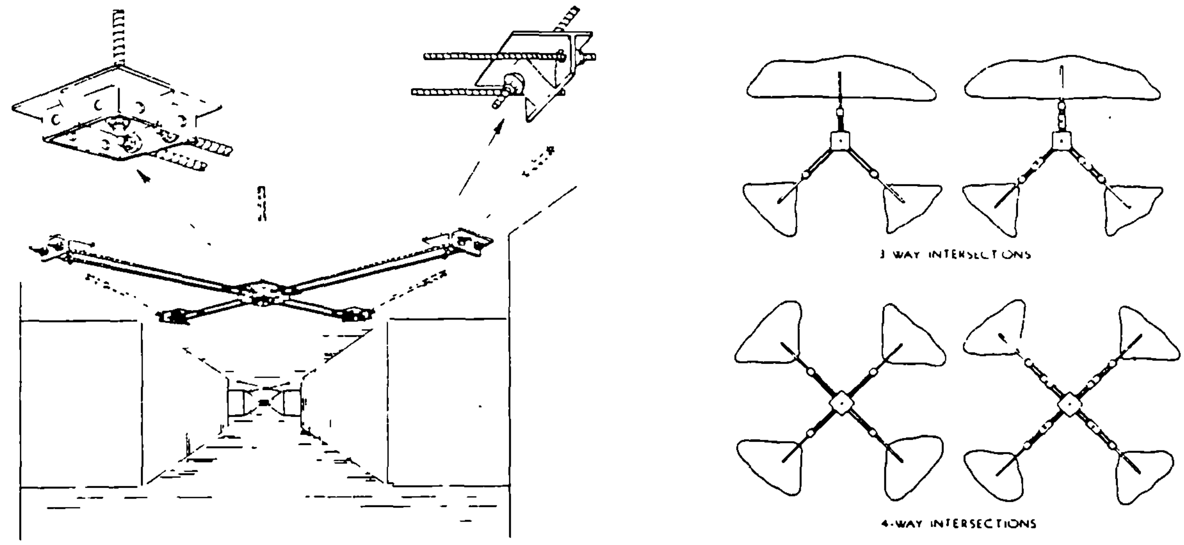


Figure 12.14 Intersection truss (after Seegmiller 1987).

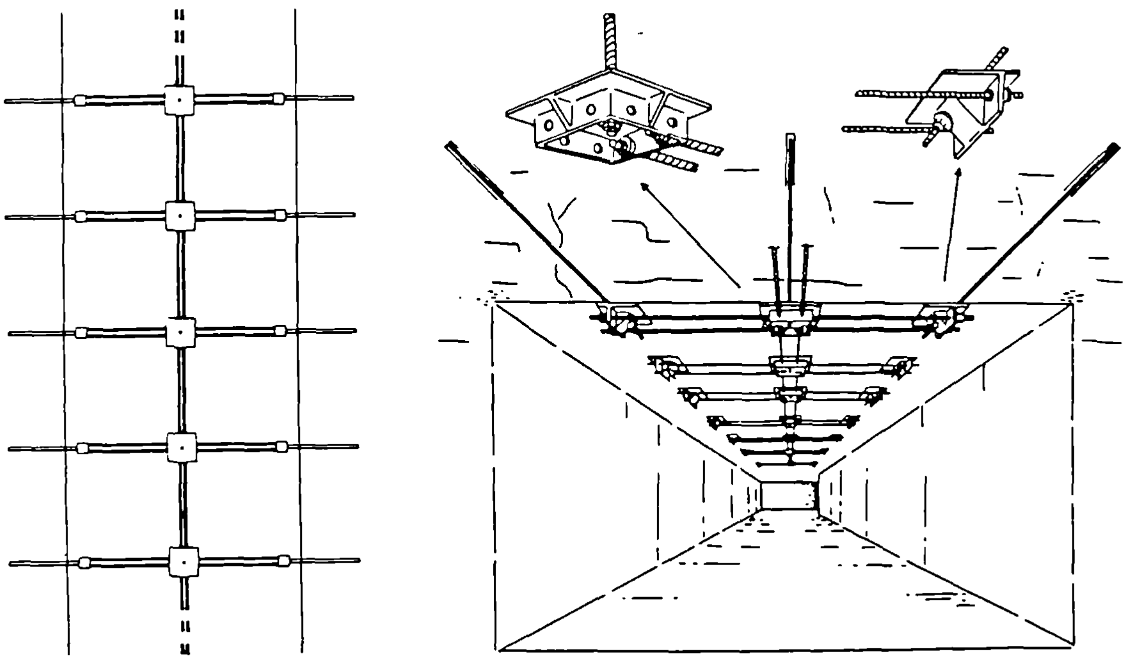


Figure 12.15 Continuous entry truss (after Seegmiller 1987).

CHAPTER 13
EXTENDED GROUND SUPPORT

13.1 Background

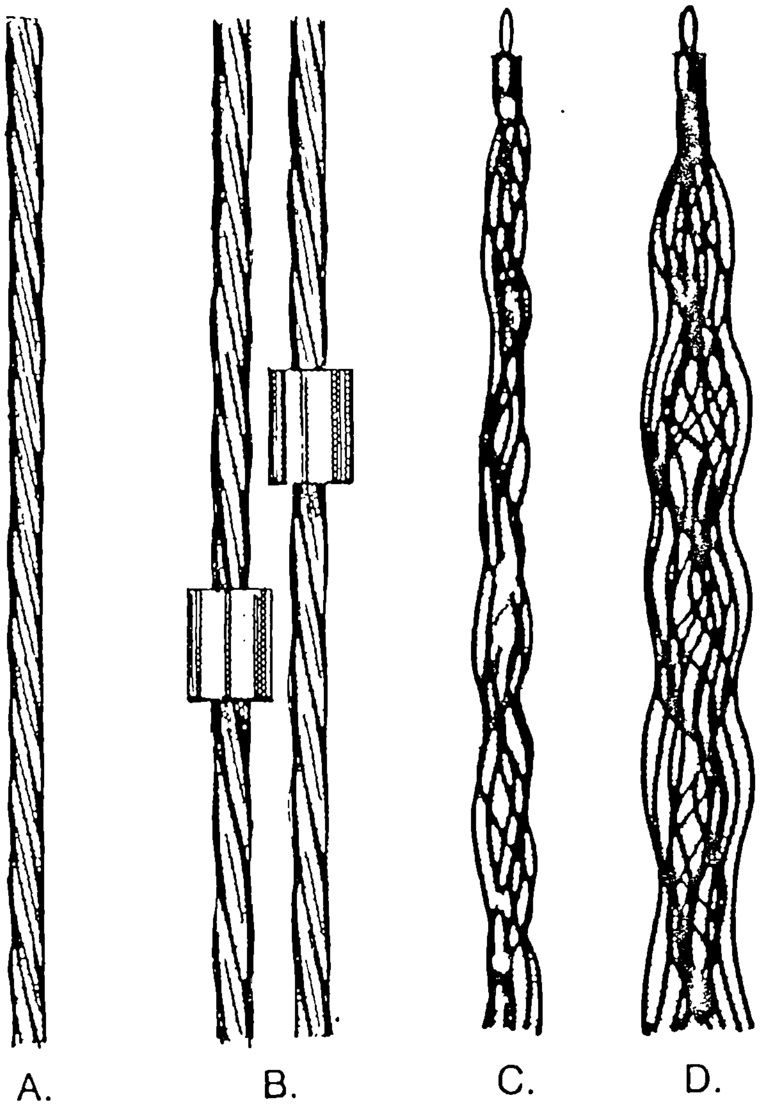
Extended ground support refers to long anchorages installed in an excavation to stabilize large volumes of rock. This type of support permits reinforcement of rock failure occurring at greater distances from the opening than standard rock bolts. Applications are generally limited to locations of high potential instability such as weak strata at depth, fault zones and junctions.

Extended ground support can be achieved by a variety of devices ranging in complexity from simple coupled bolts and cable bolts to sophisticated rock anchors. They can be fully bonded or have a tensioned free length. The use of extended ground support in the form of coupled bolts for floor reinforcement has been discussed in Chapter 11. Rock anchors are expensive and their use in underground excavations is limited to civil engineering projects. Cable bolts consist of high tensile steel cables which are usually fully bonded with injected inorganic grout. Some typical cable bolts are illustrated in Figure 13.1.

13.2 Axial Loading Characteristics Of Cable Bolts

Axial loading characteristics of various types of cable bolt stands are currently under evaluation at the USBM Spokane Reseach Center (Goris and Conway 1987). Pull tests have been conducted on sections of cable embedded in 254 mm of cement grout. After 28 days, all the cables tested could support maximium loads of at least 77.8 kN and showed good residual load carrying capacity.

Epoxy coated strands with embedded grit showed an increased load carrying capacity of approximately 31% over conventional strands (Figure 13.2a). Epoxy coated strands were originally produced for use in pre-stressed concrete members. The 0.76 mm thick coat provides corrosion resistance while the embedded grit increases frictional resistance. During the tests the bond between the coating and the strand remained intact. The increase in the load carrying capacity of the epoxy coating, together with its chemical resistance, makes this type of strand very attractive for long-term use in cable bolt support systems. However, the cost of the epoxy coated strand is approximately twice that of bare strand.



A. Single strand 15.2mm dia. 7 wire stress relieved - 250kN min. breaking load. Lengths up to 20m with swaged support fittings. Recommended hole size 38mm dia.

B. Double strand (each 15.2mm dia.) Providing min. break load of 500kN, recommended hole size 44mm dia.

C. Birdcaged single strand as per "A" but wires separated to increase adhesion factor by 100%. Recommended hole size 44mm.

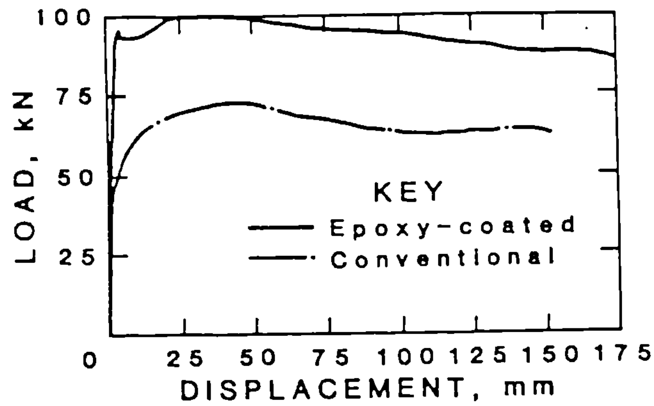
D. Birdcaged double strand providing min. breaking load of 500kN. Recommended hole size 53mm.

Figure 13.1 Typical cable bolts (after Arnall-Wickman 1986).

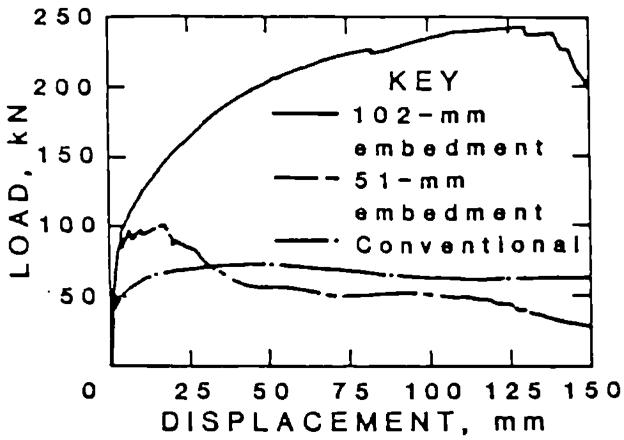
The resistance to pull out developed by a grouted strand is due initially to the mechanical interlock along the grout-strand interface. Once slippage begins, pull out resistance is due to friction along this interface. One method of increasing pull out resistance is to add a bearing surface inside the grout column perpendicular to the axis of the strand, thereby transferring the load between the strand and the grout by compression of the grout. This can be accomplished in various ways, such as attaching a stressing anchor to the strand or by pressing a thick wall sleeve onto the strand. One such device in use is referred to as a steel button. The buttons are generally 25 to 32 mm in diameter and 38 to 45 mm long. Preliminary tests reported by Goris and Conway (1987) have shown that buttons have potential for increasing the load carrying capacity of cable bolts over conventional strands by as much as 219%. Although it appears that the location of the button within the grout column will greatly influence the pull out resistance of the system (Figure 13.2b). It must therefore be ensured that the buttons are placed in the proper location i.e. at least 50 mm from the back end of the hole and any discontinuity. This is obviously a difficult task and consequently could be a major disadvantage to their use.

A recent development for the Australian mining industry is the birdcage cable bolt. Nodes are made along a cable strand by separating the seven wires of a conventional strand, rotating the outer six wires slightly and then recombining the wires to form an open strand where the surface area of all the wires comes in contact with the grout.

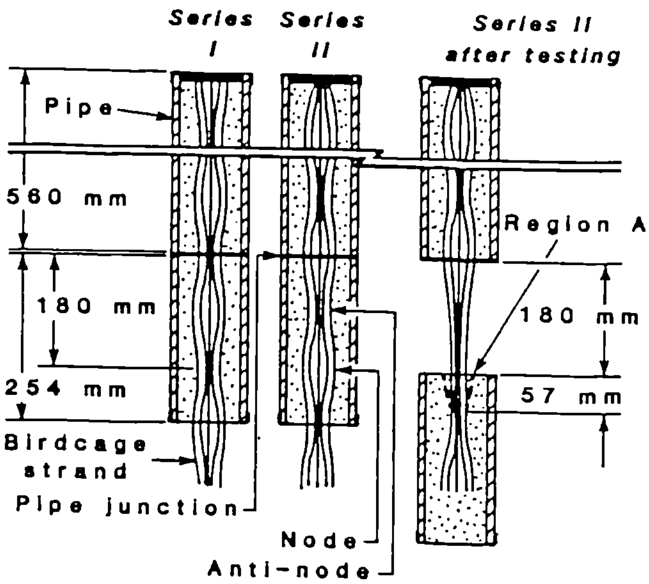
The behaviour of this bolt under load is influenced by many factors, an important one being the location of the node with respect to rock discontinuities. Goris and Conway (1987) have reported two pull tests on birdcage bolts: Series I with an antinode located at the top of the embedded length and Series II with a node located at the top of the embedded length (Figure 13.2c). The load-displacement curve for the two series are similar in shape (Figure 13.2d); however, the average maximum load achieved by the Series II samples is approximately 24% lower than for Series I. There was a loss of grout column on the Series II cables in the region "A" on Figure 13.2c because the wires in this region were deflected toward the centre due to tensile load. The grout surrounding the wires became highly fractured and offered little or no resistance to pull. This phenomenon did not occur with Series I samples. The two main peaks in the Series I load-displacement curve correlates with the



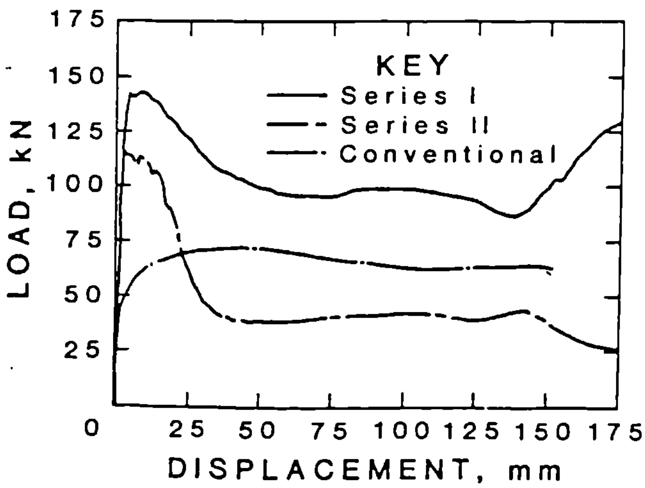
a. Results for epoxy-coated strand



b. Results for samples with steel buttons



c. Samples with birdcage strand



d. Results for birdcage strand

Figure 13.2 Cable bolt pull tests (after Goris & Conway 1987).

spacing of the nodes. To date, the test data accumulated shows that the maximum load carrying capacity of the birdcage bolt was between 47% and 94% higher than that of bolts with a conventional strand.

The grout used in the above tests was made from Type I cement and had a water-cement ratio of 0.45 to 1.0. It has become apparent from a study of this grout that water bleeding by capillary action will result in collection of water at the top of the hole. Depths of water measured were approximately 25 mm per 300 mm of grout (Brady 1987). Water bleeding is therefore an important consideration when designing cable bolt lengths.

13.3 Cable Bolt Applications

Principal applications of cable bolts in the past have been for metalliferrous mining in cut-and-fill, open stoping and block caving operations. Details of cable bolts used in some of these mines are given in Table 13.1.

Cable bolts have been installed in junctions and longwall gateroads of some Australian coal mines. The first of these trials was successfully completed at Tahmoor Colliery, New South Wales, in 1983. This technique is still in the development stage but it is considered by some Australian strata control engineers to be especially useful in areas of high stress.

Singh et al (1986) describe a method of depillaring in a 6.5 to 8 m thick seam at New Chirimiri Ponri Mine, India. Cable bolts with a diameter of 22 mm were anchored with injected cement grout through approximately 3.5 m of roof coal to at least 1.5 m within the sandstone roof. The roof coal was blasted down, leaving 1.5 m of bolt which maintained a stable roof during coal gathering operations. This system improved recovery by up to 70%.

13.4 Swellex Long Rock Bolt

According to the manufacturers (Atlas Copco 1985) flexible Swellex bolts up to 9 m long can be installed in head-rooms as low as 2.75 m. The standard Swellex is described in detail in Section 14.3. Long Swellex bolts are highly cost competitive compared with other extended ground support systems. They are cheaper than coupled rebars, do not require grouting and are quickly installed. The manufacturers quote an under-

MINE	METHOD	CABLE DIAMETER (mm)	CABLES PER HOLE	HOLE DIAMETER (mm)	LENGTH (m)	REFERENCES
New Broken Hill, Australia	Cut & Fill	15.2	2	65	20	Hunt & Askew (1977)
Con, Canada	Cut & Fill	15.9	2	57	9 - 21	Cassidy (1980)
Tsumeb, Namibia	Cut & Fill	24.0	1	57	21	Stheeman (1982)
Myra, Canada	Cut & Fill	15.9	2	51	15	Walker (1986)
Malmberget, Sweden	Open Stoping	36.0	1	N/K	15 - 25	Sellden (1983)
Kotalahi, Finland	Open Stoping	15.2	2	41 - 64	6 - 50	Lappalainen et al (1983)
San Manuel, USA	Block Caving	15.2	1	38	10 - 23	Stevens et al (1987)

Table 13.1 Details of cable bolts used in metalliferrous mining.

ground installation of a 7.3 m long Swellex bolt where the drilling for the 38 mm diameter hole was 8 minutes and it took two men just 3 minutes to insert and inflate the bolt.

To date, applications of these bolts have primarily been limited to the reinforcement of drifts ending in open stopes, where the change to large diameter blast holes has created stability problems. At Westcliffe Colliery, Australia, 7.3 m long Swellex bolts have been used in a fall recovery operation to secure the front wall and as forepoles from the tailgate (Pugh et al 1987).

13.5 Extended Ground Support Design

There is no reliable quantitative design method available to determine the optimum spacing of extended ground support systems because the distribution of loading along the anchorages cannot be accurately predicted. Consequently bolt spacing is generally chosen empirically. The Finnish Outokumpu Oy Mining Company use finite and boundary element methods of calculation qualitatively for cable bolting to locate the most critical zones to be reinforced.

CHAPTER 14

ALTERNATIVE ROCK BOLTING SYSTEMS

14.1 Background

During the last fifteen years several different rock bolting systems have been devised including yielding bolts, friction bolts and a number of systems utilizing a variety of types of rod and anchorage. A few of these have found applications in mine roadway support systems.

The so called "friction rock bolts" (eg Split Set and Swellex) are widely used in hardrock mining operations. If these bolts are to be classified together a better descriptive term would be "full column mechanical anchored bolts" as all other types of rock bolt also offer some form of frictional resistance.

14.2 Yielding Rock Bolts

The stiffness of grouted rock bolt systems offers good restraint to slowly increasing stresses. Where rapid stress changes occur at a fracture plane in a bolted strata zone or where a weak bed in predominately strong bolted strata is undergoing rapid failure, elastic rupture of the grout and steel may take place. Under these circumstances a yielding bolt may be beneficial to relieve these stresses and then take up strength once more in a new state of equilibrium.

Some simple concepts of yielding bolts designed to cope with excessive rock deformation resulting from seismic activity in the South African mining industry are illustrated in Figure 14.1.

A yielding bolt developed by the USBM (Conway et al 1975, 1977) did not prove entirely successful. The yielding effect was achieved by a smooth bore die fitted to a bolt at the borehole opening. Consequently, it was not very effective with fully grouted rebar when the bolt was stressed deeper in the borehole (Reuther and Hermulhein 1985).

