LABORATORY AND UNDERGROUND INVESTIGATIONS INTO THE STABILITY OF MINE WORKINGS IN GYPSUM

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1. INTRODUCTION.

To design underground openings, the planning engineer should be in possession of all relevant data relating to the behaviour of the surrounding strata under the different loading conditions that might be expected to exist. Where room and pillar methods are practised, this information is needed to determine the size of pillar, the percentage extraction and the roadway dimensions in general. Lack of appreciation or inadequate knowledge of such data can bring danger, and sometimes disaster to the personnel engaged in mining. One of the most tragic disasters in the history of mining occurred at Coalbrook North Colliery in the Orange Free State, South Africa, in 1960. This disaster, which involved the collapse of over one square mile of room and pillar workings, resulted in the loss of 437 lives. The inquiry following this occurrence revealed that no reliable scientific or practical method was available to design workings of the type used at the mine.

There has been a move by many scientists and engineers to establish certain basic principles relating to the large scale behaviour of rock masses. The appreciation of these principles should enable mining engineers in particular, to mine deposits in such a way that the behaviour of the surrounding strata aids, or at least does not hinder, the safe and efficient extraction of the mineral. The research concerned with the establishment of these principles is carried out within the bounds of that branch of engineering known as Rock Mechanics. It is that branch of mechanics concerned with the response of rock to the force fields of its physical environment (1).

The science of rock mechanics is a whole composed of parts taken from a number of different subjects. It brings together and utilises the knowledge and techniques of a number of sciences to aid the study of the behaviour of rock masses. The objectives of the study of rock mechanics are manifold, but to the mining engineer, it is basically to provide a method by which he may design stable mine workings which are also economically viable, both with regard to the amount of mineral extracted and possible damage to the surface environment.

While the formulation of the basic objectives of rock mechanics is a relatively simple matter, its implementation requires long term research. The present state of knowledge of this subject is far from being satisfactory. Whilst great advances have been made in the field of rock mechanics in recent years, its present state of knowledge is such that, except in certain specific cases, it is very difficult to lay down hard and fast design criteria. In the application of what is available, no standard practices or routine procedure exist except on a very limited scale. In these circumstances rock mechanics can best be projected as a valuable tool to be intelligently used, rather than something in a state of perfection capable of solving all existing problems, as it may sometimes be regarded.

When compared with the mechanics of other materials, rock mechanics is an extremely complex subject. The main reason for this is the nature of the rock material itself and its mode of occurrence. Rocks are extremely complex materials, generally made up of many different crystals, and possess characteristic properties like heterogeneity, anistropy and discontinuity. The mechanical behaviour of the rock mass is influenced by the presence of faults, joints and other geological disturbances.

When these factors are considered together with other factors such as the usually irregular shape of the underground excavations, it can be seen that the problems which must be dealt with by the rock mechanics engineer are numerous and highly complex.

To successfully apply the knowledge of rock mechanics to problems in the mining industry it is necessary to determine by accurate measurements, the existing conditions of rocks and the changes they are likely to undergo under contemplated load conditions. This entails the determination of the magnitude of the stresses acting in the rock in the zone of mining, and the understanding and description of the short and long term deformation of the rock mass. It is equally important to obtain information concerning the behaviour and strength of the rock mass under various loading and geometrical configurations. Generally, there are three basic methods by which an investigator attempts to determine this information, 'in situ' observations, laboratory experiments and theoretical studies. These methods may be employed singly or more probably in conjunction with one another, to obtain a complete solution to the problem. Each of these three basic approaches has its relative merits and in particular, its disadvantages.

The 'in situ' investigations are quite often extremely expensive to carry out and usually take a long time to complete. On the other hand they provide data which is factual and relevant to the problem under examination, though care should be taken in applying the results obtained to an apparently similar situation that may exist elsewhere.

Laboratory model work in general lacks flexibility in that it is extremely difficult to take into account all the relevant variables when designing the model. On the other hand, it is possible to carry out a large number of experiments in the laboratory on artificial models that would not be practical to do 'in situ'. Model studies in the laboratory are most useful for the obtaining of an initial appreciation of the problem.

While 'in situ' observations and experiments constitute the main approaches to a rock mechanics problem, a theoretical concept however simple, enables an analysis and assessment to be made of the significance of the results of observations and experiments. The theoretical model is extremely flexible and by using the appropriate mathematical expressions, complex processes of rock behaviour can be described through simplified symbolic models, which embody suitably chosen properties of the rock mass. This is especially true at the present time where electronic digital computers can be used to obtain a number of solutions for a selection of variables. This approach however, depends largely on the selection of the correct model and the determination of the numerical values of the physical constants, which are normally determined in the laboratory.

The conclusion to be reached from this short description of the basic methods of approach seems clear. An attempt should be made to combine the advantages of each approach to acquire both quantitative and qualitative results and thus achieve a complete understanding of mining problems. Whichever investigational techniques are employed the aim of the rock mechanics investigator remains that of attempting to find that solution to his problem which best fulfils its technical requirements.

2. INTRODUCTION TO THE RESEARCH INVESTIGATION.

The general role of the science of rock mechanics in relation to the mining industry has already been defined and an attempt will be made here to interpret the way in which the principles of rock mechanics have been utilised in determining a method for the design of stable mine workings. The evaluation of a practical method of design of the system of mining under investigation in this thesis, is of immense importance to the mining industry and is an important objective of rock mechanics research in general. The basic problems involved in the design of such workings, together with the design procedure adopted, are briefly described in this section. At the end of this section the application of the rock mechanics principles to the specific problems encountered at the underground experimental sties are also briefly discussed.

It is the ultimate aim of the research carried out within the field of rock mechanics to provide the mining engineer with a safe method of design of mine workings. There are many factors which make it extremely difficult to develop a safe and rational method of design of mine workings using the established design techniques of other branches of engineering. These can only be overcome by means of further intensive research to provide more accurate basic data with regard to the behaviour of the rock mass, and therefore, a more precise description of some of the rock mechanics phenomena which influences the design technique.

In the context of this thesis, the term 'design of workings' is taken as referring to the establishment of the limiting dimensions for systems of partial extraction within stratified deposits. In this case it refers to that system of partial extraction known as room (bord) and pillar working, where parts of the mineral seam are left unmined to form a regular pattern of square or rectangular pillars for the purpose of support. The main intention throughout this thesis is the establishment of the optimum dimensions for the pillars themselves and the surrounding rooms. These dimensions are quite simply the width, length and height of the pillars, and the width of the roadways. No attempt is made here to establish the optimum dimensions of the overall working pattern, such as the ideal panel width etc., though this should obviously be considered in a future phase of these investigations.

Room and pillar mining is usually a partial extraction system although in some cases, secondary extraction of the pillars may take place after the initial mining. If the pillars are intended to provide permanent support to the undermined strata, then their ultimate stability is the major factor in the safety of the whole mine. The main objective of the investigations described in this thesis is given to the development of a simple, but reliable method of pillar design where all the pillars are designed so that each is assumed to bear a proportional share of the weight of the overburden. This initial motivation for this research arises from considerations of mine safety. At no stage must it be forgotten however, that the aims of viable mining safety research must always be well balanced. The acceptance of any reccomendations which may arise from a rock mechanics investigation will depend to some extent on their relationship with the practical requirements of mining. The objective must always tend to the design of a mine layout giving the maximum mineral extraction together with the maximum safety. Economic advantage is attained by the maximum extraction but the extraction factor cannot be so high as to prejudice the stability of the mine.

To achieve the best results it is always preferable if circumstances permit, to combine the objectives arising from the problems of safety with those of efficient production. Clearly the correct solution, in existing mines in particular, can only be determined by a thorough examination of the pillar stability.

The roof of the roadway also has to be considered in room and pillar workings since this is normally unsupported. However, whilst a collapse of the immediate roof between the pillars could prove troublesome and provide some danger in the immediate locality of the collapse, it is unlikely to endanger the safety of the whole mine and is therefore, somewhat subordinate to the stability of the pillars. As will be shown in a following section however, a collapse of the roof indirectly has some influence on certain factors affecting the stability of the pillars, and hence the stability of the mine as a whole.

2.1 The Choice of Design Procedure.

The design procedures used by the rock mechanics investigator are influenced to some considerable extent by the particular problem to be investigated. Where an entirely new mine is planned, the complete rock mechanics process of design is utilised. At the present time for example, a new mine is being developed at Boulby on the Yorkshire Moors, where it is proposed to mine potash at a depth of 4000 feet. The rock mechanics survey to determine design parameters for this mine is being carried out in the Department of Mining Engineering of the University of Newcastle upon Tyne, under the supervision of Professor E.L.J. Potts (2). In this case, the main source of information prior to the commencement of mining, comes from rock cores obtained from surface boreholes since no mining of potash and in particular, no mining of any kind at this depth has been undertaken in this country previously. Basically, there are four main processes of design in such an instance as this, all of which are inter-related and carried out in a pre-determined order. It will be as well to briefly outline these design processes to show how design data is obtained and evaluated for a new mine such as at Boulby.

In most cases, the system of mining used at a new mine is determined prior to a rock mechanics survey. At Boulby for instance, a room and pillar mining system was chosen mainly to reduce the possible effects of surface subsidence to a minimum since the Yorkshire Moors in this area constitute a National Park. The first stage in the process of design following this was the selection of the size and shape of the mine openings, in this case, the minimum roadway span and height based on practical mining considerations and the minimum percentage extraction necessary for economic mining. Also to be considered, at this stage, is a detailed study of the geological conditions at the location of the proposed mine workings and the surrounding area. All important features of the strata which can influence the stability of the mine workings are noted and taken into consideration. In addition, the magnitude of the loads which have to be resisted, by the pillar in particular, are calculated.

The second stage consists of the determination of the mechanical and physical properties of the rock material in the vicinity of the proposed working horizon. The amount of rock material available for obtaining these properties is obviously limited. In the case of the Boulby Mine, where the surface boreholes need to be drilled 4000 feet to reach the proposed working horizon, the laboratory testing was restricted because of the limited supply of rock cores. This meant that the testing programme needed to be carefully designed to obtain the maximum amount of information from the minimum number of core samples. The mechanical properties of a rock material are critical factors in the actual design process, and include the long and short term deformation characteristics, and the ultimate strength values of the rock material under different conditions of load. These have some considerable effect on the re-distribution of the stresses due to mining and the failure mechanism of the rock material, the latter an aspect that has not been fully clarified as yet. The physical properties are not so important, but can become significant when the pillars or the roof span, not normally expected to fail due to overloading, become affected by the effects of water and exposure to the mine atmosphere.

The third step in the process of design is to examine by means of a mathematical or scale model consideration, or a combination of both, a first design of working based on the minimum dimensions set in the first stage, and utilising the data collected in these first two phases of design. If such a system of working appears to be feasible in terms of stability and deformation, then further models can be made to examine various possibilities of increasing the total extraction by such design procedures as varying the panel width, the use of barrier pillars, etc. This stage in the design process is especially flexible at the present time when computers are available to obtain a selection of solutions. The most favourable design based on all these considerations can then be chosen.

The fourth and final stage is obviously the acceptance of the proposed design from a point of view of practicability and the 'in-situ' examination of the workings during their primary development. With regard to the latter, such observations will confirm or reject the design and the initiation of re-design may be necessary.

It will be noticed that full use is made of the three basic approaches, laboratory, theoretical and 'in-situ', briefly discussed in the previous section. In this instance, the 'in-situ' approach is used mainly as a check on the design which has been determined entirely by laboratory and theoretical investigations. Salamon however, has developed a design process for room and pillar workings which relies solely on a theoretical analysis of existing workings, and upon their relative stability or instability (3). This approach will be referred to later, at this stage it will suffice to mention that by using advanced statistical techniques, data relating to existing workings may be made to yield information concerning the probable effectiveness and stability of the proposed workings. Such a technique places very little reliance upon data determined in-situ or in the laboratory.

For the successful use of a technique of this type, it is obvious that the more data available to the investigator on which to base his design, then the more reliable it is. This method has the great advantage that in the situations where it is applicable no attempt is made to make what are sometimes dubious extrapolations of laboratory data. The principal disadvantage of this analytical technique is that a large quantity of data is required before it may be legitimately carried out. Ideally, this data should be obtained from mines which are similar in most respects to the proposed workings, but this restricts the application of this design approach to those areas where there has been a history of mining by partial extraction methods, within an approximately similar type of strata. Unfortunately, these are precisely the situations where the maximum amount of design information and working experience is available to the planning engineer, where as in the case of new mines such as Boulby, neither these aids nor the statistical data can exist.

In the case of the experimental sites described in this thesis, very little information based on previous mining experience was available, and hence this statistical technique could not be applied. In the course of this research work, four experimental sites were used. In three cases, access was available to mine workings already developed. The design procedures used therefore, were based on this fact and thus did not follow that outlined previously for the planning of a new mine, though some of these processes are utilised. Taking these factors into consideration, the technique which was used and which would appear to provide the most satisfactory means of approaching the room and pillar design problem at an existing mine, is that of determining the actual state of stability of the workings in question. The research activities have been directed towards providing factual data on the state of stability of the existing workings, mainly by means of underground measurements. The selection of the most indicative measuring techniques and the interpretation of the results obtained are amongst the most important objectives of rock mechanics research.

In such an approach, in-situ measurements must obviously play a major part of the whole investigation, but unfortunately can sometimes prove to be extremely expensive. This is especially so if in-situ attempts are made to determine the actual strength properties of the rock material forming the pillars, as carried out by Bieniawski(4). In view of this, the 'in-situ' measurements were carried out in conjunction with both, laboratory experimental programmes designed to determine the various mechanical properties of the rock material forming the pillars and the surrounding strata, and theoretical considerations which simulated mathematically the underground situation. At one experimental site, the laboratory programme was extended to the construction of a scale model of the actual mine workings. Wherever possible, however, attempts were made to carry out in-situ observations, but there was always a conscious attempt to use the other techniques and concepts of design as the circumstances permitted, to produce satisfactory solutions to the problems.

2.2 The Design Problems at the Experimental Sites.

As mentioned previously, the application of the rock mechanics principles and techniques is entirely dependent upon the characteristics of the individual experimental sites. The rock mechanics problems existing at the various sites are briefly discussed here. The investigational techniques utilised to obtain data which was relevant to the problems are described in a later section.

A total of four experimental sites were used in this investigation. Though each mined the same mineral there was a systematic variation in all other design parameters. The first site at Sherburn-in-Elmet, Yorkshire, was a new gypsum mine, whilst the second and third sites were existing operating units near Kirkby Thore, Westmorland. The fourth site was also an existing operating unit at Brightling, Sussex.

The design problems of the new mine at Sherburn-in-Elmet were complicated by various external factors. Due to the nature of the surrounding strata, the Inspectors of Mines and the Local Planning Authority imposed stringent conditions on the mine operators. These factors and the ways in which they influenced the actual design will be dealt with in a later section. No previous mining had been attempted at all within this particular strata so that the design parameters would have to be established on the basis of laboratory experimentation followed by 'in situ' investigations. Various design factors were established by laboratory and theoretical investigation by Jones (5). These had indicated that the gypsum was continuous and highly homogenous. The author in this thesis has attempted to provide verification of this laboratory work, and to make available factual data by means of 'in situ' measurements during the development of the mine workings.

The problem at Sherburn which it was hoped to solve by the application of rock mechanics techniques, could be summed up as the obtaining of the efficient production of gypsum with an extremely high degree of ground stability around the workings, in order to ensure the safety of both the personnel and the mine itself.

The problem associated with the existing mine at Stamphill in Westmorland was one of attempting to improve the extraction and safety of the mine workings. It was similar to that at Sherburn insofar as the establishment of what were the optimum dimensions for the pillars and roadways was a paramount importance. The site differed from the one at Sherburn in that the overall working pattern was quite irregular as there were no limitations, apart from the normal legal ones, imposed upon the design of the workings. As at Sherburn, laboratory investigations were carried out by Jones (5) and this showed that the gypsum making up the deposit was highly heterogenous. The deposit, therefore, did not lend itself to theoretically based idealised consideration of the type it was envisaged might be used at Sherburn.

The third site was a small operating unit at Newbiggin, situated close to the Stamphill Mine. The mine itself was sited in a higher section of the strata than at Stamphill, fairly close to the surface. The problem here was to examine the possibility of mining a relatively thin gypsum seam in the near neighbourhood of a lower gypsum seam being mined at present. It was expected that the condition of the strata would have some considerable influence on whether or not the economic mining of this seam was possible. The intentions of the investigation undertaken were to assist in the initial development operation by carrying out laboratory tests to determine the characteristics of the strata, and to obtain the most suitable dimensions based on safety considerations, by means of theoretical analysis.

The fourth and final experimental site at Brightling, consisted of a duplex seam operating unit situated on the crest and limbs of an anticline, with the result that parts of the mine were working on rather steep gradients. Due to the combined factors of inclined working and close proximity of the seams, the pillars in the two seams were displaced relative to each other by an arbitrary distance to reduce the possibility of adversely distributed stresses which could possibly attain high cumulative values. The primary concern therefore was the actual stability of this mining system and the resultant provision of a more reliable design procedure on which the relative pillar displacement could be based to ensure complete stability. Unfortunately though 'in situ' measurements carried out either in the existing workings or in future workings would seem to provide the most satisfactory solution to the problem, such an approach was not possible for reasons beyond the control of the investigator. In view of this, an attempt was made to establish the best possible relative pillar layout which would provide a satisfactory state of pillar stability, by means of complementary theoretical and laboratory model studies.

PART 1.

THE STABILITY AND DESIGN OF ROOM AND PILLAR WORKINGS.

1. THE EFFECT OF MINING ON THE SURROUNDING STRATA.

Though some parts of the earth's crust still undergo considerable changes due to tectonic and other forces, it is generally accepted, in the case of stratified deposits, that the strata prior to mining is in a state of equilibrium and that the mineral seam carries the weight of the overlying strata. Mining however, and the resulting creation of underground openings upsets this equilibrium, a new form of equilibrium tending to establish itself resulting in considerable changes in the position of the rock particles and the original stress field. This change in equilibrium can be likened to an energy change in the rock mass brought about by the formation of the excavations. In some cases this energy change can be of some advantage to the mining, such as making the mineral easier to work, but in most respects it results in factors that greatly affect the stability of the actual mine workings. In fact, most strata control problems can be traced back to this energy transfer.

Prior to mining the rock mass possesses a great deal of latent energy which is normally taken as due to gravity, though tectonic and other forces may have played a large part in its concentration. The energy changes which result from the formation of an underground opening have been studied by a number of investigators. (6), (7), (8). They found that the amount of energy released when an opening is formed depends upon the volumetric convergence of the opening, and that the amount of energy stored in stress concentrations around the opening depends upon both the volumetric convergence and the total volume of the rock removed in forming the opening.

The released energy is the quantity most directly related to the failure of the rock around an underground opening, whilst the stored energy determines the stresses around the opening, and therefore affects any other opening made in this stress field. The amount and degree of rock failure around an underground opening therefore can be reduced if some form of support can be used to reduce the volumetric convergence. In some instances this may be achieved by introducing some form of packing or fill into the workings, though the amount of fill introduced should be as near as possible the volume of the mineral extracted to prevent convergence of the opening. It is obviously much better however, to use a system of mining designed to limit the amount of convergence.

When a room and pillar mining system is used, the volume of closure of the workings is usually very small when compared with the total volume of the actual opening. This may explain the fact that this system of mining is relatively free of strata control problems. Since the amount of released energy is small, the latent or stored energy of this system of mining must be very high, with the result that failure of any part of the workings induced by the normal energy transfer, may result in a much larger collapse of the workings due to this latent energy. It is important therefore that the manifestations of this transferred energy should be investigated, since their understanding will help to clarify the question of stability of room and pillar mining systems.

In practical terms, the energy released due to the formation of a room and pillar system of underground openings manifests itself in the form of the stress re-distribution and the movement distribution, and their effects on the pillars, the roof and the floor of the workings. The main factors influencing the magnitude of these changes are the magnitude of the stress prior to mining, the geometry of the excavation, and the mechanical and physical properties of the mineral and surrounding strata. These factors are discussed in some detail in a later section together with their effects on the actual stability of room and pillar mining in general.

2. POSSIBLE CAUSES OF COLLAPSE OF ROOM AND PILLAR WORKINGS.

Room and pillar mining is in widespread use throughout the world and inevitably several major and many minor collapses of this type of working have occurred. However, it was the major disaster at Coalbrook Colliery, South Africa (9), that gave some considerable impetus to the formation of a reliable and practical method of designing workings where the pillars are intended to support the supervening strata completely for an indefinite period of time. In South Africa alone, where room and pillar mining is used a great deal, some 41 uncontrolled collapses have been investigated since 1904 (10). Unfortunately in most countries in the not too distant past, statutory regulations regarding the preparation of mine plans showing the actual working dimensions, were not in force, so that even though many collapses have been recorded, it has been impossible to derive accurate information from many of them.

By far the most common type of uncontrolled collapse in room and pillar mining is that where only one seam is mined in an area. Geological factors such as the proximity of igneous intrusions, faults, slips, washouts, etc., have often been the initiating factor in some collapses in areas which would probably have been stable under normal circumstances. Other factors such as large accumulations of water overlying workings and weathering effects have also been contributory factors in certain collapses. The surface subsidence resulting from the sudden uncontrolled collapse of this type of working is sometimes considerable. This depends mainly in the depth of the workings, but normally a large surface area becomes affected as will be shown later.

From a consideration of the uncontrolled collapses that have occurred in room and pillar mining systems in more modern times, it would appear that the modes of collapse can be sub-divided into three groups :-

1. Pillar failure.
2. Failure of the roof span or the floor of the workings.
3. The effect on the stability of the workings of the behaviour of the strata above or below the working horizon.

At this stage only single seam horizontal or near horizontal workings are considered. The additional problems associated with multiple and inclined seam workings are discussed in a later section. It is proposed here to describe the possible causes of collapse under the three sub-divisions formed above. In order to exemplify each type of collapse, a number of uncontrolled collapses are described with particular relevance being paid to mining conditions and the geometry of the workings. Relatively recent examples of collapse have been used as far as was possible since more reliable data was available from these. A number of the examples of collapse referred to, have occurred in South Africa where room and pillar mining of stratified deposits, coal in particular, is used almost exclusively. It is hoped that these examples will show the effect the various design factors have on the stability of the workings, and that possible lines of research to obtain a better understanding of the mechanism of failure are also indicated.

2.1 Pillar Failure.

This is the most obvious, and often regarded after superficial examination as the only mode of failure occurring in room and pillar mining. Several reasons have been suggested for collapses in this category but in many cases it is fair to say that insufficient pillar strength was the major cause. This was especially so in the case of those collapses occurring in the older mines, where pillar stability suffered in the interest of the amount of mineral extracted. The early concept of attempting to formulate a design criterion was basically very simple. It was suggested that the weight of the overburden must be carried by the pillars. It followed from this that the load on the pillar could be expressed in terms of the depth of working and the percentage of mineral extracted. This calculated pillar load could then be compared with a value of the actual strength of the pillar determined in the laboratory in one of two ways. It could either be determined by experiment where an attempt was made to simulate the actual loading conditions, or by measurement of the compressive strength of the material making up the pillar. These two methods however, often gave widely different pillar strength values resulting in differing mine dimensions.

These design methods took into account most of the factors that directly or indirectly affect the actual stability of the pillars. These factors, which are described in more detail later, were the actual pillar strength, depth of working, amount of mineral extracted and the geometry or width/height ratio of the pillar. In spite of this however, this design approach can be criticised on several accounts. It does not consider the effects of the roadway span and the interrelationship between the mechanical properties of the pillar and the surrounding strata, each of which has some considerable influence on the distribution of the load on the pillar. The variation in strength of a pillar as its geometry or width/height ratio changes can be shown to be related to an increase or decrease in the constraining effects of the roof and floor. The amount of constraint will again depend upon the mechanical behaviour of the rock material making up the pillar and surrounding rocks. Also in some rock materials, notably evaporites, their deformation and strength properties depend upon the length of time they have been exposed to any load.

The effect of these various design factors on the stability of the pillars may be shown by describing briefly a number of cases where room and pillar workings have collapsed, the collapse in each case being attributed to pillar failure. Whenever possible the actual reason for the pillar failure is given. A total of four uncontrolled collapses are described here. The first two examples concern collapses that have occurred in South Africa, the third an unusual situation in the United States of America, and the fourth an example of pillar failure occurring in Cheshire, England.

In 1954 (10) a surface area of approximately 10 acres subsided 3 feet due to the failure of coal pillars in a section of a mine, where mining was taking place at a depth of 220 feet in a coal seam 4' 3" thick, where 12 ft.square pillars were formed by 18 ft.wide rooms. Several weeks prior to this collapse, scaling of the pillars was noticed and fracturing of the floor was imminent. The erection of artificial supports between the pillars was insufficient to prevent the collapse of the whole working section. At the enquiry into this collapse, it was suggested that whilst the influence of a nearby dyke indirectly contributed to the collapse, no account had been taken of a sudden rise in surface topography. This caused a rapid increase in the depth of cover over the whole working area with the result that the pillars were quite simply undersize for the depth being worked.

In 1962 (10), a collapse occurred at a colliery mining at a depth of 300 feet, leaving 19 ft. square pillars and 12 ft. wide rooms. The seam had been secondary extracted by 'top coaling' to an average height of 16 feet exposing a soft friable coal band approximately 1 foot thick within the pillar. The floor of the seam was extremely undulating. The total surface area affected by the collapse extended to 12 acres with a maximum depression of 9 feet in the centre. Closer investigation of this mine revealed other areas in danger of collapse. Ash filling from a nearby power station was introduced to fill these areas with some considerable success. By using this technique, only about 3 ft. remained between the roof and floor, and the pillars became encased in a cementatious material. It was noticed that after the lower third of the 16 feet high pillars became encased, surface spalling virtually ceased and there was a marked quietening in the area. In this case, the collapse of the workings was undoubtedly related to the increase of pillar height affecting the friable coal band in the pillar which together with the nature of the floor, resulted in the formation of unstable pillars. The ashfill effectively reduced the width/height ratio and applied some constraining effect to the pillars, resulting in a much more stable layout.

An unusual example of uncontrolled collapses were those occurring at a copper shale mine in Michigan, U.S.A. (11). A number of collapses were recorded at this mine, being attributed to either pillar or roof failures. Only those collapses caused by pillar failure are described here, those due to roof failure are described in the following section 2.2.

The original design of the mine openings and pillars at the mine were based upon the best information then available in 1955, most of it supplied by the United States Bureau of Mines. It was assumed that the principal load is due to the weight of overburden, and that horizontal stresses are equal to one third of the vertical value. The pillar dimensions were then calculated under an assumption that the weight of the overlying strata would be concentrated on the pillars. The calculated pillar load was then compared with the average compressive strength of the rock, and safety factors of between 4 and 6 were considered satisfactory. Up to a depth of 600 feet, the pillars were formed 20 feet square with normally 28 feet wide rooms, though sometimes 32 feet wide rooms were formed. The pillar heights were either between 8 and 10 feet or between 20 and 22 feet. Four multiple pillar failures have occurred at this mine between 1956 - 1966, one expected, three unexpected. The expected pillar failure followed some experimental pillar robbing, where pillars were practically destroyed by blasting. The second failure occurred in an area where the pillars formed were tall, thin and jointed. Although there was adequate warning of the third failure, it was not understood. About eight years after mining, a group of pillars began to split and spall, eventually collapsing and the ground subsided to the surface. The fourth failure was close to the third, and the conditions were similar. These pillars were between 8 and 10 feet high, and together with the roof, retained their good appearance for years following their formation.

These pillar failures represent only a small proportion of the area of the mine, but they are significant because they show that factors other than depth, extraction ratio, and the compressive strength of small specimens have to be considered in pillar design.

A factor that has been shown to have some considerable influence on the stability of pillars made up of certain minerals, is the time dependent deformation of the mineral. This phenomena is demonstrated in evaporites in particular, but has been shown to be present in other minerals, notably coal. The most complete record of pillar collapse where this phenomena undoubtedly played an important part, was that collapse which occurred at the Adelaide Salt Mine, Cheshire (12). The mine workings were approximately 35 acres in area at the time of the collapse. It is interesting to note that the first occurrences of failure at the mine took place at the centre section of the workings, when a pillar failed completely, to be followed by the adjacent pillars. These pillars had become very badly fractured and in some instances, severely reduced in size due to spalling of the pillar sides. Records of the roof-floor convergence at points near the centre of the mine workings, were taken over a period of 12 years prior to the collapse, in which time, a total convergence of 3 feet had been recorded. The rate of convergence increased in such an alarming manner that all personnel were eventually withdrawn and the mine declared abandoned. No record of convergence was taken after December 1927 and the exact date of the collapse is uncertain.

It will be noticed that in each collapse described, ample warning of possible pillar failure was given by the pillars spalling, so that when eventually complete pillar failure took place it was not totally unexpected. These examples, however, reveal that even when rock mechanics principles are utilised in the initial design of the mine workings, as in the collapse in the copper mine in Michigan, pillar failure still occurred. It would appear therefore, that in order to acquire a better understanding of the mechanisms associated with pillar failure, further research needs to be carried out, not only to obtain more reliable initial design data, but also of the form of control measurements during the actual mining operation.

A factor that appears to have some considerable influence on the stability of mine pillars is the actual distribution of the load taking into account the effect of the width/height ratio of the pillar and the respective mechanical properties of the pillar, the roof and the floor. This effect again may be coupled with the constraint offered to the pillar by the roof and the floor of the workings. These will be discussed in more detail in Section 3.

2.2 Failure of the Roof or Floor of the Workings.

In some cases the pillars are designed so that their strength may be quite adequate, but either the roof or floor strata cannot support the loads they are called upon to accept. This type of failure may take two forms. It appears either as a fall of the roof or as an excessive floor heave. Both in their own way can result in considerable damage to a small section of the mine, but their main effect lies in their indirect influence on the stability of the pillars themselves and therefore, on the overall stability of a much wider area. The mechanism of their effect on the mine stability can be explained in simple terms.

A localised collapse of the roof increases the height of the workings resulting in a change in the geometry of the pillars themselves. As will be shown later, and has been indicated to some extent in the previous section on pillar failure, this variation in the width/height ratio of the pillars can materially affect the actual pillar strength. Furthermore it reduces the lateral constraint provided by the roof which again results in a decrease in strength of the pillars. Failure of the roof is more common in workings where the roof is strongly laminated and the room span is too large in comparison to the lamination thickness and its inherent strength. This type of collapse is more a result of over-designed room width than insufficient pillar size and for this reason is not a collapse due to pillar failure.

Excessive movement or heave of the floor of the workings not only forms unfavourable conditions for the mine machinery, but also results in an unstable foundation for the pillars and as before, reduces the lateral constraint on the pillar provided by the floor. Such movement is caused by the transmission of the load by the pillar to the floor resulting in the formation of high pressure peaks under the pillars, in the neighbourhood of stress relieved rock beneath the roadways. This results in the flow of some softer beds towards the roadway. This is quite a common occurrence in coal measure strata and is especially prevalent where the floor strata contains one or more beds of a highly plastic nature. It is especially noticeable under wet floor conditions. It should be mentioned however, that the stability of the roof is more important than the movement of the floor, both with regard to safety and economy of mining, since the latter is normally confined to more isolated cases where geologically floor conditions are especially difficult.

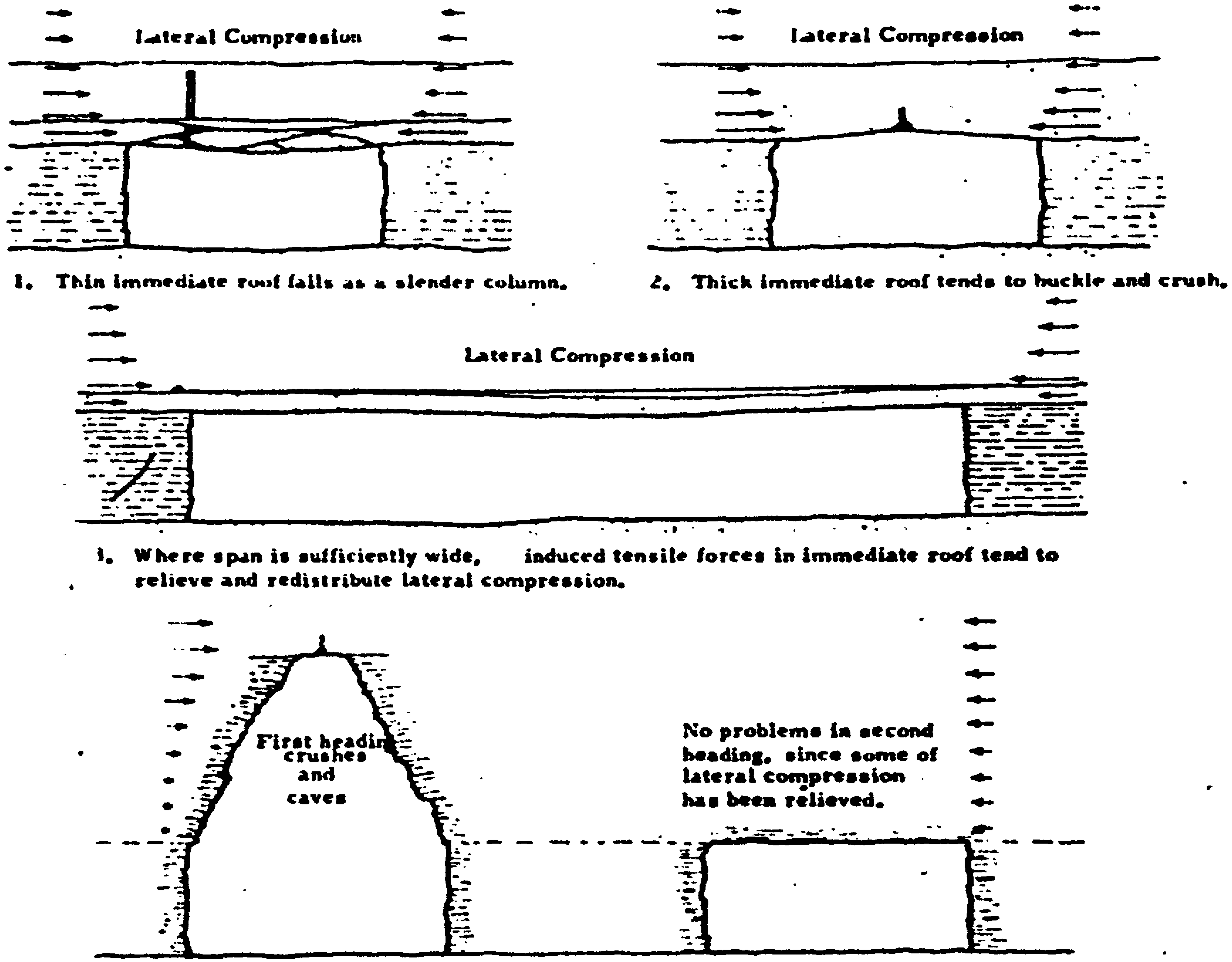
Examples of this type of failure are described here being obtained from three mines . Two concern roof failure in South Africa and the United States of America respectively and the third, a unique example of floor heave and its effects, in a coal mine in the North of England.

