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Design Criteria For Roof Bolting Plans Using Fully Resin-Grouted Nontensioned Bolts to Reinforce Bedded Mine Roof

Volume I

Executive Summary and Literature Review

Prepared for

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF MINES

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16. Abstract (Limit: 200 words)

The objective of this investigation is to identify factors governing the effectiveness of roof bolting systems using fully resin-grouted non-tensioned bolts for reinforcement of bedded mine roofs and to develop design criteria for such plans. Volume I presents a summary of work done, a background study, and a literature review on resin bolting, mine analysis, and coal and rock properties. Strength and stiffness properties of resin bolts under both axial and shear loadings are considered in detail. Mechanisms of roof reinforcement using resin bolts are reviewed. Associated mine analyses including the effects of in situ conditions are included.

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FOREWORD

This report was prepared by Michigan Technological University, Department of Mechanical Engineering - Engineering Mechanics, Houghton, Michigan 49931 under USBM Contract Number J0366004. The contract was initiated under the Coal Mine Health and Safety Program. It was administered under the technical direction of PM&SRC with Dr. Chi-Shing Wang acting as the Technical Project Officer. Mrs. Pearl A. Shapert was the contract administrator for the Bureau of Mines.

This report is a summary of the work recently completed as part of this contract during the period July 22, 1975 to July 22, 1977. This report was submitted by the authors on July 22, 1977.

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EXECUTIVE SUMMARY ON

DESIGN CRITERIA FOR ROOFBOLTING PLANS USING FULLY RESIN-GROUTED NONTENSIONED BOLTS TO REINFORCE BEDDED MINE ROOF

The objective of this contract is to identify factors governing the effectiveness of roof bolting systems using fully resin-grouted non-tensioned bolts for reinforcement of bedded mine roofs and to develop design criteria for such plans.

This is a summary of work done from July 22, 1975 to July 22, 1977 on Contract #JO366004.

The project consisted of six tasks:

Task (1). Background Study: Literature Review

Task (2). Background Study: Field Survey

Task (3). Experimental Model Studies

Task (4). Theoretical Analysis

Task (5). Design Study: Synthesis

Task (6). Design Study: Establishment of Design Criteria

The final report on this project consists of five volumes. The first four volumes report on Task (1) through Task (4) respectively, and the last volume combines the results of Task (5) and (6). The significant findings of the individual tasks are summarized in the following.

1. TASK (1), REVIEW OF LITERATURE ON RESIN BOLTING, MINE ANALYSIS AND COAL-ROCK PROPERTIES:

1.1 Introduction:

Three areas of related research were reviewed, namely resin bolting, mine analysis, and coal and rock properties. More than 250 references, both domestic and foreign, have been gathered. The more significant results are discussed in some detail in this report whereas less pertinent references are given only a cursory review.

This review related specifically to bolting of horizontally bedded mine roof with flat backs. The objective of this review then has been to classify and identify from the literature available data on the factors governing the effectiveness of roofbolting systems using fully resin-grouted nontensioned bolts for reinforcing bedded mine roof.

1.2 Conclusions on Resin Bolting:

Material property data suggests that the physical behavior of resin in tension and compression may be different. Chemical bonding of resin to steel and rock may exist under ideal conditions and low bolt loads, but appears to be absent in the mine environment.

Axial stiffness of resin bolts is nominally 10 to 20 times greater than for mechanical bolts. A ribbed resin bolt (rebar) fails by failure of the steel in hard rock, but a smooth bolt pulls out - indicating that mechanical inter-lock is the primary means of bonding. In soft rock, the resin bolt assembly fails at the rock interface indicating that the resin is stronger than the rock. A larger hole should provide a greater surface area at the resin-rock interface, reduce the shear stress there, and increase the anchorage in soft rocks and still give sufficient bolt strength.

Under axial loading the bolt load decays with distance into the rock along a fully-grouted bolt. The load is distributed over a shorter distance (15 inches) in hard rock, but over a greater distance (30 to 40 inches) in soft rock. This load transfer distance is a design factor that has to be related to the thickness of the layers in bedded mine roof. Creep of resin bolts has been found to increase the load transfer distance with time, particularly in softer rock and coal. Analytical methods are available to predict the load transfer length and stress distribution along a fully-grouted bolt as a function of the resin bolt properties.

Transverse shear strength of bolted joints is an important advantage of the resin bolt, particularly at low normal pressures for resin bolts oriented normal and at $\pm 45^0$ to the joint surface, and for both rough and flat joint planes. Under transverse shear loads, little benefit was found from either post-tensioning or pre-tensioning of resin bolts. The load transfer length under shear loading is found to be much less (about 3 to 4 inches) than under axial loading. Analytical methods are also available for estimating the shear strength and load transfer length for shear loading of resin bolts. Joint elements have been constructed for finite element programs, to account for both the axial stiffness (K_n) and the shear stiffness (K_s) of resin-bolted joints.

Some resin-bolt design charts are available giving the bond factor (anchorage) as a function of rock strength for axial loading. Design charts are lacking for shear loading and some should be constructed.

A resin bolt can provide a suspension effect in a mine roof just like a mechanical bolt, and many times with a better anchorage. Length of bolt to reach competent rock is an economic factor favoring the mechanical bolt in many cases. Friction from clamping is not a primary means of reinforcement of the untensioned grouted bolt, however, a grouted bolt can maintain the precompression and initial friction between rock layers that existed before installation - and do this better than the mechanical bolt because of resin bolt's greater stiffness and shorter load-transfer length.

A resin bolt is expected to provide a better reinforcement to the Voussoir rock-arch type of roof support mechanism. Because resin bolts can resist compression, they can help transmit the compressive thrust through the rock arch to the ribs. If a resin bolt is angled over the ribs, it can help prevent vertical shear failure at the ribs. A resin bolt with its greater shear resistance would be more effective.

A resin bolt can help build a roof beam even where the rock joints do not offer any frictional resistance themselves because of bed separation. The effectiveness depends upon the number of layers, lateral stresses and abutment support, but has been found to decrease slip of the beds during bending by as much as 50%. In laminated beams, an advantage of the fully-grouted bolt over the mechanical is that failure at any point along the grouted bolt will not reduce the total reinforcement effect since the grouted bolt adds shear resistance to each layer independently. With point anchored bolts, failure at any point eliminates all reinforcement.

It is reported in the literature that in most mines a change to resin bolting has greatly reduced roof falls. In a few exceptions, resin bolting did not help.

A new resin bolting concept, the "debondable bolt" appears to offer better anchorage in soft rock, and a capability to yield, to absorb large rock deformations, while retaining the features of the fully-grouted resin bolt at lower loads.

1.3 Conclusions on Mine Analysis:

Designing an adequate roofbolting plan requires an analysis of the mine conditions as well as the bolt assembly itself. Important in-situ conditions are stresses (gravitational, thermal, residual), moisture, and geology. Faults and surface topography can greatly alter the stresses in the mine roof. Water can cause swelling and stress in mine rock, and cause deterioration in shales.

The geology of the roof determines the type of roof support mechanism that should be designed for a particular mine. In many coal mines, a coal rider seam causes support problems. The geological column of the roof is reported in the literature for several mines and several coal seams. It varies appreciably within a seam, within a mine as well as from mine to mine. It is important to know coal cleat, rock joint and fault orientations, locations of clay veins, and overburden isopach maps - they all affect roof stability.

Blasting has been found to damage the roof and loosen roofbolt anchorage as far as 12 to 15 feet from the face, and up to 2 to 4 feet into the roof. Experiments on blasting effects on resin bolting show that resin bolting is more effective than mechanical bolting under these conditions, because of the greater damping capacity and greater stiffness of the resin bolt assembly.

Observations of roof failure have been made by many investigators and have been reported and classified in the literature. They have been classified as tensile failures, shear failures, and delamination failures. Different factors contributing to each have been identified.

Several methods of analysis have been used in the calculation of stresses around mine openings, the most useful being the finite-element method. This method requires the computer, but is most powerful because layered, anisotropic, jointed and bolted rock can be analyzed. Calculations have been made on the effect of horizontal stresses on the critical fracture surface around mine openings. The results show that the critical fracture surface (or cave) will extend higher or lower into the roof depending upon the coefficient of friction of the roof rock.

Results for multi-layered strata show that the presence of weaker strata increases the critical horizontal roof stress. Calculations show that horizontal segregation, i.e. "cross-bedding" in the roof, and inclusions in the roof can cause great variation in the roof stresses, with high tensile stresses at mid-span when the cross-bedded member or inclusion is hard rather than soft.

Calculations have been made for a coal mine entry driven under a valley and one under a hill. The results show that mid-span tensile stresses increase with the height of the hill for both openings with the opening under the hill having the greatest mid-span stress. It would be interesting to have results for an opening between the valley and the hill, where the greatest gradient in overburden occurs.

Studies are also reported for the effect of opening width-to-height ratio, the effects of joints and faults, and the effects of bolting on roof stresses and roof deflection.

1.4 Conclusions on Coal and Rock Properties:

There are data in the literature for the strengths and moduli of elasticity of coal, shales, sandstones and siltstones found in coal mine roofs. Although these data are limited, they should be sufficient for general analysis purposes in order to bracket the range of behaviour that might be expected. The coal is usually the weakest, and if a coal rider is found in the immediate roof, it can be a source of roof control problems.

Properties measured on small specimens, however, are inadequate. The properties of a rock mass with joints and faults, subject to environmental effects must be assessed. It appears that a rock quality index can be used as a measure to determine the ranges of rock quality where untensioned fully resin-grouted bolts can, and cannot, be expected to provide roof support. Attempts should be made to classify different coal mine roof sections using such a rock quality index. Methods are available in the literature and they have been used for tunnel support design.

2. TASK (2), VISITS TO UNDERGROUND MINES, RESIN MANUFACTURERS AND UNIVERSITIES AND OTHER RESEARCH FACILITIES:

2.1 Introduction:

As part of this contract, visits were made to 35 underground coal mines, a rocksalt mine, a deep potash mine and a fireclay mine. The intent was to observe first-hand a wide variety of roof-control problems, the use of resin bolts to overcome those problems, and the shortcomings of resin bolting.

Visits were made to four resin manufacturers, and to several universities and other research facilities. The intent was to establish contacts and to exchange ideas on the principles of resin bolting.

2.2 Visits to Mines:

Reception was always good, and the operators went out of their way to show us their most difficult roof conditions, and how they handled them. After making all of the scheduled visits, the following conclusions were reached:

> Many Roof-Control Problems Could and Should Be Avoided: Many conditions were attributable directly to "weathering", to design of pillars, openings and mining layouts and to some geological structures whose trend was predictable. To a large extent these problems could and should have been avoided or controlled before embarking upon roof-support plans.

In Most Coal-Mine Roofs the Rock is the Weak Link; In very few places were broken rebars seen. In most places resin and rebar could be found intact in fallen ground. It therefore appears that failure of resin bolts mostly occur at the rock interface.

Most Operators Thought that Resin Bolts Worked Better Than Mechanical Bolts: Some found that they could reduce costs by using shorter resin bolts. Most operators figured that they were using resin bolts to build beams, whereas they had formerly used longer mechanical bolts for a suspension effect. Some operators were using resin bolts for temporary support, knowing that they would have to follow up with cribs and rails within a few weeks. A couple of operators had given up on resin bolting because too much resin was lost into separations in the roof.

Some operators especially like the idea that resin plugged the roofbolt holes - to stop water coming from above, or to prevent internal weathering.

Design Criteria were Nebulous: They were based mainly upon experience. Most mines had falls of roof, all had places which needed extra support, and all had some places which looked as if they didn't really need bolting.

Other Resin Bolting Ideas: Resin bolts and wire mesh are used, especially in France, to reinforce entries in soft, crushed rock, notably in longwall development work. Overload reduces the dimensions of the entries, but they retain their shape and they stay open.

Steel rebars are used to reinforce the rock, and fiberglass or wooden dowels are used to reinforce coal which will later be cut.

Resin dowels are used to reduce floor heave in longwall entries, especially in England.

2.3 Visits to Resin Manufacturers:

All were very cooperative. Ideas on resin bolting were exchanged, and we came away with the impression that the manufacturers would rather run tests in a mine than take any theoretical approach to design criteria.

2.4 Visits to Universities and Other Research Institutions:

The authors left with the impression that all are working around the fringes of the roof-control problems and staying away from the real thing. So there are many pull tests, many model tests, and many computer analyses, but nobody is dealing with something applicable to the actual mine environment.

2.5 Visits to Foreign Countries:

In Germany and France resin bolting in combination with mesh or planks and post were used very successfully in long term entries. The rock conditions were poor and there was much swelling of the rock but the success of the program seems to depend on a national program of monitoring convergence. Bolting in these countries is not as mechanized as the United States, but drilling inclined holes is common in many places.

In England resin bolting for roof control was not used in the deep long wall mines, but grouted dowels were being used to control floor heave. A deep room and pillar potash mine was visited where fully grouted resin bolts were used for roof control. Although local difficult has prohibited the establishment of a bolting pattern, it was noted that in places no plates were used and the roof remained stable.

2.6 Conclusions of the Field Study:

The following conclusions can be drawn from the visits to 38 underground mines. There were 27 domestic coal mines, 8 foreign coal mines, a clay mine, a salt mine and a potash mine visited.

The presence of mud seams, slips and local discontinuities in the roof rocks can contribute to poor roof conditions. Surface topography and rock stratification plays a role in producing poor or good roof. Lateral stress is one of the governing factors in roof stability. Weathering of the rocks is the prominent cause of poor conditions and often leads directly to falls of ground. It is very difficult to control weathering and many mining plans require retreating from the area before weathering can seriously deteriorate the roof rocks.

The design of a mine panel, the design of pillars and the general layout of the mine can be such to minimize the effects of the local geology and surface topography. With geological knowledge of the ground, many problems could be avoided or controlled before mining. Even if difficulty is encountered during mining, it may be wise to change direction of the entries, leave top coal, etc. Although some mines have personnel involved in rock mechanics programs to obtain information about falls of ground and other geological data, most mines do not. It is the authors' belief that much could be gained through simple rock mechanics programs of measurements and observations.

Most operators believe the resin grouted bolt works better than mechanical bolts. In some cases shorter resin bolts may be more economical than longer mechanical bolts. In other cases resin bolts are required to continue mining because mechanical bolts will not support the roof.

In most coal mines the roof rocks are the weak link in the bolting system. The failure usually occurs at the rock-resin interface. In many places resin and the rebar could be found intact in fallen ground. Most operators believed the resin bolts were building beams whereas the mechanical bolts are used for suspension. The mine observations did not bear out a great amount of beam building. Roof support plans were based mainly upon experience. Most mines had falls of ground and had places where additional support was required.

3. TASK (3), EXPERIMENTAL MODEL STUDIES:

3.1 Introduction:

The experimental studies were divided into four major areas of investigation with subdivisions as necessary to facilitate the studies. The major areas were model materials, adhesion of resin to rock, laboratory pull tests, and load transfer mechanisms.

The objectives of these studies were to better understand the rock/resin/bolt interlock mechanism and to delineate the types of roof support mechanisms at work in the mine, i.e. beam-building, suspension, horizontal shear reinforcement, etc.

3.2 Modeling Materials:

Little information had been published concerning the mechanical properties of the soft shales found in most coal mines. Observations on our mine trips suggested that many coal mine roof rocks (especially the soft shales and mudstones) were relatively weak. Therefore, an attempt was made to find a model material for use in the laboratory with the following properties:

- Compressive Strength - 5,000 psi or less
- Tensile Strength - 0 psi
- Elastic Modulus - 1×10^6 psi
- Physically soft and smooth - Mohs hardness of 3 or 4, which represent the soft mudstones and shales observed in the coal mines.

Five materials were initially chosen to simulate the roof rock. After extensive laboratory work and taking into consideration the large number of models needed, it was decided to use a molding plaster of 55 gm water/100 gm plaster for all pull tests.

3.3 Adhesion of Resin to Rock:

The adhesive strength of resin to rock has been subject to question. Chemical bonding and mechanical bonding were discussed with manufacturers of resins and with mine operators, with no clear conclusion being reached. It does seem that some type of bonding does take place between resin and rock, but this bond is also a function of the conditions of the rock, i.e. roughness, moisture, dirt, etc. The purpose of the tests here was not intended necessarily to settle the debate concerning chemical bonding and mechanical interlock, but to give some data on the shear adhesive strength of the rock/resin interface.

Resin discs of 1-3/8 and 1-5/8 inch diameters and of 1/8 inch thickness were bonded to pieces of rock and pieces of modeling material. These discs were sheared off in a universal testing machine. The shear loads and the amount of adhesion were recorded in each test.

The resin does adhere to some rock types. In some types of soft rock the failure occurred in the rock and not at the interface. With hard rock from the White Pine Copper Mine, failure always occurred at the resin/rock interface.

Surface conditions of the specimens had a noticeable effect on the shear strength. In general, as the degree of surface roughness increased, the holding ability of the rock/resin interface increased.

3.4 Laboratory Pull Tests on Short Length Bolts (Bond Lengths 7-10")

Observations of bolt hole conditions in the various mines pointed out several different factors which affect the holding strength of the bolt/resin/rock interface. The main factor seemed to be the roughness and surface condition of the hole after drilling. Others such as dust left in the hole, moisture, and effect of resin thickness were also believed to be influential factors. A test matrix was set up as a guide to find the influence of these different factors on the holding strength of the resin. The same modeling material was used for all tests.

The pull tests indicate that hole surface conditions play a major part in the resin bolt's holding ability. The pull test failures occurred in model material at the rock/resin interface.

The smoother the hole, the less the load-carrying capability: The holes that were randomly grooved produced the highest load-carrying capability. Bolts examined after the tests showed model material jammed against ridges in the resin, indicating a shearing of the model material. Grooving the hole increased the pull load by as much as a factor of two or three over the as-drilled hole.

Appreciable variation in load capacity can be expected for holes of supposedly the same conditions: Standard deviation values and percent variations show that the pull test values are within a scatter band of $\pm$ 18 to $\pm$ 45% for smooth drilled holes, and within $\pm$ 10 to 27% for the grooved holes. Thus, the maximum scatter (in percent) decreases as the hole roughness is increased.

Increasing the annulus changed the load-carrying capability: In general the resin mixing became poorer as annulus increased, and load-carrying ability decreased.

In relatively soft rock, with failure occurring at the resin/rock interface, the annulus thickness is a limiting factor independent of bolt diameter: Essentially the same results were obtained for a 3/4-inch bolt in a 1-inch hole as for a 1-inch bolt in a 1-1/4-inch hole (annulus thickness 1/8-inch for both), and essentially the same results were obtained for a 3/4-inch bolt in a 1-1/4-inch hole as for a 1-inch bolt in a 1-1/2-inch hole (annulus thickness 1/4 for both). The higher values obtained were for the 1/8-inch annulus. Thus, for soft rock, if a larger hole size is needed to reduce the shear stress at the resin/rock interface then a larger bolt should be used to keep the annulus thickness at the optimum. This result is evidently related to resin mixing problems or shrinkage for larger annulus sizes.

Examination of the pull test graphs indicated three distinct zones of action: In the elastic region there was very little slippage, and if the load were released the bolt would return to its original position. The non-elastic or slippage region is divided into two parts. When slippage begins, small chips or wedges are dislodged from the modeling material, jamming the resin and hole side together - thus causing an increase in load-carrying capability. As the load increases, the wedge action induces local shearing, which breaks down the wedge action, eventually allowing the bolt to be pulled out of the hole. Maximum bolt loads were normally developed at slippages of about 1/2 inch for a bond length of about eight inches in this soft modeling material.

Pulling a strain-gaged bolt: Only a limited analysis of the 20 inch long strain-gaged roofbolt was completed, due to the large amount of data taken during the test. As the data was reduced, several interesting results emerged and are summarized as follows:

a) The load distribution for the bolt in a soft rock situation shows that the load was spread out along the entire bolt, even at the lower loads. This is quite different from the hard rock situation in which progressive debonding is indicated.

b) Total debonding or slippage occurred at a bolt load of approximately 3,500 pounds. The bolt did continue to carry increasing loads of at least twice that much before large amounts of slippage occurred. The increase in load carrying capability must be due to some type of mechanical interlock mechanism.

c) Large motions occurred at the higher loads as the load was held constant. The deformation was rapid at first but then slowed until a very slow change was occurring with time, somewhat similar to the first and second stages of a standard creep curve. This indicated that the mechanical interlock mechanism may be time dependent.

3.5 Beam Model Studies:

The objectives of the beam model studies were to better understand the support mechanism associated with beam-building. The beam models permitted controlled laboratory testing of some of the concepts discussed in Task (4).

A substitute material was needed since difficulty was encountered in making uniform layers of moulding plaster. One-half inch thick commerical grade plasterboard was selected to construct the models. The model must simulate the configuration of the mine roof in order to obtain valid results, and the flat laminates of the mine could be readily modeled by the plasterboard. The sheets also made it possible to construct the models quickly and to expedite testing. The plasterboard was scribed through the paper covering top and bottom to ensure use of the low tensile properties of the plaster alone.

Small Scale Beam: Initial experiments were conducted on a five-layered beam, four inches wide with a 25-inch span. The roofbolts were simulated with 3/16-inch all-thread rod in 1/4-inch diameter holes. Four bolts were used, five inches apart, and bonded with resin the entire length of the bolt. (No washers or plates were used on the bolts).

Strain gages were mounted at the center and at one end of the beam on both top and bottom surfaces of each layer (ten gages at each cross-section, twenty gages total).

The strain distribution measured verified a delamination hypothesis. At the center of the beam the bottom layer was in tension, while almost all the top layer was in compression. However, the layers near the clamped end acted independently.

Large Scale Beam Tests: Large Models were needed to permit testing of actual "full scale" bolts and commercially available resin. The tests were designed to demonstrate the delamination theory and check if a higher density of bolts could reduce this delamination effect.

The beam models consisted of 20 layers of 12-inch wide x 8-foot long x 1/2-inch thick sheet rock plasterboard. The beams were clamped at each end, giving a span of 64 inches.

In the first test, the beam had three resin-grouted roofbolts of 3/4-inch diameter rebar installed in 1-inch diameter holes, using a commercially available resin. The bolts were centered two feet apart. Delamination took place as the load was increased. The layers separated around the quarter points of the beam and acted independently. As the load was increased failure started to occur in the individual layers.

In a second test, a higher density of bolts (five bolts, on 10.6 inch centers) appeared to add significant reinforcement. A parabolic load distribution of 344 pounds was applied to the top of a 936 pound uniform load and the beam withstood this 1,280 pound load without any initial indication of fracture, but when allowed to creep overnight the beam did fail. The failure occurred at the clamped end and appeared to have progressed layer by layer.

Large Scale Plate: A plate model was made up of 20 sheets of 4-foot x 10-foot x 1/2-inch thick plasterboard. Two opposite edges were clamped in such a manner to give a span of 8 feet. Bolts of 3/4-inch diameter were resin-grouted in 1-inch diameter holes. Different bolt patterns were used; first no bolts, then a 2-foot by 2-foot pattern, then additional bolts one foot apart at the ribs.

Non-tensioned resin bolting was not effective in the model in reducing separations and strains. With 2-foot centers and unclamped ends the layers were for the most part acting independently, with the highest loading occurring on the bottom layers. Clamping the ends changed the magnitudes of the bending strains, yet the layers still acted independently. When the additional bolts were added, no changes were seen in the strain-gage results. Thus there was no reinforcement due to the presence of the bolts. In all cases the layers continued to bend independently even when the bolts were added.

The bending strains at the support were higher in the bottom layers than they are in the top layers for all cases. This means that the bottom layers are carrying more than their share of the load (due to the individual layer weight), even though they have equal thickness. This phenomenon is attributed to the interference of the layers due to the different curvatures on the top and bottom sides of each layer.

4. TASK (4), THEORETICAL ANALYSIS OF RESIN BOLTING AND ROOF SUPPORT MECHANISMS:

4.1 Introduction:

This report on the theoretical analysis summarizes the results of investigations on axial and shear loading of resin bolts, the Voussoir rock arch, beam-building, angle-bolted roof trusses, jointed roofs, bolting of sloping joints, and stresses in roofs due to moisture changes. Many analyses have been made of these conditions as summarized in the Task (1) report on the literature search. Available analyses are used in Volume 5 on design of resin bolting systems. The supplementary analyses described here were deemed necessary to fill gaps in the existing theory.

4.2 Conclusions:

Approximate relations were derived for the equivalent axial stiffness and equivalent bending stiffness of resin bolts. These relations show the effect of resin annulus size and resin modulus. Approximate closed-form solutions were developed for the shear stiffness of a resin-bolted joint, and the load transfer length of a resin bolt under bending. These solutions can be used to evaluate resin bolt designs, establish criteria, and guide experiments.

Finite element results were obtained for the Voussoir rock arch type of roof support mechanism. They confirm speculations of a tensile zone existing from the quarter point to the mid-span. This analysis shows that the rock arch is activated at moderate horizontal stresses in the range of 200 to 600 psi compression.

A closed form stability analysis was conducted on the Voussoir rock arch. Results give an estimate of the bolt length required to maintain the arch.

Angled bolts are required over the ribs to maintain the arch when the horizontal stress and the friction coefficient are too low. The analysis shows that a more dense bolting pattern is required near the ribs under such conditions, otherwise supplemental support such as posts will be needed, i.e. to reduce the effective span.

The analysis shows that a bolted arch is not possible when bed separations exist. The shear strength of the bolt without the additional frictional resistance of unseparated layers is not sufficient to transmit compression through the arch. This calls for early bolting before separations occur, or there will be no arch.

A plate buckling analysis of thinly-laminated roof beams shows two counter-acting effects. As the number of layers increase, the effectiveness in increasing the bending stiffness of the roof with untensioned bolts decreases. However, when the layers are separated, the resin bolt shear stiffness is most effective for layers approximately one inch thick. Based on the shear stiffness of the bolt, bolting of thicker separated layers is less effective. Therefore, there is an intermediate range of bed thickness where untensioned resin bolts give some benefit. Again, bolting can be more effective if the bolts are installed before bed separations occur.

An analysis was conducted of the delamination of thinly-laminated roofs. The analysis shows that delamination is likely to occur near the ribs - where horizontal shear is the highest between beds. This means that bolts should be installed as close to the ribs as possible to prevent delamination. A close bolt spacing near the ribs would also help the beam bending mechanism.

The analysis leads to the recommendation of a design criterion of a more dense bolt pattern, or larger bolts, as near as possible to the ribs, with bolts angled over the ribs. This would aid both the rock arch mechanism and the beam bending mechanism.

Angled resin bolts are more effective in building a roof truss if they are point anchored above the first beam layer and if the distance from rib to bolt-hole is equal to about 1/3 of room width. The truss should be tensioned at a specific value that can be calculated.

A multiplicity of joints can be analyzed by a continuum finite element approximation. It is found that joint spacing has the same effect on roof deflection as joint stiffness. High angle joints (steeply sloping) are more critical than low-angled joints. Joints that tend to open in the absence of insufficient horizontal in-situ compressive stress cannot be adequately reinforced by resin bolting alone, and additional positive support is needed.

Equations have been derived for the effect of bolting across joints at an angle.

Analysis was made of the effect of moisture changes in roof shales. It was found that both an increase in moisture and a decrease in moisture cause tensile stresses within the roof sufficient to cause failure. A sufficient increase in moisture can cause compressive failures too. This cannot be prevented by bolting, but tempering of the mine air to avoid moisture changes in the roof is recommended.

It was found that certain locations under hills and valleys at shallow depths can have tensile stresses or shear stresses that can contribute to roof support problems.

5. TASK (5) and TASK (6), DESIGN STUDY:

5.1 Introduction:

This report on design synthesis and the establishment of design criteria is based on a critical review of the results of the first four tasks. Those results considered important and useful in formulating design criteria are evaluated herein.

The design synthesis cannot be as comprehensive as it should be. There are areas of investigation in resin bolting that need more work. The synthesis is as complete as it can be within the time allotted for the task.

5.2 Conclusions on Design Synthesis:

An anchorage chart has been established. A small resin annulus 1/8-inch in thickness gives maximum anchorage. Bond factors in inches of anchorage required per ton, are given for different strengths of rock. A factor of safety of at least two is recommended for smooth holes.

Comparisons of experiments with theory have shown that the load transfer length in resin bolts is much longer (14 to 32 inches) than predicted by theory (4 to 7 inches). This is probably caused by only one third of the contact area having a mechanical interlock and this is attributed to incomplete mixing and voids in the resin, the condition of the hole, some shrinkage of the resin bolt, both from curing and from the tension load, and to the limited number of ribs on the rebar. Accordingly, a new analytical equation is recommended to predict the axial stiffness of resin bolts, using the load transfer lengths (decay lengths) observed in actual installations. More experiments are needed with strain-gaged bolts in actual installations. Even though the load transfer lengths of resin bolts are longer than commonly claimed, the resin bolt is still much stiffer than a mechanical bolt.

Shear stiffness data appears to be limited to resin-grouted bolts in Indiana limestone. More experiments are needed for softer rocks typical of coal mine roof. The shear strength of resin bolts in soft rocks may be limited by the bearing strength of the rock in contact with the bolts. A formula is given to determine the thickness of rock layer required to carry the bearing stress. In thin layers the bearing strength (and the resulting shear strength of the bolt installation) is shown to be minimal.

Actual beam building using fully resin-grouted nontensioned bolts appears to be small. From previous work the reinforcement is dependent on two factors: the shear stiffness of the bolt and bolt density. It was found that the theory was lacking in predicting reinforcement in soft rocks. Based on the large number of observations in actual mine practice and a small number of experiments, the authors believe that one should not use beam building as a design mechanism at this time. Although beam building does seem to be a valid means of roof support, more theoretical and experimental work is needed to better understand the interaction of the bolt and rock to form a competent beam. The concept of combined beam building and suspension may explain the success of the resin bolt in mine roof control. Fully grouted resin bolts provide continuous support over the total length of the bolt.

Both resin bolts and mechanical bolts can increase the buckling strength of thinly-laminated roof layers by decreasing the critical buckling length from the span of the opening to the bolt spacing. However, thin layers of 1-inch or less in thickness may still buckle between bolts on 4-foot centers. Header-boards are recommended to further increase the buckling strength - by reducing the distance between supports.

"Beam suspension", where a roof beam is supported by a bolt from competent rock above the beam, is a roof support mechanism that has been overlooked in the past. In comparison to simple suspension of loose masses of rock where anchorage is the important criterion, beam suspension depends also upon the stiffness of the bolt to stiffen the beam and prevent roof sag. One resin bolt at midspan can serve the same purpose as a post at midspan, i.e. the roof span is decreased by one-half. For the purpose of roof beam suspension, a resin bolt is shown to be five times more effective than a mechanical bolt. It may be possible to grout a resin bolt in the compression zone of a soft roof and still support the beam.

Design criteria are given to determine the bolting requirements to support a Voussoir rock arch in blocky ground. The arch is expected to form under shallow cover when the horizontal in-situ stresses are low. Angled bolts at decreased spacings are recommended to carry the shear at the ribs, to prevent ultimate failure. Planks are recommended to prevent ultimate failure between the angled bolts. Tensioned resin bolts, in the second row in from the pillar, may be required to accomplish the shear transfer between horizontal joints in the arch. Bolts at midspan need only accomplish a simple suspension of lower rock from the arch.

In roofs with numerous slips, or in a roof of mudstone that deteriorates easily, angled bolts over the ribs with planks or slings are recommended to support the dead weight of the roof. Angled bolts can be used to advantage in bolting across the maximum shear lines. These lines are nearly vertical at the ribs when the ratio of the horizontal to the vertical stress is low.

It is shown that leaving top coal in the roof can provide a compression zone in the immediate roof. In addition to the advantage of the compression, the coal can serve as a vapor barrier to roof shales that are susceptible to weathering.

Bolting of cohesionless material generally will require tensioned bolts, with the distance between bearing plates not greater than three times the average fragment size of the loose rock.

Non-tensioned resin bolts have been shown to have limitations if bed separations occur before the bolts are installed. Early bolting is called for. "Forepoling" can also reduce bed separations. In this method the bolts are installed at an angle in the roof over the face before the face is advanced.

Bolting of intersections poses additional requirements. Higher bolt densities or larger bolts are required to build or suspend plates rather than beams over intersections. Higher bolt densities are required to prevent buckling of thin plates of roof rock, if a biaxial compressive stress field exists in the roof.

Building an arch (or dome) over an intersection presents particular problems. Extra bolts in the row in the entry, at the edge of the intersection are recommended to support the arch. Mine experiments would be in order to check out this concept.

5.3 Establishment of Design Criteria:

Little or no success was obtained in applying theoretical approaches to beam building in actual mine situations. No way could the authors see how to build beams or arches out of mixed, broken, weathered rocks. Yet bolted roofs usually behaved better than unbolted roofs and that resin bolts usually worked better than mechanical bolts.

5.3.1 Review of Lab and Theoretical Approaches:

A more critical review of lab model results and theoretical approaches confirmed our suspicion that the reinforcement afforded by non-tensioned, resin-grouted bolts appears to be small in most coal-mine rocks. The beam, column and arch mechanisms all rely upon resistance to shear along pre-existing planes (bedding planes, joints, slips, faults) and, unless non-tensioned bolts are installed very close together, maybe on 10-inch centers, may not provide that reinforcement.

a) Non-tensioned bolts do not provide a clamping force to make use of friction or interlock on the planes of potential movement.

b) Direct resistance to shear is concentrated at the re-bar/resin/rock contact, and the stress there is high enough to cause coal-mine rocks to yield a little - so the reinforcement effect is reduced.

5.3.2 Support is Mainly Through a Suspension Effect:

The authors found that most of the failures and successes of resin bolting could be described in terms of suspension, and a realistic design criteria could be based on suspension. In those rocks which are not self-supporting the load is defined to be suspended, and the stable zone from which to suspend it needs to be established.

a) The Load: This is the rock which is likely to fall down. It may be weak, weathered, thin layered, disrupted by slips, faults, concretions, tree stumps or coal seams, blast damaged, or damaged by excessive stresses - either vertical or lateral.

b) The Stable Zone: The simplest case is where a strong bed of rock exists. More often - whether we know it or not - the stable zone consists of compressed and moderately-stressed rock above the opening. The lateral compression which is present in almost all mines keeps that zone stable.

   If the roof rocks are too soft or too badly crushed to be stable within reasonable bolt length, then the dead load must be suspended from rock compressed above the ribs or pillars.

5.3.3 Designing the Support System:

a) Load and Stable Zone Must be Defined: This work must be done in the mine, by observing roof failures, examining holes in the roof, and perhaps by installing sagmeters, or other simple instrumentation.

The weight of the load can be figured from a table giving typical weights of mine rocks.

b) Bolt Strength can be Selected: Tables are given for strength of Grade 40 and Grade 60 rebars. Strengths can be chosen to give reasonable bolt spacing - and this does often turn out to be 4-foot to 5-foot centers.

c) Length of Bolt can be Determined: A table is given showing how many inches of resin are needed to provide a ton of anchorage capacity in various rocks, with various rebar and hole diameters. A small safety factor is added because of inadequate mixing usually found at the end of the hole.

Total length of bolt is then equal to thickness of potential dead load plus length of anchorage. Again, this usually turns out to be reasonable, around four to six feet for vertical bolts, and eight feet for bolts inclined over the ribs.

5.4 Examination of Current Code of Federal Regulations, Title 30, Part 75:

This study has shown that more investigation must be done before any major changes in the Code are made, since full understanding of resin bolting as a mine roof reinforcement device has not been achieved. Roof behavior in actual mines needs to be monitored to determine what conditions can be classified as stable or unstable. A rock mechanics research program for the entire coal mining industry is needed. Such a program could be set up under the present code Section 75.200-1 with all data sent to a central collection point. This would allow the definition of stable rock zones, bolt burdens, width of openings, pillar sizes, etc. for different geological conditions resulting in "high confidence" designs of roof control systems. This seems like the only logical direction in which real significant contributions to roof control will be made, thus allowing a good working code for fully resin-grouted bolts to be determined. Within Section 75.200-7, some mention of fully resin-grouted non-tensioned roof bolts needs to be made. Under item (b) of this section a section should be added reflecting current recommended installation procedures for resin bolting. No good method for testing the fully resin-grouted bolts presently exists. A method needs to be developed.

5.5 Recommendations for Future Work:

Define and control those factors which cause roof failures. A better understanding of such factors as moisture effect, geological structures, relationship between surface topography and mine stability, and so on must be developed.

Find cheaper and better techniques to define conditions around real mine openings. States of rock stress and strength around mine openings need to be actually measured, so that we can design more realistically. We seek large numbers of approximate measurements, knowing that a few precise measurements are not likely to be representative.

Improve bolt anchorage in soft rocks. Field studies indicate that even resin anchorage creeps in soft rocks but the "debondable" bolt might solve that problem. It develops a "pressure bulb", like some soil anchors.

Investigate the effects of tensioned bolts in soft rocks. We suspect that they can do more to prevent shear than can untensioned bolts, but we do not know how effective they are in relatively soft coal-mine rocks.

LITERATURE REVIEW

1.0 INTRODUCTION

This review is the first task of the research project, "Design Criteria for Roof Bolting Plans Using Fully Resin-Grouted Nontensioned Bolts to Reinforce Bedded Mine Roof." The objective of this review is to classify and identify from the literature available data on the factors governing the effectiveness of roof bolting systems using fully-grouted, nontensioned bolts for reinforcing bedded mine roof.

Three areas of related research have been reviewed, namely resin bolting, mine analysis, and coal and rock properties. It is not sufficient to consider resin bolting per se, because roof bolting plans require an analysis of roof bolt interaction with the surrounding mine structure and the latter analysis requires knowledge of the mechanical properties of the ore seam (coal) as well as the adjoining rock strata.

More than 250 references, both domestic and foreign, have been gathered during the course of this literature search. Those references considered pertinent to the research project are reviewed in Sections 2-4. The more significant results are discussed in some detail whereas less pertinent references are given only a cursory review, but the latter are included for possible importance in future related work.

This review related specifically to bolting of horizontally bedded mine roof with flat backs as shown in Figure 1.1. Many different in situ conditions can be encountered as indicated in Section 3.1 of this report. Conventional mechanical bolts have been used extensively for a long time, but the fully-grouted resin-bonded bolt has found use in recent years and is the topic of this study. The configuration of a fully-grouted bolt is shown in Figure 1.2. It consists of a steel rebar of diameter d_1 and length L fully bonded by a plastic resin into a drilled hole in the rock of diameter d_2. A steel plate is used against the back to hold up loose rock (commonly called "rash"), and the plate is supported by the bolt head and sometimes by a threaded nut.

Carr [1.1] lists several advantages of the full column resin anchored bolt (or dowel):

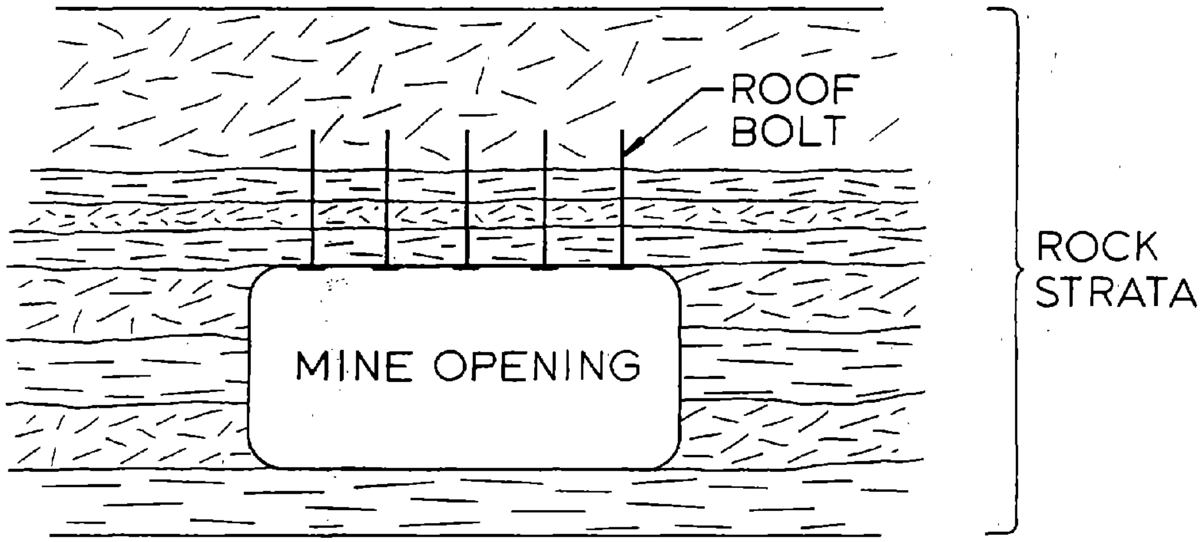


Figure 1.1 Roof-Bolting of Horizontally-Bedded Rock Strata

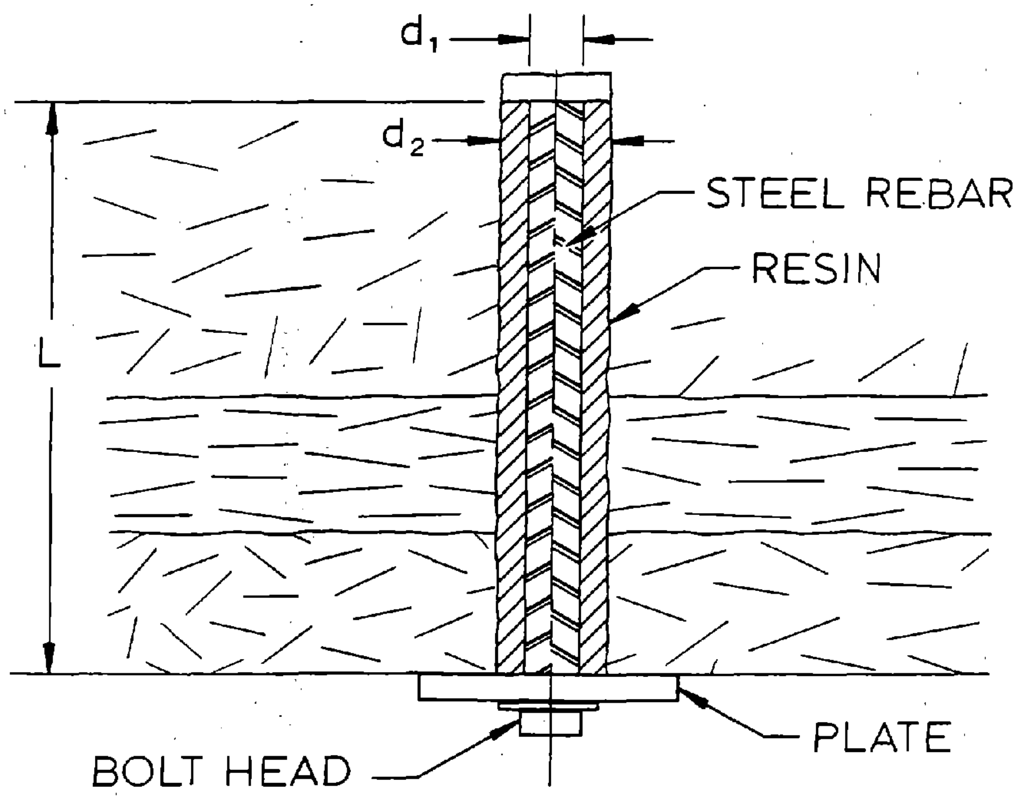


Figure 1.2 Fully-Grouted Resin-Bonded Bolt

(1) Anchorage is virtually guaranteed and independent of the operator or strata type.

(2) The bolt is permanently effective throughout its whole length.

(3) There is no loss of effectiveness by rock failure at the mouth of the hole.

(4) The hole length is not as critical as with the other bolts.

(5) The fully grouted bolt resists lateral as well as vertical forces.

(6) The fully grouted bolt seals wet holes, excludes air and reduces weathering.

(7) The resin grouted bolt is resilient to shock loads from blasting and does not lose tensile load as mechanical bolts do.

(8) Installation is simple and the time and effort of tensioning is eliminated.

Some of these advantages listed by Carr are questionable, item 4 in particular. However, many other investigators have tested the resin bolt concept and a review of their work does verify and add some certainty to the claims made for resin bolting.

During a workshop at a roof control conference in Rolla in June 1975 [1.2], recommendations were made on research and development needs on the resin-bolt-package concept. These were:

A. Develop design data and criteria for fully grouted bolts.

B. Develop proper installation techniques to assure quality control.

C. Monitor stresses along the bolt length to understand the bolt function.

D. Determine the roof support interaction between the grouted bolt and the plate or header or metal strip or timber.

E. Do research to develop an expansive resin to safeguard against shrinkage.

F. Determine whether or not uplift provided by the roof bolter during the holding period when the resin is setting up is good or bad for roof control.

G. Assess the proper geometry for resin bolting, hole size vs. steel size vs. quantity of resin.

H. Develop a testing method which will adequately record the effectiveness of a fully bonded bolt.

I. Improve education and communication between the government, suppliers and the users.

Recommendation A is the essence of this present research effort.

Corwine [1.3] gives a review of other currently supported research projects on roof control.

Moy [1.4] gives a review of roof bolting practice as of 1971. However, there has been considerable development of the resin-bolt concept since then.

1.1 References

[1.1] Carr, F., "Recent Developments in Strata Bolting in National Coal Board Mines in the United Kingdom," Paper No. 11, Symposium on Rock Bolting, February 17-19, 1971, The Australian Institute of Mining and Metallurgy, Illawarra Branch.

[1.2] Scott, J. J., "Research and Development Priorities--Roof Control Conference," USBM Contract #H0242034, University of Missouri-Rolla, Rolla, Missouri, April 15-16, 1975.

[1.3] Corwine, J. W., "Review of Roof Control Technology Research," Mining Congress Journal, pp. 25-29, January 1976.

[1.4] Moy, D., "The Design and Use of Rock Bolt Systems," Master's Thesis, Dept. of Mining and Mineral Technology, Royal School of Mines, Imperial College of Science and Technology, London, December 1971.

2.0 RESIN BOLTING

2.1 Resin Properties

An important element in the fully-grouted bolt assembly is the resin grout (Figure 1.2). Resins have been used not only for grouting bolts but for rock impregnation in general [2.1, 2.2]. Thus, research data on resins in general are considered, because these data can contribute to the knowledge of resin bolting.

Chemical Properties

Thermosetting resins are used in mine applications. The thermosetting resin forms a cross-linked polymer chain that is relatively strong. The thermo-plastic resins on the other hand require heat for molding and form relatively weak linear polymer chains.

The thermosetting resin is formed by a chemical reaction (exothermic) between a resin and a catalyst (curing agent). The thermosetting resin that has found wide use today is the polyester type resin. The epoxy type has been tried, but the polyester offers these advantages [2.1-2.5]: less expensive and faster cures at mine temperatures. The latter is important because the resin achieves its strength as it cures--a faster cure means that the strength of the resin is achieved in a shorter time.

An inherent disadvantage of the polyester resins is excessive shrinkage when the liquid mixture changes into a solid. About 8-17% shrinkage occurs in the pure resin. Accordingly fillers are used to reduce shrinkage to <1%. Fillers also reduce resin costs. Addition of thermoplastic resins as fillers are used at high temperatures in press molding operations but will not work at low temperatures [2.1, 2.2]. For use at mine temperatures, mineral fillers such as quartz and limestone are used [2.1, 2.5]. However, mineral fillers reduce the inherent ductility of the plastic resin and make it more brittle (nearly as brittle as rock, much more brittle than steel and possibly more brittle than some soft rocks in coal mines).

A question of some concern is the existence or non-existence of a chemical bond between the resin and rock or resin and steel. Daugherty [2.5] (using a Cyanamid resin) and others claim a chemical bond is formed. Brookhaven National Laboratory (BNL) [2.2] found that a chemical bond (polymer bridge) does not form but that the bonding present is all of a physical nature by relatively weak Vander Waal's forces. BNL added "silane" coupling agents to promote chemical bonding, which was achieved in laboratory tests, but pull tests in mines showed little or no chemical bonding, particularly in wet bolt holes. Monsanto [2.1] also had much difficulty achieving adhesion with polyesters.

Cyanamid appears to be the first to have come out with a resin for grouted bolts [2.6], called ROC-LOC. However, they originally quoted long gel times of 20 minutes, which are impractical for production mining. The two large suppliers in the United States today are CELTITE and DU PONT, who both use a polyester resin with shorter gel times. Different viscosities of the resin are also available. For example, CELTITE [2.7] provides resins of three viscosities "high" for installation temperatures above 60°F, "medium" for temperatures below 60°F and "low" for extremely low temperatures or extremely large holes. Also, they provide resins with different set times from one minute to 20 minutes. The set time or gel time depends on the temperature as shown in Figure 2.1 for a DU PONT FASLOC resin with a reference set time of one minute at 60°F.

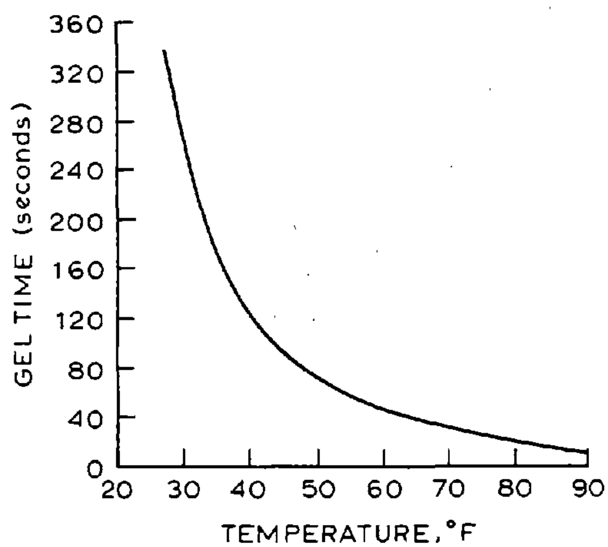


Figure 2.1 Gel Time vs. Temperature for DU PONT FASLOC Resin [2.8]

General Electric Carboloy Division has recently come out with its own polyester resin capsule, patterned after the English ICI capsule, where the catalyst is enclosed in an inner casing at the center of the package [2.89]. The current GE capsule contains the catalyst in an inner tube attached to the outside casing.

Reference [2.90] gives gel times for three grades of ICI "Rotaset" resin at different temperatures.

Celtite (Selfix) in England markets an epoxy resin as well as a polyester resin for grouted bolt applications. The epoxy resin is recommended for structural applications.

Rexnord Inc. [2.104] also markets a polyester resin for roof bolting. It is marketed under the name "Nordbak" and features a color change upon hardening that indicates whether the resin and hardener are properly mixed the correct time. Two different resins are available, one that hardens in 40 seconds and one that hardens in 100 seconds. Four different cartridge diameters for 1, 1-1/8, 1-1/4 and 1-3/8 inch diameter holes are available for use with bolts of 3/4 and 7/8 inch in diameter.