Some yielding bolt concepts have been developed for use in the German coal mining industry (Grotowsky 1981; Baur and Brune 1984; Gotze 1986; Stephan 1987). The "kombi anker" is one such device (Figure 14.2) which has a high load-bearing peak but is also highly ductile; i.e. it combines the function of a rigid bolt with that of a tensile or yielding bolt. The ductile inner core of these bolts is pre-tensioned against an

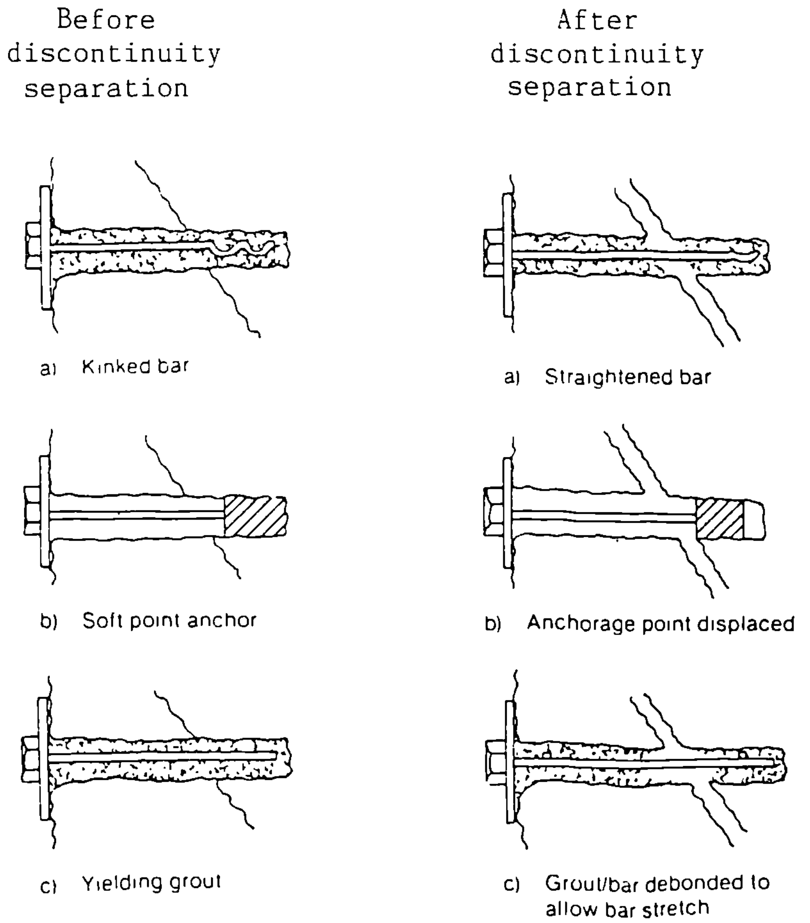


Figure 14.1 Some concepts of yielding rock bolts (after Moore & Noyons 1986).

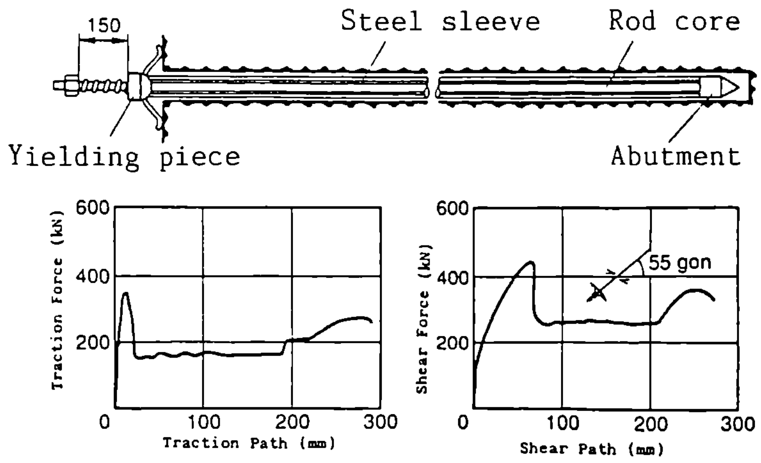


Figure 14.2 Load bearing and deformation behaviour of a kombi bolt with a length of yield of 150 mm (after Stephan 1987).

outer sleeve, the extent of which determines the load bearing peak of the bolt. The core and sleeve act together to prevent initial rock deformation. At high stresses, the outer sleeve tears off and the section of core that protrudes into the roadway is drawn into the borehole, permitting considerable axial and shear deformation without failure.

The debondable bolt (Daws 1978, 1980b) has similarities to the kombi anker. A core of high strength steel, threaded at one end and with a cylindrical block at the other, is surrounded by a series of high density polyethylene sleeves. The sleeves debond under load, permitting controlled deformation to occur.

14.3 Split Set Rock Bolts

The Split Set rock bolt was invented by J.J. Scott in 1973, developed by Ingersoll-Rand Co. and introduced into the US mining industry in 1977. The bolt is manufactured from a 2.3 mm thick hot rolled low alloy steel sheet that is formed into a tube with a 16 mm diameter longitudinal slot. The tubes are cut to specific lengths; a taper is shaped at one end and a ring flange is welded to the other end to support the bearing plate. During installation the bolt is driven into a slightly under-sized borehole. The slot permits compression of the tube but does not close completely so that a radial force is applied against the rock along its contact length (Figure 14.3). Split Sets are available with nominal outer tube diameters of 33 mm, 39 mm and 46 mm. Standard Split Set lengths range between 0.9 m and 3.7 m.

Split Set bolts require precussive or vibrating insertion equipment. The anchorage obtained during installation is dependent on the degree of interference. This is defined by Scott (1976) as "the difference in diameter between the manufactured Split Set and the borehole in which it is placed. It also includes anchorage obtained due to the frictional coefficient, borehole deviation, borehole rifling, broken ground etc".

According to the manufacturer, an initial anchorage of between 30-60 kN for the 33 and 39 mm bolts, and 50-90 kN for the 46 mm bolt should be achieved. The drilling parameters required to attain this will vary from site to site and should be determined by a series of pull tests. The tests should be carried out on bolts installed in holes drilled with bits 1 to 5 mm smaller than the outer diameter of the Split Set. Each hole must be at least 50 mm longer than the bolt. The sensitivity of

Split Set performance to hole diameter can be a major draw back with this system.

Scott (1978, 1980a) states that Split Sets act over their entire contact length and thus prevent the formation of any large stress concentrations to creep or bleed off with time. However, the method of insertion will cause the Split Set to take the shape of the smallest diameter of the borehole which in certain circumstances could limit the percentage of good contact. In fractured or laminated strata, the loosening of rock fragments inside the hole during bolt installation may impede borehole wall contact. Poitsalo (1983) overcored some Split Sets and found these bolts to be only partly in contact with the borehole. This characteristic will result in variable pull test results and tend to make the bolts flexible and deformable.

In addition to the radial forces, an axial confinement load is also produced during installation. Plate loads of 30-40 kN have been recorded (Scott 1980b; Chaiko and Scott 1977).

The yieldable aspects of Split Set bolts, without loss of restraint against the rock, are emphasized by the manufacturers (Bronder 1986). Pull tests have shown that the load-bearing capacity of Split Sets can increase with time (Scott 1976; Chaiko and Scott 1977; Scott 1977; Scott and Jackson 1977; Scott 1980; Croizat et al 1982; Liangkui and Shendou 1983; Scott 1983). The increase in anchorage is due to slight corrosion of the bolt surface and a higher radial tension caused by deformation of the surrounding strata. The tests also indicate that the bolt is capable of sliding as a unit within the borehole as bed separation occurs, without losing anchorage along its contact length. Failure of the bolt end ring can occur with continued roof dilation causing plate contact to be lost. Ring detachment will give an early warning of a potential roof failure.

Shear tests carried out by Haas et al (1978) have concluded that Split Sets are capable of withstanding large deformations while still resisting shearing.

The use of Split Sets in coal mining has been limited to date, although a monitoring programme at a trial in an Australian Colliery found that they provided a comparable support to resin anchored roof bolts (Richmond and Hebblewhite 1980).

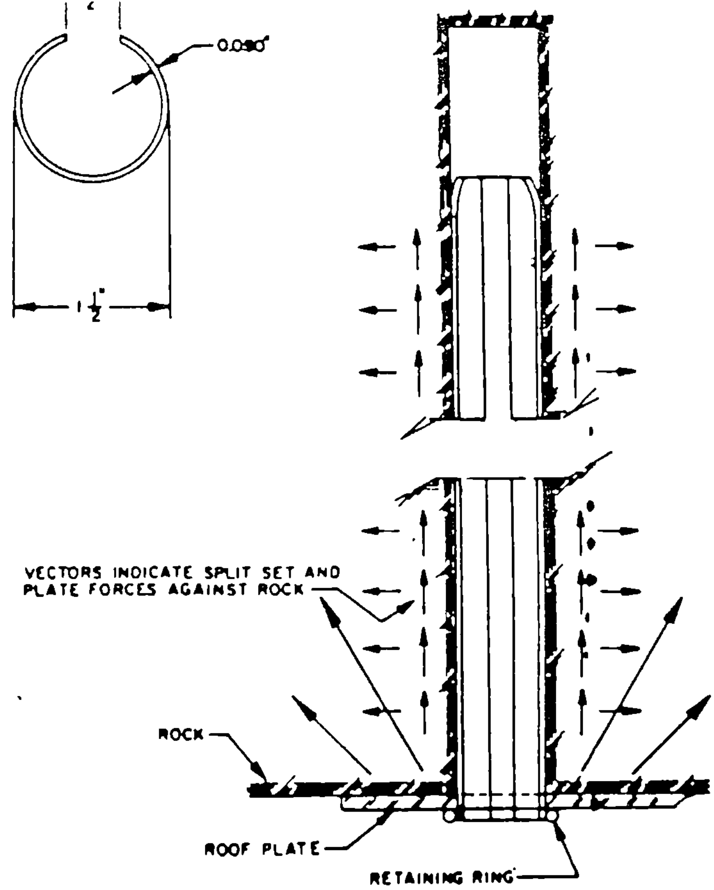


Figure 14.3 Split Set bolt.

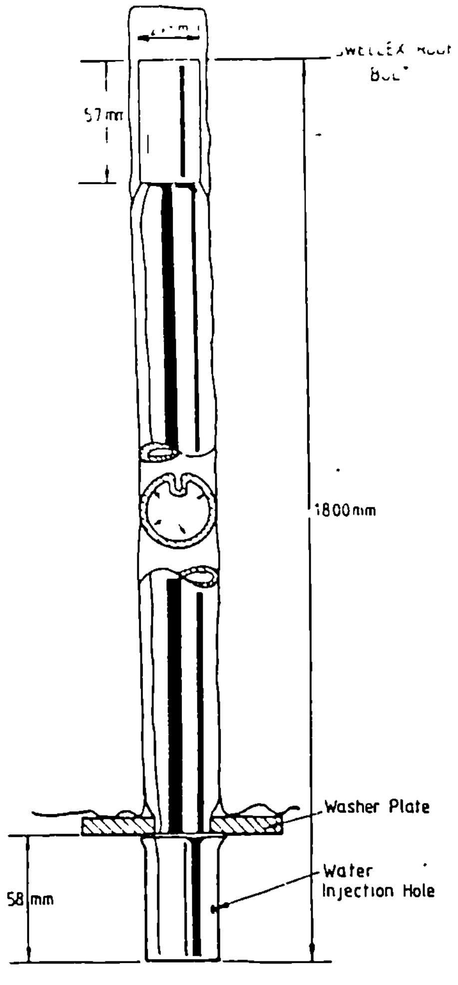


Figure 14.4 Swellex bolt.

14.4 Swellex Rock Bolts

The Swellex rock bolt (also called "Webster Rockfast") has been developed by Atlas Copco. The bolt is manufactured from a steel tube 2 mm thick with a 41 mm outer diameter that is cold rolled and reshaped to form a folded profile 25.5 mm in diameter. Short support sleeves (bushings) are pressed onto the ends and sealed by welding. The bottom bushing is more sturdy and flanged to retain a bearing plate. The bolt is installed in a borehole by injecting water from a high pressure pump (at 20-30 MPa) into a small hole drilled in the lower bushing. The water expands the bolt so that it conforms to the profile of the hole (Figure 14.4). Only 2 L of water are required per 1.8 m bolt length. During the swelling process the bolt shortens along its vertical axis and pulls the bearing plate against the rock surface, exerting an axial load of up to 15 kN. When the water injection chuck is held manually difficulties can be encountered in keeping the plate tight to the rock to obtain pre-tension (Oram 1986). The pump automatically stops at a pre-set pressure and the water drains out of the bolt. During Swellex trials for a face salvage operation at Kellingley Colliery, 38 mm drill bits were found to be producing over sized holes in weak strata at the mouth of the hole. The installed bolt ends appeared to be almost fully expanded which may have resulted in a reduction of the residual pressure between the bolt and the hole wall. Expansion of the bolt end outside the borehole can loosen pieces of rock at the borehole mouth. This can be prevented by fitting a sleeve to the bolt end (Brask and Hamrin 1983). There is a possibility that a high water pressure may initiate cracks along the borehole in rocks of low tensile strength. This is believed to have occured during Swellex field trials at Rufford Colliery (Proctor 1986). Wijk and Skogberg (1982) recommend that the water pressure should be less than four times the uniaxial compressive strength of the rock.

Destructive pull tests on correctly installed Swellex bolts generally result in failure of the bolt at the pull collar. The bolts have an ultimate pull strength of 110-120 kN. This can be obtained with contact lengths of between 0.4 and 1.0 m depending on the rock type and installation conditions. Laboratory tensile testing carried out by Ivanovic and Richmond (1984) found a small sample of Swellex bolts to have a yield load of 97 kN and a mimimum mean failure load of 119 kN.

To determine the pull out resistance of a Swellex bolt, a pipe must be placed on the bolt to reduce the free length to about 0.4 m. To compensate for the creasing effect at the free length ends a correction must be made to determine the effective length:

Effective length (m) = Free Length (m) - 0.1 m

The optimum borehole diameter and setting pressure at each installation site can be determined by a series of simple pull out resistance tests. Tests carried out by Atlas Copco (1982a, 1982b, 1983a, 1983b, 1983c) and independent research bodies (Myrvang and Hanssen 1983; Ivanovic and Richmond 1984; Tadolini 1986) have found that for most types of strata the greatest pull out resistance was achieved by bolts set at 28-30 MPa and in 36-40 mm diameter holes.

The bolt can be adapted to a variety of ground conditions by altering the inflation pressure. At the Mount Isa base metal mine (Australia) the standard setting pressure is 30 MPa. In areas prone to major convergence this is reduced to 24 MPa in order to reduce the support stiffness and consequently improve the yielding characteristics (Morland and Thompson 1985).

Corrosion of the lower bushing prior to installation can create an insufficient sealing between the chuck and the bolt preventing the required setting pressure being achieved. Corroded bushings tend to decrease the life of the O-ring chuck seals (Myrvang 1983; Oram 1986; Schmid 1986). However, seal replacement is a relatively quick and simple operation.

The Swellex bolt has a shear resistance ranging from 75 to 125 kN (Ivanovic and Richmond 1984; Redaelli 1984, 1985) which indicates that these bolts are capable of resisting a certain amount of lateral movement. Laboratory shear tests on Swellex bolts carried out by Ludvig (1983) concluded that the shear resistance is independent of the position of the fold inside the tube with respect to the direction of shearing and that bolts installed in a 38 mm diameter hole perpendicular to a shear surface will undergo 30 mm of displacement before failure. Through comparative shear tests on steel bolts, Ludvig also established that a Swellex bolt has a similar shear strength to a 14 mm diameter massive steel bolt. According to Moore (1983) in areas where lateral movement is not sufficient to deform the bolt, its axial restraint capabilities will be relatively low compared to a fully grouted rock bolt.

14.5 Alternative Rock Bolt Systems Developed By The US Bureau Of Mines

Several alternative rock bolt systems have been developed by the USBM or under USBM awarded contracts. The majority of these systems currently remain in the experimental stages. They include a helical bolt that exhibits both plastic and elastic behaviour (Babcock 1978, 1980); pumpable bolts suitable for use in deviating holes of any length (Habberstad et al 1973; Thompson et al 1974, 1975, 1984; Solomon and Rich 1983; Rich and Solomon 1986) and self drilling rock bolts to eliminate the procedure of extracting the drill rod and inserting the bolt (Engineers International 1979).

CHAPTER 15

ROCK BOLTING ACCESSORIES AND EQUIPMENT

15.1 End Plates, Straps And Roof Bars

A variety of rock bolt end plates are available in a range of strengths, shapes, thicknesses and dimensions. The choice of a particular plate will be determined by the magnitude of the load applied and the nature of the strata immediately surrounding the excavation.

End plates are an essential part of point anchored rock bolt systems to distribute the load from the bolt to the rock surface. The principal function of end plates attached to full column anchored bolts is widely regarded as a means of controlling surface spalling because generated loads are controlled along the bolt axis and interbed slips (Coates & Yu 1970, Haas et al 1974, Sinou and Dejean 1980, BMC 1986, Stillborg 1986). However, Tadolini and Ulrich (1986) have measured the load on end plates attached to untensioned fully grouted bolts that are subjected to large amounts of load, indicating that the plate may be an important part of the support system.

Steel straps, may be installed under bolt end plates to link up adjacent bolts and anchor them together in order to support the immediate strata. They commonly range from 2.5 to 10 mm thick and 100 to 300 mm wide. Holes along the strap act as a template for positioning the rock bolts. Straps are frequently very effective especially in friable strata but are relatively expensive and difficult to install particularly in fully mechanized setting operations. Sinou and Dejean (1980) have reported a coal mine application where the breakage of bolts in shear occurred due to considerable differential movement of bolt heads linked by the same steel strap.

The additional support provided by steel girders bolted to the roof compared with thin mild steel straps has been evaluated through scale model tests. Two models were tested; the first simulating 3 x 280 mm steel straps and the second simulating 65 x 110 mm steel girders. Both models were constructed with the strata configuration shown in Appendix 1a and rock bolt pattern illustrated in Appendix 2d. The model with the bolted roof girders was more effective at controlling roof deformation and could withstand hydrostatically applied pressures of 0.7 MPa without failure; whereas the roof of the model with thin steel straps failed with an inverted V-type fracture pattern between 0.6 and 0.65 MPa and suffered substantial roof lowering as a result (Figure 15.1).

15.2 Lining Materials

Linings prevent the spalling of loose rock fragments and can have the effect of confining rock surrounding an opening thus contributing to the support system. Wire mesh, either in the form of chain link netting or weldmesh sheets, is the most commonly used liner in rock bolted roadways.

A more substantial lining is achieved by the early application of layers of sprayed concrete which not only prevents rock spalling and provides confinement but also protects the rock and the bolts from the humidity of the ventilating air. The use of sprayed concrete in conjunction with rock bolts is particularly common in coal mines in West Germany and the Far East (Noche 1978; Feistkorn 1985; Zischinsky 1987; Lee et al 1987).

There are two processes for placing sprayed concrete, wet and dry. In the wet process all the ingredients are previously mixed together except the accelerator which is added in a liquid state through the nozzel at the point of placement. In the dry process all the ingredients including acceleration in dry form but not water are mixed just before placement by pneumatic projection. Water is introduced into the nozzel immediatly before spraying. The strength, shrinkage and creep properties of sprayed concrete can be improved by the addition of steel fibres to the mix (Poad et al 1975; Ryan 1975; Barfoot 1984; Masson 1985; Rose 1986).

Gotze (1977) maintains that a sprayed concrete surround can only withstand a relative roof movement of 2-3% of the extracted height. Cracks forming in the lining (once this degree of deformation has been exceeded) will give an indication of the need to introduce additional support.

15.3 Drilling And Installation Equipment

A detailed evaluation and assessment of the various types of drilling is outside the scope of this thesis. However, it is important to appreciate the limitations of the different equipment available as it can impose severe constraints on the rock bolt support configuration that can be installed in a roadway. There are four main categories of drilling equipment suitable for roadway rock bolting operations:

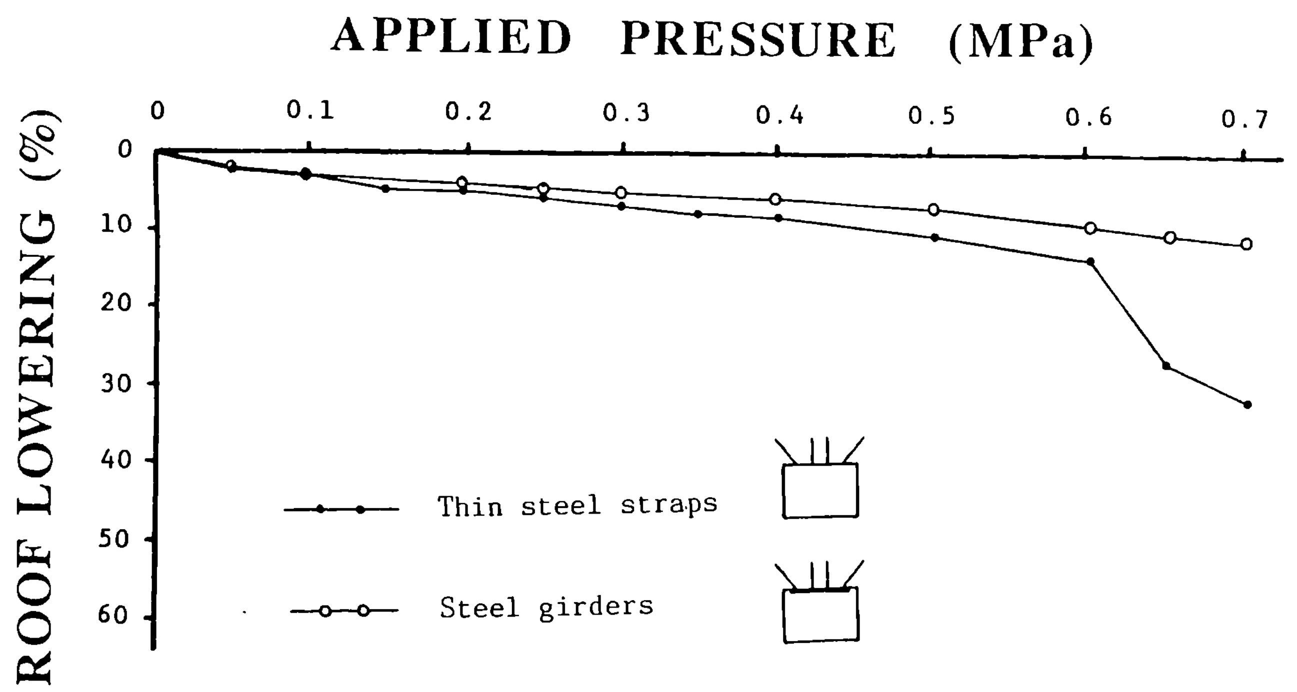


Figure 15.1 Variation in roof lowering as a percentage of initial roadway height with applied pressure - comparison of simulated thin steel straps with steel girders bolted to the roof.