Collapses between pillars or failure of the roof span, is particularly common in the more shallow mines in South Africa where room widths are quite often too great to span the strata between the pillars. This is found mainly in the sub-outcrop zones of coal mines where the overlying strata is invariably of a poor nature due to weathering. The resulting surface subsidence is rarely extensive, and more often localised to the collapse of an underground intersection with the appearance of a sink-hole on the surface. One particular colliery (10) has experienced some 60 such collapses in the last 50 years. Further away from the sub-outcrop at a depth of 70 feet at the same colliery, mining to 20 feet wide rooms with 20 feet square pillars and a working height of 10 feet, several collapses have occurred. Subsequent investigations after each collapse revealed that the pillars showed no sign of scaling and that the roadways had in fact, collapsed between the pillars. A contributory factor on several occasions was found to be heavy rainfalls, especially where marked variations in the composition of the overburden existed.

Several unusual examples of roof failure in room and pillar workings occurred at the White Pine Copper Mine in Michigan between 1956 - 1966 (11). This mine was also used in the previous section as an example of pillar failure. Roof spans were calculated as if the roof was a gravity-loaded beam or plate with a thickness equal to the length of the roof bolts, or to the thickness of the immediate roof layer, whichever was greatest. The calculated tensile stress in the lower fibres was compared with the laboratory tensile strength and then applying liberal safety factors. Spans were limited to 28 feet or 32 feet which were originally considered to be very conservative. However, local roof failures have not been uncommon in spite of the safety factors used.

One type of roof failure was attributed to the plugging of water bearing surface drill holes intersected by the mine workings. Hydrostatic pressure built up in the holes, and along bedding planes, causing local collapse of the roof. A second type of failure occurred where mine openings intersected and ran parallel to faults and joints, even though roof bolting was used and allowances made in the basic design. A third type of failure, a low angle fracture, was recognised but not understood for a long time. Investigations revealed that these roof failures had several features in common; the failures were more prevalent in certain directions, they occurred days or even months after the roof was exposed, broken edges at the fracture were thrust over or under each other, roof bolt holes were offset laterally, failure was usually violent and often confined to the lower roof horizon leaving upper thicker beds unaffected. A fourth and more subtle type of failure was observed under conditions similar to those in the third type described above, but in places where the immediate roof was thick and massive. Complete failure was preceded by the development in the roof of a grey, crushed zone a few inches wide, running down the centre of the roadway. The crushing effect worked upwards, a layer at a time, until a high, narrow arch was formed. This arch is stable and little or no trouble was experienced in other headings driven parallel and close to the original.

A longwall mining system was attempted and a strange contrast appeared in that the wider roof span was more stable than the narrow span. It was not until a rock mechanics instrumentation scheme was installed that reasons were provided to explain this phenomena. An



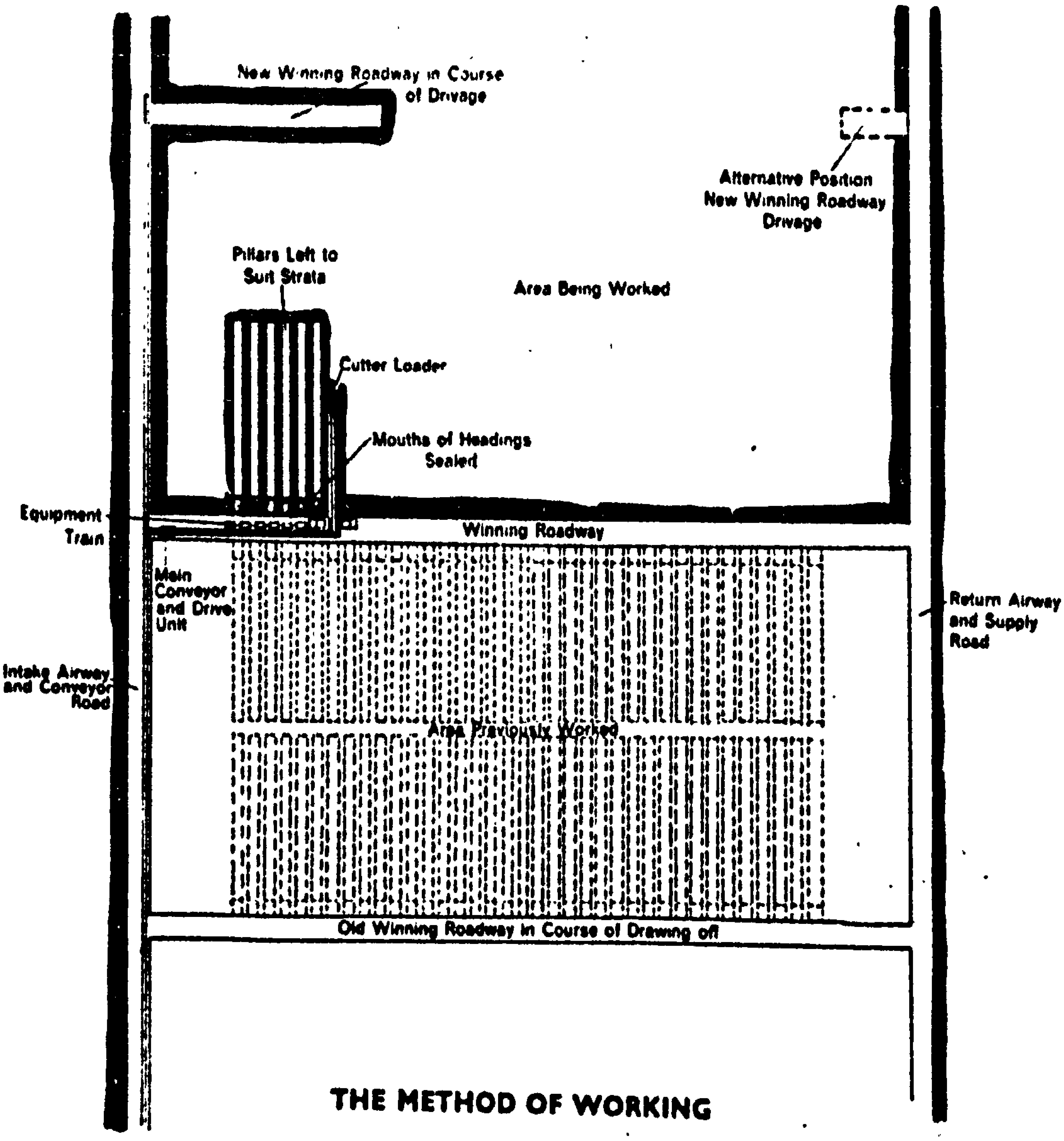
—Apparent Roof Behaviour in a Lateral Stress Field.

Fig. 1

attempt was made to define the existing stress field around the workings. In every case, the major stress field was found to be nearly horizontal or parallel to the bedding, generally parallel to faults and joints. Vertical stress magnitudes were as expected, but lateral stresses were found to be more than three times the vertical value. This readily explained the several failures which had occurred, as shown in the various diagrams in Fig. 1. The stress field was primarily lateral. Where the roof beam was thin, it buckled downwards, and where it was thick, upward buckling was produced and the roof crushed along the centre line. Where some relief was afforded by a very wide span, problems were alleviated. Though this explained the roof failures it has not as yet made clear the reasons for the pillar failures described in section 2.1, but investigations are continuing.

The third and final example that has been grouped under this type of failure is taken from an experimental system of partial extraction of a thin coal seam, carried out at the Fishburn Colliery, Co.Durham in 1963-1965 (13). A remotely controlled coal cutter/loading machine was used to form a number of parallel galleries of varying length in the coal seam, from a central roadway at a depth of approximately 600 feet. Each gallery was 6 feet 3 inches wide and normally 2 feet 10 inches high and separated by a continuous intervening pillar of varying width, which served as a temporary support. This working system is shown in Fig. 2.

It was intended that the pillar widths would be varied to determine the most stable layout. In fact the drivage lengths achieved by 141 galleries, varied between 30 and 300 feet, with pillar widths from 1 ft.8 ins.to 13 ft. 8 ins. The presence of floor heave in the galleries a short time after mining, adversely affected the mining operation, and approximately 6% of the total stoppages were found to be directly due to floor heave. This phenomena occurred in most of the galleries, but its onset was found to take place at variable times from 2 to 50 hours following completion of each gallery and therefore did not always affect the mining operation. In some cases,



THE METHOD OF WORKING

Fig. 2

considerable spalling of the pillars took place but whereas side deterioration could be tolerated to some extent, floor heave could not.

An experiment was therefore carried out to determine the relationship between pillar width and floor heave. It was found that pillars above 4 feet in thickness induced floor heave in the adjacent galleries. Pillars below 3 feet in thickness almost entirely eliminated floor heave by delaying its onset, but the pillars themselves rapidly deteriorated and became unsafe, also causing the immediate roof to fracture. Pillars between 3 and 4 feet in thickness exhibited very little spalling and the onset of floor heave was reasonably late enough to allow mining and ancillary operations to be completed. In this case however, it transpired that this range of pillar widths was extremely critical at this particular mining area, because of the nature of the rock materials associated with the coal seam, and could not in fact be tolerated. This mining system was subsequently abandoned.

These examples indicate that a clearer understanding must be obtained concerning the inter-action of the roof and floor of the workings with the pillars. In the case of the copper mine at Michigan, long-established design criteria were used originally but results were not quite as predicted. The real behaviour of the mine roof in this instance was simply a matter for speculation. A rock mechanics approach with visual observations and instrumentation, indicated that the roof was not only a gravity loaded beam, but a gravity loaded beam with even higher end loads which vary in magnitude and direction. In this case a rather unusual concept of increasing spans for greater stability has been tested with some considerable beneficiary effects.

The experiments carried out at the Fishburn Colliery indicated that the interaction of the pillars and floor could be even more important in some cases, than that between the roof and the pillars.

In this instance however, even though definite design criteria were established concerning pillar width, to counteract the floor heave phenomena, the actual method of mining used proved to be unsatisfactory under the conditions present.

It would appear therefore, that more precise design data must be obtained concerning the stress and movement conditions in the roof and floor of room and pillar mining systems in terms of the amount of mineral extracted, and the room span. This should be carried out, taking into consideration the degree of lamination of the strata, the strength of the surrounding strata and also the effects of water on the stability of the roof and floor.

2.3 The Effect on the Stability of the Workings of the Behaviour of the Strata Above or Below the Working Horizon.

It was originally believed that strata movement or subsidence did not exist above room and pillar workings. This is quite understandable since the surface structure damage due to subsidence above room and pillar workings in Great Britain and elsewhere is negligible. The reason for this lack of the necessary appreciation of the forces governing strata movement, due to room and pillar mining, can be understood and explained by the low magnitude of the measured displacements of the rock mass above such workings obtained by various investigators, in South African Coal Mines in particular (10) (14) (15). These investigations repudiated such views expressed by many people and confirmed that vertical lowering of the roof strata takes place above room and pillar districts. This vertical movement is partly the result of the yield of the pillar due to the transference of the load and the loss of lateral support, and partly the result of the vertical compression of the rocks above and beneath the pillars, due again to the increase in load on the pillars.

Under normal conditions, this vertical lowering or subsidence of the strata is initiated at the seam and develops smoothly to the ground surface. This procedure requires a gradual deformation of all rock layers between the seam and the surface. However, since the strata is generally made up of a number of beds of varying thickness and mechanical properties, some beds will resist this gradual deformation more than others. If an exceptionally strong, rigid bed is present, this could break the continuous development of the subsidence and consequently behave as a bridge with its abutments either on the ribsides or on the stabilising pillars of mined panels. In such circumstances the full weight of the overlying strata would not be resting on the pillars. Any such bed however, can only have a limited bridging capacity dependent mainly upon its span. Consequently it is always liable to collapse if the span becomes over-critical. A collapse of such a bed would exert a shock loading on the developed pillars resulting in the possible collapse of a large pillared area. It has been shown (16) that by increasing the rate of loading from 1 p.s.i. to 1000 p.s.i./sec. the failure strength of a specimen of coal was decreased by 42%. This type of collapse may take place suddenly without any prior warning, with disastrous results. Also, if such a collapse occurred, a chain reaction could occur due to the presence of the strong, rigid bed above the workings, which would maintain a peak abutment pressure in front of the line of collapsing pillars resulting in a very much wider area of collapse.

The problems connected with this type of failure are extremely complex. Basically, they can only be investigated by means of exploratory boreholes from which some considerable information may be obtained. Such boreholes permit the detection of the existence of potentially dangerous strata and determine their thickness and distance from the mineral seam, and if possible, make available cores of the relevant strata for the determination of the mechanical properties of the different layers. These factors form the criteria for any such abnormal behaviour of the strata. Field measurements by means of instrumented boreholes could possibly indicate the presence of any separation of the strata, since a major bed separation below a strong, rigid bed is not an unlikely possibility in a case where the thickness of the strata between this bed and the workings is considerable.

Such measurements could indicate the magnitude of the strains at any horizon above, or even below, the seam. The evolution of a system of mining which would not overstrain these rigid layers may then possibly be developed from such information.

As well as the strata above the workings, the strata below the working horizon also needs to be considered in the stability of room and pillar workings in particular. This has been shown to be especially so in the South African Coalfields where the presence of cavernous dolomite, both above and below the coal seams, is quite common. Such dolomites become cavernous due to leaching by acid water and there are recorded instances (17) where solution of the rock in Coal Measures at depth led in places to a collapse into such caverns. The presence of solution cavities in rock salt in the Cheshire saltfields (18) have also contributed to the failure of room and pillar workings. The detection and exact location of such caverns, the possible effect that drainage or pumping of underground water and consequent lowering of the water table will have on their support, and the possibilities of devising ways and means of providing or introducing some form of artificial support that will prevent their collapse, all present difficult problems.

Neither form of failure described here is particularly common, though it can be disastrous when it occurs. The failure of strata beneath the working horizon is particularly infrequent and no further mention of such phenomena is described here. Failure of strata above the working horizon however, is relatively a much more likely occurrence and it has been suggested that it has contributed to the collapse of workings in a number of cases in South Africa, where dolerite frequently forms a rigid bed in the strata. In the case of the disastrous collapse that occurred at the Coalbrook Mine, in 1960 (19) early consideration was given to the possibility that a thick, strong bed or band of competent strata had acted as a 'bridge', spanning a relatively large area, which presented the pillars in the area not only from taking the full load of the overlying strata, but also from showing any of the usual signs of undue pressure or weighting. The character of the strata in the area seemed to lend support to this theory and at first it appeared to be a plausible explanation. Later investigations however, revealed other features, associated with the structural geology of the area, that offered a more feasible and acceptable explanation, but it was suggested that the rigid bed played a secondary role by spreading the collapse over a wide area.

There is a fairly recent instance where the collapse of the workings has been related to the presence of rigid beds above the developed workings. This concerns a collapse in 1958 at a large potash mine in the Werra area of East Germany (20). The collapse was detected by seismic stations 2000 Kms. away. Approximately one eighth of the entire workings was affected in two potash seams, 60 m. apart at depths between 480 - 600 m. Three types of pillar damage were easily distinguished. There was pronounced scaling at the faces without these being visibly thrust into the mining cavity. This was confined mainly to the upper seam. The second type was where the faces were displaced up to 6 m. into the workings. In this case the pillars appeared superficially undamaged but were considerably crushed internally. In the third type, the pillars literally burst open and in some cases nothing but heaps of debris remained. These latter two types of failure were predominant in the lower seam. Pronounced roof falls were widespread in the zones of total failure of the pillars, the roof being much more affected than the floor.

In both seams, the greatest destruction occurred in white carnallite, which was known to have some previous history of violent failure. Evidently, the carnallites, although their failure strength is lower than that of other salt rocks, are able under certain loading conditions (particularly at high loading rates) to store large quantities of energy for a short time, and to release it abruptly on failing. To explain the failure, a change in load must be assumed, which was confirmed by measurement of pillar deformation and surface subsidence prior to failure. The pillars showed no pressure effects microscopically and measurements indicated only slight deformations, which were not in accordance with the load conditions corresponding to the depth, extraction and degree of yield of the pillar material. This suggested that even in such a large working area the pillars were underloaded. No surface subsidence was recorded prior to the collapse whilst upper seam convergence outside the failure area over a period of 25 years was between 5 and 31 mm., and 19 and 96 mm. in the later failure area.

These small deformations and the delay in subsidence were brought about by an excessive rigidity of the overlying strata which only showed subsidence where its bending strength was exceeded. The full loading of the pillars only commences at this point leading, particularly in the case of carnallite pillars, to brittle failure owing to excessive loading. In this case, the only possible rigid strata was the massive rock salt bed overlying the upper potash seam. Thus the decisive factor in this collapse was considered to be the change in the load due to the bending strength of the thick rock salt strata being exceeded. The complete failure of the pillars were not regarded as a cause but only as an effect.

This incident constitutes a text-book case, showing that even overlying strata of relatively low strength such as rock salt, if of sufficient thickness and it is associated with pillar materials of a brittle nature, may give rise to a violent failure unless extraction is planned to take these characteristics into account.

The uncontrolled collapses described in this section under the three main types of failure, have indicated the need for further research into the stability of room and pillar workings. In each case, the influence of various design factors on the stability of the workings has been shown, and possible lines of research are indicated. In this thesis, an attempt has been made, not only to solve the rock mechanics problems at the experimental sites, but to apply the information obtained from these investigations to the stability of room and pillar workings in general. The conclusions reached concerning this are described in Part 3 of this thesis.

3. THE DESIGN FACTORS IN ROOM AND PILLAR WORKING.

The supporting structures in a partial extraction system of mining consists of pillars of the mineral which are suitably designed to control the stability of the mine. In order to decide on the suitable dimensions for such pillars, it is necessary to have a knowledge of the load that is applied to the pillars and the actual bearing capacity of different sizes of pillars. A safety factor can then be determined and judgement exercised on whether the chosen pillar size is adequate or not. This applies also to the dimensions of the roadways. The problems inherent in the design of such workings have always been difficult to solve, as has been shown in the previous section, but it is hoped that in the course of the many investigations that have been carried out, sufficient data can be collected to give an indication of the normal behaviour of the pillars and the surrounding strata. This will enable attempts to be made to select appropriate parameters which are representative of the behaviour of the pillars in particular, and the surrounding strata. The measurement of the changes in these parameters may eventually be used to determine the state of stability of the workings.

The determination of the design data and its application, will be discussed under the following headings :-

(i) The magnitude and distribution of the load.

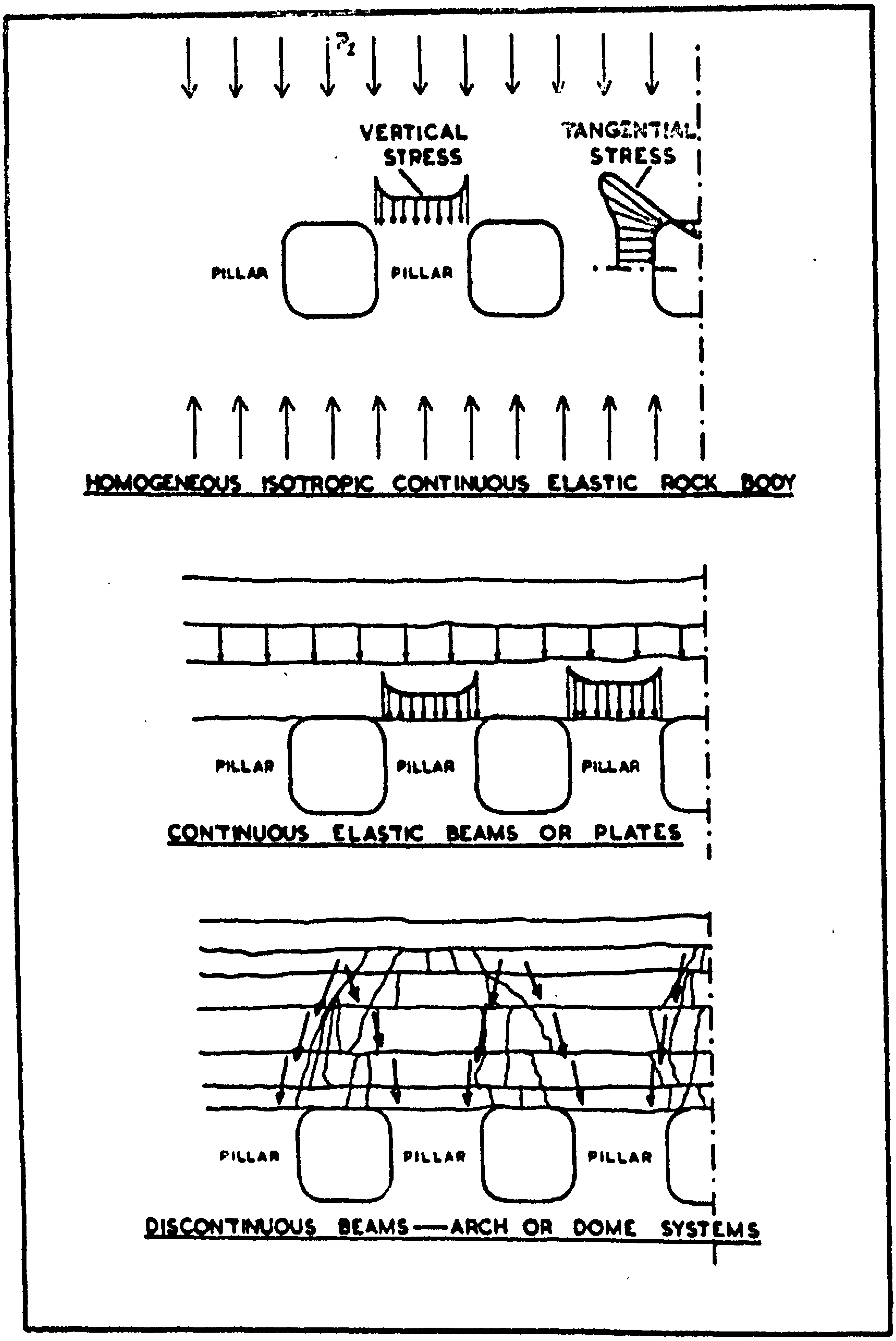
(ii) The strength of the pillars.

(iii) The roof span.

(iv) The application of the data to the design of room and pillar workings.

3.1 The Magnitude and Distribution of the Load.

The manner in which the re-distribution of stress takes place around mining excavations has been discussed by many investigators, but three basic schools of thought have emerged, namely, the continuous elastic body theory, the beam theory, and the dome or arch



P_z
VERTICAL STRESS
TANGENTIAL STRESS
PILLAR
PILLAR
HOMOGENEOUS ISOTROPIC CONTINUOUS ELASTIC ROCK BODY
PILLAR
PILLAR
PILLAR
CONTINUOUS ELASTIC BEAMS OR PLATES
PILLAR
PILLAR
PILLAR
DISCONTINUOUS BEAMS—ARCH OR DOME SYSTEMS

Fig. 3

theory. These are shown diagrammatically in Fig.3. Which of them, or which conbination of them is applicable to any particular condition is extremely difficult to decide. This is hardly surprising when it is observed that the beam theory can be considered as a special case of the continuous elastic body theory, and the arch theory can be related to the beam theory by using discontinuous beams. These theories have been described in some detail elsewhere (21),and it is felt that to discuss these theories further falls outside the scope of this thesis.

Most analyses related to strata loading usually assume that the load on any element of strata is proportional to the thickness of the overlying strata, according to the formula :-

$$Po = \gamma H \tag{1}$$

where Po = vertical pressure.

H = depth below surface.

γ = average weight per unit volume of the overburden.

It should be noted that besides the vertical pressure, lateral pressure also exists, the magnitude of which may vary from zero to a value greater than the vertical load, depending upon the local conditions (22) (23). The evaluation of the lateral stress has given rise to a great deal of controversy, and since the determination of the primitive stress components can be carried out only in certain special cases, the designer is often faced with the task of choosing a lateral primitive stress value between wide limits.

It should also be mentioned that the vertical pressure is not necessarily evenly distributed over the mine workings. Certain regions underground (24) may be partially relieved from vertical pressure as a result of 'bridging' of superincumbent strata, while others may be subjected to increased pressure due to the abutments produced by the 'bridging' effect. Moreover, geological anomalies contribute towards non-isotropy and non-homogeneity, and may also be associated with additional inherent stress.

Where a room and pillar system of mining has been developed over a fairly extensive area, the pressure on the pillars becomes greater than that given by equation (1). The assumption is made that each pillar supports a column of rock over an area which is the sum of the cross-sectional area of the pillar, plus a portion of the room area, the latter being equally shared by all neighbouring pillars, as illustrated in Fig.4. By making a second assumption, namely that the load exerted on the pillar is vertical only and is uniformly distributed over the whole pillar cross-section, the following formula is obtained, for a square pillar :-

$$P = \frac{Po\ (W + B)^2}{W^2} \qquad (2)$$

where P = load on the pillar.

Po = load due to overburden.

W = width of the pillar.

B = width of the roadway

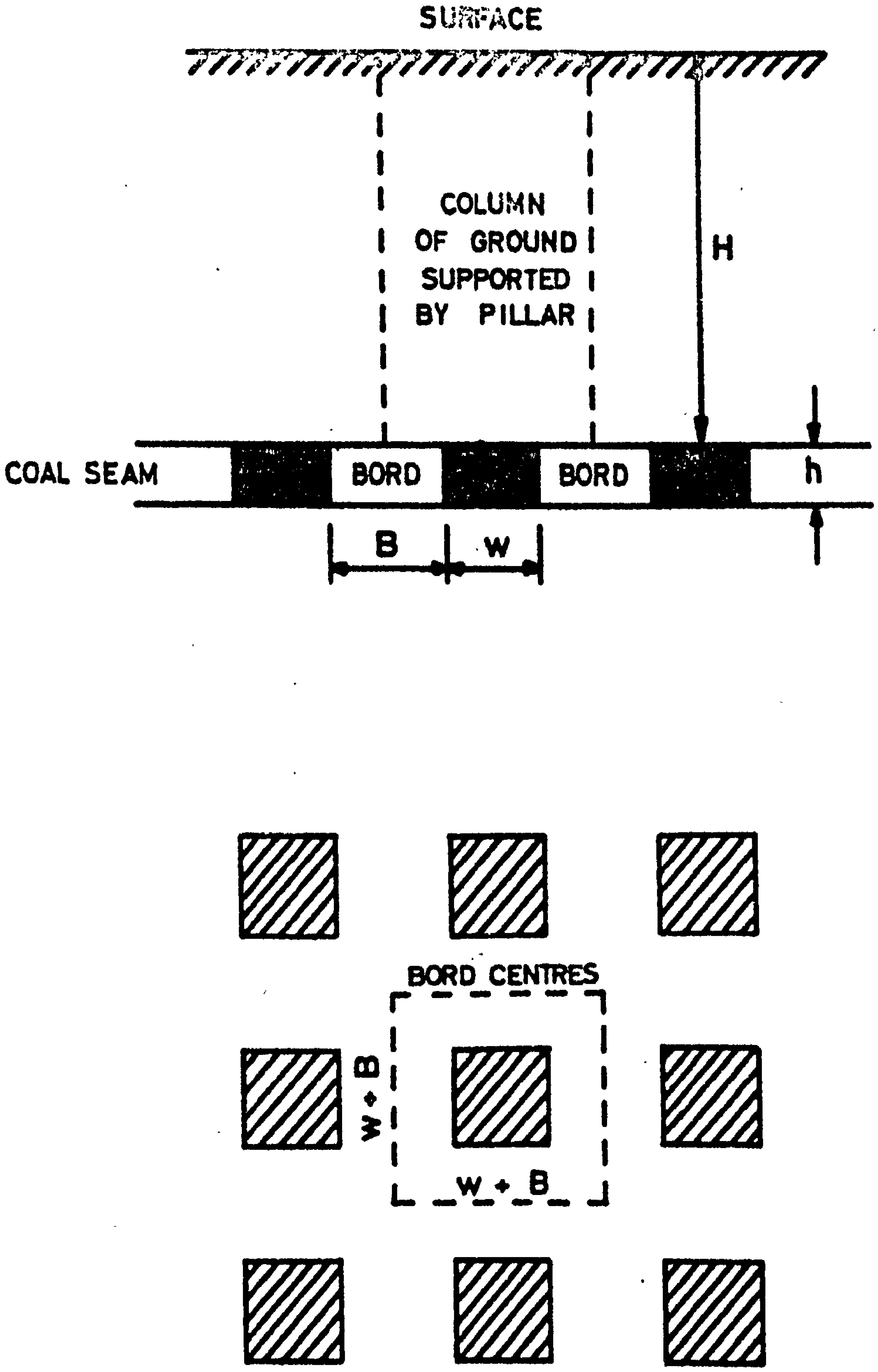
Substituting equation (1) into equation (2) gives

$$P = \gamma H \ \frac{(W + B)^2}{W^2} \qquad (3)$$

If the term 'percentage extraction', R, is introduced, which is defined as the ratio of the mined out area to the total area, that is :-

$$R = \frac{(W + B)^2 - W^2}{(W + B)^2}$$

$$= 1 - \left(\frac{W}{W + B}\right)^2 \qquad (4)$$



Column of ground assumed to be supported by one pillar

Fig. 4

then equation (3) may be re-written as

$$P = \gamma H \frac{1}{1 - R} \qquad (5)$$

It must be emphasised that these equations are based on certain simplifying assumptions, and must be treated as approximate formulae only, and are only really suitable for regular room and pillar workings. When pillars are of different shapes and sizes and spaced at irregular intervals, the calculation becomes more complex. Sen (12) and Dreyer (25) have shown by laboratory experiments that the greater the area/perimeter ratio of a particular specimen, the greater the load it can support. Hence a circular pillar is able to accomodate a greater load than an adjacent square pillar of similar cross-section. Secondly, for two adjacent pillars of similar shapes but having a difference in areas, the room span supported by the large pillar is likely to be greater than that apportioned to the smaller pillar. Accurate graphical methods of load assessment have been developed by Hast (22) and Sen (12) for use in extraction areas where irregular pillars are formed, in each case the roof span supported by a pillar, being represented by a proportionate polygon surrounding the pillar.

A more precise calculation for determining the pillar load, has been carried out by Coates (26). He proposed a general, comprehensive theory for the prediction of pillar loads based on the statically indeterminate net deflection of the pillars, this deflection being used as a measure of the increase in pillar stress resulting from mining. He obtained an equation containing 15 variables, for the determination of the pillar load in elastic ground. It is yet to be proved however, that this equation can be applied to practical design with the same amount of confidence as equation (5).

When a room and pillar area is only of limited extent, the loads exerted on the pillars are unlikely to be of a magnitude approaching the fully distributed cover load as calculated from equations (3) and (5). The probable form of transfer and accumulation of load

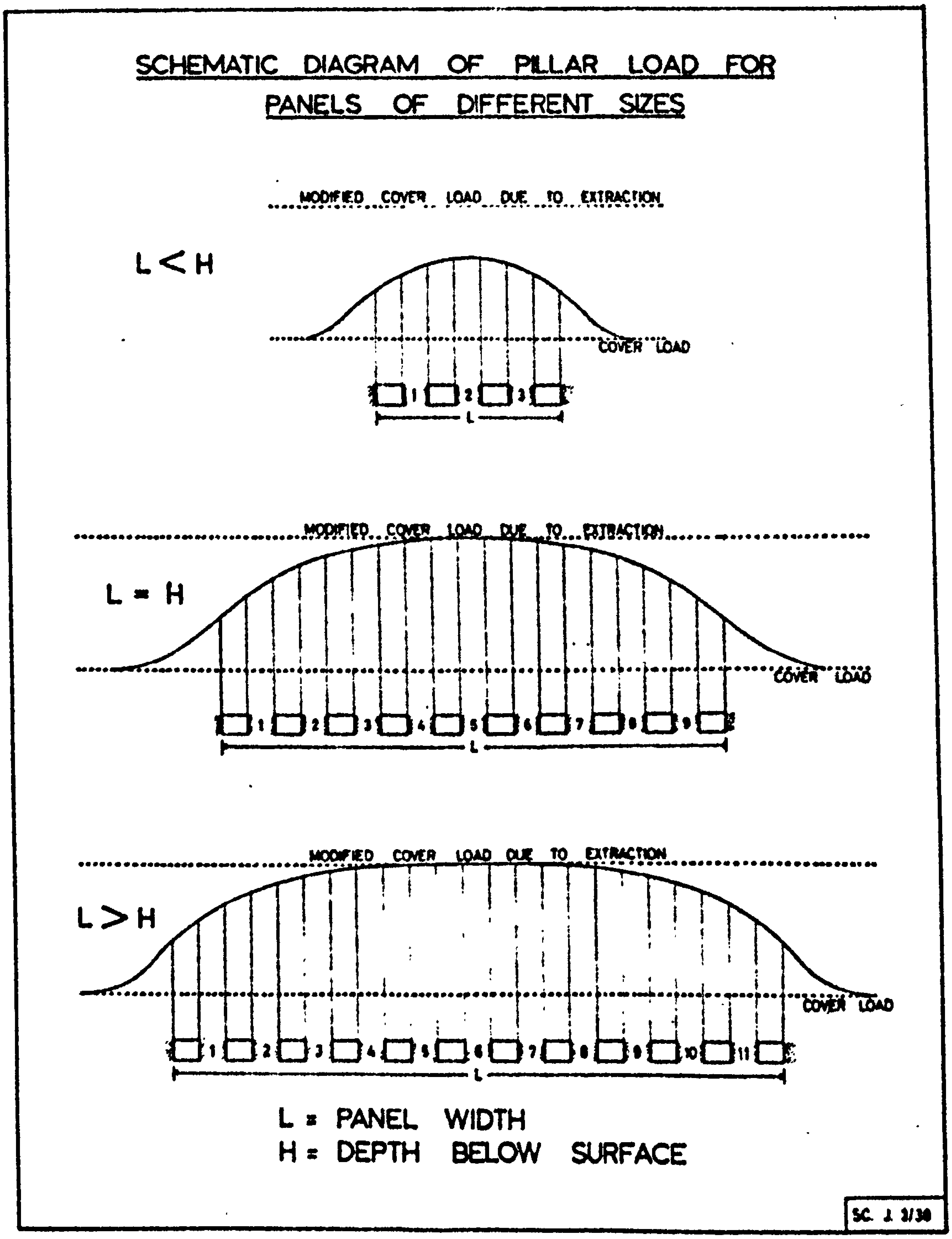

SCHEMATIC DIAGRAM OF PILLAR LOAD FOR PANELS OF DIFFERENT SIZES
MODIFIED COVER LOAD DUE TO EXTRACTION
L < H
COVER LOAD
L
MODIFIED COVER LOAD DUE TO EXTRACTION
L = H
COVER LOAD
L
MODIFIED COVER LOAD DUE TO EXTRACTION
L > H
COVER LOAD
L
L = PANEL WIDTH
H = DEPTH BELOW SURFACE
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Fig. 5

on the pillars as the excavation is widened is shown in Fig.5 It is usually considered that when the width of the excavation is the same as the depth, then the fully re-distributed cover load is exerted upon the centre pillar. As the width increases still further, an increasing number of pillars in the central zone of the excavation are subjected to the fully re-distributed cover load. More recently however, field investigations carried out into the subsidence effect above room and pillar workings in South Africa, by Potts and Szeki (14) have indicated that the maximum surface subsidence takes place when the width of the excavation reaches a distance equal to approximately 1.4 times the depth of working.