The chemical composition of resin used by The National Coal Board in England is reported to be a stabilized unsaturated polyester resin mastic containing 75% limestone filler for the resin component, and a benzoyl peroxide paste with 30% active peroxide organic plasticizer and limestone filler for the catalyst component. U.S.A. resins are reported to contain generally polyester resin and benzoyl peroxide. Exact percentages are not reported but are secret. A special variation of the polyester resin called E-C-Phix is supplied by SELFIX for the particular application to wooden dowels.

Physical and Mechanical Properties

The specific gravity of filled polyester resins in general is in the range of 1.8 to 2.2 [2.1]. Of the commercial resins used in the United States, the value for CELTITE resin, for example, is 1.85.

The mechanical properties of roof-bolt resins and other filled polyester resins are reported in Table 2.1. Data on the DU PONT resin was not found in their literature. Michigan Tech tests on CELTITE resin show a higher modulus value in tension than those reported by SELFIX (original British trade name for CELTITE) for compression and also show an extremely low ductility of .2%. Flexure modulus values reported on another filled resin, ALTEK 71-63, are

Table 2.1 Mechanical Properties of Mineral Filled Polyester Resins

Type	Tensile Strength, psi	Compressive Strength, psi	Shear Strength, psi	Modulus of Elasticity, psi	Poisson's Ratio	Elongation	Ref.
Celtite	2,500	16,000	7,500	----------	-------	--------	[2.7]
Selfix	2,500	17,300	-----	$.38\text{x}10^6$ compression	.30	--------	[2.10]
Celtite	2,650	------	-----	$1.4\text{x}10^6$ tension	-------	.2%	[MTU]
Altek (71-63)	4,400.- (flexure)	12,500	-----	$1.3\text{-}1.6\text{x}10^6$ (flexure) $1.9\text{x}10^5$- compression	0.30	---------	[2.1]
Cyanamid EPX-295-1	4,200-8,900 (flexure)	------	-----	----------	-------	--------	[2.9]
Cyanamid EPX-289-4	1,600-3,600	------	-----	$(.68\text{-}.9\text{x}10^6)$ flexure	-------	--------	[2.1]

---indicates properties not available

about the same as the tensile modulus found by Michigan Tech. Flexure strengths are higher than tensile strengths, as expected, because flexure is a combination of tension and compression. The compression modulus value for the ALTEK resin is also an order of magnitude low, leading to the general conclusion that the compression moduli for filled polyester resins are lower than the tensile moduli. Compressive strengths are much higher than the tensile--much like rock. The shear strength reported by CELTITE appears too high if one considers a Mohr-Coulomb failure criteria. Figure 2.2 shows how the strength of the filled polyester resin varies with temperature, decreasing at higher temperatures as shown. Mechanical properties of the "Rotaset" resin are reported in [2.91] as follows:

Compressive Strength = 86.8 N/mm^2 (12,600 psi)

Beam Tensile Strength = 22.1 N/mm^2 (3,200 psi)

Strengths of polyester-rock bonds are also reported in the literature. Flexure strengths of 60 mil slate-resin-slate sandwiches [2.9] ranged from 6000 psi to 12000 psi with a flexure modulus of 1.86 to 3.5 × 10^6 psi. The shear strengths in double shear of resin-bonded shale specimens were 450-610 psi. BNL [2.2] reports bond strengths of 220 psi (wet) to 620 psi (dry) for laboratory pull-out tests. Askey [2.4] reports a bond strength of 350 psi in coal and implies higher values for shale. (Additional resin properties were found and are discussed in Volume 2 of this report.)

2.2 Axial Loading of Resin-Bolt-Rock Assemblies

2.2.1 Experimental Data For Axial Loading

A common test of roof bolt performance is the pull test. Askey [2.3] found a difference in results for a ribbed bar (rebar) and a smooth bar for both bars fully bonded with resin, as shown in Figure 2.3. He also found that the fully bonded resin bolt has a higher pull load than the mechanically anchored bolt.

DU PONT advertises pull test loads of 37,000 lbs for 3/4 inch resin bolts as compared to 16,000 lbs for the same size mechanical bolts [2.8]. They report that with the resin bonded bolts, the bolt head breaks with the bond still intact. Much, however, depends upon the competency of the rock and the strength of the bond. In soft rock, shear occurs at the resin-rock interface and the bolt pulls out as shown in Figure 2.4 [2.11, 2.17]. The load-extension curve also exhibits different behavior for soft rock than for hard rock as shown.

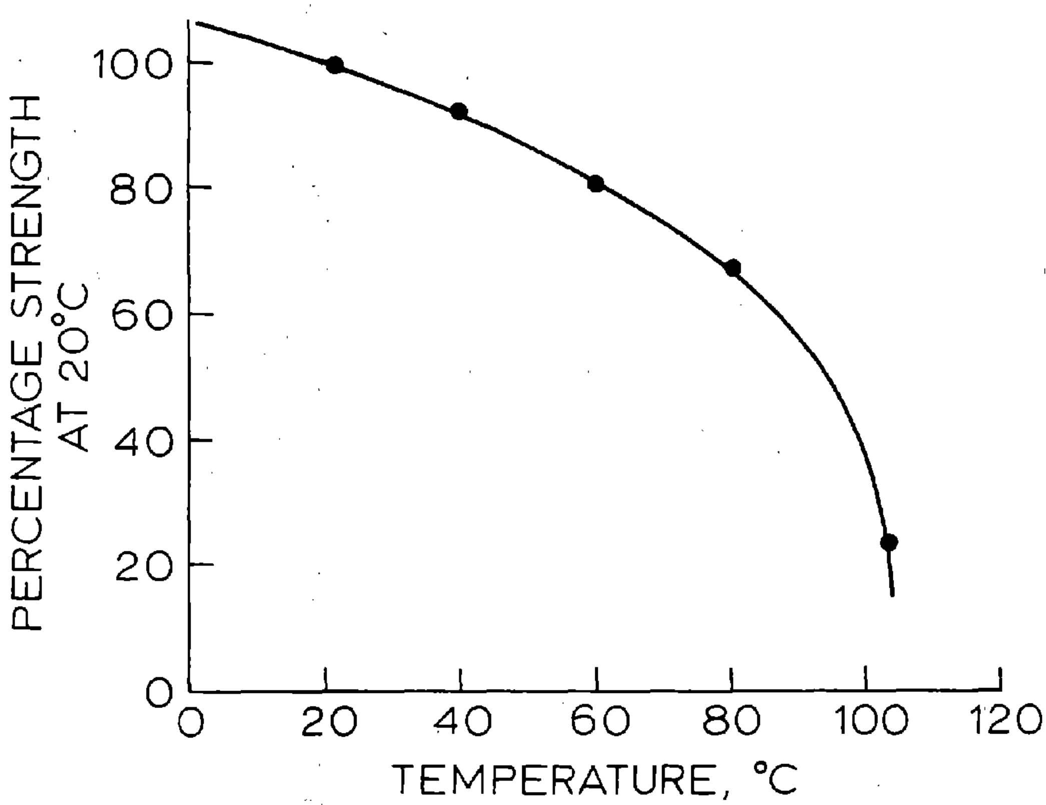


Figure 2.2 Strength of Filled Polyester Resin (ICI) at Various Temperatures [2.90]

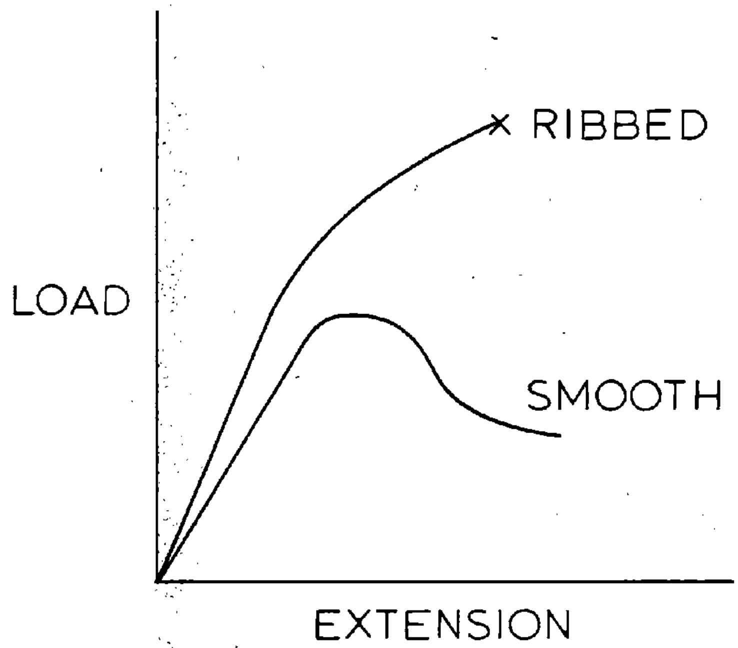


Figure 2.3 **Pull** Tests of Resin Bonded Bolts, **Ribbed** versus Smooth [2.3]

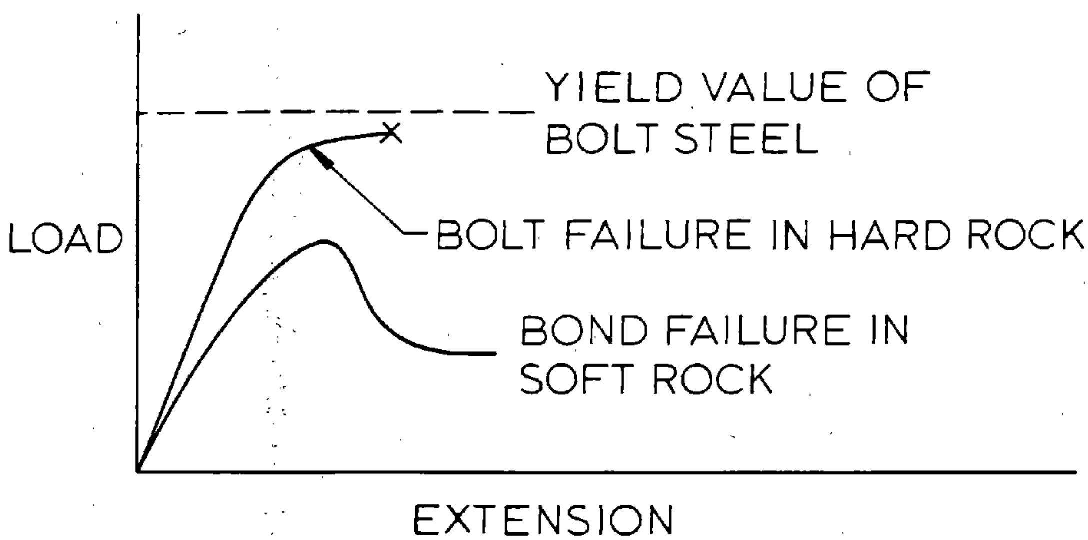


Figure 2.4 **Pull** Tests of Resin Bonded Bolts, **Bond** Failure versus Bolt Failure

The resistance to pullout is only one factor. Another important factor is the axial stiffness of the resin-bonded bolt assembly. The fully-bonded resin bolt is stiffer than a mechanical bolt because the load is carried by a much shorter length of the bolt. In fact, part of the same bolt can be in compression and part in tension because of differential rock movement in the roof [2.12].

Stefanko and Choi [2.13] show that the resin bolts are much stiffer (stretch less) than mechanical bolts, from tests in shale and sandstone. Others all show similar results [2.6, 2.14-2.18]. These results for fully grouted bolts are summarized in Table 2.2, where the axial stiffnesses (spring constants) are given for extensions up to 0.1 inch. Lower stiffnesses (but still higher than mechanical) are found for partially grouted resin anchored bolts [2.6, 2.11, 2.19-2.21]. The axial stiffnesses for fully grouted resin bolts are ten to 20 times larger than for mechanical bolts [2.15, 2.16, 2.19].

References [2.3, 2.4, 2.22] also discuss pull tests on bolts.

The results in Table 2.2 are for fully grouted bolts. When bolts are fully grouted only a portion of the length is loaded and the pull test results for axial stiffness are not dependent upon the overall length--if sufficiently long. Results by Dunham [2.11] show that bonded bolt lengths of 25 or more diameters appear to be sufficiently long to develop sufficient load to break steel bolts (for sufficiently strong rock).

Dunham [2.11] also experimented with the effect of hole diameter and found that too large a hole size results in inadequate mixing and very low pull loads. He recommends the following hole diameter for optimum results:

$$d_2 = d_1 + (8 \text{ to } 12 \text{ mm}),$$

or (2.1)

$$d_2 = d_1 + (0.3 \text{ to } 0.5 \text{ inch})$$

(Resin suppliers are recommending a difference of 0.25 to 0.375 inch between the hole diameter and diameter of the bolt.)

Dunham also investigated the differences in anchorage between wet and dry drilling and rotary and percussion drilling and found them negligible. He also investigated the effects of installing resin bolts at temperatures of -1°C and -7°C.

Table 2.2 Axial Stiffnesses (Force/Elongation) For Resin Bonded Steel Bolts

Diameter of Bolt, inch d_1	Diameter of Hole, inch* d_2	Rock Material	Axial Stiffness K, kip/inch	Ref.
1.0		Shale	360	[2.13]
1.0		Sandstone	280	[2.13]
0.625-1.0		"Sound firm strata"	250-1000	[2.15]
0.91	1.06** mim	Mudstone, Sandstone	400	[2.14]
0.75	1.0	Indiana Limestone	500	[2.16]
0.50	1.0	Indiana Limestone	290	[2.16]
0.75	1.375	Indiana Limestone	1390	[2.16]
0.875	1.375	Indiana Limestone	1130-3130	[2.16]
0.79	1.18	Sandstone and Diorite	450	[2.17]
0.87	1.18	Sandstone and Diorite	450	[2.17]
1.0	1.375	Sandstone and Diorite	250	[2.18]
1.375	1.75	Sandstone and Diorite	700	[2.18]

* See Figure 1.2
** Minimum

At -1°C there appeared to be little change in anchorage capacity, but at -7°C there were drastic reductions in capacity, believed to be caused primarily by too much dissipation of heat from the exothermic resin reaction and consequent prevention of proper curing of the resin.

Creep of resin-bonded bolts in sandstone was also studied by Dunham [2.11]. It was found that the load on the bolt has to be increased to near the pull strength before significant creep occurs if the bonded length is sufficiently long (~30 diameters). For shorter length (~15 diameters) which have bond failures rather than bolt failures, the load has to be at least 75% of the pull strength to cause appreciable creep.

Haas and Patrick [2.16, 2.24] found similar creep when increasing the load to two-thirds the yield strength of the bars. Haas and Patrick found other interesting results: (1) that the load is redistributed with time a greater distance along the bolt, and (2) that stress relaxation occurs after pretensioning resin bolts. After one month, 38-68% of the load remains, depending on the diameter of bolt and type of resin used (and rock type).

Creep rates are listed in Table 2.3. The ranges of creep given in [2.16] vary with the resin type used. Comparing values in Table 2.3 and ignoring other variables, one finds, as one would expect, that larger creep occurs for bars bonded in the softer sandstone than in the relatively harder limestone.

Liddle and Tisdale have measured stress relaxation in bolts in mines [2.25]. They report that anchor capacity is much less for anchorage in coal than in shale.

Another important result of experiments concerns the load transfer along the length of the resin bolt. Experimental results agree qualitatively with theory, but not quantitatively. Theory predicts a short decay length for load transfer and a nonlinear load distribution. Experiments show a larger decay length (but still relatively short) and a more linear distribution due to actual bond failure or less than perfect bonding. Extensive measurements have been made by Haas et al [2.16] for bolts bonded in limestone. For example, Figure 2.5 shows results of strain gage measurements on a typical resin bolt installation. Theoretical results were obtained by the finite element method. Experiments show that the load decays to zero along the bolt within 16-20 inches for one type resin and within 20-32 inches for another type resin. Evidently poor resin properties or mixing problems can be a factor in causing large decay lengths, because the shorter

Table 2.3 Creep Rates on Resin Bonded Bolts

Diameter of Bolt, inch d_1	Diameter of Hole, inch [a] d_2	Rock Material	Load, lbs.	Creep Extension	Ref.
0.79	1.10	Scotswood Sandstone	16,400	0.016 in./10 days	[2.11] [b]
0.79	1.10	Dunhouse Sandstone	16,400	0.027 in./10 days	[2.11] [b]
0.875	1.375	Indiana Limestone	8,000	0.0007-.0029 in./30 days	[2.16] [c]
0.75	1.375	Indiana Limestone	8,000	0.012 in./30 days	[2.16] [c]
0.75	1.0	Indiana Limestone	8,000	0.014 in./30 days	[2.16] [c]
0.50	1.0	Indiana Limestone	3,540	0.020 in./30 days	[2.16] [c]
0.875	1.375	Indiana Limestone	16,000	0.004-0.016 in./30 days	[2.16] [c]
0.75	1.375	Indiana Limestone	16,000	0.007 in./30 days	[2.16] [c]
0.75	1.0	Indiana Limestone	16,000	0.034 in./30 days	[2.16] [c]
0.50	1.0	Indiana Limestone	5,890	0.090 in./30 days	[2.16] [c]

(a) See Figure 1.2

(b) Bonded length of 16 diameters

(c) Bonded length of 24 inches

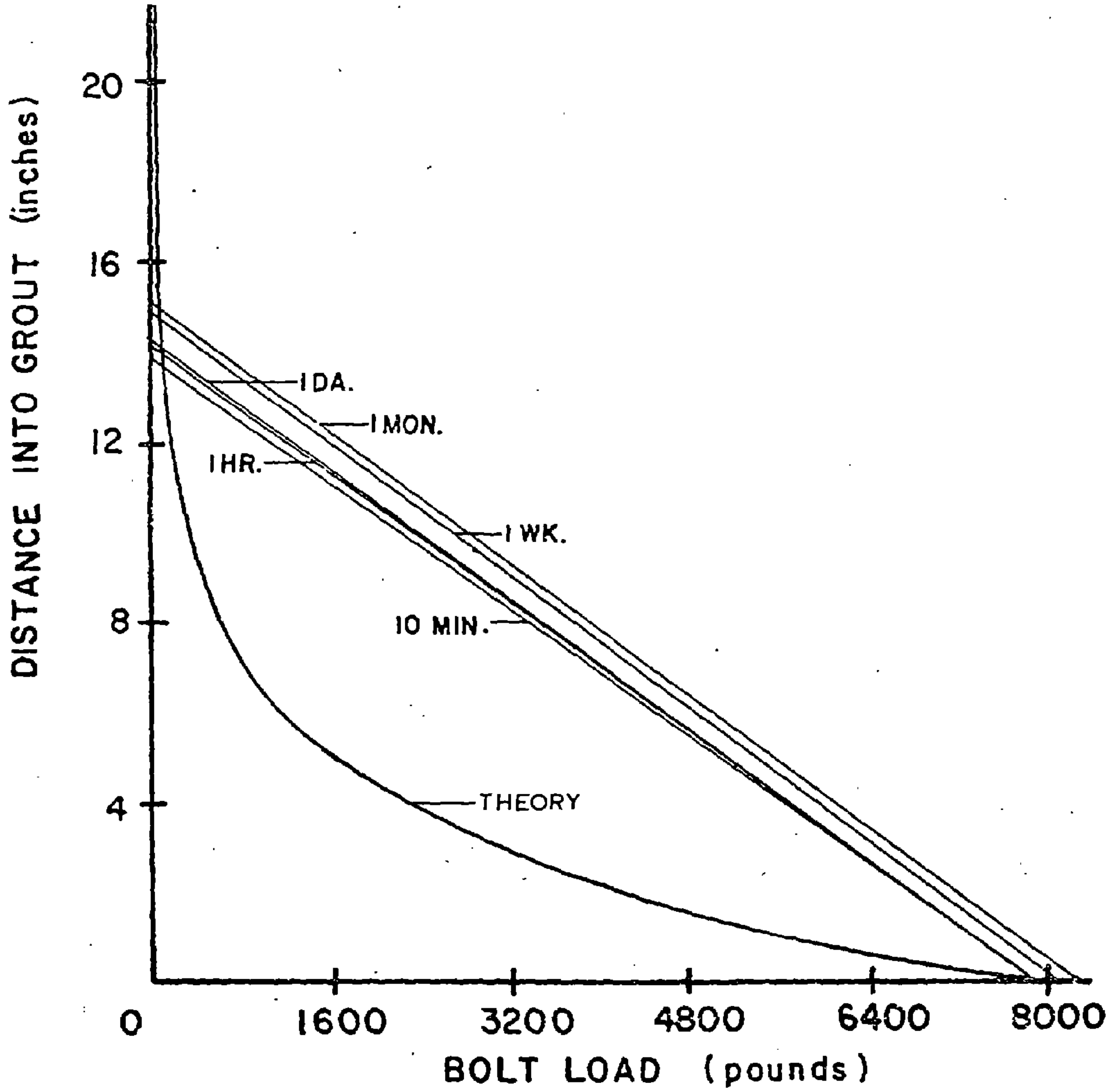


Figure 2.5 Variation in Bolt Load with Distance into the Grout at Several Times (7/8 inch bar in 1 3/8 inch Resin Grouted in Indiana Limestone) [2.16]

decay length is to be preferred to take full advantage of the load differentiation characteristics of the fully bonded resin bolt.

Farmer and Dick [2.26] have conducted experiments in three materials: concrete, weathered limestone, and chalk using 20 mm (.787 inch) diameter steel bars resin grouted into 28 mm (1.10 inch) diameter holes. They found similar results. Results conformed closer to theory for concrete at low loads, less at high loads where bonding breaks down, and less for limestone and chalk where bonding breaks down at lower loads because of lower elastic moduli for these materials. In the softer rock the resistance is mainly frictional shear when the resin-rock interface breaks down. Decay lengths are shorter in concrete and longer in limestone and chalk.

Farmer and Dick's results show a nonlinear distribution for weathered limestone and concrete as shown in Figure 2.6. (The experimental results of Haas et al--shown previously in Figure 2.5, were actually not linear but were approximated by the best linear fit.) For chalk the distribution was nearly linear, as shown in Figure 2.6.

A creep test of an ICI "Rotaset" resin anchor is reported in [2.90]. Results are shown in Figure 2.7 for a 20 mm bar bonded over 350 mm length and loaded to 9.5 tons. The maximum creep extension was 0.5 mm after five days. Thereafter, negligible creep occurred for the 80-day test. The rock material in this test was not reported, but will have an effect on the magnitude of creep, as for example in [2.91], for the same size bar bonded in Scotswood Sandstone, where the creep at lower load (73 KN) was 0.7 mm in 18 days with most of this amount occurring in the first five days. The creep deformation was measured at the end of the bar.

Dunham [2.92] reports on field testing of 20 mm diameter EN8 ribbed bolts bonded with ICI resin in 28 mm diameter holes, 4 foot long. Bonded lengths from 178 mm up to 585 mm (7 inches to 23 inches) were used in five different rocks. Results are shown in Table 2.4. The first three materials were considered together because no significant difference in results was found. Dunham concluded that rock strength alone had no direct influence on anchorage capacity in his tests.

Although not reported, axial stiffnesses K (force/displacement) were calculated from the results (graphs) of [2.92]. For the first four materials in Table 2.4, the axial stiffnesses were about the same, K ≈ 20 kN/mm or 114 kips/inch. For gypsum, however, the stiffness was about half as much. (Modulus of elasticity values for these rocks were not reported.)

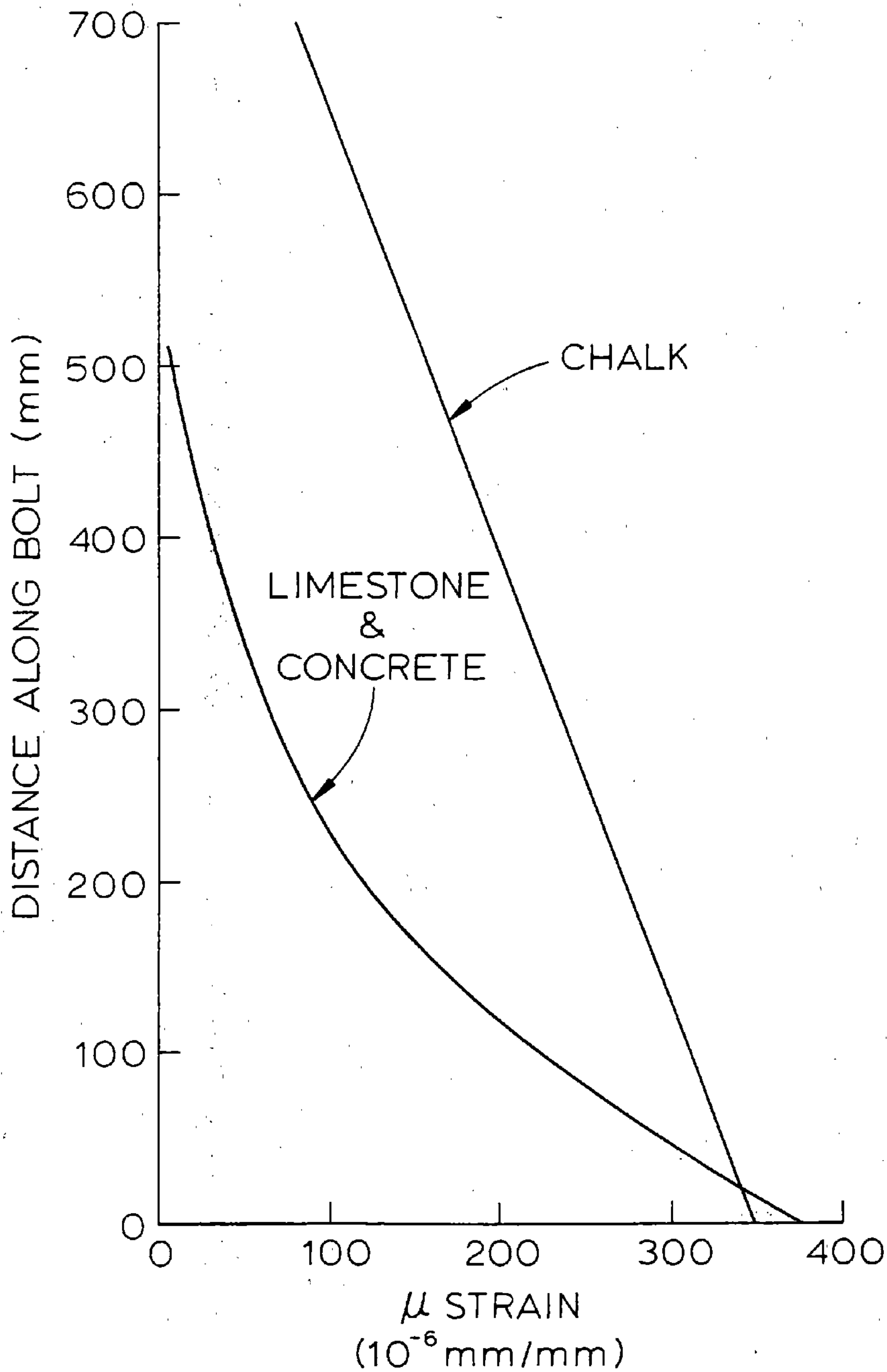


Figure 2.6 Strain Distribution in Resin-Grouted Bolts at a Load of 20 KN (Kilo-Newtons) [2.26]

Dunham [2.92] also investigated the effect of three hole conditions in a concrete block. Pre-cast holes were produced in the concrete by means of removable plastic tubes. Drilled holes were made in hardened concrete by percussive drilling. Thirdly, a steel/concrete anchorage (with no resin) was produced by embedding the bars in the concrete during casting. Results in Figure 2.8 show that bolts resin bonded in pre-cast holes failed at the lowest loads by slippage at the resin/concrete interface. For bolts resin bonded in drilled holes, the threaded portion of the bolt broke with the anchorage remaining intact. For the embedded bars (with no resin) the anchorage failed at a load slightly less than the yield strength of the steel, followed by a decrease in tension as the concrete progressively failed. These results showed that resin-bonded rebar in drilled holes gives a strength equivalent to embedded rebar, but smooth holes result in severe reductions in strength.

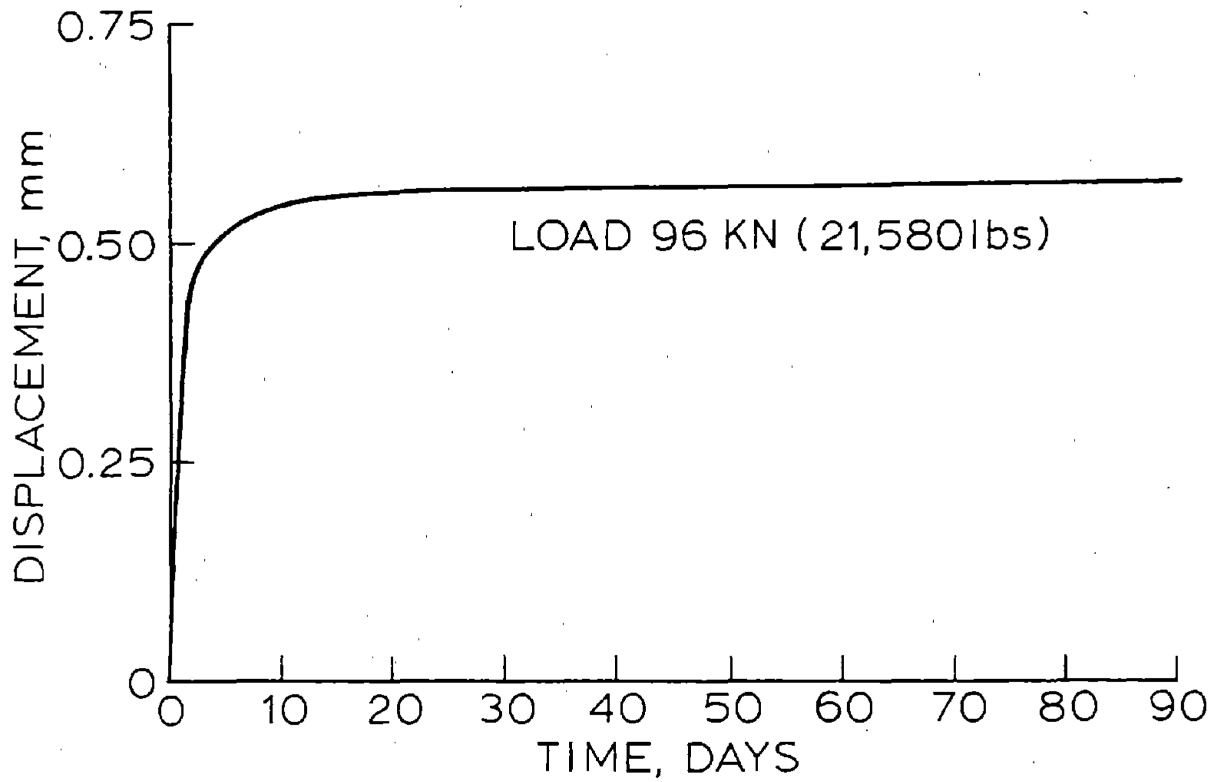
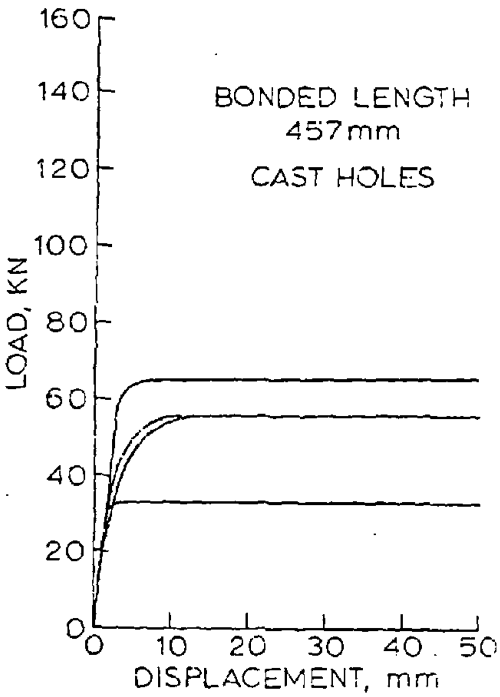


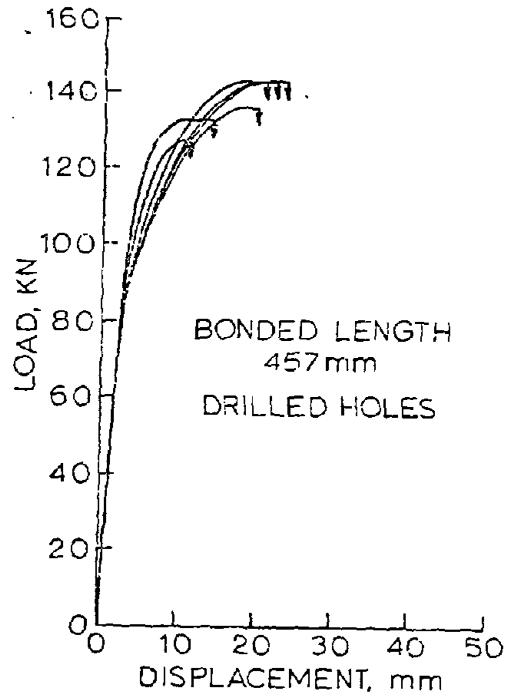
Figure 2.7 Creep of Anchorage under Constant Load [2.90]

TABLE 2.4 THE INFLUENCE OF BONDED LENGTH AND ROCK TYPE [2.92]

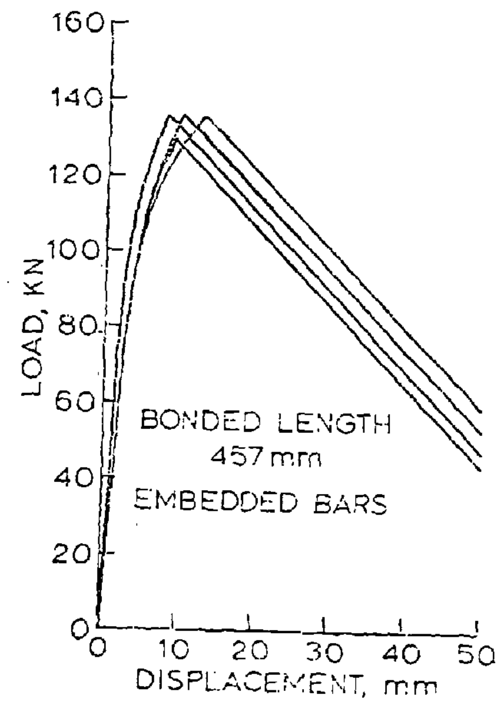
Rock Type	Uniaxial Comp. Strength MN/m^2	ksi	No. of Tests	'Bond Factor' (cm. of bond per KN of anchorage strength)	inch/kip
Four fathom limestone	136	20.	24		
Nattrass Gill Hazel	134	19.	26	0.25	.44
Shale	40	6.	26		
Anhydrite	175	25.	40	0.19	.33
Gypsum	35	5.	20	0.26	.46



(A) Pull tests in concrete - cast holes



(B) Pull tests in concrete - drilled holes



(C) Pull tests in concrete - embedded bars

Figure 2. 8 Effect of Hole Condition on Pull Strength of Resin Bonded Bolts in Concrete [2.92]

Karabin and Debevec [2.93] report some new results on resin bolting. Figure 2.9 shows the pull test results for 3/4-inch rebars bonded in various diameters of holes. The smaller hole size, 1.0-inch diameter, with a 1/4-inch diameter differential gives superior strength, with the strength decreasing with increasing diameter differential.

Figure 2.10 shows results of borehole size in weak rock, for resin bolts all with the same 1/4-inch diameter differential, i.e. 3/4-inch bars in 1-inch holes, 1-inch bars in 1 1/4-inch holes, and 1 1/8-inch bars in 1 3/8-inch holes. As shown, resin bolt installations with the larger hole size are superior in weak rock, because the shear stress at the resin/rock interface is lower for the higher surface area. (Failures in weak rock occur at the resin/rock interface.) The material used to simulate weak rock in this case was a mortar-sand mix with a 700 psi compressive strength, and a 100 psi shear strength.

Bolt hole roughness was also investigated [2.93]. Figure 2.11 illustrates the roughness of each hole diameter. Figure 2.12 shows results for resin bolt installations, all again with the preferred 1/4-inch diameter differential. The 1 3/8-inch and 1 1/4-inch diameter hole results again are ordered the same as previously in Figure 2.10, but this 1-inch diameter hole installation now shows superior results because of the added shear strength at the resin-rock interface due to the rougher hole. (The material used in this case was a cement with a 3000 psi compressive strength, and a 250 psi shear strength. These strengths are comparable to shales overlying coal seams.)

Daws [2.97] studied the strain distribution along resin bonded bolts in concrete. Nearly identical results to those of Farmer and Dick in Figure 2.6 were found by Daws.

Daws [2.97] also investigated the effect of load transfer by bonding strain gages to the hole surface in a block of Darley Dale sandstone. Strain gages were also bonded to a 19 mm rebar which was resin bonded into the 43 mm hole to a depth of 0.2 m. The strain at the borehole walls was found to be only 12-14% of the strain in the bolt at the same axial location. It is agreed with Daws that these results are very significant, but for a different reason. This does not mean the load transferred to the rock is low. Because the axial strain was low, it means the load is transferred to the rock primarily by shear strain. A strain gage does not measure shear strain when mounted parallel to the direction of shear. As a result of faulty reasoning [2.97], it was suggested that bond stress calculations should be abandoned. They should not be.

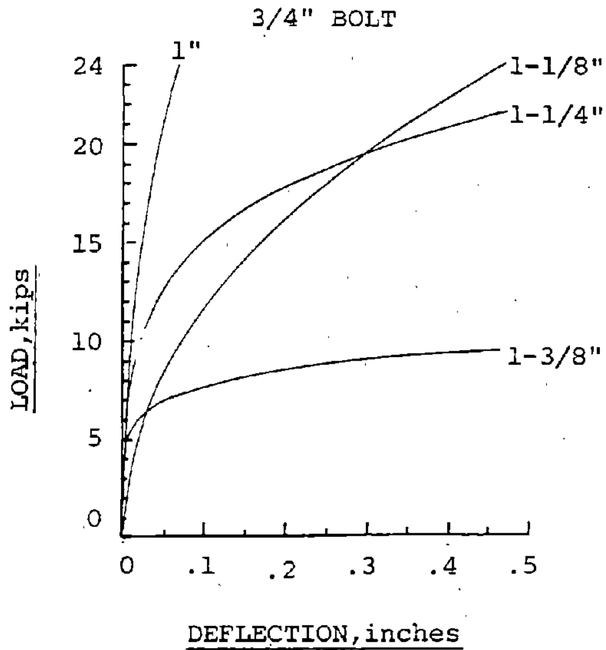


Figure 2.9 Pull Test Results of 1-foot Lengths of 3/4-inch Reinforcing Bar Grouted in Various Boreholes [2.93]

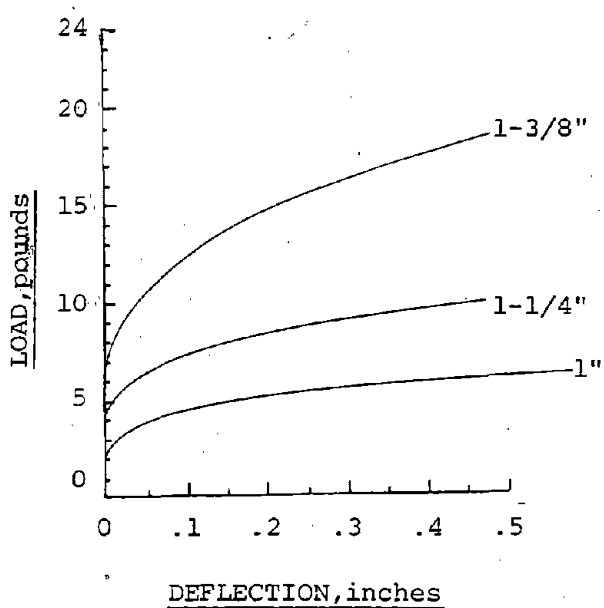


Figure 2.10 Pull Test Results Illustrating Effects of Borehole Size on Anchorage in Weak Rock [2.93]

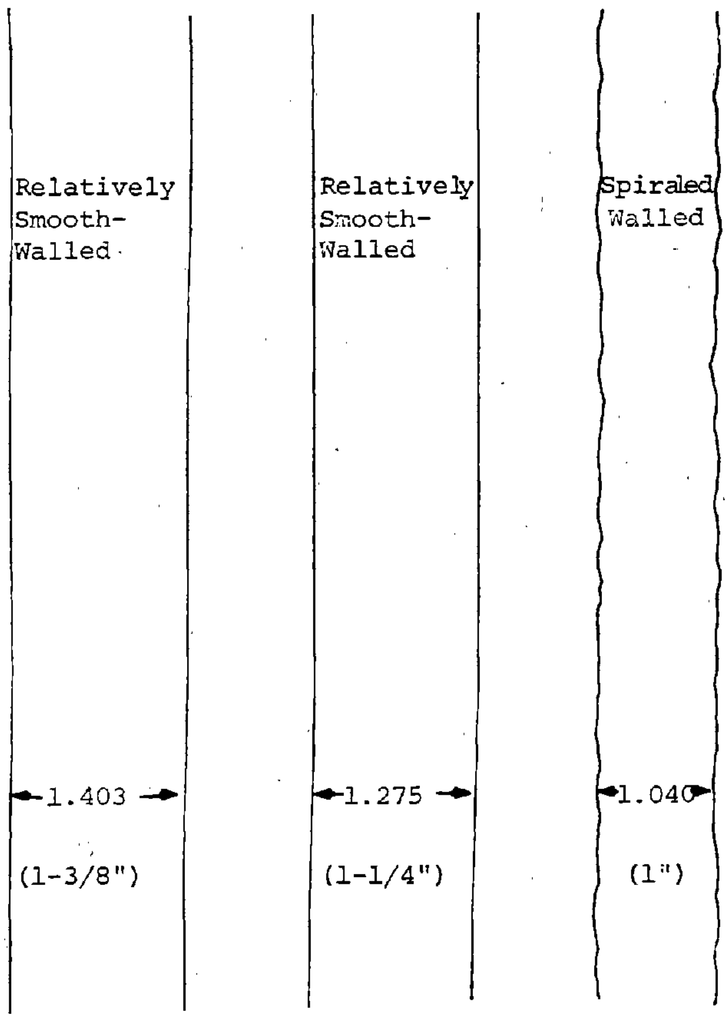


Figure 2.11 Illustration of Borehole Roughness of Each Hole Diameter [2.93]

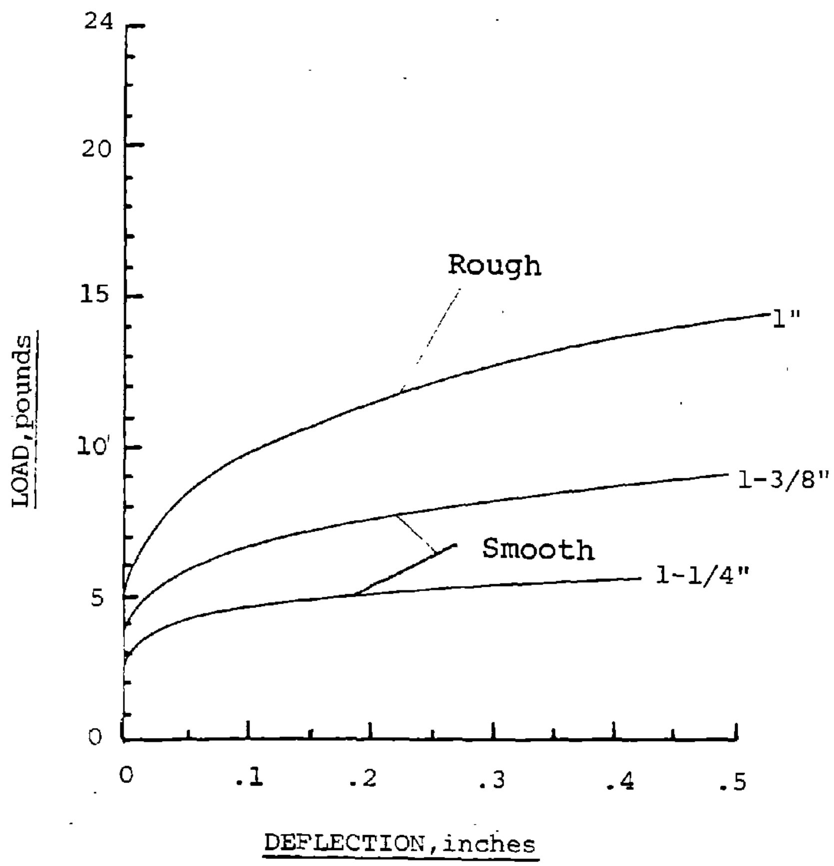


Figure 2.12 Pull Tests Reflecting the Effects of Borehole Roughness and Hole Size on Anchorage [2.93]

The mechanisms of bonding are discussed by Littlejohn and Bruce [2.28, 2.29] for cement grouted rock anchors, and the same mechanisms can be extended and applied to resin grouted roof bolts. Three mechanisms can be identified as shown in Figure 2.13. These are:

(1) Adhesion. This is the initial microscopic bond provided by chemical bonding or physical Vander Waal's forces--which disappears once the displacement exceeds microscopic size.

(2) Mechanical interlock. The shearing resistance of the irregularities of the resin and rock interfaces is important in preventing slip. A rough hole and a ribbed rebar are more effective than smooth holes and smooth bars.

(3) Friction. When the shearing strength of the interfacial materials is exceeded further shear resistance is due mainly to friction.

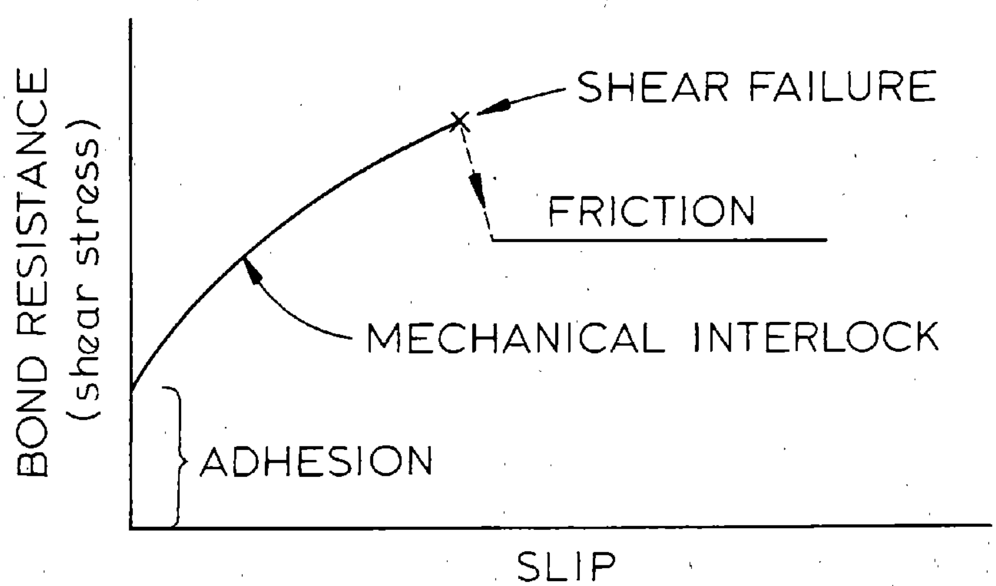


Figure 2.13 Mechanisms of Bonding of Grouted Bolts [2.28, 2.29]

In hard rock with a rough hole the resistance is primarily by mechanical interlock. In soft rock the mechanical interlock breaks down at low loads and a frictional resistance is mobilized.

As a sidelight, it is worth mentioning that cement-grouted reinforcing bars in concrete show results closer to theory, with decay lengths of 6-9 inches [2.27]. This is a two-material system of steel and concrete. Thus, the addition of the resin (as shown in the previous results in the three-material system of steel, resin and concrete) serves to distribute the load to a greater depth.

Statebam and Sun [2.112] have used the temperature rise due to the exothermic reaction of the resin as it gels to determine the integrity of the resin bolt installation. By measuring temperature rises on the bolt head they were able to distinguish between fully grouted bolts and bolts only partially grouted. A finite-difference analysis confirmed qualitatively the experiments.

2.2.2 Analysis of Axial Loading

Experiments on axial loading have been reviewed above. At this point it is of value to review the analyses that have been made of axial loading of resin bolt and rock assemblies.

Reed [2.30, 2.31, 2.32] points out the advantage of the fully grouted bolt in accommodating localized strain. If a small joint perpendicular to the bolt opens up, only a few inches of the resin bolt is stretched (see decay length above) and the resin bolt offers a greater resistance to this motion. In terms of the K values (force/length), in Table 2.2, this can be expressed as:

$$F = K \cdot \delta \qquad (2.2)$$

where F is the resistive force, δ is the joint opening displacement, and K is the axial stiffness of the bolt. As shown by experiments, K is 10-20 times larger for fully bonded resin bolts than for point anchored mechanical bolts. In terms of simple tension of a bar of length L,

$$K = \frac{AE}{L} \qquad (2.3)$$

where A is the area of bar, and E is the modulus of elasticity of the bar. Thus, K decreases (resistance or bolt effectiveness decreases) inversely with L. Applying Equation (2.3) approximately (because the grouted bar is not in a state of pure tension) to soft rock and hard rock, it can be deduced that the joint opening resistance of the fully grouted resin bolt will be less for soft rock than for hard rock because the decay length (L) of load transfer is longer for the soft rock (Figure 2.6).

Farmer [2.26], Hawkes [2.27] and Littlejohn and Bruce [2.28, 2.29] use an exponential law to represent the load transfer and decay along grouted bolts. The shear stress distribution along the bolt is represented by:

$$\tau_x = \tau_o e^{-(\alpha x/d)} \tag{2.4}$$

where τ_x is the shear stress at any position x, τ_o is the shear stress at x equal to 0, α is a constant, and d is the diameter of the bar. The applied axial load F can be written

$$F = \frac{\pi d^2 \tau_o}{\alpha} \tag{2.5}$$

Equation (2.5) is based upon competent bonding (mechanical interlock) along the entire length over which it is applied. Farmer uses α = 0.1 for resin bonded bolts. From (2.4) a decay length x = ℓ can be calculated. For decay to 5%,

$\frac{\alpha \ell}{d} = 3$, or $\ell = \frac{3d}{\alpha} = 30d$, which agrees with the experimental findings shown previously in Figure 2.6.