(a) Portable

(b) Heading machine mounted

(c) Roof support mounted

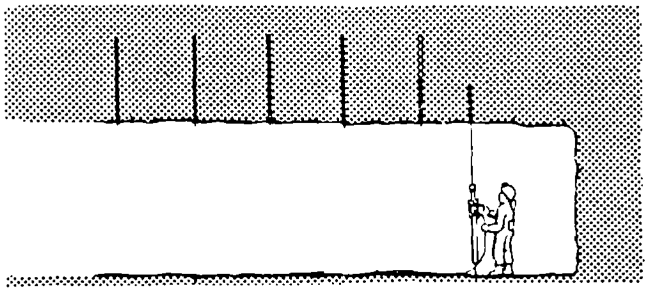
(d) Mobile

Examples of equipment from each of these categories are illustrated in Figure 15.2.

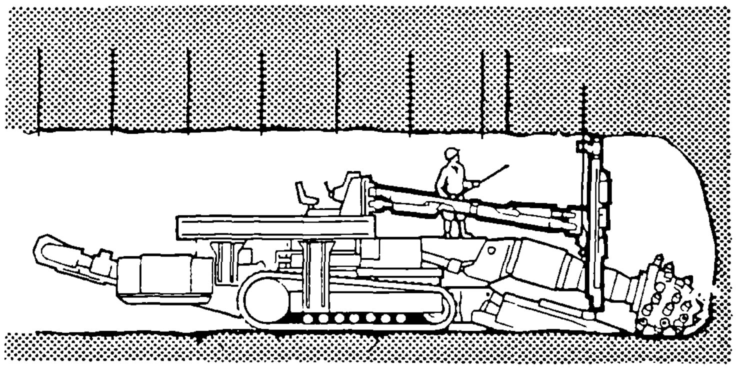
Portable drills are either fixed to a mast of a free standing twin-tyred carriage, mounted on a single telescopic leg or hand held. The free standing type were widely used in the UK during the 1970s and early 1980s. The compressed air leg mounted drills have recently become very popular following their successful use in the Australian mining industry. Portable units are relatively inexpensive and highly manoeuverable (particularly the leg mounted type), although they are generally unsuited for drilling roof over 3 m high as the drills tend to become unstable. The mast type drills often have remotely operated control units but the leg type require the driller to be standing adjacent to the machine during operation. Both types must be manually positioned. Consequently, at sites where bolts are installed at the head end it is difficult for operators to avoid standing under unsupported ground (even if only for a very short period of time). This is obviously hazardous if the roof strata is friable or has a low stand-up time. Hand held drills are generally limited to ribside bolting applications.

Heading machine mounted bolting modules are either fixed to a roadheader boom or in the case of dintheaders and in-seam/continuous miners are located behind the cutting unit. Boom mounted devices are generally capable of drilling most roof bolt patterns at the immediate head end although the arm may be a nuisance during cutting operations. Units mounted at the rear of the cutting machine lack manoeuverability but the use of multiple units will permit the drilling of most rock bolt positions and orientations. These devices cannot be used to install bolts at the immediate head end and are therefore most suited to rapid drivage applications where the roof has been proven to undergo minimal early deflection and bed separation.

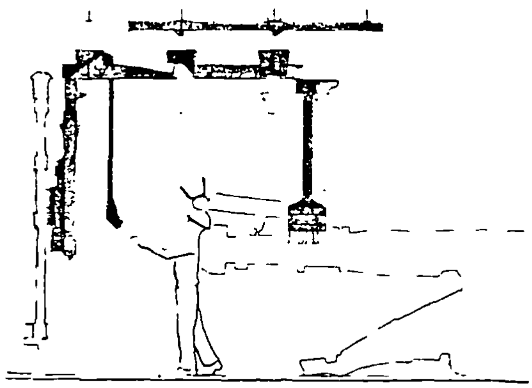
Roof support mounted drills have only recently been developed and consequently little operational experience has been gained. One such device consists of twin drills suspended from a monorail which is fixed to free standing steel supports. This equipment is therefore only



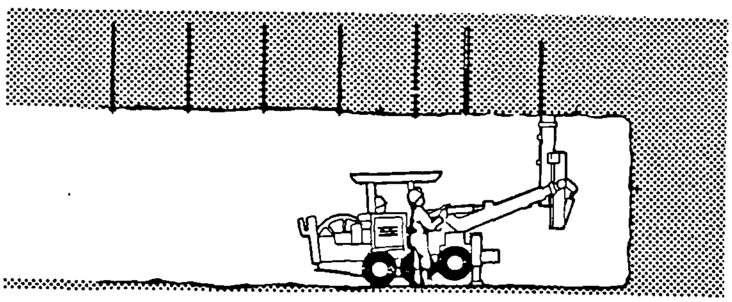
a. Portable



b. Heading machine mounted



c. Roof support mounted



d. Mobile

Figure 15.2 Examples of rock bolt drilling equipment.

suitable for use with dual rock bolt/steel work support systems. Retraction of the rig allows unrestricted access to the heading machine. In addition the operator can stand under erected steel supports while bolts are installed at the immediate head end.

Mobile bolting machines range in size from small compact units to large, expensive "drilling jumbos". In relatively wide roadways the smaller units can pass in front of the heading machine. Applications of large drills are generally limited to bore-and-fire operations, particularly where the rig can be used for both shot and bolt hole drilling.

Full mechanisation of the bolting operation is possible using a turret which has all the tools required for the complete bolting cycle on a single rotary assembly. The operator remains under supported ground at all times and does not come into close contact with grouting materials. Rapid high quality installations are frequently possible with this type of equipment.

Clearly no one type of drill is suitable for all applications and there is still a considerable amount of development required to improve equipment performance and reliability. Additional research should be undertaken to determine the design of drill bits most suited to each drilling rig to give optimum performance in different strata.

CHAPTER 16

GENERAL COMMENTS, CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

The optimum rock bolt support configuration for a coal mine roadway depends upon the relationship between geological conditions, in situ stress and excavation dimensions. Detailed geotechnical investigation and monitoring of specific rock bolt installation sites, in conjunction with scale model studies, will help to establish the precise nature of these complex relationships and consequently lead to the development of safer and more economical roadway support systems.

A detailed site investigation is essential when designing a rock bolt support configuration for an underground opening. A geotechnical assessment of a potential rock bolt installation site must be undertaken by experienced personnel who are familiar with all the types of geological anomalies and discontinuity configurations likely to effect the stability of a mine roadway in coal measures strata.

Preliminary investigations in the Deep Hard/Piper Seam to the east of Mansfield have shown that the position of parting planes and the presence of water, faulting and adjacent/superjacent workings can cause roadway stability problems. Provided adequate instrumentation is installed during mining operations and geotechnical mapping is continued, these potential hazards may be located, assessed and the appropriate support installed.

Many of the elements that constitute the New Austrian Tunnelling Method philosophy are very applicable to coal mine roadway drivage and support. Measurement of strata deformation and support system loadings will provide valuable information on the effectiveness of the rock bolt support system employed. This data can then be used to determine the optimum support and excavation parameters.

Established empirical and analytical design methods should only be used to obtain general guidelines concerning rock bolt parameters and should never be used in isolation. Many of these design methods are simplistic and frequently do not consider critical factors effecting the stability of mine roadways (e.g. in situ stress).

The British Coal HQTD roadway model rig has proved to be a valuable tool to assist in the design of rock bolted mine roadways. The modelling technique, originally developed 25 years ago, has been improved so that it is now possible to obtain semi-quantitative as well as qualitative information concerning the factors influencing the closure of specific underground roadways.

Qualitative scale model studies simulating support systems in a 4.75 m wide, 2.54 m high, rectangular roadway with moderately strong laminated roof strata, indicate that:

(a) Stress concentrations that develop in strata surrounding a roadway are influenced by the magnitude and orientation of the in situ stress field which will determine the mode of strata failure.

(b) Commonly used roof bolt support systems are significantly better at maintaining roof stability than steel standing support under high horizontal and hydrostatic stress fields. Differences in the capacity of the two support systems is not so marked in a high vertical stress field or where the roof strata is relatively weak.

(c) The position and inclination of roof bolts is a very important factor influencing the critical load that a roadway roof can withstand before the onset of failure.

(d) The practice of inclining the outer bolts in a pattern over the roadway ribsides is probably not worthwhile in areas affected by anisotropic stress fields with a high lateral component and in hydrostatic stress fields of low magnitude.

(e) The stability of roadways driven in strata with a high hydrostatic in situ stress field or roadways subjected to a stress field with a high vertical component can be significantly improved by inclining the shoulder roof bolts over the roadway ribsides.

(f) Where geological conditions are favourable and the magnitude of the in situ stress field is low or moderate, relatively short bolts may be capable of providing adequate roof support.

Laboratory scale model studies and field investigations both indicate that roof and floor bolting are more effective at increasing the stability of a roadway driven in moderately strong strata than in weaker rock (particularly when subjected to high loading conditions).

To gain the maximum benefit from rock bolt support systems the bolts should be installed prior to the commencement of significant roadway deformation.

The use of roof bolts in addition to steel standing supports has been shown to bring about reductions in roadway support costs by permitting the use of smaller section RSJs and an increase in the spacing between the steel work.

There is currently a trend to increase the number of retreat faces in British coal mines. Rock bolting is particularly suited to the support of retreat drivages and in some circumstances could be capable of acting as the sole means of support prior to face retreat. Partial extraction operations are generally not subjected to severe mining induced stresses, consequently a number of such sites could possibly also be supported by rock bolt systems alone.

Attention must be paid to safety at all times. Personnel involved in the installation of rock bolts must receive adequate training and supervision to ensure effective strata control. Poor quality installation or insufficient monitoring of rock bolt support systems could lead to catastrophic failure of large sections of mine roadways.

The widespread introduction of rock bolting in British coal mines could possibly result in a decrease in the number of dangerous incidents recorded through a reduction in the number of accidents caused by the transport and setting of heavy steel supports.

The mechanisms of rock bolt reinforcement appear to be relatively complex, affecting the deformation characteristics of the entire underground excavation. A number of roadway monitoring investigations have concluded that the introduction of roof bolting in a roadway can result in a reduction in floor heave and conversely floor bolting can bring about a reduction in roof lowering. Rock bolting systems may therefore be capable of initiating the redistribution of unfavourable stress concentrations.

Full column resin grouted rock bolts are suited to a wide variety of strata and mining conditions. However, other types of rock bolt reinforcement systems have their special applications.

Inorganic grouts can provide a viable low cost alternative to polyester resin, especially where high or variable temperatures could effect the setting characteristics of resin grouts.

Angle bolt trusses have been shown to be very effective in the support of unstable roof strata in some United States mines and may be equally effective in the support of some British coal mine roadways, particularly in low stress conditions (e.g. shallow partial extraction workings). The use of truss bolts may also permit the adoption of a rectangular roadway profile in certain cases where previously an arch shape has been essential.

Point anchored rock bolts are commonly used in the USA, although they have limited applications in the UK due to the generally weaker roof rock and higher in situ stress conditions. In circumstances where the use of point anchored bolts are viable (e.g. for the suspension of a weak layer from an overlying competent bed), point anchored grouted expansion shell rock bolts can provide a relatively high capacity anchorage.

End plates of adequate strength are essential for the operation of point anchored rock bolts and also form an important part of fully grouted rock bolt support systems. Bolting steel straps or girders to the roadway roof may improve the capacity of certain rock bolt support systems as well as act as a template for the bolt pattern. Lining materials are also capable of improving rock bolt support systems, particularly in friable strata.

Further research is recommended in the following areas:

(a) Accumulation of information from monitored rock bolted sites to form a data base which can then be used to develop the basis of an empirical design approach for rock bolted British coal mine roadways.

(b) Studies of the influence of geological anomalies, structures and strata weatherability on the effectiveness of rock bolt support systems.

(c) Accurate measurement of the orientation and magnitude of stress components in British coal mines and the development of a simple means of identifying the stress state in a mine roadway.

(d) Studies of mine drifts or other major drivages with excavation and support procedures based on the New Austrian Tunnelling Method.

(e) Development of an accurate and reliable multi-point rock bolt load measuring device to assist in the design of rock bolt configurations at specific mine sites.

(f) Mine studies to determine whether any benefits will gained from the development of rock bolt supported multiple entry drivages in British mining conditions.

(g) Continuation of scale model studies to determine the effect of different rock bolt support configurations on the stability of various types of mine roadway in a range of strata and stress conditions.

(h) Mine studies to establish under which circumstances the practice of inclining bolts over the roadway ribsides is beneficial and to verify scale model observations (detailed above) concerning inclined bolting.

(i) Mine studies to evaluate circumstances where it is beneficial to use 19 or 20 mm rather than 25 mm diameter rebar.

(j) Laboratory and field point anchorage pull tests to evaluate the most suitable grout for specific mining conditions (i.e. in different strata types, at a range of temperatures and water in-flow rates etc).

(k) Detailed mine studies in a variety of underground conditions to determine under which circumstances pre-tensioning of full column resin grouted rock bolts can be achieved and if it has a significant effect on roadway stability.

(l) Mine studies to evaluate the effect of forms of extended ground support on the stability of roadways where strata deformation extends some distance into the surrounding rock mass.

(m) Mine studies to determine circumstances where the use of Swellex bolts, Split Sets, trusses, yielding bolts and point anchored bolts might be beneficial compared with full column grouted bolts.

(n) Mine studies to determine the most effective type of strapping and lining materials used in conjunction with rock bolt support systems in a variety of roadway conditions.

(o) Studies of methods for improving bolt installation rates, ensuring that a high degree of safety is maintained at all times.

(p) Studies to determine the design of drill bits most suited to each type of drilling rig to give optimum performance in different strata.

ACKNOWLEDGEMENTS

The author would like to thank British Coal for sponsoring this research project. The advice and assistance of Dave Bigby, Alan Bloor, Chris Morris, Brian Mosley, John Sharp and other staff from British Coal HQTD Rock Mechanics Branch is gratefully acknowledged.

Sincere thanks are also due to Clarke Mangelsdorf (University of Pittsburgh), Guy McDowell (Peabody Coal Co.), Zeke Serbousek (USBM), Ray Stateham (USBM) and Nevil Tomlin (University of Newcastle upon Tyne).

REFERENCES

Adams, C.I. 1987. A study of roof bolting in the USA. Unpublished report. 47pp.

Adams, G.R. & Jager, A.J. 1980. Petroscopic observations of rock fracturing ahead of stope faces in deep-level gold mines. *J. South Afr. IMM.* V.8, No.6, p.204-209.

Adcock, W.J. 1955. Roof bolting. Its application in the East Midlands Division. *Iron & Coal Trades Rev.* p.425-433.

Adcock, W.J. & Wright, A. 1957/58. Further progress with strata bolting. *Trans. IME,* V.117, p.185-206.

Adcock, W.J. 1959. Strata control and support development - a survey of mining engineering. *Industrial Newspapers Ltd*, London.

Agarwal, R.K. & Boshkov, S. 1970. Theory and practice gap closes in rock mechanics. *Min. Engng,* V.22, No.2, p.74-76.

Airey, E.M. 1974. The derivation and numerical solution of equations relating to stresses around mining roadways. Ph.D. Thesis, University of Surrey.

Albers, H.J., Gallhoff, U. & Jagsch, D. 1982. The effect of workings on the New Austrian Tunnelling Method. *Gluckauf*, V.118, No.16, Eng. Transl. p.327-331.

Albers, H.J. 1985. Conventional supports and the New Austrian Tunnelling Method. *Gluckauf*, V.121, No.11, p.833-839. Eng. Transl. p.250-252.

Altounyan, P.F.R. 1986. Single point rock bolt extensometer user guide. British Coal HQTD Rock Mech. Branch, 8pp.

Altounyan, P.F.R. 1987. Pillar size analysis for A Block Allerton Bywater Colliery. British Coal HQTD report, 6pp.

Anderson, J.G.C., Arthur, L.J. & Powell, D.B. 1977. The engineering geology of the Dinorwic underground complex. *Conf. on Rock Engng*, Newcastle Upon Tyne, p.491-510.

Anon. 1969. Roof trusses on trial at Crown Mine. *Coal Min. & Proc.* V.6, No.6, p.64-65.

Anon. 1984. Investigating in Sweden's zinc mine. *World Min. Equip.* Jan.

Armstrong, T.L. 1976. Scale model studies of strata movement around mine roadways: Resin dowel reinforcement of gateroad and pack floors. Unpublished notes & test results.

Arnall-Wickman. 1986. Publicity Literature.

Atlas Copco. 1982a. Monitoring rock movement. Swellex Tech. Bull. No.9, 4pp.

Atlas Copco. 1982b. Test summary - Canada. Swellex Tech. Bull. No.3, 3pp.

Atlas Copco. 1982c. Test summary - USA. Swellex Tech. Bull. No.2, 2pp.

Atlas Copco. 1983a. Test summary - Sweden. Swellex Tech. Bull. No.10, 1pp.

Atlas Copco. 1983b. Swellex pull tests at Udden Mine in Sweden. Swellex Tech. Bull. No.20, 2pp.

Atlas Copco. 1983c. Operator's instructions for Swellex pull test equipment. Company Literature. 4pp.

Atlas Copco. 1985a. Underground rock stabilization with longer-than-tunnel-height Swellex bolts. Publicity Literature. 1pp.

Atlas Copco. 1985b. Swellex in new applications. Swellex Tech. Bull. No.40, 5pp.

Atrott, G. 1972. The application of the New Austrian tunnel construction method to the construction of the underground railway in Frankfurt/Main. *Baumasche & Bautechnik*, No.2, p.65-71.

Aughenbaugh, N.B. & Bruzewski, R.F. 1976. Humidity effects on coal mine roof stability. USBM OFR 5-78, 164pp.

Auriol, P. 1972. Roadway planning and maintenance and layout of workings. *Proc. 5th Int. Strata Control Conf.* 9pp.

Azuar, J. & Panet, M. 1980. Shear behaviour of passive steel in rock masses. *Rock Bolting*. Ed de la revue Industri Minerale, p.85-90.

Babcock, C.O. 1977. Sensitive vibrating wire rock bolt load gage. *Trans. AIME*, V.262, No.4, p.293-299.

Babcock, C.O. 1978. Flexible helical rock bolt. USBM RI 8300r, 26pp.

Babcock, C.O. 1980. Plastic-elastic helical roof bolt. *Proc. 21st US Symp. on Rock Mech.* p.769-779.

Babenderde, S. 1980. Application of the New Austrian Tunnelling Method for metro construction in the Federal Republic of Germany. *Eurotunnel '80*, IMM, London p.54-58.

Bains, A.S. 1978. Experience with floor reinforcements at Birch Coppice Colliery. *Min. Eng.* V.137, No.201, p.515-526.

Barfoot, J. 1984. Sprayed fibrous concrete for tunnel support. *Tunn. & Tunnlg*, July, p.58-59.

Barish, K. 1985a. Mine controls bad roof with trusses bolted on cycle. *Coal Age*, V.90, No.5, p.62-66.

Barish, K. 1985b. Truss bolting on cycle in Jane Mine Lower Freeport Seam. *Proc. 4th Conf. on Ground Control in Min.* p.1-10.

Barns, E.L. 1971. Rock bolt testing. *Proc. Symp. on Rock Bolting*, Wollongong, 9pp.

Barratt, D.R. & Altounyan, P.F.R. 1980. The use of strata reinforcement techniques in underground roadways as a means of reducing the density of steel arches. NCB MRDE Tech. Memo. TU(80)1.

Barratt, D.R. 1980. Proposals for roof bolting trials at Bullcliffe Wood Colliery, Barnsley Area. NCB MRDE Tunnelling Branch report, 24pp.

Barratt, D.R. 1981. Case for planned roof bolting. *Colliery Guard. Coal Int.* V.229, No.3, p.9-13.

Barry, A.J., Panek, L.A. & McCormick, J.A. 1953. Use of torque wrench to determine load on roof bolts. Pt.1 Slotted-type bolts. USBM RI 4967, 7pp.

Barry, A.J., Panek, L.A. & McCormick, J.A. 1954a. Use of torque wrench to determine load on roof bolts. Pt.2 Expansion-type bolts. USBM RI 5080, 17pp.

Barry, A.J., Panek, L.A. & McCormick, J.A. 1954b. Anchorage testing of mine roof bolts. Pt.1 Slotted-type bolts. USBM RI 5040, 12pp.

Barry, A.J., Panek, L.A. & McCormick, J.A. 1956. Anchorage testing of mine roof bolts. Pt.2 Expansion-type bolts. USBM RI 5194, 19pp.

Barton, N., Lien, R. & Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mech.* V.6, No.4. p.189-236.

Barton, N. 1976. Recent experiences with the Q-system of tunnel support design. *Proc. Symp. Exploration for Rock Engng*, Johannesburg, V.1, p.107-117.

Barton, N. & Bakhtar, K. 1983. Bolt design based on shear strength. *Proc. Int. Symp. Rock Bolting*, Abisko, p.367-376.

Bauer, E.R. 1985. Ground control instrumentation. USBM IC 9053, 68pp.

Baur, J. & Brune, F. 1984. The stretch-bolt: a new development in strata bolting. *Gluckauf*, V.120, No.13, p.823-825. Eng. Transl. p.228-230.

Beadnall, D.M. 1978. A summary report on the truss bolting undertaken in the United Kingdom 1972 to 1978. Perard Torque Tension report.

Bellier, J. & Debreuille, P. 1977. Three new instruments for measurements in tunnels. *Proc. Int. Symp. Field Measurements in Rock Mech.* V.1, p.351-360.

Bello, A. & Serano, F. 1974. Measurements of the behaviour of grouted bolts used as reinforcing elements for the support of underground openings. *Proc. 3rd Cong. ISRM*, V.2B, p.1189-1193.