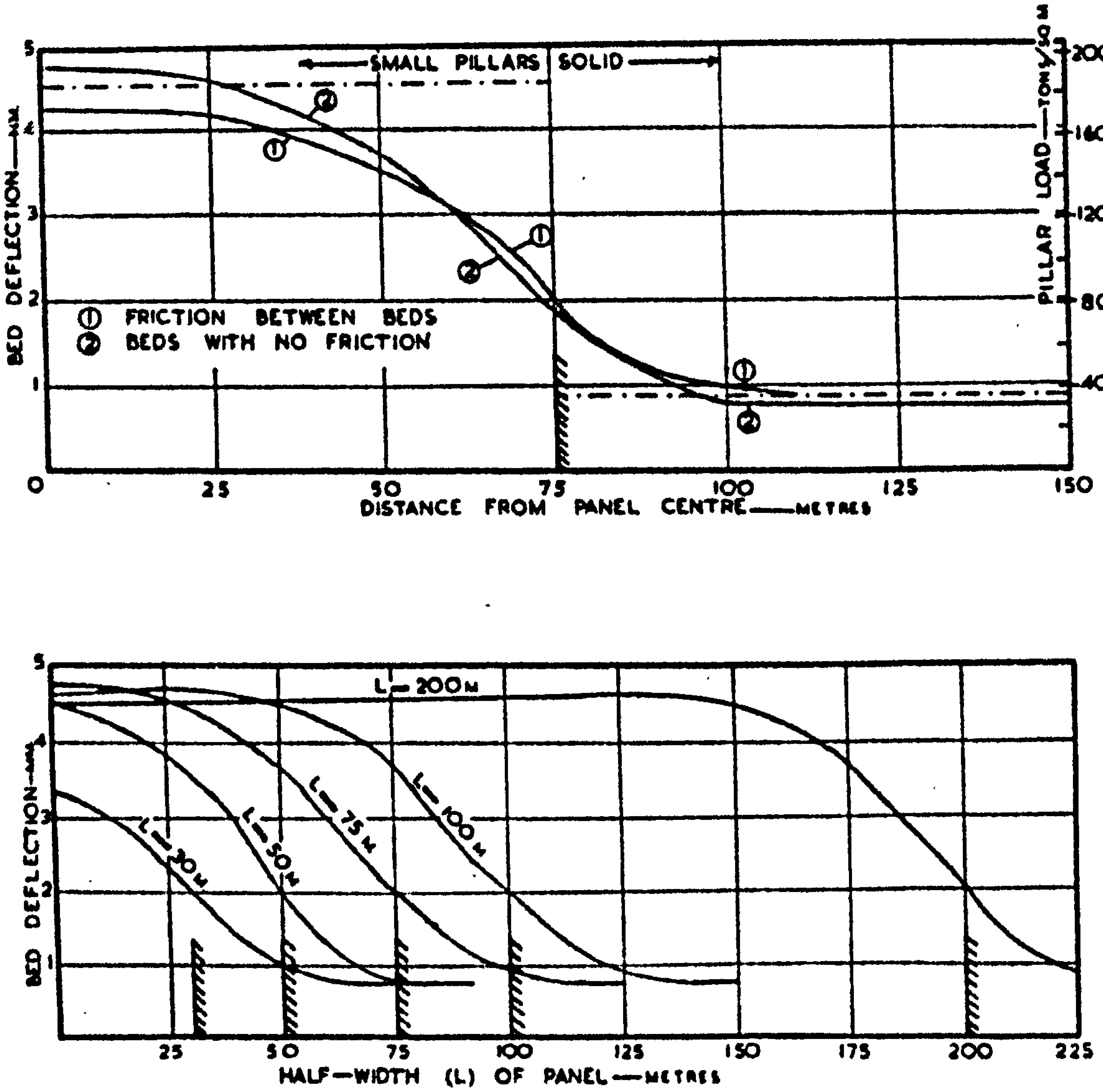
A brief literature survey on the distribution of the load onto a room and pillar mining district, represented in tabulated form, is given in Table 1. A second literature survey on the distribution of the load in the pillars themselves is given in Table 2. In this case, some reference is made to the attempts that have been carried out by a number of investigators, to measure the actual load on the pillars. This is by no means a comprehensive review, this being adequately covered elsewhere, by Singhal (45) in particular. It will be noticed that some references are referred to in both Tables. This is inevitable since the distribution and magnitude of the load in the pillar is very closely related to the distribution of the load above the working area.

An important factor affecting the stress distribution around mine workings is the behaviour of adjacent rock materials with respect to each other when subjected to load. Although very little is understood of the mechanisms involved, it has been shown (27) that such knowledge is necessary with respect to partial extraction systems. The ratio of the elstic moduli of the different materials involved may be as large as 10 to 1, and the influence of soft or hard roofs and floors on the pillar material can be such that an adequate pillar size falls between extremely critical limits. Some preliminary investigations to obtain a clearer understanding of the stresses at

TABLE 1

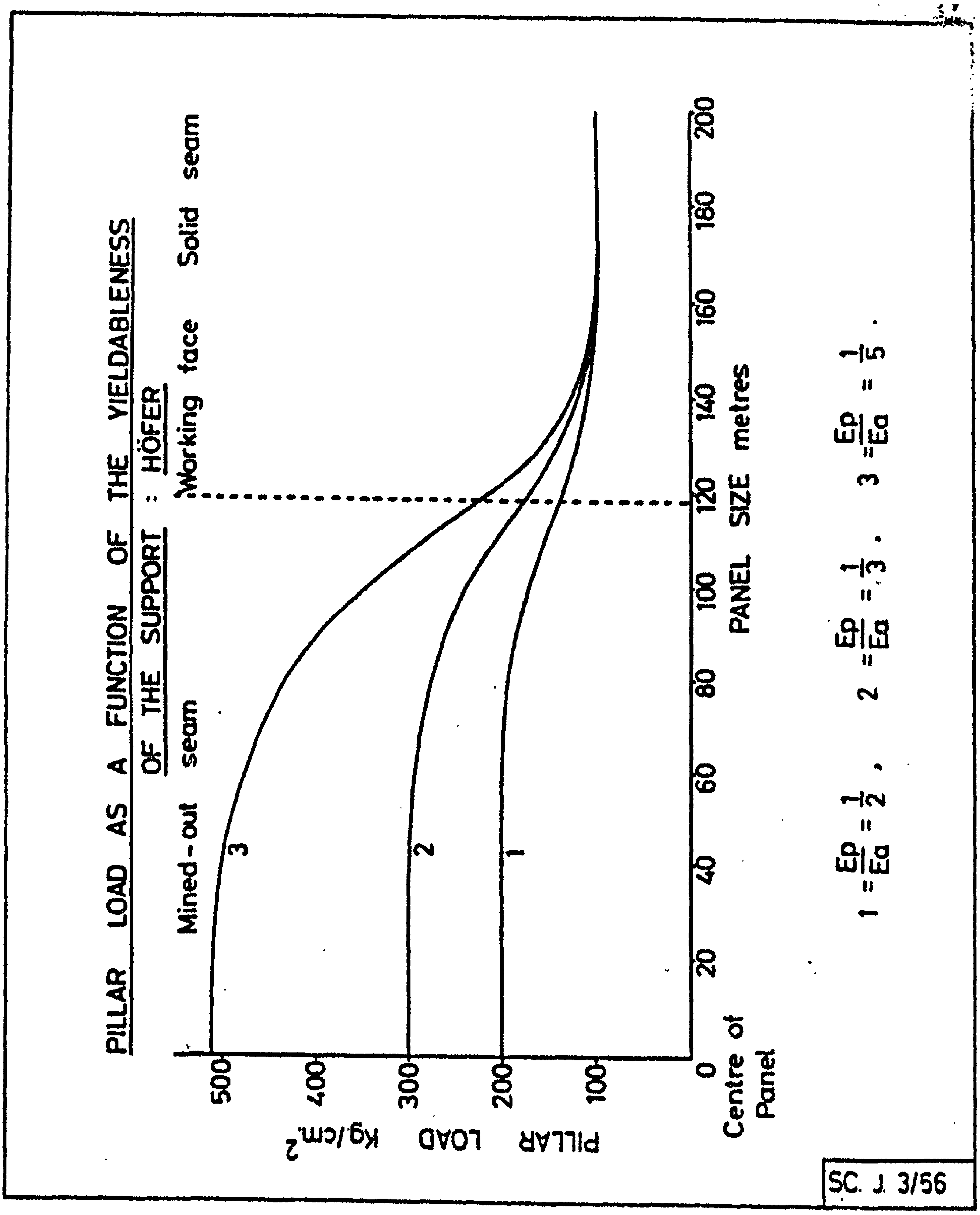
LITERATURE SURVEY ON THE DISTRIBUTION OF THE LOAD ON ROOM AND PILLAR WORKINGS.

Name of Investigator	Year	Mineral	Investigational Technique	Conclusions (Numbered) and REMARKS
DUVALL, W.I. (28)	1948		Two-dimensional photoelastic models representing a vertical cross-section through a room and pillar system. Models contained row of five ovaloidal openings. Continuous elastic body theory used. Fig.3a.	1. The pillars in the centre of the excavation are under more stress than those near the rib-sides. 2. Five openings, or four pillars, were the minimum number that allowed a fairly constant stress concentration factor in the model.
ALDER, H. POTTS, E.L.J. WALKER, A. (29)	1951		Photoelastic models used to show how the stress distribution around the workings changed with a change in 'stiffness' of the pillar. Single pillar used.	1. The pillar of lower stiffness than the model material deflected the field stress around the two rooms and intervening pillar onto the outside abutments. No analysis of data to provide a method for determining the obviously reduced pillar load.
STEART, F. (30)	1954	Coal	Large number of undergr und observations.	1. The load is not evenly distributed even under normal geological conditions,-assumed a conical or parabolic distribution in the case of a developed area of circular plan, so that the maximum pressure was exerted on the centre pillars. 2. The load gradually increases with increasing area of development to a maximum- equation (5)- this maximum being reached when the development, if roughly circular in plan, attains a radius equal to the depth of overburden divided by the area extraction ratio. Neither theoretical analysis nor measurements given to justify these statements.
AVERSHIN, S. (31)	1958		Photoelastic models.	1. When the width of the mining zone is equal to the depth of working, the central pillars in the mining zone will carry the full load as given by equation (5). No experimental results provided.
COATES, D.F. (32)	1961		Theoretical analysis to calculate the deflection of any point in the overlying strata.	1. When the ratio of the width of the workings to the depth is low, the average pillar stress in the central pillar should be less than that given by equation (5). The average pillar stress can only be calculated for the central pillar.
TINCELIN, E. SINOU, P. (33)	1960	Iron ore	Constructed a mathematical model based on the beam concept. Using elastic theory, calculated the resultant deflections of roof beds over pillared area as function of pillar loads and abutment pressures.	1. The relationship obtained is shown in Fig.6. The pillar load varied according to their position in relation to the abutment zones, and according to the width of the development. 2. A critical condition was recognised whereby the strata passed from a homogenous to a stratified state, due to shearing stresses resulting from the deflection of the beams Fig.6a, could lead to roof failure. Did not take into account the contribution to closure of the deflection of the floor.
HOFER, K.H. MENZEL, W. (34)	1964	Potash	Used the mathematical work of Tincelin and Sinou. Calculated the effects on pillar loads of the deflection characteristics of the overlying strata. Overburden considered as consisting of fixed slabs or beams resting on elastic supports.	1. The relationship obtained is shown in Fig.7. The greater the pillar yield, the greater the pillar load. The more rigid the pillar, pillar loads less and reached more consistent values in the region close to the rib-side. 2. As the rigidity of the overburden increased, so the pillar loads reduced, with a corresponding increase in load on the solid rib-side. It was claimed that the relative effects were qualitatively substantiated by underground observations, but no such measurements were provided.



THEORETICAL PRESSURE PROFILES FOR PARTIAL EXTRACTION PANELS (AFTER TINCELIN)

Fig. 6



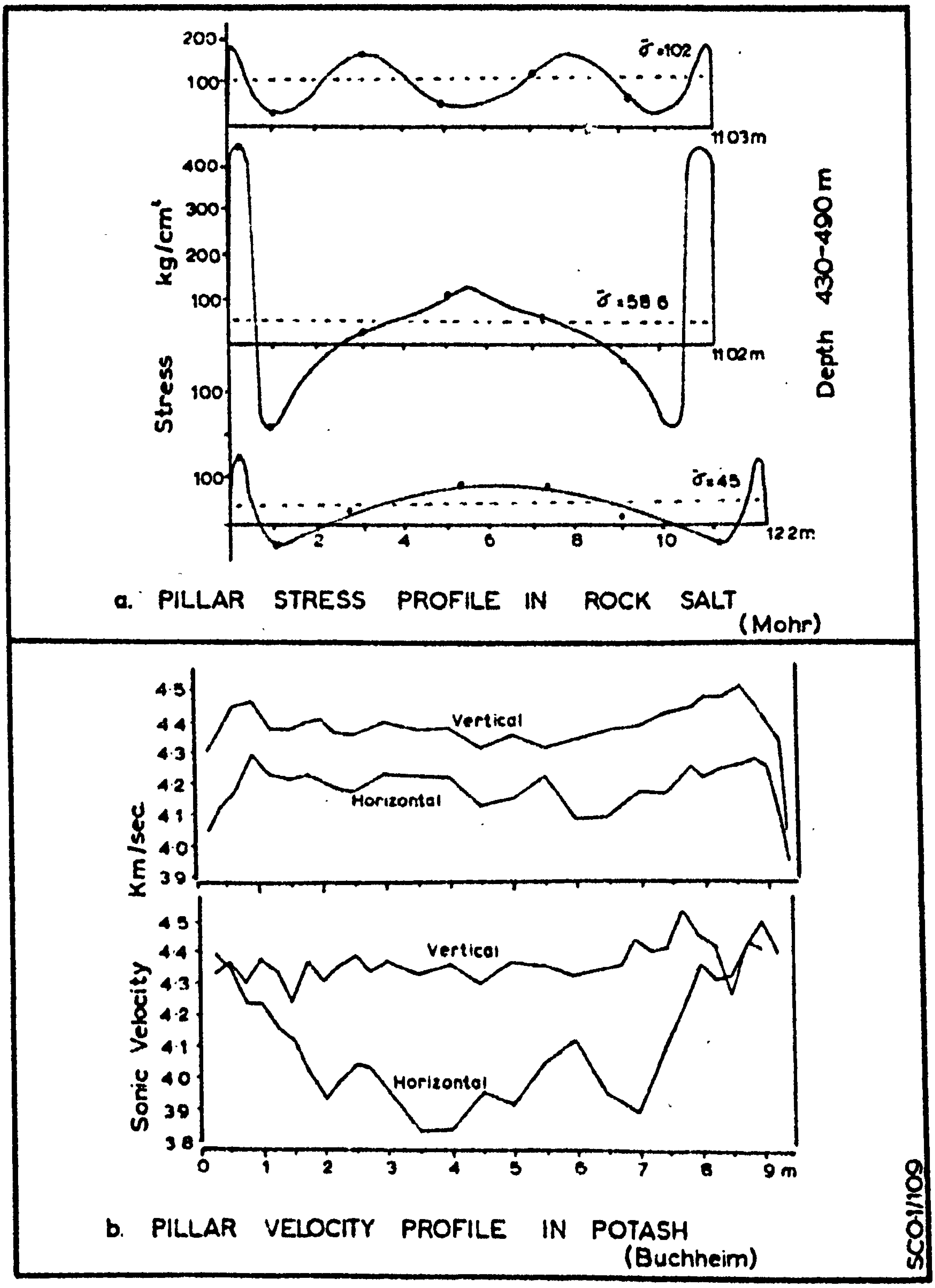
PILLAR LOAD AS A FUNCTION OF THE YIELDABLENESS OF THE SUPPORT : HÖFER
Mined-out seam
Working face
Solid seam
PILLAR LOAD kg/cm.²
500
400
300
200
100
0
Centre of Panel
20
40
60
80
100
120
140
160
180
200
PANEL SIZE metres
3
2
1
1 = Ep/Ea = 1/2 , 2 = Ep/Ea = 1/3 , 3 = Ep/Ea = 1/5 .
SC. J. 3/56

Fig. 7

TABLE 2

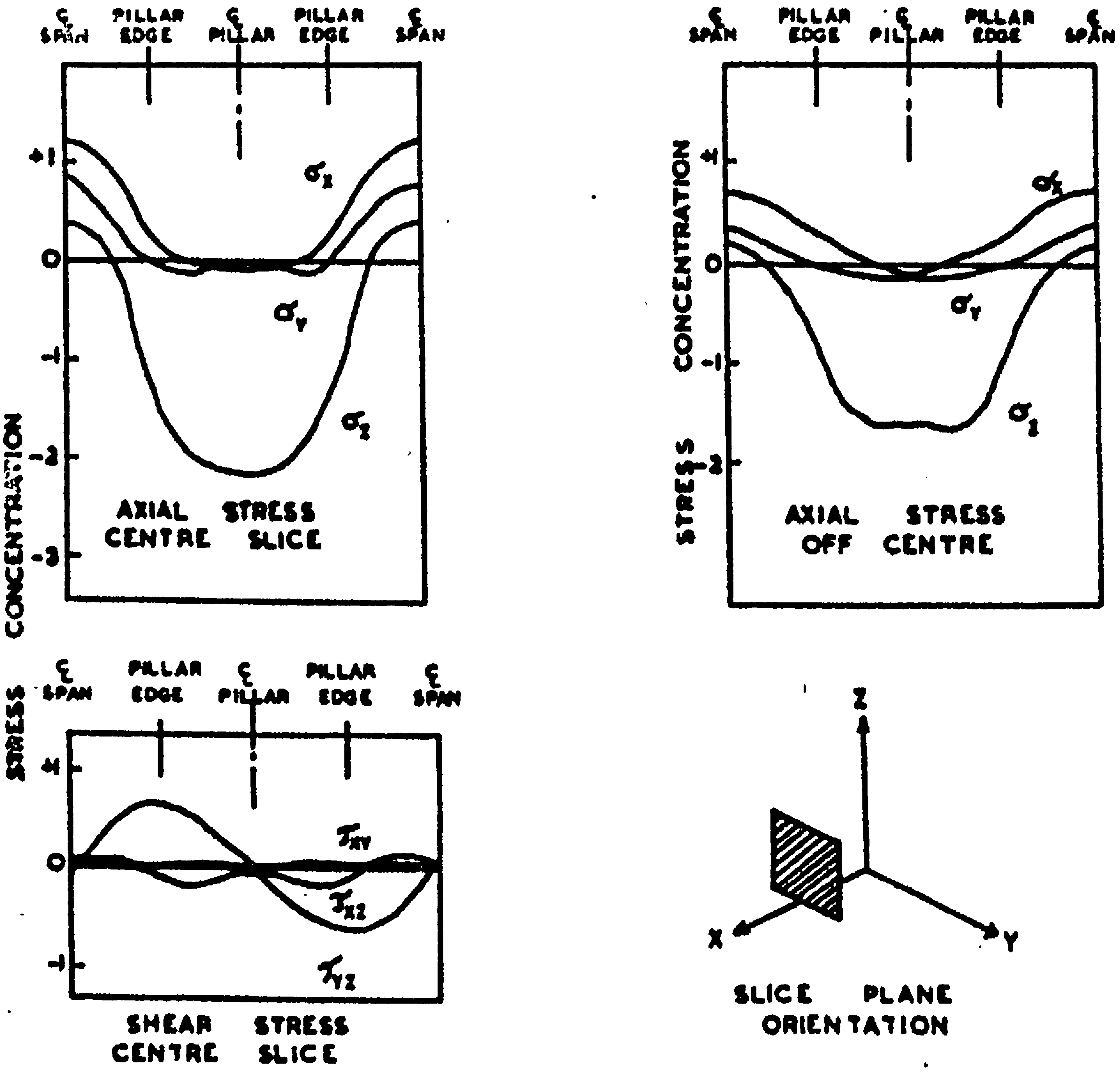
LITERATURE SURVEY OF THE PILLAR STRESS MAGNITUDE AND DISTRIBUTION

Name of Investigator	Year	Mineral	Investigational Technique	Conclusions (Numbered) and REMARKS
OBERT, L. (35) (36)	1939 - 1940		Measured the velocity of sound through the pillar before and after the pillar was cut free from the roof.	1. The velocity through the pillar fell from 4755 m/sec to 300 m/sec, whereas the velocity in the adjacent pillars remained the same. Impossible to deduce stress magnitude.
DUVALL, W.I. (28)	1948		Photoelastic Models - Table 1.	1. The stress is not evenly distributed over the cross-section of the pillar, the maximum stress occurring at the pillar edges. 2. The maximum stress occurs at the corner between pillar and roof or floor. 3. The stress on the pillars depends upon the W/H ratio of the pillar. Results valid for 2-D homogenous, isotropic, linear-elastic continuous system, and should be treated as a first approach only.
PANEK, L.A. (37)	1951		Photoelastic models containing two rectangular openings.	Same conclusions as Duvall, but slightly higher stress concentrations at corners due to shape of opening.
MOHR, H. (38)	1956	Potash	Two unidirectional electrical wire strain gauges were cemented at the end of a borehole, at right angles to each other. These then overcored and the stress relieved.	1. Pillar stress measurements obtained were lower than the theoretical load - equation (5). At the time this research was done, the state of the art of bonding strain gauges to rock was questionable. Also the effects of stress concentrations around the end of the borehole were ignored. See Fig. 8a.
AVERSHIN, S. (31)	1958		Theoretical.	1. The distribution of the stresses in the pillar depends on the relative values of the strength of the rock in the roof and in the floor.
HAST, N. (22)	1958	Lead	Stress relieved a pre-stressed magnetostrictive pressure cell.	1. Measured stress in pillar near centre of excavation indicated that pillar was carrying 125% of the cover load, calculated from equation (5). Pillar near to rib-side only carrying 16% of same coverload. 2. Measurements in the abutments of the mining area indicated that the vertical stress was twice the value of the stress calculated from equation (5).
MAUCHI, I. (39)	1959		Glass stressmeters used to measure the change in pillar stress resulting from further mining. No overcoring - relative measurements only.	1. Showed that the direction of the major principal pillar stress was influenced by the existence of an inclined ground surface. i.e. the major principal stress can be greater than the immediate overburden stress, and can be inclined to the vertical.
SOKOLOVSKI, V.V. (40)	1961		Proposed a mathematical appreciation of the distribution of stresses in pillars, involving the integration of stresses acting along a number of finite bands constituting a pillar section.	1. Deduced a network of slip lines related by the cohesion and angles of friction of the rock materials which make up the pillar structure.
TRUMBACHEV, V. MEL'NIKOV, E. (41)	1961		Photoelastic and mortar models.	1. Pillar loadings were invariably less than predicted by equation (5). No explanation offered for the findings.
BUCHHEIM, W. (42)	1958	Potash	Used a sonic method to obtain pressure profiles in pillars.	1. Results obtained shown in Fig.8b. Very high stress concentrations exist within 1 m. of the pillar edge. No calibration carried out so results were given in sonic velocity.
SCOTT, J.J. (43)	1964		Three-dimensional photoelastic model to determine the stress distribution in pillars; Model containing 25 pillars and representing 75% extraction.	1. The results, some of which are shown in Fig.9, provided information on the overall stress distribution in points of stress concentration, shear and tension, showed the relative magnitude of stress throughout the model. 2. The vertical and horizontal stress concentrations were nearly zero over the pillar area. 3. Indicated that there was a high vertical shear component associated with the edge of the pillar. Some doubt as to the applicability of findings.
LEEMAN, E.R. Van HEERDEN, W.L. (44)	1964	Coal	'In situ' stress measurements using linear variable differential transformers as the transducers for measuring the changes in a borehole diameter, as the borehole was 'stress relieved' by overcoring, or 'slotting'.	1. Obtained pressure profiles of the vertical and horizontal stresses in the pillar, some of which are shown in Fig.10. 2. The average pillar stresses were found to be greater than the pillar stresses calculated from equation (5).
COATES, D.F. (26)	1965		Combined existing scientific theories into a rational hypothesis for predicting pillar loads. A theoretical formula was developed from this hypothesis, and examined to some extent with laboratory model studies, supplemented with extensive field measurements of pillar stress. The hypothesis was based on the relation between the pillar stress and pillar deflection.	1. Proposed a general equation containing 15 variables for the determination of pillar load. 2. The laboratory model work indicated that the range of applicability of the equation was smaller than the theoretical analysis originally suggested. 3. The measured pillar stresses did not correspond to the predicted stresses. 4. The most significant parameters, aside from the extraction ratio, affecting the pillar loading, were shown to be the ratio of the compressibility of the pillar rock to wall rock, and the W/H ratio of the pillar. It was not proved that the hypothesis could be applied to anything but elastic ground.
SINGHAL, R.K. (45)	1970	Rock Salt	Underground attempt to measure the 'in situ' state of stress in pillars using the 'doorstopper' stress relieving technique.	1. Clearly demonstrated that this technique for absolute stress determination is capable of giving satisfactory results in rock salt. 2. Difficulties mainly concerned with the bonding of the strain cell to the flat end of the borehole. 3. Several pillar stress values obtained, but repetitive results difficult to achieve.



σ̄=102
1103m
σ̄=58.6
1102m
σ̄=45
122m
Stress kg/cm²
Depth 430-490 m
a. PILLAR STRESS PROFILE IN ROCK SALT (Mohr)
Vertical
Horizontal
Vertical
Horizontal
Sonic Velocity Km/sec
b. PILLAR VELOCITY PROFILE IN POTASH (Buchheim)
SCO1/109

Fig. 8

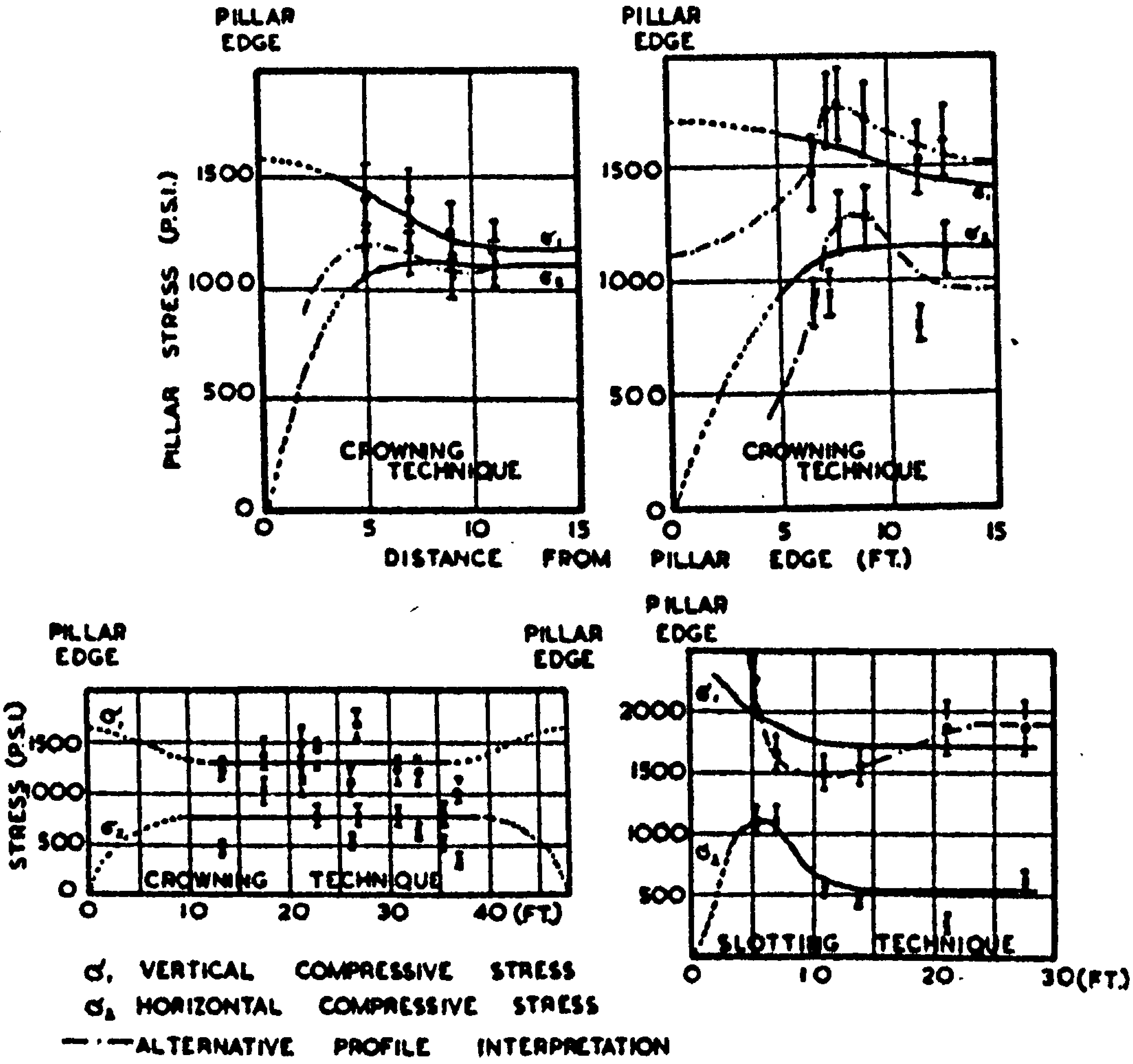


THEORETICAL STRESS DISTRIBUTIONS FOR PILLARS

(AFTER SCOTT)

No 1999

Fig. 9



COAL PILLAR STRESS PROFILES
FROM STRESS RELIEVING TECHNIQUES
(AFTER LEEMAN AND VAN HEERDEN)

Fig. 10

the interface have been carried out (46) (47) (48), but the determination of such phenomena is very difficult, and has been confined mainly to the theoretical approach.

The load carried by a pillar may not be distributed uniformly across the horizontal cross-section of the pillar. Stress concentrations can occur in the vicinity of the pillar edges (28). To compensate for this, a factor of safety is used, which also takes into account the difference in compressive strength of samples tested in the laboratory and the actual strength of mine pillars. The determination of this factor of safety will be discussed later.

3.2 The Strength of Pillars.

The determination of the effective strength of a pillar or a series of pillars, i.e. the load on the pillar which will cause it to collapse, disintegrate or yield excessively, is a problem which as yet, has not been solved satisfactorily. Various attempts have been made to establish design criteria for pillars by the formulation of relationships between their dimensions and a laboratory determined constant for the pillar material. The most readily assessable design property in the case of mine pillars is their strength in compression determined in the laboratory.

The procedure adopted in all laboratory testing techniques to determine the compressive strength of the material forming the pillar, is to load the specimen to failure in a hydraulic press, the specimen being considered as a model of a mine pillar since the mode of failure of the small specimen appeared to be similar to the manner in which pillars fail underground. A large number of objections may be raised regarding the validity of this type of test for underground application, not least of which is the degree of extrapolation involved. The extrapolation between the relative sizes of the laboratory specimen and the mine pillar is so great that any errors in the strength of the small specimens are considerably exagerated in the process. A further objection may be raised concerning the factors affecting the compression test itself. This has been discussed in great detail elsewhere, notably by Buzdar (49). In this case, a possible means of overcoming this is to standardise the form of the test specimen used as well as the testing procedure, as has been suggested by the International Bureau of Rock Mechanics (50).

Where any form of laboratory testing is involved, the extrapolation constants required to apply the findings to field conditions are the most difficult to determine. Denkhaus (51), basing his principle on Griffiths'(52) reasoning in his theory of rupture, propounded that the strength of a material depends upon the number of inherent 'deficiencies' or flaws present within the material. The influence of such 'deficiencies' on strength becomes less pronounced with increase in volume of the material. Hence, the difference in strength between a specimen with say, ten 'deficiencies' and one with a hundred 'deficiencies' is enormous, whilst the difference between three hundred and two thousand 'deficiencies' is negligible, as far as strength is concerned. He used this argument to explain the large scatter obtained when testing small specimens.