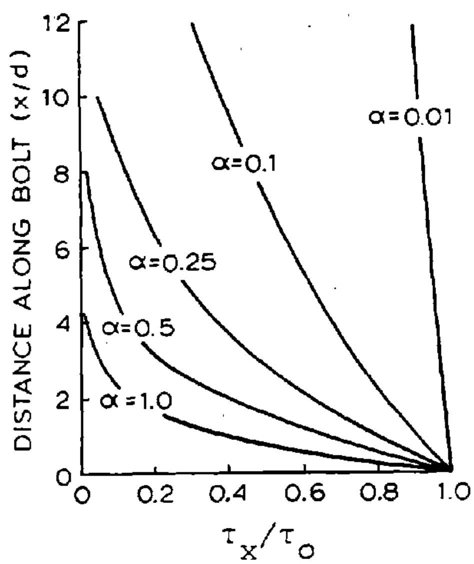


Figure 2.14 Theoretical Stress Distribution Along an Anchor [2.28, 2.29]

Littlejohn and Bruce give the change in stress distribution as a function of the exponential factor or constant α as shown in Figure 2.14. Comparison of Figures 2.14 and 2.6 indicates that α can be adjusted to match experimental results. However, α will change as the bond breaks down at high loads and the load is distributed further along the bar.

Using the solution of Farmer [2.26], one can also write an analytical expression for the axial stiffness K:

$$K = \frac{F}{\delta} = \frac{3AE}{\ell} \qquad (2.6)$$

where A is the area of bolt, E is Young's Modulus, and ℓ is the decay length. Using $A = \frac{\pi d_1^2}{4}$, $E = 30 \times 10^6$ psi and $\ell = 20$ inches, one finds the following results:

d_1, in.	K, kips/in.
0.75	1988
0.875	2706

These values are in the range given in Table 2.2 from reference [2.16] for limestone, but perhaps a little on the high side. One can use Equation (2.6) by a judicious choice of ℓ based upon the type of rock and resin used. (Kamal [2.39, 2.40] has also derived formulas for the axial stiffness of resin bolts, but the above formulas by Farmer are considered to have a more realistic basis.)

Haas et al [2.16, 2.33-2.35] have conducted axisymmetric finite element analyses of tension of bolt-resin-rock assemblies. A theoretical result for the pull test case has been given in Figure 2.5, which is similar to the exponential form given by Equation (2.4) above. They have also analyzed the effect of post tensioning of resin bolts by tightening a nut onto a bolt. A similar result is obtained, with the post tensioning effect decaying about 16 inches into the rock surface. Thus rock strata separations greater than 16 inches away from the bolt head cannot be pulled together by post tensioning in certain rock--the harder types. Any benefit can only be realized near the bearing plate.

Coates and Yu [2.36, 2.28, 2.29]* also have conducted finite element analyses and have varied the ratio of the moduli of the anchor and the rock. To correlate this analysis to the problem at hand, an equivalent modulus of the steel rebar plus resin is assumed to represent the anchor modulus.

---

* Littlejohn and Bruce in [2.28, 2.29] reference Coates and Yu

This permits a conclusion to be reached concerning the effect of the relative stiffness of the rock: Figure 2.15 shows the result that soft rock with a relatively low modulus will have a longer decay length, i.e. the load will be distributed over a greater length of the bolt.

Roegiers [2.37] has also conducted finite element analysis of resin grouted bolts and Wade and Stahl [2.38] give finite element results for stresses around mechanical anchors.

Dejean and Tournaire [2.107] have conducted a linear-elastic finite-element model study of fully grouted and point anchored bolts of two lengths in both homogeneous soft rock and homogeneous hard rock. The loading was the removal of insitu stresses due to excavation of the mining cavity with the assumption that the bolt was installed prior to excavation. In summary, their conclusions were:

- the width of the zone of ground re-compressed under the action of the bolt depends on the nature of the ground and is quite unaffected by the bolt length;

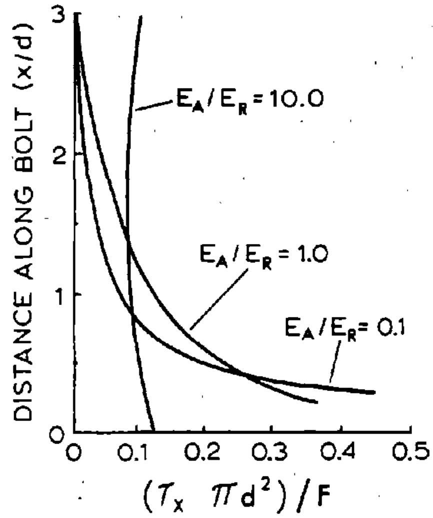
- in hard ground, short bolts (1 metre) are quite as effective as longer bolts;

- in soft ground, it is advisable to use the longest bolts possible. But, in this case, there is reason to expect either breakage of the bolt or displacement of the bolt at its extremities;

- as introduced into the model, the point anchorage bolt is less effective than the distributed anchorage bolt;

- the differences observed between the two types of ground are largely explained by the differences in rigidity - essentially in the Young's modulus - between the bolt and the ground.

The second and third conclusions especially must be considered limited to the particular model studied and should not be generalized. The last conclusion verifies Figure 2.15.



E_A = Modulus of Elasticity of anchor.

E_R = Modulus of Elasticity of rock

Figure 2.15 Variation of Shear Stress With Depth Along the Rock/Resin Interface [2.36]

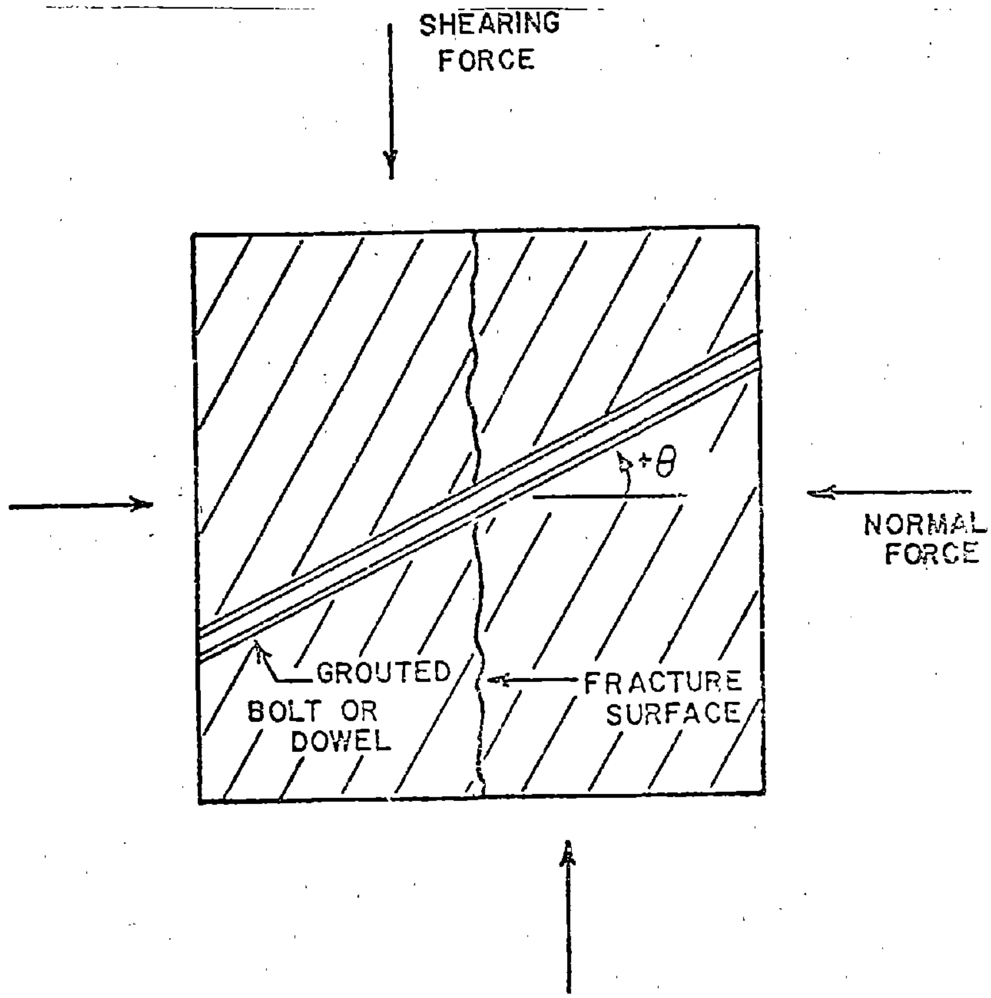


Figure 2.16 Schematic Sketch of Loading Geometry Showing Section of Rock, Fracture Surface, and Grouted Bolt. [2.41]

2.3 Shear Loading of Resin-Bolt-Rock Assemblies

An important reinforcement mechanism of the resin bonded bolt is that it offers a greater resistance to transverse shear loading than does the mechanical bolt. This has been recognized by many investigators and their results are now reviewed.

2.3.1 Experimental Data for Shear Loading

The most extensive experiments have been conducted by Haas et al [2.16, 2.34, 2.41] for shear loading of limestone and shale blocks, both resin bolted and mechanically bolted. Figure 2.16 shows the configuration used for the shear tests. The angle θ between the normal to the joint surface and the bolt was given two values 0° and 45°. When θ was +45° to the direction of loading as shown, tension built up in the bolt and a greater resistance was offered. When θ was -45° a compression effect occurred. Experiments were conducted at two values of normal pressure on the joint, 25 psi and 250 psi. Figures 2.17 and 2.18 show typical results for conventional mechanical bolting versus untensioned grouted rebar; showing the greater reinforcement effect for the latter. The greatest relative reinforcement effect of the resin bolt is realized at low normal pressures as evident from these data.

Additional results are given in Tables 2.5, 2.6 and 2.7. The shear stresses at 0.1 inch shear displacement are given because these may be used as an estimate of the initial elastic shear stiffness of the joint before yielding and progressive breakdown of the joint occurs. The shear stiffness is defined here as:

$$K_s = \frac{\tau A}{d} \text{, lbs/inch} \tag{2.7}$$

where τ is the average shear stress on joint surface in psi, A is the area of joint surface (24 x 24 in^2 in this case), and d is the shear displacement in inches.

It is evident from Tables 2.5 and 2.6 that there is little benefit derived from post tensioning or pre-tensioning the resin bolts. These results were for initially smooth flat lapped joints. Additional results have been obtained for natural joints in limestone produced by fracturing the blocks [2.16]. The results for resin bolting of natural joints are given in Figures 2.19 and 2.20, and Tables 2.8 and 2.9. The initial strength of the joint without the bolt is now larger, particularly for the higher normal stress case. Also, the maximum shear stress is reached at smaller displacements for the high normal stress for the natural surface. Pull out of the bolts was also found for the natural joint tests as

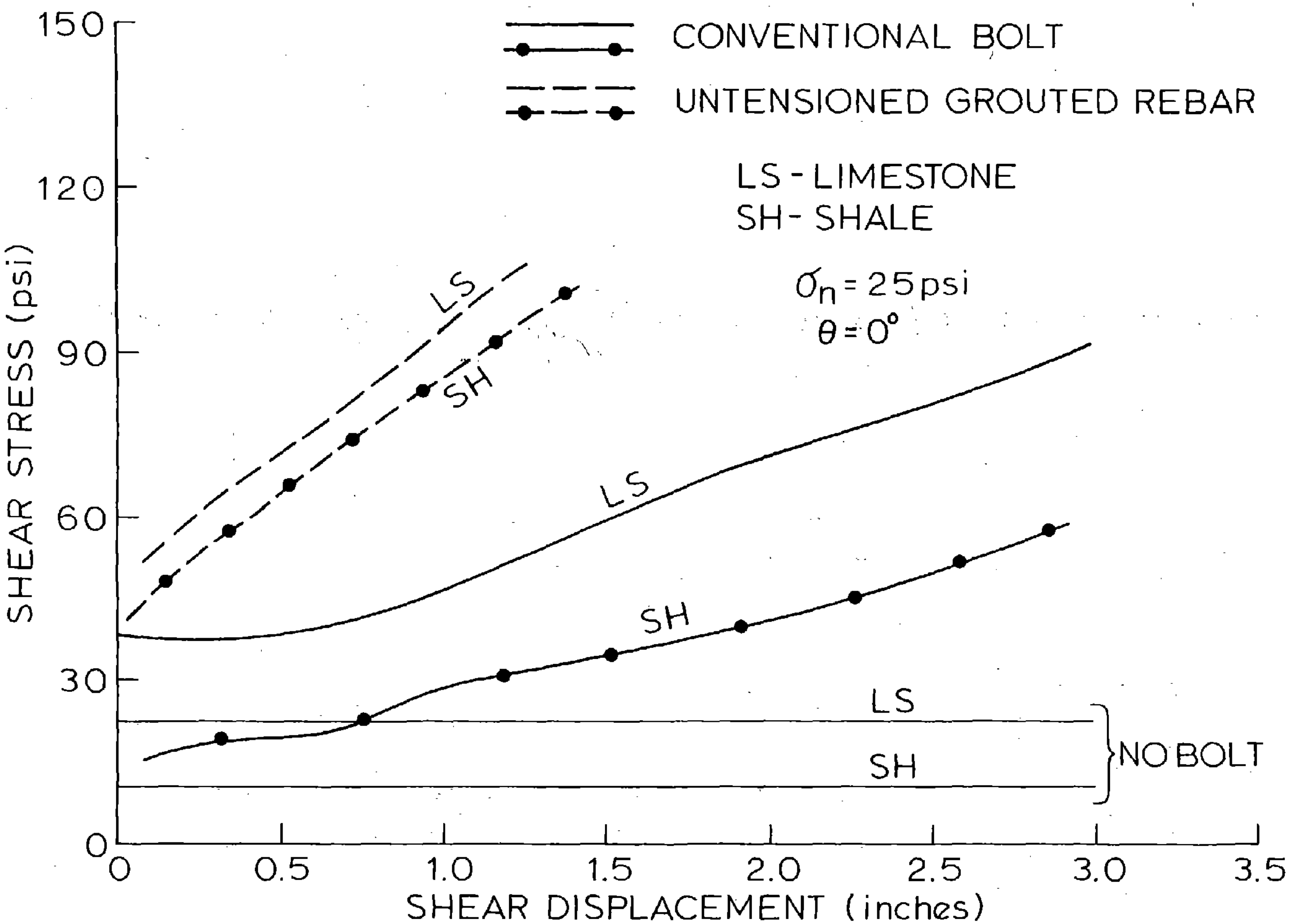


Figure 2.17 Comparison of Shear Resistance for Joints in Limestone and Shale at Low Normal Pressure (25 psi) [2.41]
(5/8 inch conventional bolt and 7/8 inch grouted rebar installed in 1 3/8 inch holes)

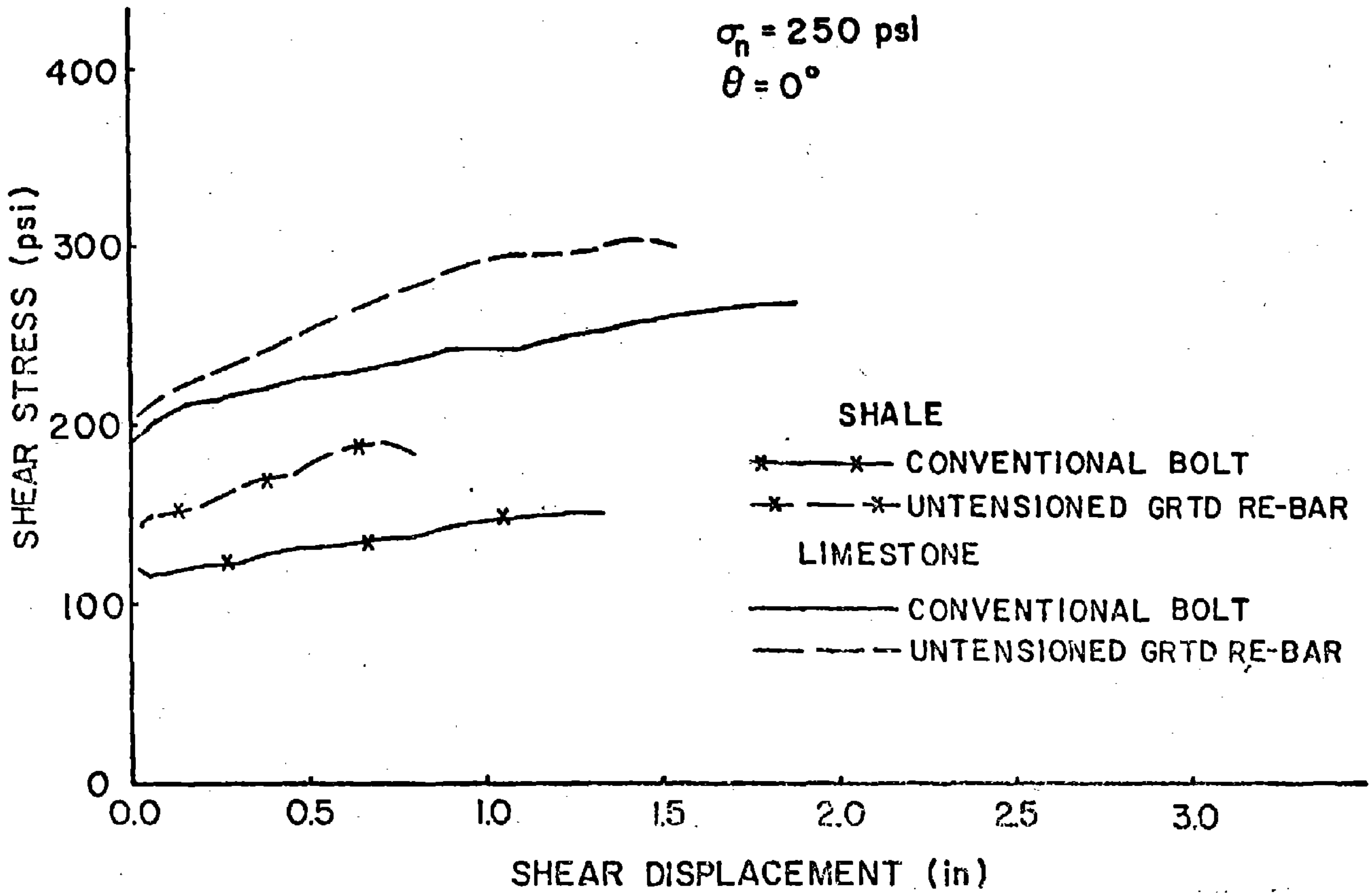


Figure 2.18 Comparison of Shear Resistance on Joints in Limestone and Shale at High Normal Pressure (250 psi) [2.41] (5/8 inch conventional bolt and 7/8 inch grouted rebar installed in 1 3/8 inch holes)

Table 2.5 Summary of Shear Stresses on Bolted Joint in Limestone. (Low Normal Pressure, σ_n = 25 psi.) [2.41]

Bolt Configuration[b]	Shear Stress at[a] d = 0.1 in., psi	Max. Shear Stress[a]		Increase in Max. Shear Stress Due to Bolt[a], psi
		Shear Stress psi	Corr. Shear Displ., in.	
Grouted Pretensioned $\theta = +45^o$	81	130	1.78	107.4
Grouted Post tensioned $\theta = +45^o$	68.5	120.5	0.94	97.9
Grouted Untensioned $\theta = +45^o$	76.5	116.5	1.02	93.9
Grouted Post tensioned $\theta = 0^o$	57.5	113	1.62	90.4
Grouted Untensioned $\theta = 0^o$	52.5	106.5	1.28	83.9
Grouted Pretensioned $\theta = 0^o$	53	102	1.04	79.4
Conventional $\theta = 0^o$	36	94	3.13	71.4
Conventional $\theta = +45^o$	56	78.5	1.76	55.9
Grouted Post tensioned $\theta = -45^o$	39.5	41.5	0.02	18.9
Grouted Pretensioned $\theta = -45^o$	35	40.5	0.81	17.9
Grouted Untensioned $\theta = -45^o$	35	37.5	0.05	14.9

(a) Average values
(b) 5/8 inch diameter conventional bolts and 7/8 inch diameter rebar installed in 1 3/8 inch diameter holes

Table 2.6 Summary of Shear Stresses on Bolted Joint in Limestone. High Normal Pressure, σ_n = 250 psi [2.41]

Bolt Configuration	Shear Stress at* d = 0.1 in., psi	Max. Shear Stress*		Increase in Max. Shear Stress Due to Bolt*, psi
		Shear Stress psi	Corr. Shear Displ., in.	
Grouted Untensioned $\theta = 0^{\circ}$	217	310	1.65	92.3
Grouted Pretensioned $\theta = 0^{\circ}$	234	302.5	1.44	84.8
Grouted Post tensioned $\theta = 0^{\circ}$	223	301.3	1.46	83.6
Grouted Untensioned $\theta = +45^{\circ}$	258.5	299.5	0.65	81.8
Conventional $\theta = +45^{\circ}$	231	279	1.86	61.3
Conventional $\theta = 0^{\circ}$	203.5	272	2.40	54.3
Grouted Untensioned $\theta = -45^{\circ}$	222	253	2.88	35.3
Conventional $\theta = -45^{\circ}$	204	246.5	2.90	28.8

* Average values and d is the shear displacement

Table 2.7 Summary of Shear Stresses on Bolted Joint in Shale [2.41]

Test Configuration	Shear Stress at* d=0.1 in., psi	Max. Shear Stress*		Increase in Max. Shear Stress Due to Bolt,* psi
		Shear Stress psi	Corr. Shear Displ., in.	
	Low Normal Pressure $\sigma_n = 25$ psi			
Grouted Untensioned $\theta = 0°$	46	102	1.434	91.2
Conventional $\theta = 0°$	16	60	2.95	49.2
	High Normal Pressure $\sigma_n = 250$ psi			
Grouted Untensioned $\theta = 0°$	150	222	2.001	101
Conventional $\theta = 0°$	116	165	3.002	44

* Average values and d is the shear displacement

(7/8-inch resin grouted rebar in 1 3/8-inch holes)

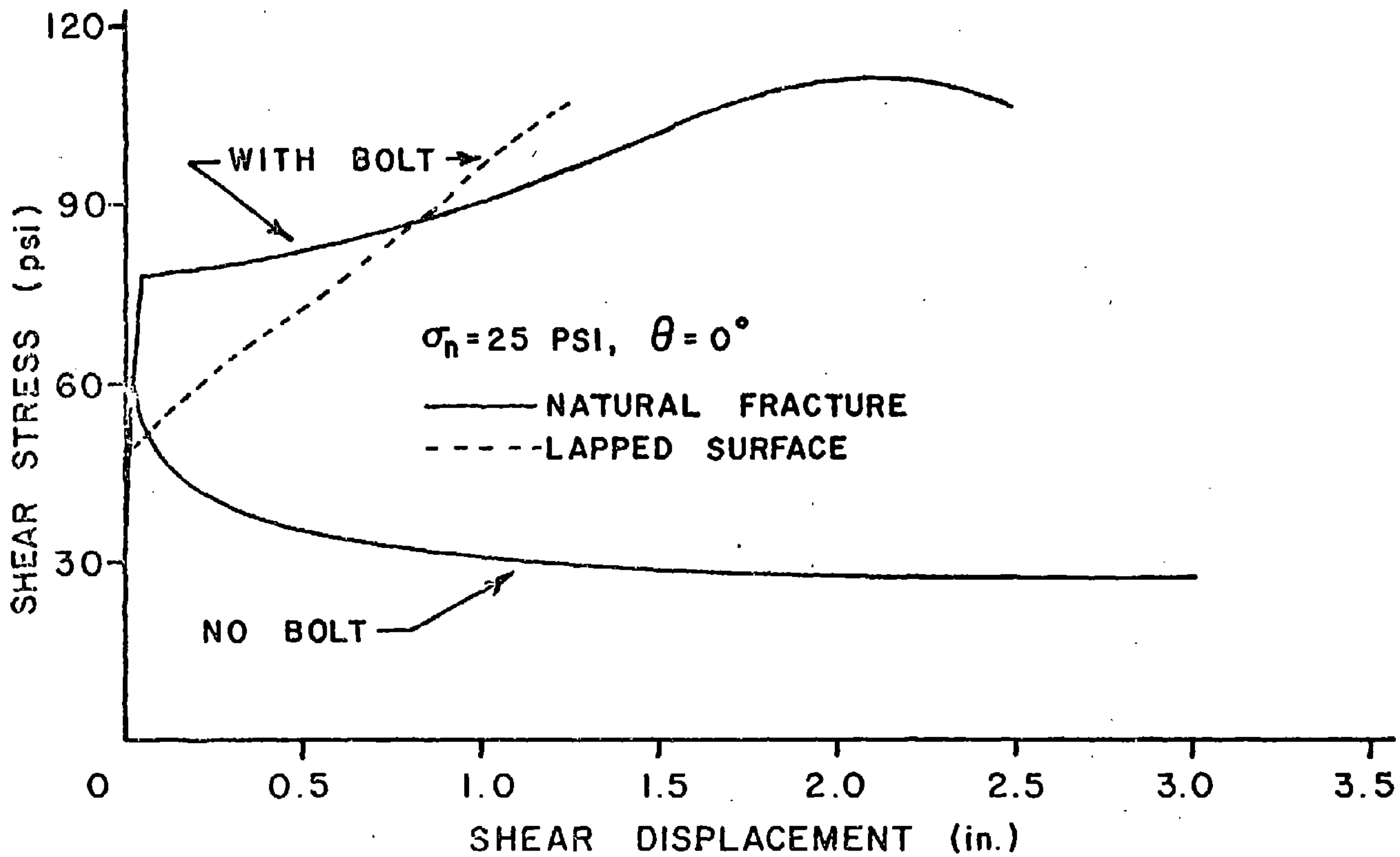


Figure 2.19 Average Shear Stress on Bolted Joint in Limestone versus Shear Displacement for Normal Bolt, Natural Fracture, and Low Normal Pressure (25 psi). [2.16]
(7/8 inch resin-grouted rebar in 1 3/8 inch holes)

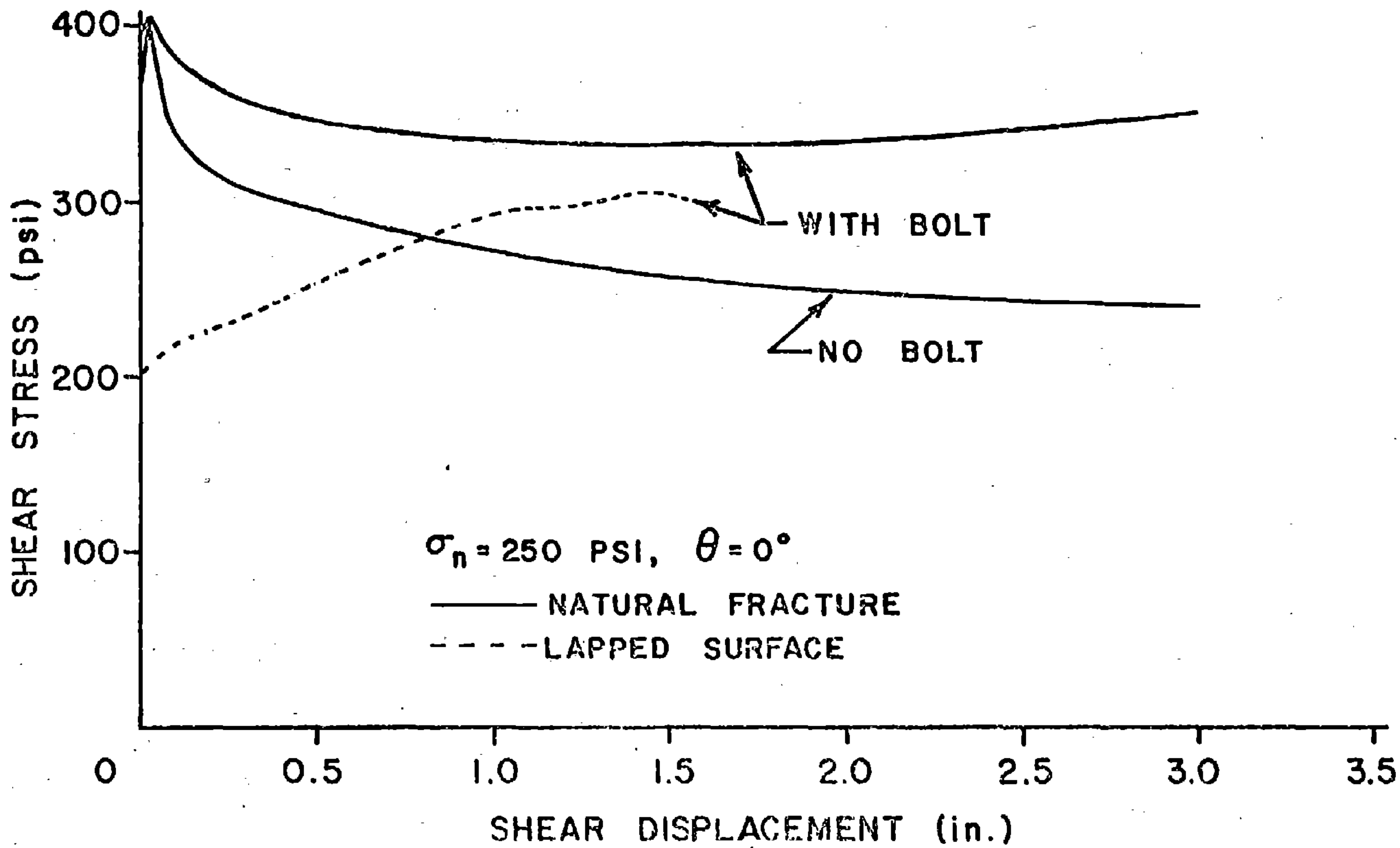


Figure 2.20 Average Shear Stress on Bolted Joint in Limestone versus Shear Displacement for Normal Bolt, Natural Fracture and High Normal Pressure (250 psi). [2.16]
(7/8 inch resin-grouted rebar in 1 3/8 inch holes)

TABLE 2.8 SUMMARY OF SHEAR TEST RESULTS* WITH NORMAL BOLTS AND NATURAL FRACTURES. LOW NORMAL PRESSURE σ_n = 25 psi

(7/8-inch resin grouted rebar in 1 3/8-inch holes) [2.16]

Performance Criterion	Natural Fracture			Lapped Surface		
	Without Bolt	With Bolt	Increase with Bolt	Without Bolt	With Bolt	Increase with Bolt
Shear stress at d = 0.1 in., psi†	50	79.5	29.5	19.5	52.5	33
Maximum Shear Stress, psi	61	111	50	27.5	106.5	79
Shear Displ. for Max. Shear Stress, in.	0.032	1.956	1.924	3.00	1.275	-1.725
Maximum Pull-** Out of Bar, in.		-0.033 to 0.432			-- --	

† d is the shear displacement of the test blocks

* Shear stresses on bolted joints in limestone. Average values of several tests.

** Separation of surfaces.

TABLE 2.9 SUMMARY OF SHEAR TEST RESULTS* WITH NORMAL BOLTS AND NATURAL FRACTURES. HIGH NORMAL PRESSURE σ_n = 250 psi

(7/8-inch resin grouted rebar in 1 3/8-inch holes) [2.16]

Performance Criterion	Natural Fracture			Lapped Surface		
	Without Bolt	With Bolt	Increase with Bolt	Without Bolt	With Bolt	Increase with Bolt
Shear Stress at d = 0.1 in., psi †	333	377	44	206	217	11
Maximum Shear Stress, psi	399	405	6	239	310	71
Shear Displ. for Max. Shear Stress, in.	0.025	0.017		2.50	1.652	-0.848
Max. Pull-Out ** of Bar, in.		0.137 to 0.336			0.020 to 0.043	

† d is the shear displacement of the test blocks

* Shear stresses on bolted joints in limestone. Average values of several tests.

** Separation of surfaces

indicated in Tables 2.8 and 2.9. Sliding of the rough surfaces over one another in these tests causes greater joint separation and greater tension in the bolts.

Bjurstrom [2.42] has also conducted direct shear tests of bolted joints, but on granite blocks with a cross section of 25 x 40 cm. These were not resin bonded, but cement grouted bolts. Both smooth joints and natural joints were tested under a range of normal pressures and a range of angles, θ = 15° to 90° (see Figure 2.16). The maximum reinforcement effect was found at $\theta \cong 50°$. Failures occurring for angles greater than 55° seemed to be of the tensile type and those for angles less than 50° were a combination of tension and shear. It was found that the increase in maximum shear resistance (due to the bolts) was 220-290 psi for normal pressures of ~ 430 psi and bolts of 16 mm (0.63 inch) diameter. The maximum shear stress was reached at a displacement of about 10 mm (0.39 inch).

Horino, Duvall and Brady [2.43] have conducted experiments on mechanically bolted pillar models, of sandstone, marble, Indiana limestone and oil shale, containing a plane of weakness at 45°, and have found that bolting increased the compressive strength by 84, 31, 52, and 28 percent respectively.

Heuze and Goodman [2.44] have also conducted shear tests on bolted joints using 3/16-inch dia. steel bolts, grouted but untensioned, in sandstone with θ = 0° (see Figure 2.16). The cross section of the shear specimens was a 4.75 inch square or 22.5 in^2. The results for variable normal stress σ_n were as follows:

σ_n, psi	Increase in Shear Strength $\Delta\tau$, psi
100	46.
500	82.
1500	137.

If the dimensions are scaled by a factor of 5, the specimen and bolt size is then nearly that of Haas (Table 2.6) and the results for $\Delta\tau$ are the same order of magnitude.

Kwitowski and Wade [2.45] have instrumented resin bolts with strain gages along the length and have measured the bending strain distribution for direct shear of 14 inch x 14 inch square Indiana limestone blocks bolted together with 3/4 inch steel rebar resin bonded in 1-inch diameter holes. Figure 2.21 shows some results and shows that the bending effect decays out in about 4 inches. The decay length for bending is about 25% of that for tension (Figure 2.5). An

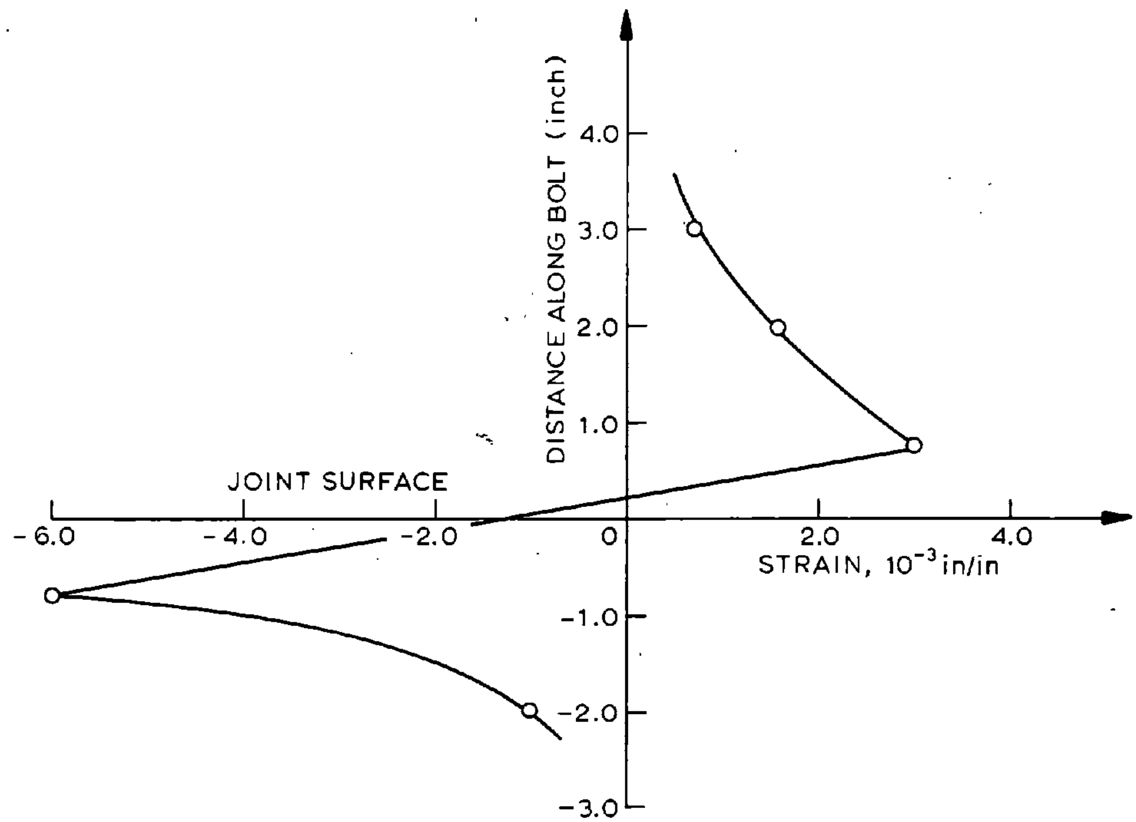


Figure 2.21 Bending Strain Along Resin Bolt For a Shear Displacement of 0.10 Inch [2.45]

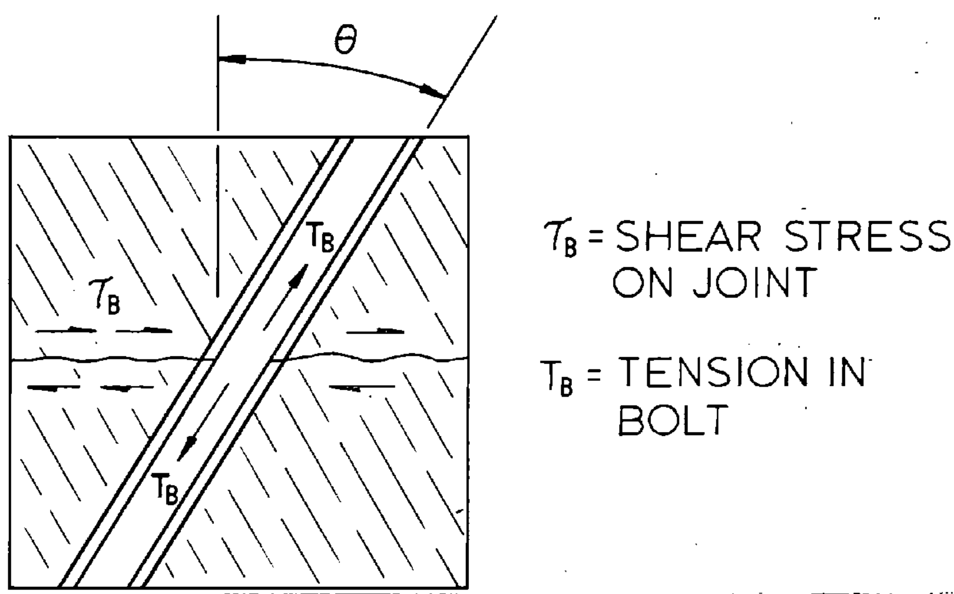


Figure 2.22 Bolted Joint Showing Bolt Force and

inflection point occurs in the bending strain near the joint surface, but the distribution is not exactly symmetrical as shown.

Wade, Kwitowski and Judeikis [2.108] have also measured strain in bolts installed in mines. They found that:

1. For laboratory shear tests on full column resin bolts with a 3/4 inch (1.91 cm) diameter steel core in a 1 inch (2.54 cm) diameter borehole in limestone, the shear stiffness of the bolt was calculated to be 208,000 lb/in (25,000 kg/cm) for differential lateral displacement up to 0.09 inches (2.29 mm).

2. For laboratory axial tension tests, the axial stiffness of the bolt was calculated to be 1,620,000 lb/in (194,000 kg/cm) for an axial tension up to 17,000 lbs. (7,700 kg).

3. Full column resin bolts in service as the primary means of roof support in underground coal mines can be subjected to significant loadings resulting from both differential lateral displacement and axial tensions.

4. The axial tensions developed can be greater than those expected from simple dead weight suspension of immediate roof. (This is a result of shear between layers stretching the bolt at the joint.)

5. Based on comparisons with existing experimental and analytical results, a full column resin bolt of the type discussed above is approximately 36 times stiffer than an expansion shell anchored tension bolt 5/8 inches (1.59 cm) in diameter and of equal length for the load types (both shear and axial) described in this report.

2.3.2 Analysis of Shear Loading of Joints

Experiments on shear loading have been discussed above. Analysis of this type of loading is now discussed.

Many have analyzed the reinforcement effect of bolting of joints. Lang [2.46], Obert and Duvall [2.47], Alexander and Hosking [2.48], and Horino, Duvall and Brady [2.43] all have analyzed the increase in shear strength due to the effect of the tension of a mechanical bolt across a joint or failure plane in rock. Haas et al [2.16] have included as well a relationship between slip of the joint and tension in a mechanical bolt.

Bjurstrom [2.42] has reinterpreted the method of analysis for mechanical bolts and reapplied them to the fully-grouted untensioned bolt. The shear reinforcement effect due to tension developed in the bolt can be represented by the equation:

$$\tau_B = T_B \; (\sin\theta + \mu \cos\theta)/A \qquad (2.8)$$

where τ_B is the reinforcement effect or increase in shear stress resistance due to bolting, T_B is the tensile force developed in the bolt due to shear displacement along the joint, θ is the bolt angle (see Figure 2.22), μ is the coefficient of friction of joint ($\mu \approx \tan\phi$, cohesion of the joint is negligible) and A is the specific area of joint surface per bolt.

Bjurstrom finds that Equation (2.8) accounts for 50-60% of the reinforcement effect when compared with experimental data for granite. The additional effect is attributable to the "dowel effect" which includes the strength of the bolt itself and the bearing strength of the rock in contact with the lateral surface of the bolt as shown in Figure 2.23. The shear reinforcement due to the "dowel effect" (for $\theta \doteq 0°$) is represented by the Equation [2.42]:

$$\tau_{dowel} = \frac{d^2}{A} \sqrt{\frac{\sigma_B \cdot \sigma_R}{6 \cdot \xi}} \qquad (2.9)$$

where σ_B is the yield strength of bolt steel, σ_R is the average rock stress under bolt compression, and is a constant indicating the point of peak bearing stress.

Figure 2.23 shows σ_R and ξZ the point of peak bearing stress. The distance ξZ is assumed to be constant and is related to the decay length under shear, shown previously in Figure 2.21.

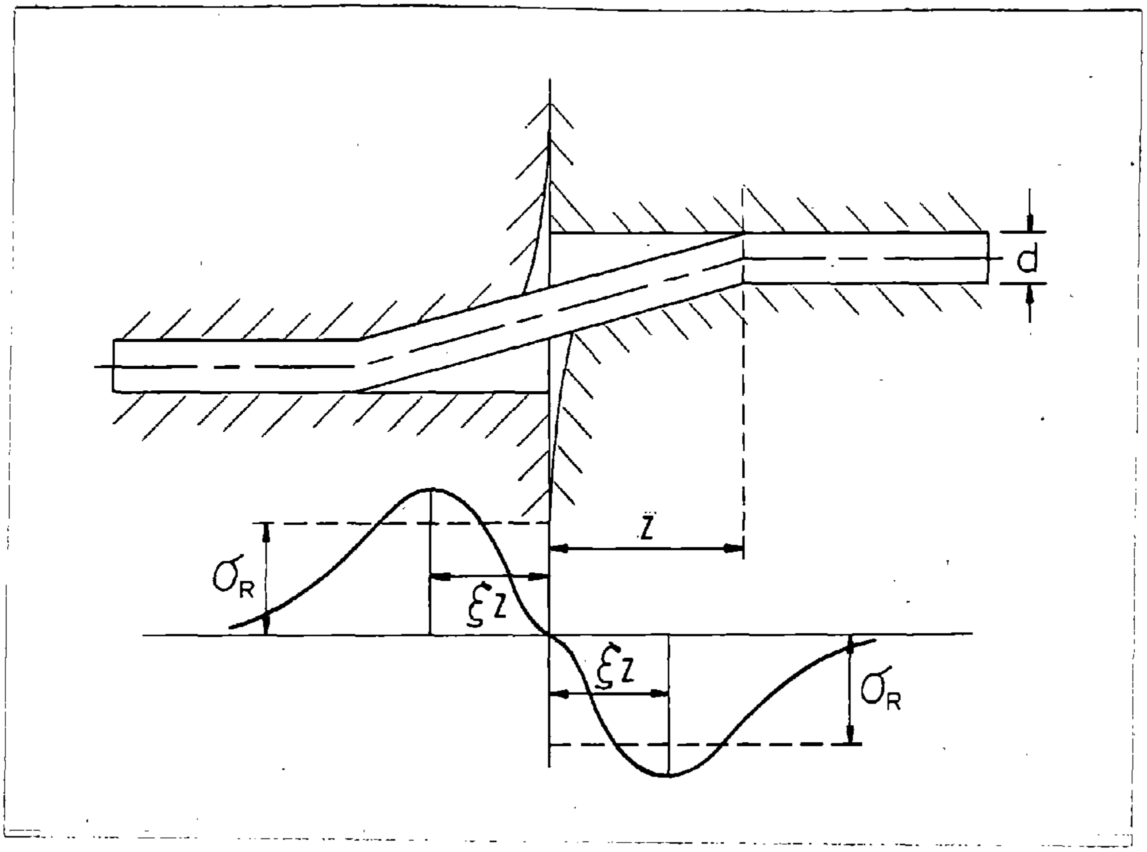


Figure 2.23 Stress Distribution for Dowel Near Failure [2.42]

From Equation (2.9) one can readily perceive that shear reinforcement will be less effective in soft rock than in hard rock. For smaller σ_R the shear reinforcement is smaller. For soft rock the load transfer length parameter ξ will be larger, therefore the shear reinforcement effect will be less. Thus, in soft rock one should expect the reinforcement to be due more to tension developed from slip according to the first Equation (2.8). Bjurstrom [2.42] has found very good agreement between theory and full scale experiments on granite using Equations (2.8) and (2.9).

In addition to the shear strength of bolted joints, the stiffness of bolted joints under shear is important. Fairhurst and Singh [2.49] assume the model shown in Figure 2.24. This model is equivalent to a cantilever beam of length three-fourths the diameter, loaded by a force P as shown in Figure 2.25. The shear stiffness K_s, then is:

$$K_s = \frac{P}{2\delta} = \frac{\pi E d_1}{31.5} \tag{2.10}$$

where δ is the deflection shown resulting from bending and shear. The term 2δ in the denominator represents the total deflection of both halves of the bolt.

For an effective length of $(3/4)d_1$, Equation (2.10) will reduce to $\pi E d_1/31.5$ in units of lbs/inch. For E equal to 30×10^6 psi, and d_1 equal to 1.0 inch, the shear stiffness K_s is calculated to be 3.0×10^6 lbs/inch.

In the above approach the properties of the resin and the hole diameter (or annulus) are not accounted for. Gerdeen has taken a different approach [2.50] that does account for these factors, in that the steel bolt is assumed to rest upon an elastic foundation (the resin). (The rock is assumed rigid.) The result of this approach is:

$$K_s = \frac{P}{2\delta} = EI\beta^3 \tag{2.11}$$

where β is equal to $\sqrt[4]{K/4EI}$ and I is equal to $\pi d_1^4/64$. The K in the β expression is the elastic foundation modulus and is equal to $2E_R/(d_2/d_1 - 1)$, where E_R is the modulus of elasticity of the resin and d_2 is the hole diameter. For E_R equal to 1.5×10^6 psi, for d_1 equal to 1.0 inch and for d_2 equal to 1 1/3 inches, Equation (2.11) gives a value of the shear stiffness K_s of 1.6×10^6 lbs/inch. This is about half the value found by Fairhurst and Singh [2.49].

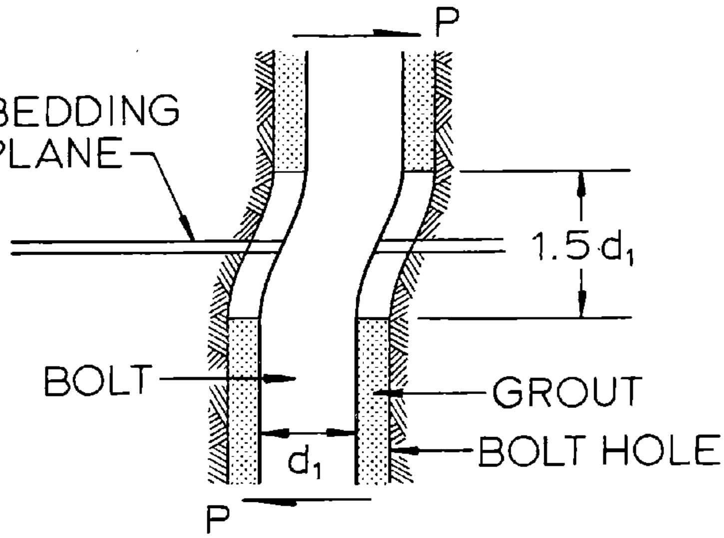


Figure 2.24 Model Assumed for Shear Stiffness of Bolted Joint [2.49]

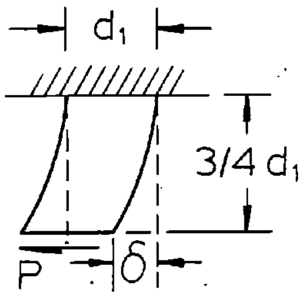


Figure 2.25 Equivalent Cantilever Beam

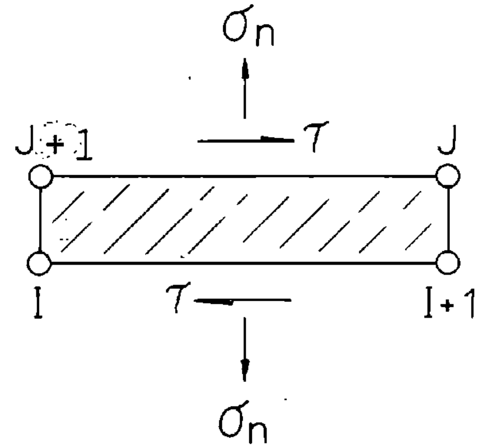


Figure 2.26 Joint Element

The beam on the elastic foundation approach enables the calculation of a decay length under bending because the solution is of the exponential decay type. For decay of δ to 5% of maximum value, the decay length becomes

$$\ell \cong 3/\beta \tag{2.12}$$

For the above data, with d_1 equal to 1.0 inch, the decay length ℓ becomes equal to 2.7 inches. This compares with $\ell \simeq 4$ inches found in experimental results [2.45], which indicates that elasticity of the rock evidently extends the load transfer distance somewhat.

An experimental value of K_s is taken from graphical data of τ versus δ from Haas [2.16, Appendix B, Figure B-8], for $\delta \leq .05$ inch. The value estimated is $K_s \cong 0.86 \times 10^6$ lb/in, which is about half that predicted by Equation (2.11) which again indicates that elasticity of the rock is increasing the deflection somewhat, as expected.

2.4 Finite Element Analysis of Bolted Joints

The finite element method has found extensive use in rock mechanics. This method is discussed in more detail in Section 3 on mine analysis, however, at this point in the discussion it is appropriate to indicate how the bolted joint is represented in a finite element analysis. Considered here is the joint element, Figure 2.26, developed by Goodman, Isenberg and Heuze [2.51, 2.52, 2.53]. In Figure 2.26, τ is the shear stress acting on the joint, σ_n is the stress normal to the joint and the directions I to I+1 and J to J+1 are parallel to the joint surface. The joint element has been used and modified to include the bolting effect by Kamal and Gerdeen [2.39, 2.40], as shown in Figures 2.27 and 2.28.