Bergman, S.G.A., Krauland, N., Martna, J. & Paganus, T. 1983. Non-destructive field test of cement-grouted bolts with the Boltometer. *Proc. 5th Cong. ISRM*, V.1, p.A177-A181.

Beus, M.J. & Phillips, E.L. 1974. Development of titanium load cells for support load determination. USBM RI 7972, 35pp.

Beveridge, R.L.W. 1974. The resin anchor in strata reinforcement. Paper to Finnish Soc. of Engng Geol.

Bieniawski, Z.T. 1973. Engineering classification of jointed rock. *Trans. S. Afr. Instn Civ. Engrs*, V.15, P.335-344.

Bieniawski, Z.T. 1974. Geomechanics Classification of rock masses and its application in tunnelling. *Proc. 3rd Cong. ISRM*, V.2A, p.27-32.

Bieniawski, Z.T. 1976. Rock mass classification in rock engineering. *Proc. Symp. Exploration for Rock Engng*, Johanesburg, V.1, p.97-106.

Bieniawski, Z.T. 1979. The Geomechanics Classification in rock engineering applications. *Proc. 4th Cong. ISRM*, V.1, p.41-48.

Bieniawski, Z.T., Rafia, F. & Newman, D.A. 1980. Ground control investigations for the assessment of roof conditions. *Proc. 21st US Symp. Rock Mech.* p.691-700.

Bieniawski, Z.T. 1984. *Rock mechanics design in mining and tunneling.* Balkema: Boston. 272pp.

Bieniawski, Z.T. 1987. *Strata control in mineral engineering.* Balkema: Rotterdam. 211pp.

Bikerman, & Mahtab, 1986. *Proc. Min. Latin Amer. Conf.* Santiago.

Bischoff, J.A. & Smart, J.D. 1975. A method of computing a rock reinforcement system which is structurally equivalent to an internal support system. *Proc. 16th US Symp. Rock Mech.* p.175-184.

Bjornfot, F. & Stephansson, O. 1983. Interaction of grouted rock bolts and hard rock masses at variable loading in a test drift of the Kiirunavaara Mine, Sweden. *Proc. Int. Symp. Rock Bolting*, Abisko.

Bjurstrom, S. 1974. Shear strength of hard rock joints reinforced by grouted untensioned bolts. *Proc. 3rd Cong. ISRM*, V.2B, p.1194-1199.

Blevins, C.T. & Dopp, D. 1985. Ground control experiences in a high horizontal stress field at Inland Steel Coal Mine No.2. *Proc. 4th Conf. Ground Control in Min.* p.227-233.

Bloor, A.S. 1980. Scale model studies of strata movement around mine roadways: reproducibility of model strata strengths and effects of pack width and composition on roadway closure. NCB MRDE Internal Report No.80/40, 10pp.

Bloor, A.S. 1985. British Coal HQTD. Personal communication.

Bloor, A.S. 1987. Pillar stress measurement at Allerton Bywater. British Coal HQTD report in preparation.

Bobeck, G.E. & Clifton D.F. 1973. Cause and prevention of failure of freshly exposed shale and shale materials in mine openings. USBM OFR 31-74, 116pp.

Bohnlein, A. 1981. Rock bolting in square section roadways at Ensdorf Colliery. *Gluckauf*, V.117, No.17, p.1101-1103. Eng. Transl. p.496-497.

Boldt, H. & Fritz, W. 1980. Experience with rock bolting as the support method in a gateroad. *Gluckauf*, V.116, No.2, p.51-57. Eng. Transl. p.20-23.

Bollier, C.W. 1982. Hydraulic tensioning of a Birmingham roof truss. *Proc. 2nd Conf. Ground Control in Min.* p.104-107.

Bolstad, D.D., Hill, J.R.M. & Karhnak, J.M. 1983. US Bureau of Mines rock bolting research. *Proc. Int. Symp. Rock Bolting,* Abisko, p.313-320.

Bourbonnais, J. 1985. New developments in rock testing and monitoring equipment for tunnelling projects. *Proc. 5th Canadian Tunnelling Conf.* p.23-26.

Brady, T. 1987. USBM Spokane Research Center. Personal communication.

Brask, C.G. & Hamrin, H. 1983. New type of rock bolt simplifies rock reinforcement procedures. Altas Copco, 17pp.

Breckels, I.M. 1978. A study of the distribution and magnitude of strata loading around mine roadways and tunnels. Ph.D. Thesis, University of Nottingham.

Brest Van Kempen, C.J.H., Maleki, H.N. & Hardy, M.P. 1986. Field tests of uniformly tensioned roof bolts. USBM Final report contract H0222008 158pp.

British Mining Consultants 1986. Guidelines for the application of rock bolt reinforcement for systematic roadway support. Report for British Coal Western Area. 30pp.

BS 4449 1978. Hot rolled steel bars for the reinforcement of concrete. *British Standards Institution.*

Bronder, D.S. 1986. Ingersoll Rand Co. Personal communication.

Brook, N. 1977. Model studies of mine roadway deformation. *Min. Eng.* April, p.375-384.

Brown, E.T. & Hoek E. 1978. Trends in relationships between measured in-situ stresses and depth. *Int. J. Rock Mech. & Min. Sci.* V.15, p.211-215.

Brown, E.T. 1981. Putting the NATM into perspective. *Tunn. & Tunnlg* Nov. p.13-17.

Brown, E.T., Bray, J.W., Landanyi, B. & Hoek, E. 1983. Ground response curves for rock tunnels. *J. Geotech. Engng.* V.109, No.1, p.15-31.

Bullock, M. 1968. Floor bolting in the Doubles Seam at Granville Colliery. *Colliery Guard.* Aug. p.571-573.

Bywater, S. & Fuller, P.G. 1983. Cable bolt support of lead stope hanging walls at Mount Isa Mines Ltd. *Proc. Int. Symp. on Rock Bolting,* Abisko.

Carr, F. 1971. Recent developments in strata bolting in National Coal Board mines in the UK. *Proc. Symp. on Rock Bolting*, Wollongong, 20pp.

Carr, F. 1972. Strata reinforcement. *Proc. 5th Int. Strata Control Conf.* Preprint No.10, 11pp.

Cassidy, K. 1980. Implementation of a cable bolting program at Con Mine. *Proc. 13th Canadian Symp. Rock Mech.* p.67-72.

Celtite Selfix Ltd 1983. Strata reinforcement for the UK mining industry. Publicity Literature, 14pp.

Chaiko, W.M. & Scott, J.J. 1977. Roof support experience with friction rock stabilizers. *Min. Cong. J.* V.63, No.3, p.36-41.

Charlesworth, T.J. & Stokes, H. 1970. Roof support at Whitwell Colliery. Paper presented to Chesterfield and District Mining Society. 28th Nov. 40pp.

Chase, F.E. & Sames, G.P. 1983. Kettlebottoms: their relation to mine roof support. USBM RI 8785, 18pp.

Chase, F.E. 1985. Clay veins: their physical characteristics, prediction and support. *Proc. 4th Conf. on Ground Control in Min.* p.212-219.

Chekan, G.J. & Babich, D.R. 1982. Investigation of longwall gateroad roof support at Powhatan No.4 Mine. USBM RI 8628, 9pp.

Clarke, A.M. 1963. A contribution to the understanding of washouts, swalleys, splits and other seam variations and the amelioration of their effects on mining in South Durham. *Min. Eng.* V.122, p.667-706.

Coates, C.D. 1987. Initial experience in No.7 Seam - Snowdown Colliery. *Min. Eng.* May, p.674-681.

Coates, D.F. & Yu, Y.S. 1970. Three dimensional stress distributions around a cylindrical hole and anchor. *Proc. 2nd Cong. ISRM*, p.175-181.

Coates, D.J., Cater, P.G. & Smith, I.M. 1977. Inclined drilling for the Kielder Tunnels. *Q. J. Eng. Geol.* V.10, p.195-205.

Commercial Plastics Special Products Ltd. 1981. Publicity Literature, 6pp.

Conway, J.P., Dar, S.M. & Stears, J.H. 1975. Laboratory studies of yielding rock bolts. USBM IC 8058.

Conway, J.P., Cox, D.J. & Gooch, A.E. 1977. Yielding rock bolt holds promise for future ground control applications. *Min. Engng*, V.29, No.4, p.53-55.

Coulthard, M.A., Crotty, J.M. & Fabjanczyk, M.W. 1983. Comparison of field measurements and numerical analysis of a major fault in a mine pillar. *Proc. 5th Cong. ISRM*, p.D53-D60.

Cox, R.M. 1974. Why some bolted mine roofs fail. *Trans. SME-AIME*, V.256, June, p.167-171.

Cox, R.M. & White, C.C. 1977. Design and application of mine roof truss system. *Proc. 6th Int. Strata Control Conf.*

Croizat, G., Lauret, G. & Meyer, J.L. 1982. Split Set friction rock stabilizer and its use in French mining operations. *Industrie Minerale*, V.64, No.12, p.653-662.

Culver, R.S. & Jorstad, T. 1967. Fracturing around a rock bolt anchor. *Proc. 9th US Symp. Rock Mech.* p.222-234.

Cummings, R.A., Singh, M.M., Sharp, S.E. & Laurito, A.W. 1981. Control of shale roof deterioration with air tempering. Vol.1: Field and laboratory investigations. USBM OFR 41(1)-82, 164pp. Vol.2: Annotated bibliography. USBM OFR 41(2)-82, 64pp.

Cummings, R.A., Singh, M.M. & Moebs N.N. 1983. Effect of atmospheric moisture on the deterioration of roof shales. *Min. Engng*, V.35, No.3, p.243-245.

Cyrul, T. 1985. A low-cost ring dynamometer for monitoring the performance of rock bolts. *Geotech. Test. J.* V.8, No.1, p.30-36.

Daly, W. & Abramson, L. 1986. Mt Lebanon Tunnel uses NATM American-style. *Tunn. & Tunnlg*, Jan. p.35-38.

Davis, R. 1978. A catalogue of strength properties of some coal measure rocks. NCB MRDE Report 72. 5pp.

Davis, R. 1981. A catalogue of strength properties of some coal measure rocks. NCB MRDE Report 98. 5pp.

Davis, R.L. 1979. Split Set rock bolt analysis. *Int. J. Rock Mech. Min. Sci.* V.16, No.1, p.1-10.

Daws, G. 1975. A study of the stability of mine roadways driven in solid ground. Ph.D. Thesis, University of Nottingham, 122pp.

Daws, G. 1977. A method of designing a rock bolting scheme which is structurally equivalent to a standing support system. Celtite Selfix Ltd publication.

Daws, G. 1978. Resin anchors. *Civil Engng*, Oct. p.71-75 & Dec. p61-63.

Daws, G. 1980a. Calculations for rock bolting scheme - Bullcliffe Wood Colliery, Celtite (Selfix) Ltd., Report to N.C.B.

Daws, G. 1980b. The design and installation of rock anchors in granite gneiss for foundation stabilization. *Proc. Int. Conf. on Structural Foundations on Rock*, Sydney, V.1, p.105-108.

Daws, G. 1986a. The design of rock bolting systems for the coal mining industry. *Proc. Int. Symp. Underground Min. Sci & Tech.*, Nottingham.

Daws, G. 1986b. Coal mine roof bolting - the state of the art. *Min. Methods '86 Symp.*, Harrogate, p.279-286.

Deere, D.U. 1964. Technical description of rock cores for engineering purposes. *Rock Mech. & Engng Geol.*, V.1, No.1, p.17-22.

Deere, D.U., Peck, R.B., Parker, H.W., Monsees, J.F. & Schmidt, B. 1970. Design of tunnel support systems. *Highway Res. Rec.* No.339, p.26-33.

Dejean, M. & Raffoux, J.F. 1976. Mining drifts and tunnels: role of rock bolting and parameters in its selection. *Tunnelling '76 Conf.*, London, p.321-327.

Dejean, M. & Raffoux, J.F. 1980a. The determining of roof bolting parameters. *Rock Bolting*. Ed de la revue Industri Minerale, p.141-152.

Dejean, M. & Raffoux, J.F. 1980b. Monitoring of rock bolt effectiveness. *Rock Bolting*. Ed de la revue Industri Minerale, p.153-164.

Dejean, M. & Raffoux, J.F. 1983. Rock bolting in France - a keynote lecture. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.269-284.

Dhar, B.B., Ratan, S., Reddi, H. & Mehta, R. 1983. Laboratory study for the design of optimum rock bolting in bedded strata. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.219-223.

Douglas, T.H. & Arthur, L.J. 1983. Guide to the use of rock reinforcement in underground excavations. CIRIA Report N101, 74pp.

Dunham, R.K. 1973. Some aspects of resin anchored rock bolting. *Tunn. & Tunnlg*, V.5, July.

Dunham, R.K. 1974. Field testing of resin anchored rock bolts. *Colliery Guard.*, V.222, No.5, p.146-151.

Egger, P. & Gindroz, C. 1979. Anchored tunnels with small overburden. Comparative study on physical and mathematical models. *Proc. 4th Cong. ISRM*, V.2, p.121-130.

Ellenberger, J.L. 1979. Slickenslide occurrence in coal mine roof of the Valley Camp No.3 Mine near Wheeling, W.VA. USBM RI 8365, 17pp.

Elliott, R.E. 1965. Swilleys in the coal measures of Nottinghamshire interpreted as palaeo-river courses. *Mercian Geologist*, V.1, p.133-142.

Engineers International Inc. 1979. Development and testing of self-drilling roof bolts. USBM Report: Contract HO272022, 225pp.

European Coal & Steel Community (ECSC) 1980. Analysis of factors affecting the method, shape and reliability, and extent of mine roadways in relation to their duty. Report on Research Project 6220-AB/8/806, 13pp.

Evans, W.H. 1960. Roof bolting and stabilization of natural arches on roadways. *Colliery Engng*.

Everling, G. 1982. Models for strata control and support dimensioning: successes and present achievements. *Gluckauf*, V.118, No.1, p.16-23. Eng. Transl. p.9-12.

Ewan, V.J., West, G. & Temporal, J. 1983. Variation in measuring rock joints for tunnelling. *Tunn. & Tunnlg*, April, p.15-18.

Fairhurst, C. & Singh, B. 1974. Roof bolting in horizontally laminated rock. *Engng. Min. J.*, V.175, No.2, p.80-90.

Farmer, I.W. 1975. Stress distribution along a resin grouted rock anchor. *Int. J. Rock Mech. Min. Sci.*, V.12, p.347-351.

Farmer, I.W. & Shelton, P.D. 1978. Rock bolt support of underground structures - design and performance studies. *Proc. Conf. on Geotech. Engng.*, New Delhi.

Farmer, I.W. & Shelton, P.D. 1980. Factors that affect underground rock bolt reinforcement systems design. *Trans. IMM.*, V.89, April, p.A68-A83.

Feistkorm, E. 1985. State of the art in shotcreting. *Gluckauf,* V.121, No.2, p.126-133. Eng. Transl. p.48-51. -

Finch, M.E. 1987. Assessment of "PC Bolting": a rock bolting design program from Cerchar. British Coal HQTD Rock Mech. Branch Tech. Memo. No.(87)4.

Fish, E.L., Turnbull, L.A. & Toenges, A.L. 1944. A study of summer air conditioning with water sprays to prevent roof falls at the Beach Bottom Coal Mine, West Virginia. USBM RI 3775, 20pp.

Fitzsimmons, J.R., Stateham, R.M. & Radcliffe, D.E. 1979. Flexible, fibreoptic stratascope for mining applications. USBM RI 8345, 12pp.

Fraley, J.E. 1984. Inorganic grouts for roof bolting. USBM IC 8973, p.138-145.

Fraley, J.E. & Serbousek, M.O. 1987. Bolt anchorage with gypsum-plaster water capsule cartridges. USBM RI 9067, 23pp.

Franklin, J.A. & Woodfield, P.F. 1971. Comparison of a polyester resin and mechanical rock bolt anchor. *Trans. IMM*, July, p.A91-A100.

Frazer & Jones 1987. Mine roof supports anchors. Publicity Literature, 6pp.

Freeman, T.J. 1978. Behaviour of fully bonded rock bolts in the Kielder experimental tunnel. *Tunn. & Tunnlg,* V.10, No.5, p.37-40.

Gale, W.J. & Fabjanczyk, M.W. 1985. In situ measurement to determine roof stability and bolt performance and its implication to the stability of mine roadways. *Proc 21st Int. Symp. on Safety in Mines Research Institutes.*

Gale, W.J. 1986. Design considerations for reinforcement of coal mine roadways in the Illawarra Coal Measures. *Proc. Symp. on Ground Movement & Control Related to Coal Min.* AusIMM, p.82-92.

Gale, W.J. & Fabjanczyk, M.W. 1986. Application of field measurement techniques to the design of roof reinforcement systems for underground coal mines. *Proc. 13th Cong. of Council IMM*, Singapore, p.135-141.

Gale, W.J. 1987. Australian Research Industry Research Laboratories. Personal communication.

Gale, W.J. & Blackwood, R.L. 1987. Stress distributions and rock failure around coal mine roadways. *Int J. Rock Mech. Min. Sci.* V.24, No.3, p.165-173.

Geodynamik AB 1986. Publicity Literature.

Geological Society Engineering Group. 1977. The description of rock masses for engineering purposes. *Q. J. Engng Geol.*, V.10, No.4, p.355-388.

Georgel, P. & Raffoux, J.F. 1968. A trial with complete roof bolting in a gateroad in an advancing face. *Charbonnage de France, Doc Techn.* No.4, p.153-159. NCB Transl. A2718.

Gerdeen, J.C., Snyder, V.W. & Viegelahn, G.L. 1977. Design criteria for roof bolting plans using fully resin-grouted nontensioned bolts to reinforce bedded mine roof. USBM OFR 46-80, 713pp.

Golder Associates 1986. Field tests for in situ stress determination by hydraulic fracturing method. Pt 1: Back Lane Borehole. Report for DOE Contract PECD 7/9/299.

Goodman, R.E., Heuze, F.E. & Bureau G.J. 1972. Modelling techniques for the study of tunnels in jointed rock. *Proc. 14th US Symp. Rock Mech.* p.441-479.

Goris, J.M. & Conway, J. 1987. Grouted flexible tendons and scaling investigations. USBM Spokane Research Center, unpublished report.

Gotze, W. 1977. Roof bolting in arch shaped roadways. *Gluckauf*, V.133, No.15, p.753-755. NCB MRDE Transl. M25565, No.1386.

Gotze, W. 1981. Planning of rock bolt support in in-seam roads and gateroads. *Gluckauf*, V.117, No.17, p.1107-1111. Eng. Transl. p.499-501.

Gotze, W., Stephan, P. & Wiegand, H.A. 1982. Practical limits, scope and future development of rock bolting. *Gluckauf*, V.118, No.21, p.1083-1091. Eng. Transl. p.425-429.

Gotze, W. 1986. Bolting at great depths. *Gluckauf*, V.122, No.2 p.123-128. Eng. Transl. p.41-43.

Gouilloux, C. & Piguet, J.P. 1977. Ground reinforcement by bolting and grouting. *Proc. 6th Int. Strata Control Conf.*, Paper 18.

Gray, R. 1968. Stage II trials on Roc-Loc floor bolting at Birch Coppice Colliery. NCB Central Engineering Establishment Report No.FT(68)4, 6pp.

Grotowsky, U. 1977. Recent developments in roadway support. *Proc. Int. Strata Control Conf.*

Grotowsky, U. 1981. Progress in the field of strata control. *Gluckauf*, V.117, No.17, p.1111-1114. Eng. Transl. p.501-503.

Guccione, E. 1978. Conkle's warning monitor: the miners guardian angel. *Coal Min. & Proc.* Sept.

Guo, L.B. & Peng, S.S. 1984. Boundary element method of analyzing the interaction between roof strata and roof bolts

Gutberlet, W. 1987. Operational experience with bolting at Niederberg Colliery. *Gluckauf*, V.123, No.9, p.534-538. Eng. Transl. p260-261.

Haas, C.J., Clarke, G.B. & Nitzsche, R.N. 1974. An investigation of the interaction of rock and types of rock bolts for selected loading conditions. USBM OFR 2-77, 342pp.

Haas, C.J. 1976. Shear resistance of rock bolts. *Trans. SME-AIME*, V.260, No.1, p.32-40.

Haas, C.J., Davis, R.L., Keith, H.D. & Patrick, W.C. 1978. An investigation of the interaction of rock and types of rock bolts for selected loading conditions. USBM OFR 29-79, 287pp.

Haas, C.J. 1981. Analysis of rock bolting to prevent shear movement in fractured ground. *Min. Engng*, V.33, No.6, p.698-704.

Habberstad, J., Waide, C. & Simpson, R. 1973. The pumpable rock bolt. A new roof control concept. *Engng Min. J.*, V.174, No.8, p.76-79.

Hansagi, I. 1965. Numerical determination of the mechanical properties of rock and rock masses. *Int. J. Rock Mech. Min. Sci.*, V.2, p.219-223.

Hansagi, I. 1974. A method of determining the degree of fissuration of rock. *Int. J. Rock Mech. Min. Sci.*, V.11, p.379-388.

Hansen, J.S. & Gerdemann S.J. 1985. Pull and creep tests on gypsum-bonded roof bolts. USBM RI 8937, 36pp.

Hartman, I. & Greenwald, H.P. 1941. Effects of changes in moisture and temperature on mine roof.1. Report on strata overlying the Pittsburgh Coal Bed USBM RI 3588, 40pp.

Haynes, C.D. 1975. Effects of temperature and humidity variations on the stability of coal mine roof rocks. USBM OFR 8-77 385pp.

Herget, G. 1982. TV borehole inspection and testing of roof strata. *Proc. 2nd Conf. on Ground Control in Min.*, p.209-213.

Hill, J.L. 1986. Cutter roof failure: an overview of the causes and methods for control. USBM IC 9094, 27pp.