The strength of mine pillars has been studied by many research workers who have in many cases, produced formulae for the pillar strength. These investigations have indicated that the strength of a pillar is invariably determined by the intrinsic properties of the pillar material and the geometries of the pillar and the mine. Some factors tend to reduce the load bearing capacity of pillars and others tend to increase it. A brief review of the investigations carried out, and the formulae obtained for the calculation of pillar strength is given in tabulated form in Table 3.

In general, the strength of mine pillars depends upon four groups of factors, the micro-structural strength of the pillar material, environmental conditions causing a time dependent strength condition, the height to width ratio and shape of the pillar, and the mine structure in general. The micro-structural strength of the pillar material

TABLE 3

LITERATURE SURVEY ON THE STRENGTH OF MINE PILLARS

Name of Investigator	Year	Investigation Technique	Equation	Remarks
Holland, C.T. and Gaddy, F.L. (57)	1938	Laboratory and Underground tests on coal pillars	$S = K\sqrt{L}$ S = Strength (psi) L = least pillar width (ins) T = pillar thickness (ins)	The results suggested that a mixture of large and small pillars should be avoided.
Greenwald, H.P. Howarth, H.C. and Hartmann, I. (58)	1941	Underground compressive strength tests on 12 small coal pillars standing on a fireclay floor. Pillars carefully isolated without using explosives and crushed by means of hydraulic loading equipment	$S = 2800W^{0.5}H^{-0.85}$ S = strength (psi) W = width of pillar (ins) H = height of pillar (ins)	The floor failed in 2 tests, so that concrete restraining rings were placed around the bases of the other pillars. Seven of the results were used to produce the formula. The results indicated that pillar height was more important than pillar width.
Steart, F.A. (30)	1954	Laboratory crushing strength tests carried out on small coal specimens, supplemented by underground visual observations. Specimens: 9in. sq.base, 4 - 27 in.h.t.	$S = KW^{0.5}H^{-1}$ S = strength (ton/sq.ft. W = width of pillar (ft) H = height of pillar (ft)	Concluded that the same mechanical principles - wedge action - were involved in the failure of laboratory specimens and coal pillars. This equation, partly hypothesis and partly laboratory work, is questionable but has been used by many South African collieries.
Gaddy, F.L. (61)	1956	Laboratory strength tests on 80 cubical coal specimens, 2in - 9in size.	$S = 5{,}551.L^{-1/2}$ S = strength (psi) L = side of cube (in)	This equation is markedly different from that produced by the 'in-situ' tests on the same seam material by Greenwald et al (58). This formula was found not to be applicable to pillar design.
Potts, E.L.J. and Szeki, A. (14)	1966	Laboratory strength tests on 409 Coal specimens Width constant at 3"; height varied 0.75" - 3" Height constant at 2"; width varied 1" - 4".	$S = 4213W^{0.38}H^{-0.49}$ S = strength (psi) W = width of pillar (ins) H = height of pillar (ins)	This equation is based on the minimum crushing strength value determined in the laboratory, and was used as a basis for the design of coal pillars in South African collieries. The validity of the formula was analysed statistically and found to provide effective design data with an adequate factor of safety.
Salamon, M.D.G. and Munro, A.H. (3)	1967	Derived statistically, an empirical pillar strength formula based on a survey of actual coal mining dimensions from 125 collieries, of which 27 had experienced collapse	$S = 1320W^{0.46}H^{-0.66}$ S = strength (psi) W = width of pillar (ft) H = height of pillar (ft)	This pillar strength equation was regarded as being sufficiently reliable to form the basis of a method of design, Fig.21, which was subsequently used by many South African collieries.
Bieniawski, Z.T. (4)	1967	Underground compressive strength tests on coal pillars of varying dimensions. The pillars were separated from the roof and the load applied through a concrete pad by means of hydraulic jacks.	$S = 1100W^{0.16}H^{-0.55}$ S = strength (psi) W = width of pillar (ft) H = height of pillar (ft)	Used the equation to construct a nomograph, Fig. 22, which formed a basis for the design of workings. This nomograph included similar factor of safety data to that used by (14) and (3) above.

is determined by its rock constituents, the mode of formation of the material, and the number and size of micro-flaws in the material. The environmental factors causing a time dependent alteration of the strength of the pillars, are indirectly related to mining and include the chemical alteration of the pillar material by solution and oxidation, the deformation of the pillar not necessarily associated with changes in stress, where the pillar may continue to deform until it fails, and dimension changes of the pillar due to progressive fracturing and surface spalling. The time dependent deformation or creep of the pillar, has been shown to be an important factor in the design of pillars of certain minerals, mainly evaporites and coal, and a great deal of information concerning the effects of this phenomena on the stability of mine pillars, has been obtained at the University of Newcastle upon Tyne (53) (54) (18).

The effect of the width to height ratio and the shape of the pillar on its failure strength has been extensively examined. It has been shown that the stress distribution in pillars is very much affected by the width to height ratio, which by producing different stress concentrations for different pillar geometries Fig.11 affects directly the strength of the pillars. Consequently, pillar strength must depend on its width to height ratio. Where formulae were obtained for determining pillar strength, as can be seen in the tabulated literature in Table 3, it was invariably presupposed that the failure strength, S, could be expressed by an equation of the type

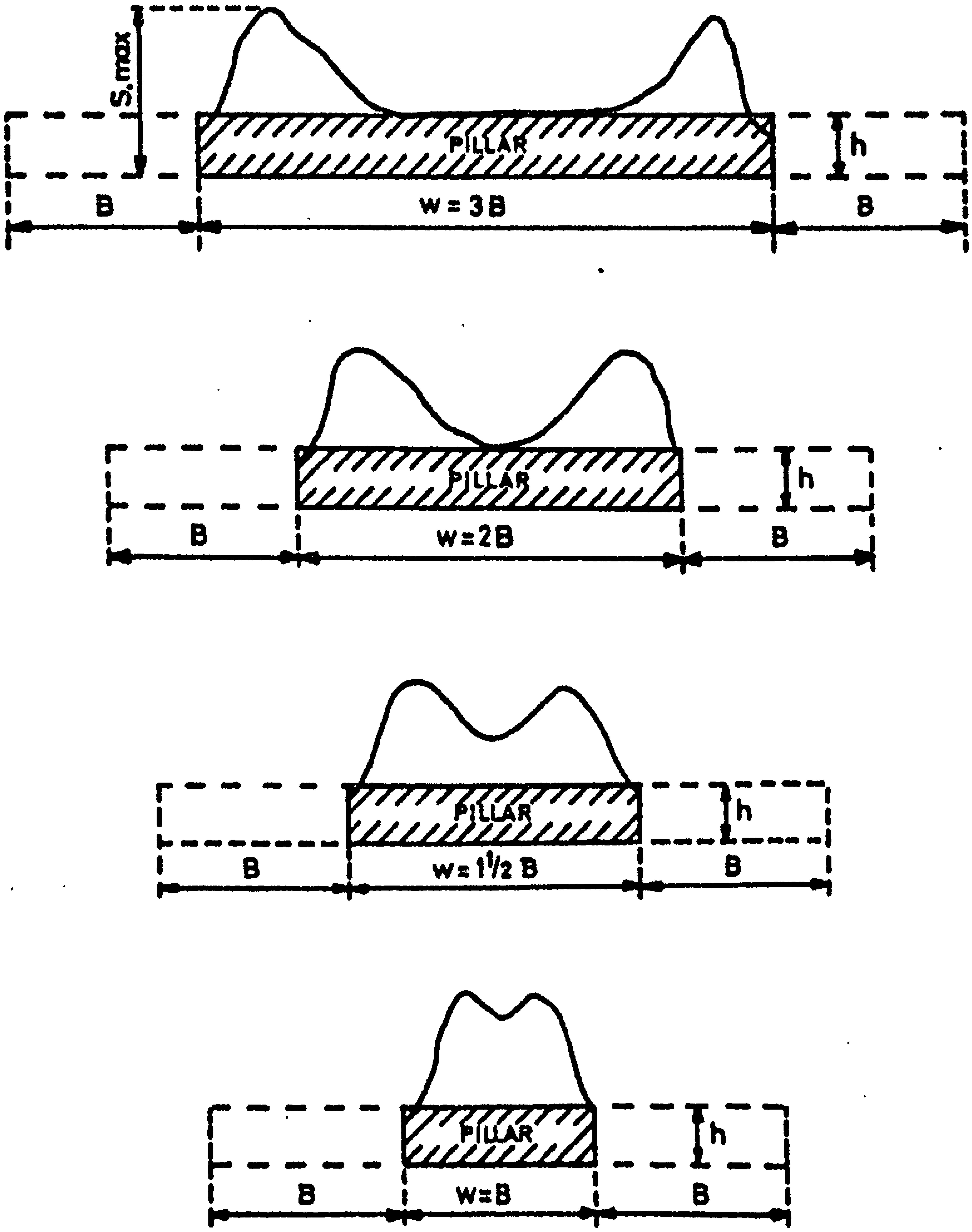
$$S = K W^{a} \times H^{b} \qquad (6)$$

where W = width of pillar.

H = height of pillar.

K, a and b = constants.

It is generally accepted that as the width to height ratio increases, so does the strength. Also, the strength decreases with



Pillar stress magnitude and distribution as affected by width of pillar relative to its height

(after Holland)

Fig. 11

increasing pillar height for a given pillar width. This is due to the restraining effect of the roof and floor strata on the ends of the pillar, and also the constraint imposed on the central part of the pillar by the outside edges. This constraint tends to impose conditions of triaxial loading on the central core of the pillar with a consequential increase in compressive strength. As the width of the pillar becomes proportionally greater than the height of the pillar, i.e. as the W/H ratio increases, the roof and floor of the workings have a greater and greater restraining effect on the pillar ends with a corresponding increase in strength.

The factor of pillar shape for a given plan area has also been found to have a bearing on pillar strength. A square pillar is regarded (55) as capable of accepting higher loads before failure than one rectangular in shape. If however, we consider the respective strengths of two pillars, one rectangular and one of square plan shape, the rectangular pillar having its shorter sides equal to the sides of the square pillar, then the rectangular pillar will certainly be much the stronger.

The fourth and final factor, the structural effect of the mine layout on the pillar failure strength, is caused by the relative deformation of the roof, the pillar and the floor materials when they are stressed, and by their relative failure strengths. This has been discussed in an earlier section 2.2. Associated with this is the possible presence of anomalous interstitial bands running through the pillar. The effect of the presence of such anomalous layers on the pillar stress distribution had been investigated photoelastically by Trumbachev (56).

The fact that the value of the ultimate compressive strength as determined in the laboratory cannot be applied directly to practical situations within the mine has been stated by many investigators, and much discussion about the relevance of laboratory testing has taken place (4) (51) (15). Most investigators agree that, if laboratory testing is to be taken into account, the strength value obtained must be modified after a careful consideration of the factors which are likely to influence the behaviour of the rock material in the mine. Underground strength testing however, possesses distinct advantages over laboratory testing, particularly in the case of the softer minerals which deteriorate rapidly when exposed to different environmental conditions. The determination of the failure strength of pillars 'in situ' however, is costly, time consuming and dangerous, though the results obtained offer directly, interpretable data for practical application. This type of testing eliminates selective sampling which can bias laboratory test results. The most successful underground strength testing was carried out by Greenwald, Howarth and Hartman (58), and more recently by Bieniawski (4), both on coal pillars of varying dimensions (Table 3). Whilst they have undoubtedly enjoyed a fair measure of success, Bieniawski probably more so, their experimental processes have a certain number of practical drawbacks.

To overcome the difficulties of both the laboratory and underground technique for determining pillar strength, Salamon and Munro (3) attempted to determine a formula for pillar strength based on actual mining experience. It was argued that during the seven decades of coal mining in South Africa, a vast amount of experience had been accumulated which, if it could be rationalised, might form the basis of the most practical method of design. The aim therefore, was to derive an empirical pillar strength formula from the accumulated data. A formula shown in Table 3 was derived statistically, expressing the approximate strength of coal pillars in South African Collieries, based on data obtained from a survey of actual coal mining dimensions from 125 collieries, of which 27 had already collapsed. It was assumed that those pillars which were still intact had safe dimensions whilst the collapsed pillars had been too small. Since however, in over fifty per cent of the collapsed workings, failure occurred 3 - 5 years after mining, it is possible that a number of presently unfailed pillars may collapse in the future. The validity of the assumption that presently stable pillars are safe could not be verified since the available data was not sufficient for consideration of the time effect.

3.3 A Consideration of the Roof Span.

The rock mass of a stratified deposit can be considered to be made up of a large number of 'plates' of various and varying thicknesses. Consequently, where a relatively flat roof is formed in a horizontal deposit, the lower portion of the roof often becomes detached from the overlying rock - either immediately or after sometime - and forms a gravity loaded layer. This layer is often considered as a 'beam' or 'plate' resting upon supports formed by the pillars, and may be considered as either a beam clamped at the edges by the pressure of the overburden bearing on the pillars, or as a freely supported beam. The decision as to whether the roof should be considered as a simply supported or a clamped beam is usually rather difficult, but is normally taken after preliminary investigations into the strength characteristics of the overlying strata, the state of the beam ends having a marked effect on the magnitude and position of the maximum tensile stresses.

No further mention is made here of simply supported beams, a clamped beam being considered in most cases, to provide the closest analogy for the behaviour of the roof beam. The roof layer may consist of one or more members that deflect as a single unit, the lowest single, double or multi member layer in the roof being defined as the immediate roof. The stress and deflection in the immediate roof can then be evaluated from beam theory provided certain assumptions are made :-

(i) the roof is composed of a rock which is elastic, homogenous and isotropic, and uniformly thick.

(ii) the thickness of the members is small in comparison to the roof span.

(iii) the length of the excavation is more than twice the roof span.

(iv) the deflection of the layer is small in comparison to its thickness.

(v) each layer is assumed to act as a beam supported by elastic abutments, and supports a uniformly distributed load that is equal to its own weight, plus any load that is applied by the overlying strata.

3.3.1 When the immediate roof is a single-member layer which is clamped at the edges by the pressure of the overburden on the pillars, the maximum deflection will occur at the centre of the beam, and the maximum shear and tensile stresses occur at the ends of the beam. At the centre of the beam the shear stress is zero and the tensile stress is one half of the maximum value. Thus the point of initial failure would be expected to occur at the ends of the span rather than the centre. The maximum values of deflection and tensile and shear stresses are given by the following equations :-

$$D\ max. = \frac{\rho L^4}{32Et^3} \qquad (7)$$

$$T\ max. = \frac{\rho L^2}{2t^2} \qquad (8)$$

$$S\ max. = \frac{3\rho L}{4t} \qquad (9)$$

where

D max. = maximum deflection.

T max. = maximum tensile stress.

S max. = maximum shear stress.

ρ = density.

L = length of beam.

t = thickness of beam.

E = Young's modulus of beam.

When the beam is long in comparison to its thickness, the tensile stress developed is greater than the shear stress, and the beam will will tend to fail in tension before it will fail in shear. For this reason, shear strength is normally omitted from design calculations.

3.3.2 When the immediate roof consists of two members clamped at the ends, there are two cases to consider :-

(i) When the thicker of the two members overlies the thinner one, each member acts independently due to a greater down sag of the lower member; a cavity appears between the two members. In this case, the deflection and stresses can be calculated from equations (7 - 9).

(ii) When the thinner member overlies the thicker one, it loads the lower member. The two members bend exactly like a single layer equal in thickness to the total thickness of both, but made up of rocks with different elastic properties.

In the second case, the increase in load on the lowest member can be considered as an increase in density of thelowest member. This 'apparent' density is given by :-

$$\rho a = \frac{E_1 \, t_1^{\,2} \, (\rho_1 \, t_1 + \rho_2 \, t_2)}{E_1 \, t_1^{\,3} + E_2 \, t_2^{\,3}} \qquad (10)$$

where ρa = apparent density of the lowest member.

ρ_1, ρ_2 = density of each member.

E_1, E_2 = modulus of elasticity of each member.

t_1, t_2 = thickness of each member.

The maximum values of deflection and shear and tensile stresses can be determined by applying equations (7 - 9) and using ρa to replace ρ. This procedure can be extended to a multi-member beam. As above, the additional loading occurs when the thicknesses of the upper members are less than the lowest members. For a multi-member beam the apparent density is :-

$$\rho a = \frac{E_1 \ t_1 2 \ (\rho_1 \ t_1 + \rho_2 \ t_2 + \ldots\ldots + \rho_n t_n)}{E_1 t_1{}^3 + E_2 t_2{}^3 + \ldots\ldots + E_n t_n{}^3} \qquad (11)$$

The maximum values of deflection and shear and tensile stresses are then obtained by substituting equation (11) in equations (7 - 9).

Most roof deformations however, are not perfectly elastic and may contain defects within the layers, such as joints, fractures, faults and fissures. Also, the roof may contain horizontal planes of weakness below which wholly detached roof layers will form. In some instances the rock exhibits a degree of weakening with time (12), which may cause a change in the number of layers, the physical properties of the roof, or any of the factors that would affect the stability of the roof with time.

Many investigators have done work in this field, some have used models and others have approached the problem in a purely theoretical manner. The following references are representative of the investigations carried out, (65) (66) (67) (68) (33) (34) (64).

Stephanson (64), provided mathematical solutions of deflection, bending moments and fibre stress for seven different configurations of single and multi-layer roofs in horizontally bedded rock. The work, based on the theory of elastic beams on elastic supports, shows the importance of abutment compression in contributing to the deflection of the roof layers. The theories and results from finite element model experiments were put into practice at a lead mine in Sweden, and are continuing at the present time. The results obtained to date are in accordance with the results from proposed theories. An attempt has been made by the author to apply some of the equation obtained by Stephanson to one of the experimental sites. The results obtained are described in a later part of this thesis.

3.4 The Evaluation of Design Data.

The information and data of the type described in the previous three sections are of little value until arranged into readily assimilable form for practical use. For a particular set of circumstances the criterion for design must first be established, and then all variables likely to affect this limit of design correlated, in order to build up a range of combinations of circumstances by which the design limits can be satisfied.

In order to derive the formulae given in the preceding sections, some assumptions had to be made. These assumptions constitute uncertainties which must therefore be incorporated into an appropriate factor of safety. The ratio of the estimated ultimate compressive strength of the pillar to the estimated imposed stress is generally maintained at a value considerably greater than one, to give the pillared area a factor of safety. Obert, et al (66) found that factors of 2 to 4 were adequate for pillars, whereas a safety factor of 4 - 8 was required for a bedded roof. Salamon and Wilson (71) conducted a statistical survey of collapsed and stable workings in South Africa, and found that over 50% of the stable workings gave a factor of safety between 1.2 and 1.9. It was emphasised however, that local mining experience together with geological features of the workings should be taken into consideration when deciding on a suitable factor of safety.

In a room and pillar mining system, the final design criteria necessary for practical application are the dimensions of the pillars and the width of the roadways. The form in which combinations of parameters relating to these are presented for the design of new workings and for checking the stability of existing ones, fall broadly into (a) simple formulae, (b) complex design nomographs.

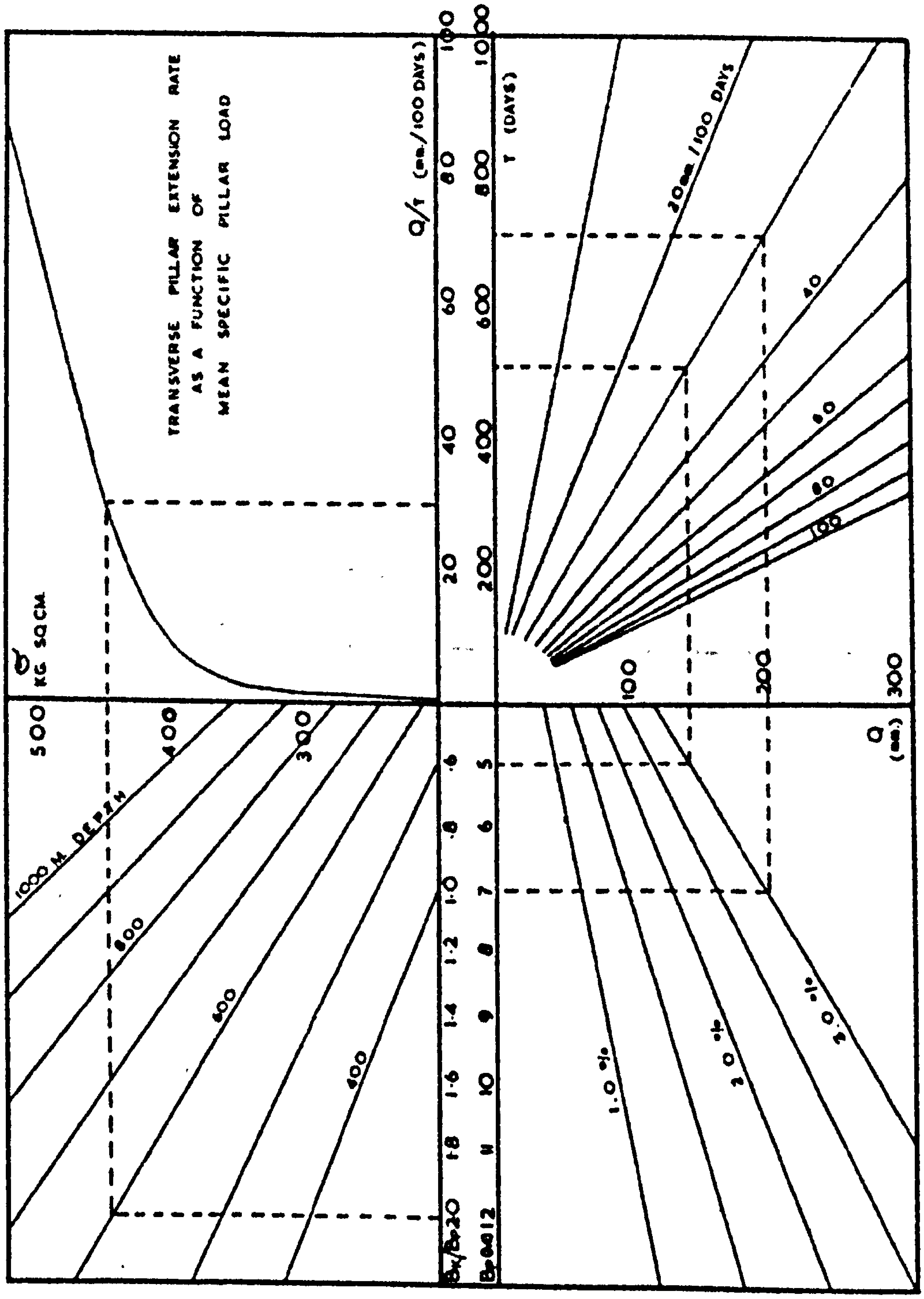
Several design formulae have been propounded and represent in various degrees of complexity, the ultimate strength of a pillar with respect to its dimensions and other relevant factors. The effectiveness of this type of relationship however, tends to rely heavily on the strength of the pillar material and the safety factor, the determination of both of which has many objections.

The most convenient presentation method for design data where a large number of inter-related parameters are involved, is by nomographic representation, first used in the design of room and pillar workings by Hofer (72). This method has now been used extensively to provide design information for workings in the following media :-

Potash	E.Germany	Hofer (72)
Rock Salt	Gt.Britain	Potts (53), Sen (12), Hedley (54)
Anhydrite	Gt.Britain	Salamon (70)
Coal	S.Africa	Taylor (73)
Coal	S.Africa	Potts and Szeki (14)
Coal	S.Africa	Salamon and Munro (3)
Coal	S.Africa	Bieniawski (4)

The basis for Hofer's design nomograph consists of data obtained from underground measurements in a number of East German potash mines. The nomograph, shown in Fig.12, consists of two distinct parts. The upper portion of the graph consists of a means of derivation of transverse pillar extension rate from the depth of working and the rate of extraction. In the lower portion, values of pillar width, $\beta\rho$, are plotted against pillar extension, Q, determined by 'in situ' measurement. The range for which percentage pillar extensions are taken (up to 3%), represent conditions near those allowed underground before it is considered that pillar 'failure' has occurred. The adjacent graph is a relationship between pillar extension and time.

Based upon an extensive laboratory programme of creep testing, Potts (53) and Sen (12) developed a method to estimate the maximum stress which pillars of rock salt would support, without failure, for an indefinite



DESIGN NOMOGRAPH FOR POTASH PILLARS (AFTER HÖFER)

Fig. 12

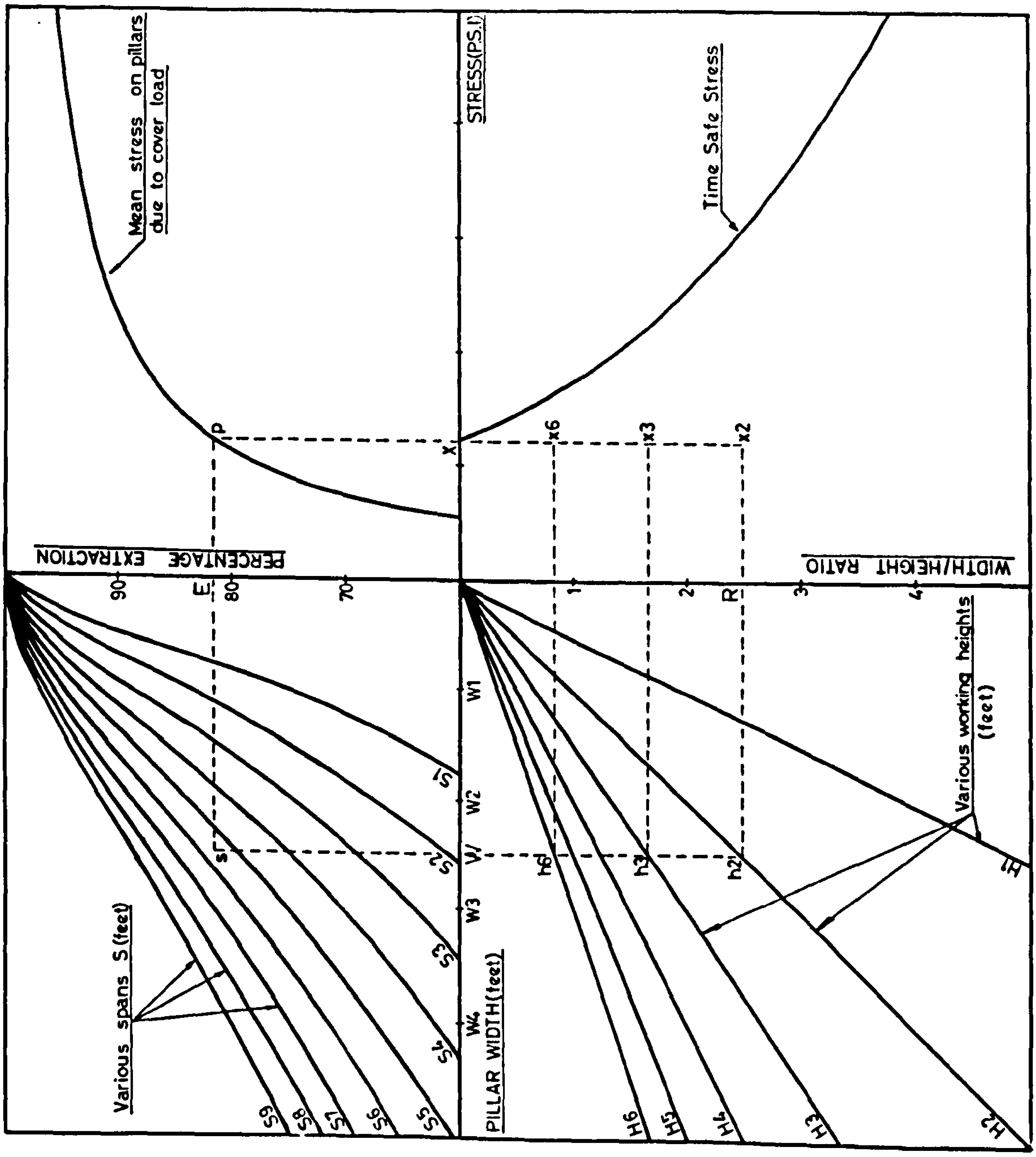
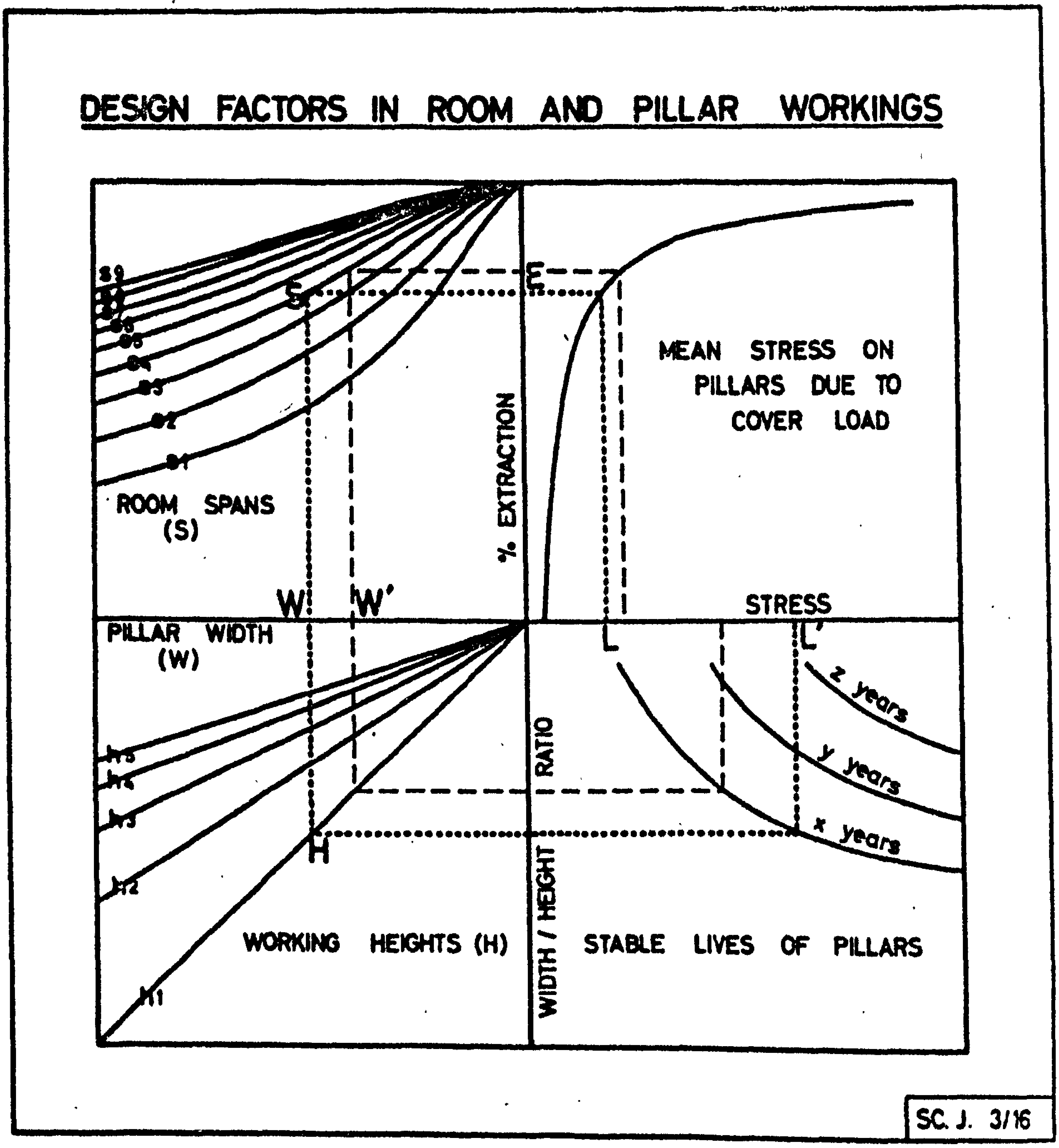
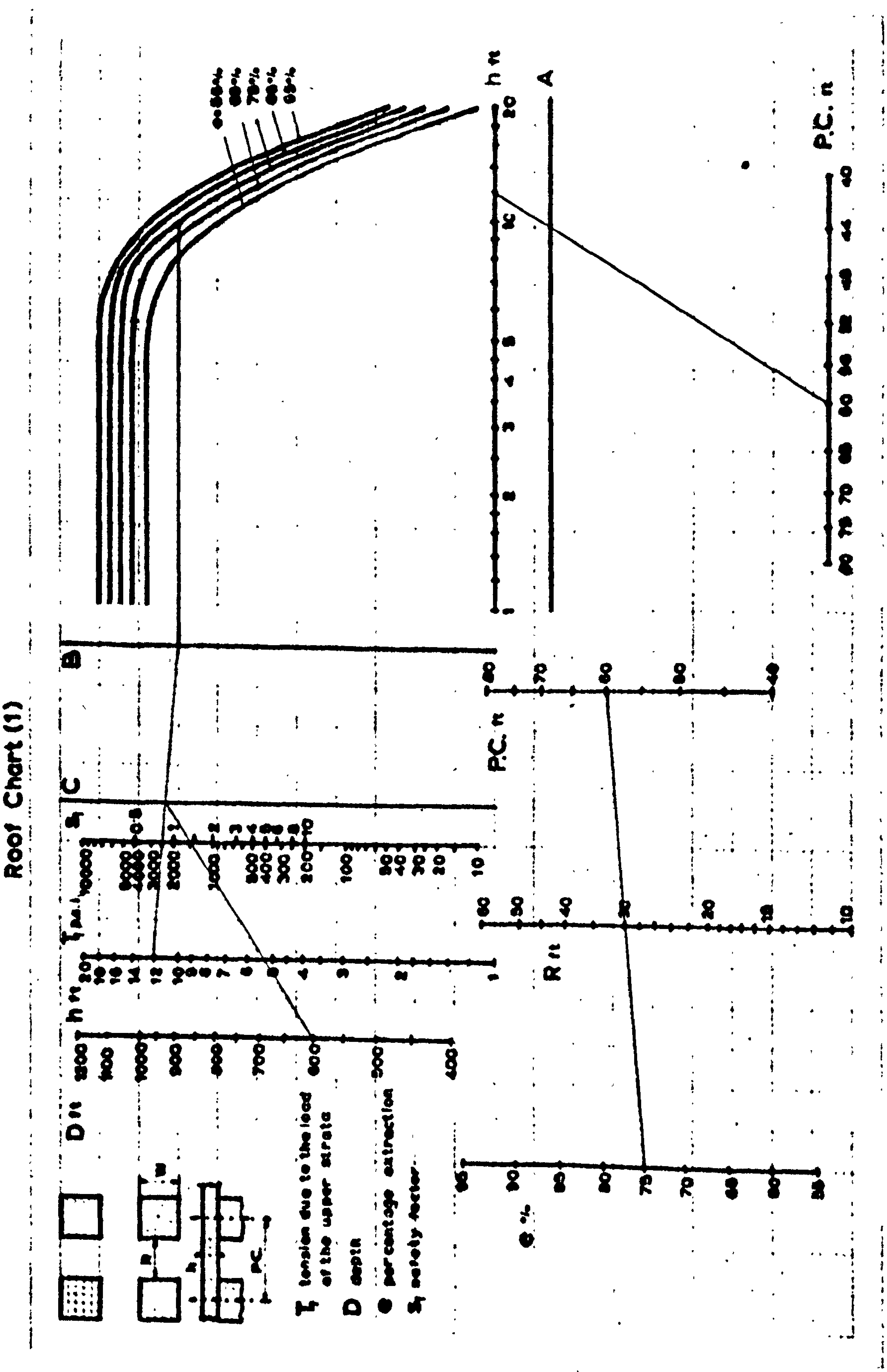