In the finite element analysis the normal stiffness K_n and the shear stiffness K_s are defined to have units of psi/inch displacement rather than lb/inch as in the previous section. The influence of the bolt upon the joint can be described by referring to Figures 2.27 and 2.28 as follows:

(1) The bolt increases the value of the shear and normal stiffnesses of the joint, K_s and K_n.

(2) The bolt increases the value of the inherent shear strength of the joint, C_i.

(3) The joint develops a capacity to resist a tensile normal stress ρ_i.

(4) There is no change in the internal friction angle ϕ_f and the maximum joint closure t_i.

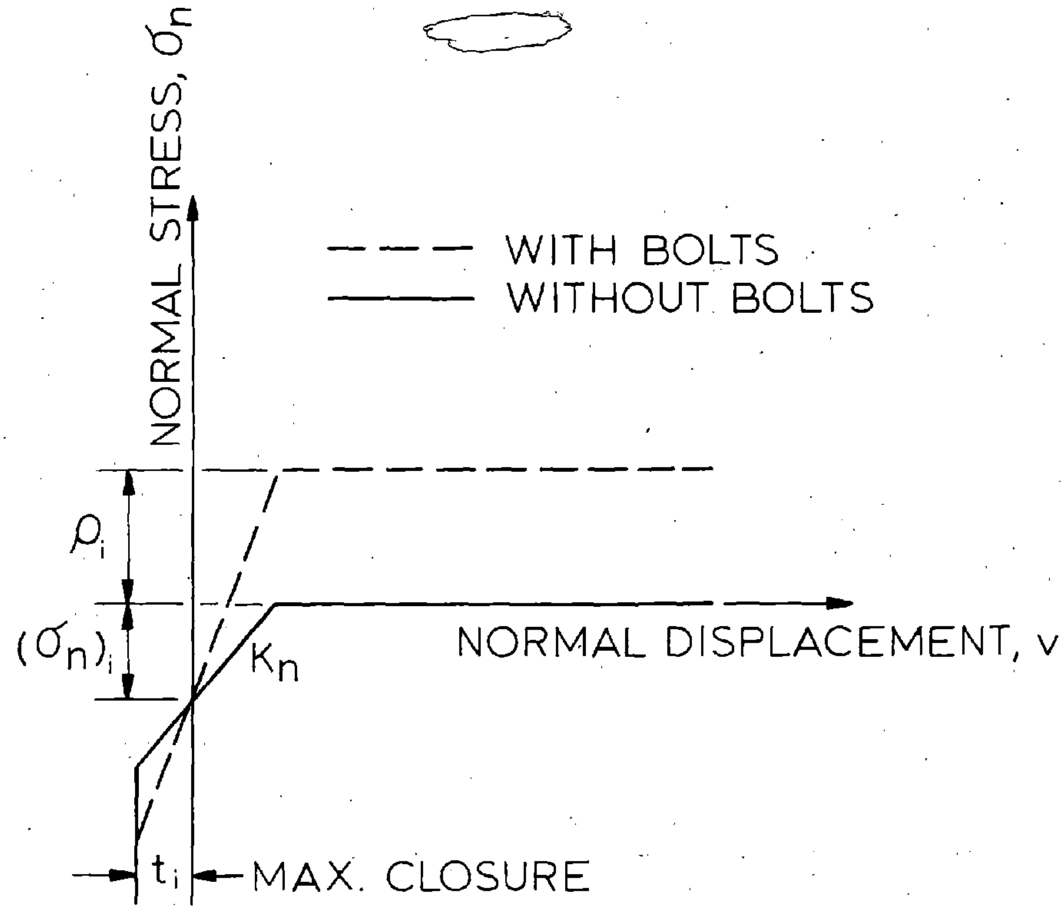


Figure 2.27 Effect of Bolting on Joint Normal Stiffness [2.39, 2.40]

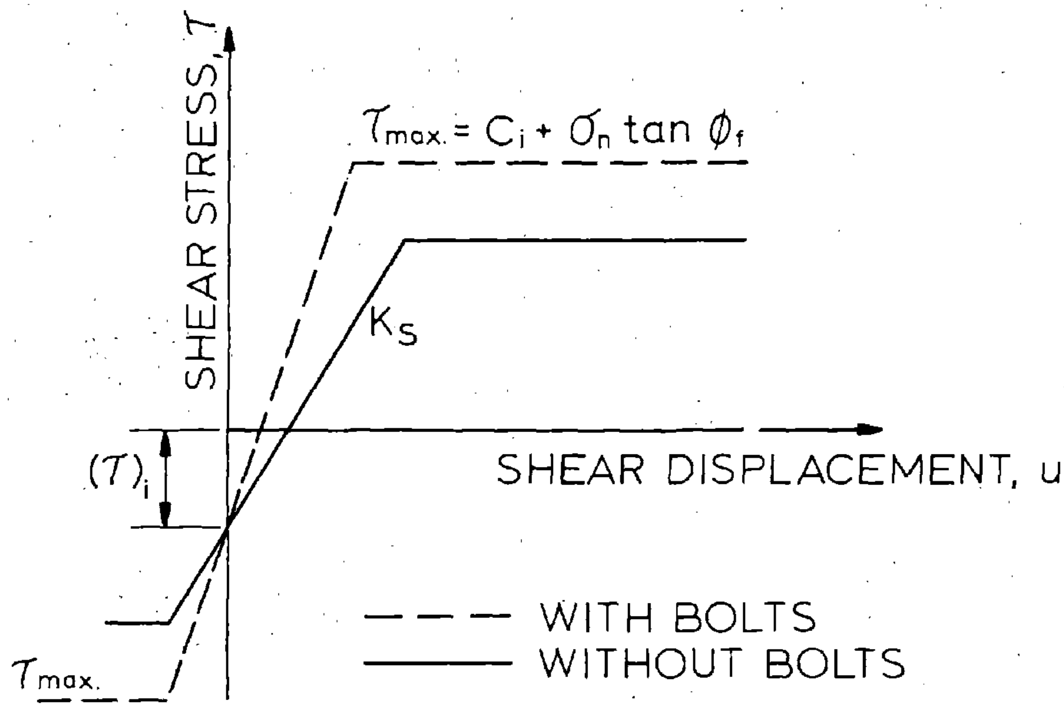


Figure 2.28 Effect of Bolting on Joint Shear Stiffness [2.39, 2.40]

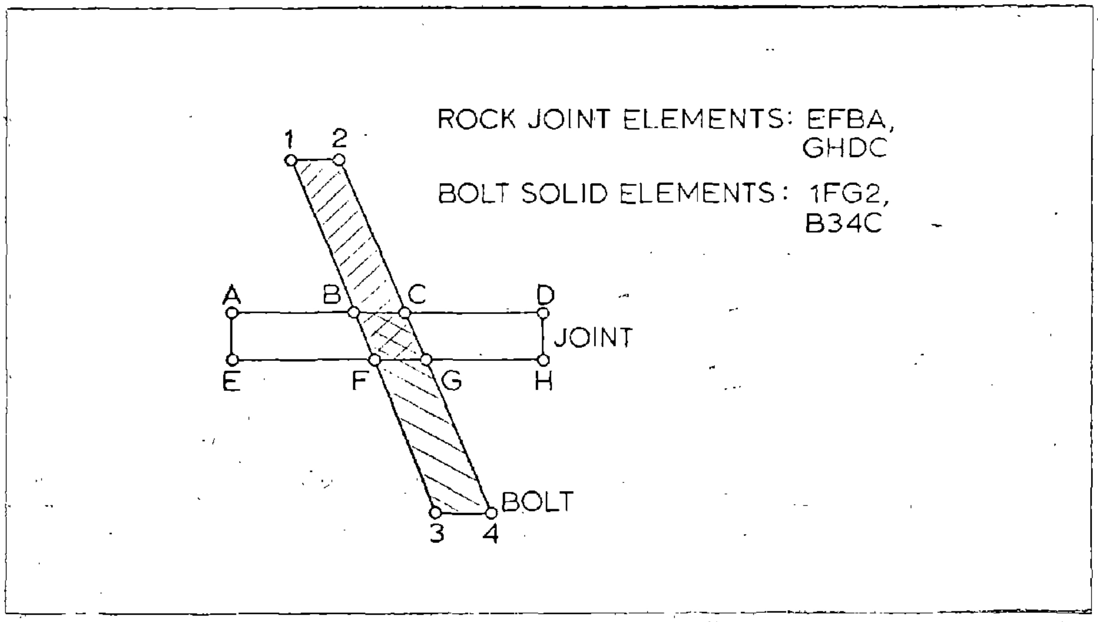


Figure 2.29 Finite Element Representation of a Bolt Across a Joint [2.44]

The values of K_s and K_n found experimentally or analytically and reported in the previous sections need only be divided by the cross-sectional area of the joint element (a variable depending on the mesh size used in finite element model) to be used in this analysis.

Heuze [2.44] has gone a step further and used another large slender solid element to represent the bolt as shown in Figure 2.29. He uses an artifice to make the rock joint "feel" the presence of the bolt. The two steel (bolt) elements are numbered so that the upper one is tied to the lower joint plane and the lower steel element is tied to the upper joint plane.

Londe and Tardieu [2.54] have also used a modified joint element approach to represent bolting, but use a "truss" element to represent the bolt. However, this approach represents neither the bending stiffness of the bolt nor the effect of shear stresses transmitted nonlinearly along the bolt.

Kulhawy [2.111] uses a one-dimensional joint element and reviews the work of Clough and Duncan (1969) to show how the shear stiffness K_s can be related in a nonlinear way to the cohesion and angle of friction of the joint, the shear stress and the normal stress on the joint. This approach, however, makes the finite element analysis more nonlinear. Kulhawy lists experimental values of normal stiffness and shear stiffness for various types of rock joints. Some of these data are repeated here in Table 2.10. Most of the data are originally from Goodman (1968). The overall average K_s at yield is 3.02×10^6 KN/m^3 or 1.11×10^4 lb/in^3.

TABLE 2.10 STIFFNESS VALUES FOR JOINTS [2.111]

Joint	Average Stiffness, 10^6 KN/m^3		
		Secant Shear, K_s	
	Normal, K_n	Yield	Peak
Berea Sandstone, Dry Sawed Joint	-	-	29.8
Boise Sandstone, Dry Saw Cut	35.1	-	1.29
Sandstone, Marl Contact	-	0.84	0.15
Marly Sand Filled Joint	1.96	-	2.34
Limestone, Dry Sawed Joint	-	-	8.73
Marly Partings in Limestone, Saturated	-	2.74	2.89
Marly Partings in Limestone, Saturated	-	1.47	7.41
Limestone with Marly Joints, Dry	-	-	9.75
Limestone - Slightly Rough Bedding	-	2.17	0.84
Limestone - Rough Bedding Surfaces	-	5.93	3.06
Limestone - Rough Unfilled Fractures	-	8.46	1.98
Limestone - Thin Shale Seams At Joint	-	5.96	3.08
Limestone - Smooth Unfilled Fractures	-	1.07	0.51
Closely Jointed Shale in Limestone	0.24	-	0.02
Shale Interbedded - Wet	0.26	-	0.02
Slate - Dry Natural Clevage Surface	-	0.88	0.79
⋮	⋮	⋮	⋮
Overall Average	-	3.02	2.82

2.5 Design Data for Resin Bolts

Various investigators have determined design data for the support capability of resin bolts based upon rock strength and other factors. These design data are expressed in terms of the bond strength which in turn is related to the design load to be imposed on the resin bonded bolt. The design load depends upon the support requirements that are to be determined from an analysis of the mechanisms of roof reinforcement that should be achieved in the particular mine situation. These support requirements are reviewed in the next section, Section 2.6.

Franklin and Woodfield [2.18] have determined design data for the SELFIX (CELTITE) resin. Some of these data are repeated in references [2.3, 2.4, 2.7]. Figure 2.30 shows the bond length required to achieve a certain pull strength in five different rock materials: (1) granite, (2) limestone, (3) sandstone, (4) coal,

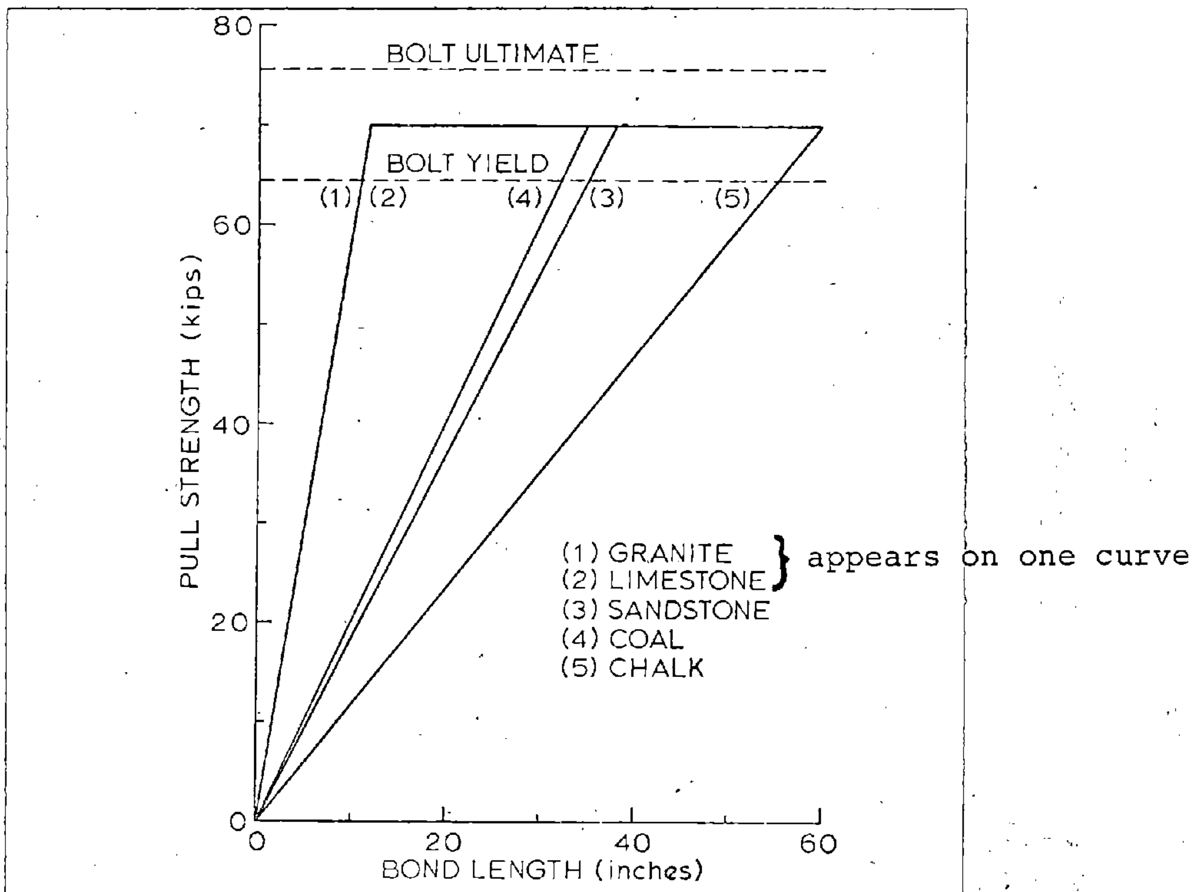


Figure 2.30 Bond Length Required to Achieve Maximum Pull Strength in Various Materials (1.0 inch diameter bolts resin-grouted in 1.25 inch diameter holes) [2.18]

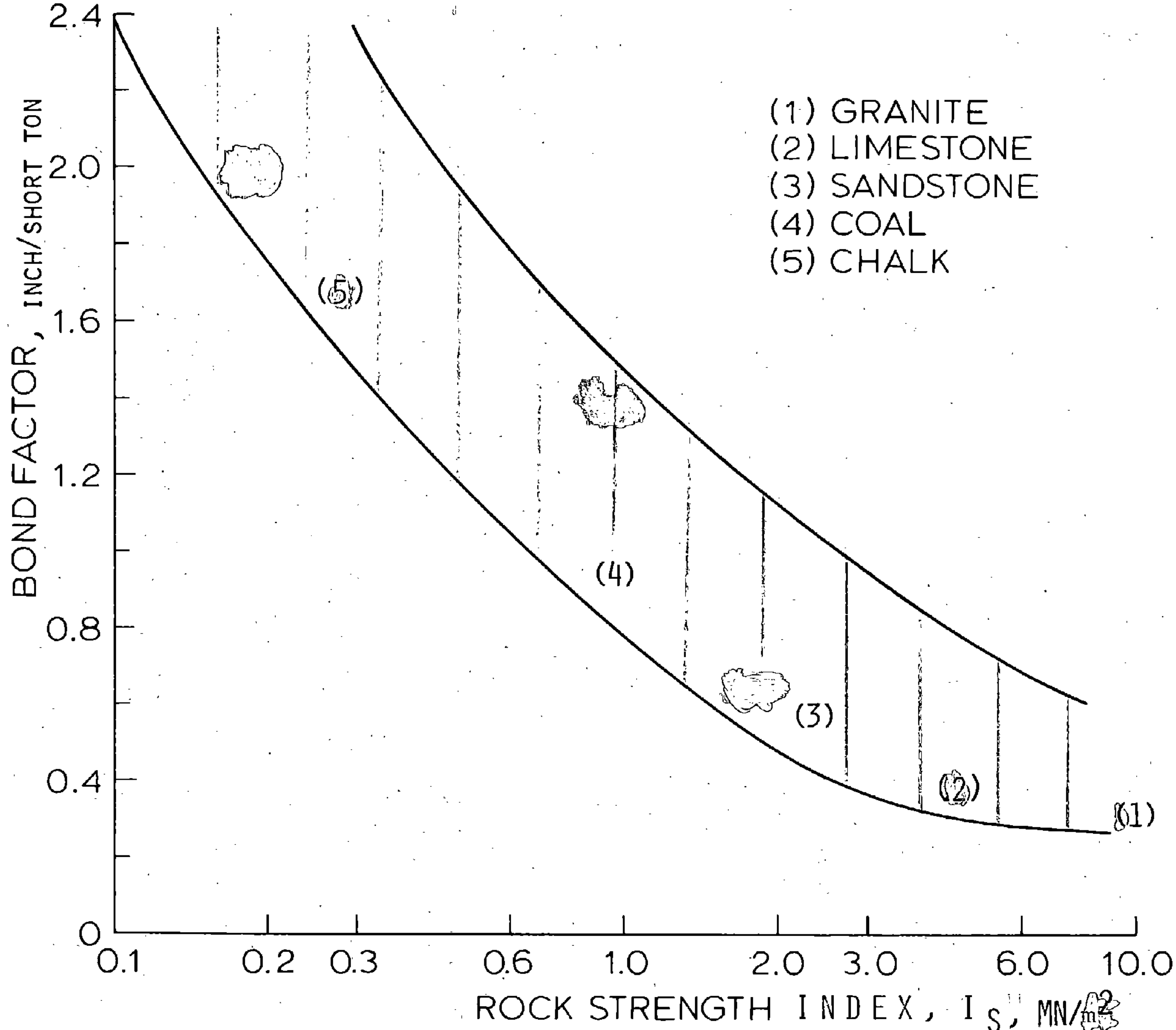


Figure 2.31 Resin Bolt (Anchor) Design Chart Showing the Relationship Between Bond Factor and Rock Strength Index. (1.0 inch diameter bolts in 1.25 inch diameter holes.) [2.18]

and (5) chalk. It is evident that the weaker rocks require a greater bond length to achieve the same overall strength, in agreement with Figure 2.15 discussed previously. The maximum anchorage of the resin bonded bolts ranged from 1.7 to 3.0 times that of mechanical anchors, depending upon the rock material. The greatest advantage of the resin bolt over the mechanical was achieved in the weaker rocks. (The maximum loads in Figure 2.30 depend upon the strength of the bolt steel which evidently was relatively high.) The results of Figure 2.30 have been replotted in [2.18] for design purposes and are shown in Figure 2.31 where the "bond factor" is defined as bond length/short ton, (inch/ton). The abscissa is the rock strength index I_s defined as $\sigma_u/16$ where σ_u is the unconfined compressive strength.

Albritton [2.21] has extended the above work to longer bolt (anchorage) lengths up to 9 feet and to weaker rock--a Hannibal Shale with $\sigma_u \simeq 200$ psi. Design charts are given in Figures 2.32 and 2.33.

One can consider using the data of Figures 2.31 and 2.32 in evaluating results of a finite element analysis where the load per inch of resin bolt is determined locally along the bolt for specific in situ conditions.

For shear loading, analytical results have not been reduced to design data to the extent done for axial loading. This is mainly due to the fact that research on shear loading was begun more recently and results are still incomplete for a variety of rock materials. However, even for the axial loading case ultimate loads have been used as design data, whereas another useful parameter for safe design is the axial stiffness (lb/in) which limits movement prior to failure. The axial stiffnesses as well as the shear stiffnesses should be given more attention in design considerations.

2.6 Support Requirements

A mine design computer program has been developed for selecting roof bolt plans for mechanical bolting, [2.114], that is based on support requirements. The basis used is to determine the closest competent bed in the geological section for the anchor horizon and use the distance to this bed as the required bolt length. Where a competent bed is absent, provisions are made for additional conventional support. Bolt spacing is allowed to vary only between four and five feet. The spacing and the need of straps is determined by the presence of disintegrable immediate strata. This computer program was applied to fifteen test cases. In six of the cases additional support by steel posts and bars were required by the program, whereas actual support practice was to use only header boards in two of these six cases, and nothing additional in one. In eight of the fifteen cases the program required additional steel straps. In actual practice no straps

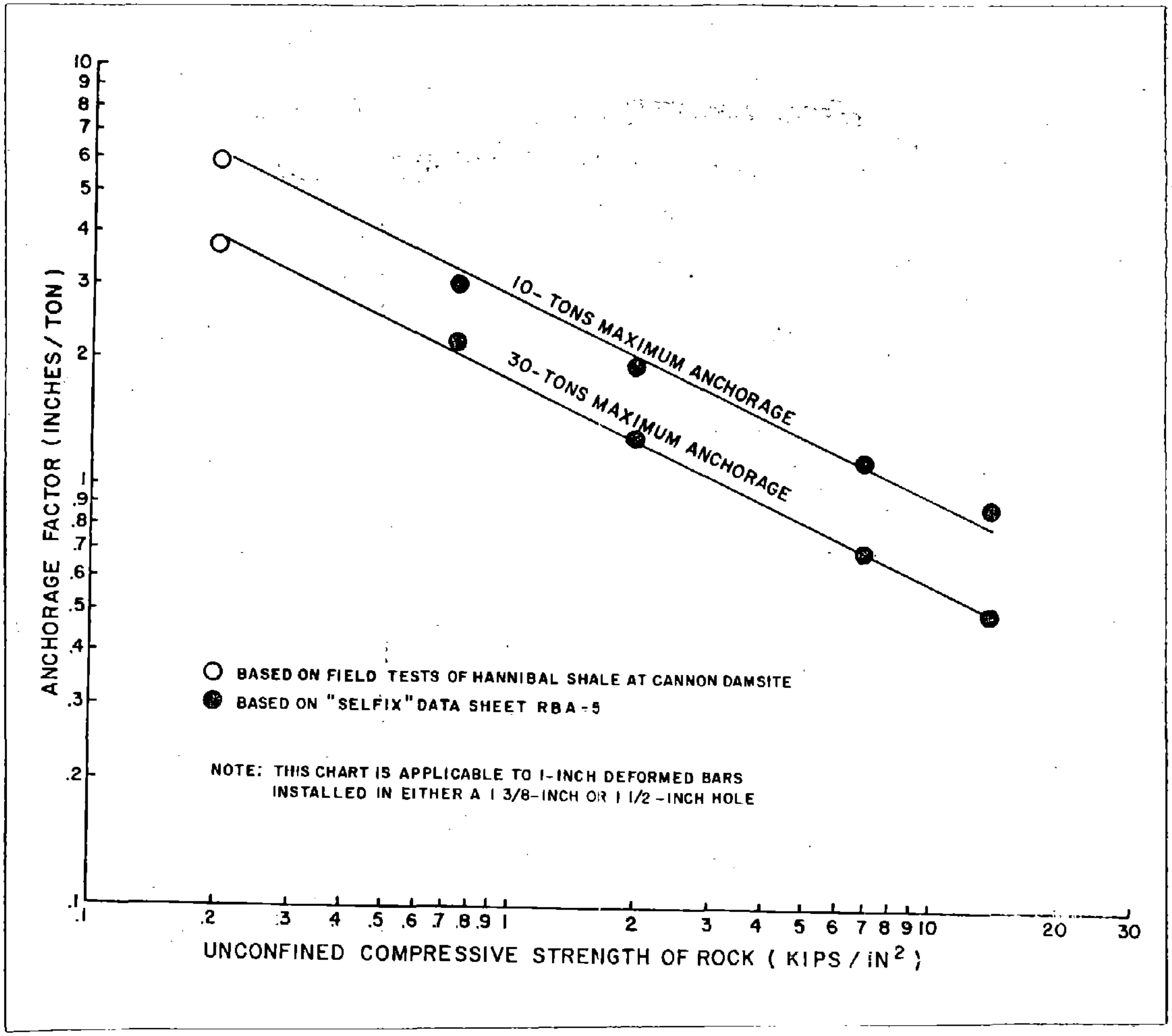


Figure 2.32) Guide for Estimating Anchorage Factor of Resin Bolts Based on Unconfined Compressive Strength of Rocks. [2.21]

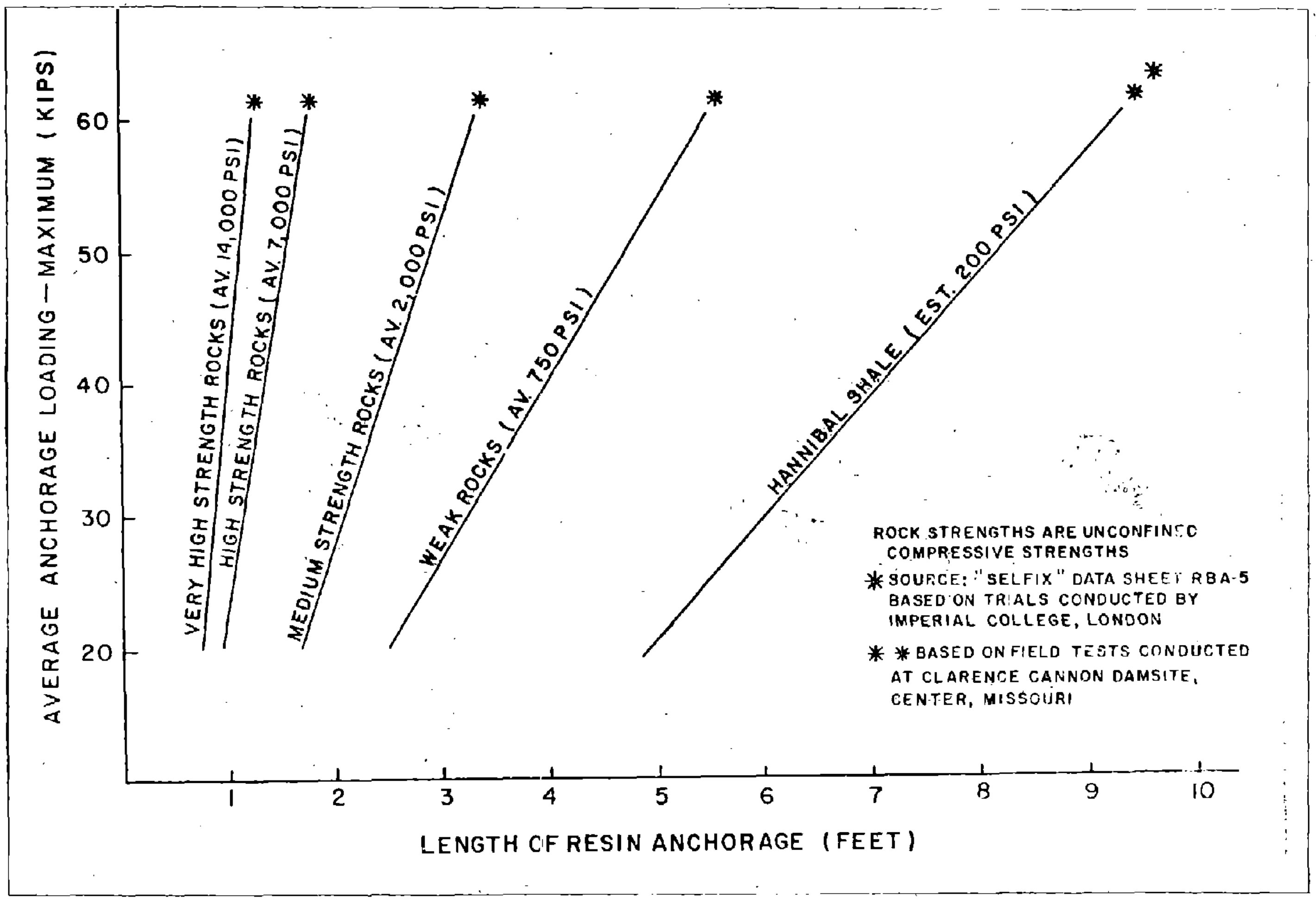


Figure 2.33 Guide for Estimating Length of Anchorage of Resin Bolts Required for Stress Level. [2.21]

were used, but in one of these cases posts were actually used. In general the program was conservative, perhaps too conservative. The basis for the program is simplistic and too much so - anchorage only determines bolt length. The objective here is to determine where resin bolting can overcome some of the shortcomings of mechanical bolting, remove excess conservatism from bolting design and establish a broader basis for bolt requirements.

Support requirements such as suspension and beam building that have been applied to mechanical bolting can also be applied to fully-grouted resin-bolting. These are considered here in addition to the shear reinforcement capability of the resin bolt. This discussion is restricted to the scope of this contract: bolting of horizontal flat-bed roofs. Support requirements for round tunnels are not considered here but are discussed in references [2.48, 2.55, 2.56].

2.6.1 Suspension

Simple suspension of the weight of loose slabs of roof rock has been considered by Panek [2.57], and Obert and Duvall [2.47]. Figure 2.34 shows a loose portion of mine roof that must be suspended by a row of bolts. The bolts must simply support the weight of the loose rock. It is shown in Appendix 1-A for bolts on 4-foot by 4-foot centers, that the load per bolt is about 10,000 to 13,000 lbs (or 5 to 6½ tons) when supporting a loose slab of thickness t equal to 4-feet. Using Figure 2.31 as a design chart and assuming suspension from a competent bed of sandstone with a strength index of I_s equal to 2, one finds a bond factor of 1.2 inches per ton or in this case $5 \times 1.2 = 6.0$ inches of bond length is required as a minimum for the length ℓ shown in Figure 2.27. For a safety factor of two one would take ℓ equal to 12 inches for design purposes. Thus the total resin bolt length in this case should be 5 feet to support the 4-foot slab.

A suspension effect can also be mobilized during the bending of multiple roof beams with built-in ends under gravity loading as shown in Figure 2.35. When the upper beam is thicker than the lower beam, as shown, the lower one tends to sag more and tends to be suspended from the upper beam. When the upper beam is thinner than the lower there is no suspension effect and the upper beam rests on the lower and causes the lower beam to bend more. Thus, as far as bending is concerned, the greatest bending strain will occur in the thickest beam [2.57].

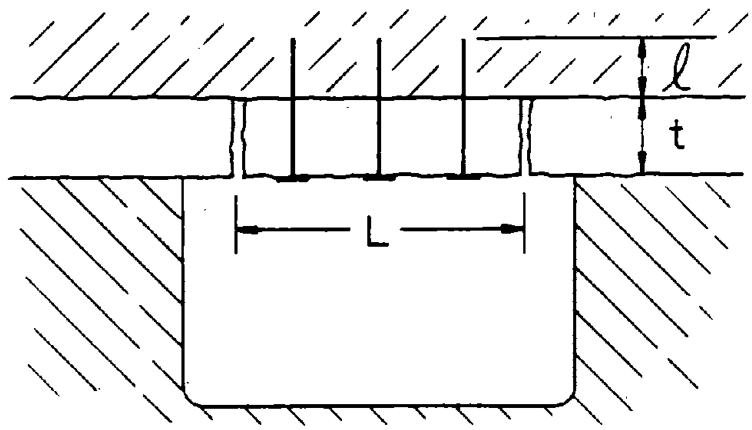


Figure 2.34 Simple Suspension of a Roof Slab [2.47]

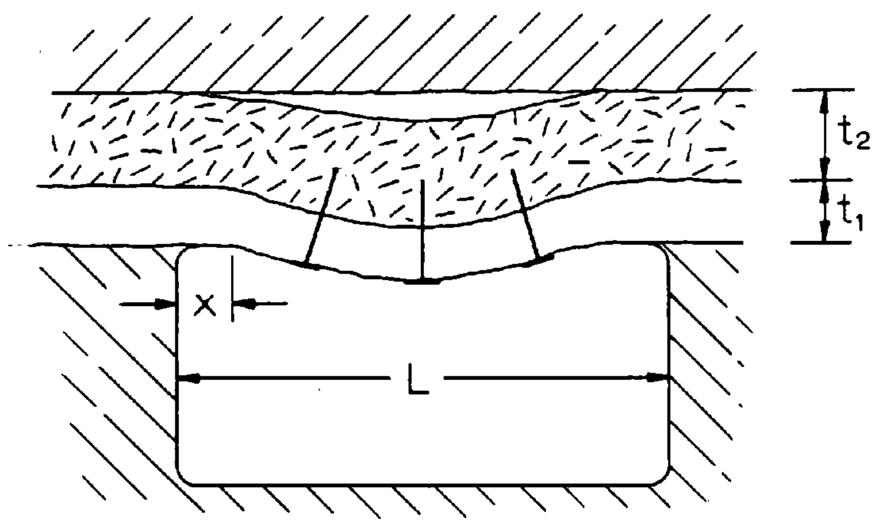


Figure 2.35 Gravity Loading of Multiple Roof Beams with Built-In Ends [2.47]

2.6.2 Friction

Next reinforcement of bedded mine roof by the friction effect of bolting is considered. If the roof beams or laminae are of the same material and thickness, they will bend the same amount under their own weight, and there will be no suspension effect. However, bolting can be effective if a friction effect is produced between layers due to clamping of the layers together by pretensioning of the bolts. Panek [2.58] analyzed this effect and his log-log design chart for the reinforcement factor RF from friction for mechanical bolting was developed. This effect cannot be realized for untensioned bolts except in the case where one could imagine an untensioned bolt developing tension after installation due to rock movement and consequently maintaining a compression and friction between rock layers that existed prior to bolt installation. However, this possibility is difficult to predict and to attach any numbers to for design purposes. It is more likely the case that bed separation has occurred before the bolts are installed.

Although friction from clamping is not a primary means of reinforcement for untensioned fully-grouted resin-bonded bolts, a comparable effect, the transverse shear resistance of the bolt itself, is an effective means of reinforcement as discussed later in Section 2.6.4.

2.6.3 Rock Arch

Mine roof can also be reinforced by the creation of a roof-bolt reinforced rock arch. This approach is promoted by Cox [2.61] and is illustrated in Figures 2.36 and 2.37. The rock arch is assumed to be made up of jointed rock with little or no tensile strength. As shown in Figure 2.36, as the beam sags, a crack opens at the middle and a horizontal thrust H is carried over one-quarter of the thickness of the beam. The dashed line indicates the boundary of the rock arch. Below this boundary tensile stresses can be developed 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 it and reinforce it. Cox [2.61] finds that if the roof-bolt reinforced arch is to remain stable the roof bolt length t must satisfy the following conditions:

$$t \geq \sqrt{\frac{whL^2}{216C}} \quad \text{to prevent compressive failure} \qquad (2.13)$$

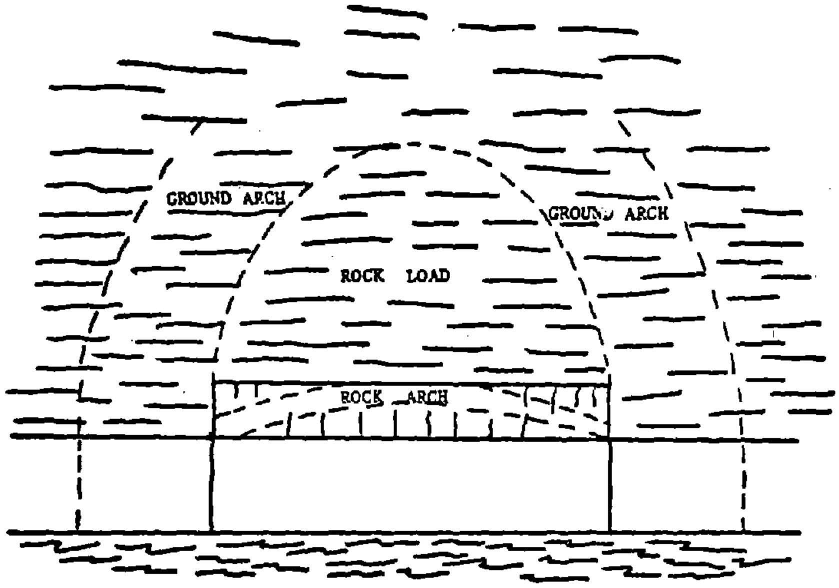


Figure 2.36 Illustrative Concept of Ground Arching and the Rock Arch. [2.61]

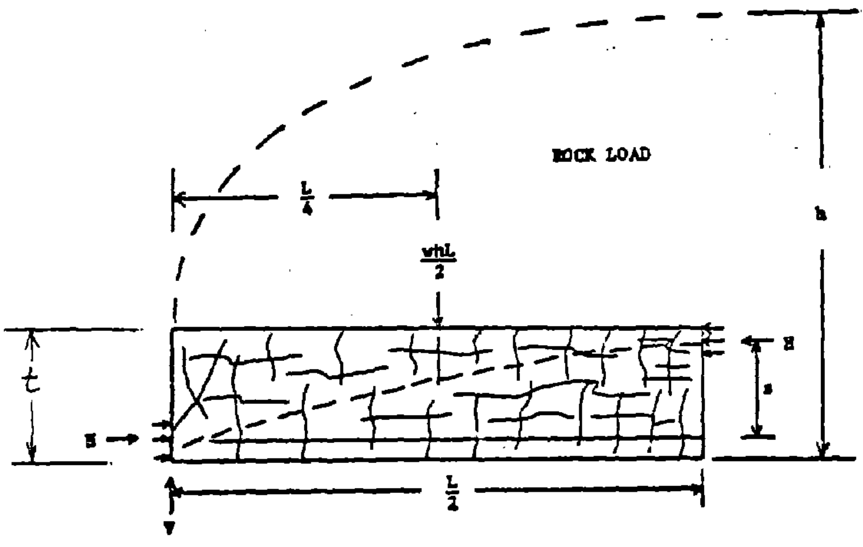


Figure 2.37 Mechanics of Rock Arch. [2.61]

$$t \geq \frac{whL}{72S} \quad \text{to prevent shear failure, and} \qquad (2.14)$$

$$t \leq \frac{\mu L}{3} \quad \text{to prevent slip along vertical fracture planes at the abutments} \qquad (2.15)$$

(It is assumed that the modulus of elasticity E of the rock is large enough to prevent significant changes of geometry due to sag of the roof.) In the above limits w is the weight density of the rock, lb/ft^3, h is the height of the rock load, ft., L is the opening width, ft., C is the compressive strength of the rock, psi, and μ is the coefficient of friction on the vertical fracture plane. These results are plotted in the form of design curves in Figures 2.38-2.41. Examination of these results leads to the following conclusions [2.61]:

(1) Compression failures are unlikely because relatively short bolts* are needed even for low compressive strength.

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

(3) Slip failures along vertical planes are always a potential problem for typical roof bolt patterns of 3-5 foot bolts with $0.4 \leq \mu \leq 0.6$.

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

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

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

The above conclusions relate only to this one mechanism of roof support, the rock arch. For example, conclusion (1) precludes any existence of in situ lateral stresses.

In the above analysis no distinction was made between mechanical and resin bolts. This remains yet to be done. One observation can be made, however. Angled bolts at the abutments can help prevent vertical shear failure. A resin bolt with its greater shear resistance would be more effective.

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*Note however from Figure 2.38, that the bolt lengths of 6-7 inches over a 32-foot span, as required by this theory [2.61], cannot be considered practicable. A new theory is presented in Volume IV of this report.

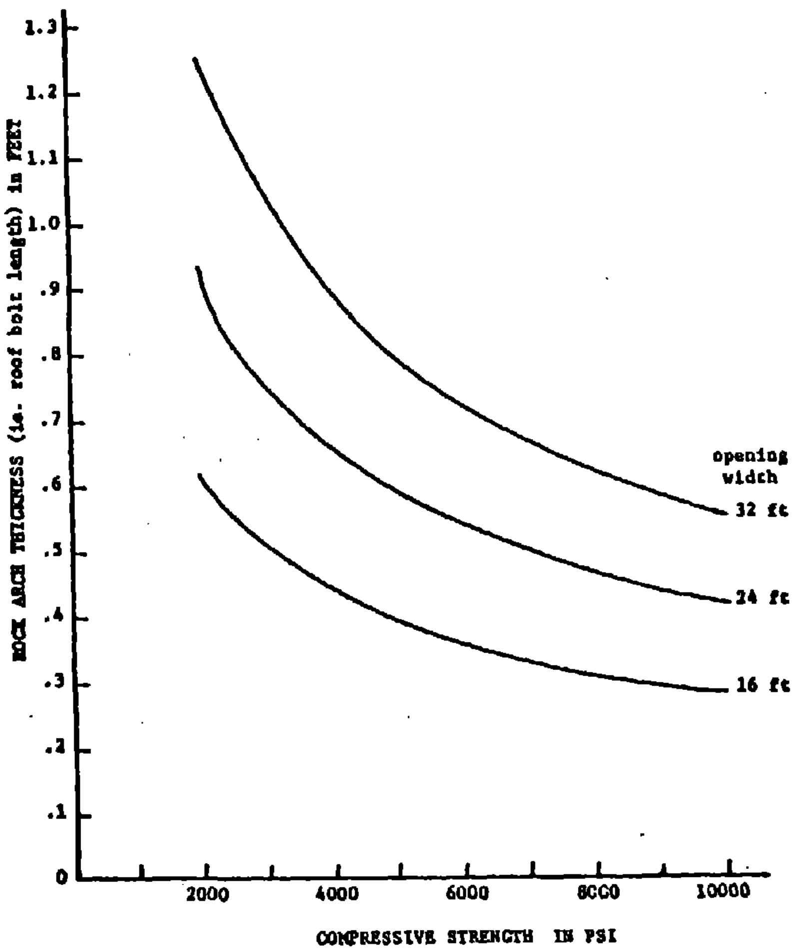

Figure 2.38 Relationship Between Roof Bolt Length, Compressive Strength of Roof Rock, and Opening Width for a Stable Rock Arch. [2.61]

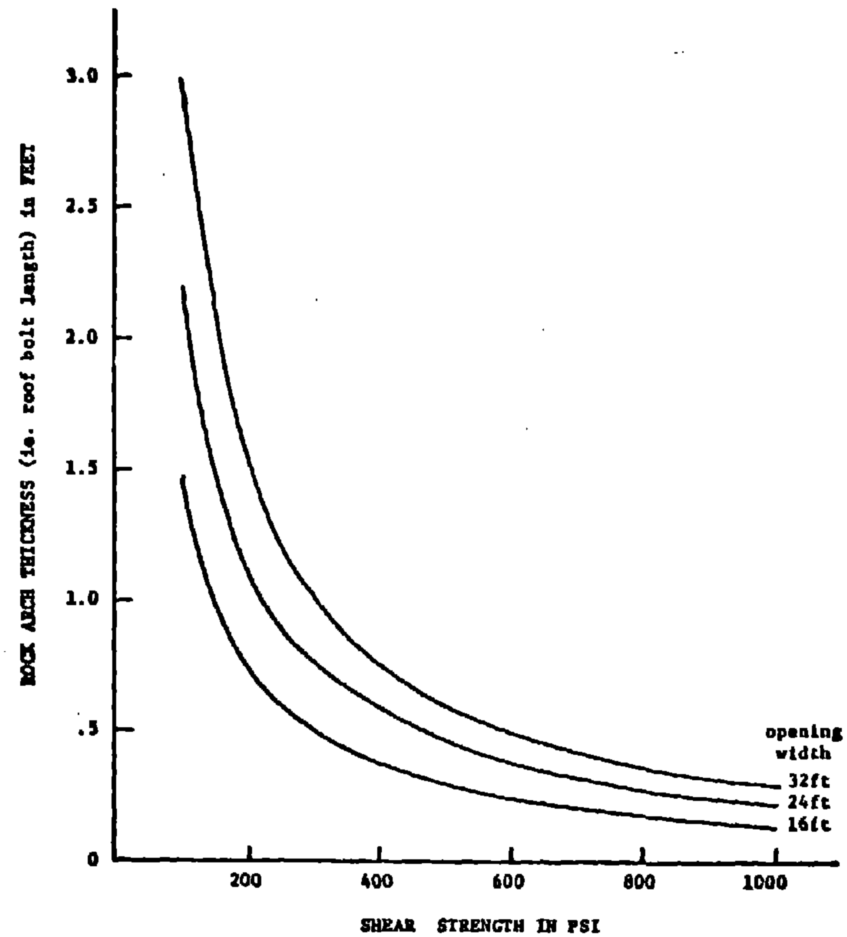

Figure 2.39 Relationship Between Roof Bolt Length, Shear Strength of the Roof Rock, and Opening Width for a Stable Rock Arch. [2.61]

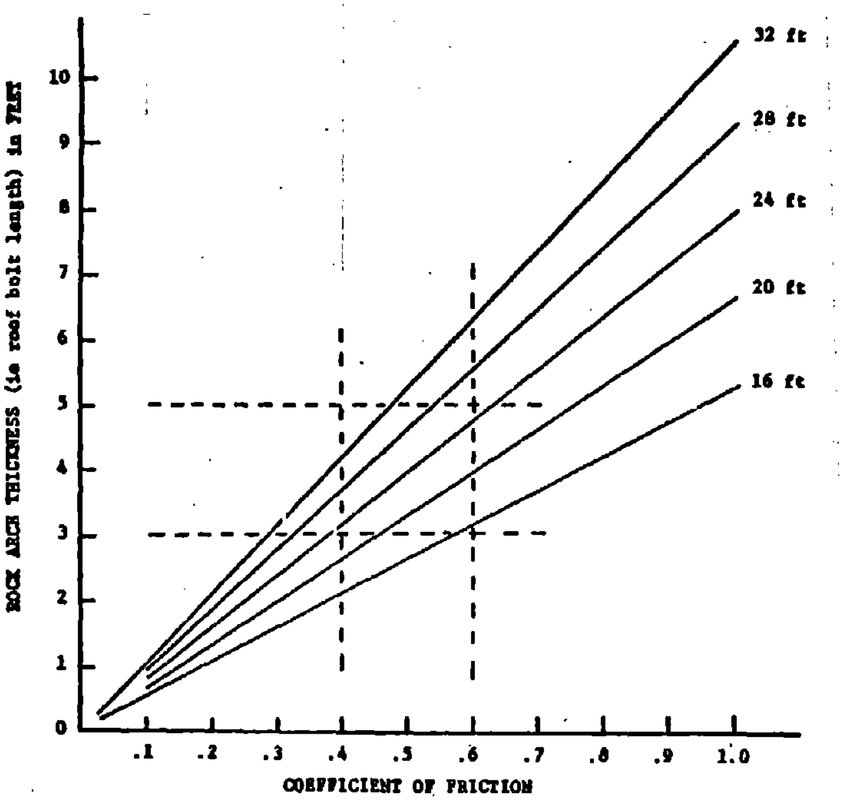

Figure 2.40 Relationship Between Roof Bolt Length, the Coefficient of Friction Along the Vertical Shear Plane, and Opening Width for a Stable Rock Arch.

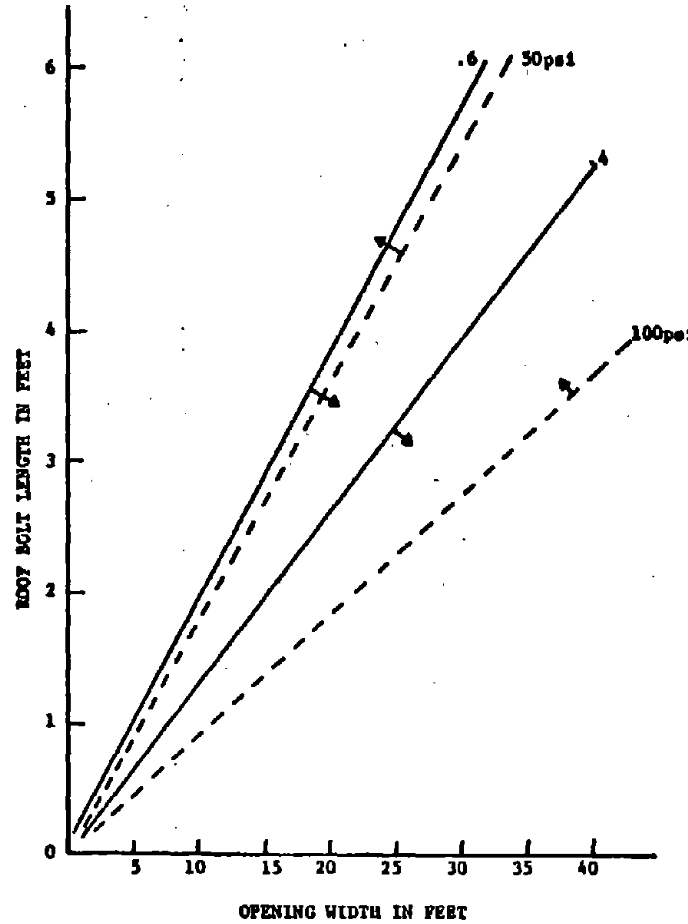

Figure 2.41 Rock Arch Stability as a Function of Roof Bolt Length, Opening Width, Shear Strength and the Coefficient of Friction. [2.61]

2.6.4 Interlaminar Shear Resistance

In Section 2.6.2 the friction effect due to mechanically clamping of laminar beams was discussed. The fully-grouted untensioned resin bolt can accomplish the same net result due to its shear resistance. Fairhurst and Singh [2.49] have analyzed this problem. They assumed a built-in beam of 4 rock layers each of thickness 4 inches*, a span length L of 20 feet, a 4' x 4' bolting pattern, and rock with a modulus of elasticity of 10^6 psi. Results are shown in Figure 2.42 where T_b is the total shear force developed in the bolts and T_r is the required shear force to prevent all interlaminar slip-point A in Figure 2.42. Point A is computed by calculating the shear stress at the various levels in a single competent beam of thickness 4t equal to 16 inches. Point C represents the extreme where all four layers act as independent beams and full differential slip occurs.