Hind, J.G. 1959/1960. Some experiences in roadway support. *Min. Eng.* V.119, p.613-626.

Hirst, P. 1987. British Coal HQTD. Personal communication.

Hobbs, D.W. 1965. Scale model studies of strata movement around mine roadways: apparatus, technique and some preliminary results. NCB MRDE Report No.2276 (Int. J. Rock Mech. Min. Sci., 1965 V.3, p.101-127).

Hobbs, D.W. 1966. Scale model studies of strata movement around mine roadways: the effect of slotting the rib. NCB MRDE Report No.2297 (*Int. J. Rock Mech. Min. Sci.*, 1968 V.5, p.245-251).

Hobbs, D.W. 1967a. Scale model studies of strata movement around mine roadways: the dependence of roadway closure upon rock strength. NCB MRDE Report No.2311 (*Int. J. Rock Mech. Min. Sci.*, 1968 V.5, p.219-235).

Hobbs, D.W. 1967b. Scale model studies of strata movement around mine roadways: the effect of dinting. NCB MRDE Report No.2312 (*Int. J. Rock Mech. Min. Sci.*, 1968 V.5, p.237-244).

Hobbs, D.W. 1968a. Scale model studies of strata movement around mine roadways: roadway shape and size. NCB MRDE Report No.2325 (*Int. J. Rock Mech. Min. Sci.*, 1968 V.6, p.365-404).

Hobbs, D.W. 1968b. Scale model studies of strata movement around mine roadways: circular roadways. NCB MRDE Report No.2326.

Hobbs, D.W. 1968c. Scale model studies of strata movement around mine roadways: quality of roadway profile, support spacing and pack material. NCB MRDE Report No.2336 (*Int. J. Rock Mech. Min. Sci.*, 1969 V.6, p.405-414).

Hobbs, D.W. 1968d. Scale model studies of strata movement around mine roadways: ribside support. NCB MRDE Report No.2343 (*Int. J. Rock Mech. Min. Sci.*, 1970 V.7, p.183-192).

Hodgkinsdon, D.R. 1971. A study of mine roadway deformation. Ph.D. Thesis, University of Nottingham, 352pp.

Hodkin, D.L. 1975. An investigation into the support of mine roadways by means of roof trusses at Ellington Colliery. M.Sc. Thesis University of Newcastle upon Tyne.

Hoek, E. & Brown, E.T. 1982. *Underground excavations in rock*. IMM, London.

Hunt, R.E.B. & Askew, J.E. 1977. Installation and design guidelines for cable dowel ground support at ZC/NBHC. *Proc. Underground Oper. Conf.* Aus.IMM, Broken Hill, p.133-122.

Hunter, J.W. 1986. The effect of water on the performance of capsule resins for strata reinforcement. British Coal HQTD Test Report No.MT(86)164, 8pp.

Hylbert, D.K. 1978. The classification, evaluation and projection of coal mine roof rocks in advance of mining. *Min. Engng*, Dec. p.1667-1676.

International Society for Rock Mechanics (ISRM) 1981. *Rock characterization testing and monitoring: Suggested methods*. Ed E.T. Brown. Permagon Press, 211pp.

International Society for Rock Mechanics (ISRM) 1985. Suggested method for rock anchorage testing. *Int. J. Rock Mech. Min. Sci.* V.22, No.2, p.71-84.

International Society for Rock Mechanics (ISRM) 1987. Suggested methods for rock stress determination. *Int. J. Rock Mech. Min. Sci.* V.24, No.1, p.53-73.

Isaac, A.K. & Livesey, D.B. 1975. Strata mechanics in the South Wales Coalfield. *Min. Eng.* Aug/Sept. 1975. p.609-619.

Ivanovic, D. & Richmond, A. 1984. Underground coal mine roof control research: technical evaluation of Swellex roof bolts. ACIRL Report No.84-4, 45pp.

Jacobi, O. 1976. Praxis der geibirgsbeherrschung. *Verlag Gluckauf*.

Jacobi, O. 1981. Praxis der geibirgsbeherrschung. *Verlag Gluckauf*.

Jeffery, R.I. 1986. Rock bolting in M25's development roads. British Coal HQTD Geological Dept. report, 7pp.

Jeremic, M.L. 1980. Rupturing criteria of coal bearing strata, W. Canada. *Modern Geol.*, V.7, p.191-199.

John, M. 1980. Investigation and design for the Arlberg Expressway Tunnel. *Tunn. & Tunnlg,* April p.46-51, May p.54-57, June p.45-50, July p.66-68.

Johnson, G. 1973. Strata reinforcement of the floor of the Parkgate Seam in the gateroads and S.W. drivages in the vicinity of 30's face. NCB MRDE Rock & Coal Mechanics Branch Visit Report No.(73)36, 2pp.

Jones, M.B. 1986. Examining the bond between anchor and rock. *Tunn. & Tunnlg,* V.18, No.11, p.55-58.

Kammer, W. 1977. Die ausbauplanung duch vorausberechnung der endkonvergenz in abbaustrecken. *Gluckauf*, V.113, p.746-748.

Kammer, W. 1980. Calculation of convergence and support systems planning in gateroads by means of an EDP programme. *Gluckauf*, V.116, No.5, p.200-205. Eng. Transl. p.91-94.

Karabin, G.J. & Debevec, W.J. 1976. Comparative evaluation of conventional and resin bolting systems. MSHA IR 1033, 22pp.

Karabin, G.J. & Hoch, M.T. 1979. An operational analysis of point resin anchored bolting systems. MSHA IR 1100, 14pp.

Keck, G. Experiences with strata bolting and roadside packs at Niederberg Colliery. *Gluckauf*, V.117, No.17, p.1104-1107, English Transl. p.498-499.

Kegel, W.G. 1969. Roof truss installations. *Min. Cong. J.*, V.55, No.7, p.22-27.

Kendorski, F.S., Cummings, R.A., Bieniawski, Z.T. & Skinner, E.R. 1983. A rock mass classification scheme for the planning of caving mine drift supports. *Proc. Conf. on Rapid Excavation & Tunnlg*, V.1, p.191-223.

Kennedy-Skipton, H. 1987. Nobel's Explosives Company Ltd. Personal communication.

Khair, A.W. 1983. Physical and analytical modelling of the behaviour of truss bolted mine roofs. *Int. Symp. on Rock Bolting*, Abisko, p.125-142.

Kmetz, W.J. 1970. Roof trusses support problem strata. *Coal Age*, Jan. p.64-68.

Knight, D. 1979. Rock mechanics/geotechnical investigations their application to the mining industry. *Colliery Guard*. March p.162-169, May p.212-220.

Krishna, R. 1974. A study of floor deformation in mine roadways. Ph.D. Thesis, University of Nottingham.

Kwitowski, A.J. & Wade, L.V. 1980. Reinforcement mechanisms of untensioned full column resin bolts. USBM RI 8439, 37pp.

Lang, T.A. 1958. Rock bolting speeds Snowy Mountains Project. *Civil Engng*, Feb. p.40-42.

Lang, T.A. 1961. Theory and practice of rock bolting. *Trans. AIMM & Pet. Engs*, V.220, p.333-348.

Lang, T.A. 1972. Rock reinforcement. *Bull. Assoc. Eng. Geol.*, V.9, No.3, p.215-239.

Lang, T.A., Bischoff, J.A. & Wagner, P.L.A. 1979. Program plan for determining optimum roof bolt tension theory and application of rock reinforcement systems in coal mines. Report to USBM.

Lang, T.A. & Bischoff, J.A. 1981. Research study of coal mine rock reinforcement. USBM OFR 72-82, 277pp.

Lang, T.A. & Bischoff, J.A. 1982. Stabilization of rock excavations using rock reinforcement. *Proc. 23rd US Symp. on Rock Mech.* p.935-944.

Lang, T.A. & Bischoff, J.A. 1984. Stability of reinforced rock structures. *Proc. ISRM Symp. on Design & Performance of Underground Excavations*, Cambridge, p.11-18.

Langland, R.T. 1977. Experimental investigation of room and pillar coal mining in the eastern United States. *Proc. Int. Symp. on Field Measurements in Rock Mech.*, Zurich, p.949-967.

Lappalainen, P., Pulkkinen, J. & Kuparinen, J. 1984. Use of steel strands in cable bolting and rock bolting. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.557-562.

Laubscher, D.H. & Taylor, H.W. 1976. The importance of geomechanics classification of jointed rock masses in mining operations. *Proc. Symp. on Exploration for Rock Engng*, Johannesberg, V.1.

Laubscher, D.H. 1984. Design aspects and effectiveness of support systems in different mining conditions. *Trans. IMM*, V.93, p.A70-A81.

Lawrence, D. 1972. Scale model studies of strata movement around mine roadways: effects of horizontal and vertical pressure. NCB MRDE report No.30 (*Int. J. Rock Mech. Min. Sci.* 1973, V.10, p.173-182).

Lawrence, D. & Silvester, G.V. 1972. Scale model studies of strata movement around mine roadways: floor dowelling. Unpublished notes and test results.

Ledvina, C.T. 1986. Geologic mapping can boost productivity, safety underground. *Coal Age*, April, p.54-61.

Lee, K.W., Kim, B.Y., Kim, H.Y. & Lee, C.H. 1987. Tunnelling experiences with NATM in a Korean colliery. *Int. Symp. on Coal Min. & Safety*, Soel, p.431-438.

Liangkui, C. & Shendou, F. 1983. Mechanism and strengthening effect of split set bolts. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.429-437.

Locotos, F.M. 1987. Personal communication.

Lorentzen, A.J. & Moore, R.K. 1984. Strata reinforcement - history, techniques and material development. *Colliery Guard.*, V.232, No.2, p.45-53.

Ludvig, B. 1983. Shear tests on rock bolts. *Proc. Int. Symp. on Rock Bolting.*, Abisko, p.113-123.

Lumetzberger, F. 1982. In-seam drivages with selective roadheaders and a combined rock bolts and wiremesh support system. *Gluckauf*, V.118, No.9, p.452-455. English Transl. p.170-172.

Mahtab, M.A., Bolstad, D.D. & Pulse, R.R. 1973. Determination of attitudes of joints surveyed with a borescope in inclined boreholes. USBM IC 8615, 12pp.

Mahyera, A. & Jones, A.H. 1985. A novel system for automatic installation of cement grouted roof bolts. *Proc. 4th Conf. Ground Control in Min.*, p.11-17.

Mahyera, A. & Jones, A.H. 1986. A novel roof bolting system for coal mines. *Proc. Int. Symp. on Underground Min. Sci. & Tech.*, Nottingham.

Mahyera, A. 1987. Terra Tek Inc. Personal communication.

Maidl, B. 1984. *Handbuch des Tunnel und Stollenbaus. Bd.1: Konstruction und Bauverfahren.* Essen: Gluckauf.

Maiweg, N. 1981. Rock bolt and porch set systems at Lohberg Colliery. *Gluckauf*, V.117, No.17, p.1099-1101. Eng. Transl. p.495-496.

Maleki, H.N., Hardy, M.P. & Brest van Kempen, C.J.H. 1985. Evaluation of roof bolt tension measuring techniques. *Proc. 26th US Symp. on Rock Mech.*, p.425-437.

Mallicoat, W.R. 1978. Truss bolting with point resin anchorage. *Min. Cong. J.*, V.64, No.6, p.47-50.

Mallory, R. 1981a. Floor reinforcement trials at Birch Coppice Colliery. Pt 1: 28's tail gate. NCB MRDE Internal Report No.81/48, 8pp.

Mallory, R. 1981b. Floor reinforcement trials at Birch Coppice Colliery. Pt 2: 46's tail gate. NCB MRDE Internal Report No.81/49, 6pp.

Mallory, R. 1984. Strata reinforcement in the Lower Fenton Seam at Bullcliffe Wood Colliery. NCB MRDE Rock Mech. Branch Tech. Memo. No.(84)17.

Mangelsdorf, C.P. 1979a. Evaluation of roof trusses, phase 1. USBM OFR 56-82, 109pp.

Mangelsdorf, C.P. 1979b. Recent progress in roof truss technology. *Proc. 1st Conf. on Ground Control Problems.* p.118-123.

Mangelsdorf, C.P. 1980. Role of friction in roof truss behaviour. *Trans. AIME.* V.268, p.1869-1879.

Mangelsdorf, C.P. 1985a. Design of a roof truss bolting plan for Bear Mine. *Proc. 4th Conf. on Ground Control in Min.* p.11-17.

Mangelsdorf, C.P. 1985b. Use of frequency of vibration to determine the tension in the horizontal chord of a roof truss. *Min. Engng.* p.1148-1150.

Mangelsdorf, C.P. 1986. Roof trusses. Contribution to *Coal Mine Ground Control,* 2nd Ed. By S.S. Peng, p.390-398.

Mangelsdorf, C.P. 1987a. Optimum design of roof truss installations based on bending strain energy. *Proc. 28th US Symp. on Rock Mech.* p.1115-1122.

Mangelsdorf, C.P. 1987b. University of Pittsburgh. Personal communication.

Martin, D. 1987. Dry run for Washington Metro gives NATM an American boost. *Tunn. & Tunnlg,* May, p.16-18.

Masson, C. 1985. Use of steel fibre shotcrete to line stonedrifts. *Gluckauf,* V.121, No.2, p.145-148.

Mattila, G. & Boyd, R. 1985. New method allows for anchorage check for rock bolts. *Min. Engng,* April, p.320-322.

McCabe, K.W. & Pascoe, W. 1978. Sandstone channels: their influence on roof control in coal mines. MSHA IR 1096, 24pp.

McCoy, A.E., Vnenk, L. & Hargraves, A.J. 1971. Rotary boring and resin bolting at Metropolitan Colliery. *Proc. Conf. on Rock Bolting,* Wollongong, 16pp.

McCullock, C.M., Diamond, W.P., Bench, B.M. & Duel, M. 1974. Cleats in bituminous coalbeds. USBM RI 7910, 19pp.

McDowell, G. 1987. Peabody Coal Corp. Personal communication.

Merritt, A.H. 1972. Geologic prediction for underground excavations. *Proc. 1st N. Am. Rapid Excavation & Tunnlg Conf.*, p.115-132.

Mills, L.J. 1985. Retreat vs advancing mining faces. *Colliery Guard.* June, p.247-249.

Moebs, N.N. 1984. Geologic structures in ground control. USBM IC 8973, p.4-14.

Moebs, N.N. & Stateham, R.M. 1986. Coal mine roof instability categories and causes. USBM IC 9076, 15pp.

Moore, J.F.A. 1967. Trials with Roc-Loc resin grouted floor bolts at Birch Coppice Colliery. NCB Central Engineering Establishment Report No.FT 760, 19pp.

Moore, R.K. 1983. Developments in rock bolting technology. *Tunn. & Tunnlg,* V.15, No.6, p.85-88.

Moore, R.K. 1986. Ground control techniques. *World Min. Equip.* Nov. p.22-24.

Morland, R.L. & Thompson, J. 1985. Development in ground support practices at Mount Isa Mine (Australia). Atlas Copco Tech. Bull. No.43, 4pp.

Morrow, R.E. 1981. New easy-to-use mine roof support system reduces bolt bleed-off and anchor movement. *CIM Bull.* V.74 No.831, p.137-138.

Mosley, J.T.B. 1974. Manton Colliery 30's return gate. NCB MRDE Interim Report, 7pp.

Mosley, J.T.B. 1986a British Coal HQTD, Personal communication.

Mosley, J.T.B. 1986b. Roadway stability in the No.7 Seam. British Coal HQTD Rock Mech. Branch Visit Report No.(86)14, 4pp.

Moulder, G.R. 1983. Ultrasonic stress transfer: a direct test of rock bolt integrity. USBM OFR 31-84.

Muller, L. 1978. Removing misconceptions on the New Austrian Tunnelling Method. *Tunn. & Tunnlg*, V.10, No.8, p.29-32.

Muller, S. 1987. Austrian road tunnel supported by water-expanded bolt. *Tunn. & Tunnlg*, March, p.35-36.

Mullins, D.R. 1985. Physical modelling of underground support systems. *Univ. of Leeds Min. Ass. Mag.*, p.36-41.

Murphy, J.M., Whittaker, B.M. & Blades, M.J. 1972. Strata bolting. *Colliery Guard.*, V.220, No.7, p.317-321.

Myrvang, A. & Hanssen, T.H. 1983. Experiences with friction rock bolts in Norway. *Proc. Int. Symp. on Rock Bolting,* Abisko, p.419-423.

National Coal Board (NCB) 1977. NCB cone indentor. MRDE Handbook No.5.

National Coal Board (NCB) 1978. Truss bolting at Park Mill Colliery, Barnsley Area. NCB HQ Min. Dept. Production & Productivity Directorate, Broadsheet No.30, 10pp.

National Coal Board (NCB) 1980. Castlehill Mine - truss bolting. NCB Scotish Area Min. Dept. Mechanisation Strata Control Branch Report, 5pp.

National Coal Board (NCB) 1984. Procedures in coal mining geology. NCB Mining Dept. Publication. 241pp.

National Coal Board (NCB) 1986. Bolts, plates and accessories for strata reinforcement. NCB Specification 131, 32pp.

Neall, G.M., Haycocks, C., Townsend, J.M. & Johnson, L.P. 1976. Influence of some critical design parameters on roof truss support capacity. - A preliminary report. *Proc. 17th US Symp. on Rock Mech.*, p.5E21-5E26.

Neall, G.M., Haycocks, C., Townsend, J.M. & Johnson, L.P. 1978. Optimizing roof truss installations with body-loaded photoelastic models. *Min. Engng,* June, p.660-666.

Neall, G.M. 1981. Design optimization in underground coal systems - volume VIII - the roof truss: an analysis with applications to mine design. Final report, Contract No. US DOE DE-AC01-76-ET-10722.

Neff, T.L. 1970. An evaluation of several types of rock extensometers. *Proc. 6th Canadian Rock Mech. Symp.*, p.103-118.

Newson, S.R. 1986. The use of rock bolting in mines in West Germany and France. *Min. Eng.*, Jan. p.281-288.

Nobel's Explosives Co. Ltd. 1986. Publicity Literature, 4pp.

Nocke, H., Rasche, O. & Schuermann, F. 1968. Experience with combined roof bolting and porch sets in a gateroad. *Gluckauf*, V.104, No.16, p.701-704. NCB Transl. No.A.2724.

Nocke, H. 1970. New experience with combined roof bolting/porch set support system. *Gluckauf*, V.106, No.20, p.976-983. NCB Transl. No.A.2849.

Nocke, H. 1978. Strata bolting in conjunction with gunited concrete in coal mine drifts. *Gluckauf*, V.114, No.4, p.163-168.

Norbury, B. 1986. Roof bolting trials, Brass Thill Seam, Ellington Combine. NCB North East Area Mech./Strata Control Branch Report 23pp.

Northard, J.H. 1986. Roadway formation in British coal mines. *Min. Eng.*, April, p.435-443.

Nyga, J. 1987. The course of convergence in gateroads supported with arches and rock bolts. *Gluckauf*, V.123, No.9, p.528-530. Eng. Transl. p.255-257.

O'Beirne, T. & Shepard, J. 1984. The failure of coal pillar ribs and possible methods of control. *Proc. 4th Aus.-N.Z. Conf. on Geomech.*, Perth, 6pp.

O'Beirne, T., Shepherd, J., Rixon, L.K. & Napper, A. 1986. Coal rib stabilization. - A new perspective. *The Coal J.*, No.14, p.7-11.

O'Beirne, T., Shepherd, J., Rixon, L.K. & Napper, A. 1987. Instability and support of coal mine ribs. ACIRL Published Report No.87-3, 130pp.

Obert, L. & Barry, A.J. 1955. Roof bolt compression pad. *Min. Cong. J.* V.41, No.7, p.35-36.

Obert, L. 1966. Summary of stress determinations made in the proximity of underground openings. *Proc. 8th US Symp. on Rock Mech.*, p.217-223.

Obert, L. & Duvall, W.I. 1967. *Rock mechanics and the design of structures in rock.* J. Wiley & Sons Inc. Chap. 20.

Oram, J.S. 1986. A comparison of full column resin anchored and full column friction anchored (Swellex) bolt reinforcing systems. British Coal HQTD Rock Mech. Branch Tech. Memo. No.(86)11, 18pp.

Oram, J.S. 1987. Observations and measurements related to roof bolt installation in S3's main gate in the Red Vein Seam at Betws Mine, South Wales. British Coal HQTD Rock Mech. Branch Interim Report, 4pp.

Oldengott, M. 1979. Floor bolting in a gateroad. *Gluckauf*, V.115, No.14, p.688-692. Eng. Transl. p.320-323.

Ounanian, D.W. & Cardenas, R.L. 1986. The design, construction and mine testing of a device for installing cement-grouted roof bolts. Final Report USBM Contract No.H0202027, 130pp.

Panek, L.A. 1955. Analysis of roof bolting systems based on model studies. *Min. Engng*, Oct.

Panek, L.A. 1956a. Theory of model testing as applied to roof bolting. USBM RI 5154, 11pp.

Panek, L.A. 1956b. Design of bolting systems to reinforce bedded mine roof. USBM RI 5155, 16pp.

Panek, L.A. 1956c. Principles of reinforcing bedded mine roofs with bolts. USBM RI 5156, 26pp.

Panek, L.A. 1962. Combined effects of friction and suspension in bolting bedded mine roof. USBM RI 6139, 31pp.

Panek, L.A. 1964. Design for bolting stratified roof. *Trans. SME-AIME*, V.229, No.2, p.113-119.

Parker, J. 1973/1974. Practical rock mechanics for the miner. *Eng. Min. J.*, Six Part Series.

Parsons, E.W. & Osen, L. 1969. Load loss from rock bolt anchor creep. USBM RI 7220, 26pp.

Patrick, W.C. & Haas, C.J. 1980. Strata separations and loads on grouted bolts in coal mine roofs. *Proc. 21st US Symp. on Rock Mech.*, p.757-768.