Mean stress on pillars due to cover load
STRESS(P.S.I)
Time Safe Stress
PERCENTAGE EXTRACTION
WIDTH/HEIGHT RATIO
Various working heights (feet)
Various spans S (feet)
PILLAR WIDTH (feet)

Fig. 13



DESIGN FACTORS IN ROOM AND PILLAR WORKINGS
ROOM SPANS
(S)
% EXTRACTION
MEAN STRESS ON PILLARS DUE TO COVER LOAD
STRESS
PILLAR WIDTH
(W)
W
W'
S
E
L
L'
H
RATIO
WIDTH / HEIGHT
z years
y years
x years
WORKING HEIGHTS (H)
STABLE LIVES OF PILLARS
SC. J. 3/16

Fig. 14



Roof Chart (1)
T_t tension due to the load of the upper strata
D depth
e percentage extraction
S_f safety factor
D ft
h ft
R ft
P.C. ft
e %
A
B
C

Fig. 15

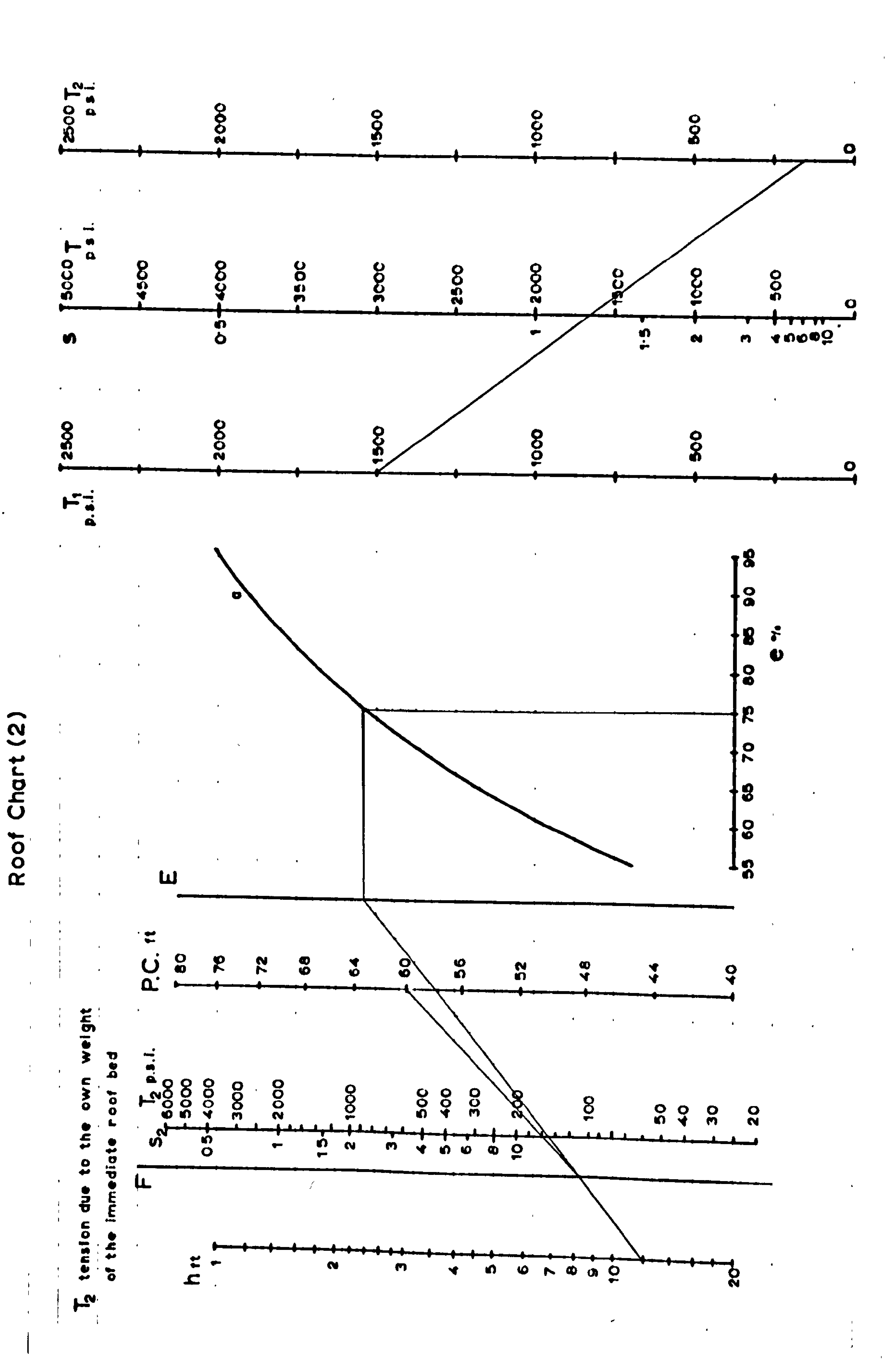

Roof Chart (2)
T_2 tension due to the own weight of the immediate roof bed
h ft
F
S_2
T_2 p.s.i.
P.C. ft
E
e %
a
T_1 p.s.i.
S
T p.s.i.
T_2 p.s.i.

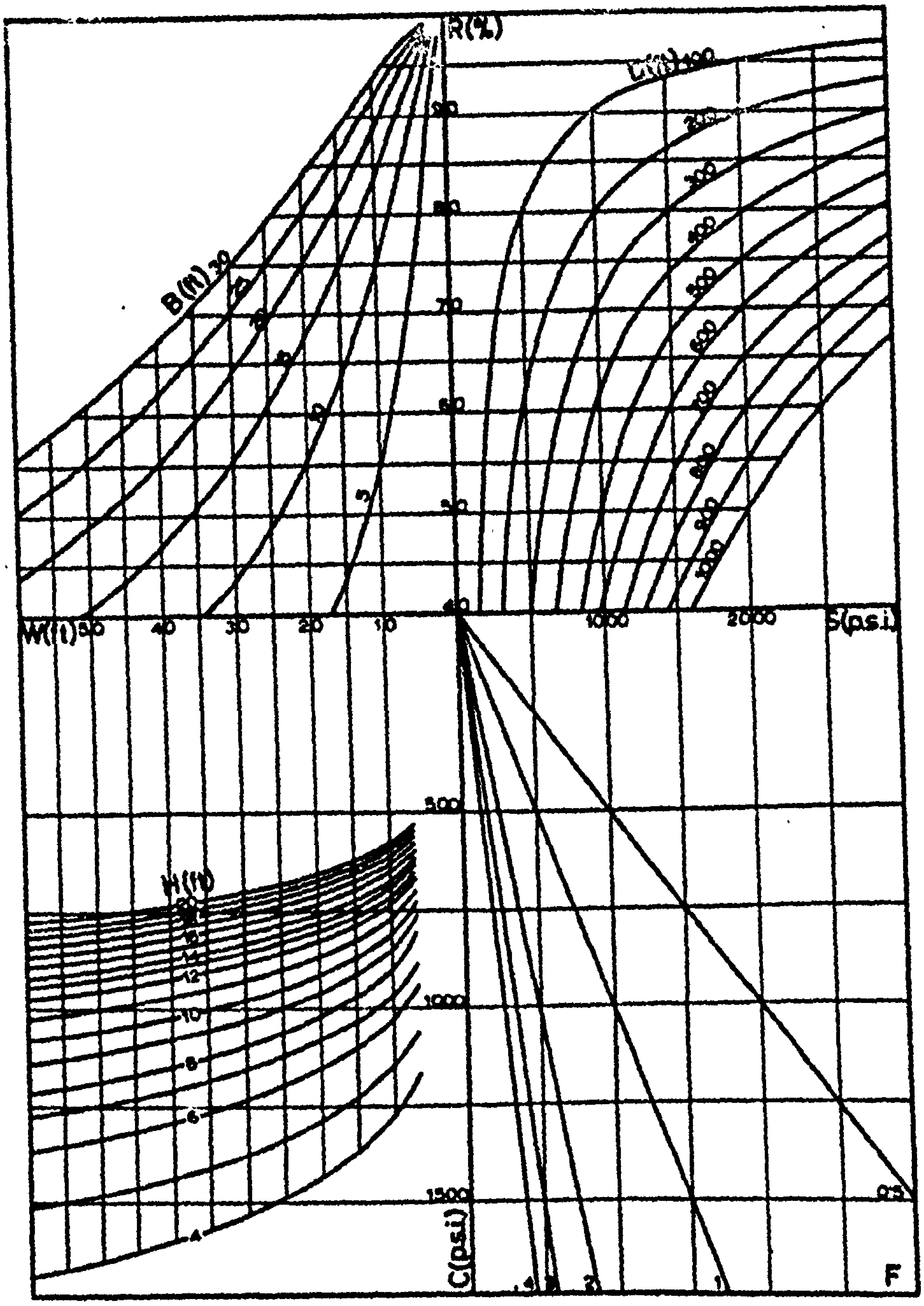
Fig. 16

length of time. This method, termed the 'Time-Safe Stress Concept', consists of a quadrant diagram, Fig.13, which correlates pillar width, working height, percentage area of extraction, room span, estimated pillar load, and time-safe stress. The part of the quadrant to the right hand side of the time stress curve represents unstable conditions, whereas that to the left, stable conditions for pillars.

If the time-safe-strain method of design, Pótts (53) and Hedley (54), is used instead of that of time-safe-stress, then a similar type of nomograph can be drawn as shown in Fig.14. Only the lower right quadrant is different. Instead of one curve separating stable and unstable conditions, a number of curves can be drawn which indicate the stable lives of pillars of different width/height ratios and under different stresses.

Salamon (70), constructed three nomographs for determining the state of stability of the rooms and pillars at an anhydrite mine. The nomographs, Figs.15, 16 and 17, were constructed on the principle that the principal dimensions of the then present and projected mining development were known, and that the existing future stability of the mine was to be examined. The roof charts were based on theoretical formulae derived by Salamon to determine the tensile stresses induced in the roof beam by mining. Roof Chart 1, Fig.15, shows the tensile stress due to the load of the upper strata. Roof Chart 2, Fig.16, shows the tensile stress induced by the weight of the immediate bed only. The nomographs can also be used to determine the maximum safe roof span or extraction rate, but were primarily intended for determining mine stability.

A number of investigators, Taylor (73), Potts and Szeki (14), Bieniawski (4) and Salamon and Munró (3), produced nomographs for the evaluation of design data for room and pillar mining in the coal mines of South Africa. Taylor (73) produced several quadrant diagrams based on the ultimate compressive strength concept. The diagrams correlated percentage area extraction, imposed stress, room span, pillar width, width/height ratio and working height, along with laboratory determined values of

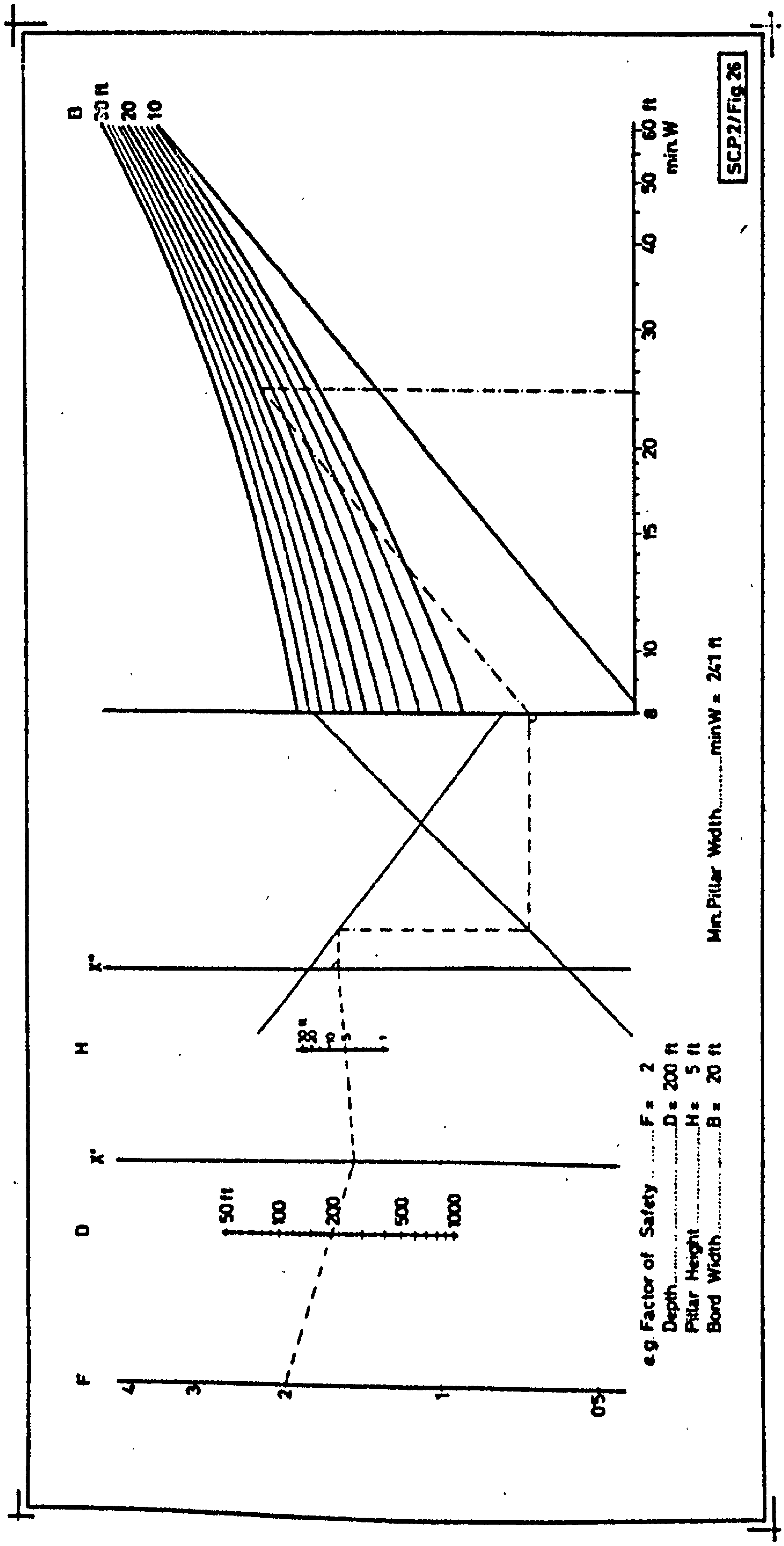


Pillar Width..........W
Working Height..........H
Factor of Safety..........F
Stress on Pillars..........S

Bord Width..........B
Depth..........D
Rate of Extraction..........R
Crushing Strength..........C

SC.P.2/ Fig.25

Fig. 18



SCP.2/Fig.26
D
30 ft
20
10
8
10
15
20
30
40
50
60 ft
min.W
Min. Pillar Width.........min W = 24.1 ft
e.g. Factor of Safety ....F = 2
Depth.........D = 200 ft
Pillar Height.........H = 5 ft
Bord Width.........B = 20 ft
F
D
H
X'
X''
50 ft
100
200
500
1000
4
3
2
1
0.5

Fig. 19

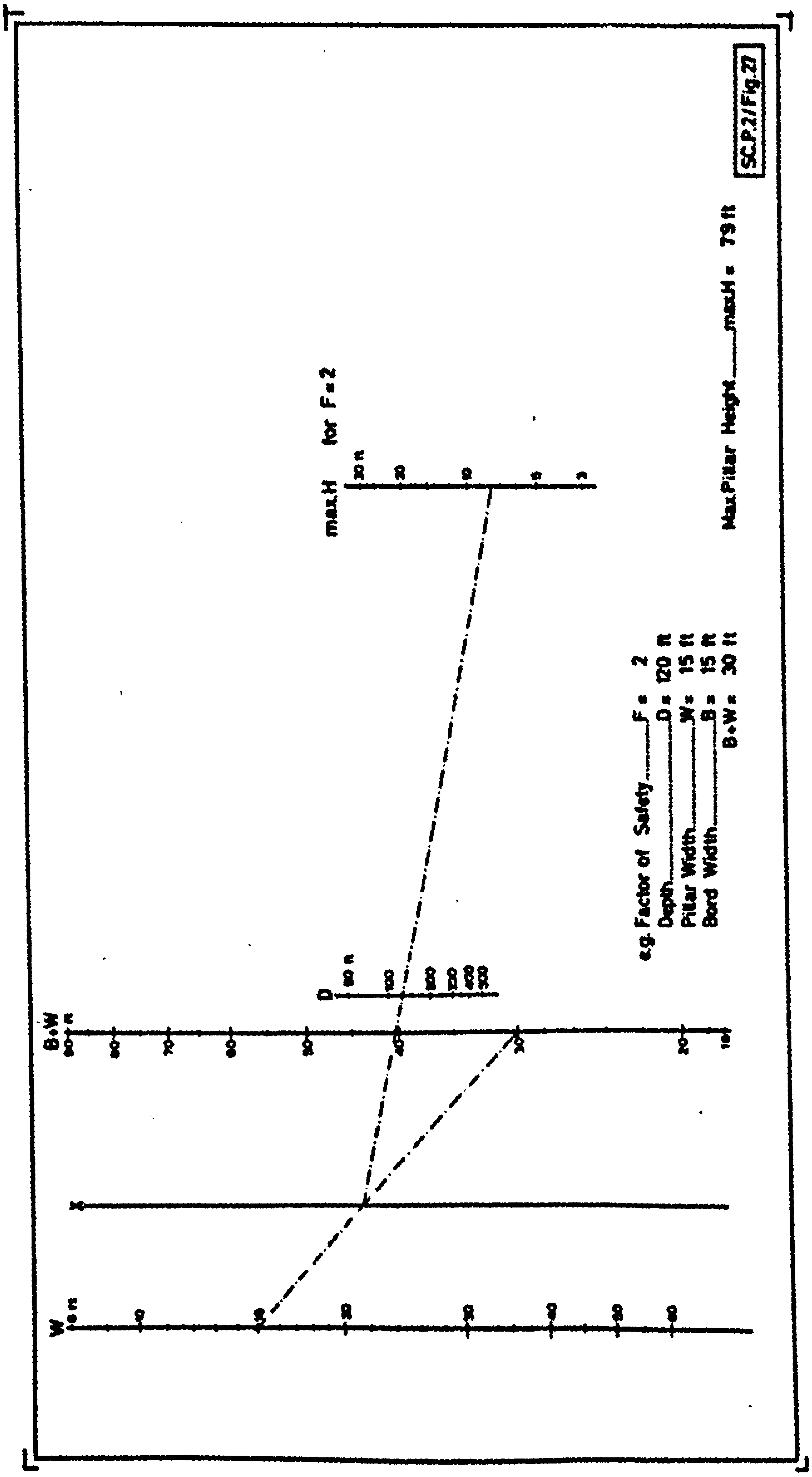

W
B+W
X
D
max.H for F = 2
e.g. Factor of Safety.........F = 2
Depth.........D = 120 ft
Pillar Width.........W = 15 ft
Bord Width.........B = 15 ft
B+W = 30 ft
Max.Pillar Height.........max.H = 7.9 ft
SC.P.2/Fig.27

Fig. 20

ultimate compressive strengths of specimens having differing width/height ratios.

Potts and Szeki (14) produced a modified version of the Taylor nomograph (73), based on a formula obtained in the laboratory for the minimum strength of any size of coal pillar,(see Table 3), and incorporating factor of safety data. This quadrant nomograph shown in Fig.18, differed from Taylor's in two respects. The failure strength of the pillar was not expressed as a function of width/height ratio, but the actual pillar dimensions were introduced. Also, the actual stresses on the pillars were compared with the theoretical rupture strength in the factor of safety quadrant. The nomograph itself was self-explanatory, and was applicable to existing development to determine the stability of the area mined.

To facilitate easier application of design data for planning new development, the nomographs shown in Figs.19 and 20 were included, to be used in conjunction with the quadrant nomograph shown in Fig.18. The nomograph, Fig.19, related the factor of safety with depth, working height, and room width in an easily discernable form. In planning for mechanised mining, it is usual that a minimum room width is defined. A corresponding minimum pillar width to provide a chosen factor of safety at a particular mining height and depth could then be defined. The values obtained can be transferred to the quadrant diagram, Fig.18, to determine the maximum rate of extraction. Conversely, in Fig.19, the nomograph provided the facility to relate the maximum pillar height at a chosen factor of safety of 2.

Salamon and Munro (3) constructed a nomograph, Fig.21, combining the influence of the depth of working, working height, pillar width, room width and safety factor, to be used to determine the safety factor of existing workings and the pillar width and height in proposed workings. In order to give the most effective guidance, some recommendations were made concerning the suitable value of safety factor which had to be made. A statistical analysis of stable and unstable workings was carried out and

DESIGN OF COAL PILLARS.

(SALAMON and MUNRO)

FIG. 21

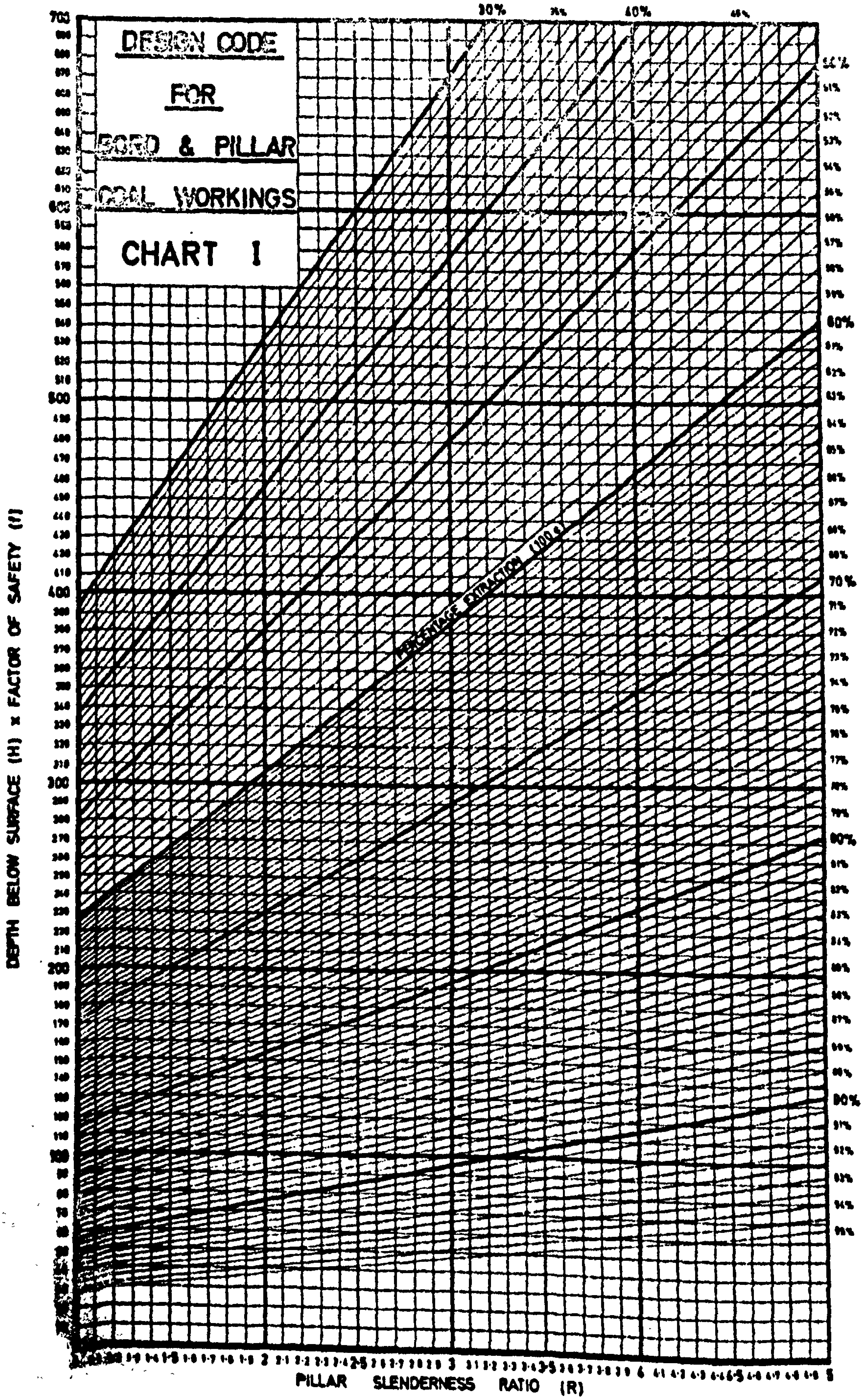
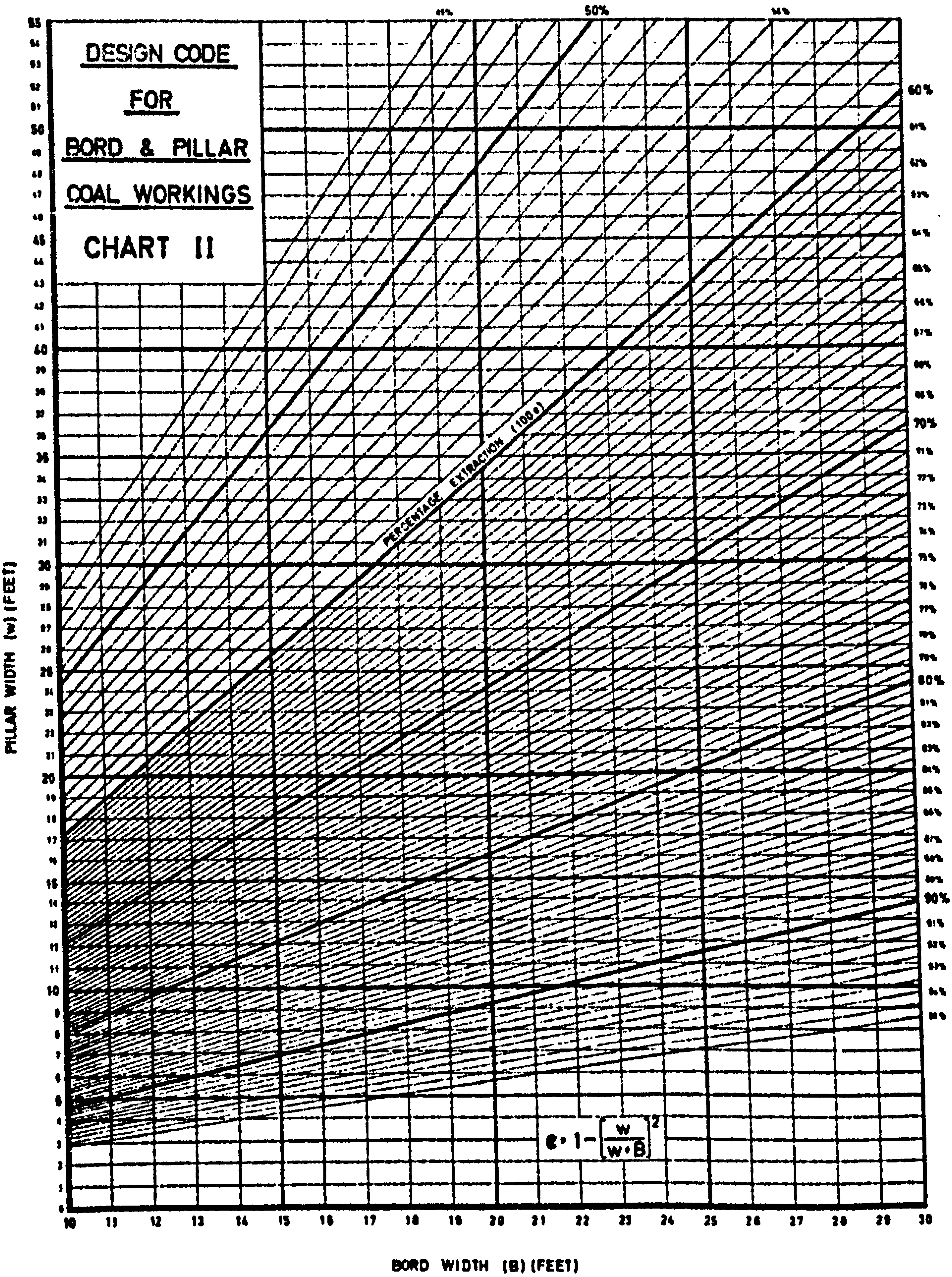

DESIGN CODE
FOR
BORD & PILLAR
COAL WORKINGS
CHART I
PERCENTAGE EXTRACTION (100 e)
DEPTH BELOW SURFACE (H) x FACTOR OF SAFETY (f)
PILLAR SLENDERNESS RATIO (R)

Fig. 22



DESIGN CODE
FOR
BORD & PILLAR
COAL WORKINGS
CHART II
PERCENTAGE EXTRACTION (100 e)
50%
60%
70%
80%
90%
$e = 1 - \left[\frac{w}{w+B}\right]^2$
PILLAR WIDTH (w) (FEET)
BORD WIDTH (B) (FEET)

Fig. 23

a pillar strength formula obtained (Section 3.2), together with a suggested optimum range of safety factors. It was suggested by Salamon and Wilson (71), that the optimum value of safety factor lies in the range where 50% of the stable cases were most densely concentrated (Section 3.4). This nomograph is now used by almost all collieries in the Transvaal and Orange Free State and has proved to be both reasonably reliable and practical.

Bieniawski (4) presented a nomograph which formed a basis for the design of new workings and for checking the stability of existing ones. The nomograph, split into two charts as shown in Figs.22 and 23, incorporated factor of safety data based on a number of 'in situ' compressive strength tests (Section 3.2), and the statistical knowledge obtained by Salamon and Wilson (71) concerning the state of stability of South African Collieries. These two charts are self explanatory. When new workings are to be designed, the pillar width or height - one must be determined independently from practical considerations - can be determined after the choice of a suitable factor of safety and the percentage extraction required. Having determined the pillar width from Chart 1, the room width can be determined from Chart II. When existing workings are to be checked, all the mining dimensions are known and the factor of safety can be determined.

4. ADDITIONAL STABILITY PROBLEMS ASSOCIATED WITH MULTI-SEAM AND INCLINED SEAM WORKING.

The two previous sections have been concerned with the determination of the design parameters and their effects on the actual state of stability of room and pillar workings in horizontal, or near horizontal, single seam mining systems. These have indicated the degree of importance, in the design of such workings of the various parameters, and the many attempts that have been made to investigate the effect of these design factors on the stability of the workings. Where the possibility of mining several seams in the same area or the inclination of the strata have to be considered however, additional factors are introduced into the state of stability of the workings. These additional stability problems can become rather complex if the combined effects of inclined and multiple seam working are introduced, as at one of the experimental sites used in this thesis.

4.1 . Multiple Seam Working.

As the result of the re-distribution of the load in room and pillar mining, two types of pressure zones are formed in the strata above and below the workings. A zone of high compressive stress is formed above, in and below the pillars, and a zone of tensile stress formed above and below the rooms. These zones develop more or less systematically during the mining operation, together with zones of high stress in front of the moving face line and on the rib-sides of the panel. These changes in the stress condition resulting from the partial extraction of the seam, causes corresponding displacements in the roof strata, the pillars and the floor strata. The magnitude of these stresses and strains are attenuated with increasing distance from the seam.

In the event of two or more seams being mined simultaneously in the same section of strata, each set of workings has the tendency to develop the above pattern of stresses and strains. If the distance between neighbouring seams is so large that the stress and strain peaks produced by each seam diminish, then most of the interference between each set of workings disappears. In this case the only remaining effect of working multiple seams would be the result of differential subsidence over the rib-sides of the lower seam. If however, the distance between the seams is quite small, then these stresses and strains surrounding each set of workings will be superimposed on one another, and their individual magnitudes will be affected by this proximity. In these circumstances, an unfortunate geometrical relationship between neighbouring seams, such as non-superimposition of pillars, can result in a very unfavourable distribution of the stresses and strains with very high cumulative magnitudes. Such a relationship could possibly induce failure of one, both, or even several sets of workings, even if each was individually stable.

Multiple seam mining, using room and pillar methods of working, is a relatively common system of mining either in thick, massive deposits such as rock salt or potash, or in a series of individual seams such as coal, gypsum,etc. The uncontrolled collapses thathave occurred in this system of mining were more common in the older mines where at least one seam was extensively mined without superimposing pillars, irrespective of the distance between the seams. In South Africa, where coal seams are found in close proximity to each other, superimposition of workings in each seam is strictly adhered to. When seams are more than 50 feet apart however, workings are generally not superimposed, although there appears to be no criteria on which this non-superimposition is based.

As recently as 1959 (10), a collapse of workings occurred where a seam, approximately 50 feet in thickness at a depth of 600 feet, was being mined near the top and bottom of the seam to a height of 18 feet leaving a 12 feet parting between the two sets of workings. In this particular incident, 16 acres of surface land subsided about 7 feet due to the failure in both levels of 52 feet square pillars, formed by 18 feet wide roadways. The major contributing cause of this collapse, given at the subsequent inquiry, was the non-superimposing of the pillars in one part of the area affected.