Points B and E in Figure 2.42 are situations where the bolts offer all the shear resistance with no shear resistance from the joints. Points between A and B or between A and E are situations where the rock joints also offer some frictional shear resistance. The slip S between layers and the beam deflection δ are related as follows:

$$\frac{S\ \max - S}{S} = \frac{\delta\ \max - \delta}{\delta - \delta\ \min} \quad \text{(within } \pm 10\%) \qquad (2.16)$$

As indicated in Section 2.3.2 realistic values of the resin-bolt shear resistance K_s are of the order of 1×10^6 lb/in. Thus the curve AE is realistic and untensioned resin-bonded bolts can be expected to reduce slip by 50% and roof deflection accordingly. However, results by Gerdeen [2.62] on a two-layered simply-supported beam showed only a reduction of 10% in slip for two feet thick layers. The shear effectiveness of resin grouted bolts will depend upon the number of layers and support conditions, and this needs further analysis.

More information on the interlaminar shear resistance shown in Figure 2.42, is found in [2.100]. Figure 2.42 was the result for zero in-situ horizontal stress. The effect of in situ stress is shown in Figure 2.43, where the difference between bolted and unbolted roofs is also shown. Greater slip and consequently a greater shear resistance is required when the in situ stress is greater than 950 psi, the Euler buckling stress for an individual unbolted layer. A plate buckling theory was used in [2.49] as explained in [2.100], and is discussed in Section 3.4.

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* One might question the practicality of 16-inch length bolts.

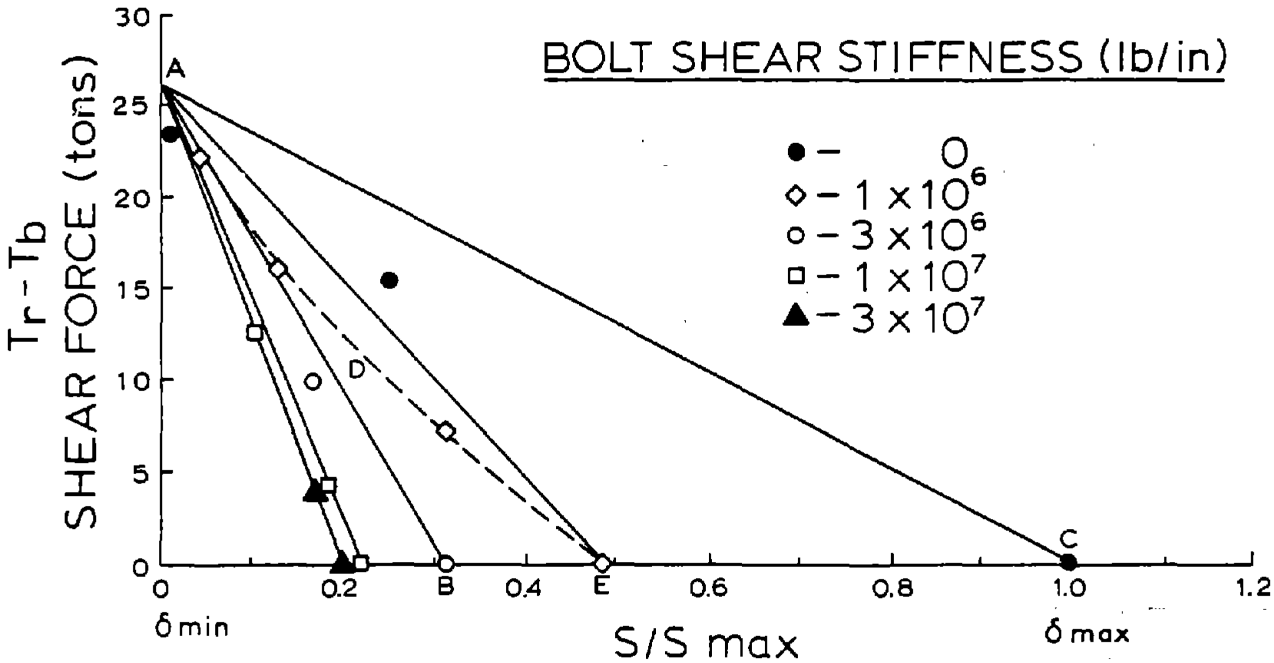


Figure 2.42 Shear Force That Must Be Developed By Sliding Resistance of Middle Bedding Plane to Maintain a Given Average Slip (S) or Deflection (δ) of the Layers

A reinforcement factor is developed based in the plate bending theory of [2.100]. The reinforcement factor (RF) is defined as:

$$RF = (I-nI_o)/(nI_o-I/n^2) \tag{2.17}$$

where I is the moment of inertia (second-moment of area) of the bolted layer system, I_o is the moment of inertia of a single layer, and n is the number of layers. EI is the bending stiffness, therefore (2.17) is equivalent to:

$$RF' = \frac{RF}{n^2} = \frac{\text{Bending Stiffness of Bolted System} - \text{nx(Bending Stiffness of 1 Layer)}}{n^3\text{x(Bending Stiffness of 1 Layer)} - \text{Bending Stiffness of Bolted System}} \tag{2.18}$$

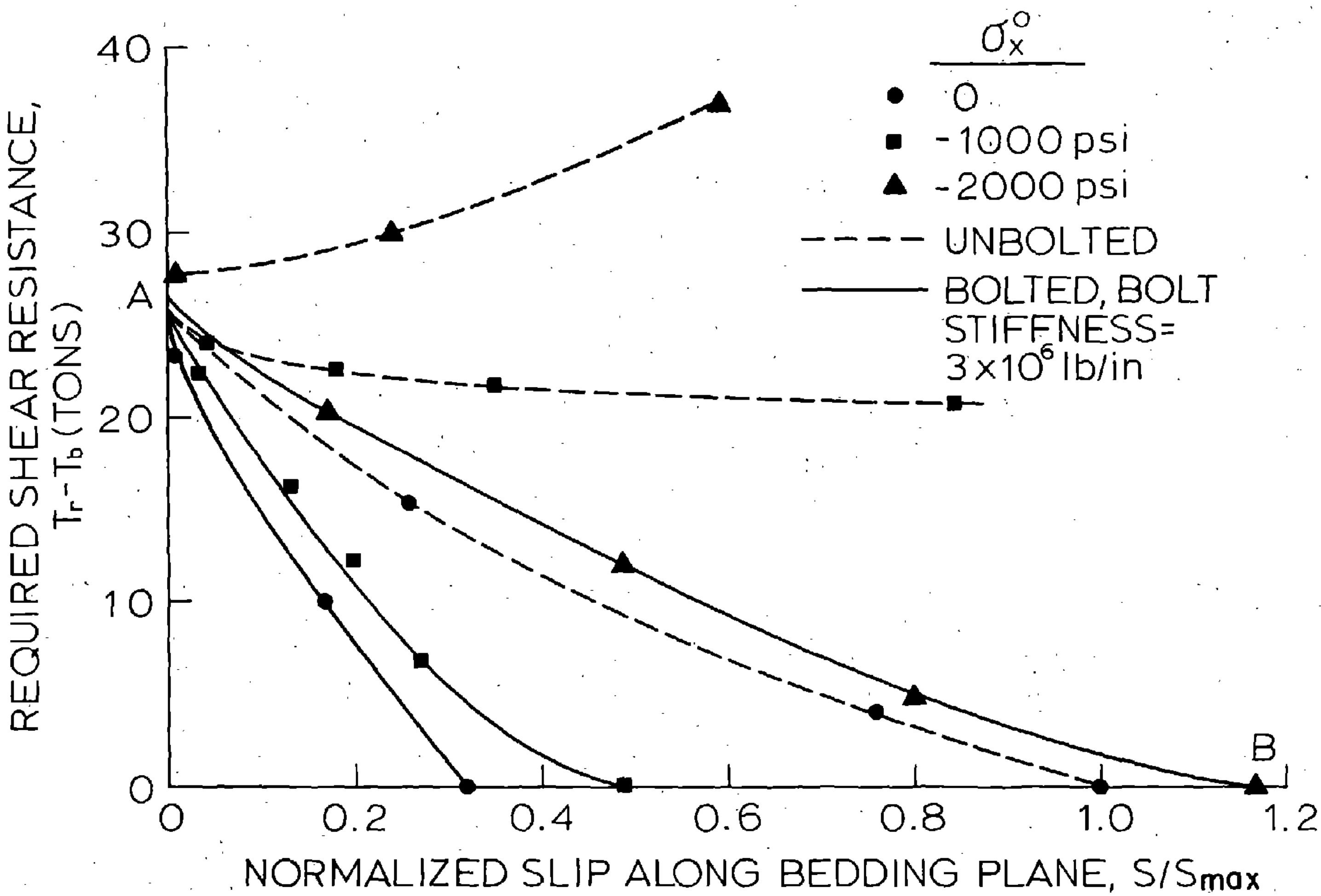


Figure 2.43 Shear Force That Must Be Developed By Sliding Resistance of the Middle Bedding Plane As Influenced By In Situ Stress [2.100]. σ_x^0 = In-situ Horizontal Stress.

For no bolts $I = nI_o$. For a monolithic beam of thickness equal to n layers, $\bar{I} = n^3 I_o$. Therefore, (2.17) gives RF = 0 for unbolted layers and RF = ∞ for 100% reinforcement equivalent to a monolithic beam.

A parameter study was conducted for bolted layers with zero resistance along bedding planes sagging under their own weight [2.100]. It was found that RF could be correlated to the parameters, K_s/Et, N, L/b and n as follows:

$$RF = \frac{L}{b} N n^{1/5} f(K_s/Et) \tag{2.19}$$

where L is the length of the span, b is the spacing between rows of bolts, N is the number of bolts in a row, K_s is the shear stiffness of the bolts, E is the modulus of elasticity of the rock and t is the thickness of a single layer. The function f is shown in Figure 2.44. (A simple expression could be written for f.)

It was found that RF was independent of t/L, when other parameters are held constant. The correlation (2.19) shown in Figure 2.44 led to the following conclusions [2.100]:

(i) A change in the spacing of bolts in a row has the same effect on the reinforcement factor as an equal change in the spacing of bolt-rows.

(ii) A bolting system ceases to significantly increase in effectiveness as the shear stiffness of a bolt (K_s) approaches the axial stiffness of a layer (Et).

Wright and Gesund [2.101], using their computer approach [3.50, 3.51], have calculated reinforcing factors for bolted multi-slab roofs under different conditions. They define the reinforcing factor (R.F.) as:

$$R.F. = \frac{\text{Load carried by n layers each of thickness t}}{\text{nx(Load carried by 1 layer of thickness t)}} \tag{2.20}$$

In contrast to the Fairhurst, Singh and Christiano definition based on stiffness [2.100], Wright and Gesund base their definition on load carrying capacity [2.101]. The latter include friction between layers, but does not account for stiffness of bolts. It may be assumed then that the latter may represent bolted layers beyond the elastic limit of the bolts (see Figure 2.28). Results are shown in Table 2.11. The first and third sets of results show how R.F. increases with number of layers. The second set of results show how

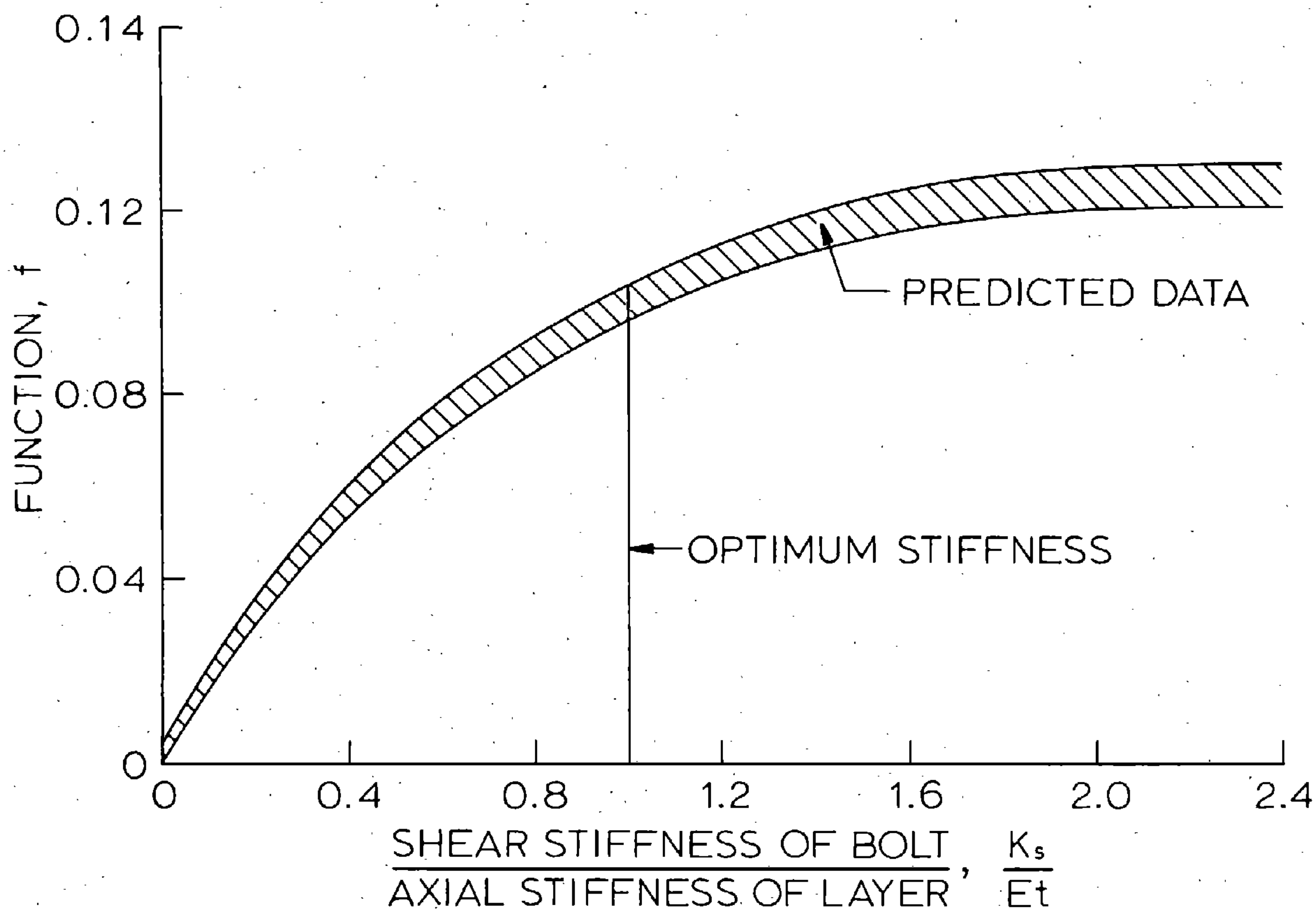


Figure 2.40 Theoretical Correlation Between f Calculated from Equation (2.19) and K_s/Et, [2.100].

TABLE 2.11 REINFORCEMENT FACTORS (R.F.) FOR BOLTED MULTISLAB ROOFS AT INTERSECTIONS [2.101]

Number of Layers n	Layer Thickness, t, in.	Interlayer Friction, psi	Tensile Strength* of Layer, Ksi	R.F.
2	4.	5.	3.0	1.49
3	4.	5.	3.0	1.68
4	4.	5.	3.0	1.82
5	4.	5.	3.0	2.40
4	4.	.5	3.0	1.32
4	4.	5.	3.0	1.82
4	4.	10.	3.0	2.01
4	4.	50.	3.0	2.85
2	12.	.5	3.0	1.9
3	12.	.5	3.0	2.96
4	12.	.5	3.0	3.81
5	12.	.5	3.0	4.02
2	12.	.5	0.3	2.0
3	12.	.5	0.3	2.4
5	12.	.5	0.3	3.2

* Compressive Strength of each layer equals 13.0 ksi
Modulus of elasticity of each layer equals 2×10^6 psi

R.F. increases with interlayer friction. (Interlayer friction is assumed here to simulate the effect of bolting.) The last set of results shows how a lower tensile strength material reduces the reinforcement effect for the greater number of layers.

An advantage of the fully-grouted resin bolt over the mechanical bolt can be appreciated [2.49]. For the former, failure at any point along the bolt length will not significantly reduce the total reinforcement effect since the resin bolt adds shear resistance to each layer independently. With point-anchored bolts, however, failure at any point completely eliminates all reinforcement.

2.6.5 Fragmented or Loose Granulated Rock

Alexander and Hosking [2.48] discuss the support of loose granulated rock by roof bolting. Tensioned bolts are required to compress the loose rock into a somewhat competent whole. Also wire mesh is required to hold up loose fragments between the bolts. The untensioned fully-grouted bolt is not applicable in this case (other than a hanger). However, perhaps something can be learned and adapted for the situation where a highly jointed roof, precompressed by in situ stresses and bolted with untensioned grouted-bolts, later has the tendency to develop cracks. Alexander and Hosking use an F number defined as the ratio of the distance between bolt bearing plates to the diameter of the rock fragment that would result in the formation of small stable arches between the bolts. The F number depends upon the shape of the fragments and the coefficient of friction of the material. Experiments showed the results given in Table 2.12.

The above experiments [2.48] were conducted against horizontal restraints, i.e. horizontal compression existed as well. Coates and Cochrane [2.63] also conducted experimental analysis of the F number using bolted beams of blocks of various materials, but evidently without horizontal support--they appear to have been simply supported beams. Their results are shown in Table 2.12. No difference was found for blocks oriented vertically or at 45°. When the spacing of the bolts exceeded their length, a stable beam could not be constructed. A bolt length at least three times the width of the joint blocks is recommended in both [2.48] and [2.63].

Table 2.12 Experimental Results for F Number* [2.48, 2.63]

Shape of Fragment	Material	Maximum Stable F Number	Ref.
Spherical	Glass	3	[2.48]
Angular (flaky)	Crushed quartz diorite and crushed granite	5	[2.48]
Cubical	Crushed quartz diorite and crushed granite	7	[2.48]
Rectangular	Plaster	5	[2.63]
Rectangular	Hardwood and Plexiglass	3	[2.63]

* The F number is the ratio of the distance between bolt bearing plates to the diameter of the rock fragments.

From the above results, to be safe, (considering "slickensided" cone blocks to be similar to plexiglass) an F number of 3 as a maximum would be recommended, i.e. the space between bolts should not exceed three times the average block size of fragmented rock, unless there is horizontal stress, of course. The degree of confinement is important.

Certainly bearing plates have to be used with resin bolts to support blocky and fragmented rock. Coates and Cochrane [2.63] and Panek and McCormick [2.60] give requirements on the size of the bearing plates required. Steel bearing plates are usually 1/4 to 3/8 inch thick and 6 to 8 inches square and may be flat or embossed into a doughnut shape to aid positioning of a bolt when the roof surface is not perpendicular to the bolt hole. Some mines use wood header blocks (2 x 6 x 18 inches or 2 x 8 x 24 inches) in addition to steel bearing plates to reduce spalling of loose rock between bolts.

2.6.6 Rock Quality

The above discussion leads naturally to more general support requirements based upon rock quality. How to classify rock quality will be covered in more detail in Section 4 of this review. Here the use of rock quality indices to determine roof bolt requirements will be reviewed briefly. Barton, Lien and Lunde [2.64, 2.65] have done extensive work along this line. Their design criteria are illustrated in Figure 2.45. Plotted on the ordinate (for our purposes) is the span/ESR. ESR is the "excavation support ratio" which reflects the safety and support desired. For temporary mine openings ESR = 3 to 5 is suggested. The rock mass quality Q is defined later in Section 4 as already mentioned. For present purposes, the qualitative classification form "exceptionally poor to exceptionally good" rock should suffice. Untensioned grouted bolts as the sole means of support are recommended only for support categories 1-13 in Figure 2.45--only for rock in the "good to exceptionally good" quality range. For poorer rock other means of support in addition to bolting are recommended: mesh, chain link mesh, etc. for tunnels. (Coal mines appear to get along without such.)

The authors' (Barton, Lien and Lunde) [2.64, 2.65] experience has been primarily with circular or arched tunnel support. More work is needed to be done to classify rock quality support requirements for flat-roof mine openings.

Back in 1954, Rabcewicz [2.102] in Sweden classified the suitability of rocks for mechanical roof bolting based on rock softness measured with an impact tester. The impact tester consisted of a small steel ball shot by a spring with constant force against the rock. The impact leaves a small cavity. The diameter of the cavity is measured and used as a measure of "softness" of the rock. Results are shown in Figure 2.46. This was before the day of the resin bolt. However, the better anchorage for the concrete anchor may give an indication of the capability of resin anchorage in soft rock.

The "Penetrometer" reportedly developed by The Bureau of Mines for hardness testing could be used to develop a chart similar to Figure 2.46, but for resin bolting.

In a recent paper, Dejean and Raffoux [2.103] classify rock and the role of roof bolting as shown in Table 2.13, (a) and (b). Examples of Type A rock are thick beds of sandstone or conglomerate or massive igneous rock. Type B rocks include thick schistose roofs, more or less argillaceous. Examples of Type C rock are sandstone and limestone beds. Type D rocks include thinly laminated shales and thin seams of coal riders in the roof. The rock is classified by using the hardness type test discussed in [2.105].

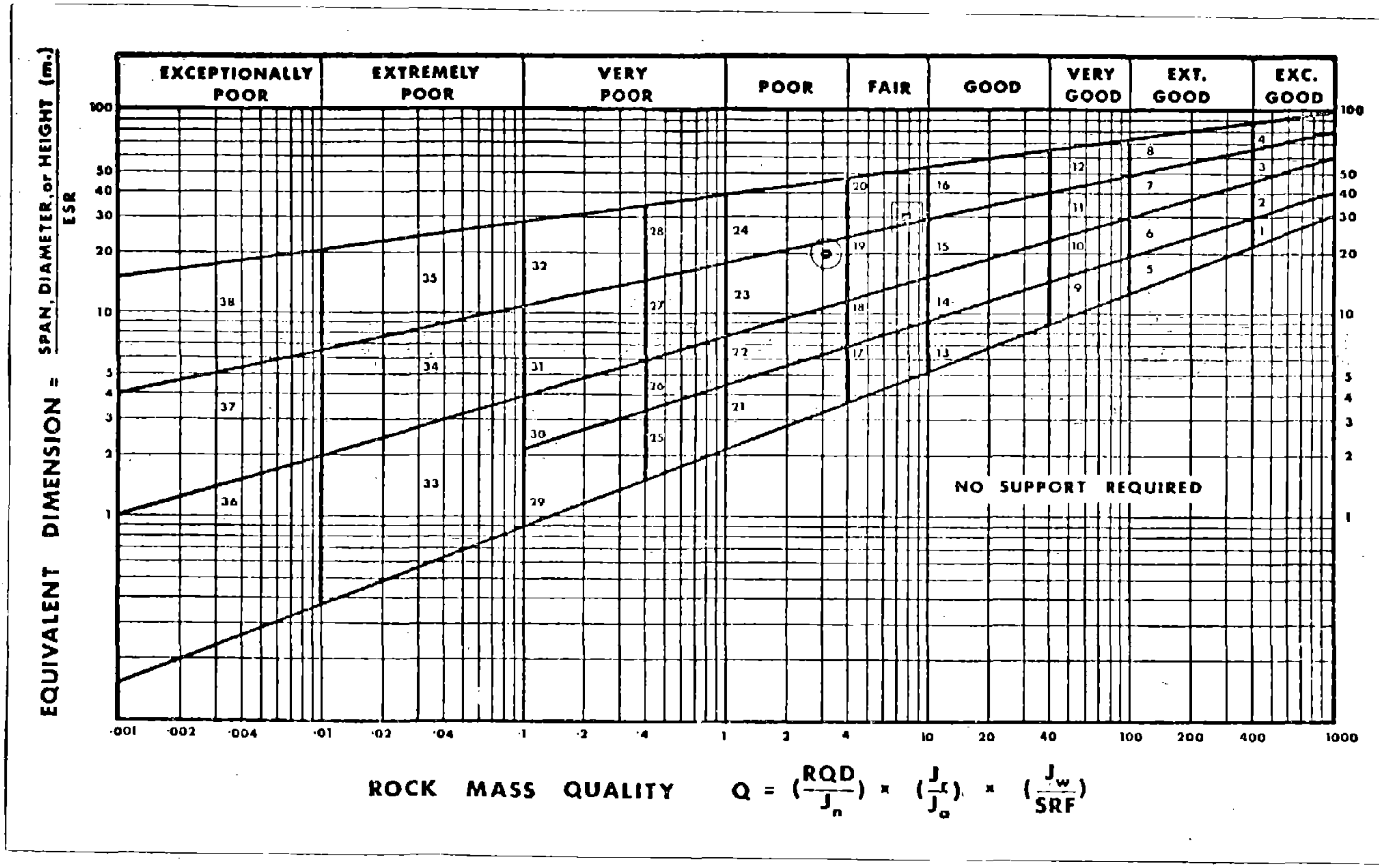


Figure 2.45 Excavation Support Chart Based Upon Rock Mass Quality [2.64, 2.65]

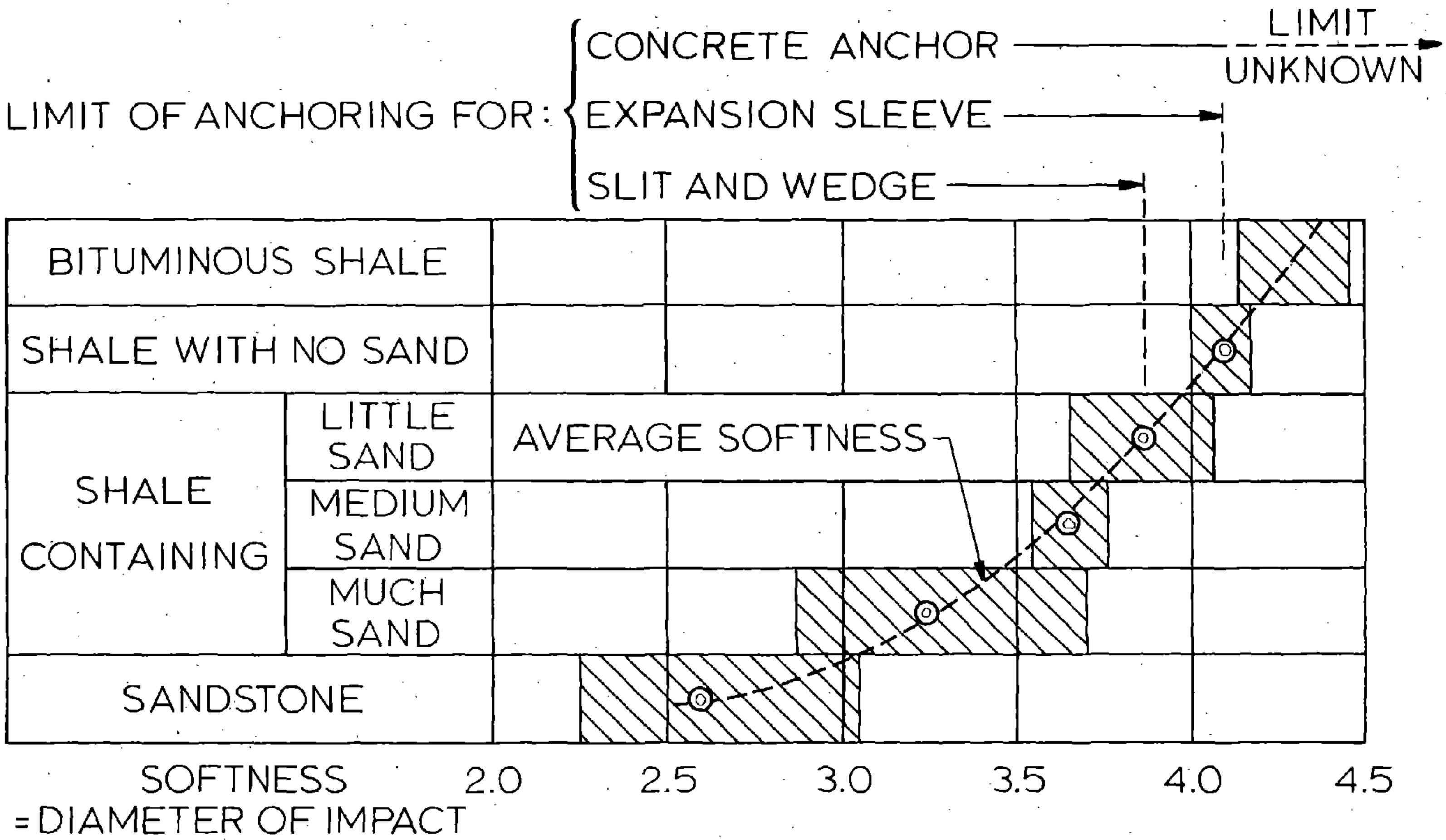


Figure 2.46 Classification of the Suitability of Rocks for Roof Bolting by Means of the Impact Tester [2.102]

TABLE 2.13 TYPE OF GROUND AND ROLE OF ROOF BOLTING [2.103]

(a) Type of Ground

	Strong	Weak
Homogeneous	Type A	Type B
Stratified	Type C	Type D

(b) Role of Bolting

Type A	To resist superficial flaking and prevent collapse of loosened slabs.
Type B	To increase the cohesion of the rock.
Type C	To resist relative horizontal and vertical displacements of beds. To increase rigidity of compound beam. To tie together blocks of broken beam.
Type D	Same as for Type C, but in addition to increase the cohesion of the rock.

2.7 Mine Experience With Grouted Bolts

Use of fully-grouted resin bolts in coal mines is the subject of another task of this research contract, namely the field survey. However, some mine experiences have been reported in the literature and are reviewed here.

Initial trials of the resin bolt concept in the United States involved resin bonded anchorages [2.20, 2.25] using resin cartridges about 12 inches long imported from Germany. Little difference was found between resin anchors and conventional expansion shells in firm shale roof. However, in tests in four coal mines [2.20] with roofs composed of soft shale and coal, significant differences were found. Pull loads on the resin anchored bolts ranged from 24,600 to 32,000 lbs with negligible bolt displacement as compared to erratic results with less than 14,000 lbs capacity for the conventional bolts. Similar results were found in another coal mine [2.25], but lower values were found for resin bolts anchored in the coal rider--pull loads of only 10-15,000 lbs were found. This was a roof with abnormally adverse conditions, however.

Davidson, Grosvenor and Gardiner [2.22] report on the trial of resin anchored bolts in a trona mine in Wyoming and found superior results over the conventional mechanical anchor, based on load cell and roof sag data. McCormick, Hollop and Debevec [2.66] reported in 1974 that 50 coal mines had been approved for fully-grouted resin bolts as the sole means of support. They also reported on four experimental mine installations. In one mine six foot resin bolts were found to perform better (significantly less roof sag) than 12-foot conventional bolts. In another mine four foot resin bolts successfully replaced four to eight foot mechanical bolts and supplemental supports. In another, five foot resin bolts replaced 10 to 12 foot mechanical bolts plus supplemental supports. In still another six foot resin bolts replaced four to eight foot mechanical bolts with the additional benefit that sloughing and deterioration was less because the resin sealed off the bolt holes and prevented water problems. Some roof falls were still reported in resin bolted mines but the frequency of failures was reduced. "Glove fingering" was mentioned as a problem when the difference between bolt and hole diameter exceeded 1/4 inch--the Mylar package failed to shred causing improper mixing.

Extensive evaluation of the fully grouted resin-bolt has been made in the shale roof of the White Pine Copper Mine [2.37, 2.67, 2.68, 2.69]. Superior performance over conventional bolts was found, based on convergence data and pull tests. Roof falls were dramatically reduced. Material cost comparisons for four-foot bolts were made (in 1974) as follows:

5/8 mechanical bolt in 1 3/8 inch hole $2.10

7/8 resin bolt in 1 1/8 inch hole $3.70

3/4 resin bolt in 1 inch hole $3.40

Although costs are more for the resin bolt (of the same length), safety has been the No. 1 priority. Further efforts have been aimed at cost reduction. One such effort has been the development of the capability of small hole drilling.

Additional efforts were made to reduce installation time. A "bent bolt" concept was developed whereby a slightly bent bolt lodges and holds itself in the hole while the resin hardens. Additional efforts are being aimed at reducing bolt length to a uniform four foot length or less. It is stated [2.69] that results under White Pine conditions indicate that for the most part the roof bolts act to suspend loose, blast-damaged rock, and to pin broken slabs together rather than form a monolithic beam equal in thickness to the bolt length. The White Pine Mine is relatively deep and subject to high horizontal stress. The "Voussoir" rock arch (Section 2.6.3) may be the primary mechanism of roof support in parts of the mine. Tensioning of mechanical bolts may prevent the formation of a stable arch whereas untensioned resin bolts may allow it [2.69]. Under the high horizontal stresses near dipping faults, the resin bolts withstand appreciable transverse shear and accommodate both compressive and tensile stresses along their length [2.12].

Resin bolts have drastically reduced roof falls and water problems in a clay mine [2.70], and eliminated the need of posts and cribbing. Overall a 30% decrease in roof support costs was realized.

Adamek [2.71] and Moebs [2.72] report on USBM investigations in an experimental resin bolt area in a coal mine in West Virginia. Four and five foot, 3/4 inch resin bolts were installed in 1-inch holes between mechanical bolts and then the mechanical bolts were removed and the area was "dangered off." Deflections of bolt heads of resin bolts were monitored, and differential sag stations were installed and monitored. Roof motions were small and without any pre-indications (or means to observe) part of the area caved in one year later. Some of the observations were:

(1) The trend and magnitude of vertical bolt head movement correlated with movement of points nine feet up in the overburden.

(2) In the caved area, the rock had separated at two feet above the resin bolts--about six feet up.

(3) Mechanically bolted areas adjacent to the resin bolted area did not cave, however, parts of the resin bolted area adjacent to the mechanical bolts did not cave either.

(4) In the resin bolted area, the head coal (3-4 inches) fell down or separated from the black shale--a phenomenon unique to the resin bolted area.

(5) The combined effects of slickensides (slips) and time-deteriorating drawslate may have weakened the roof structure.

A tentative conclusion reached [2.72] was that untensioned resin bolts were not as effective as tensioned mechanical bolts under these conditions. The actual stress condition was unknown. The overburden was 700 feet.

Kwitowski [2.73] reported on a USBM study of the trial usage of full-column resin-bolts in the Jenny Mine. The roof bolting involved 4 foot, 7/8 inch diameter bolts in 1-1/8 inch and 1-3/8 inch diameter holes on 4 foot centers. Mechanical bolts 8 foot long were also used for comparison purposes. The roof consisted of 6-8 feet of shale overlaid by competent sandstone. It was found in general that at the lower strata levels the resin bolted roof tended to approach stability sooner and showed less overall roof deflection than the conventionally bolted roof. Bed separations occurred at similar horizons for both types of bolting. The 1 1/8 inch resin system appeared to be more effective than the 1 3/8 inch system.

Anderson [2.74] reported on the use of full-column resin bolting in a U.S. Steel Mine with a roof of soft drawslate, laminated coal and rock ash, ranging up to 9 feet thick, where the presence of water had caused severe roof problems. The use of resin bolts dramatically reduced roof falls, reduced raw coal ash from 40 percent to 28 percent and increased production of clean coal by 23 percent. The use of 4-foot bolts on 4-foot centers both lengthwise and crosswise was unsuccessful, but the use of 4-foot bolts on 4-foot centers crosswise and 3-foot centers lengthwise was successful. The latest plan utilized 5-foot bolts on 4-foot centers in both directions, and it was successful.

Resin bolting appears to be used less in Europe as a primary means of support than in the United States. Curth [2.75] reported on a study of 25 coal mines. Longwall mining is used extensively and only roadways are permanently supported--generally by yielding arches. Roof bolting has not found much application, and because of the hardness of the rock, stopers are used more than rotary bolting machines. In France roof bolts are used more frequently than in Germany. Bolts are installed on a radial pattern (angled near the ribs), on 30-40 inch centers both lengthwise and crosswise. Usually full column 7/8 inch resin bolts are used, 6-7 feet long, combined with steel channels and posts. Recommendations from the Essen Research Center [2.75] are that the lengths should be equal to one-half the road width, where bolts are used exclusively and equal to one-third the road width, where bolts are combined with steel sets. Wood resin bolts are used near the face if caving is a problem between the canopies and the longwall face. Wood bolts are used because they are easily cut off and do not interfere with mechanical plows or shearers.

Raffoux [2.76, 2.77] reported on resin bolting of arched and flat roofed roadways in France. Carr has reviewed his work and has made recommendations to the National Coal Board in the United Kingdom [1.1, 2.78, 2.79]. Raffoux gives two rules governing roof bolting:

(1) Do not plan to use roof bolting as a sole means of support immediately below a pillar, pillar edge or alongside a fault.

(2) When coal beds are found within the first five meters of the roof they have a harmful effect. Specifically, if the thickness of the coal in that five meters is more than 20%, the bed separation will exceed 50mm, the limit of acceptable roof behavior, and therefore roof bolting should be prohibited. (Bed separations are measured in the first two meters of roof. If bed separation exceeds 50 mm (2.5% of two meters) (after 50 m of drivage, then additional support besides roof bolting is necessary).

Raffoux tested various lengths of resin roof bolts and found a minimum length of 2.2 m was recommended in the roadways of the collieries of the Lorraine Coal Basin. The ribbed rebar was found superior to smooth bar. Wire mesh and steel lagging is used in addition to bolts to prevent spalling in soft coal.

Bello and Serrano [2.56] reported success on using cement grouted bolts in circular tunnels. Salykov et al [2.80], reported on the use of reinforced concrete roof bolts in Russian mines. Such bolts did not always ensure stability and the authors recommend research to replace the sand and cement mortar to strengthen roof bolts and recommend synthetic resins as a substitute. It was also reported [2.80] that a shorter roof bolt is required at greater mine depths in the Russian mines, because of less convergence at greater depths.

Kwitowski [2.114] reported on measurements of mine roof deflections in both a resin-bolted test panel and an adjacent conventionally-bolted entry in a mine in the Pittsburgh No. 8 Coal Seam. The mine had 550 feet of cover and the immediate roof consisted of sporadic roof slate and 12 inches of roof coal overlaid by about 4 feet of clay slate. Above the immediate roof the main roof consisted of 4 feet of bastard lime overlaid by about 10 feet of limestone. The resin bolts were 3/4-inch x 5-foot rebar. The mechanical bolts were 5/8-inch x 8-foot length. Results showed the following:

1. Removal of posts in the resin area showed no significant effects on the roof.

2. Most of the separation occurred within the bolted levels for both bolt types.

3. There exists a zone between the 10-1/2 and 8-foot levels that is in compression for both bolt types. (This may indicate a Voussoir rock arch mechanism.)

4. The resin bolts performed better or at least equal to the conventional bolts.

Sawyer and Karabin [2.94] reported on the use of strain-gaged resin bolts in three different bituminous coal mines; one in Illinois and the others in Pennsylvania. (They confirm similar findings by authors in a copper mine [2.12].) The important findings are: (1) compressive and tensile axial loads develop in fully resin-grouted bolts, (2) at the horizon of a strata separation, extremely high tensile loads develop in resin bolts, sometimes exceeding their elastic limit, (3) axial loads vary considerably along the bolt's shaft, and (4) considerable fluctuation in axial loads occurs when mining in the vicinity of the bolt.

Figure 2.47 shows axial loads immediately after installation and one week later. Initially, 2000 lbs tension at the bolt head and compression at other locations was recorded. One week later, tensile loads were recorded at 20, 34 and 48 inches while the load at the bolt head remained constant. The bolt appeared to yield at the 20 inch level. This correlated with a significant strata separation (>.100 inch) measured on an adjacent differential sag station.

Figure 2.48 shows correlation between cyclic loads in a strain gaged resin bolt and variation in strata separation with time in a bituminous U. S. coal mine.

Mine experience with resin bolting in France has also been reported. It is reported that 475,000 bolts were installed in roadways in French coal mines in 1970, [2.95]. In 1974, 127,000

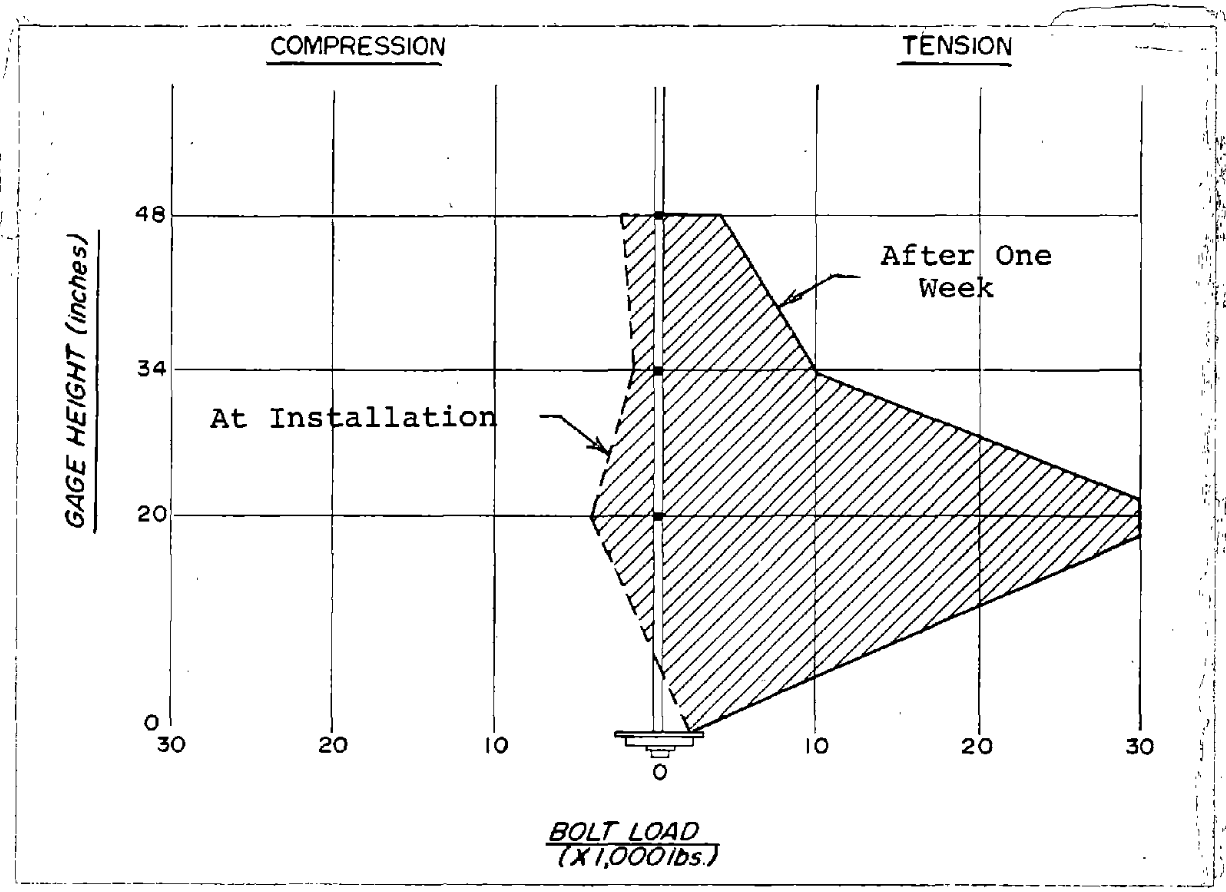


Figure 2.47. Axial Loads in a Strain-Gaged Resin Bolt Immediately After Installation and a Week Later [2.94]

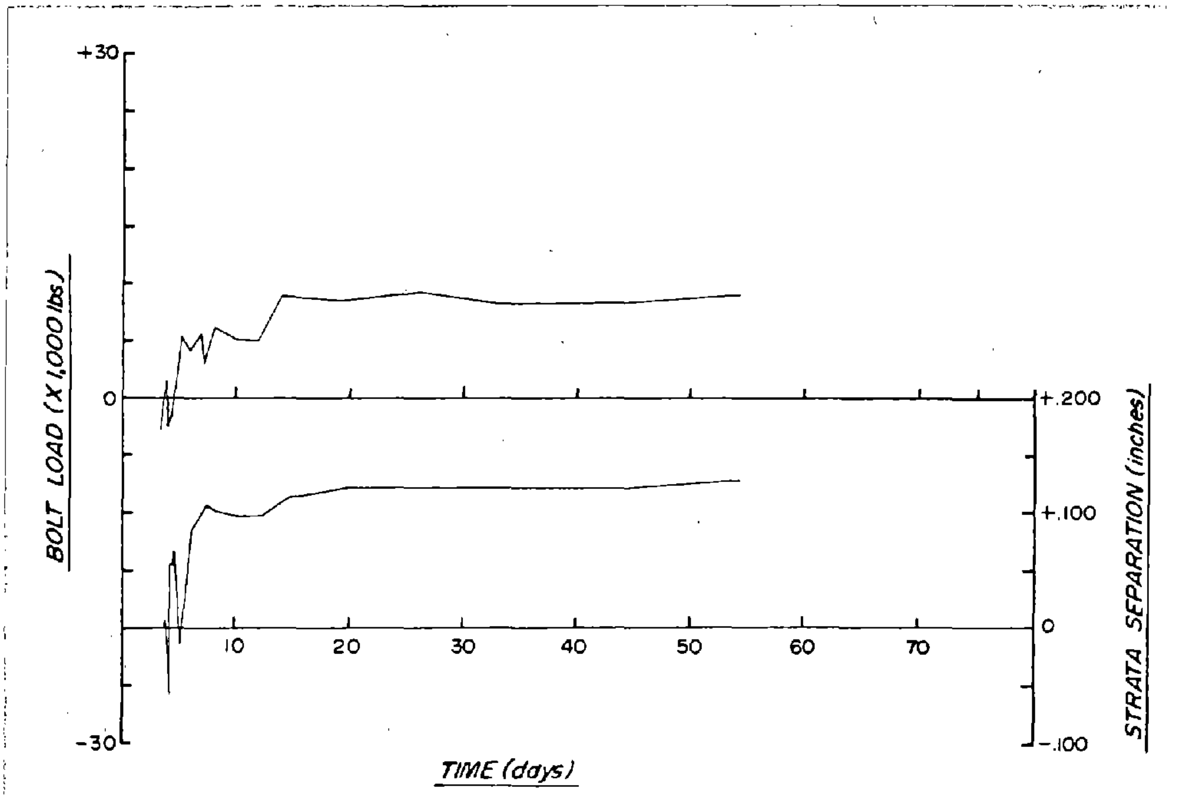


Figure 2.48 Typical Strain-Gaged Bolt Load-Strata Separation Time Series [2.95]

mechanical bolts and 930,000 resin bolts were installed in coal mines, and 1,790,000 mechanical and 1,453,000 resin bolts were installed in iron mines [2.103]. The resin anchored bolt has been highly successful in strata reinforcement of rectangular profiles of widths of 3.5 to 6 m. Routine inspection is carried out in the Lorraine Coalfield. Bed separation measurement are made at 15 m intervals, once or twice a week, and data from hundreds of measuring stations are fed into a computerized handling system. Where bolts are used as the sole means of support, guidelines are recommended:

(a) bolt length should be 1/3 of roadway width, but not less than 1.8 m,

(b) a 20 mm diameter bolt rod with a 15% elongation is preferred,

(c) systematic bolting patterns with bolts normal to the strata, but with side bolts inclined 15-20° into the solid, are recommended,

(d) bolt density should be 1 per 1.5 m^2,

(e) wire mesh should be used in conjunction with roadway bolting,

(f) side bolting is recommended wherever there is the likelihood of significant side spalling.

Similar rules are established for German mines where 203,000 resin and 63,000 mechanical bolts were used in 1974. (See the section on the foreign trip under Task II, the Field Survey.)

2.8 Laboratory Beam Models and Mine Models

A number of investigators have constructed beam models and models of mine openings in order to test out theories of roof bolting. Panek has done extensive laboratory work on beam models to test the effect of mechanical bolting. In [2.81] Panek described the theory of model similitude and used dimensional analysis to derive the equations assuring the same strains in the beam model as in the actual bolted material. In [2.82] the development of a centrifugal testing apparatus to simulate the effect of dead weight (gravity) loading is described. Panek [2.83] conducted experiments on laminated beams, 12 inches long, made of Indiana limestone and hydrostone with bolts 1/16 to 5/64 inch in diameter. In addition to tensioned mechanical bolts, untensioned closely-fitted bolts (threaded into the holes) were tested. In the latter case, clamped beams of 6 and 12 laminae were tested and little reinforcement effect was found when comparing outer fiber bending strains on the bottom lamina. Therefore, Panek concluded that a multimember roof cannot be reinforced by using untensioned bolts even if they are held in their holes by cement or friction along their length.

In [2.57] Panek describes experimental results that confirm his theory on the effect of tensioned mechanical bolts in providing a suspension effect in a bedded deposit. In [2.58] experimental results were found to verify the theory for combined suspension and friction effects.

Fairhurst and Singh [2.49] have also conducted beam experiments to check on the reinforcement effect of grouted bolts. Bolted beams with four layers made of both plexiglas and masonite were tested. The size of each layer was 16 inches x 3 inches x 1/4 inch. Bolt holes were 3/16 inch in diameter and 1/8 inch diameter steel was used for the bolts. The beams were simply supported and loaded at midspan. Deflections at midspan were used as a basis of evaluation. The resin grouted bolt was found to be superior to the point anchored bolt. This result is contrary to that of Panek [2.83] mentioned above. There may be a number of reasons for this apparent contradiction:

(1) Relatively larger bolts were used by Fairhurst and Singh. This would give a greater reinforcement effect.

(2) A fewer number of layers were used by Fairhurst and Singh. The reinforcement effect may be influenced by this.

(3) Panek's tests were limited to small strains. Fairhurst and Singh carried their tests out to larger deflections. A sufficient amount of interlaminar slip may need to be activated before the shear resistance of resin grouted bolts is realized.

The different conclusions reached in [2.83] and [2.49] point out the need for more research on this problem before the mechanisms can be fully understood.

Heuze and Goodman [2.44] reported on beam tests with mechanical bolting and tensioned-cable roof-truss reinforcement in sandstone beams. No significant results were found, except perhaps that the bolts and cables compensated for the weakening effect of machined bolts.

Gambrell and Haynes [2.84] report on a photoelastic model study of tensioned mechanical roof bolts angled 45° over the pillars and connected by a metal plate to simulate a roof truss. However, the roof bolts did not produce the favorable compressive prestress at midspan that was produced by the roof truss.