Peerlkamp, P. & Watt, K.L. 1971. Chemicals for bolt anchorages. *Proc. Symp. on Rock Bolting*, Wollongong, Paper No.3.

Pelissier, P. 1980. Face-ends. *Min. Eng.*, Dec. p.381-386.

Pells, P.J. 1974. The behaviour of fully bonded rock bolts. *Proc. 3rd Cong. ISRM*, V.2, p.B1212-B1217.

Peng, S.S. 1986. *Coal mine ground control*. 2nd Ed, J. Wiley & Sons, 491pp.

Perard Torque Tension Ltd. 1985. Publicity Literature.

Pettibone, H.C., Dar, S.M. & Smelser, T.W. 1985. Modelling of coal mine roof reinforcement. *Proc. 26th US Symp. on Rock Mech.*, p.1273-1280.

Phillips, F.C. 1971. *The use of stereographic projection in structural geology*. 3rd Ed, E. Arnold, London.

Piteau, D.R. 1970. Geological factors significant to the stability of slopes cut in rock. *Proc. Symp. on Planning Open Pit Mines*, Johannesburg, p.33-35.

Poad, M.E., Serbousek, M.O. & Goris, J. 1975. Engineering properties of fibre-reinforced and polymer impregnated shotcrete. USBM RI 8001, 25pp.

Poitsalo, S. 1983. Strengthening efficiency of different rock bolts. *Proc. Int. Synp. on Rock Bolting*, Abisko, p.459-464.

Potts, E.L.J. 1955. The practical application of scientific measurement to problems in strata control. *Iron & Coal Trades Rev.*, V.421, p.1169-1179.

Potts, E.L.J. 1957/1958. Further progress in the scientific approach to strata control. *Trans. IME*, V.117, p.203-219.

Precht, K. 1979. Grouting rock bolts using a perforated sleeve. *Tunn. & Tunnlg*, V.11, No.8, p.54-56.

Priest, S.D. & Hudson, J.A. 1976. Discontinuity spacings in rock. *Int. J. Rock Mech. Min. Sci.*, V.13, p.135-148.

Priest, S.D. & Hudson, J.A. 1981. Estimation of discontinuity spacing and trace length using scanline surveys. *Int. J. Rock Mech. Min. Sci.*, V.18, p.183-197.

Proctor, R. 1986. British Coal Rufford Colliery. Personal communication.

Rabcewicz, L.V. 1964/1965. The New Austrian Tunnelling Method. *Water Power*, Nov. p.453-457, Dec. p.511-515, Jan. p.19-24.

Rabcewicz, L.V. & Golser, J. 1974. Application of the NATM to the underground works at Tarbela. *Water Power*, V.26, No.9, p.314-321, No.10, p.330-335.

Radcliffe, D.E. & Stateham, R.M. 1978. Effects of time between exposure and support on mine roof stability, Bear Coal Mine, Somerset, Colo. USBM RI 8298, 13pp.

Radcliffe, D.E. & Stateham, R.M. 1980. Stress distribution around resin grouted bolts. USBM RI 8440, 39pp.

Raffoux, J.S., Sinou, P. & Tincelin, E. 1970. Bolting in adits or galleries. *Rev. Industrial Min.*, V.52, No.12, p.795-814.

Raffoux, J.S. 1971. Resin bonded bolting in French coal mines. *Proc. Symp. on Rock Bolting*, Wollongong, 18pp.

Raistrict, A. & Marshall, C.E. 1939. *The nature and origin of coal and coal seams*. The English Universities Press Ltd, London, 282pp.

Raju, N.M., Verma, B.P. & Singh, B. 1972. A new method of grouting bolts for roof support. *Can. IMM Bull.*, V.65, No.721, p.37-40.

Raju, N.M., Ali, S.V., Sheorey, P.R., Verma, B.P. & Singh, B. 1972. Experimentation of roof truss in a coal mine. *Proc. Symp. on Rock Mech.*, Dhabad, p.220-227.

Redaelli, L.L. 1985. Roof bolting in coal mines with evaluation of Swellex. Atlas Copco Tech. Bull. No.46, 51pp.

Redaelli, L.L. 1986. Design of rock bolting underground with evaluation of Swellex. Atlas Copco Tech. Bull. No.58, 15pp.

Reuther, E.U. & Hermuelheim, W. 1985. Rock bolting in the US hard coal mining industry. *Gluckauf,* V.121, No.9, p.677-681. Eng. Transl. p.216-218.

Rich, T.R. & Solomon, B.A. 1986. Design, fabricate and test a pumpable bolt system. USBM Report: Contract H0222077, 72pp.

Richmond, A.J. & Hebblewhite, B.K. 1980. An assessment of the installation and performance of split set roof bolts at Huntley Colliery. ACIRL Published Report No.80-10.

Roberts, A. & Hawkes, I. 1963. The application of photoelastic devices for measuring strata pressures and support loads. *Mine & Quarry Engng.*, July, p.298-308.

Roberts, A. & Hawkes, I. 1965. Optical load measurement techniques on mine support systems. *Colliery Guard.*, Aug., p.240-250.

Roberts, M. 1980. New roof bolt passes US tests. *Coal Age,* V.85, No.7, p.122-127.

Rocha, M. & Barrow, M. 1971. Some applications of the new integral sampling method in rock masses. *Proc. ISRM Symp. on Rock Fracture,* Nancy, Paper 1-21, 12pp.

Roko, R.O. & Daemen, J.J.K. 1983. A laboratory study of bolt reinforcement influence on beam building, beam failure and arching in bedded mine roof. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.205-217.

Rose, D. 1986. Steel-fibre reinforced shotcrete for tunnel linings. *Tunn. & Tunnlg,* May, p.39-44.

Round, C. 1979. Retreat mining with integrated dirt stowing. *Min. Eng.* Dec. p.495-505.

Ryan, T. 1975. Steel fibres in gunite - an appraisal. *Tunn. & Tunnlg,* July, p.74-75.

St John, C.M. & Van Dillen, D.E. 1983. Rock bolts: a new numerical representation and its application to tunnel design. *Proc. 24th US Symp. on Rock Mech.*, p.13-25.

Sames, G.P. 1985. Coal mine air tempering: effectiveness, design and roof support. USBM RI 8955, 20pp.

Schmid, W. 1986. Swollen rockbolts secure Alpine road tunnel. *Tunn. & Tunnlg,* July, p.32-34.

Schuermann, F. 1960. Measures against the relaxation of movements occurring in roadway walls. Wissenschaftler beim Steinkolenbergbeauverein in Essen.

Schuermann, F. 1978. Improving roadway condition by closer adaptation to the load bearing ring. *Gluckauf*, V.114, No.11, p.463-468. Eng. Transl. p.242-245.

Scott, J.J. 1976. Friction rock stabilizers a new reinforcement method. *Proc. 17th US Symp. on Rock Mech.*, p.242-249.

Scott, J.J. 1977. Friction rock stabilizers - a different approach to ground support. *Min. Engng*, July, p.59-62.

Scott, J.J. & Jackson, D. 1977. Continuous miner and friction bolts play key roles in Highlands move underground. *Engng Min. J.*, Dec. p.61-67.

Scott, J.J. 1978. Friction rock stabilizers how and why they work. *Proc. 19th US Symp. on Rock Mech.*, p.492-496.

Scott, J.J. 1980a. A new innovation in rock support - friction rock stabilizers. *Gluckauf*, V.116, No.3, p.112-116. Eng. Transl. p.50-52.

Scott, J.J. 1980b. Interior rock reinforcement. *Engng Min. J.*, V.181, No.9, p.79-91.

Scott, J.J. 1983. Friction rock stabilizer impact upon anchor design and ground control practices. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.407-417.

Seegmiller, B.L. 1986. Seegmiller truss bolt system - design and installation. Publicity Literature, 10pp.

Seegmiller, B.L. 1987. Seegmiller intersection truss system - Seegmiller continuous entry truss system. Publicity Literature.

Sellden, H. 1983. Cable bolting in Fabian ore body, Malmberget. *Proc. Int. Symp. on Rock Bolting*, Abisko.

Sen, G.C. 1959. Strata bolting in South Wales. *Colliery Engng*, p.247-250.

Serbousek, M.O. & Bolstad, D.D. 1981. Inorganic cement grouted bolt system. *Proc. 1st Conf. on Ground Control in Min.*, p.137-140.

Serbousek, M.O. & Signer, S.P. 1985. Load transfer mechanics in fully grouted roof bolts. *Proc. 4th Conf. on Ground Control in Min.*, p.32-40.

Serbousek, M.O. 1987. USBM Spokane Research Center. Personal communication.

Serbousek, M.O. & Signer, S.P. 1987. Linear load transfer mechanics of fully grouted roof bolts. Final draft of unpublished USBM Report of Investigation.

Shelton, P.D. 1980. PhD. Thesis University of Newcastle upon Tyne.

Sheorey, P.R., Verma, B.P. & Singh, B. 1973. An analysis of the roof truss. *J. Mines, Metals & Fuels*, Aug. p.233-236.

Shepherd, J. & Fisher, N.I. 1978. Faults and their effect on coal mine roof failure and mining rate. A case study in a New South Wales Colliery. *Min. Engng*, Sept. p.1325-1334.

Shepherd, J., Humphreys, D.R., Rixon, L.K. & Creasey, J.W. 1984. Geotechnical investigations of roadway rib instability during mine development at Harrow Creek Trial Colliery, Queensland. *Proc. 5th Australian Tunnlg Conf.*, Sydney, p.6-19.

Shepherd, J., Rixon, L.K. & Walton, K.P. 1986. Borescope techniques for assisting colliery roof control. *AusIMM, Ground Movement & Control Related to Coal Min. Symp.*, p.32-40.

Shi, G.H. & Goodman, R.E. 1983. Keyblock bolting. *Proc. Int. Symp. on Rock Bolting,* Abisko, p.143-167.

Sikora, W. & Kidybinski, A. 1977. Roof stability evaluation for proper choice of roadway supports. *Proc. 6th Int. Strata Control Conf.*, Banff.

Silvester, G.V. 1975. Scale model studies of strata movement around mine roadways: roof dowelling. NCB MRDE Report No.57.

Silvester, G.V. 1982. Strata reinforcement in the Barnsley Area. *Proc. Symp. on Strata Mech.*, Newcastle upon Tyne, p.82-91.

Simpson, R.E. 1978. Cartridge for grouting an anchor element in a hole of a support structure. US Pat.4,096,944.

Simpson, R.E., Fraley, J.E. & Cox, D.J. 1980. Inorganic cement for mine roof bolt grouting. USBM RI 8494, 32pp.

Simpson, R.E. 1980. New methods of grounting roof bolts with hydraulic cement. *Gluckauf,* V.116, No.23, p.1243-1245. Eng. Transl. p.491-492.

Singh, B. 1986. Status of research on roof supports in board and pillar workings. Central Mining Research Station Report, Dhanbad, India 42pp.

Singh, M.M. 1978. Effectiveness of inclined roof bolts in coal mines. USBM OFR 40-80, 115pp.

Singh, R.N. & Buddery, P.S. 1983. Assessment of the efficiency of roof bolt anchorage based on laboratory and field experimentation. *Proc. Int. Symp. on Rock Bolting,* Abisko, p.445-457.

Singh, T.N., Gupta, R.N., Dubey, B.K., Verma, B.P. & Singh, B. 1986. Cable bolted slicing of thick seam standing on pillars. *J. Mines, Metals & Fuels,* July, p.350-358.

Sinou, P. & Dejean, M. 1980. Types of rods for strata bolting, accessories and setting equipment. *Rock Bolting.* Ed de la revue Industri Minerale.

Smith, C.S. 1978. Clog-free inorganic grout emplacement gun. US Pat.4,116,368.

Smith, R.J. & Pearson, G.M. 1961. Tensional behavior of floor bolts in advancing longwall faces. *Min. Cong. J.* Nov. p.74-77.

Snyder, V.W., Gerdeen, J.C. & Wiezelahn, G.L. 1979. Factors governing the effectiveness of roof bolting systems using fully grouted and nontensioned bolts. *Proc. 20th US Symp. on Rock Mech.*, p.607-614.

Snyder, V.W., & Krohn, R.L. 1982. An experimental study of beam building mechanisms using fully grouted bolts in bedded mine rock. *Proc. Symp. on Strata Mech.*, Newcastle upon Tyne, p.234-236.

Solomon, B.A. & Rich, T.R. 1983. Chemical modification of pumpable bolt resin. USBM Report on contract H0282012, 173pp.

Spaun, G. & Jagsch, D. 1983. Tunnelling with the New Austrian Tunnelling Method in a 1100 m deep German coal mine. *Proc. 5th Cong. ISRM*.

Stateham, R.M. & Sun, M. 1976. Temperature investigations on resin anchored bolts. USBM RI 8178, 29pp.

Stateham, R.M. & Radcliffe, D.E. 1978. Humidity: A cyclic effect in coal mine roof stability. USBM RI 8291, 19pp.

Stateham, R.M. 1982. Roof bolt bond tester, a device for nondestructive testing of grouted bolts. *Proc. 2nd Conf. on Ground Control in Min.*, p.183-187.

Stateham, R.M. 1987. USBM Denver Research Center. Personal communication.

Stears, J.H. 1965. Evaluation of a penetrometer for measuring roof bolt anchorage. USBM RI 6646, 23pp.

Stephan, P. 1987. Extending the scope of application of rock bolting, and new developments. *Gluckauf*, V.123, No.9, p.538-543. Eng. Transl. p.262-264.

Stevens, C.R., Sanbak, L.A. & Hunter, J.J. 1987. LHD production and design modifications at the San Manuel Mine. *Proc. 28th US Symp. on Rock Mech.*, p.1175-1185.

Stheeman, W.H. 1982. A practical solution to cable bolting problems at the Tsumeb Mine. CIM Bull., V.75, Feb., p.65-77.

Stillborg, B. 1986. *Professional users handbook for rock bolting*. Trans Tech Publ., 145pp.

Stimpson, B. 1983. Influence of rock bolt location on the reinforcement of horizontally bedded roofs by full column grouted bolts. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.195-204.

Su, W.H, & Peng, S.S. 1987. Cutter roof and its causes. *Min. Sci. & Tech.*, V.4, p.113-132.

Sykes, G. 1987. Allerton Bywater 56's partial extraction . British Coal North Yorks. Area Interim Reports, Jan., Feb., March.

Szlavin, J. 1971. Relationships between some physical properties of stone determined by laboratory tests. NCB MRDE Report No.19.

Tadolini, S.C. 1986. Anchorage capacities in thick coal roofs. USBM IC 9058, 13pp.

Tadolini, S.C. & Ulrich, B.F. 1986. Evaluation of bearing plates installed on full column resin grouted bolts. USBM RI 9044, 12pp.

Tamames, B.C. 1983. Fourteen years of experience on rock bolting in Spain. *Proc. Int. Symp. on Rock Bolting*, Abisko, p.295-311.

Taylor, R.K. & Spears, D.A. 1970. The breakdown of British coal measure rocks. *Int. J. Rock Mech. Min. Sci.*, V.7, p.481-501.

Terzaghi, K. 1943. *Theorectial Soil Mechanics*. J. Wiley, New York, Art 20.

Terzaghi, K. 1946. Rock defects and loads on tunnel supports. In *Rock Tunnelling With Steel Supports*. Eds: Proctor R.V. & White T.L. (Youngstown, Ohio: The Commercial Shearing & Stamping Co.) p.17-99.

Terzaghi, K. 1965. Sources of error in joint surveys. Geotechnique, V.15, p.287-304.

Thiel, P.S. 1964. Recent developments in roof bolting and roof bolt installation procedure. *CIM Bull.*, V.57, N.630, p.1050-1062..

Thomas, E. 1962. Stabilization of rock by bolting. *Rev. Engng Geol., Geol. Soc Amer.*, V.1, p.257-279.

Thomas, L.J. 1966a. Rock movement around roadways at Babbington Colliery. *Colliery Guard.* 15 April, p.475-481 & 22 April, p.507-510.

Thomas, L.J. 1966b. A strata introscope. *Colliery Guard.*, V.212, No.5477, p.447-450.

Thompson, R.R., Habberstad, J.L. & Bates, R.C. 1974. Pumpable polymeric bolts. *Proc. 3rd Cong. ISRM,* V.2, p.B1223-B1228.

Thompson, R.R. & Bates, R.C. 1975. Field testing the pumpable polymeric roof bolt. *Proc. 1st Symp. on Underground Min.*, V.2, p.80-89.

Thompson, R.R. 1984. Development of epoxy grouts and pumpable bolts. In: *Mine Ground Control.*, USBM IC 8973, p.126-128.

Thurner, H.F. 1979. Non-destructive test method for rock bolts. *Proc. 4th Cong. ISRM,* V.3, p.254-255.

Thurner, H.F. 1983. Detection of invisible faults on rock bolts in-situ. *Proc. Int. Symp. on Rock Bolting,* Abisko, p.477-480.

Tifft, R.D. 1987. Cyprus Coal Co. Personal communication.

Tilman, M.M., Jolly, A.F. & Neumeier, L.A. 1984. Corrosion of friction rock stabilizers in selected uranium and copper mine waters. USBM RI 8904, 23pp.

Tilman, M.M., Jolly, A.F. & Neumeier, L.A. 1985. Corrosion of roof bolt steels in Missouri lead and iron mine waters. USBM IC 9055, 9pp.

Tinchon, L. 1980. An example of quadrangular section strata bolting in a gateroad. *Rock Bolting* Ed de la revue Industri Minerale, p.29-31.

Ting, F.T.C. 1977. Origin and spacing of cleats in coal beds. *Trans. ASME,* Nov p.624-626.

Tully, D.M. 1984. The use and effectiveness of remedial measures for unstable rock slopes. M.Sc. Dissertation, Queen Mary College, University of London, 352pp.

Tully, D.M. 1985. Instrumentation of shaft inset at North Selby. Report to NCB MRDE, 2pp.

Unal, E. 1983. Design guidelines and roof control standards for coal mine roofs. Ph.D. Thesis, The Pennsylvania State University.

US Army Corps Of Engineers 1980. Engineering and design: rock reinforcement. Engineer manual EM1110-1-2907. Available from the Office of Chief Engineer, Washington D.C.

US Government 1977. Coal mine health and safety act. Code of federal regulations. V.30.

University College Cardiff 1987. Special report on strata bolting in S3 maingate. Dept. of Mining, Geological and Minerals Engng, 51pp.

University Of Nottingham 1983. *Support and closure of main drivages.* Dept. of Mining Engng, 151pp.

University Of Nottingham 1985. *Strata loading of mine roadway supports.* Dept. of Mining Engng, 280pp.

Unrug, K.F. 1986. Monitoring of the performance of mechanical roof bolts in an Appalachian coal mine. *Proc. Underground Min. Sci. & Tech. Symp.*, Nottingham.

Venkateswarlu, V., Sinha, A. & Raju, N.M. 1983. Prediction of roof conditions through geotechnical studies - an approach. *J. Mines, Metals & Fuels*, March, p.94-100.

Wade, L.V. 1977. Investigation of full column resin rock bolt reinforcement mechanisms. *Proc 6th Int. Strata Control Conf.*, Banff.

Walker, S. 1986. Westmin's H-W, where mining methods meet. Int. Min., Nov. p.32-41.

Wallis, S. 1986. Greeks grapple with NATM. *Tunn. & Tunnlg*, Sept. p.17-19.

Wallis, S. 1987. First NATM contract in Britain guarantees water tight drifts. *Tunn. & Tunnlg*, Sept. p.42-44.

Walton, R.J. & Fuller, P.G. 1980. Investigation of a bolted coal mine roof during mining at Nattai North Colliery. CSIRO Div. Applied Geomech. report GCM 24, 53pp.

Wang, F.D., Zink, G., Wolgamott, J. & Maxwell, B. 1976. Test of inorganic replacement for resins. USBM OFR 59-78, 84pp.

Ward, W.H., Coats, D.J. & Tedd, P. 1976. Performance of tunnel support systems in the Four Fathom Mudstone. *Proc. Tunnlg '76 Conf.*, London, p.329-340.

Weidmann Ltd 1986. Fibreglass rock bolts. Publicity Literature. 13pp.

Wester, A. 1971. Floor reinforcement in roads and gates. *Gluckauf*, 29 April. NCB Transl. No.A2901.

White, C.C. 1967. In situ mine roof trusses combining rock compression with steel tension members. *Proc. Coal Min. Inst. Amer.*

White, C.C. 1969. Mine roof support system. US Patent No.3,427,811.

White, C.C. 1970. Mine roof support system. US Patent No.3,505,824.

White, C.C. 1982. Development cooperation between operator and manufacturer - point anchor resin bolts. *Proc. 2nd Conf. Ground Control in Min.* p.198-201.

Whittaker, B.N. & Hodgkinson, D.R. 1971. Reinforcement of weak strata. *Min. Eng.*, V.130, p.595-609.

Whittaker, B.N. & Singh, R.N. 1980. The estimation of strata loading at a ripping lip. University of Nottingham, Report to N.C.B.

Whittaker, B.N. & Scoble, M.J. 1980. Rock mechanics instrumentation in underground ironstone mining in the UK. *Proc. 13th Canadian Rock Mech. Symp.*, p.111-116.

Wijk, G. & Skoberg, B.O. 1982. The Swellex rock bolting system. *Proc. 14th Canadian Rock Mech. Symp.*, p.106-115.

Williamson, I.A. 1967. *Coal Mining Geology.* Oxford Univ. Press, 266pp.

Wilson, A.H. 1972. Research into the determination of pillar size. *Min. Eng.*, No.141, p.409-430.

Wilson, A.H. 1977. The stability of tunnels in soft rock at depth. *Rock Engng*, University of Newcastle upon Tyne.

Wilson, A.H. 1980. The stability of underground workings in the soft rocks of the coal measures. Ph.D. Thesis, University of Nottingham, 188pp. (*Int. J. Min. Engng*, V.1, No.2, p.91-187.)