Investigations have been carried out to determine a better understanding of the various factors associated with this mining system. Kvapil (74) used a photoelastic technique to demonstrate the most stable layout of multiple horizon mining in a thick seam. The characteristic stress distribution around a vertically superimposed layout and a 'chess-board' pattern layout, were obtained and compared. The 'chess-board' layout of the rooms was seen to be less advantageous than the vertical superimposed layout, since the stresses between the room corners of successive horizons were adversely distributed and attained high values. As a result, the rock failed easily between the lower corners of the upper rooms and the upper corners of the lower rooms.

Field investigations were carried out in South Africa by Wilson (10) in conjunction with Potts (74), to obtain data concerning the mining of seams both over and below unmined and mined areas. One aim of the investigation was to define the displacement of the strata above and below the seams being mined. It was hoped that some data could be provided concerning the effects of overmining and undermining existing workings, by means of underground, inter-strata and surface strain measuring instruments. It was found however, that very little information was obtained concerning this phenomena and since this only played a small part in the overall objectives of the investigations carried out, no definite conclusions were drawn. It was thought that the seams were in fact too far apart for any mutual interference of the strata stresses and displacements induced by each seam.

It would appear that there are two basic methods of investigation by which a better understanding of this system of mining may be obtained. The first concerns the determination of the variation and magnitude of the stress and strain existing in a multiple seam area, and their attenuation with distance from each seam. This will define the minimum distance at which each set of workings can be considered to behave independently of one another. The second method concerns the determination of the magnitude and effect of the differential subsidence over the lower seam workings. This will enable a decision to be made concerning the correct sequence in which the seam should be mined. By carrying out these investigations it should then be possible to determine the optimum method and geometries of mining any number of seams in close proximity.

4.2 Inclined Seam Working.

The influence of the angle of inclination of the seam on the actual stability of the workings is a factor that is not as yet completely understood, and it has been the subject of a number of investigations. Basically, the lateral component of the primitive stress field (Section 3.1) acting on the pillars in an inclined seam, would tend to increase the value of the shearing stresses appearing in the pillars, as compared with the shearing stresses in a level seam. These shear stresses are not normally considerable when compared with the mean stresses normal to the stratification plane, but under certain unfavourable conditions these stresses could result in pillar instability. A better understanding of the factors associated with room and pillar mining in an inclined seam, can probably be obtained by briefly describing an investigation that has been carried out into this phenomena.

The investigational techniques used have been confined to the laboratory, the photoelastic technique mainly being utilised to determine the stress distribution, both in the pillar and in the surrounding strata. This technique has been used by Tsimbarevich (75), and Trumbachev and Mel'nikov (76),the former using a glycerine gelatine base substance as an optically sensitive material, and the latter using models made of igdantine and celluloid. Trumbachev and Mel'nikov simulated a room and pillar mining system to study, amongst other things, the influence of the angle of inclination of the seam upon the stressed condition of the simulated rock material. The distribution of the stresses formed in the pillars were studied for angles of inclination of 30°, 45°, 60° and 90°. The data obtained indicated that the stress condition of the pillars substantially differed as the angle of inclination increased.

At an inclination of 30°, it was found that the mean vertical section of the pillar was the weakest point. At an inclination of 60°, the pillars were considerably, unevenly loaded, the most loaded parts being at the roof on the up-dipside and at the floor on the down-dipside. At the same time, the other two opposite corners were underloaded, and the supporting strength of the pillar was not fully utilised.

The investigations were taking further to study the influence of the lateral primitive stress and the shape of the pillars, upon their stressed condition. For the conclusions on these aspects, the literature should be referred to.

As mentioned previously, one of the experimental sites used by the author, concerned the possible pillar stability of an inclined, duplex seam mining system. As a result, a combination of the two stability problems described in this section, was investigated, and is described in Part 2 of this thesis. It was originally hoped that 'in situ' investigations could be carried out at this mine, but for reasons beyond the control of the author this was not possible, and the investigations had to be confined to the laboratory, though with some considerable success.

PART 2.

THE EXPERIMENTAL SITES.

A. THE SHERBURN-IN-ELMET MINE.

1. DESCRIPTION OF THE GENERAL CONDITIONS AT THE SHERBURN MINE.

1.1 Geology of the Mining Area.

The new mine at Sherburn-in-Elmet is situated in the extensive post-glacial plains which cover a large area of the East Riding of Yorkshire. These plains extend inland from the coast for a distance of about 15 miles and are an extremely prosperous farming area. The gypsum deposit being mined at Sherburn dates from Permian times and it is considered to have been associated with the western-most edges of the ancient Zechstein Sea, which was also responsible for the formation of the extensive evaporite deposits in north-east Yorkshire. The gypsum seam itself, which varies in thickness from 10 to 20 feet, is extremely pure and is found at comparatively shallow depths. In the Sherburn area the overburden is only 120 feet thick.

The shallowness of the deposit however, tends to give a somewhat false idea of the magnitude of the problems associated with its extraction, these problems are partly natural and partly dependent on the existence of conflicting interests in the vicinity of the mine. They do however, all stem from the nature of the overlying strata. As has already been mentioned, the surface deposits are post-glacial and extend down from the surface to a depth of approximately 75 feet. From the bottom of the post-glacial deposits to the seam, a distance of approximately 45 feet, the strata consists of what has been termed Upper Permian Marl which, although inherently incompetent material, is appreciably stronger than the post-glacial material which, to all intents and purposes, may be considered to have negligible strength. The precise nature of the strata may be seen in the stratigraphical column shown in Figs.24 - 26.

The upper 75 feet of post-glacial strata are essentially lake deposits, and consist of varve clays, fine silts and sands, the varves becoming more silty with depth until an area of almost pure laminated silt is encountered at a depth of approximately 70 feet. Despite the fact that the presence of the large amounts of clay in these deposits

SHERBURN IN ELMET.

LOG OF EXPLORATION SHAFT NO. 1.

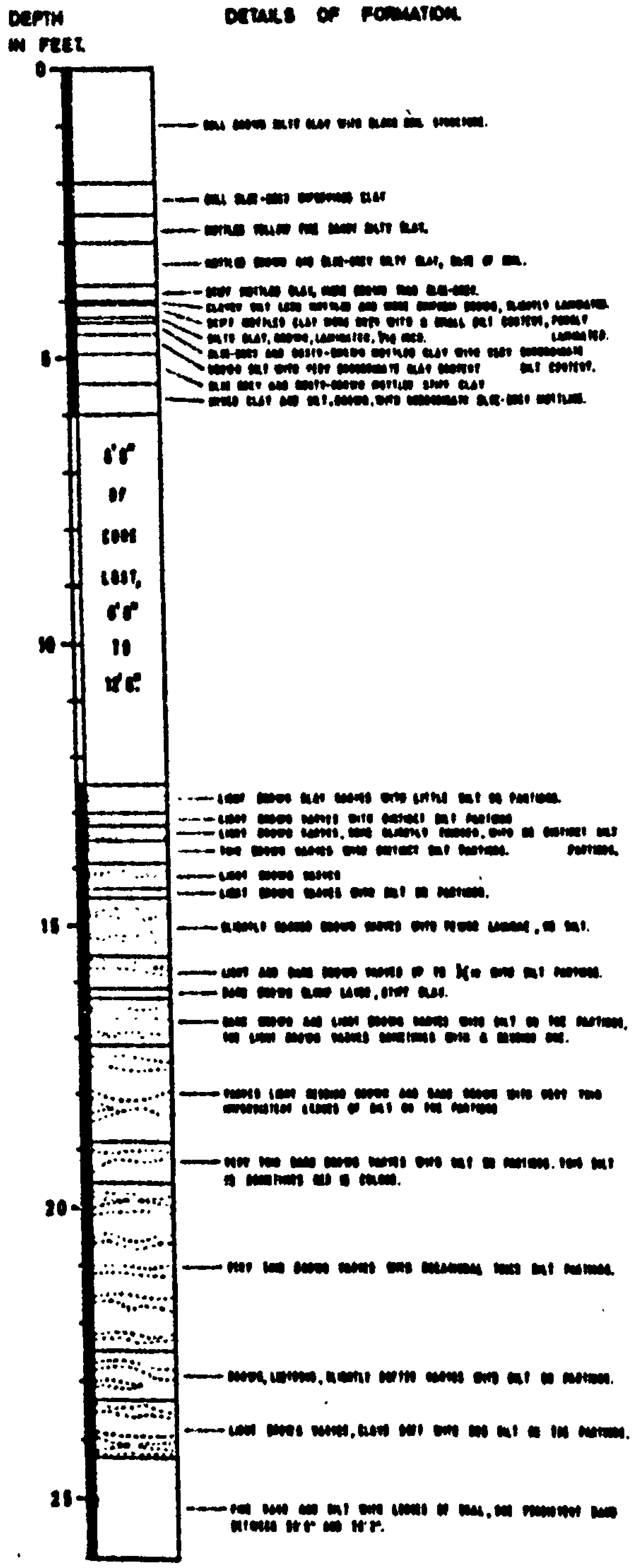


Fig. 24

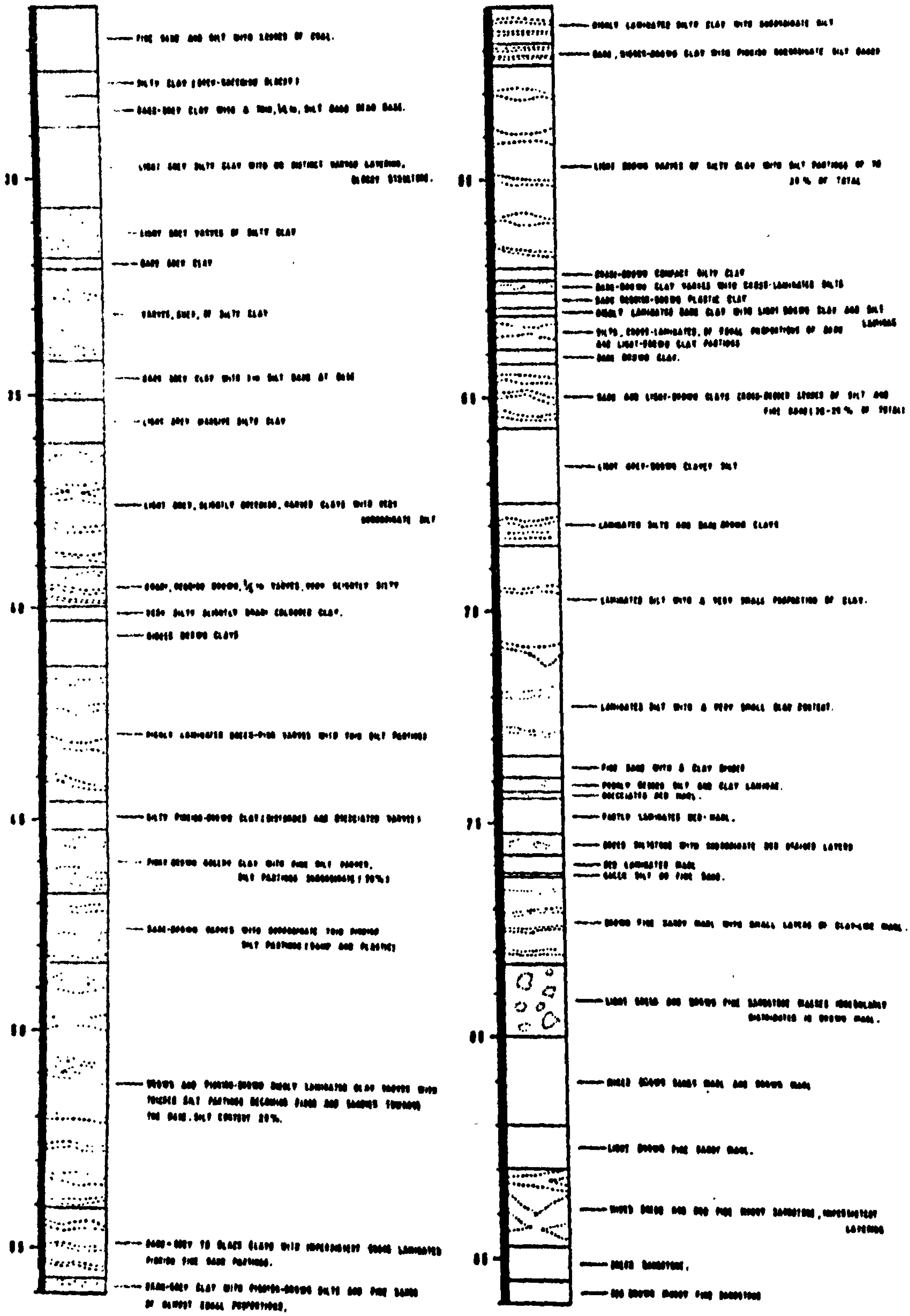


Fig. 25

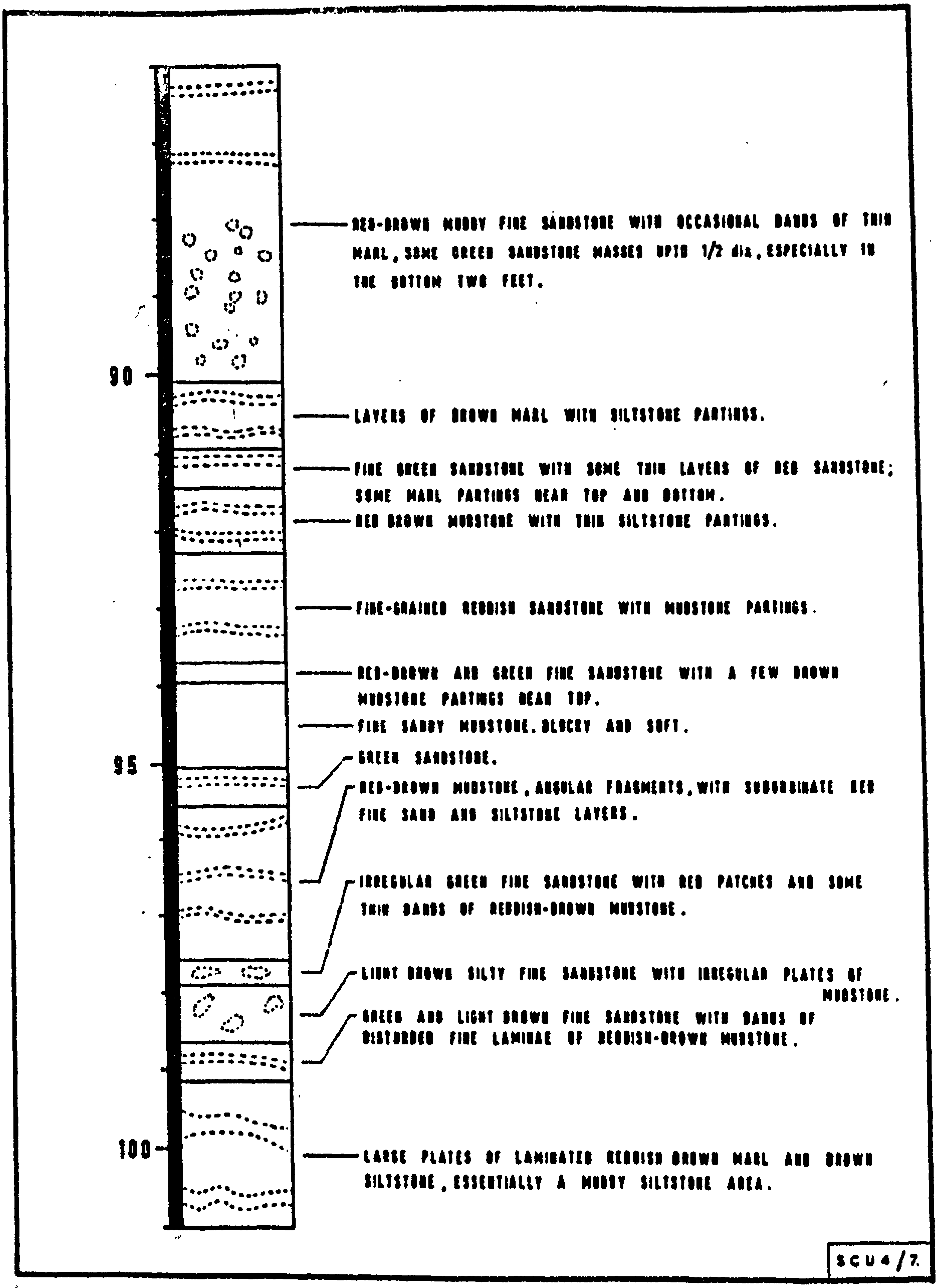
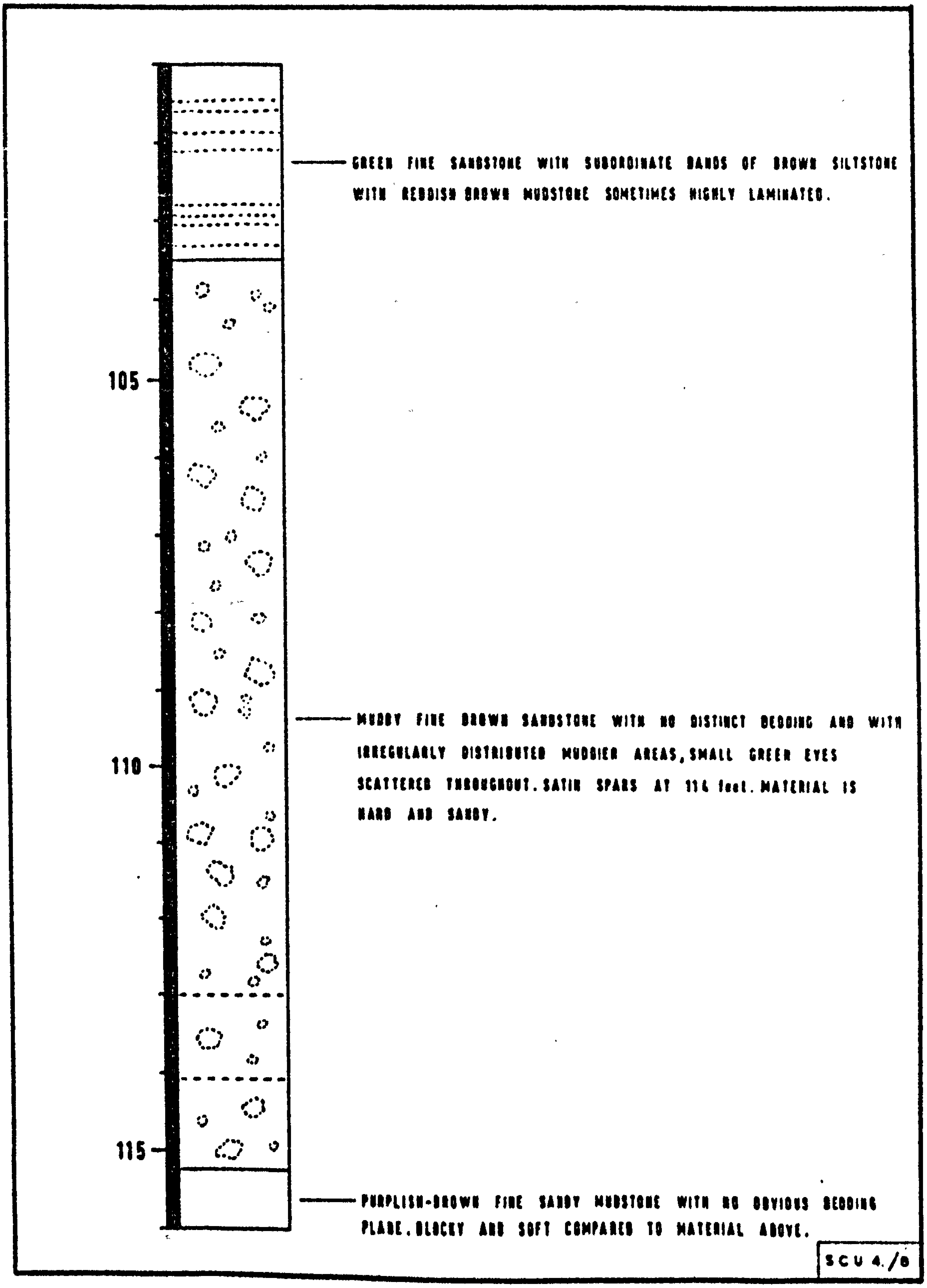
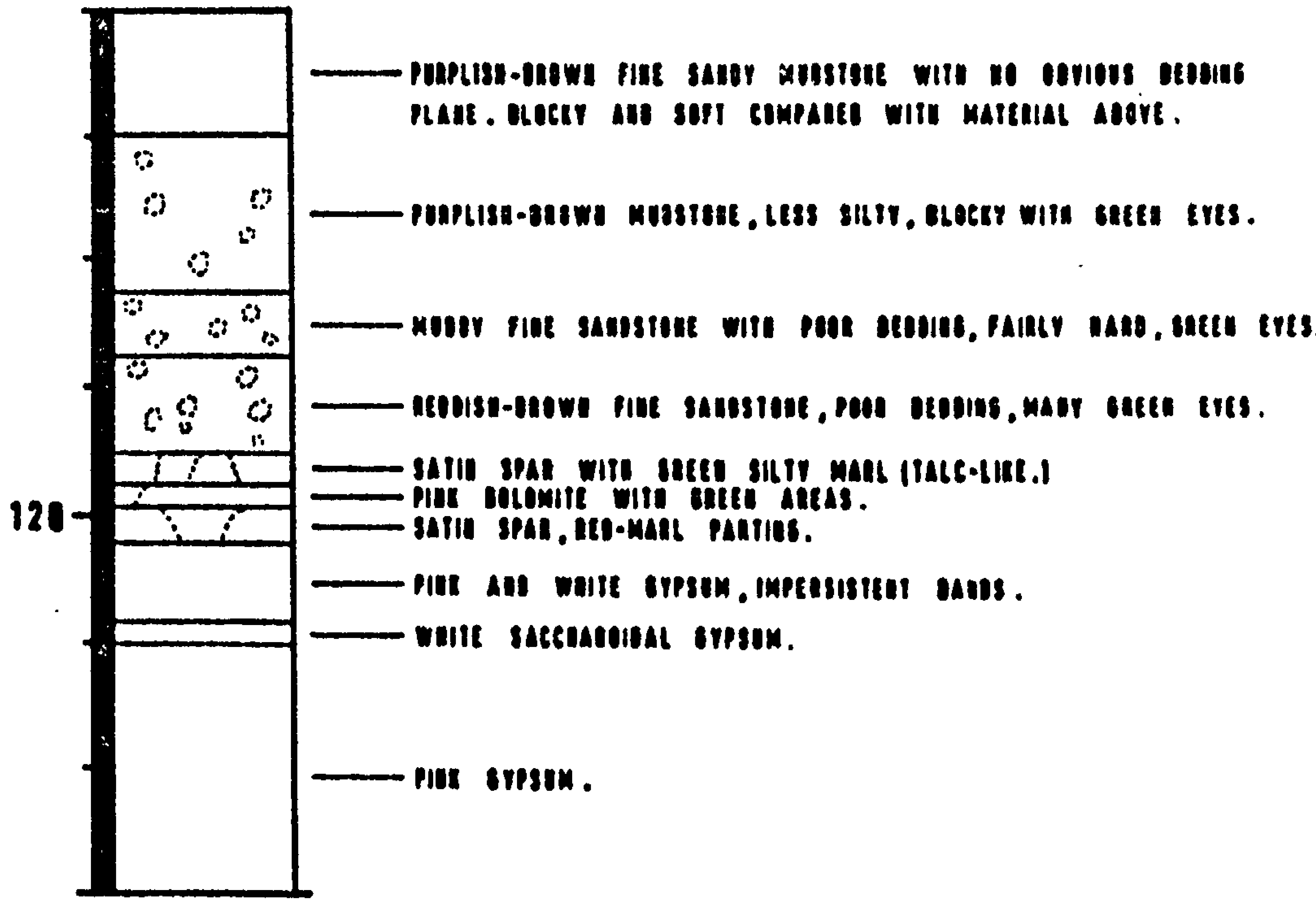

90
95
100
RED-BROWN MUDDY FINE SANDSTONE WITH OCCASIONAL BANDS OF THIN MARL, SOME GREEN SANDSTONE MASSES UPTO 1/2 dia, ESPECIALLY IN THE BOTTOM TWO FEET.
LAYERS OF BROWN MARL WITH SILTSTONE PARTINGS.
FINE GREEN SANDSTONE WITH SOME THIN LAYERS OF RED SANDSTONE; SOME MARL PARTINGS NEAR TOP AND BOTTOM.
RED BROWN MUDSTONE WITH THIN SILTSTONE PARTINGS.
FINE-GRAINED REDDISH SANDSTONE WITH MUDSTONE PARTINGS.
RED-BROWN AND GREEN FINE SANDSTONE WITH A FEW BROWN MUDSTONE PARTINGS NEAR TOP.
FINE SANDY MUDSTONE, BLOCKY AND SOFT.
GREEN SANDSTONE.
RED-BROWN MUDSTONE, ANGULAR FRAGMENTS, WITH SUBORDINATE RED FINE SAND AND SILTSTONE LAYERS.
IRREGULAR GREEN FINE SANDSTONE WITH RED PATCHES AND SOME THIN BANDS OF REDDISH-BROWN MUDSTONE.
LIGHT BROWN SILTY FINE SANDSTONE WITH IRREGULAR PLATES OF MUDSTONE.
GREEN AND LIGHT BROWN FINE SANDSTONE WITH BANDS OF DISTURBED FINE LAMINAE OF REDDISH-BROWN MUDSTONE.
LARGE PLATES OF LAMINATED REDDISH BROWN MARL AND BROWN SILTSTONE, ESSENTIALLY A MUDDY SILTSTONE AREA.
SCU4/7.

Fig. 26a



105
110
115
GREEN FINE SANDSTONE WITH SUBORDINATE BANDS OF BROWN SILTSTONE WITH REDDISH BROWN MUDSTONE SOMETIMES HIGHLY LAMINATED.
MUDDY FINE BROWN SANDSTONE WITH NO DISTINCT BEDDING AND WITH IRREGULARLY DISTRIBUTED MUDDIER AREAS,SMALL GREEN EYES SCATTERED THROUGHOUT.SATIN SPARS AT 114 feet.MATERIAL IS HARD AND SANDY.
PURPLISH-BROWN FINE SANDY MUDSTONE WITH NO OBVIOUS BEDDING PLANE.BLOCKY AND SOFT COMPARED TO MATERIAL ABOVE.
SCU 4./6

Fig. 26b



SCU 4/9

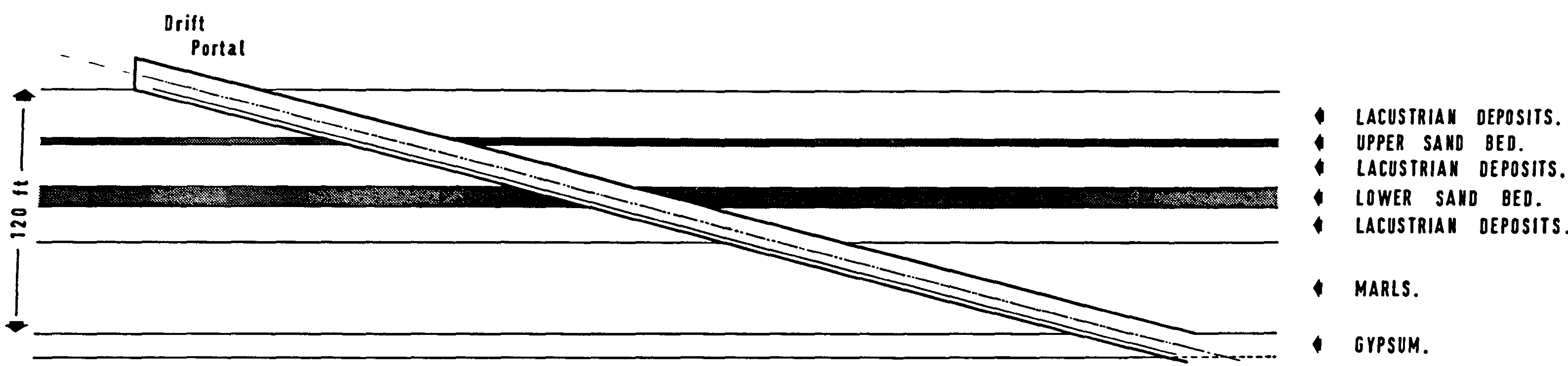
Fig. 26c

renders them almost completely impervious to the vertical movement of water, there are two continuous beds of fine sand which readily permit its lateral movement. These beds play an important role in the design of the mine workings, hence it will be as well to name them and establish their positions. The upper one, 3 feet thick, is centred at a depth of 27 feet and is referred to as the Upper Sand Bed. The lower one, 10 feet thick, is centred at 65 feet and is known as the Lower Sand Bed. The relative position of these sand beds and a stylised section of the main access drift is shown in Fig.27.

It is the presence of these sand beds which presents one of the natural problems. Any disturbance of the beds causes them to become fluid, as one abortive attempt to sink a shaft had shown. In this condition they present an extremely serious hazard to any unprotected excavation to which they have access.

The 45 feet of strata below these lake deposits and above the top of the gypsum seam, at a depth of approximately 120 feet, has been termed Upper Permian Marl, but such a description as marl is most erroneous. It has partial resemblances to Permian Marl, but it is essentially a very mixed strata of muddy fine sandstones, pure mudstone and silty mudstone with gradations between. In general, these mudstone beds would not permit the vertical passage of water, though porosity and permeability is slightly higher in some of the thin sandstone beds in spite of the interstitial mud tending to reduce the permeability.

A. soft mudstone is found just above the seam, between 115 feet and 118 feet from the surface. This is rather soft and well jointed but of a very low permeability. This mudstone becomes more sandy below and is a sandstone from 118 feet to 119 feet 6 ins. This sandstone is porous and underlain by a thick bed of satinspar, erosion of which could lead to a channel for water. Just above the seam, but



SCHEMATIC SECTION OF SHERBURN DRIFT SHOWING THE POSITION OF THE UNSTABLE UPPER AND LOWER SAND BEDS.

Fig. 27

separated from the seam by a lower bed of satinspar, is a pink and green mixed dolomite and gypsum band.

The seam itself is on average 14 feet - 20 feet thick, and consists for the most part of extremely pure, high quality gypsum, though there are variable thicknesses of anhydrite at the base of the seam. Physically, the gypsum occurs as a massive pink crystalline material with a few minor faults, and some thin bands of satinspar. Fairly thin fragments are clearly translucent, and even at the interface with the overlying marls there is no deterioration in quality.

The seam is situated 35 feet above the thick Upper Magnesian Limestone. The intervening strata consists of varying thicknesses of red mudstone with gypsum spars, and blue and grey shales.

The structure of the area is one of a general dip just south of east, cut by a number of faults of throw varying in magnitude from 15 feet to 100 feet. The East-West strike of the faults invariably gives a block nature to the structure. The mine at present is confined to one of these blocks and is not affected by major faults, though future development will necessitate a change in mining horizon due to the presence of these faults.

1.2 Preliminary Appreciation of the Natural Hazards.

The majority of the problems which exist at this mine can be related to the nature of the overlying material and the ability of the upper and lower sand beds to flow given the least opportunity to do so. These sand beds constitute a hazard which is extremely difficult to assess with certainty, but the presence of 10 feet of unconsolidated sand and silt lying at a distance of approximately 50 feet above the seam on what was most likely the original surface of erosion, necessitates consideration being given to the probability that this could, under certain circumstances, have access to the workings.

In addition, the upper sand bed has been recorded flowing into boreholes. However, between the upper and lower sand beds, the fine silty varved clays can possibly be expected to prevent the vertical movement of the upper sand bed downwards. Though, if the lower sand bed does flow at any point, the possibility must be considered that these intervening beds may yield sufficiently and could provide access of this upper fine sand to a lower level.

A further factor that had to be considered in the preliminary design of the mine workings was the distribution of water around the workings. Preliminary boreholes which had penetrated the seam and the underlying material had indicated that an aquifer lay below the gypsum and that there was an artesian head of approximately 170 feet. Attempts were made to establish whether an appreciable volume of water could flow from this aquifer, which was established at a distance of approximately 150 feet below the gypsum seam, but it was felt that the intervening marl would prevent access of the artesian water from the aquifer to the seam even though a thick limestone bed was situated only 35 feet below the seam. However, there is a possibility that lines of faulting might provide the means of access of water to higher permeable horizons.

Varying amounts of water had also been found in exploratory boreholes in the strata overlying the seam, suggesting that there was probably one general water source for the strata above the seam, and for the water that had been encountered in the seam itself, the most likely source being a limestone escarpment towards the outcrop. If this is correct and there is such a source, then if the water taken from the mine by pumping exceeds the amount of rainfall and percolation, it could result in a slow dewatering of the overlying strata. This dewatering, of the sand beds in particular, might show itself as slight subsidence on the surface, brought about by the compaction due to the weight of the beds above this sand as the water is removed.

Knowledge of the unstable structure overlying the workings caused the Inspectors of Mines to impose certain restrictions on the working dimensions of the seam, these included :-

(i) Room and pillar workings should be used.

(ii) The roadways should not be more than 15 feet in width or 10 feet high, and at least 6 feet of gypsum should be left between the roof of the workings and the gypsum-marl interface.

(iii) The workings should be developed by means of sets of parallel advance headings, each set comprising not more than five parallel headings.

Further restrictions were imposed upon the mine owners by the local Planning Authority. Before granting planning permission for the mine, the Authority required an assurance that there would be no ground movement at the surface. The reason which lay behind the imposition of this condition is again connected with the nature of the upper strata, since the material forming the surface layers form a very large extent of flat, valuable farming land. Because of the extent of the plain an extremely complex system of artificial drainage ditches has been constructed to facilitate the removal of excess ground water. The Planning Authority feared that if the method of mining the gypsum caused the surface to subside, there would be an immediate tendency for the subsidence basin to flood. In addition to this, should there be drainage ditches passing through the affected area, the water flow in them would be either impeded or reversed, with extremely deletrious effects on the overall drainage pattern.