Lang [2.46] gives photoelastic results for stresses in bolted jointed rock where tensioning is required to hold a fragmented mass together. Also, full scale tests of mechanically bolted masses of crushed rock were reported. It was found that the form of the load deformation curve for bolted crushed rock is similar to that of a ductile metal. Loading and reloading loosens the bolts, however, and retightening was required. Alexander and Hosking [2.48] also report on the mechanical bolting of loose jointed rock. They found that a beam could be formed by bolting. Tension on the bolts in the vertical direction tended to cause the jointed rock mass to expand horizontally. Lateral restraints against expansion produced a horizontal compression in the rock mass to hold it together.

Cecil [2.85] studied the behavior of jointed rock models made up of small rectangular blocks and investigated the effects of lateral stress, and horizontal, vertical and dipping joints (modeled with Teflon seams) on block fallout. The effect of bolting was not included.

Wuerker [2.86] tested roof bolts in solid concrete beams. Tensioned mechanical bolts and wedged wooden bolts were used. The only important results that may pertain to testing of resin bolts in concrete beams are these: The experimenter can expect a lot of scatter in results for a brittle model material like concrete. Many of the model beams cracked at bolt holes and the author [2.86] expects that a cemented bolt (or resin today) may prevent this failure by filling the hole.

Stimpson [2.87] has published a comprehensive paper on modeling materials for rock mechanics. He gives a list of mechanical properties of these materials. Twenty-two references to the application of "dry plaster," twelve with plaster plus an additive, and nine with plaster of Paris plus an additive are cited. For the dry plaster the values of Poisson's ratio range from .06 to .304, the modulus of elasticity from 3.7×10^5 to 2.6×10^6 psi, and the uniaxial compressive strengths from 360 to 1100 psi. The ratio of compressive to tensile strength of most dry plasters ranges from 2 to 8, so that the majority of rock types are not modeled accurately. However, lime added to plaster of Paris is reported to give a better ratio of these strengths. If one of these model materials is selected, it is recommended that the experimenter conduct mechanical tests to determine the specific properties of his own mix of the model material since properties have been found to vary appreciably.

Wright experimented with weak mortar consisting of coarse sand and a fairly high water cement ratio [2.96]. Compressive strengths of 1500 psi down to 200 psi were found for water/cement ratios of 1.0 to 2.5. Tensile strengths ranged from 290 down to 30 psi. A modulus of 0.6×10^6 psi was found for sand/cement of 17/1 and water/cement of 2.4/1.

Lang [2.46] has used the experimental photoelastic modeling technique to study the stress state under a stream valley. Figure 2.49 shows the stress concentration that occurs under gravity loading at the base of a valley in a horizontal plateau.

Wang, Boshkov, and Wane have conducted photostress model studies of laminated limestone beams under centrifugal loading, [2.109, 2.110]. They concluded that the use of non-tensioned resin grouted dowels at the center of the roof span may be destructive to the stability of the immediate roof layer because the dowels introduce a stress concentration in a tensile stress area. Comparisons between unbolted and bolted beams do not lead to any valid conclusions because of the strengthening effect of the photostress plastic coating.

Research at the Safety Research Coal Mine of U.S. Bureau of Mines at Bruceton, Pa. [2.23] has been undertaken on a roof pulling technique to evaluate bolted roofs. The testing has been checked out, but more tests are needed to determine if the method can distinguish the effectiveness of different bolting plans.

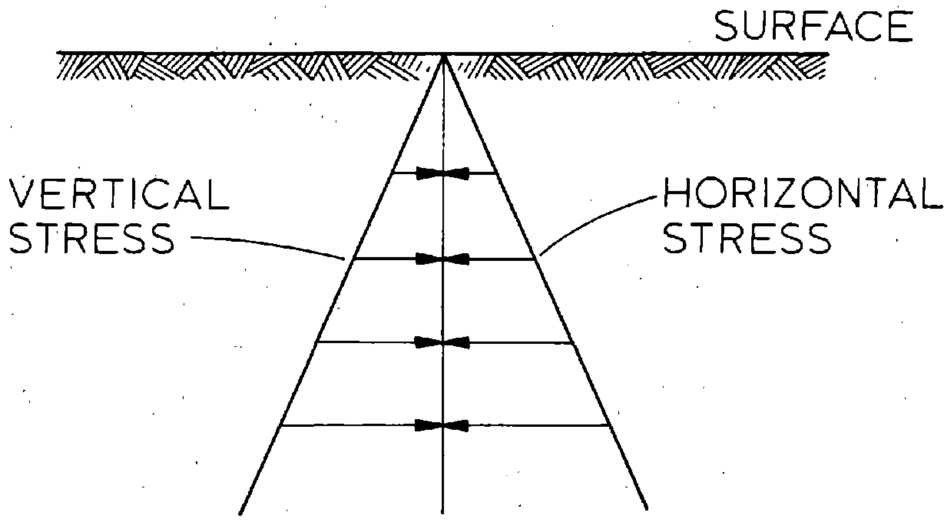
2.9 Use of Wooden Dowels in Resin Bolting

Wooden dowels have been used in place of steel rebar in resin bolting installations, particularly in Europe, because the wooden dowel is more easily cut or sheared at a long-wall face or in the floor in case of floor heave. Use of wood is not out of question in American coal mines either, not just for long-wall mining, but for another reason as well. In soft coal roofs, the roof rock usually is weaker than the resin and the steel, and failure occurs in the rock. The shear stress on the rock at the resin-rock interface can be reduced, if a larger hole size is used to distribute the load over a greater surface area of hole in the rock. In such a case, wood of a larger diameter can be used, to save resin costs and to give the same strength as the steel rebar of a smaller diameter, as shown in Figure 2.50 from reference [2.12].

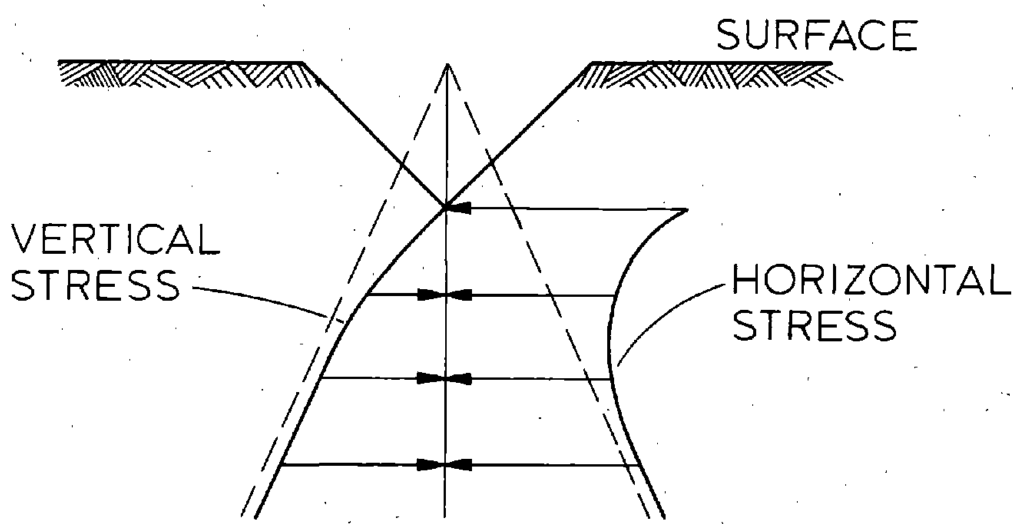
Carr [1.1] reports on 22 pull tests on resin-bonded wood dowels at the National Coal Board Mining Research Development Establishment in the United Kingdom. The results showed that adequate pull loads can be developed.

Strength values for some American woods are given in [2.88].

Celtite (Selfix) Ltd. in England has come out with a composite wood/steel resin bonded bolt [2.98]. The steel core is a 19 mm diameter smooth steel bolt. The hardwood outer sleeve has a 36 mm outside diameter. The bolt is intended for resin grouting in 43 mm diameter holes. The wood evidently is used primarily to fill the big holes. This composite bolt is referred to as a "yielding" bolt or a "debondable" bolt. As the bolt is loaded there is a



(a) Stress Distributions without Notch.



(b) Stress Distributions with Notch.

Figure 2.49 Effect of V-Notch in Gravity Stress Field [2.46]

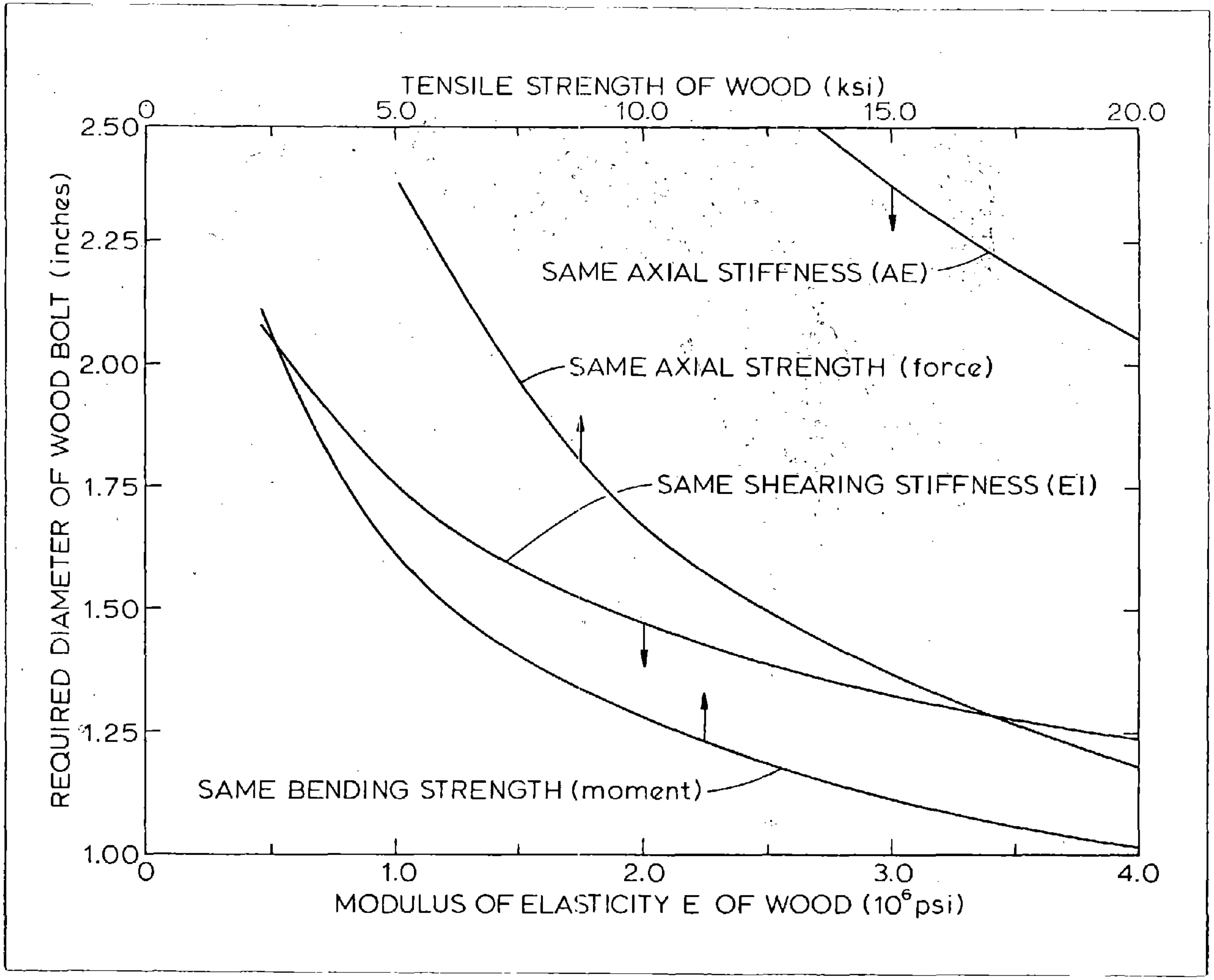


Figure 2.50 Required Size of Wood Bolt to Give Same Strength and Stiffness as a 0.75-Inch Steel Bolt with an alternate tensile strength.
σ_u = 50 ksi, E = 30(10^6) psi, [2.12]

progressive debonding of the steel core from the surrounding hardwood dowel with considerable elongation before the ultimate load is reached. In addition to wood, a polyethylene sleeve is also used on another composite bolt design. The "debondable" bolt is a concept that can be used without wood sleeves as discussed below.

2.10 The Debondable Resin Bolt

The debondable bolt concept introduced by Selfix [2.98] has been applied in a simple way that can be used without too much modification in U. S. A. coal mines. The "Style 4" bolt of Selfix uses a smooth bar with threads on each end. The threaded end at the top produces an anchor there. At low loads the bolt behaves like a fully resin grouted rebar, but as the smooth bolt is loaded further it progressively debonds from the smooth unthreaded portion and thereby elongates considerably before reaching ultimate failure load, Figure 2.51. This bolt can also be used with a welded washer near the top end to create a wedging action at the anchor.

As shown in Figure 2.51 the debondable bolt reaches higher load values. (This should be contrasted with the smooth bolt in Figure 2.3 without the threaded top portion.)

The load transfer mechanism in the debondable bolt is shown in Figure 2.52. The resin/rock interface is loaded from the top. The shear stress is higher at the top and decays toward the bottom just the opposite of the grouted rebar, (Figures 2.5 and 2.6). As shown in Figure 2.52, as the resin/rock bond breaks down the remaining resin is loaded partially in compression. This causes a lateral Poisson expansion and a lateral bearing stress, thereby tending to increase the resistance at the rock/resin interface. In contrast the rebar installation creates tension in the resin causing it to pull away laterally from the hole. The compression feature is advantageous also because the resin is stronger in compression than in tension. It is reported that debondable bolts have been pulled to the breaking point of the steel whereas the standard rebar installations have been pulled intact out of the holes in soft rock.

The debondable bolt concept appears to have been first reported in the literature in 1972, [2.115].

2.11 Installation Procedure

The recommended installation procedure for fully grouted resin bolts is now generally common among the resin manufacturers, particularly because of the work of MESA. The procedure is as follows:

1. Use a drill steel 1-inch longer than the bolt length.

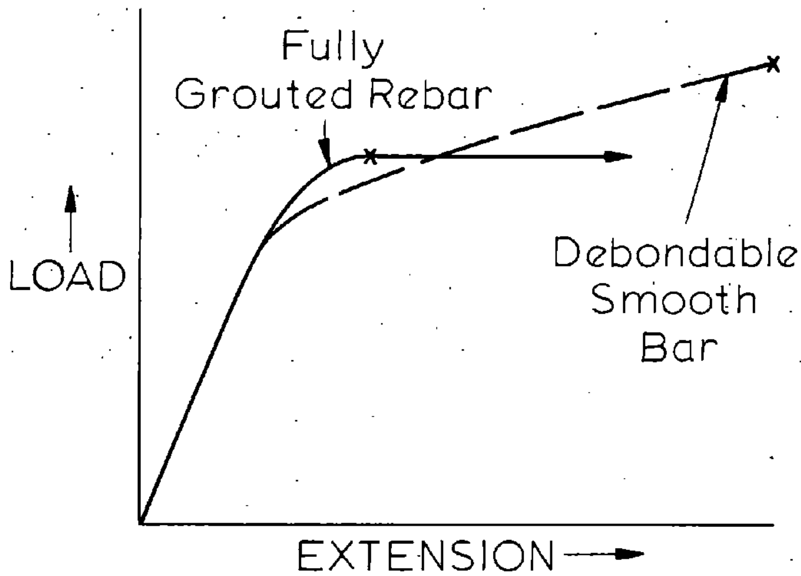


Figure 2.51 Load-Extension Behavior of The Debondable Resin Bolt

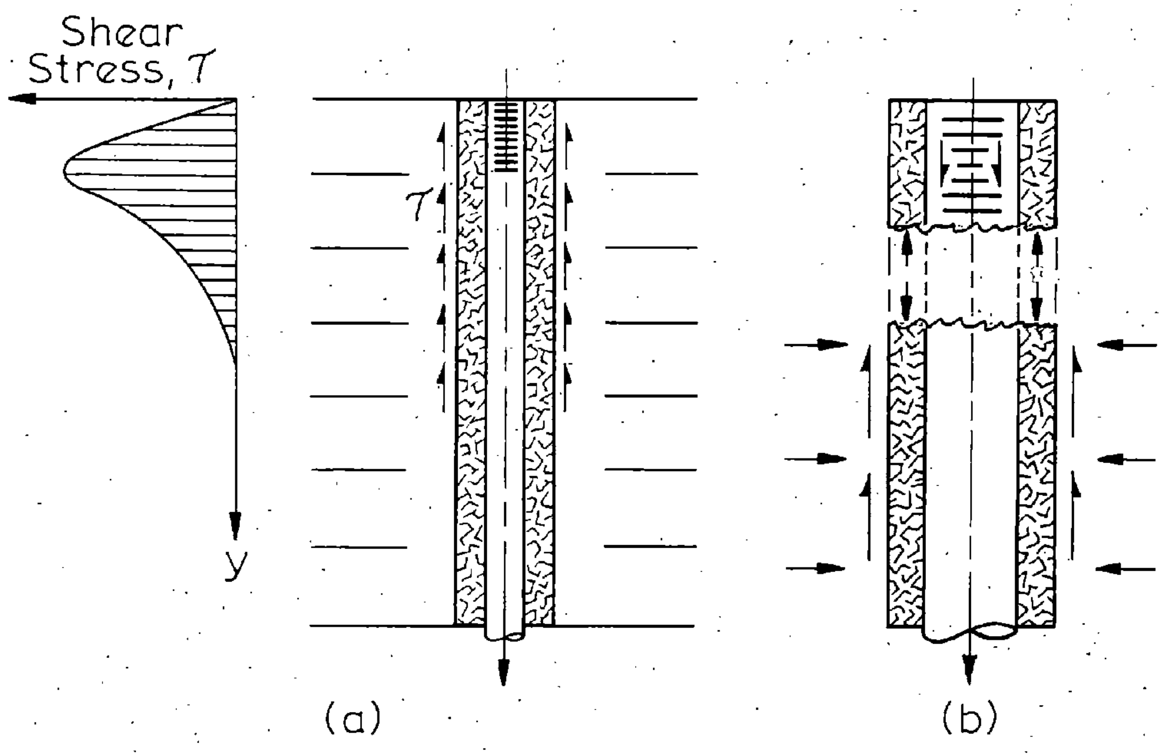


Figure 2.52 Load Transfer in The Debondable Resin Bolt

2. Drill bolt hole to correct depth, shorter than bolt length depending on whether header boards or thick plates are used.

3. Clean dust and dirt out of hole.

4. Insert the required number of resin cartridges into the hole.

5. Push the bolt into the hole to a point within 1-inch of the roof. A slow rotation during insertion is optional to assure better rupture of resin package.

6. With the bolt within 1-inch of the roof, stop the upward movement, and spin the bolt at 200-450 rpm for 15-20 seconds.

7. Stop rotation and push bolt upward with maximum thrust from the machine until the resin gels, depending on gel time for the particular resin.

8. If the bolt moves downward after drill chuck is removed, push the bolt back up and hold until resin gels.

9. Excess resin should run out of hole if the bolt hole has been properly filled. If not check for incorrect drill size, incorrect depth of hole, or cracks in roof.

Hole sizes are recommended to be 1/4-inch to 3/8-inch greater in diameter than the bolt steel for maximum strength. An annulus greater than this may result in poor mixing.

Caution is advised in Step 5 to avoid excessive rotation resulting in whipping of the bolt and possible injury to the operator.

One manufacturer states, under Step 6, to rotate the bolt at least 20 seconds. Beware of the word "least." Too great a spin time is not advisable. Another warns: never re-rotate the bolt after final fast spin because damage to partially gelled resin may occur. This warning should be heeded. The bolt should not be spun too long, otherwise the resin may have gelled and then be ground to powder. There may be the possibility of applying excessive thrust in Step 7, thereby damaging the roof.

2.12 Installation Equipment

Jamison in a recent paper discusses the equipment available and the development work in the area of bolting, [2.106]. Improvements have been made in temporary roof supports mounted on the bolters, hands-off drilling, centralizers to align the drills, and remote control miner-bolters. Some of the latter use magazines or turntables holding five or seven bolts. One machine has been developed for automatic insertion of bendable bolts longer than the seam height. Work is underway on drilling longer holes and smaller holes, 1-inch or smaller. What is still needed is the development of better equipment to drill angled holes over the ribs.

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3.0 MINE ANALYSES

An analysis of resin bolting by itself would be insufficient without considering as well the roof bolting plan for a specific mine and an analysis of the variety of stress conditions, other in situ conditions and rock types that will definitely have an influence on the support capability of resin bolting. Accordingly, analyses of mine effects are considered next in this literature review.

3.1 In Situ Conditions

Hooker [3.1] indicates that in situ stresses may be composed of gravitational, thermal and tectonic stresses. The gravitational vertical stress is directly related to the depth of overburden, and the horizontal is assumed to be smaller by a Poisson factor as follows:

$$\sigma_z = \rho z = \text{the vertical stress} \qquad (3.1)$$

$$\text{and} \quad \sigma_x = \sigma_y = \frac{\nu}{1-\nu}\, \sigma_z = \text{the horizontal stresses} \qquad (3.2)$$

where ρ is the density of rock, z is the depth, ν is Poisson's ratio and lateral expansion is prevented. In one coal mine, Lu [3.2] reports in situ stress measurements of 1600-1800 psi vertical stress in the ribs and 600-900 psi horizontal stresses in the roof and floor. However, high topographical relief can affect principal stress magnitudes and directions [3.1, 3.3, 3.4] and excess horizontal stresses do exist above the Poisson effect from overburden. Magnitudes and orientation of stress components seem to correlate with geological features such as streams, valleys and mountains [3.1, 3.3, 3.4]. High stresses can also be expected in areas near major faults [3.5]. Hooker [3.1] gives a tectonic map of the United States showing plots of the principal horizontal stress ellipses and indicates that the horizontal components (compressive) generally differ in magnitude by a factor of 2 to 1. The magnitude of the horizontal stress may be as much as 1500 psi for near surface mines in the Appalachian-Piedmont area [3.1], as much as 7500 psi in a 2300 foot deep mine in Ohio [3.1], and as much as 15,000 psi in a 5500 foot deep mine in Idaho [3.5].

Thermal stresses can also exist due to fluctuations in daily and seasonal temperatures [3.1]. However, their effects are confined to a few feet near the air-rock interface. Thermal stresses are also caused by geothermal gradients in deep mines in the west [3.4].

Ground water can also be a cause of pressure in mine rock [3.6]. This water pressure can open cracks in the rock if the in situ stresses are not sufficient.

Brady [3.7] also points out the importance of in situ stresses and the need to measure them in order to effectively design a mine opening. Obert [3.8] shows that the measured stress distribution near an opening can be quite different than the theoretical prediction because of stress relief in the fractured rock around the opening. The result of stress relief near the opening is to shift the point of high stress further into the rock. In situ stresses have been measured with bore hole deformation gages using an overcoring technique [3.9], flat jack procedures, and more recently, by hydraulic fracturing [3.10]. Bore hole deformation gages and pressure cells have also been used to determine in situ values of elastic moduli [3.11, 3.12, 3.13, 3.14, 3.4]. The borehole jack has also been used [3.5].

The geology of the roof section is another important in situ condition that must be accounted for in resin roof bolting. Although their emphasis was on roof noise as a roof warning signal, Prout et al [3.15] give the roof sections of 18 mines covering five coal seams. These data are summarized in Table 3.1.

As evident in Table 3.1, the roof section can vary within a mine, and from mine to mine within a coal seam. The overburden or cover varies from 100 to 2200 feet in these mines in the Appalachian region. A few mines may have a coal rider a few feet up (Mine Q) which can cause problems for bolting. (The coal rider problem is also mentioned in [2.25] in the Taggert seam.) Some mines have a competent sandstone bed within reach of bolts, some do not. (Mines in the Illinois seam tend to have limestone in the roof whereas the others do not. The immediate roof in Mine J was eight inches of top coal.) Not shown in Figure 3.1 is the geology of the floor which was generally not reported. However, a fireclay floor was reported for Mine D and a shale floor for Mine B both from the Pocahontas #3 seam.

Some information on the roof bolting plans was given in [3.15], as shown in Table 3.2, but no distinction was made between resin and mechanical bolting in the report. Bolt spacing was generally 48 inches except for Mines M and Q. In most cases, posts were used only as additional support near the face, in retreat sections and in critical areas. Where cross bars were used, posts were set under each end of the crossbars. It would be interesting to lay out the bolt lengths in the roof sections to see if there are any correlations, especially taking into account comments on other roof conditions noted in [3.15].

The roof strata above six mines have been plotted in the form of "fence diagrams" by McCulloch, Jeran, and Sullivan [3.16]. The fence diagrams are determined from data from corehole logs. The mines and their coal beds were as follows:

TABLE 3.1 BED THICKNESSES IN SOME COAL MINES [3.15]
(Reference Figure 3.1)

Mine	Name of Coal Seam	Bed Thickness, ft						
		h_1	h_2	h_3	h_4	h_5	h_6	h_7
A	Lower Kittanning	4.1	0.	2.-3. (S)	0.	0.	0.	780.
B	Pocahontas #3	4.6	1. (D)	0.-1. (S)	0.	0.-2.	0.	95.-343.
C	Brookville "A"	4.8	0.-4. (S)	0.	0.	80.-120.	0.	0.
D	Pocahontas #3	4.7	2. (S)	0.	0.	0.	0.	150.-198.
E	Pocahontas #3	5.-9.	0.	0.	0.	.2-6.	0.	950.-1100.
F	Pocahontas #3	4.4	0.	0.	0.	~1.	0.	2000.-2200.
G	Pocahontas #3	5.5	0.	0.	0.	.5	3.7	1500.
H	Illinois #6	7.5-8.	0.	0.	0.	0.	0.	600.-650. (L)
I	Illinois #6	6.3-6.5	100.-120. (S)	4.-11. (L)	0.	0.	0.	550.-650. (L)
J	Illinois #6	6.7	0.	0.	.7	0.	2.-3. (S)	640. (L)
K	Illinois #6	6.	5. (S)	~2. (L)	0.	0.	0.	430.
L	Illinois #6	7.-8.5	3.-5. (S)	0.	0.	0.	0.	650.-745. (L)
M	Elkhorn #3 (Cedar Grove)	3.8-4.2	30. (S)	0.	2.	168.-0.	0.	0.-168.
N	Elkhorn #2 (Lower Cedar Grove)	4.	0.	6.-8. (S)	0.	25.	0.	267.
N	Elkhorn #2 (Lower Cedar Grove)	4.2	4.	0.	0.	~50.	0.	200.
O	Taggart (Cedar Grove)	4.2-4.5	0.	0.	0.	?	0.	470.-950.
P	Taggart (Cedar Grove)	4.7	0.	0.-3. (S)	0.	10.	0.	480.
Q	#2 Gas	6.6-8.	0.	1.-3. (S)	~2.	?	475.-695.	?
R	#2 Gas (Campbell Creek)	6.6	0.	0.-4. (S)	0.	20.	40.	590. (L)

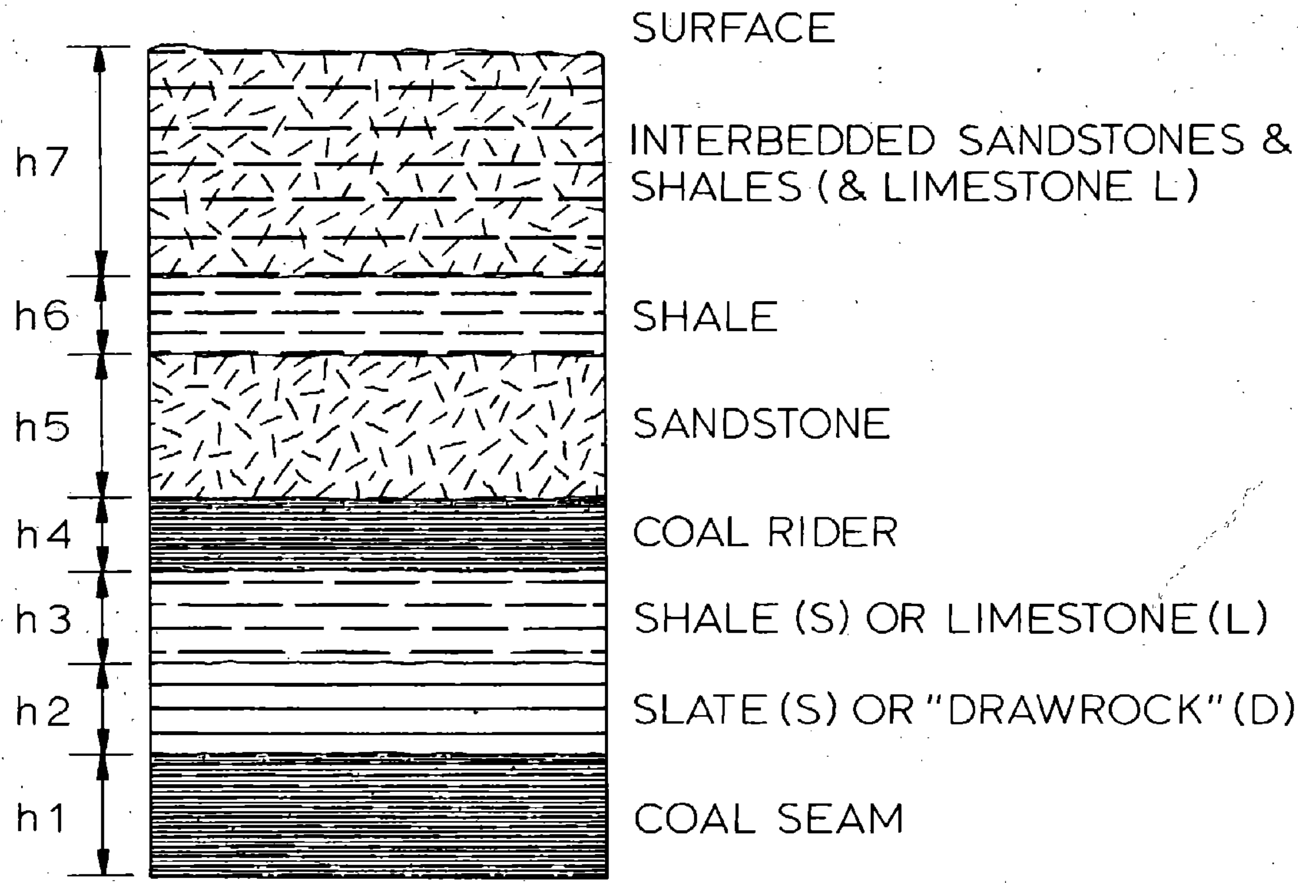


Figure 3.1 Roof Section (Geological Column) of Some Coal Mines [3.15]

Table 3.2 Roof Bolting Plans in Some Coal Mines [3.15]

Mine	Bolt Length (Inches)	Bolt Spacing (Inches)	Other Supports
A	72	48	6"x8" Crossbars, 6"x8" posts
B	42	48	4"x6" Crossbars, 6" posts
C	48	48	6" posts
D	84	48	2"x8"x18" Header boards, 6" posts
E	48	48	2"x8"x24" Header boards, 6" posts
F	48,60	48	6" posts
G	48	48	6" posts
H	60,66	48	Header boards, 6" posts
I	96	48	6" posts
J	60	60	6" posts
K	60,108	48	
L	60,84	48	
M	42	66	
	36,72	48	4"x4" posts
N	48,60	48	6" posts
O	42	48	6" posts
P	54	48	
Q	60,84,96	36	6"x8" Crossbars, 6" dia. posts
R	72	48	

Marianna No. 58 Mine	Pittsburgh coalbed
Lucerne No. 6 Mine	Upper Freeport coalbed
Somerset No. 60 Mine	Pittsburgh coalbed
Federal No. 2 Mine	Pittsburgh coalbed
Howe Mine	Lower Hartshorne coalbed
Beatrice-Pocahontas Mine	Pocahontas #3

For example, Figure 3.2 shows a fence diagram for the first mine mentioned above. As shown, the roof section varies appreciably. Roof problems are associated with the "wild" coal interval consisting of thin bands of claystone and coal varying from 1 to 9 feet in thickness. Isopach maps of coal beds, individual rock strata and overburden are also presented in [3.16]. For example, Figure 3.3 shows an overburden isopach map for the Beatrice-Pocahontas mine which shows sharp gradients in the overburden. They [3.16] also point out the importance of knowing coal coal cleat, rock joint and fault orientations, and the locations of clay veins, because they greatly affect roof instability.

Coal cleat orientations are plotted in [3.17] for a Pennsylvania coal mine. Geological maps of roof sections and isopach maps are also given by Moebs [3.3].

Floor swelling and softening, and rib expansion due to high humidity or presence of water in the mine can cause problems with roof control as pointed out by Morgan [3.18]. These effects can cause rotation of the ends of the roof beam, and can cause horizontal stress changes to shift to the roof. Moisture also can cause weathering and weakening of soft roof rock. Variations in humidity can work in two ways. Oitto, Zona and Stears [3.19] reported that a shale roof gave up moisture and shrunk when the relative humidity was low and absorbed moisture and swelled when the humidity was high. Some mechanical bolts lost as much as 2000 lbs tension when the relative humidity decreased and gained 3000 lbs tension when it increased. This working or cyclic stress change in the roof can lead to failure.

Haynes [3.20] conducted laboratory experiments on humidity effects and found that seasonal changes can induce strains of 6000 microinches/inch in coal mine roof rock, which may be sufficient to cause spalling. Significantly less swelling strain was found underground, perhaps because of confinement.

Einstein and Bischoff [3.21] also discuss swelling problems in tunnels and indicate that swelling in shales is primarily due to the presence of clay minerals. They also describe a laboratory procedure for determining swell pressure-displacement curves and swell pressure-time curves.

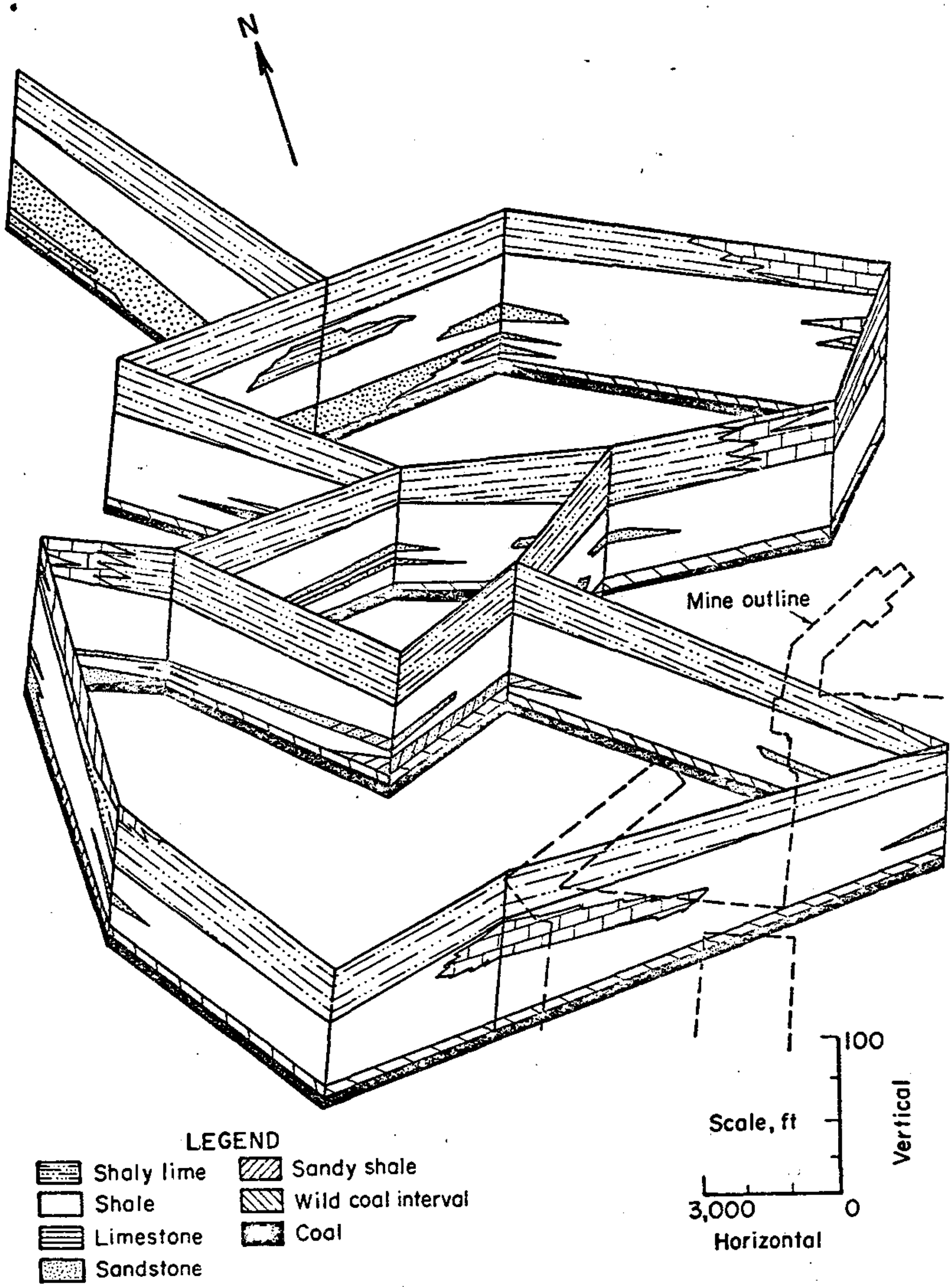


Figure 3.2 Fence Diagram of Strata above the Pittsburgh Coalbed, Marianna No. 58 Mine. [3.16]

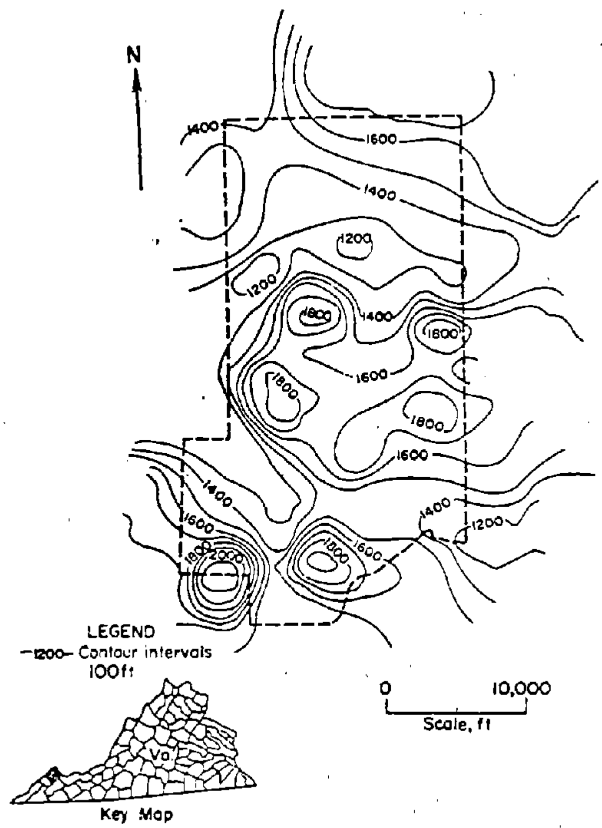


Figure 3.3 Overburden Isopach Map of Interval From Pocahontas No. 3 Coalbed to Surface, Beatrice-Pocahontas Mine [3.16]

Adam [3.22] found clay mineral content in a mine shale, and found that the shale lost strength with a moisture increase. The shale had a greater affinity for water parallel with the bedding planes than it did perpendicular. Roof falls were identified where moisture had caused deterioration of the roof rock.

Martna of Sweden [3.23] points out that the mineral pyrrhotite (an iron sulphide) weathers easily and causes expansion of rocks. The weathering of pyrite (another iron sulphide: is much accelerated when in contact with pyrrhotite.

Palowitch and Zarhar [3.68] give the geology of the roof section in three coal mines, Figures 3.4 - 3.6. In the Federal No. 1 mine (Figure 3.4), the roof caved well in shortwall operations in pillar recovery. Gas and oil wells penetrate this coal bed occasionally and extremely heavy sulfur balls inclusions are found in the bed.

The Hendrix No. 22 Mine (Figure 3.5), had a good roof requiring only 3-foot bolts on 5 foot centers. The Gary No. 14 Mine (Figure 3.6), had a poor roof which was highly friable because of the 20 feet of thin alternating laminations of coals and shales.

Pratt and Levinson [3.69] conducted a study of roof rocks in bituminous coal mines in Utah, Pennsylvania, West Virginia and Ohio. Data were collected from 640 measured sections, and the roof sections are presented in several figures in [3.69]. As a summary, the percentages of rock types in the roofs are given in Table 3.3. It is found that shale, sandy shale, and sandstone predominate in the roofs in both the eastern and western coal areas.

McCulloch, Diamond and Bench [3.70] present data on sandstone channels, clay veins, cleat directions and roof sections of the Pittsburgh coal bed in southwestern Pennsylvania and northern West Virginia. The sandstone channels extend into and sometimes completely through the coalbed. When the coalbed is completely cut out, the sandstone may be 40 to 60 foot thick.

The clay veins are another problem [3.70]. They can penetrate the coalbed from either above or below, and may be vertical or form an angle of 45° to the vertical as shown in Figure 3.7. The clay veins are crooked and irregular as shown, range from 1 inch to several feet in thickness, and may have a total length up to 1000 feet.

Cleat orientations in the Pittsburgh coalbed have also been mapped. In 18 mines surveyed the face cleat trends range from N62°W to N80°W, and the butt cleat trends from N12°E to N30°E. (Cleats compose the natural vertical fracture system in bituminous coal beds. The more dominant is the face cleat, and the minor fracture plane is the butt cleat approximately at 90° to the face cleat.)

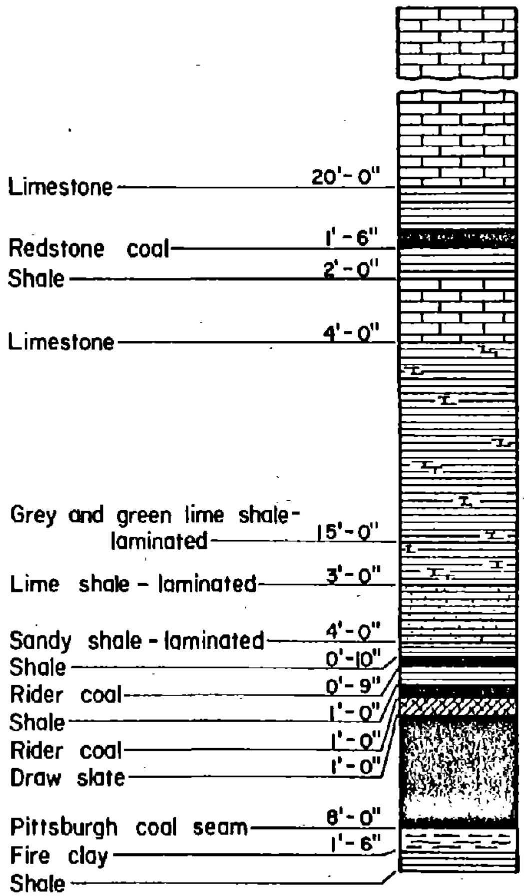


Figure 3.4 Strata Overlying the Pittsburgh Coal Seam, Federal No. 1 Mine [3.68] (800 to 1000 ft. cover)

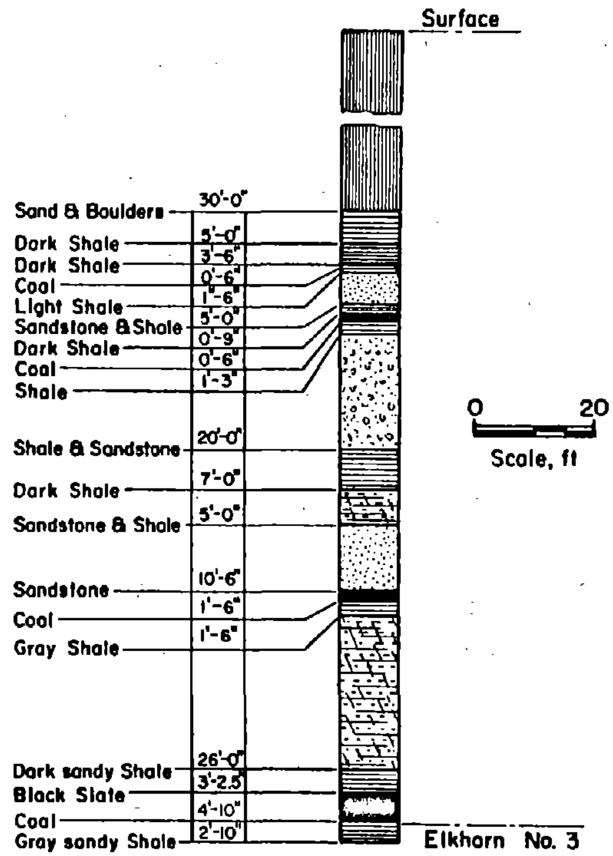


Figure 3.5 Strata Overlying the Elkhorn No. 3 Coalbed, Hendrix No. 22 Mine [3.68] (150 to 800 ft. cover)

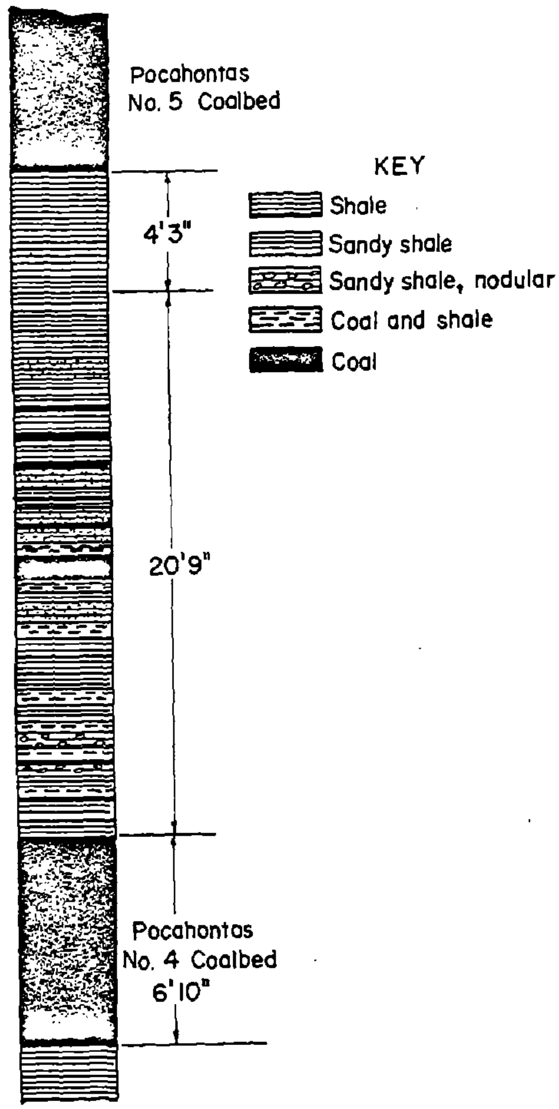


Figure 3.6 Strata Overlying the Pocahontas No. 4 Coalbed, Gary No. 14 Mine [3.68] (200-450 ft. cover)

TABLE 3.3 PERCENTAGES OF ROCK TYPES IN COAL MINE ROOFS [3.69]

	Percentage	
Rock Type	Utah	Pennsylvania, West Virginia, Ohio
Shale	48.0	54.0
Sandy Shale	29.0	18.0
Sandstone	22.0	23.0
Conglomerate	.5	--
Bone	.5	--
Limestone	--	5.0

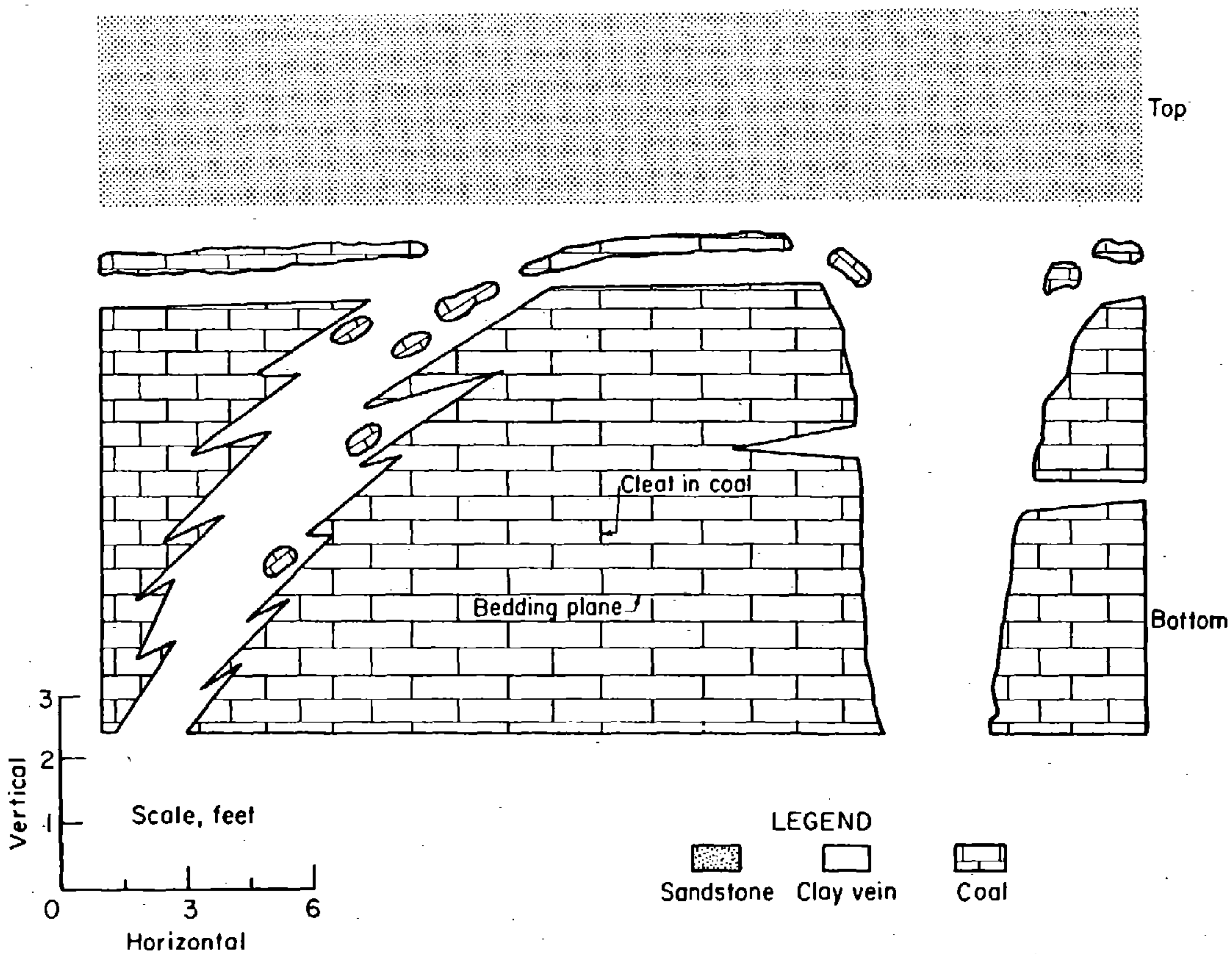


Figure 3.7 Generalized Cross Section of Clay Veins Intersecting Coalbeds [3.70]

It is also found that the roof section can rapidly change above the Pittsburgh coalbed [3.70], as shown in Figure 3.8. This abrupt lithologic change often coincides with unstable roof areas.