Wright, F.D. 1974. Lateral thrust, bedding and jointing in roof stability calculations. USBM IC 8630.

Yagi, Y. 1978. Application of the NATM to the Nakayama Tunnel. *Tunnelling In Difficult Ground.* Ed: I. Kitamura, Pergamon Press, Oxford, p.193-197.

Zadeh, A.M.H. 1982. Support characteristics of underground mining excavations with special reference to rock reinforcement techniques. Ph.D. Thesis, University of Nottingham, 359pp.

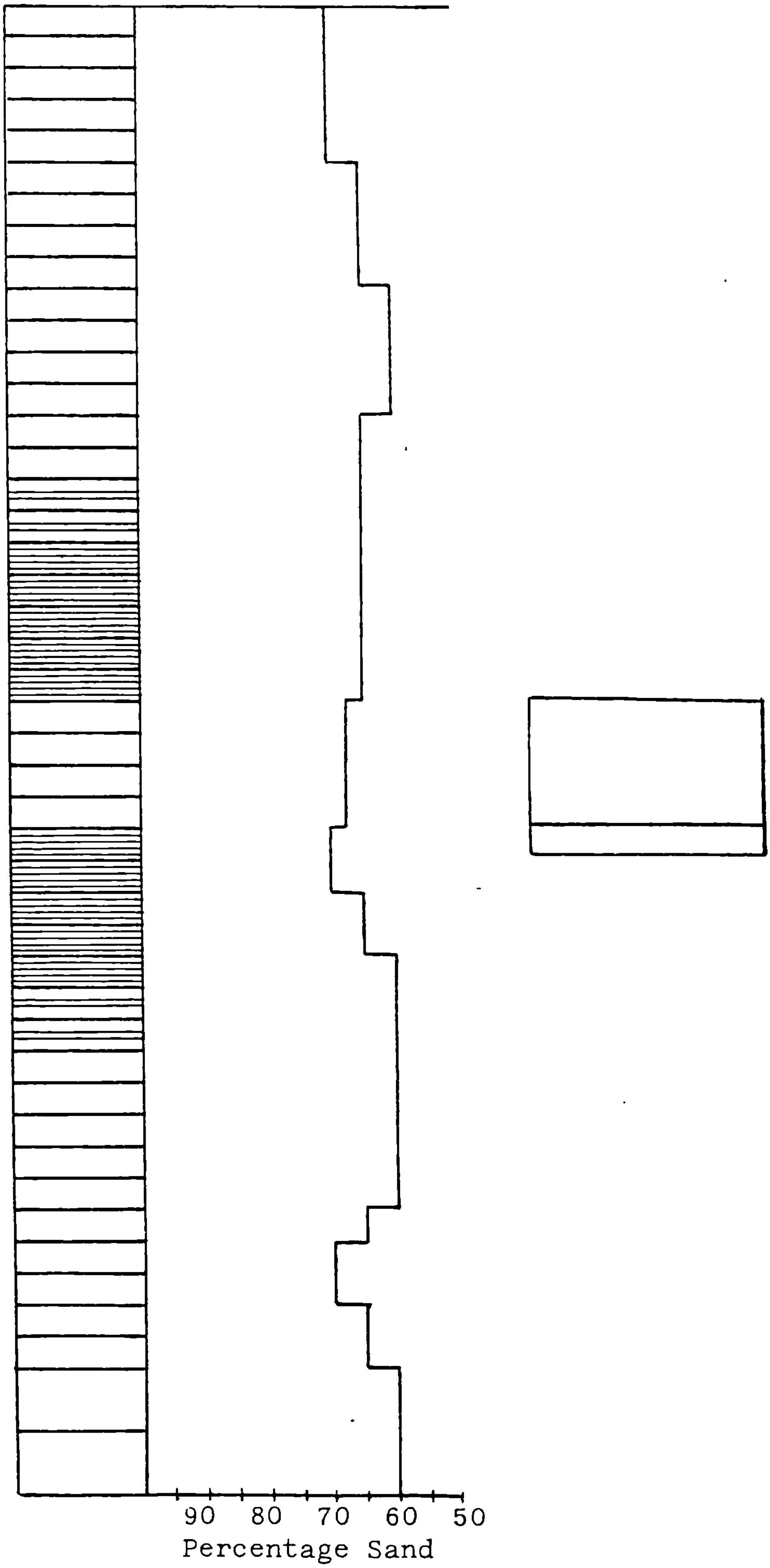
Zillessen, C. 1978. New Austrian tunnel construction methods as supports for shaft cavities. BAG Lippe, T.2.2. NCB MRDE Transl. No.541.

Zischinsky, U. 1987. Bolting-shotcreting linings in large mine structures. *Gluckauf*, V.123, No. 9, p.552-556. Eng. Transl. p.272-274.

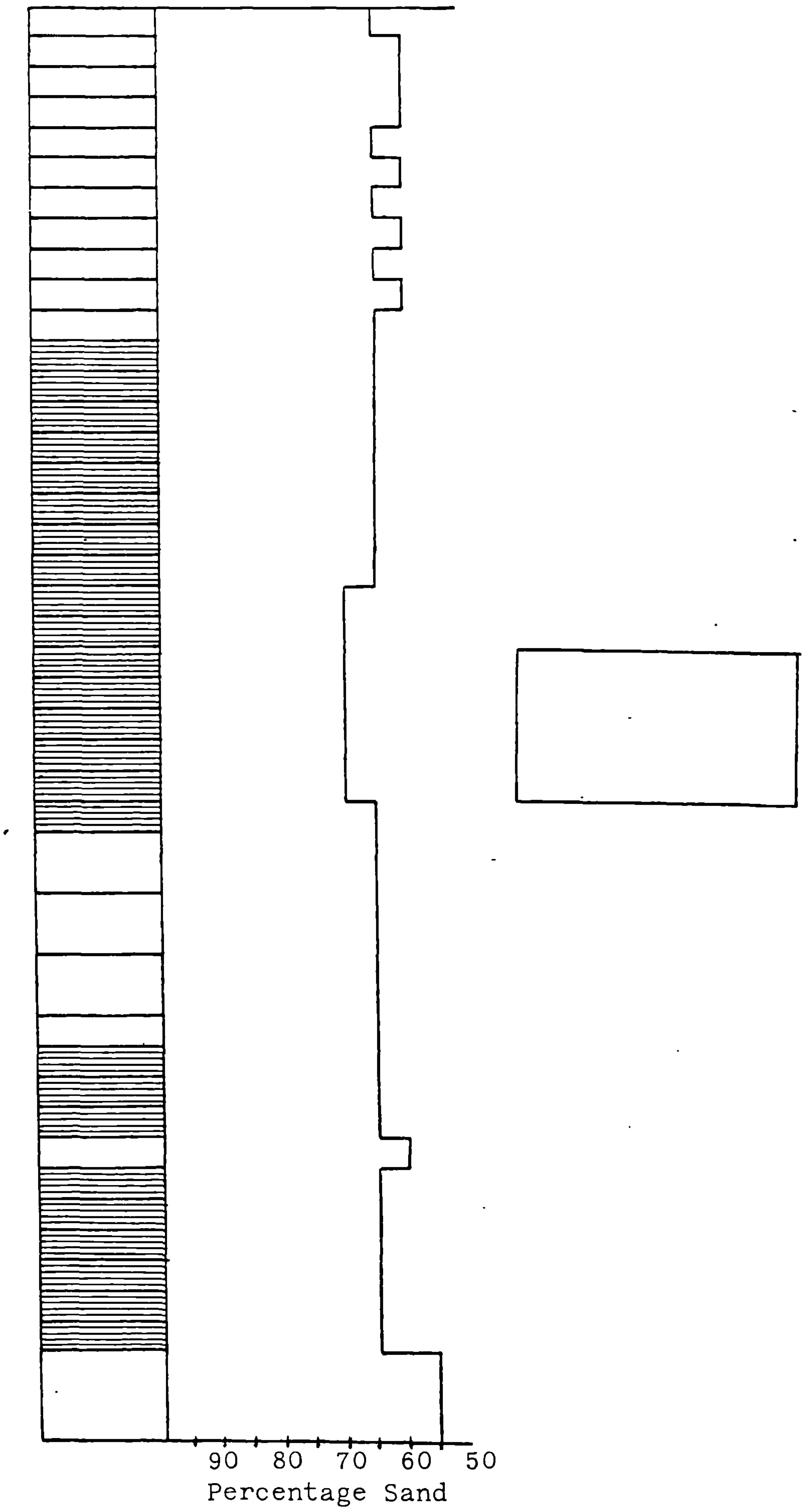
APPENDIX 1

SCALE MODEL STRATA CONFIGURATIONS

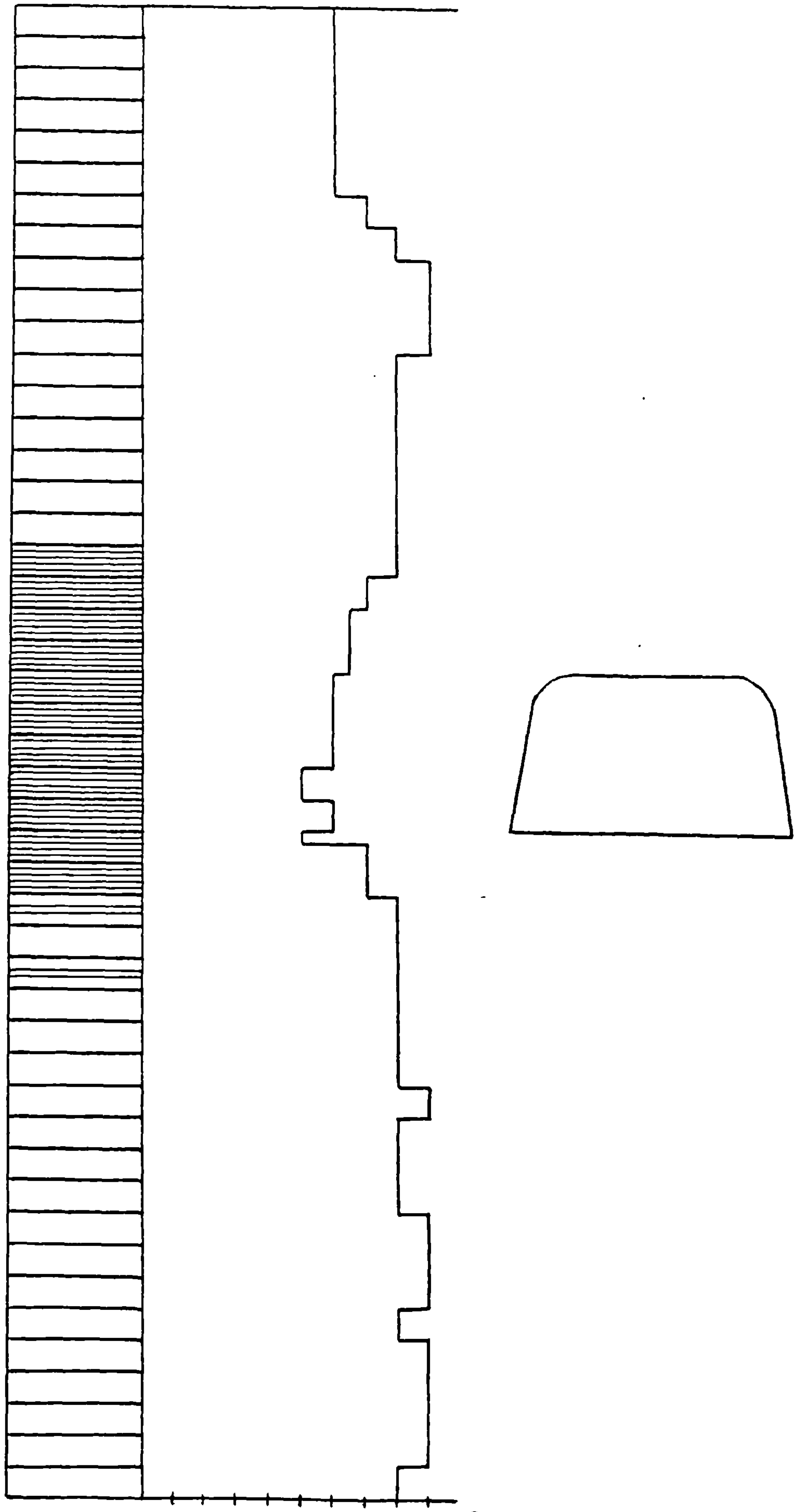
Showing:
Postion of laminations
Constituent proportions
Position and shape of modelled roadway



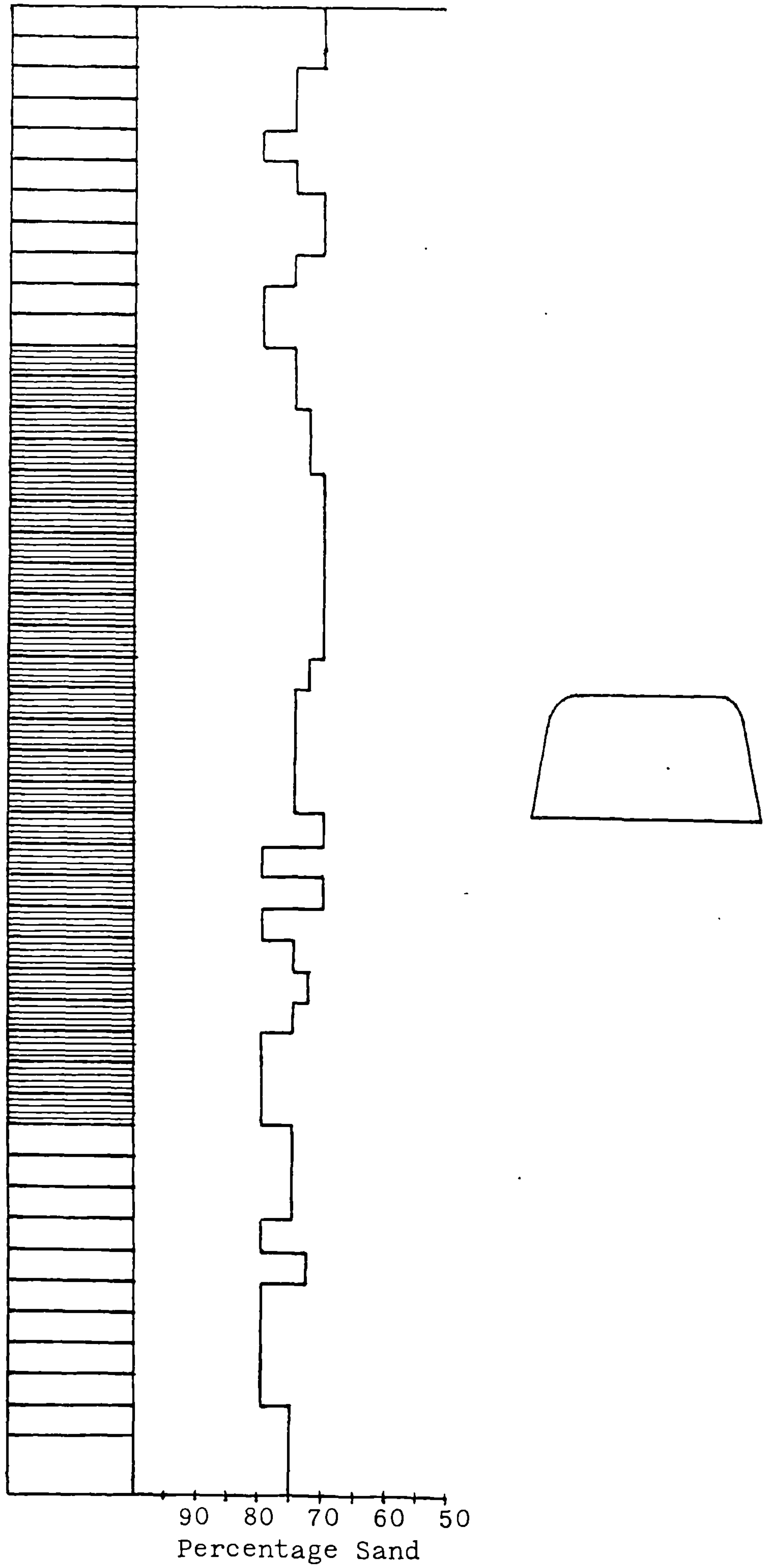
1a Qualitative series.



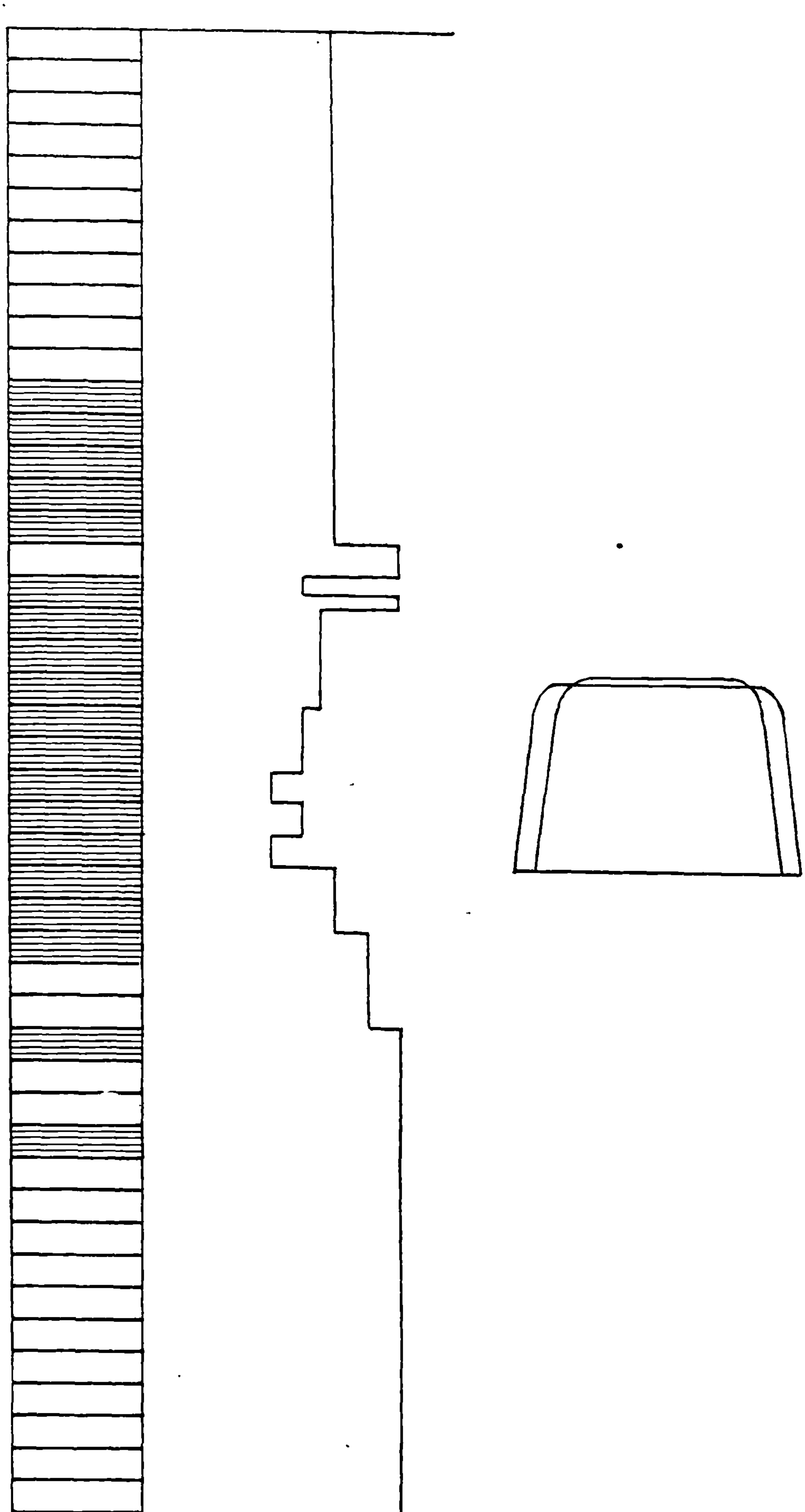
1b. Sherwood Colliery - Deep Hard/Piper partial extraction.