In view of these conditions, it was decided that whatever layout of room and pillar mining was employed the strata overlying the deposit would have to be disturbed as little as possible in order that there should be negligible vertical movement at the surface. Therefore, even before development work commenced at the mine, and before the results of laboratory testing became available, certain conditions relating to the design of the workings became apparent.

To prevent the possible ingress of water into the workings from the aquifer below the seam, it was decided that at least two feet of gypsum would be left at the bottom of the seam to act as a water barrier. It has been found in fact, that at least four feet of gypsum has been left in the floor during the initial development whilst conforming to the dimensions laid down by the Inspectorate. At least one benefit has occurred from the presence of the gypsum floor in the workings, in that the movement of machinery has been much easier than it would have been on the underlying marl.

Thus, the situation prior to mining was that the mine workings would be completely contained within the gypsum seam and the method of extraction used would effectively produce no surface subsidence. This latter condition meant in effect, that the mine workings would have to be stable on a long term basis.

2. METHODS OF OBTAINING DATA RELATING TO THE STABILITY OF THE MINE.

Extensive laboratory investigations were carried out by Jones (5) to determine the values of those mechanical and physical properties which, it was felt, would have particular relevance as far as their effect on the stability of the mine workings was concerned. The rock testing programme was designed to yield statistically significant values for the strength properties of the gypsum making up the deposit so that the values could be utilised in any theoretical appreciation of the mine stability that may be carried out. The mine stability problem at Sherburn lends itself to a possible idealised approach for a number of reasons, these are :-

a) The material making up the deposit is uniform.

b) The boundaries of the deposit and the limits of the working zone are sharply defined.

c) The external loads which are applied to the deposit may be estimated quite precisely. This will be discussed later.

Given these conditions it was hoped that a theoretical appreciation could be produced which would prove capable of being applied directly to the mine.

A brief synopsis of the results obtained together with any conclusions that were made is given here.

2.1 Synopsis of Data Obtained from the Compression Testing of the Gypsum.

The compressive testing programme carried out on the seam material was extremely extensive, not because the material was inhomogenous and therefore necessitated the testing of many samples, but because investigations were carried out into the effect of varying both the W/H ratio of the specimens and their volume. These enquiries were carried out in order to establish to what extent the ultimate compressive strength of the gypsum varied as the W/H ratio or the specimen volume changed. A complete record of the uniaxial compressive strength data relating to Sherburn gypsum is given below :-

Total number of specimens tested	191	
Number of specimens, 3" dia. x 6" long	15	Group (i)
Number of specimens, 1.625" dia. x 3.2" long	10	Group (ii)
Number of specimens of varying W/H ratio, diameter constant at 1.625"	83	
Number of specimens of varying W/H ratio, diameter constant at 3.2"	47	
Number of specimens of W/H ratio 0.5, but of varying dimensions	36	Group (iii)
Mean ultimate compressive strength of Group (i)	7123 p.s.i.	
Mean ultimate compressive strength of Group (ii)	6237 p.s.i.	
Mean ultimate compressive strength of Group (iii)	5862 p.s.i.	

Mean ultimate compressive strength of all specimens having a W/H ratio of 0.5	6234 p.s.i.
Standard deviation of total	± 791 p.s.i.

Jones (5) suggested that the figure obtained in the laboratory for the compressive strength of the gypsum could be applicable as a basis for establishing the 'in situ' strength, as it was thought reasonable to suggest from the results that the gypsum strength was not particularly sensitive to specimen size. The justification for this conclusion was based upon the results obtained and the work of Bieniawski (4), Denkhaus (51) and Protodyakanov (63). In addition to this, the W/H ratio of the pillars will be much greater than that of any of the specimens tested. Therefore, it is likely that there will be more lateral restraint applied to the pillars when they are loaded, than there was on the laboratory specimens. The main effect of this would be to proportionately increase the degree of compressive triaxial loading on the pillar core, and hence raise its effective ultimate compressive strength.

2.2 Synopsis of Data Obtained from the Tensile Testing of the Gypsum.

The method used to determine the ultimate tensile strength of the gypsum was an indirect one commonly referred to as the Brazilian Disc Test; this test was used exclusively by Jones (5) and by the author in investigations to be described later.

Total number of tensile specimens	172	
Number of specimens cut from cores obtained from Surface Borehole 1/51	89	Group 1
Number of specimens cut from blocks obtained from the working zone	66	Group 2
Number of specimens cut from cores obtained from horizontal seam boreholes	17	Group 3

Mean tensile strength of Group 1	411 p.s.i.
Mean tensile strength of Group 2	513 p.s.i.
Mean tensile strength of Group 3	420 p.s.i.
Mean ultimate tensile strength of the total sample	451 p.s.i.
Standard deviation of total sample	76 p.s.i.

2.3 An Appreciation of the Stability of the Mine Workings.

In 1965, work commenced on the construction of an inclined drift which would provide primary access to the mine workings, and in conjunction with a previously constructed monolithic reinforced concrete shaft, provide an intake airway for the mine ventilation. The difficulties encountered during the construction of this drift and the investigations carried out to determine its subsequent stability have been described by Jones (5). In 1966, this drift connected with the initial underground development, but in the summer of 1966, the underground development work ceased and did not recommence for a further two years.

A plan of the development workings showing the stages in mine development at approximately 6 monthly intervals up to July 1970, is shown in Fig.28. It will be noticed that from July 1968, the mine was developed initially by means of a set of 5 parallel development headings, referred to as 1 - 5 South, driven Eastwards to form the Southern boundary of a 600 ft.square central panel. These headings were each 15 ft. wide and 10 ft. high, whilst the pillars were 65 ft. long by 25 ft. wide, giving a 50% extraction. These headings were formed by means of repetitive drilling and blasting. The level of the workings was governed by the requirement that a minimum of 6 ft. of gypsum had to be left between the roof of the workings and the overlying strata. To ensure this, inclined roof boreholes were drilled every 10 - 15 ft. as the face of each heading advanced, each such borehole being drilled as near as practicable to the face. In

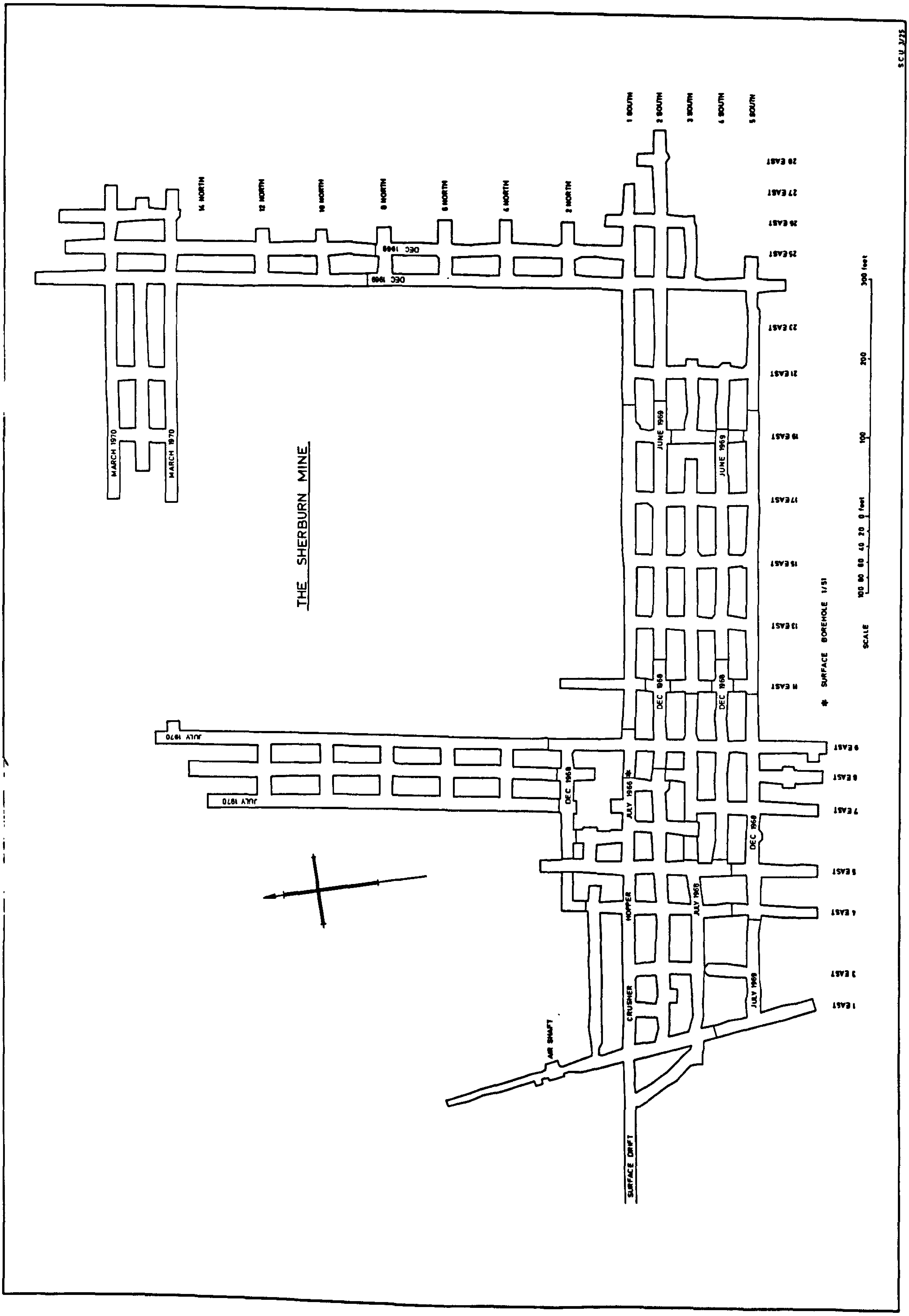

THE SHERBURN MINE
SURFACE DRIFT
CRUSHER
HOPPER
AIR SHAFT
MARCH 1970
MARCH 1970
JULY 1970
JULY 1970
DEC 1968
DEC 1968
JUNE 1969
JUNE 1969
JULY 1966
JULY 1968
JULY 1969
1 SOUTH
2 SOUTH
3 SOUTH
4 SOUTH
5 SOUTH
2 NORTH
4 NORTH
6 NORTH
8 NORTH
10 NORTH
12 NORTH
14 NORTH
SURFACE BOREHOLE 1/51
SCALE
300 feet

Fig. 28

some cases these boreholes penetrated the overlying strata and water was encountered, the hole subsequently being plugged though not always satisfactorily as will be discussed later.

When these headings were turned Northwards after the required 600 ft. had been driven, a two heading mining system was used. Also at this time, metric units were introduced which resulted in the following changes in mine dimensions :-

	Before	After
Basis for roadway centres	40 ft. 80 ft.	12 m. (39.37 ft) 24 m. (78.74 ft)
Roadway width	15 ft.	5 m. (16.41 ft)
Working height	10 ft.	3 m. (9.84 ft)
Thickness of gypsum roof	6 ft.	2 m. (6.56 ft)
Extraction Rate (Development)	50%	54%
Extraction Rate (Proposed in Panels)	61%	66%

It will be noticed that the only significant dimension changes were an increase in roadway width of 1.4 ft. and an increase in the thickness of the gypsum roof beam of approximately 6 in.

In March 1970, working in the development headings forming the Northern boundary of the central panel, was transferred to roadways 7, 8 and 9 East, the position of the working faces on July 1970 being shown in Fig.28. It is expected that this panel will be isolated by September 1970.

The fact that the seam is situated at a depth of 120 ft. and thus subjected to a vertical load due to the weight of the overburden of approximately 120 p.s.i., means that with the 50% extraction of the development workings, the compressive stress on the remaining rock material forming the pillars is only 240 p.s.i. Preliminary calculations of this type prior to mining, were the consequence of the Inspectorate's ruling that the mining system to be used would be of the room and pillar type.Even with a 75% extraction rate, which is higher than any contemplated, the average compressive pillar stress is less than 500 p.s.i. Thus, in view of the comparatively low pillar load, together with the compressive strength value of the seam material obtained in the laboratory, it is thought that the load bearing capacity of the pillar is not a limiting factor in the stability of the mine. The laboratory compressive strength data suggests that not only are the pillars formed by the development headings stable, but also those which it is planned to form in the working panels. The reason for this is simply that the ultimate compressive strength of the gypsum forming the pillars appears to be a great deal larger than the compressive stress that the pillars are required to bear.

It was suggested by Jones (5) that the gypsum forming the Sherburn deposit is remarkable on account of its homogeneity and continuity, and that although slight variations do occur in the constituents of the seam they do not appear to be sufficiently numerous and marked to justify their being taken into account in the overall pillar design. As mine development proceeded however, it became more and more obvious that this statement was not completely valid. It was found that there was a marked vertical variation in the rock material forming the seam, giving an apparently laminated effect to the seam though there were no definite planes of lamination in the working height associated with this. The most important factor that became obvious however, was the occurrence of a very large number of planes of discontinuity in the seam. This quite often resulted in the formation of blocks of gypsum of various and varying sizes, that were completely free from the rest of the seam material. These planes of discontinuity, sometimes vertical and horizontal, but more often inclined, quite often influenced the shape of the sides of the roadways, especially at intersections. Hence, whilst it is not suggested that, at this depth of working and using the pillar sizes that are contemplated, pillar stability could be affected to any great deal, some form of 'in situ' investigation should be carried out to confirm the conclusions obtained from the laboratory testing.

The stability of the roof strata at Sherburn is a factor which limits the feasibility of a particular working design to a much greater extent than the factor of pillar stability. Since the thickness of the immediate roof layer is governed by the Inspectorate's ruling, the most likely limitation would be imposed by the maximum permissible roof span, or possibly, the dimensions of the floor beam as it overlies the weaker water bearing marls. As mentioned previously mine development revealed the presence of a large number of discontinuities in the working zone of the seam, and whilst these discontinuities were not always apparent in the roof section of the seam, other factors were encountered which could have some influence on the stability of the immediate roof layer.

To ensure that a minimum of 6 ft. of gypsum was left between the roof of the workings and the overlying marls, inclined roof boreholes were drilled at regular intervals in each heading. When these holes penetrated the overlying marls, water was encountered and the holes subsequently plugged. The plug however, was inserted inside the seam boundary, and whilst this prevented the water from entering the workings through the borehole, it still permitted the passage of water to the seam itself. The gypsum is normally impermeable, but after some time this water was able, by solution, to penetrate along bedding planes, ultimately reaching the workings through the planes of discontinuity described previously. Over a much larger period of time the flow of water increases considerably as the water penetrates further along bedding planes gaining access to further joints in the seam. In addition to these boreholes, the water gains access to both the roof beam and the workings through these joints, fracture planes and fault lines.

The effect that this water has on the stability of the roof is an unknown factor. Visually, there is obvious evidence in the development workings of roof sag at an intersection, though admittedly this was partly due to pressure grouting in an attempt to stop the flow of water. It is possible that the effect of the water forcing its way along bedding planes is to reduce the effective thickness of the roof beam overlying the workings. Since, for a roof beam of a given thickness composed of a material with a certain ultimate tensile strength, there is a specific limit to the length of a beam which can carry a particular load, any alteration in the effective thickness of the beam must affect the state of stability of the roof beam. Also, if there is a full hydrostatic head of water acting in the roof beam itself, the stresses induced in the lower section of the beam would be considerably increased. An appreciation of the possible loading of the roof beam is dealt with in a later section.

A further factor that should be considered is that although the roof itself may be stable from the point of view of localised stresses, the additional influence of the stresses induced by any large scale strata movement associated with the working pattern could conceivably cause instability of the roof.

2.4 Method of Approach Used in this Investigation.

A detailed knowledge of the various mechanical properties of the seam material was already available and since access could be gained to the mine workings, the objectives of the investigations carried out by the author were directed towards obtaining 'in situ' measured data on the stability of the existing development workings. The 'in situ' approach can be used to provide quantitative data which can assist in evaluating the state of stability of the underground workings. Although a number of techniques are available to carry out 'in situ' measurements, they fall into two main categories depending upon whether the purpose of the measurement is to determine the state of stress or to quantify the deformation of the strata in and around the mine workings.

The determination of the load acting on the pillars and, in some cases the roof layer, at various stages of extraction, is necessary from both theoretical and practical design points of view. At the present state of development of stress measuring instruments however, no such technique is entirely satisfactory. Nevertheless, where the conditions of the rock are such that stress measurements can yield meaningful results, useful design data and a measure of the stability of the workings may be deduced from the information so obtained. An attempt was therefore made to measure the absolute stress in the mine pillars using a technique based on the stress relief principle.

In most circumstances however, the measurement of the deformation of the support elements and the overlying strata gives a more realistic picture of the state of stability of the mine workings. Deformation measurements provide a convenient and simple means of obtaining 'in situ' results which even in their qualitative form can aid the understanding of local or overall stability of the workings. The design of the 'in situ' investigation intended to define stability therefore, was based on the measurement of strata displacement in the roof layers and in the pillars. The instrumentation was designed not only to enable data to be obtained relating to the stability of the underground workings but also to enable a check to be made on the strata stability in the immediate vicinity of the workings, so that a check could be made on any strata movement which could give rise to subsidence at the surface. The installation of the equipment and the information subsequently obtained from it will be described in a later section.

In addition to the 'in situ' instrumentation, a brief theoretical appreciation of the stability of the immediate roof was carried out. The technique used in this approach was similar to that developed by Stephanson (64). Computer techniques were used to determine the magnitude of the induced stresses and the deflection of the roof beam. The techniques involved and the results obtained are described in the following section.

3. A THEORETICAL APPRECIATION OF THE STABILITY OF THE ROOF.

The problem of determining the state of stability of the roof is influenced considerably by the Inspectorate's ruling that a continuous roof slab of solid gypsum at least 6 ft. thick be left to form the immediate roof. The laboratory investigations into the ultimate compressive strength and ultimate tensile strength of the gypsum suggested that the gypsum was a homogenous and continuous rock material. The roof is known to consist of a uniform slab of similar material and it is justifiable, in the first instance, to view it from the point of classical mechanics and consider it to be either a plate supported by the pillar edges or one supported above the roadway intersections by the pillar corners. It should, however, be remembered that the pillars and roof are integral parts of the solid seam. As mentioned in the previous section however, factors that became more and more apparent during the development of the mine were the laminated structure of the seam and the occurrence of vertical jointing, though mainly in the mid-part of the seam, and a theoretical appreciation should therefore be treated as a first approximation only.

As a preliminary measure, the roof plate between the pillars may be treated as either a clamped or free ended beam of unit width. The decision as to whether the roof should be considered as a simply supported or a clamped beam will be discussed later. An approximation to the actual conditions in the roof beam can then be obtained by treating the roof layer as an elastic plate on elastic foundations.

It is possible to analyse the problem of a slab lying on elastic supports by the mathematical methods of the theory of plate flexure, providing the loading conditions of the structure are known together with the spacial and elastic parameters. In this case however, certain assumptions have been made to reduce the problem to one of manageable proportions. Some of these assumptions are usual when dealing with beam problems and have been mentioned previously in Part 1, Section 3.3 of this thesis. Others are specifically designed to reduce the complexity of this problem.

One of the assumptions is that all the applied loads are normal to the beam. These consist of the weight of the beam, the load applied from above and the reactive load applied by the pillars. The entire structure consists of the pillar and the underlying rocks, the immediate roof and the overlying rocks. It is assumed that the load applied to the roof beam by the overlying material is uniform and normal - this is probably quite correct because of the overlying weak marls. The reactive loading of the pillars is assumed to be proportional to the deflection of the beam, the pillars are treated as an elastic supporting medium. This reactive loading acts vertically and opposes the deflection of the beam.

It is felt that, in the case of the mine at Sherburn, the assumptions are fairly accurate, and hence it is justifiable to use them as a basis for a theoretical appreciation of the state of stability of the immediate roof.

3.1 The Application of Theoretical Methods to the Problem of Roof Stability.

The problem of the roof above the actual roadways at Sherburn may be dealt with in either of two ways, the roof beam may be considered to have clamped or unclamped ends - this has already been mentioned in Part 1, Section 3.1. The state of the beam ends has a marked effect on the magnitude and position of the maximum tensile stresses. With a situation such as the one which prevails at Sherburn, it is rather difficult to decide which type of beam provides an analogy for the behaviour of the roof beam. The lower part of the ends of the beam are most certainly firmly attached to the pillars for the simple reason that they are all part of a continuous mass. On the other hand the upper parts of the beam ends are in contact with the overlying marls and the degree of restraint in this position is going to depend both upon the frictional forces between the gypsum and the marl, and upon the forces parallel to the plane of the beam imposed by the ends of abutting beams. The former forces are most likely to be great as the interfaces are in intimate contact, the latter forces depend upon the continuity of the beams in the zones above the pillars. Bearing all these points in mind it is suggested that the behaviour of the roof beam at Sherburn is more akin to the behaviour of a clamped rather than a simply supported elastic beam.

A factor which must be taken into account before presenting calculated data relating to the behaviour of the roof beam is the magnitude of the load which is applied to the roof beam by the overburden. Although the 45 ft. of marls overlying the gypsum may be sufficiently rigid to support both themselves and the overlying lacustrian deposits, it would be one of the extreme cases to assume that these marls have no rigidity and hence the roof beam has to support a distributed load composed of the entire weight of the superincumbent strata. It is proposed, therefore, to carry out the theoretical appreciation for three conditions of the possible loading of the roof beam. The three cases may be described as follows :-

1. The gypsum beam carrying the entire weight of the superincumbent strata - the marls are assumed to carry no load.
2. The gypsum beam supporting nothing but its own weight - the marls are assumed to be supporting the entire weight of the lacustrian deposits.
3. The gypsum beam carrying a load due to a hydrostatic head equal to full working depth, i.e. 120 ft. In this case the marls are assumed to be supporting their own weight plus the weight of the lacustrian deposits, but to be permitting the passage of water, i.e. the load on the beam is Case 2 + hydrostatic head.

It may be seen that the beam supports the greatest load in Case 1 and the smallest in Case 2, Case 3 is obviously an intermediate one.

The theoretical approach used is based on a technique developed by Stephanson (64) for determining the deflection and longitudinal fibre stresses induced within a single layer roof beam. A computer program, shown in Appendix A, was used to produce values of deflection of various points on the roof beam and the longitudinal fibre stresses induced in the lower section of the beam at the same points. The program used was developed from the equations produced for a single layer roof beam, which disregarded the effects of the shear stresses induced in the beam. Strictly speaking, the theory for a single layer roof beam is valid for a ratio between the beam thickness, H, and the roof span, L , of H < L/5, whereas the roof beam dimensions at Sherburn gave a ratio slightly greater than this. However, the value L/5 = H is by no means an exact limit since shear stresses will always influence the elastic equations for supported and unsupported parts of the layer. The effect of neglecting the shear stresses in the calculation is to introduce an error of between 10 - 15% in the stress values obtained in this investigation, but a negligible additional deflection. For a first consideration therefore, the simpler equations according to the theory for a thin single layer beam, H < L/5, can be used.

In addition to the single layer roof beam, an attempt was also made to determine the deflection and longitudinal stresses induced in a double layer roof beam. In this case, programs were used for a double layer beam in which the layers were either free to separate or had welded contact. However, since this required considerable computer time and the results that were obtained were indeterminate, coupled with the fact that there was some doubt as to the actual validity of the theory as applied to the Sherburn mine, the calculations for the double layer roof beam configuration were not completed.

The mathematical expression used in the calculations carried out have been, for the most part, obtained from the paper of Stephanson (64), and it is not proposed to repeat the derivations. The computer program is shown in Appendix A and the calculations carried out can be deduced from it. The following relevant data was used in the calculations.

The roadway width,	= 180 ins.
The roadway height, h	= 120 ins.
The roof beam thickness, H	= 72 ins.
Young's Modulus for the pillar and roof beam	= 2.0 - 3.0 x 10^6 lb/in^2.
Poisson's ratio	= 0.28
Specific gravity of the roof beam	= 0.083 lb/in^3

Assuming these known values of roadway height, roof span and elastic constants,the deflection and stresses occurring in different parts of the roof beam above a single opening were calculated for the three possible load conditions described previously. The origin of a rectangular co-ordinate system was chosen as the pillar edge and the calculations carried out for a range of values for Young's modulus at 10 in. increments on the 'X' -axis, between values for x of - 90 ins. and + 90 ins., the remainder of the roof beam being symmetrical. A range of values for Young's modulus of 2.0 to 3.0 x 10^6 lb/in^2 was used since the laboratory determined value of 2.6 x 10^6 lb/in^2 was the mean of several widely varying values.

3.2 Discussion of the Results.

The calculated results for the deflection of the roof beam and the longitudinal fibre stresses induced in the beam are shown in Figs.29 and 30; for the three conditions of loading of the beam and a range of values for Young's modulus of 2.0 to 3.0 x 10^6 lb/in^2. The deflection shown in each case includes the deflection of the abutment or pillar, the calculated value of which is given in the Figures for each load condition.

In case 1, the roof beam was assumed to be carrying the full weight of the superincumbent strata in addition to its own body weight and obviously, this gave the highest values for both the deflection and the tensile stresses. The effect of varying the Young's modulus

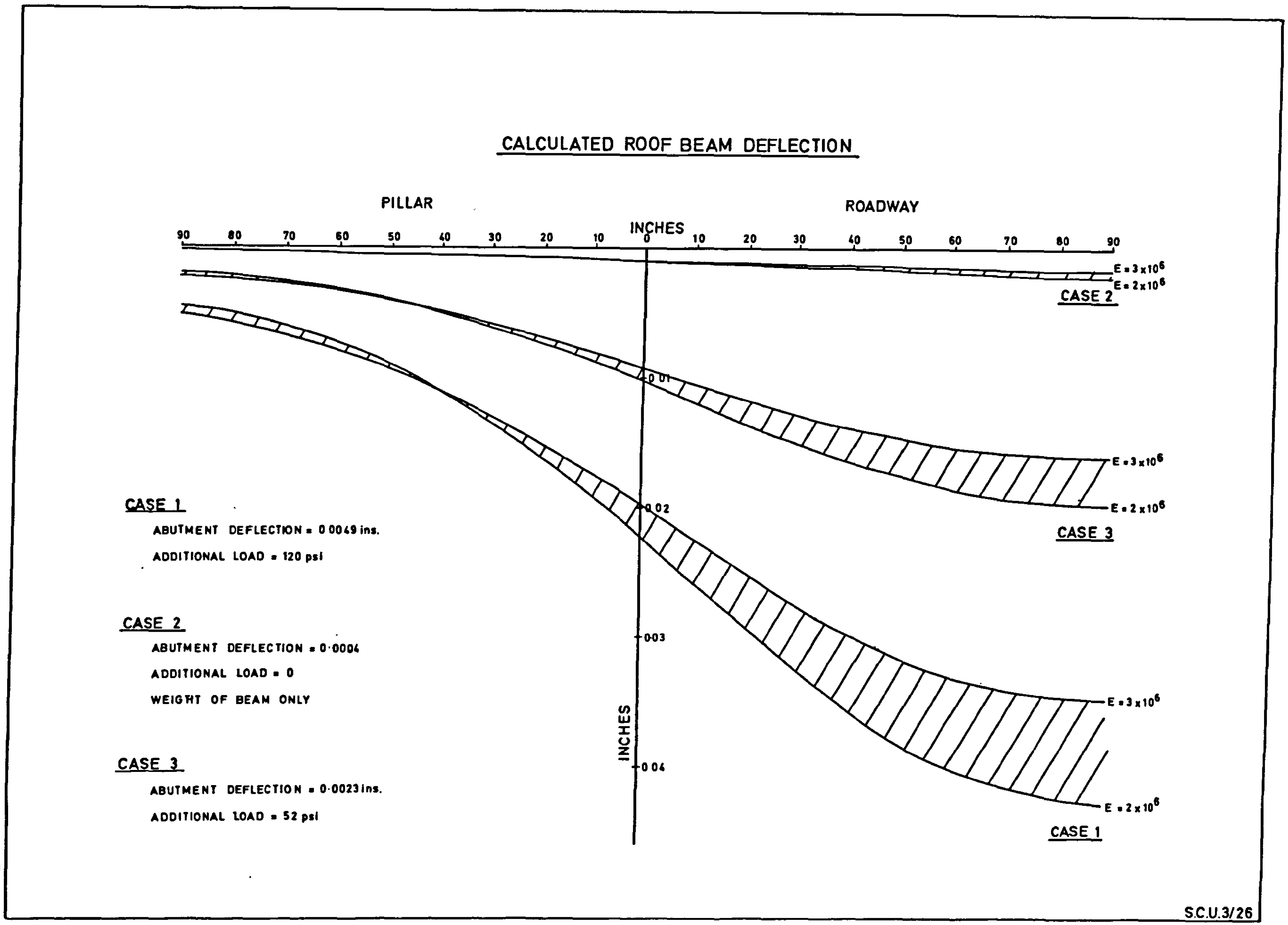
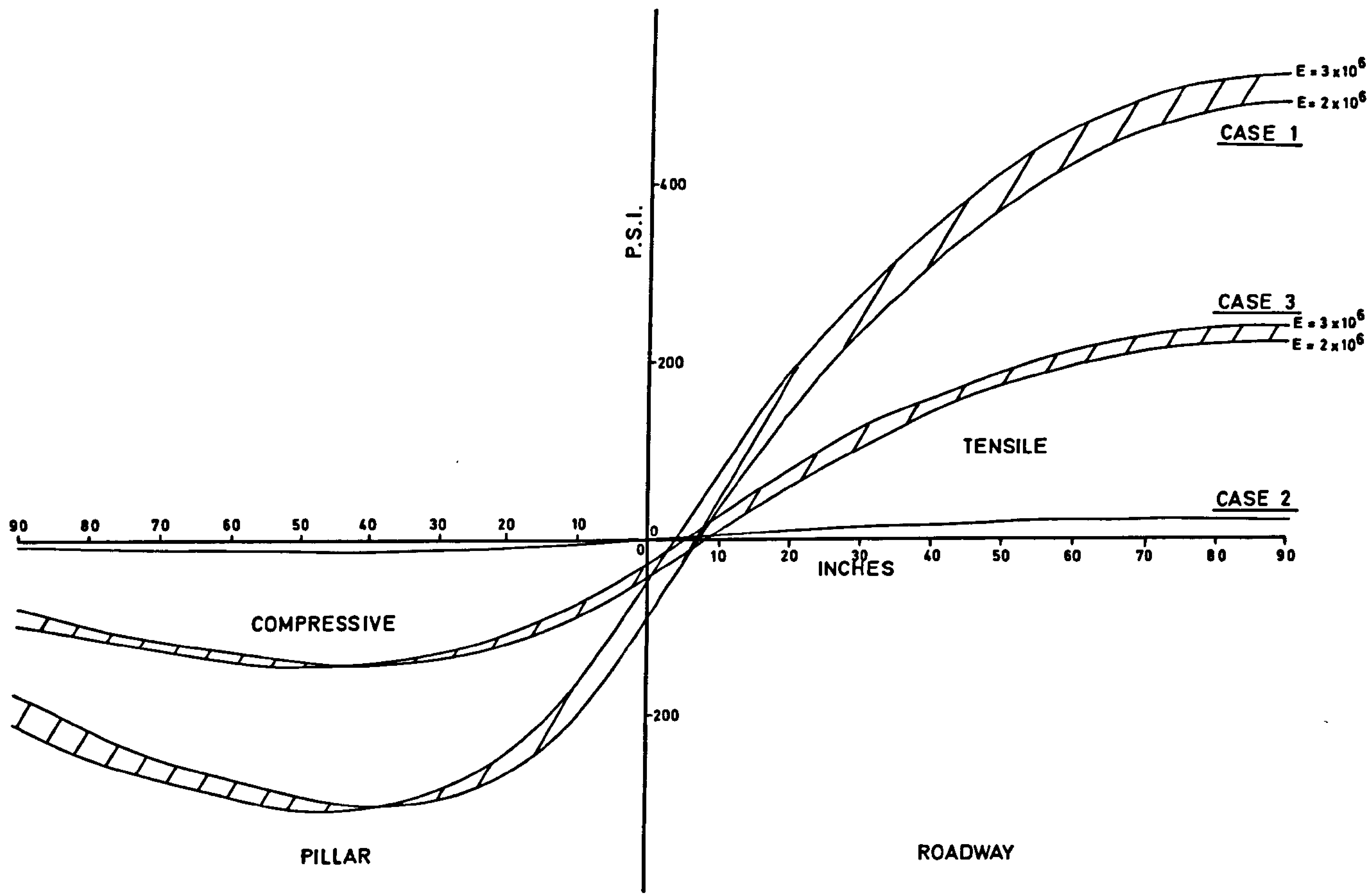

CALCULATED ROOF BEAM DEFLECTION
PILLAR
ROADWAY
INCHES
90 80 70 60 50 40 30 20 10 0 10 20 30 40 50 60 70 80 90
0·01
0·02
0·03
0·04
INCHES
E = 3 x 10^6
E = 2 x 10^6
CASE 2
E = 3 x 10^6
E = 2 x 10^6
CASE 3
E = 3 x 10^6
E = 2 x 10^6
CASE 1
CASE 1
ABUTMENT DEFLECTION = 0·0049 ins.
ADDITIONAL LOAD = 120 psi
CASE 2
ABUTMENT DEFLECTION = 0·0004
ADDITIONAL LOAD = 0
WEIGHT OF BEAM ONLY
CASE 3
ABUTMENT DEFLECTION = 0·0023 ins.
ADDITIONAL LOAD = 52 psi
S.C.U.3/26

FIG. 29

CALCULATED LONGITUDINAL FIBRE STRESS INDUCED IN THE ROOF BEAM



S.C.U.3/27

FIG. 30

from 2.0 to 3.0 x 10^6 lb/in^2 resulted in a decrease in the maximum values of the deflection at the centre of the roadway of 0.0081 ins. and an increase in the maximum longitudinal fibre stress at the same point of 35 p.s.i. In case 2, the roof beam was assumed to be supporting its own weight only, and in this case the deflection and stress values obtained for the 6 ft. thick beam was found to be negligible. The effect of varying the Young's modulus was also found to be negligible. In the final load condition, case 3, the roof beam was assumed to be supporting its own weight plus the load due to the hydrostatic head of water of 120 ft., and it may be seen that the calculated values were intermediate between case 1 and case 2. The effect of varying the Young's modulus from 2.0 to 3.0 x 10^6 lb/in^2 resulted in a decrease in the maximum deflection of 0.004 in. and an increase in the longitudinal fibre stress of 16 p.s.i.