3.2 Blasting and Vibration Effects

Blasting, machine vibration, and vibrations from earthquakes and rock bursts can damage the roof and loosen the roof bolt anchorage. Habenicht and Scott measured the exponential decay of strain amplitudes from blasting in an experimental mine [3.24]. They found that the decay distance was approximately 15 feet. Beyond this distance strain losses in the bolts were below 1% per blast. Within this distance, approaching the blast hole, strain losses exponentially increase.

Stehlik [3.25] measured changes in tension in mechanical bolts and roof damage from blasting in the White Pine mine. The greatest damage to bolts occurred in the first row of bolts two feet from the face where the bolts ruptured, bent, or lost all their tension. Beyond 12 to 15 feet from the face, changes in bolt tension were small (±1000 lbs). A zone of rock damage (fracturing) occurred repeatedly at a distance into the roof of about 21 inches.

Snodgrass and Siskind [3.26] report on vibration effects from blasting in four different mines with a wide range of geological conditions. Reviewing research results from previous studies they found that square-root-scaled regression data can be used with fairly good accuracy to predict particle velocities and acceleration amplitudes from subsurface blasting. The square-root relation for acceleration is:

$$aw^b = K\ (D/w^b)^{-m}$$

where a is the acceleration, w is the charge weight/per delay period, and D is the distance from the charge. The parameters b, m and K used in Equation (3.3) are in the range of 1/3 to 1/2, 2.07 to 2.65, and 14,000 to 82,000, respectively. For a 1.0 lb charge, this equation shows that the acceleration decreases from 10 g to 1.0 g in about 40 feet.

Siskind, Steckley and Olson [3.27] measured the radii of damaged or fractured rock around blast holes in shale pillars in the White Pine mine. Damage radii of two to four feet were found.

Ortlepp in South Africa [3.28] found that in a tunnel, blasting in holes (parallel to the tunnel) two to three feet from the anchors of mechanical bolts, completely dislodged the anchors but "yielding" rock bolt anchors (of a new type) stayed intact.

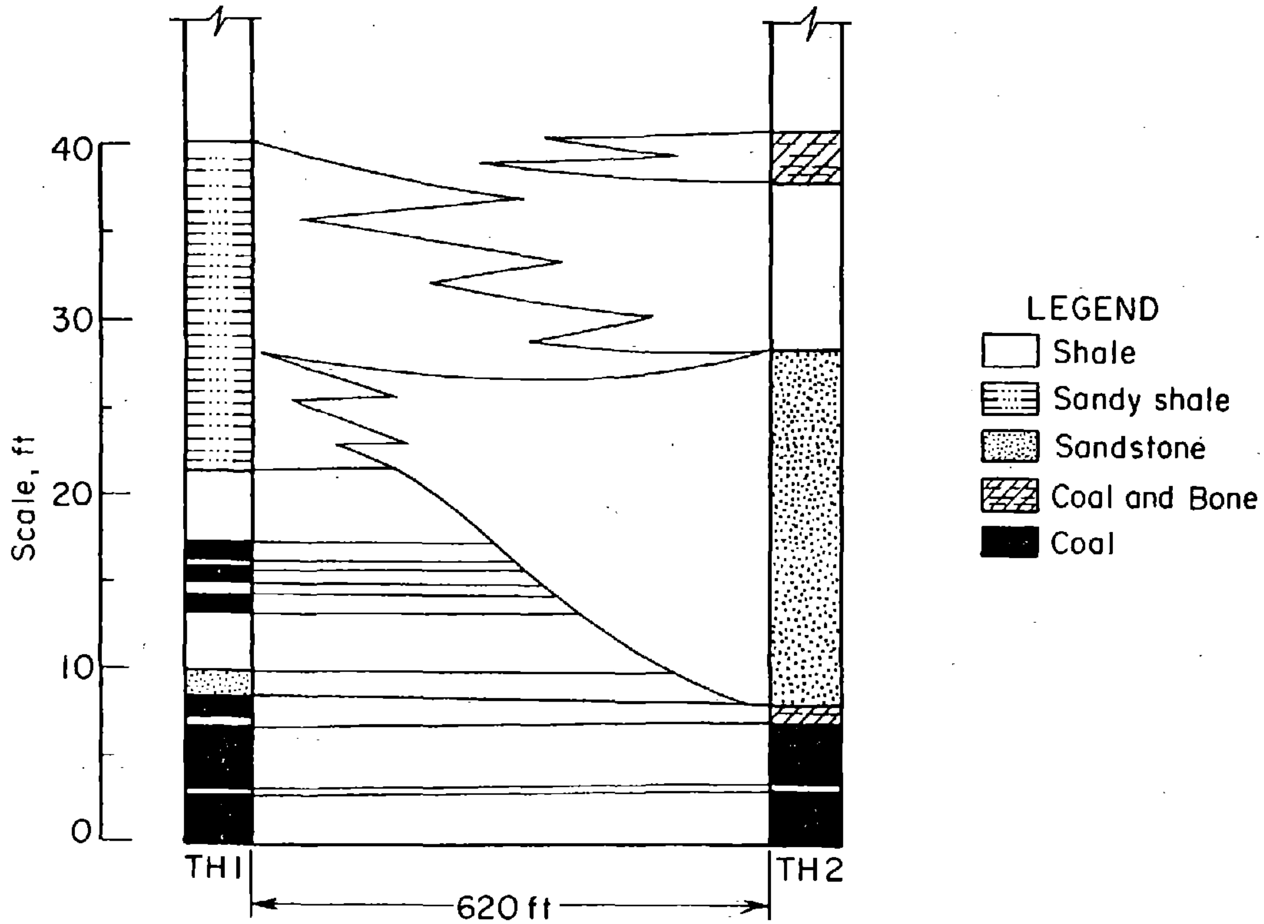


Figure 3.8 Rapid Lateral Variation in the Strata Directly Above the Pittsburgh Coalbed [3.70]

What may be considered as important, from these studies, are possibly two conclusions: blasting effects along the roof are sensed by the bolts up to 15-40 feet away, and damage in the roof can exist up to 2-4 feet into the roof rock. (These conclusions may change quantitatively depending on the rock quality to start with.) Using continuous mining machines with mechanical cutters, instead of blasting, to reduce such damage certainly appears to be advantageous.

During blasting, one might expect greater damping of vibration in a fully resin grouted bolt. Amplitudes of initial vibration can be expected to be less, because the load transfer length is much less (greater stiffness) for the resin bolt than for the mechanical bolt.

Beveridge [2.91] reports on the effects of vibration from blasting on the load relaxation in point anchored resin bolts (ICI resin, 20 mm rebar in 28 mm holes) bonded in a limestone tunnel close to a face being drilled and blasted. Results in Figure 3.9 show only an initial relaxation* of 15-20% during the first 2-3 days when the bolts were 10-12 m from the face, and no change thereafter. (Reference [2.91] indicates 10-12 mm from the faces, but this is obviously a typographical error.) Mechanical bolts are reported to relax up to 80%.

Dunham reports similar findings on bolts installed in a lead mine and in a gypsum mine [2.92]. Only a 10-20% loss in load occurred within two days, and no loss thereafter, for bolts initially 9 m from the face in the lead mine. Figure 3.10 shows the comparison of blasting effects on mechanically anchored and short bond length resin anchored bolts in the gypsum mine.

The importance of pre-reinforcement in preventing blast damage in tunnels is demonstrated by Korbin and Brekke [3.71]. The authors claim that reinforcement generally does not work by "pinning" loosened material in place, but by preventing loosening. Pre-reinforcement, ahead of the face in an advancing tunnel, prevents rock loosening and prevents excess deformation from occurring. Figure 3.11 shows how fully grouted spiles were installed at a 30° angle ahead of the face. It was found that pre-reinforcement far ahead of the advancing face was more effective in restraining deformation than spiling reinforcement at the face.

*At the bolthead/bearing plate.

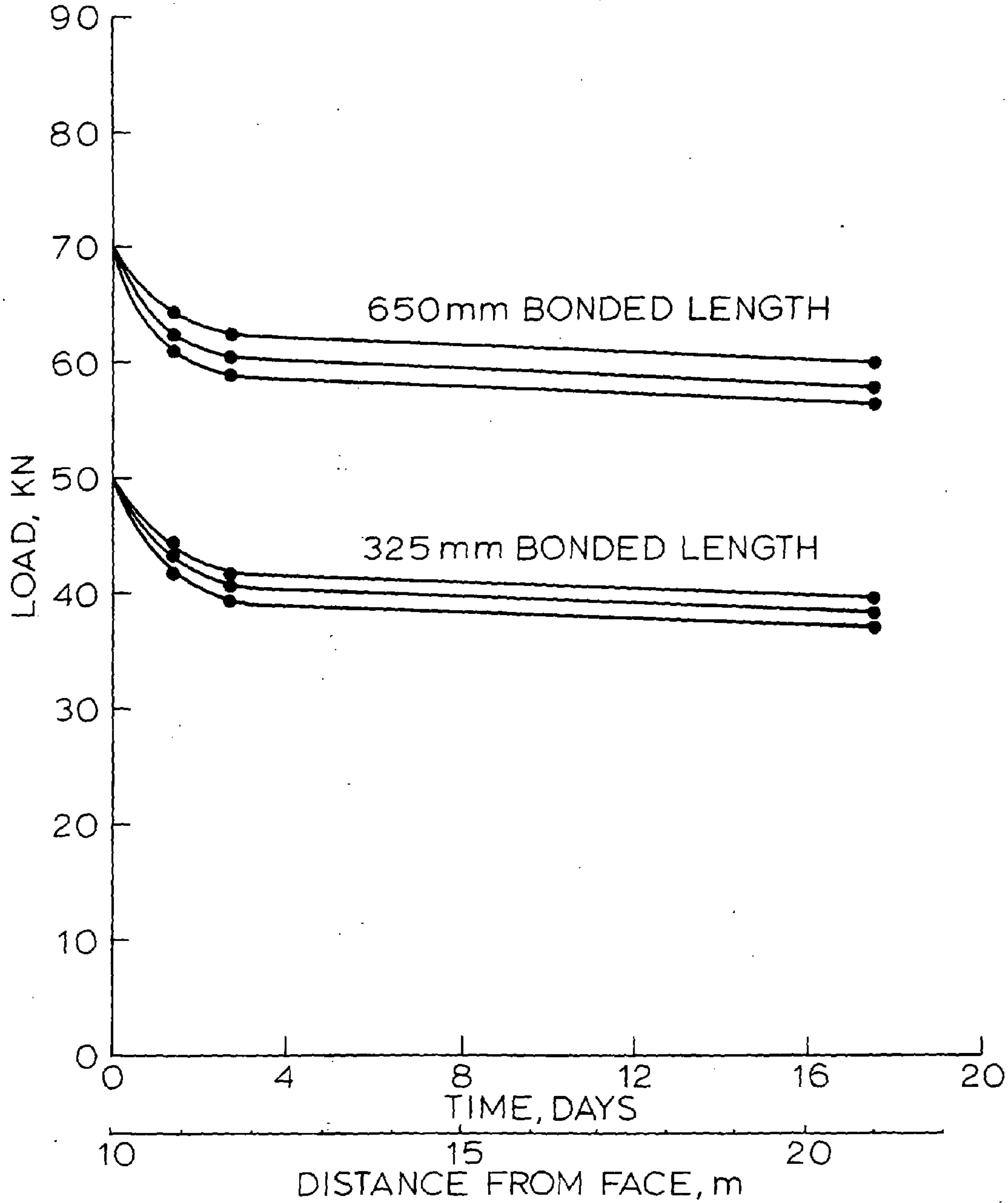


Figure 3.9 Limestone Tunnel. 20 mm Bolts Subjected to Repeated Vibration from Blasting [2.91]

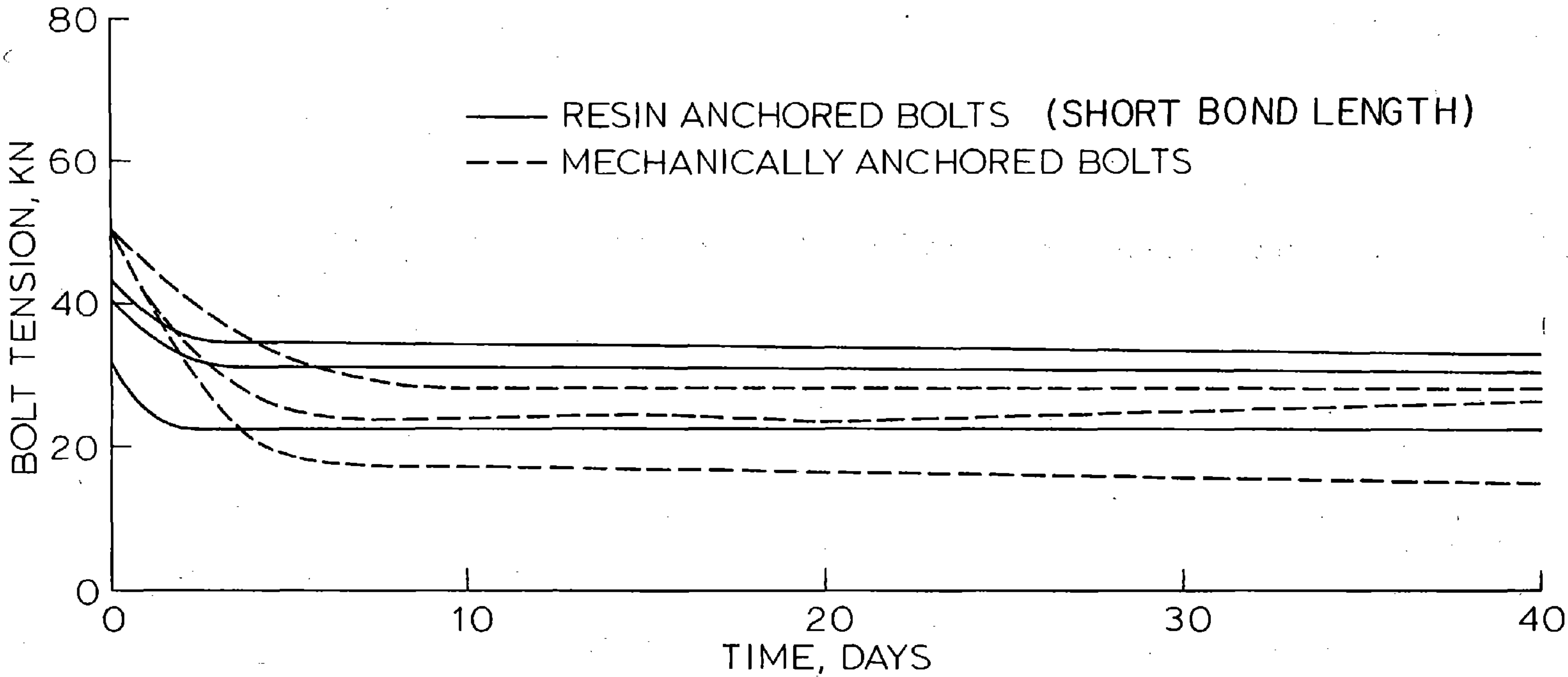


Figure 3.10 Bolt Dynamometer Tests at Long Rigg Gypsum Mine [2.92]
Bonded Length 500 mm.

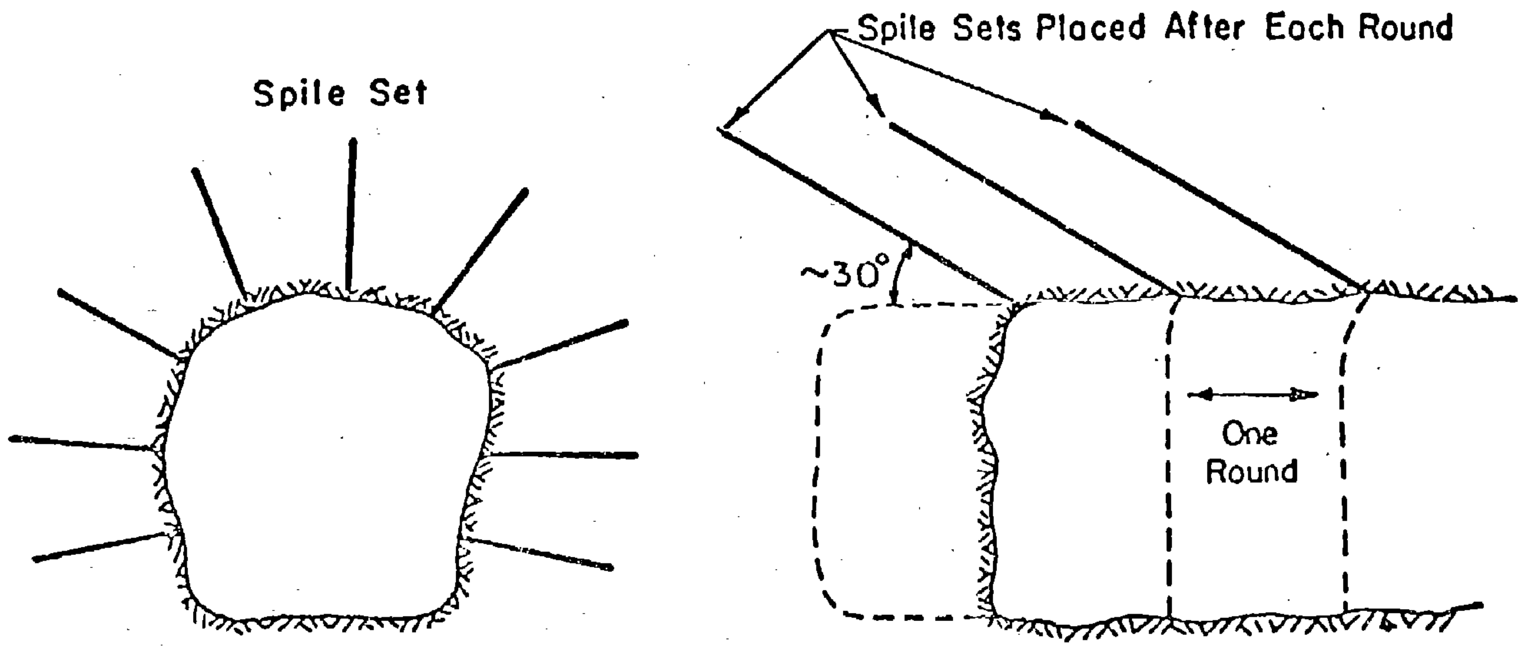


Figure 3.11 Spiling Reinforcement Ahead of the Face (a form of pre-reinforcement) [3.71]

3.3 Roof Failure Observations

In an article by Caudle [3.29], roof failures are attributed to three sources:

(1) tensile strains in the back that result in the opening of cracks

(2) delamination and buckling of the immediate roof

(3) shear failure in the immediate roof at the rib, arching upward to terminate at the bedding planes.

Morgan [3.18] states that the two major types of roof falls are characterized by (1) and (3) above, i.e. tensile or shear, with the tensile type developing slowly with some warning and the shear type more quickly and with little warning. According to Morgan, the conditions causing the shear type of failure are as follows:

"(1) Low shear strength of the immediate roof, often due to joints, slickensides, and other geological features,

(2) Great depth of cover or high abutment loads,

(3) High horizontal stresses,

(4) Wide spans,

(5) Rib and the floor under the rib, being relatively stiff compared to the immediate roof, and

(6) Soft shale occurring above a relatively rigid immediate roof."

He lists the following conditions causing the tensile type of failure:

"(1) Low ratio of horizontal to vertical stress,

(2) Thinly bedded or delaminated roof layers,

(3) Wide spans,

(4) Jointing in the roof and/or coal,

(5) Low modulus of crushing ribs and/or yielding of the floor from under the rib, and

(6) Low modulus roof layers."

Parker [3.30] has illustrated various roof falls and has characterized the contributing conditions. Figure 3.12 shows blast damaged roof rock tied together by roof bolts. Figure 3.13 shows a high lateral stress condition in massive roof rock and in laminated roof rock. In some areas there is not enough lateral stress and friction between blocks to hold them together--and the roof collapses like a pile of rubble. Figure 3.14 shows a condition of insufficient lateral stress.

In some places the beds will not be thick enough, or the span will be too great, or the lateral stress will not be sufficient, or the rocks will soften with time--and a Voussoir arch will sag and buckle through. Figure 3.15 shows this condition. Under certain conditions the vertical loads may be excessive, causing the ribs or pillars to punch into the roof, producing shears which are nearly vertical, as shown in Figure 3.16.

Stateham [3.31] observed a roof fall in a coal mine in New Mexico which appears to be of the type shown in Figure 3.14. Roof movement prior to failure was cyclic in nature, probably related to the mine work cycle (work days to idle days) and "pillar robbing" upon retreat in nearby entries.

In commenting why resin bolts may have worked better in the White Pine mine than mechanical bolts, Maher and Bennett [2.69] believe that a Voussoir arch (Figure 3.15) and zone of compression tend to form as the roof begins to sag, that mechanical bolts act to remove or lessen the favorable compressive rock arch by pulling the blocks back toward their original position, but that untensioned resin bolts pin the roof rock in the stable position.

Moebs [2.7] reports on an examination of a roof fall in a resin bolted area in a coal mine, and found that large slickensides (slips) shown as dashed lines in Figure 3.17 could have contributed to the roof failure. Perhaps stress gradients could relate to the variation in draw slate thickness in the roof and also be a contributing factor. Also the bolts may not have been of a sufficient length to extend beyond the zone of the slickensides.

Failures associated with discontinuities in shale thickness between two competent layers of sandstone have also been found in a tunnel [3.32]. These discontinuities were believed to have caused stress concentrations.

In Britain, the idea of a "pressure arch" or ground arch has been formulated based upon underground observations [3.33]. (A ground arch has been shown earlier in Figure 2.36.) The width of the ground arch that is formed depends upon the mine depth. Generally, it has been found that

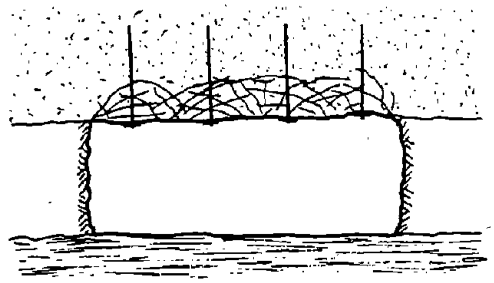
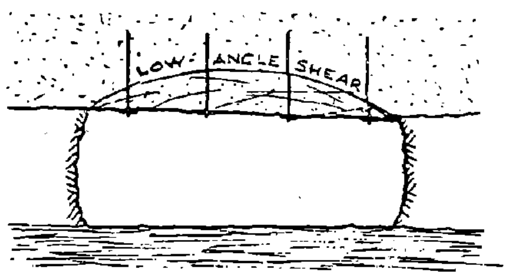


Figure 3.12 Blast Damaged Roof Rock [3.30]



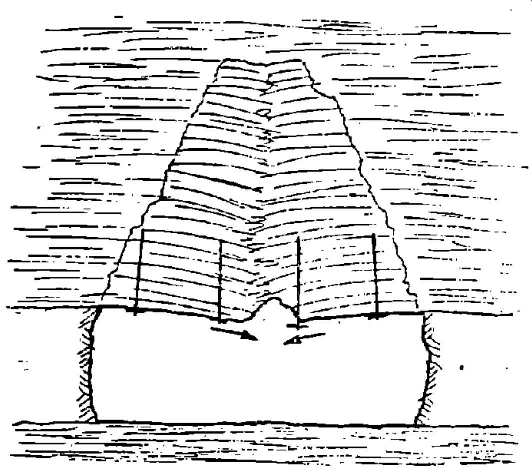


Figure 3.13 High Lateral Stress [3.30]

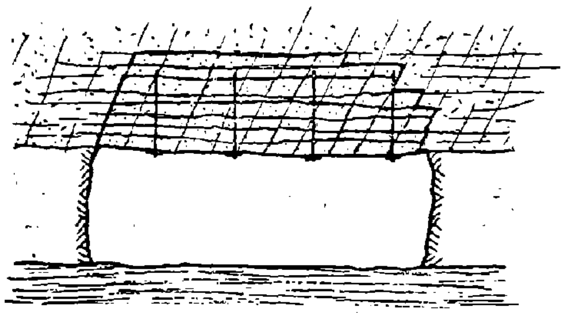


Figure 3.14 Insufficient Lateral Stress [3.30]

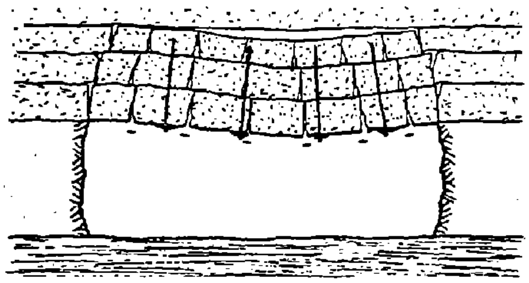


Figure 3.15 Buckling Through of a Voussoir Arch [3.30]

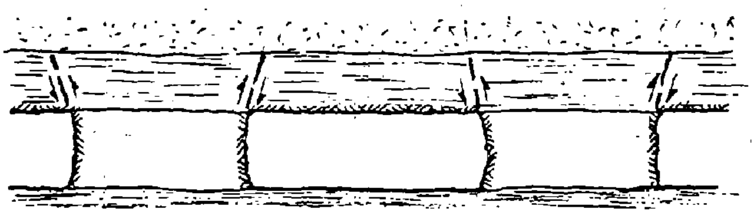


Figure 3.16 Excessive Vertical Load [3.30]

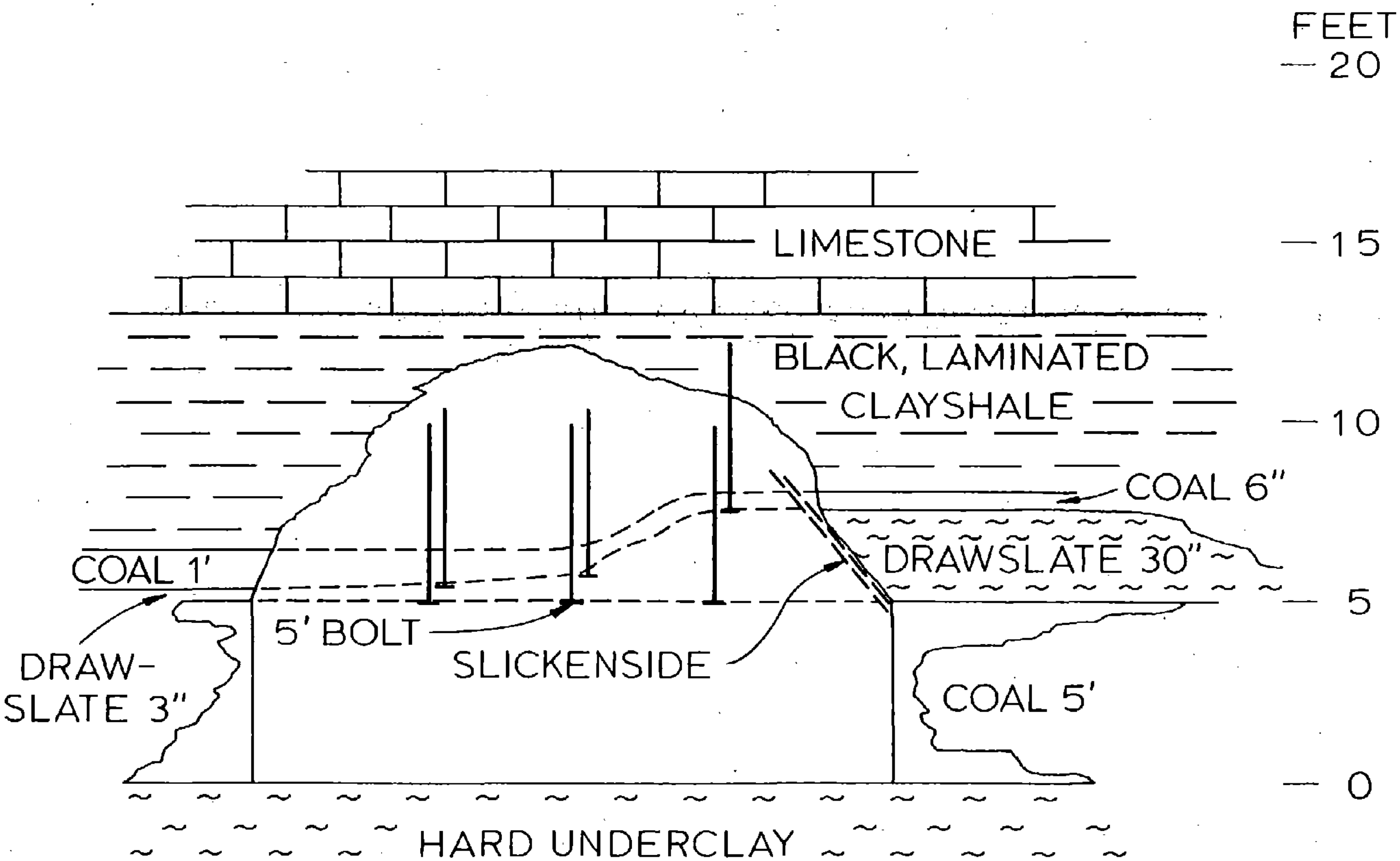


Figure 3.17 Cross Section of Roof Fall in Resin Bolted Test Area of a coal Mine [2.72]

$$0.18 \leq \frac{W}{D} \leq 0.33 \tag{3.4}$$

where W is the width at the base of the arch, and D is the depth of the mine opening. The arch is elliptical in shape with a height above the seam generally about twice the width. The ground arch may form in room and pillar mining at shallow depths, but more generally may be expected at greater depths in longwall mining and during pillar robbing upon retreat in a room and pillar operation or a narrow panel operation. If the rock within the arch is sufficiently weak it may then cave and add load to any roof beam that may be formed by bolting.

Adler [3.34] discusses the pressure arch theory and the influences of span (width) of the opening and jointed rock upon caving over mine openings. Adler [3.35] has also used both the pressure arch theory and a cantilever beam theory to explain caving in controlled longwall mining.

Documentaries of roof falls, in the spirit of Figures 3.12 to 3.16, are lacking in the literature. More effort is needed to record and classify roof fall observations to add to the fund of knowledge.

It is reported [3.72] that there are approximately 1000 disabling injuries per year in underground coal mines that are attributed to roof falls. Of the injuries occurring to the roof bolter operator, 45.2 percent occur while drilling and 52.9 percent occur while inserting the bolt.

Figure 3.18 shows a roof fall in a mechanically bolted area (5-foot bolts on 4-foot pattern) at a 1600 foot depth in the Pocahontas #3 Coalbed in Virginia [3.73]. Evidently the thinly laminated roof, slickensides, and lateral thrust contributed to the problem. Some 4-foot, fully grouted resin bolts and some 8-foot mechanical bolts had been spotted in the face area but to no avail.

In another mine [3.73], at 1300 feet, 4-foot fully grouted, 7/8-inch diameter resin bolts on 4-foot centers fell out with broken rock. It was observed that only a small portion of the resin had adhered to the rebars, that the rebars had twisted by the weight of the rock and the lateral pressure and slipped from their bond with the resin still clinging to the rock material.

The authors [3.73] recommended a fanned resin bolting pattern like that used in France, combined with steel straps or channels and posts, as shown in Figure 3.19. The length of the bolts should be equal to one-half of road width when bolts are used exclusively and to one-third when bolts are combined with steel sets.

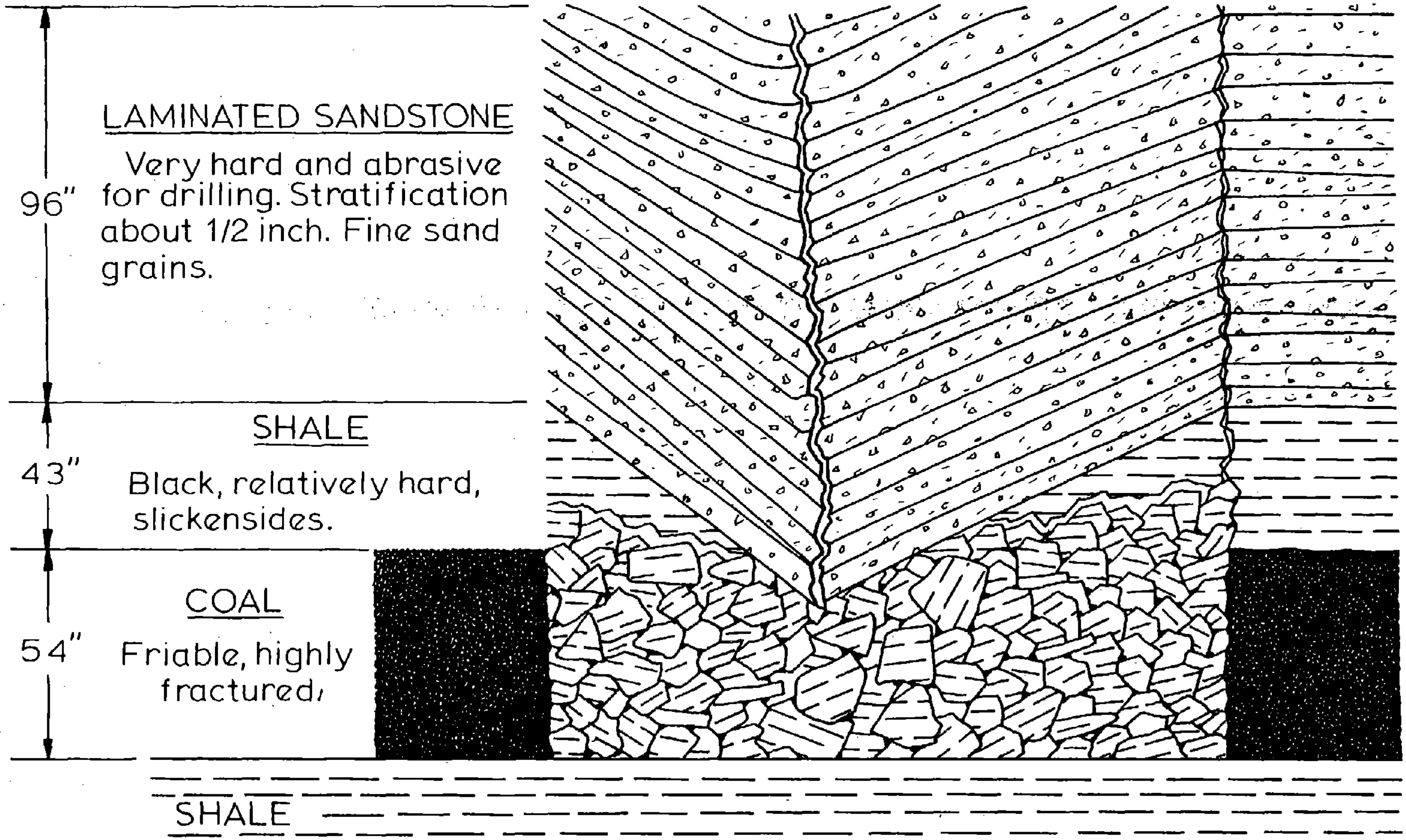


Figure 3.18 Cross Section and Lithology of a Roof Fall Area in Pocahontas #3 Coalbed [3.73]

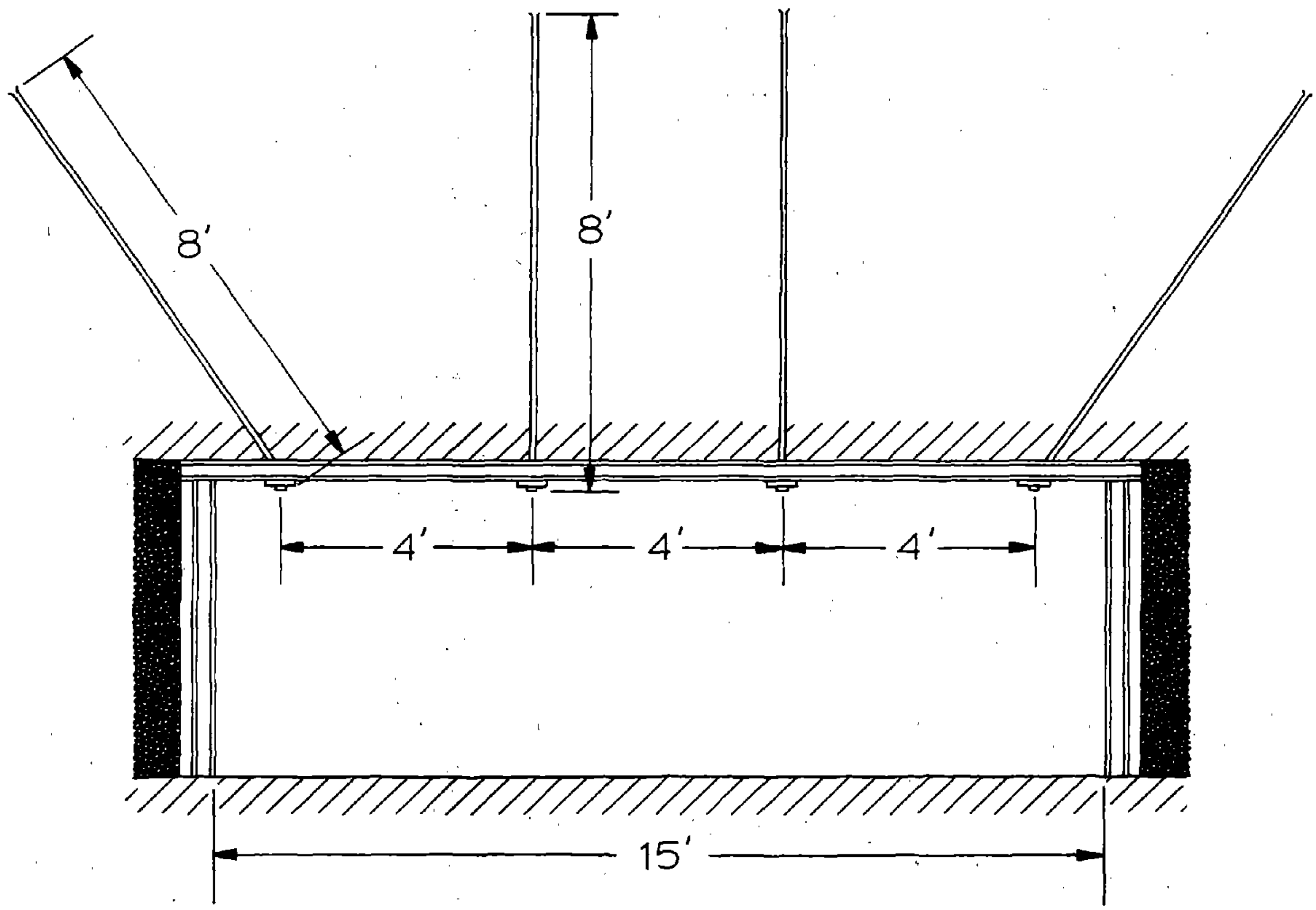


Figure 3.19 Recommended Fanned Resin Bolting Pattern for Development Entries in a Long Wall Operation with Severe Roof Problems [3.73]

A roof fall study [3.80] of five coal mines in the Pocahontas #3 Seam showed that rock type, rock sequence and slickensides in mine roofs are related to the occurrence of roof falls. The best roof is the hard grey sandstone 10 foot or more in thickness of large horizontal (2000 foot) extent. However, sandstone conglomerates composed of shale or coal pebbles tend to weaken sandstones.

The next best roof is the sequence of shale grading upward through interbedded shale and sandstone that can be held by bolting, but where separations occur on sandstone-shale lamination planes, some falls do occur.

The worst roofs are those cut by slickensides, seat earths (root penetrated silty clays - "kettlebottoms"), and rider coal seams within 30 feet of the main seam. The slickensided planes were found to cut the silty claystones and siltstones at angles varying between 90° and 120°.

Hylbert [3.81] studying roof falls in The Harlan Coal Seam (Kentucky) found that a thin-bedded sandstone/shale roof provided poor anchorage for mechanical bolts. A coal rider seam 8 feet to 10 feet up also contributed to roof instability. However, a change to resin bolts greatly improved roof stability in this type of roof.

3.4 Analytical Techniques

The in situ conditions, environmental factors and other conditions contributing to roof falls have been discussed. It is possible to analyze the effect of these factors, to some extent, by conducting stress analyses both experimental and analytical. Here the methods of analysis as well as some of the pertinent results are discussed.

The methods of analysis that have been used to determine stresses around underground mine openings may be classified as follows:

(1) The two-dimensional finite element technique such as used in references [2.40, 2.44, 3.11, 3.17, 3.36-3.47, 3.67]

(2) The three-dimensional finite element technique such as used in references [2.2, 2.52, 2.48, 3.49],

(3) The two-dimensional plate bending analysis of Wright et al [3.50, 3.51],

(4) The electrical resistance analog and elastic potential methods of solution such as used by Salamon, Fairhurst, Crouch and others [3.52-3.59],

(5) Closed-form analytical techniques [3.60-3.64], and

(6) Rigid-body computer graphic techniques [3.65, 3.66].

The finite-element method is considered first, because it is the most powerful; layered, anisotropic, jointed and bolted rock can be analyzed using this method. The mathematics of the solution will not be elaborated upon here, but the intent rather is to give some of the results of the method that are pertinent to the problem at hand, namely the stresses in the roof above rectangular mine openings. Suffice it to say that the finite-element method is a force-displacement method involving the solution of large algebraic equations on the computer. The method involves dividing the mine structure up into a number (hundreds) of small finite elements, constituting a "mesh". Each element of the mesh can be assigned different material properties--stiffness and strength, which is an important feature. In addition to solid elements, joint elements can also be introduced to represent faults and bed separations within the rock strata.

Zienkiewicz, Valliappan, and King [3.36] have used the finite-element method to study the stress state under a valley, around a tunnel and around a power station opening. In all cases, they studied the effect of relief loading (removal of material during excavation or mining). Using a "no tension" material to represent weak fissured rock, they obtained lower bounds on failure loads. Pretensioned mechanical bolts were included. The solution was nonlinear and iterative.

Panek et al [3.37, 3.38] reported on the finite element analysis of roof arching failures over coal mine entries and showed that the height of the arch depended on the ratio of horizontal to vertical pressures and the coefficient of internal friction. Figure 3.20 shows these results. Panek et al [3.11, 3.37, 3.38] studied the effect of different strata stiffnesses in coal mine entries in bedded deposits. Figure 3.21 shows that the presence of weaker strata increases the critical horizontal roof stress in the immediate roof rock. Wang, Ropchan and Sun [3.67] gave more details on three-layer and seven-layer systems such as have been shown in Figure 3.21, but they also gave results for other problems: where there is horizontal variation in roof properties, where there are inclusions in the roof, and where there is variation in surface topography. Horizontal variation of roof properties occurs for "crossbedding" or horizontal segregation of materials in addition to the vertical segregation in the beds. It was found that horizontal segregation causes considerable variation in roof stresses with high tensile stresses occurring in the midspan of the roof when the crossbedded member above the opening is hard rather than soft. The presence of a hard inclusion in the roof had a similar effect. Figure 3.22 shows the model of variation of surface topography for mine

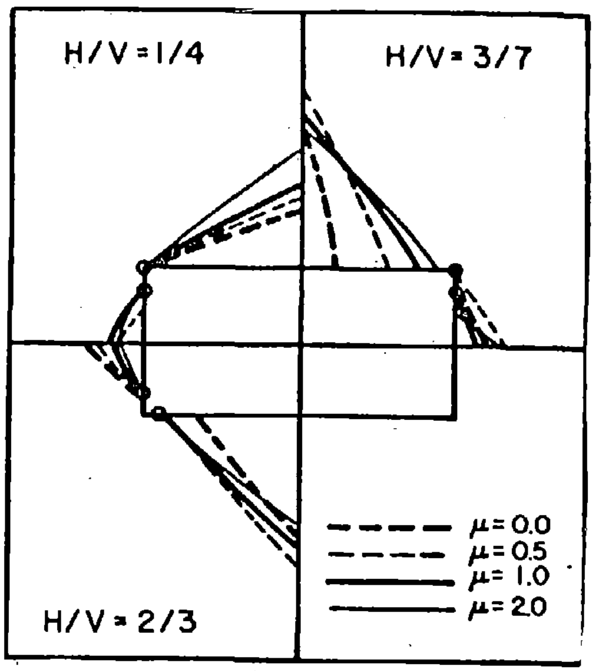


Figure 3.20 Change in Critical Fracture Surfaces with Horizontal Stress and Internal Friction. (H and V are the horizontal and vertical stresses, and μ is the coefficient of internal friction.) [3.37]

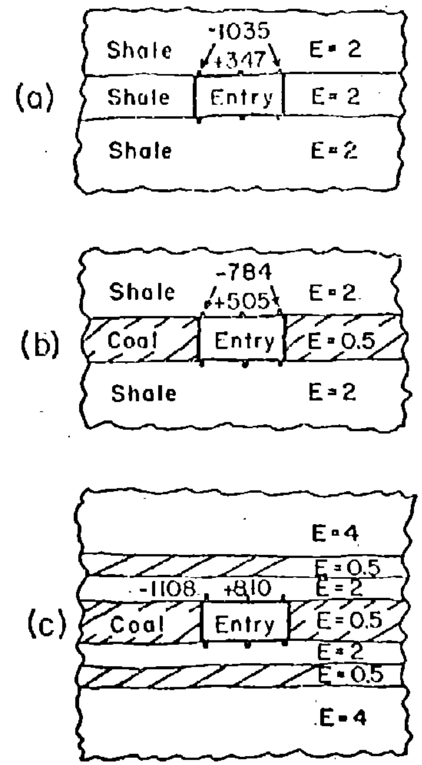


Figure 3.21 Change in Critical Horizontal Roof Stresses (psi) With Strata Stiffnesses (8 x 16-foot entry at 1000 foot depth, E x 10^6 psi) [3.37]

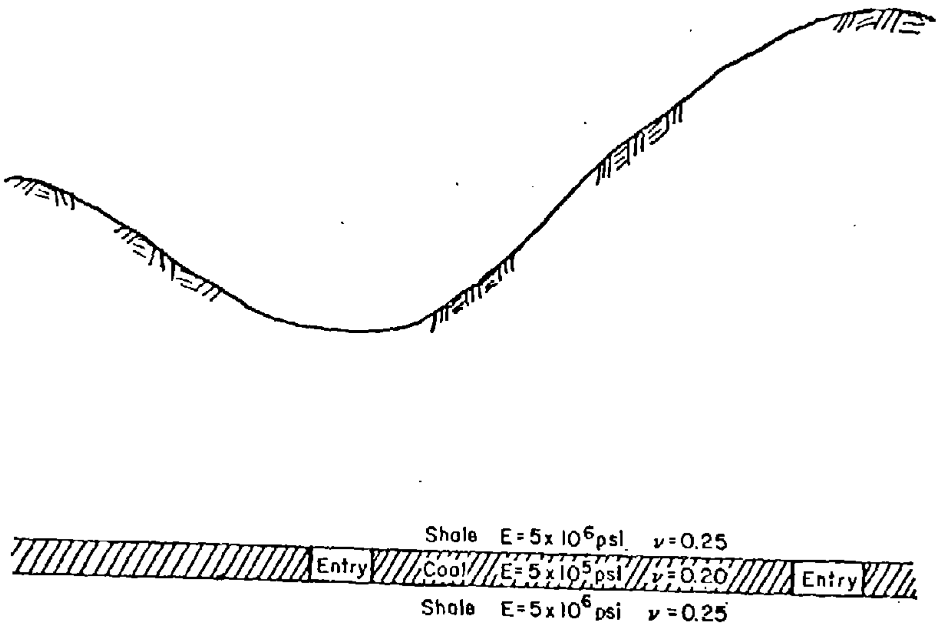


Figure 3.22 Location of Entries Under a Hill and a Stream Valley [3.67]

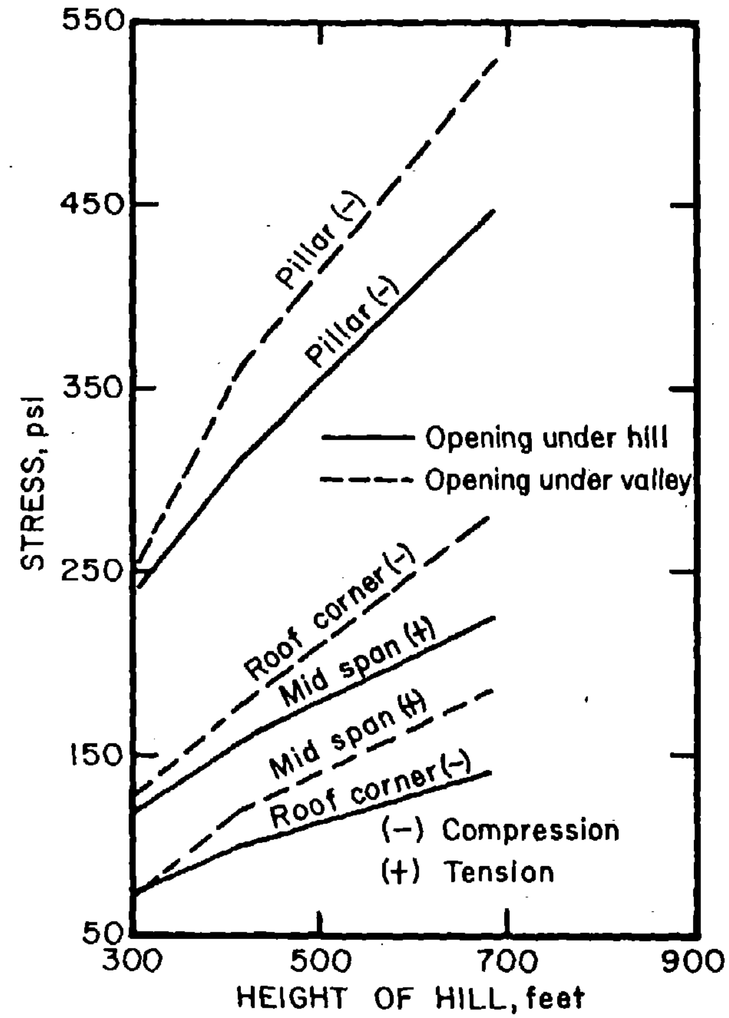


Figure 3.23 Changes in Roof Corner and Midspan Stresses Produced by Elevation Changes [3.67]

openings below a valley and under a hill. Figure 3.23 shows how the mid-span tensile stress increases with the height of the hill for both openings. (The height of the valley above the coal seam was kept constant at 115 feet.) It would be interesting to know the stress distribution when an opening lies between the valley and the hill. Because of the gradient in overburden, the vertical shear stress may reach a maximum halfway in between.