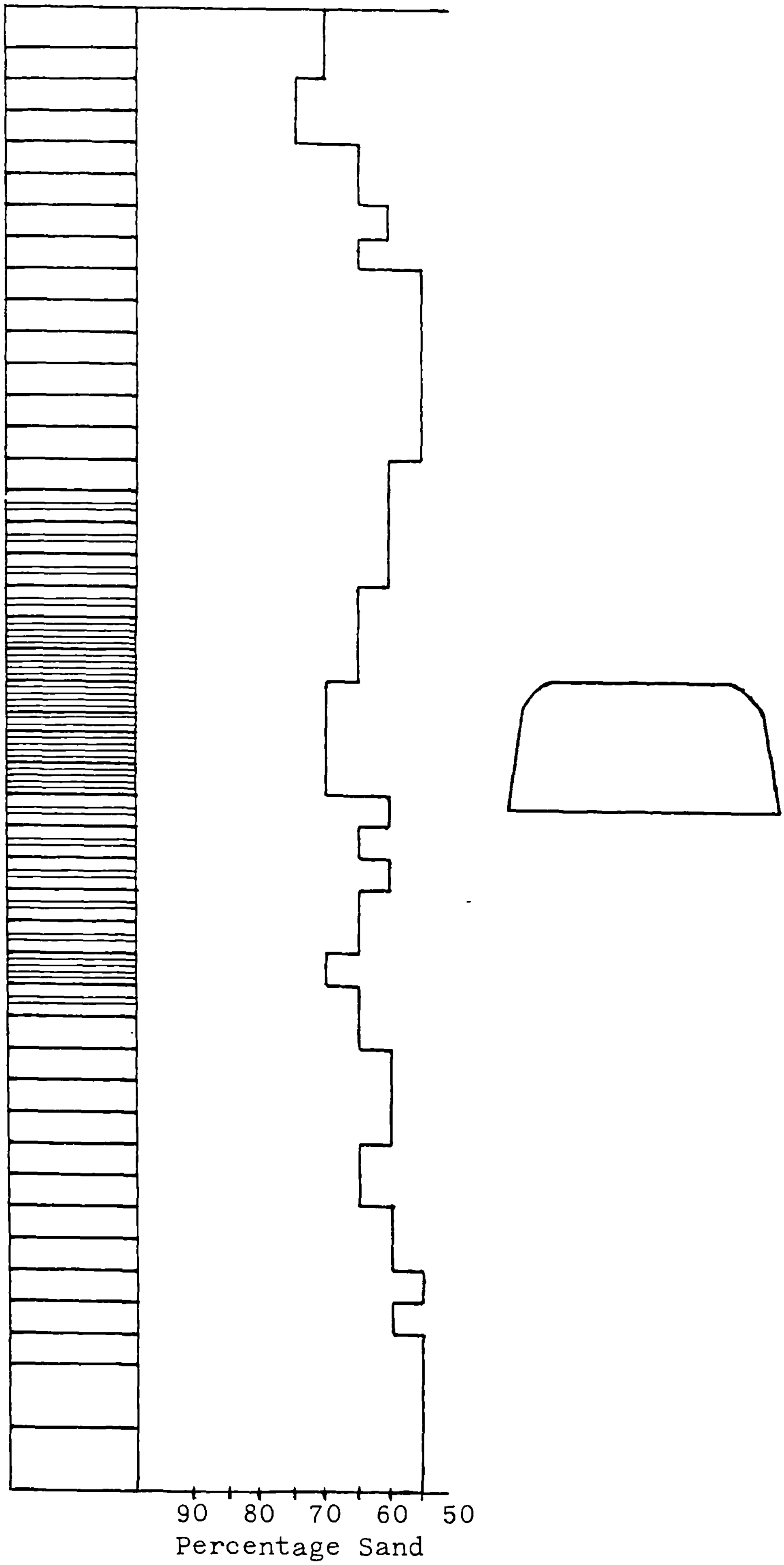
1c Mansfield Colliery - 202's gateroad.



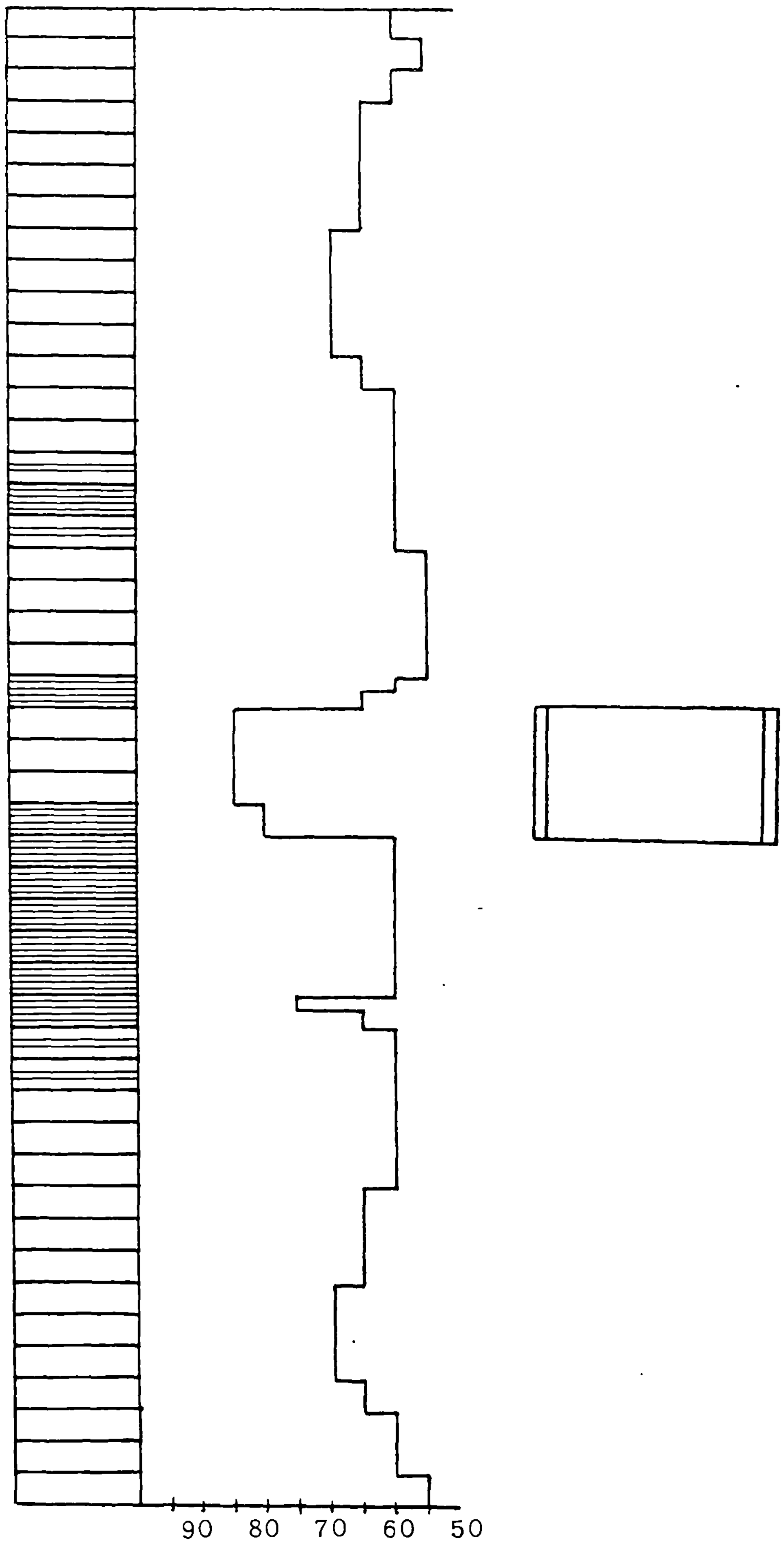
1d Snowdown Colliery - 1's main gate.



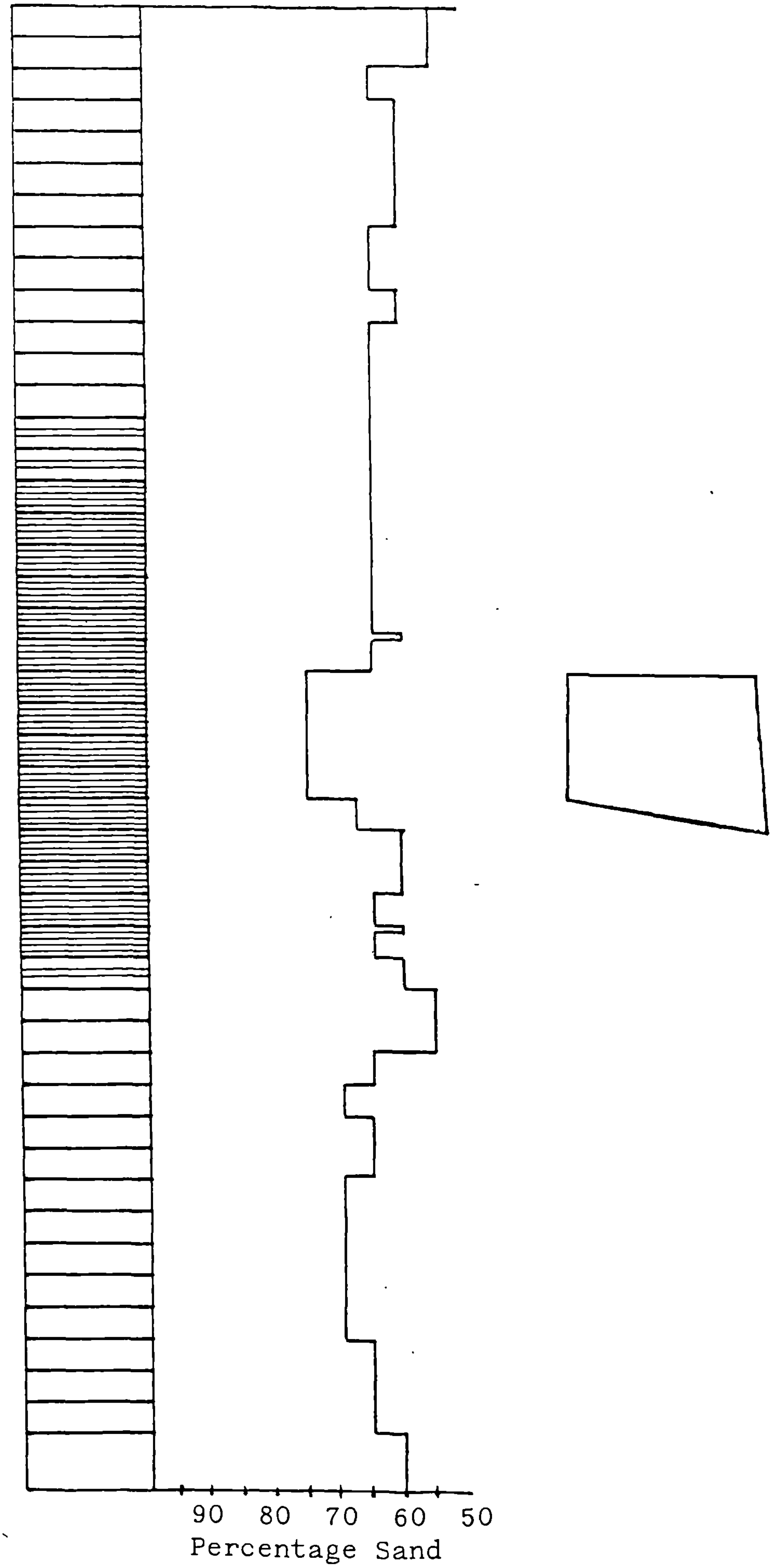
le Rufford Colliery - 206's main gaτe.



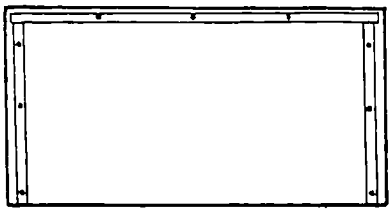
1f Welbeck Colliery - 88's gateroad.



1g Penallta Colliery - M25's main gate.



1h Margam Prospect.



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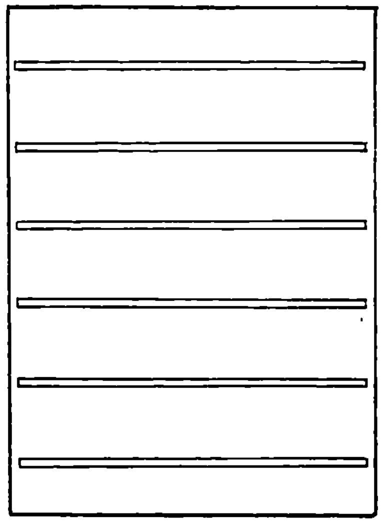

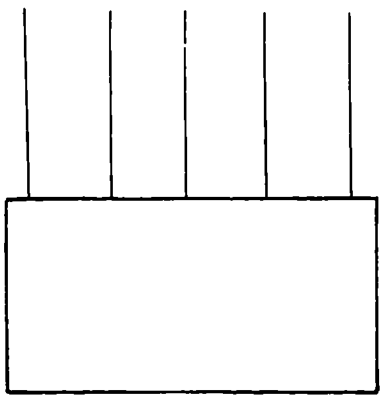

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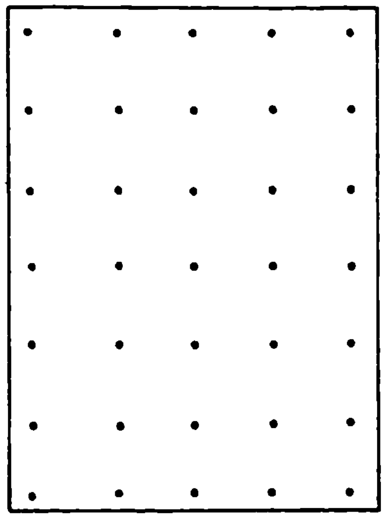

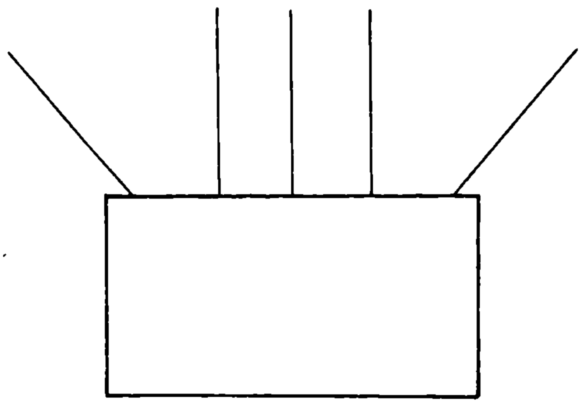

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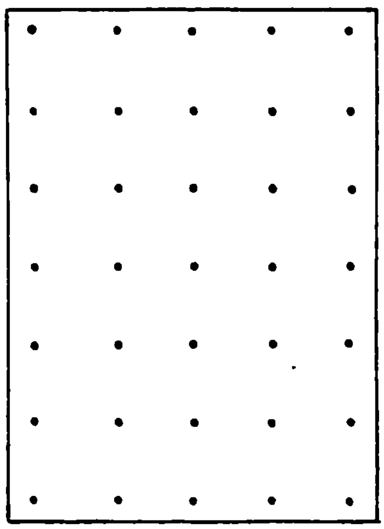
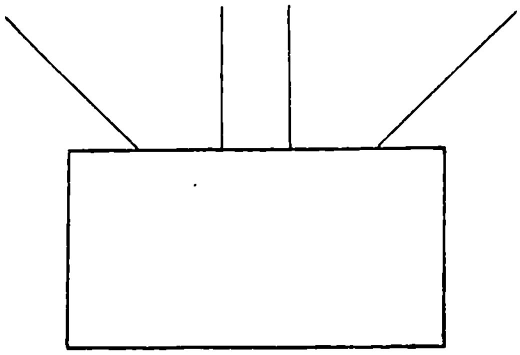
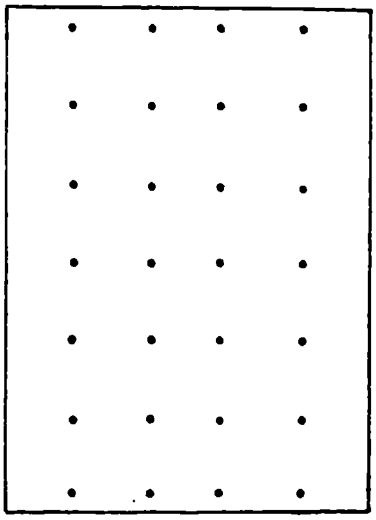


APPENDIX 2

SUPPORT CONFIGURATIONS FOR MODELLED ROADWAYS





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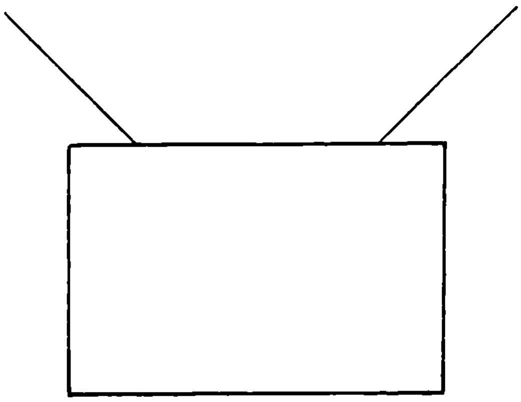

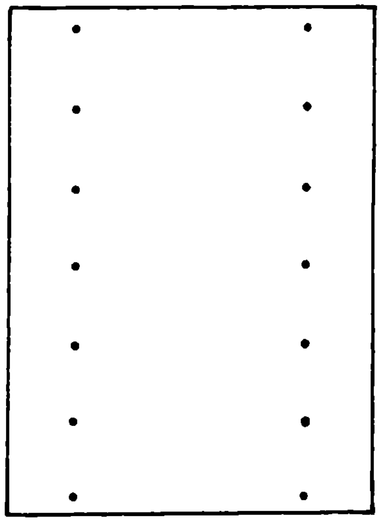

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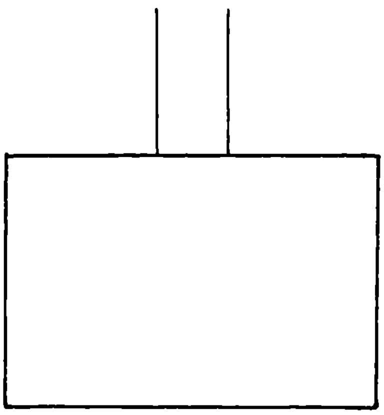

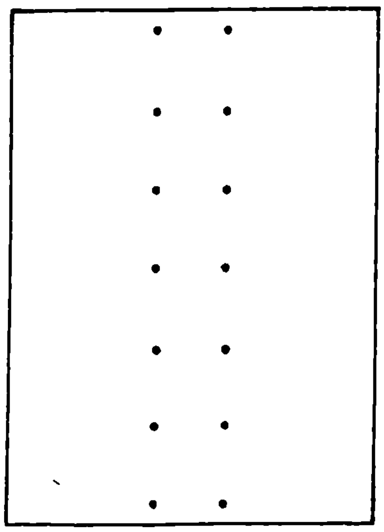

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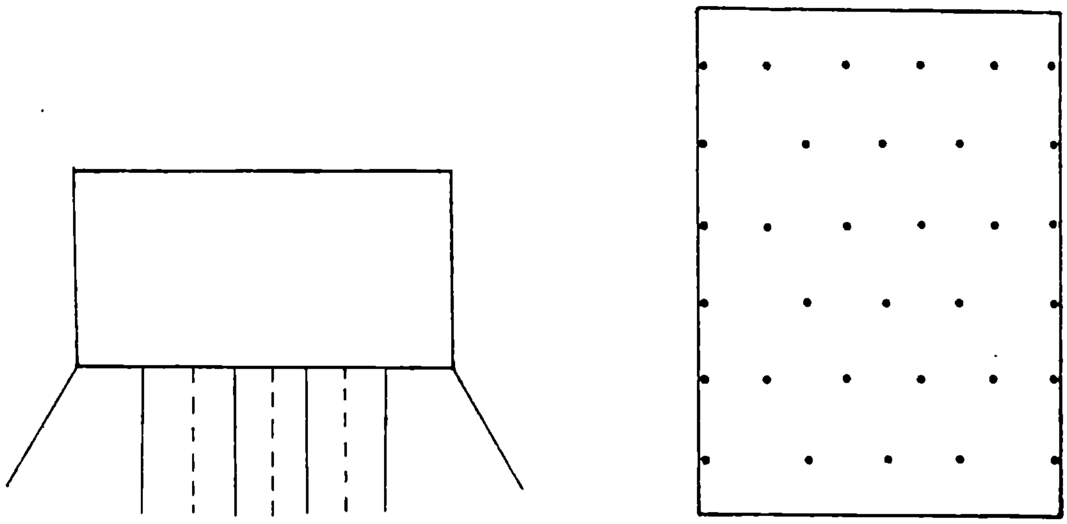

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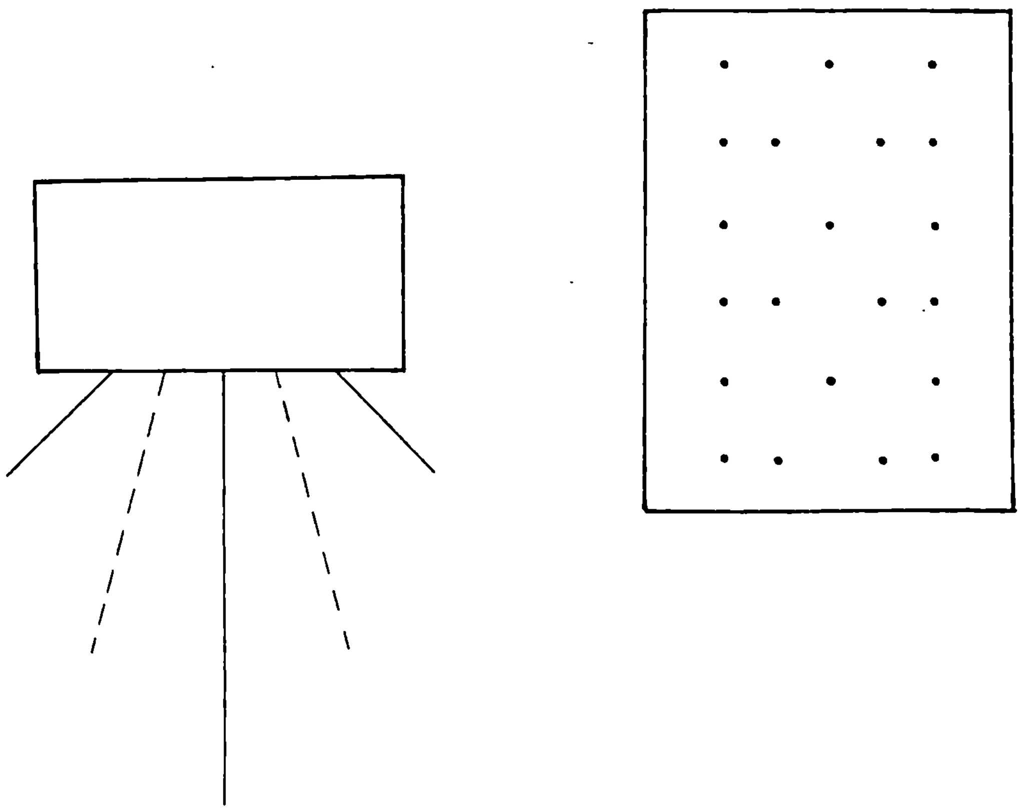

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