Caudle [3.39] reported on a study of the effect of opening height-to-width ratio and showed how the tensile stress in the roof at mid-span increases with the width of the opening in a soft seam.

As many as twelve layers of strata in a copper mine were analyzed using the finite element method in [2.40]. The effects of bed separations, faults, bolting and creep were included. Although the absolute creep of an individual rock layer may be insignificant in itself, it was found that differential creep between rock layers can be a cause of appreciable stresses. References [3.38, 3.40, 2.44, 3.47, 2.52] contain joints, references [2.44, 3.45] contain bolts, and references [3.41, 3.46] contain plasticity in the analysis but pertain to round tunnels or other problems of secondary interest to this project. However, the methods described in these references are worthy of further study.

Stacey [3.48] reported on a three-dimensional (3-D) finite element analysis but the emphasis was on pillar stresses and on comparisons with the electrical analog method discussed below. A 3-D analysis of an opening (hoist room), including the effects of creep and excavation, was reported in [2.52]. The addition of a joint analysis in a 3-D program was reported in [3.49], but the application was to slopes. Bolting was introduced into a 3-D computer program by Reich [2.2] at Brookhaven to study the reinforcement effect in preventing crack propagation, but evidently the work was discontinued before results were completed. The effect of bolting in a 3-D analysis was studied at Michigan Tech [2.40].

The difficulty with a 3-D analyses is that the size of the problem is generally limited to a quadrant of an intersection and a quadrant of a pillar because computer storage becomes a problem. Wright et al [3.50, 3.51] have found one way around this problem by conducting plate bending analyses. Rectangular plates were used to model competent roof slabs over intersection, whereas beam theory could not. (Beam theory is only acceptable between pillars, midway in entries, where bending is generally uniaxial.) It was shown in [3.50] that beam theory was as much as 100% in error when applied at the intersection. Maximum bending moments and maximum deflections were found in [3.50] in the roof for various pillar lengths and widths, for both regular and staggered patterns of entries. This method was extended to multi-layered roof slabs [3.51]. A finite difference computer program was developed and roof bolts were accounted for only by varying the friction on an element in the program. Although agreement of computer results with laboratory experiments was good, agreement with mine measurements was not. The reviewers suspect the reason for this was that continuous contact between layers was assumed in the plate model, and therefore bed separation in the mine roof was not accounted for.

The electrical resistance analog model based upon the elastic potential solution [3.52-3.59] has been primarily useful to determine pillar stresses and to analyze "coal bump" type failures of pillars and similar failures of the mine faces. This method is limited to a single homogeneous material media above and below the seam.

A closed form solution for coal mine pillar design was presented in [3.61]. An elasticity approach using point loads was used to represent bolt loads in homogeneous media around circular tunnels in [3.62]. A closed form elasticity approach was used to analyze inclusion type problems (stresses around ore bodies before they are mixed in [3.63]). Again the media around the inclusion must be homogeneous. A closed-form plasticity solution was presented in [3.64] to determine the conditions under which a roof may be supported and prevented from caving over rectangular openings. This paper appears worthy of more study.

In [3.60] closed form solutions, beam solutions and Voussoir arch solutions were presented as a basis for mine design.

Rigid body solutions for the dynamic motion of blocky ground has been used to study the stability of slopes [3.65, 3.66]. It appears the approach may be extended to study the stability of fragmented blocky type roofs also.

Brookhaven National Laboratory has conducted three-dimensional finite-element analyses of room and pillar models with roofs bolted with plastic bolts and resin bolts fully bonded and installed both perpendicular and at an angle to a roof with a pre-existing crack. Even with the bolts the crack was open at certain nodal points when a non-uniform pressure was applied [3.74]. Tensile stresses were also found to develop in the unbolted roof using the unbolted model. These progress reports are sketchy and no details are given in the final report [2.2] either. On another project [2.40] the authors attempted to obtain more details from BNL, but none were forthcoming. This is unfortunate, because it is believed some important information has been lost due to insufficient reporting.

Lawrence Livermore Laboratory [3.75] has been modifying the Agbabian finite-element program [2.52], with the purpose of analyzing coal mine roof behavior. The program is called ROCK3D. Three-dimensional results obtained on a room and pillar coal mine show close agreement with experimentally obtained displacement data, [3.79]. Viscoelastic and elastic-plastic analyses were made in addition to the linear elastic analysis. Changes in roof bolt (mechanical) loads showed discrepancies between analytical and experimental values, possibly because the mechanical anchors slipped in the mine. Later work [4.23] has lead to these conclusions:

- The presence of roof bolts appears to have little effect on the gross behavior of the mine. Reducing the bolt stiffness has virtually no effect on roof sag.

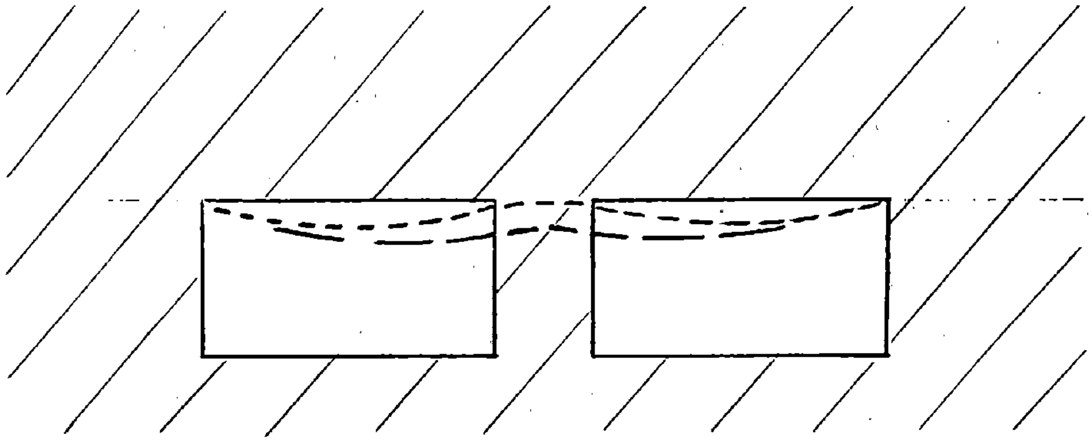
- Two-dimensional models underestimate the sag and roof bolt load changes for a given overburden. A factor of 4 increase should be applied to the overburden to achieve the same results for two-dimensional modes as for three-dimensional ones.

- The load change in bolts is strongly dependent upon excavation and bolt placement sequence as well as upon the distance from the bolt to the face.

Two-dimensional and three-dimensional finite element analyses were also compared in [3.82] and the conclusion was that the two-dimensional analysis wrongly predicts maximum roofsag at the center of the entry rather than at the pillar in a two-entry model. However, it is shown by Gerdeen, et al [3.83] that if the pillar modulus is reduced to compensate for the entries then a truer deflection curve is predicted by the two-dimensional analysis as shown in Figure 3.24. The authors [3.82] also had difficulty calculating correct load changes in roof bolts and attribute this to the relatively low stiffness of the bolts with respect to the surrounding rock.

In [3.83], a procedure was tried to further make the two-dimensional analysis more accurate. The stiffness change of the pillar was made variable across its section and the load was increased on the roof beam. A factor of two gave the right value of maximum deflection at the middle of the intersection. (A factor of four was used by Langland, cited above.) A factor of 3 must be applied to give the correct value of maximum pillar stress at the corner. It was concluded that these factors depend on geometry: pillar width to pillar height to opening width ratios.

In [3.84] a two-dimensional elastic-plastic finite-element analysis of a rectangular opening is reported. No effect of bolting was included.



------ Unmodified, pillar modulus E

—— —— Modified, pillar modulus E'

Figure 3.24 Comparison of Roof Deflection of Two-Dimensional Finite Element Models, Modified and Unmodified, [3.83]

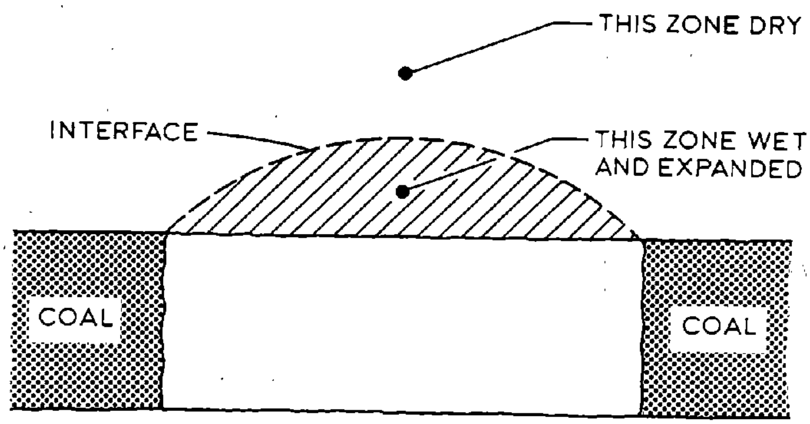


Figure 3.25 Possible Model of Swelling Problem in Roof

Based upon this review of analytical techniques, it is recommended that the finite element approach be used in a parameter study to generate results of the type shown above in Figures 3.20-3.23, that realistic rock properties and joints be included, and that the effects of resin-bolting be included in the models. Models of various roof conditions, shown in the previous section of this report and identified in the field survey, should be analyzed to determine the reinforcement mechanisms of resin bolting. It would seem possible as well to study the effects of swelling from moisture, in a finite element model, by using an analogous thermal strain approach with swelling beginning at the air/rock interface and creeping into the rock at a wet/dry interface, thus giving differential expansion and stress. This is shown in Figure 3.25.

3.5 Model Studies

Ergun has conducted model studies of the stability of square underground openings in jointed media, [3.76]. Glass and plaster models with two sets of joint planes were used. Joints sets at 45°/45° were found to critically affect stability. This effect and the zone of influence increased as the horizontal pressure was decreased relative to the vertical.

Lawrence [3.77] has found from plaster model studies of thinly laminated roofs (unbolted) under horizontal and vertical pressure that the failure was due primarily to Euler type strut buckling. Silvester [3.78] conducted similar tests on resin bolted roofs installed in a 4/3 (alternating rows of bolts with 4 bolts in one row and 3 in the next) pattern with the midspan bolt vertical, the bolts closest to midspan inclined at 10° to the vertical, and the bolts closest to the rib inclined at 20° to the vertical and over the ribs. He found that where the immediate roof was strengthened by the clamping of individual layers together by means of an effective bolting system, bed separation and the fracturing of the roof were delayed and movements in the higher stress range were reduced. However, bolted supports tended to give high vertical movements at low stresses. This could be explained by the fact that the bolts were not prestressed during the installation and perhaps were less effective until tightening took place as the result of first movements in the roof.

Roof dowels in the 4/3 pattern produced a higher inverted V zone of influence in the roof. This may be explained by the suspension action of the dowel anchored in the unfractured zone. This theory could be proved by installing longer dowels to see whether it would further increase, as expected, the height of the inverted V.

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4.0 COAL AND ROCK PROPERTIES

To reiterate, a thorough analysis of resin bolting requires knowledge of mine conditions, as discussed in the previous section of this report, and it also requires knowledge of the mechanical properties of the coal and rock constituting the structure around the mine openings in coal mines.

4.1 Property Data

When the question is posed, "What is the compressive strength of rock A?," for example, it is found that there is disagreement concerning the answer. Some will argue in favor of in situ properties over laboratory properties. In situ properties are what eventually are needed, but there is an alternative way of assessing these, i.e. accept the values from laboratory tests, but account for size effect, humidity, triaxial stresses (or hydrostatic pressure), and joints in the rock when conducting the stress analysis. Both in situ and laboratory properties are reviewed here.

A modulus of elasticity profile (for both shear and normal stresses) is plotted along the columnar section of a coal mine in Figure 4.1 as reported by Lu [3.2]. In situ properties were measured by a cylindrical, hydraulic, borehole pressure cell. The modulus values for the floor and roof are for the horizontal direction (vertical boreholes), and the values for the coal seam are for the vertical direction (horizontal boreholes). As evident the properties can vary. For example, the shear modulus G varied from 0.21 (10^6 psi) in the coal bed, to 0.77 (10^6 psi) for the shale 10 feet above the coal bed, and to 0.87 (10^6 psi) for the sandy shale 8 feet below the coal bed [3.2]. Properties of four different coals in the vertical direction are given in [3.55]. Two of these coals are from the Pocahontas seams No. 3 and No. 4 in West Virginia and two are from the Geneva and Sunnyside Mines in Utah. The Pocahontas coals are relatively soft and have pronounced cleat systems, the others do not. Average values for the compressive strength are given in Table 4.1, as determined from least squares fits of data from specimens with different diameter-to-height ratios.

Table 4.1 Compressive Strengths of Coals,

$P = P_o(D/H)^a$ [3.55]

Coal	Strength Coefficient P_o, psi	Exponent a
Pocahontas No. 3	1733	0.68
Pocahontas No. 4	2035	0.61
Sunnyside	4078	0.54
Geneva	2122 (2-in. D)	0.34
	5130 (4-in. D)	0.34

D = diameter, H = height

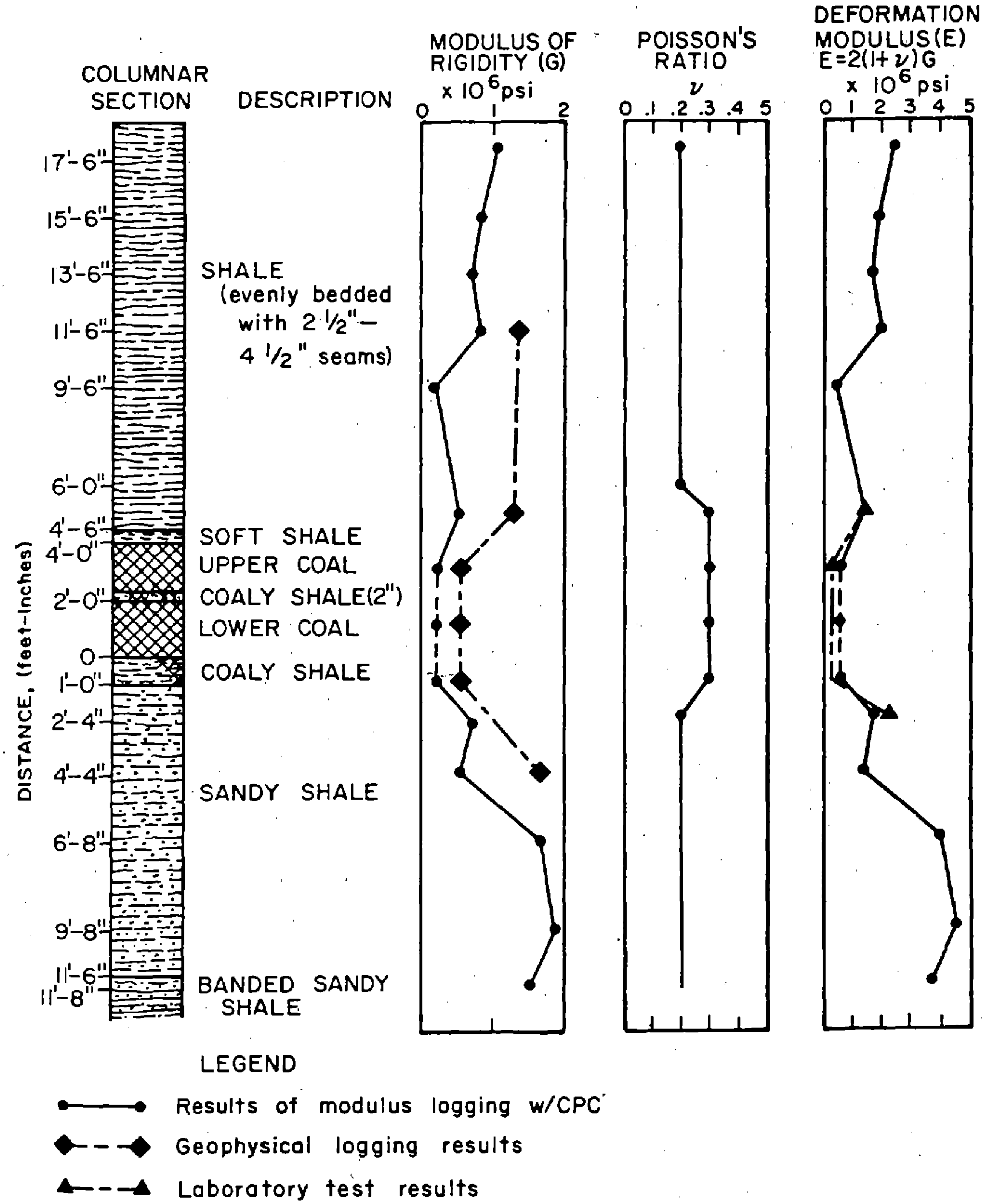

Figure 4.1 Modulus Profile of Coal Measure Strata [3.2]

The Geneva coal showed a size effect--different strengths for different diameters. The Utah coals were found to be about twice as strong, but five times as brittle as the West Virginia coals.

Triaxial tests were also conducted on the Geneva coal [3.55]. Results are shown in Table 4.2. Post-peak slopes of stress strain curves of all of the coals were also determined for use in pillar failure (coal bump) analysis. These are not reported here but may be found in [3.55].

Mechanical properties of shales from coal mines were reported in [2.2], and are listed here in Table 4.3. (Omitted here are values for polymer impregnated shales.) Vertical properties are given generally, but for some tests the direction was not reported, and was assumed vertical. One value of compressive strength of a coal, Moss No. 3, was reported to be 860 psi, much lower than the shale strengths.

Compressive strengths of Pittsburgh coal were reported in [4.1] and were shown to depend on specimen size. The lowest strength for the larger size cubical specimens (100 to 200 cm) was about 5-7 (10^6) N/m^2 (~1000 psi), similar to that for the Moss No. 3 above. Creep data are reported in [4.1] for British coals but not for American.

In situ properties of a Western Coal in Colorado (Bear Creek No. 4, Mid-Continent Coal and Coke Co.) are reported in [4.2]. Young's Modulus values of .144 to .149 (10^6) psi were found. Laboratory values on 4 inch cubes were 0.100 to 0.122 (10^6) psi perpendicular to the bedding planes and 0.080 to 0.091 (10^6) psi parallel to the bedding plane. Laboratory values for compressive strength were 770 to 1170 psi perpendicular and 730-780 psi parallel.

Properties of a Kentucky roof shale were determined by Augenbaugh [4.3]. Unconfined compression tests on 40 specimens showed variations from less than 2000 psi up to 24,000 psi. Within the mine, strengths varied by as much as 8000 psi. Triaxial tests were made on 60 specimens and Mohr-Coulomb envelopes were constructed as shown in Figure 4.2. There was scatter in the data as shown. A Mohr-Coulomb envelope has also been determined for a Sunnyside Sandstone (from Utah) and is shown in Figure 4.3 [4.4]. For the 45^0 inclination (Figure 4.2), failure occurred across the layers. Table 4.4 gives some values of tensile strengths of some limestone and sandstones [4.5].

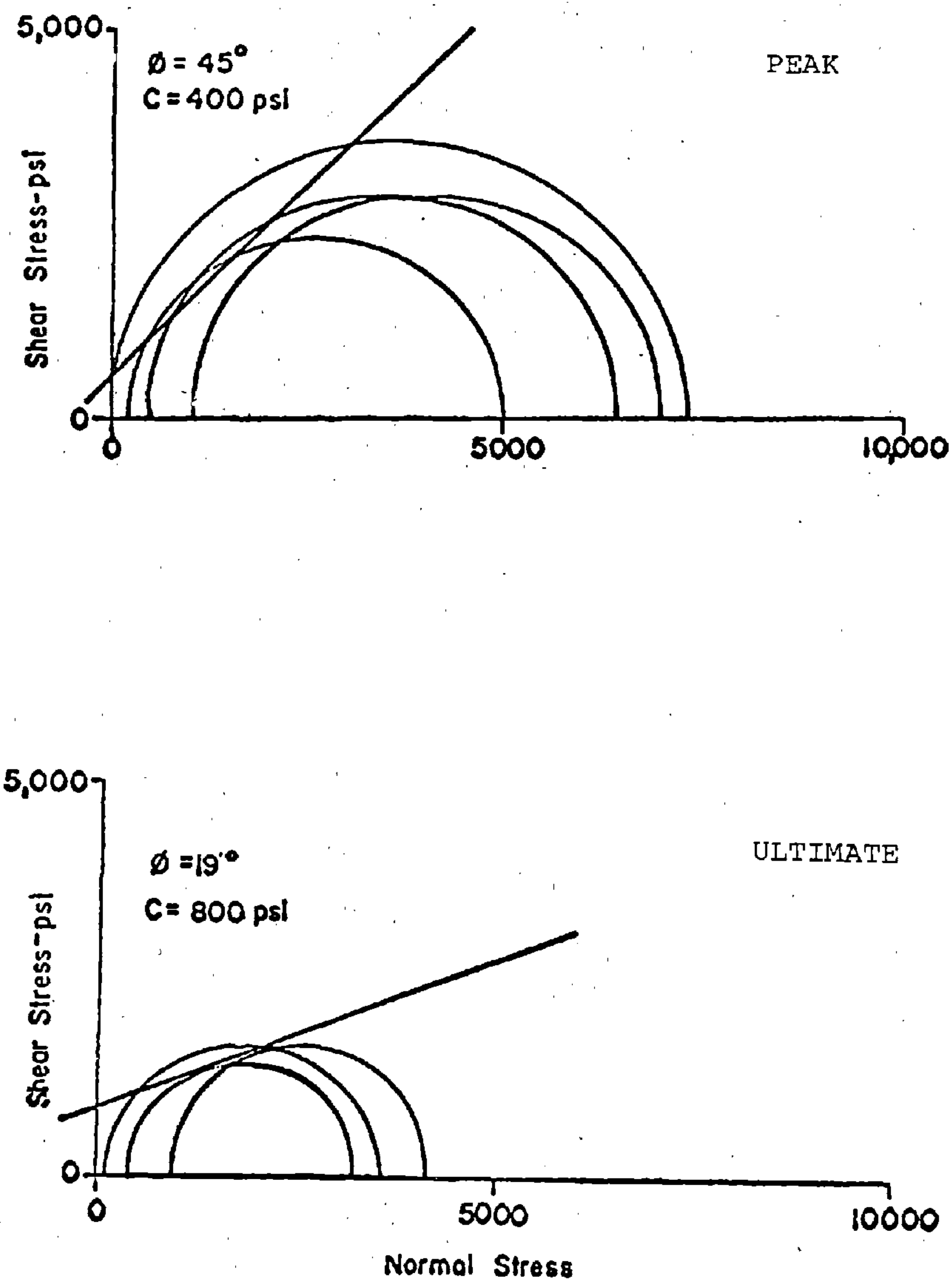

Figure 4.2 Triaxial Shear Strength of a Kentucky Roof Shale [4.3]

Table 4.2 Summary of Results From Triaxial Compression Tests on Geneva Coal [3.55]

Confining Pressure (psi)	Compressive Strength (psi)	Modulus of Elasticity (10^6 psi)	Poisson's Ratio
0	1,030 3,100 2,350	.22 .46 .38	- - -
500	7,200 5,250 9,950	.45 .40 .72	0.41 0.38 0.36
1,000	9,200 10,300 8,350	.72 .68 .92	0.47 0.35 0.41
2,000	8,600 13,800 12,700	.73 .76 .78	0.35 0.41 0.36
3,000	16,600 15,300 7,300	.73 .80 .64	0.39 0.35 0.37

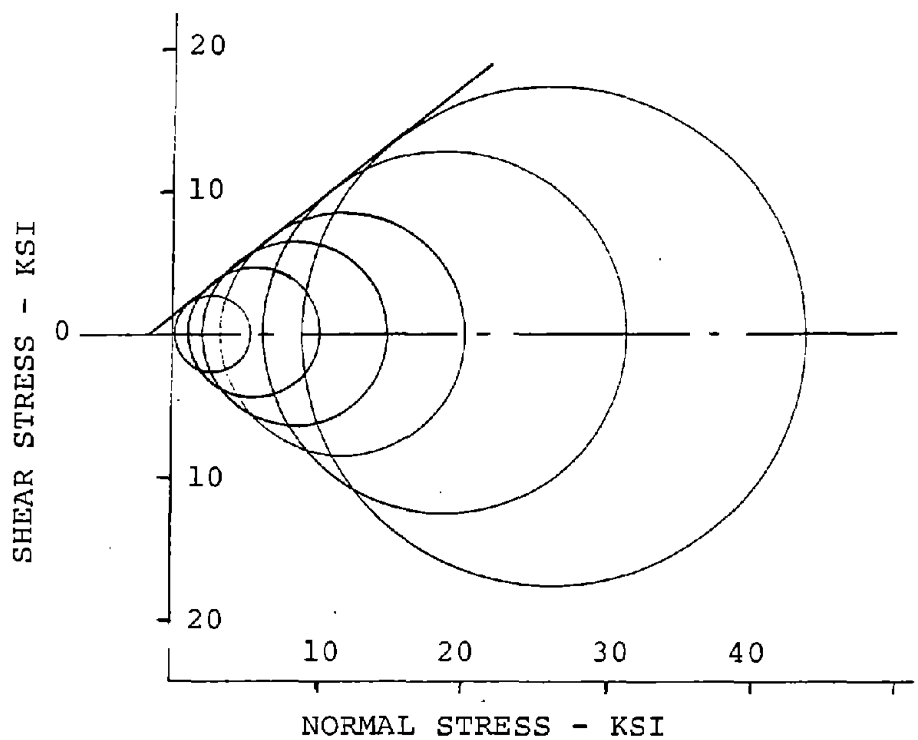

Figure 4.3 Mohr-Coulomb Envelope for Sunnyside Sandestone at a 75° Inclination [4.4]

Table 4.3 Mechanical Properties of Coal Bearing Shales [2.2]

Shale	Compressive Strength psi	Tensile Strength, psi	Modulus of Elasticity psi	Poisson's Ratio
Kepler	11,850	---	---	
Itmann No. 1	4,500			
Itmann No. 4	6,000 9,000			
USS No. 4B	10,400	---	1.9 (10^6)	0.32
USS No. 4B	---	80	0.8 (10^6)	---
Pike 26D	---	70		
Pike 26E	---	140		
Pike 26F	---	438		
Bruceton	---	26		
USS 4A	---	660-1185 (flexure)	0.2-4.9 (flexure)	

Table 4.4 Tensile Strengths of Limestones and Sandstones [4.5]

Rock Type	Tensile Strength, psi
Indiana Limestone	500
Lenders Limestone	700-800
Berea Sandstone	300
Tennessee Sandstone	1000

Mohr-Coulomb data are presented in [4.6] for Lovilia, Iowa coal. The envelope data found are:

$C = 1154$ psi and $\phi = 36$, loading parallel to bedding, and

$C = 800$ psi and $\phi = 42$, loading perpendicular to bedding, where $\tau = C + \sigma \tan\phi$. (Data for plaster and cement are also given in [4.6] which may be useful in laboratory models of coal.)

Reference [4.7] gives a host of test data on "Old Ben" and Pittsburgh coals. Included are static strengths, elastic moduli (or compliances), and creep parameters. Although useful for this project, the data are quite extensive and the format of data presentation is somewhat unconventional, so it will take some effort to reduce the data to a useful format for analysis purposes.

References [4.8], [4.9], [3.14], and [3.33] give properties of South African, Indian, Russian and English coal respectively. Reference [4.10] emphasizes the dynamic and cutting properties of rock.

Although intended for use in analysis of dams, the data in [4.11] for siltstones and sandstones from northwest Pennsylvania may prove useful. This reference is somewhat unique for it includes the important normal and shear stiffnesses, K_n and K_s, for joint surfaces in these materials. The uniaxial compressive strengths averaged 21,100 and 20,800 psi respectively for the siltstone and the sandstone. Sandstone had the highest shear stiffness, with the effect of normal stress on shear stiffness small, whereas the normal effect was apparent in the siltstone.

Anisotropy is an important factor in rock materials. Batugen and Nirenburg [4.12] give approximate relations between the elastic constants in different directions for anisotropic bedded rocks which should be useful.

Table 4.5 gives some rock property data for the "Fort Union Formation" of Wyoming and Montana, [4.19]. These data are quite complete - they include C and Φ data.

Figure 4.4 gives measured properties for a stratigraphic column in the Pittsburgh coalbed, [4.22]. The properties include tensile strength, compressive strength, modulus of elasticity, shore hardness and specific gravity. These data are more complete than Figure 4.1. The values for E are approximately the same for shale in Figures 4.1 and 4.4.

Table 4.6 gives rock property data for a coal mine in Ohio.

TABLE 4.5 AVERAGE MECHANICAL PROPERTIES OF THE FORT UNION FORMATION, [4.19]

	ROCK TYPE	YOUNG'S MODULUS E ($10^9 N/m^2$)	POISSON'S RATIO ν	BRAZILIAN TENSILE T_B (KN/m^2)	DENSITY ρ (Kg/m^3)	COHESION C ($\times 10^6 N/m^2$)	ANGLE OF INTERNAL FRICTION ϕ (°)
A.	Siltstone	0.85	0.38	430	2,300	2.28	33.33
B.	Shale	1.3	0.30	1,600	2,300	5.55	29.03
C.	Sandstone	7.0	0.47	10,750	2,700	5.33	60.46
D.	Clay	0.22	0.49	407	2,390	0.26	36.58
E.	Coal	1.98	0.39	1,880	1,320	6.16	31.30
F.	Shale	0.22	0.30	3,850	2,380	0.26	36.58

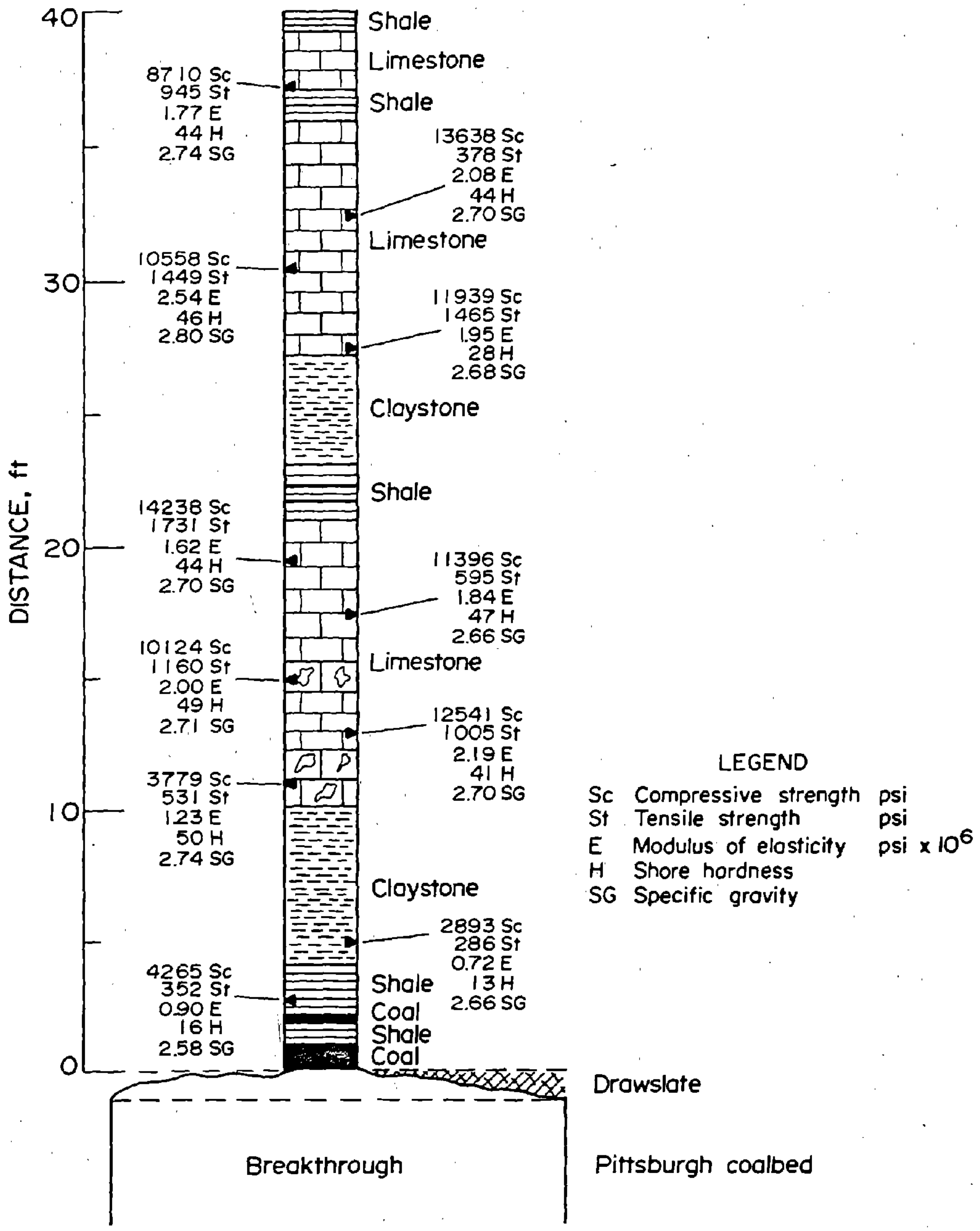

Figure 4.4 Stratigraphic Column Showing Measured Properties [4.22]

TABLE 4.6 PHYSICAL PROPERTIES MEIGS NO. 2 COAL MINE, ATHENS, OHIO, [4.23]

Description	Mean compressive strength, in pounds per square inch (1)	Tensile strength parallel to bedding, in pounds per square inch (2)	Modulus of rupture in pounds per square inch (3)	Young's modulus, in 10^6 pounds per square inch (4)	Poisson's ratio (5)	Unit weight in pounds per cubic feet (6)
Sandstone	11,685	500	--	2.2	0.105	160
Limestone	22,500	670	1653	11.1	0.29	166
Carbonaceous shale	7,580	400	986	0.72	0.19	107
Coal	5,699	150	--	0.568	0.35	82
Fire clay[a]	5,500	-	--	2.34	0.125	160
Weak shale	6,670	-	--	1.6	0.16	150

Note: 1 psi = 6.9 kN/m^2; 1 lb/ft^3 = 16.2 kg/m^3; 1 lb = 4.45N.

4.2 Rock Quality and Rock Classification

As shown above, rock properties may vary greatly, especially when jointing, weathering and other factors are considered. The condition of the rock will determine to a large extent what type of roof support is needed, whether it be resin bolting or something else. Many have recognized this fact and have recognized the need to classify rock quality in some way.

Barton, Lien and Lunde [2.64, 2.65], Cecil [2.85], and Bieniawski [4.13, 4.14], for example, have been working toward this end. Following [2.64, 2.65], an equation for estimating rock mass quality (Q) can be expressed as

$$Q = (RQD/J_n)(J_r/J_a)(J_w/SRF) \qquad (4.1)$$

where RQD is the rock quality designation (Deere [4.15]), J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, and SRF is the stress reduction factor. The three pairs of parameters are crude measures of block size (RQD/J_n), inter-block shear strength (J_r/J_a), and active stress (J_w/SRF). The six parameters are further defined in Tables 4.7 to 4.12.

In Table 4.7, the RQD can be established from rock cores or can be estimated from the number of joints per unit volume. For clay free rock masses, a simple relation has been used [2.64]:

$$RQD \simeq 115 - 3.3\, J_v \qquad (4.2)$$

where J_v is the total number of joints per m^3 and RQD is taken as 100 if J_v is anything less than 4.5. Goodman [4.16] discusses the estimation of RQD in discontinuous rocks.

Barton et al [2.64, 2.65] have tested this method on about two hundred case records and have come up with a support-requirement design chart shown previously in Figure 2.45. The "Evacuation Support Ratio," (ESR) in Figure 2.45 needs some explanation. It reflects mining practice and the degree of safety required. For temporary mine openings, a value of ESR of 3-5 is suggested, and for permanent mine openings a value of 1.6 is suggested.

The experience of Barton, et al [2.64, 2.65] has generally been more with hard rock and round tunnels. This method has some potential and perhaps can be extended to the problem of roof bolting of coal mines if the roof rocks can be categorized in this way. Thus, this approach is worthy of further study. The following steps would need to be followed [2.64, 2.65]:

(1) Classify the relevant rock mass quality,

(2) Choose optimum dimensions of mine openings and pillar widths, and degree of safety required,

Table 4.7 Descriptions and Ratings for the Parameter RQD [2.64, 2.65]

	1. ROCK QUALITY DESIGNATION	(RQD)
A.	Very poor	0 - 25
B.	Poor	25 - 50
C.	Fair	50 - 75
D.	Good	75 - 90
E.	Excellent	90 - 100

Note: (i) Where RQD is reported or measured as = 10, (including 0) a nominal value of 10 is used to evaluate Q in Equation (4.1)

(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate.

Table 4.8 Descriptions and Ratings for the Parameter J_n [2.64, 2.65]

	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
G.	Three joint sets plus random	12
H.	Four or more joint sets, random, heavily jointed, "sugar cube" etc.	15
J.	Crushed rock, earthlike	20

Note: (i) For intersections use (3.0 x J_n)

(ii) For portals use (2.0 x J_n)

Table 4.9 Descriptions and Ratings for the Parameter J_r [2.64, 2.65]

3. JOINT ROUGHNESS NUMBER

	(a) Rock wall contact and *(b) Rock wall contact before 10 cms shear*	(J_r)
A.	Discontinuous joints	4
B.	Rough or irregular, undulating	3
C.	Smooth, undulating	2
D.	Slickensided, undulating	1.5
E.	Rough or irregular, planar	1.5
F.	Smooth, planar	1.0
G.	Slickensided, planar	0.5

Note: (i) Descriptions refer to small scale features and intermediate scale features, in that order.

	(c) No rock wall contact when sheared	
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

Note: (ii) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.

(iii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are orientated for minimum strength.

Table 4.10 Descriptions and Ratings for the Parameter J_a [2.64, 2.65]

	4. JOINT ALTERATION NUMBER	(J_a)	(ϕ_r)
	(a) Rock wall contact		(approx.)
A.	Tightly healed, hard, non-softening, impermiable filling, i.e. quartz or epidote	0.75	(-)
B.	Unaltered joint walls, 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	(25-35°)
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 claying (see G,H,J for description of clay condition)	10, 13, or 13-20	(6-24°)

Table 4.11 Descriptions and Ratings for the Parameter J_w [2.64, 2.65]

5. JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pres. (kg/cm^2)
A. Dry excavations or minor inflow, i.e. < 5 l/min. locally	1.0	<1
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
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

Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
(ii) Special problems caused by ice formation are not considered.

Table 4.12 Descriptions and Ratings for Parameter SRF [2.64, 2.65]

6. STRESS REDUCTION FACTOR

(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated. (SRF)

		(SRF)
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10
B.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leqq$ 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 $\leqq$ 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

Note: (i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

(b) Competent rock, rock stress problems

		σ_c/σ_1	σ_t/σ_1	(SRF)
H.	Low stress, near surface	>200	>13	2.5
J.	Medium stress	200-10	12-0.66	1.0
K.	High stress, very tight structure (usually favorable to stability, may be unfavorable 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

Note: (ii) For strongly anisotropic virgin stress field (if measured): when $5 \leqq \sigma_1/\sigma_3 \leqq 10$, reduce σ_c and σ_t to $0.8\sigma_c$ and $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, 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.

(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 imcompetent rock under the influence of high rock pressure (SRF)

		(SRF)
N.	Mild squeezing rock pressure	5-10
O.	Heavy squeezing rock pressure	10-20

(d) Swelling rock: chemical swelling activity depending on presence of water

		(SRF)
P.	Mild swelling rock pressure	5-10
R.	Heavy swelling rock pressure	10-15

(3) Estimate the appropriate support required (e.g. resin bolting, bolt spacing, bolt length; or mechanically bolting; or cribbing and posting), and

(4) Compare estimates with actual case records (from a field survey).

Franklin [4.17] tried to classify all rocks according to their mechanical properties, but found considerable overlap in parameter values. Rather than demonstrating that rock types are mechanically identical, this finding illustrated the inadequacy of traditional geological nomenclature when used for mechanical classification.

Nag [4.18] has used plaster models to study the decrease in strength due to two sets of regular joint patterns of vertical joints; horizontal joints; orthogonal joints at 0°/90°, 45°/45°, and 60°/30°; and horizontal joints with nonaligned vertical joints. The loading was vertical (σ_1) with various values of horizontal confining pressure (σ_2). The 60°/30° joint system was the weakest showing about a 50% decrease in the shear strength on the Mohr-Coulomb plot. Failure occurred in two stages for the jointed models: first, slip along the joints and secondly, tensile failure perpendicular to the σ_1, σ_2 plane after joint interlock. The Mohr-Coulomb envelope was nonlinear in all cases and intercepted the origin.

Lee, Smith and Savage [4.19] summarize results of Walsh [4.21] on the effect of small randomly oriented joints in rock. For open joints the effective Young's modulus is

$$E_{eff} = E/[1 + \frac{4}{3}\pi(1-\nu^2)\bar{c}^3/\bar{v}] \tag{4.3}$$

where E is the modulus of intact rock, $\bar{c}$ is the average joint length, and $1/\bar{v} = N/V$ is the number of fractures in a specific volume. The effective Poisson's ratio in this case is

$$\nu_{eff} = \nu E_{eff}/E \tag{4.4}$$

For rock with closed joints:

$$E_{eff} = E/[1 + \frac{4\pi\bar{c}^3}{15\bar{v}}(\frac{2 + 3\mu^2 + 2\mu^4 - 2\mu(1+\mu^2)^{2/3}}{(1 + \mu^2)^{2/3}})] \tag{4.5}$$

$$\text{and } \nu_{eff} = 1/2 - \frac{(1-2\nu)}{2} E_{eff}/E, \tag{4.6}$$

where μ is the coefficient of sliding friction.

The above equations were used in [4.19] to predict that E in siltstones, shale and coal is reduced by 80% for open joints and by 62% for closed joints where there are 1250 fractures of 10 cm average length per m^3. For more massive sandstone and claystone, a 50% and 28% reduction were found respectively when there are 300 fractures of 10 cm average length per m^3.

The Drucker and Prager yield criterion has been used in finite-element computer programs in place of the Mohr-Coulomb relation to predict yield of rock. The Drucker-Prager criterion is

$$f = \sqrt{J_2'} + \alpha J_1 = k \tag{4.7}$$

where

$$J_2' = \frac{1}{6}[(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2]$$

$$J_1 = \sigma_1 + \sigma_2 + \sigma_3, \tag{4.9}$$

and where σ_1, σ_2, σ_3 are principal stresses. The constants α and k can be evaluated in a triaxial test from the relations:

$$\alpha = 2 \sin\phi/(\sqrt{3}(3-\sin\phi)) \tag{4.10}$$

$$k = 6C \cos\phi \ (\sqrt{3}(3-\sin\phi)) \tag{4.11}$$

where C is the cohesion and ϕ is the internal angle of friction.

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5.0 SUMMARY AND CONCLUSIONS

The review of the literature has shown that there are considerable data available on resin bolting, analysis of mine conditions, and coal and rock properties.

5.1 Resin Bolting

Material property data available on the resin suggests that the behavior (including the modulus of elasticity) in tension and compression may be different. This should be checked further. Chemical bonding of resin to steel and resin to rock may exist under ideal conditions and low loads, but appears to be absent in the mine environment.

Axial stiffnesses of resin bolts are 10 to 20 times greater than for mechanical bolts. A ribbed resin bolt (rebar) fails by failure of the steel in hard rock, but a smooth bolt pulls out--indicating that mechanical interlock is the primary means of bonding. In soft rock, the resin bolt assembly fails at the rock interface--the resin is stronger than the rock. A larger hole would provide a greater surface area at the resin rock interface, reduce the shear stress there, and increase the anchorage in soft rocks. In such a case, use of wood of a larger diameter may save steel costs and still give sufficient bolt strength. This should be investigated further.

Under axial loading the load decays with distance into the rock along a fully grouted bolt. The load is distributed over a shorter distance (15 inches) in hard rock, but over a greater distance (30-40 inches) in soft rock. This load transfer distance is a design factor that has to be related to the thickness of the lamellae in the bedded roof. Creep of resin bolts has been found to increase the load transfer distance with time, particularly in softer rock and coal. Analytical methods are available to predict the load transfer length and stress distribution along a fully grouted bolt as a function of the resin bolt properties.

Transverse shear strength of bolted joint is an important advantage of the resin bolt, particularly at low normal pressures across the joint (bedding plane). Experimental data are available for resin bolts oriented normal and at ±45° to the joint surface, and for both rough and flat joint planes. Under transverse shear loads, little benefit was found from either post-tensioning or pre-tensioning of resin bolts. The load transfer length under shear loading is found to be much less (about 3 to 4 inches) than under axial loading. Analytical methods are also available for estimating the shear strength and load transfer length for shear loading of resin bolts. Joint elements have been constructed for finite element programs, to account for both the axial stiffness (K_n) and the shear stiffness (K_s) of resin bolted joints.

Some resin-bolt design charts are available giving the bond factor (anchorage) as a function of rock strength for axial loading. Such design charts are lacking for shear loading and should be constructed.

A resin bolt can provide a suspension effect in a mine roof just like a mechanical bolt, and many times with a better anchorage. Length of bolt to reach competent rock is an economical factor favoring the mechanical bolt in many cases. Friction from clamping is not a primary means of reinforcement of the untensioned grouted bolt, however, a grouted bolt can maintain the pre-compression and initial friction between rock layers that existed before installation--and do this better than the mechanical bolt because of the resin bolt's greater stiffness and short load-transfer length.

A resin bolt is expected to provide a better reinforcement to the Voussoir rock-arch type of roof support mechanism. Because resin bolts can resist compression, they can help transmit the compressive thrust through the rock arch to the ribs. If a resin bolt is angled over the ribs, it can help prevent vertical shear failure at the ribs. A resin bolt with its greater shear resistance would be more effective.

A resin bolt can help build a roof beam even where the rock joints do not offer any frictional resistance themselves because of bed separation. The effectiveness depends upon the number of layers, lateral stresses and abutment support, but has been found to decrease slip of the beds during bending by as much as 50%. In laminated beams, an advantage of the fully-grouted bolt over the mechanical, is that failure at any point along the grouted bolt will not reduce the total reinforcement effect since the grouted bolt adds shear resistance to each layer independently. With point anchored bolts, failure at any point eliminates all reinforcement.

It is reported in the literature that in most mines a change to resin bolting has greatly reduced roof falls. In a few exceptions, resin bolting did not help.

A new resin bolting concept, the "debondable bolt" appears to offer better anchorage in soft rock, and a yielding capability to absorb large rock deformations, while retaining the features of the fully grouted resin bolt at lower loads.

5.2 Mine Analysis

Designing of an adequate roof bolting plan requires an analysis of the mine conditions as well as the bolt assembly itself. Important in situ conditions are stresses (gravitational, thermal, tectonic), moisture, and geology. Faults and surface topography, valleys and hills, can greatly alter the stresses in the mine roof.

Water can cause swelling and stress in mine rock, and cause deterioration in clay-bearing shales.

The geology of the roof section determines the type of roof support mechanism that should be designed for a particular mine. In many coal mines, a coal rider has been found in the roof, causing support problems. The geological column of the roof is reported in the literature for several mines and several coal seams. It is found to vary appreciably within a seam, within a mine as well as from mine to mine. It is important to know coal cleat, rock joint and fault orientations, locations of clay veins, and overburden isopach maps--they all affect roof instability. Some of these data are available in the literature for certain mines, and will be useful in analysis on this project.

Blast damage has been found to damage the roof and loosen the roof bolt anchorage, up to 12 to 15 feet from the face, and up to 2 to 4 feet into the roof. Experiments on blasting effects on resin bolting show that resin bolting is more effective than mechanical bolting under these conditions, because of the greater damping and greater stiffness of the resin bolt assembly.

Roof failure observations have been made by many investigators and have been reported and classified in the literature. They can be classified as tensile failures, shear failures, and delamination failures. Different factors contributing to each have been identified.

Several methods of analysis have been used in the calculation of stresses around mine openings, the most useful being the finite-element method. This method requires the computer, but is most powerful because layered, anisotropic, jointed and bolted rock can be analyzed. Calculations have been made on the effect of horizontal stresses on the critical fracture surface around mine openings. The results show that the critical fracture surface (or cave) will extend higher or lower into the roof depending upon the coefficient of friction of the roof rock.

Results for multi-layered strata show that the presence of weaker strata increases the critical horizontal roof stress. Calculations show that horizontal segregation, i.e. "cross-bedding" in the roof, and inclusions in the roof can cause great variation in the roof stresses with high tensile stresses at mid-span when the cross-bedded member or inclusion is hard rather than soft.

Calculations have been made for a coal mine entry driven under a valley and one under a hill. The results show that mid-span tensile stresses increase with the height of the hill for both openings with the opening under the hill having the greatest mid-span stress. It would be interesting to have results for an opening between the valley and the hill, where the greatest gradient in overburden occurs.

Studies are also reported for the effect of opening width-to-height ratio, the effects of joints and faults, and the effects of bolting on roof stresses and roof deflection.

5.3 Coal and Rock Properties

There are data in the literature for the strengths and moduli of elasticity of coal, shales, sandstones and siltstones found in coal mine roofs. Although these data are limited, they should be sufficient for general analysis purposes in order to bracket the range of behavior that might be expected. The coal is usually the weakest, and if a coal rider is found in the immediate roof, it can be a source of roof control problems.

Individual specimen properties, however, are inadequate. The properties of a rock mass with joints and faults, subject to environmental effects must be assessed. It appears that a rock quality index can be used as a measure to determine the ranges of rock quality where untensioned fully resin-grouted bolts can, and cannot, be expected to provide roof support. Attempts should be made to classify different coal mine roof sections using such a rock quality index. Methods are available in the literature and have been used for tunnel support design.

5.4 Recommendations

A parameter study should be made, using the finite element method to analyze different roof support mechanisms that might be offered with resin bolting. Bolt data, in situ conditions, mine opening geometry, geology and rock quality should all be parameters in this analysis. The effect of swelling due to moisture needs to be included in this analysis. Experiments should be conducted on resin bolts investigating in particular the effect of hole condition--wet or dry, rough or smooth, and the mechanism of failure at the resin rock interface, especially in soft rock. Different sized holes should be experimented with. A laboratory resin-bolted beam model should be tested to study the effect of shear reinforcement between layers and supplant the limited data available in the literature. More mine investigations need to be made to classify roof conditions, and support requirements.

These recommendations fall within the scope of this research project and are reported on in other volumes.