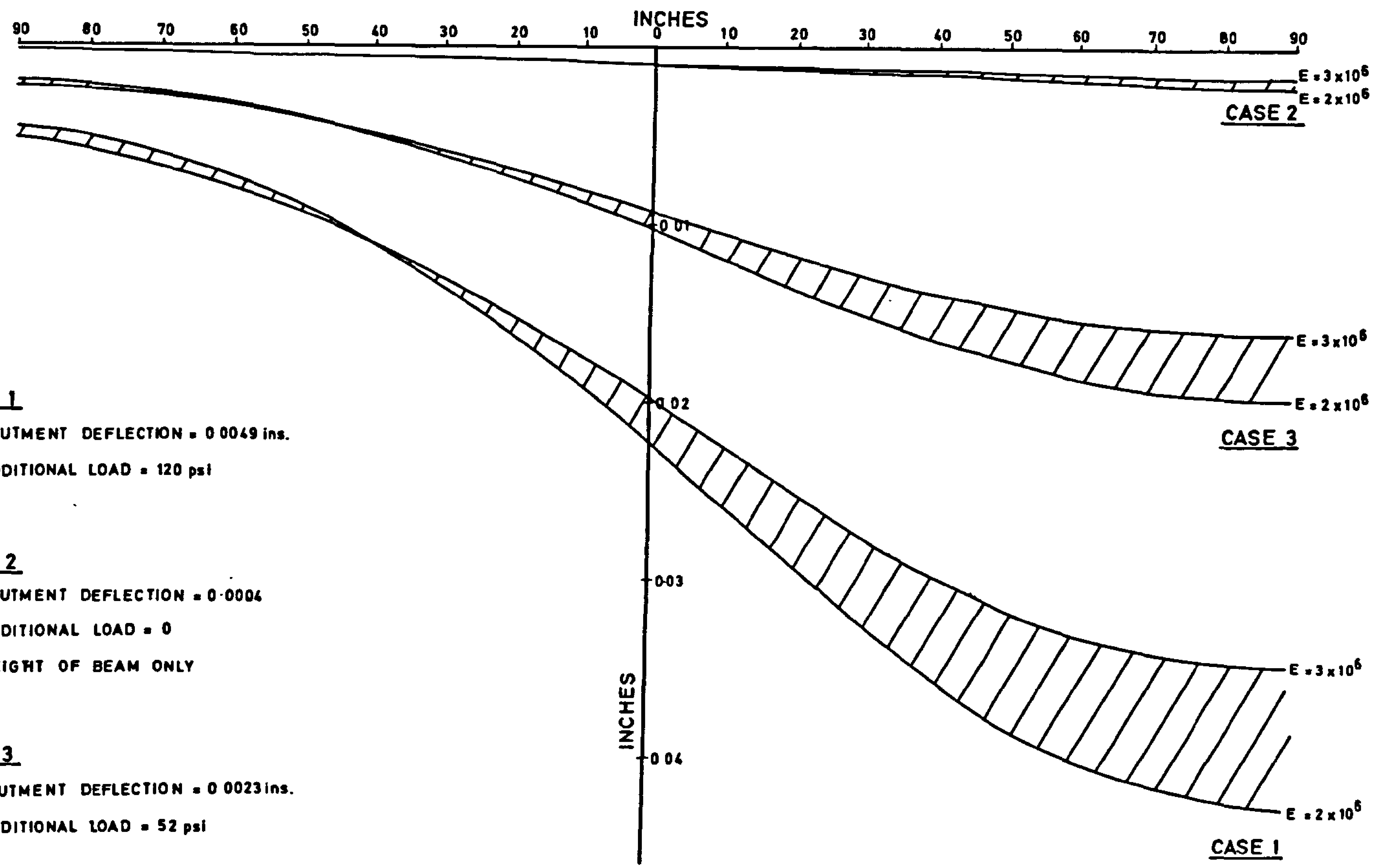
Before these results are discussed further, it would be as well to consider the loading condition which is most likely to apply in practice, and to discuss the relative rigidities of the gypsum and marl beds. If the marl bed is more rigid than the gypsum roof layer the gravity loading on the two beds would cause the gypsum to sag to a greater extent than the overlying marl and consequently the marl would impose no additional loading on the gypsum beam. Since the rigidity of a layer depends upon two factors, the thickness of the layer and its Young's modulus, even if the modulus is assumed to be 1/10th that of the gypsum, the fact that the marl has a thickness of 45 ft. against the 6 ft. of the gypsum beam causes the marl to be six times as rigid. Therefore, provided that the marl layers do not separate, it is reasonable to suggest that the presence of the overlying marls cause little or no increase in loading of the gypsum beam. It is suggested therefore, that the calculations in case 1 refer to an unrealistic case and that the calculated values for the deflection and stress are those which exist under the most extreme conditions.

In view of this therefore, it is necessary to decide which of the other two cases provide the most satisfactory loading condition of the roof beam. The amount of water extering the workings through the roof beam suggests that in all probability, water applies an hydraulic loading to the roof layer, and that it is justifiable to assume that the water present is subjected to the full hydraulic head due to the depth of working. Consequently, it would seem reasonable to suggest that provided the marl does not separate, and hence become less rigid, the loading condition as depicted by case 3 may be taken to represent the probable situation. It is proposed therefore, to discuss further only those results obtained for this loading condition.

The maximum deflection of the roof beam at the centre of the roadway for a Young's modulus of 2.0×10^6 lb/in^2 is only 0.020 (0.51 mm.). The corresponding maximum deflection for a Young's modulus of 3.0×10^6 lb/in^2 is 0.0162 in.(0.41 mm). The maximum tensile stress values at the same point for moduli of 2.0 and 3.0 $\times 10^6$ lb/in^2 are 228 p.s.i. and 244 p.s.i. respectively, approximately equal to 60% of the mean tensile strength of the seam material obtained in the laboratory.

In a later section,the values of the beam deflection will be compared with vertical convergence measurements obtained from an underground instrumentation scheme. It will suffice to mention at this stage however, that the calculated values for the deflection of the roof beam above the roadway are very low. It will be noticed that the beam deflection is not confined to that section overlying the roadway, but is also apparent for some distance into the pillar. If, in this case, the calculated abutment deflection of 0.0044 ins. is subtracted from the total deflection of the beam, zero deflection of the roof beam occurs at a distance of between 80 - 90 ins. from the pillar edge.

CALCULATED ROOF BEAM DEFLECTION



CASE 1

ABUTMENT DEFLECTION = 0.0049 ins.

ADDITIONAL LOAD = 120 psi

CASE 2

ABUTMENT DEFLECTION = 0.0004

ADDITIONAL LOAD = 0

WEIGHT OF BEAM ONLY

CASE 3

ABUTMENT DEFLECTION = 0.0023 ins.

ADDITIONAL LOAD = 52 psi

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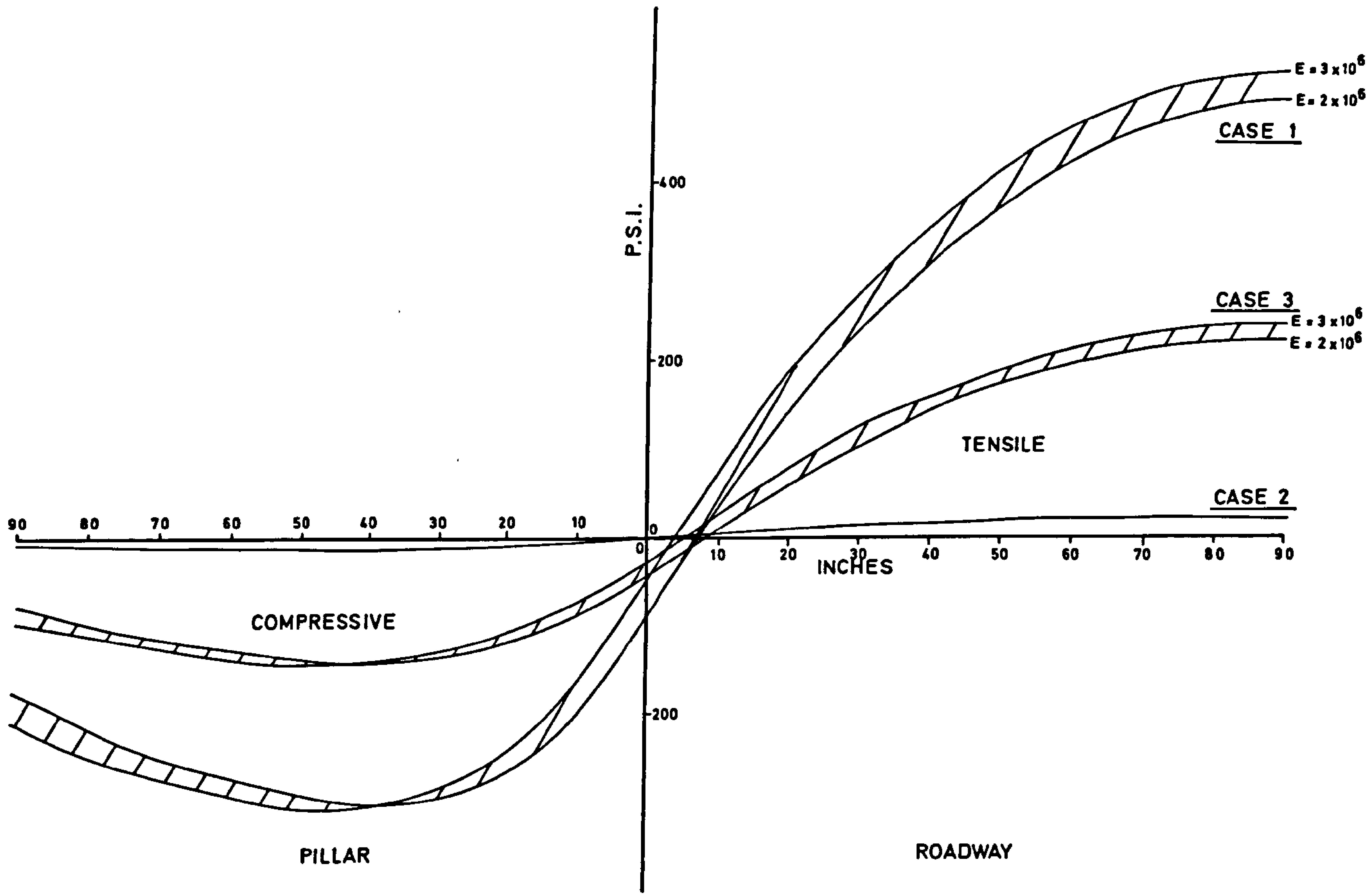
FIG. 29

CALCULATED LONGITUDINAL FIBRE STRESS INDUCED IN THE ROOF BEAM



S.C.U.3/27

FIG. 30

from 2.0 to 3.0 x 10^6 lb/in^2 resulted in a decrease in the maximum values of the deflection at the centre of the roadway of 0.0081 ins. and an increase in the maximum longitudinal fibre stress at the same point of 35 p.s.i. In case 2, the roof beam was assumed to be supporting its own weight only, and in this case the deflection and stress values obtained for the 6 ft. thick beam was found to be negligible. The effect of varying the Young's modulus was also found to be negligible. In the final load condition, case 3, the roof beam was assumed to be supporting its own weight plus the load due to the hydrostatic head of water of 120 ft., and it may be seen that the calculated values were intermediate between case 1 and case 2. The effect of varying the Young's modulus from 2.0 to 3.0 x 10^6 lb/in^2 resulted in a decrease in the maximum deflection of 0.004 in. and an increase in the longitudinal fibre stress of 16 p.s.i.

Before these results are discussed further, it would be as well to consider the loading condition which is most likely to apply in practice, and to discuss the relative rigidities of the gypsum and marl beds. If the marl bed is more rigid than the gypsum roof layer the gravity loading on the two beds would cause the gypsum to sag to a greater extent than the overlying marl and consequently the marl would impose no additional loading on the gypsum beam. Since the rigidity of a layer depends upon two factors, the thickness of the layer and its Young's modulus, even if the modulus is assumed to be 1/10th that of the gypsum, the fact that the marl has a thickness of 45 ft. against the 6 ft. of the gypsum beam causes the marl to be six times as rigid. Therefore, provided that the marl layers do not separate, it is reasonable to suggest that the presence of the overlying marls cause little or no increase in loading of the gypsum beam. It is suggested therefore, that the calculations in case 1 refer to an unrealistic case and that the calculated values for the deflection and stress are those which exist under the most extreme conditions.

In view of this therefore, it is necessary to decide which of the other two cases provide the most satisfactory loading condition of the roof beam. The amount of water extering the workings through the roof beam suggests that in all probability, water applies an hydraulic loading to the roof layer, and that it is justifiable to assume that the water present is subjected to the full hydraulic head due to the depth of working. Consequently, it would seem reasonable to suggest that provided the marl does not separate, and hence become less rigid, the loading condition as depicted by case 3 may be taken to represent the probable situation. It is proposed therefore, to discuss further only those results obtained for this loading condition.

The maximum deflection of the roof beam at the centre of the roadway for a Young's modulus of 2.0 x 10^6 lb/in^2 is only 0.020 (0.51 mm.). The corresponding maximum deflection for a Young's modulus of 3.0 x 10^6 lb/in^2 is 0.0162 in.(0.41 mm). The maximum tensile stress values at the same point for moduli of 2.0 and 3.0 x 10^6 lb/in^2 are 228 p.s.i. and 244 p.s.i. respectively, approximately equal to 60% of the mean tensile strength of the seam material obtained in the laboratory.

In a later section, the values of the beam deflection will be compared with vertical convergence measurements obtained from an underground instrumentation scheme. It will suffice to mention at this stage however, that the calculated values for the deflection of the roof beam above the roadway are very low. It will be noticed that the beam deflection is not confined to that section overlying the roadway, but is also apparent for some distance into the pillar. If, in this case, the calculated abutment deflection of 0.0044 ins. is subtracted from the total deflection of the beam, zero deflection of the roof beam occurs at a distance of between 80 - 90 ins. from the pillar edge.

The longitudinal fibre stresses gradually decrease towards the pillar and become compressional close to the pillar edge. They attain a maximum compressive value above the pillar of approximately 140 p.s.i. at a distance of 45 ins. from the pillar edge.

In order to determine the effect on the roof beam of the change in mine dimensions due to the decision to use metric values, the programs for each load condition were re-calculated using the revised dimensions. It was found that the calculated values of the beam deflection and longitudinal fibre stresses, increased by a negligble amount. It would seem that the increase in roadway width from 15 ft. to 16.41 ft. (5 m.) is offset by the increase in beam thickness from 6 ft. to 6.56 ft. (2 m.).

This theoretical appreciation of the roof stability was intended as a preliminary assessment only to which actual 'in situ' measurements could be related. It should be remembered that the various values for deflection and longitudinal fibre stress were calculated using the assumption that the roof strata behaves as if it were an elastic beam on elastic supports. The application of this theoretical approach and the relevance of the conclusions drawn will be considered in connection with some other aspects of the research work, in the conclusions, Section 7.

4. 'IN SITU' INVESTIGATIONS TO DETERMINE THE DEFORMATION OCCURRING IN AND AROUND THE MINE WORKINGS.

In any room and pillar system of mining, the re-distribution of the load due to the extraction of the seam, manifests itself in the form of deformation of the pillars and surrounding strata, and it may be supposed that it is the magnitude of this re-distributed load, and hence the magnitude of the resulting deformation which determines the stability of the workings.

The initial 'in situ' investigations carried out at Sherburn were designed to obtain information regarding the stability of the strata overlying the workings. It was decided that some form of instrumentation should be installed to provide information concerning any vertical movement which might occur in the strata above the workings, and also any which might occur in the gypsum layer forming the roof of the mine roadways.

In addition to this, an underground instrumentation scheme was installed in the development workings. This was designed to measure the lateral deformation occurring in selected pillars together with roof-floor convergence measurements throughout the mine.

4.1 Instrumentation.

The instruments used in this investigation, to measure the deformation occurring in and around selected areas of the development workings, were developed and manufactured in the Department of Mining Engineering, University of Newcastle upon Tyne. The descriptions and techniques of using these instruments have been extensively published by Potts (79) (80), Edwards (81), Sen (12), Leigh (82) and Johnson (83). Therefore, since this part of the thesis is concerned primarily with the obtaining and analysis of underground measurements, only a brief description of the instruments and measuring techniques used is given here.

The primary instrument used in the 'in situ' investigations, both at Sherburn and Stamphill, was the borehole extensometer and its associated forms.

4.1.1 The Extensometer.

This instrument was designed to measure the relative movement between a point within a borehole and the mouth of the hole. The principle of the instrument relies upon one elastic property of metallic specimens, namely that a specimen of metallic wire or tape under a certain load will assume the same length at two instants of time providing the same load is applied to it, and that the temperature remains constant.

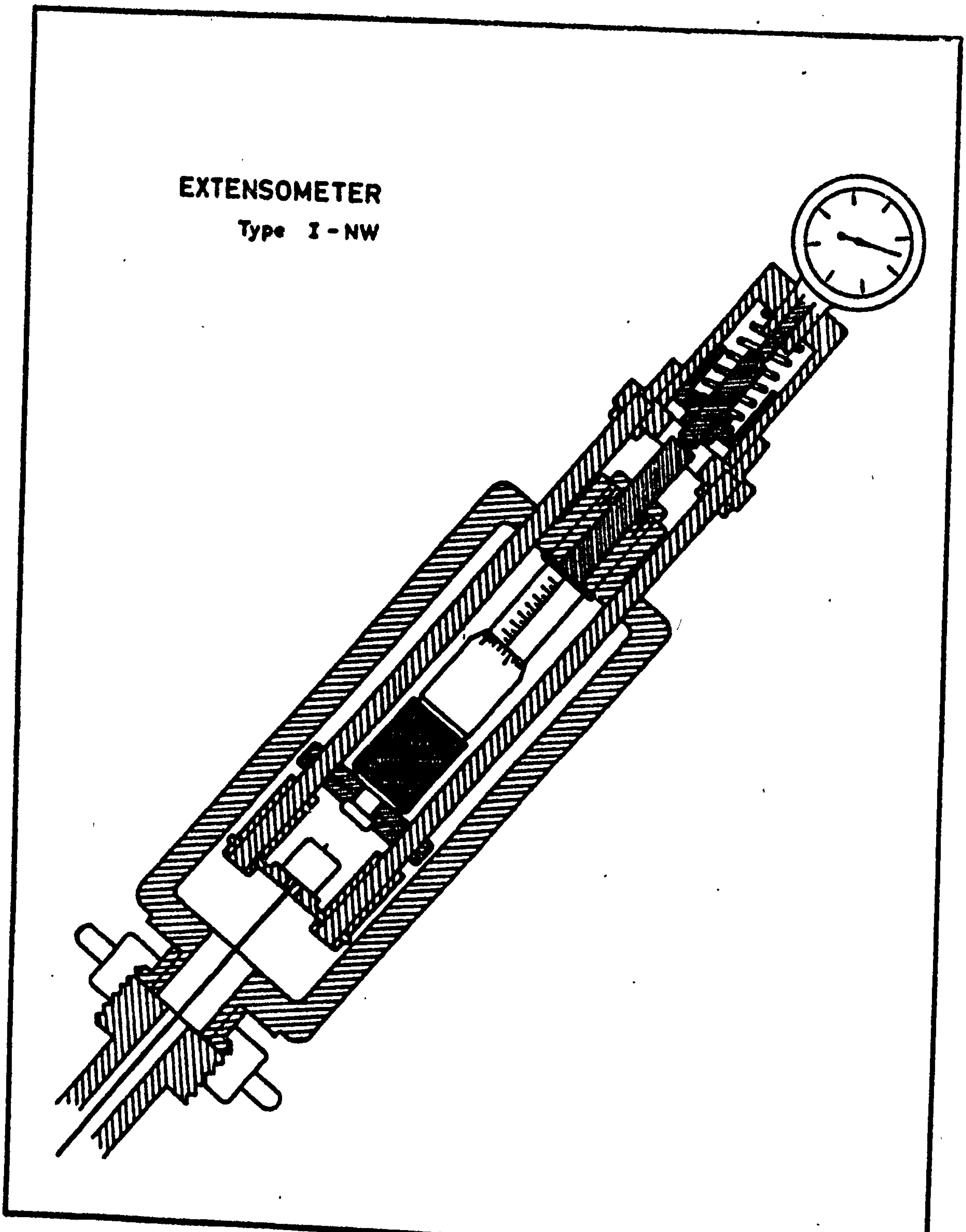


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Fig. 31.

Fig. 32

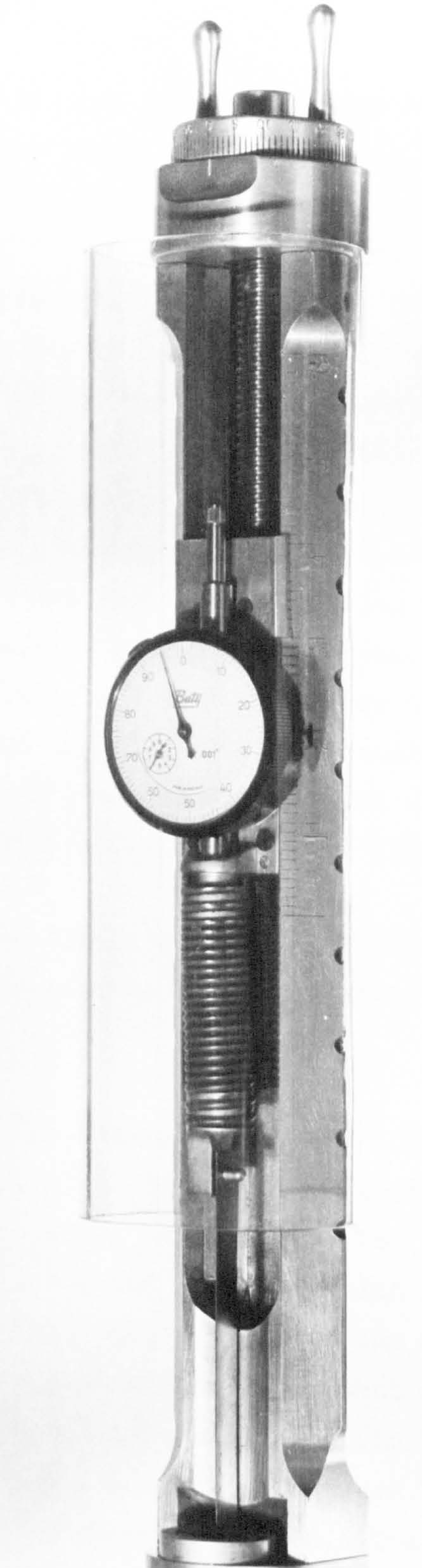


Fig. 33

To facilitate this, one end of the wire or tape is anchored in the borehole and the other end is taken to the mouth of the hole and attached to the tensioning device. Once the designated load has been applied to the tape, the distance between a reference mark on the tensioning saddle and the fixed datum at the hole mouth is measured by means of a micrometer. Over a period of time, as the borehole contracts or extends, the reference mark on the end of the constant length of tape moves relative to the datum at the hole mouth, thus giving a measurement of the change in distance between the plug in the borehole and the datum at the mouth. Once the initial reading has been taken in a borehole, it is possible to relate all subsequent measurements to preceding one or ones, and thus a picture of the axial deformation of the borehole is obtained. The tensioning-measuring device used to obtain the actual measurement is termed the Mk.I Extensometer. This instrument is shown in Fig.31.

Alteration to the frame of the Mk.I instrument gives the Mk.I reversed which is used to measure roadway convergence as shown in Fig.32.

The latest borehole extensometer is the Mk II, shown in Fig.33. This consists of a rigid frame containing a sliding saddle mounted on, and moved by,a screw. The saddle carries a dial gauge, helical spring and a pin to which is attached the measuring tape. The dial gauge indicates the deformation of the spring, hence by adjusting the instrument until the dial gauge gives a pre-determined reading, one can be sure that the spring has been extended a certain amount and hence that the measuring tape is at the correct tension. The measuring technique of this instrument is the same as that for the Mk.I Extensometer.

The Mk.II Extensometer is designed for use generally in boreholes of larger diameter than that for which the Mk.I was intended. It is used to carry out similar types of measurements as the Mk.I Extensometer, on longer lengths of wire or tape. It differs from the Mk.I version in that it has a range of movement of 6 ins. instead of the 1 in. or 25 mm. of the Mk.I Extensometer, before the effective length of the wire or tape needs to be altered. All the extensometers read to 0.001 in. (0.01 mm.).

4.1.2 <u>The Borehole Anchors.</u>

The anchors used for securing the extensometer tapes or wires in boreholes are of two types :-

a) <u>Mechanical anchors</u> - these consist of a slightly modified 1.11/16 in. B.J.B. roof bolt anchor. The wire is secured to the anchor by means of a grub screw and the anchor itself is installed in the hole using a box spanner welded to an insertion pipe. For intermediate anchors a hole is drilled down the long axis of the anchor and the wires from any anchors further up the hole threaded through, the intermediate anchors are then installed using the box spanner insertion rod. The mouth anchor consists of a screw thread that is welded to a B.J.B. anchor shell, the screw thread with machined end face, being used by the Extensometer to take the actual measurement. The three anchors used are shown in Fig.34.

b) <u>Hydraulic anchors</u> - in long boreholes it is very difficult to turn a long length of insertion rods to tighten the screw anchor. To deal with this problem the hydraulic anchor - Figs.35 and 36 - was developed. Sprung steel bow sections are forced outwards and flattened against the sides of the hole as a tapered section is forced forward by means of a hydraulic ram to which the anchor is attached with shear pins. These eventually shear allowing

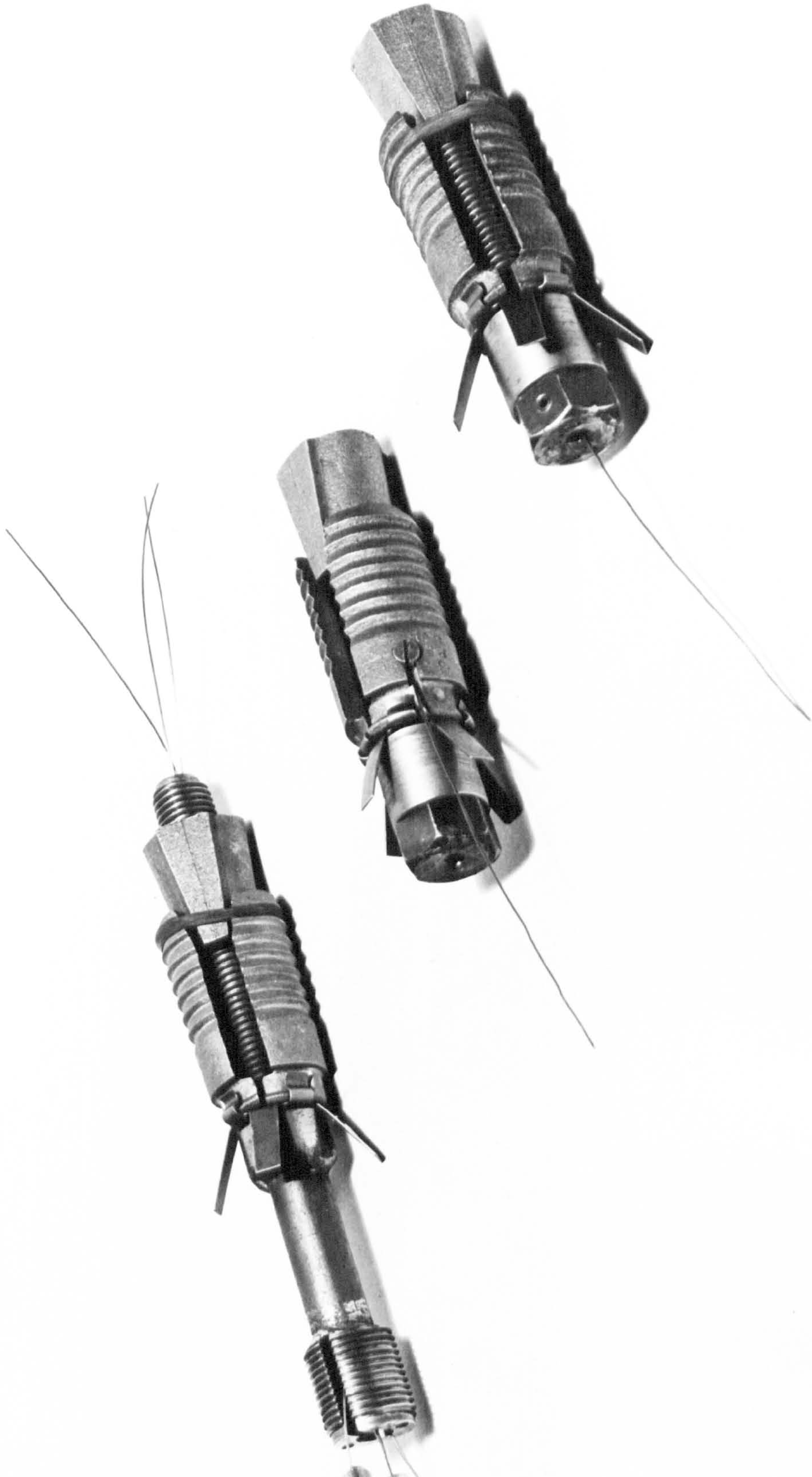


Fig. 34

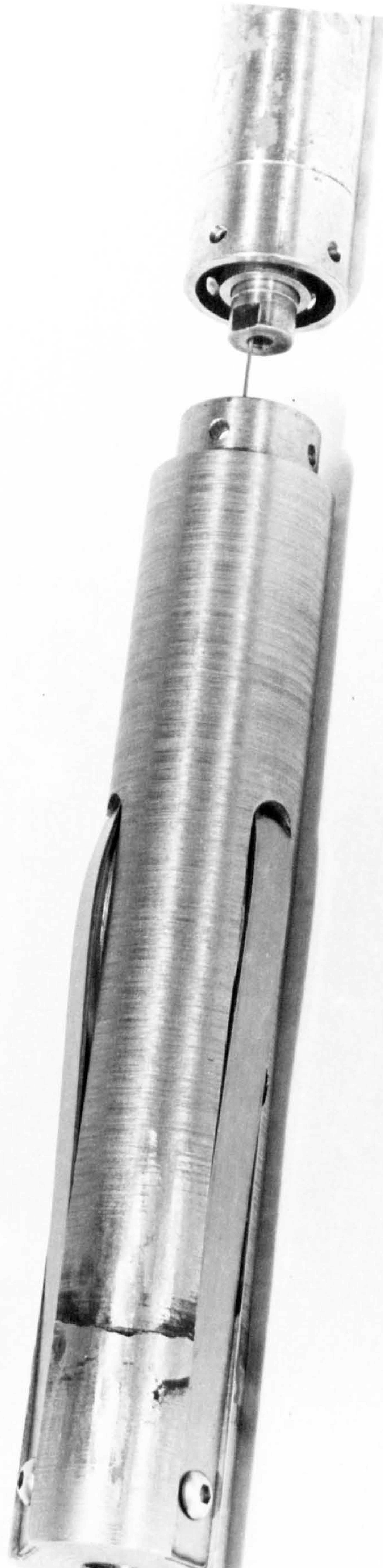


Fig. 35

the ram to be withdrawn from the hole leaving the anchor, with the wire or tape attached, in place in the hole. These anchors require a larger diameter borehole than the mechanical anchors. The hydraulic anchor shown in Fig.36 is specially designed for use in large diameter boreholes in unconsolidated strata.

The accuracy of this type of deformation measurement has been discussed in some detail elsewhere, most recently by Hedley (77). However, provided that only a limited number of anchors are installed in any one borehole and that care is taken to ensure that the wires do not become entangled during subsequent measurements, it is possible to obtain measurements using the Mk.I to an accuracy of 0.001 in., and using the Mk.II to an accuracy of 0.002 in.

4.2 <u>The Displacement of the Strata Overlying the Workings.</u>

It was felt that information concerning the stability of the overlying strata could best be obtained by drilling a borehole from the surface down through the seam and measuring the displacement of points within this borehole relative to a datum. This technique is by no means new and it has been used by numerous workers (14) (82) (84), in connection with their investigations into the strata movement in the vicinity of mine workings.

It was decided that the site of the investigation would be an exploratory borehole which was situated about 200 yds. East of the Sherburn shaft; the borehole was referred to as No.1/51, and its position shown in the underground plan, Fig.28. The installation of the equipment in this hole took place in February 1966 and observations commenced at that time, extensometer readings being taken at weekly intervals. The equipment consisted of a single hydraulic anchor, of the type shown in Fig.36, at a depth of 118 ft., i.e. two feet above the projected roof horizon in roadway 1 South, and a Mk.II Extensometer station welded to the top of the borehole casing. This casing extended down to a depth of 70 ft. and was designed to prevent the blocking of the hole by the unconsolidated surface deposits. The ground surrounding the top of the borehole was excavated to form a pit about 4 ft.square and 3 ft. deep; the floor of this pit was concreted and the pit covered with a substantial wooden lid for protection. With this equipment, it was hoped to detect any significant movements which might occur as the working headings advanced towards the borehole. Unfortunately, in July 1966, development work at the mine ceased just as the headings were about to reach the borehole, and did not recommence until July 1968. However, during the entire period of the investigations there was little or no detectable surface movement. This fact is commented upon later.

As soon as development work recommenced, weekly extensometer measurements were taken as before. Since this borehole had penetrated the proposed working horizon, it was thought inadvisable to form the roadway directly beneath the borehole until it had been sealed. This was to prevent the probable ingress of water from the overlying strata. In view of this, and in order to obtain the maximum amount of information from the borehole before it was sealed, the roadway heading 1 South, was not worked immediately. The other four headings were advanced, together with their associated cross-cuts, for some distance before the borehole was finally sealed and roadway 1 South advanced beneath the borehole. The hole was sealed in December 1968, by means of pressure grouting from the surface, at which time measurements ceased. The results obtained are discussed later.

4.3 The Design of the Underground Instrumentation Scheme.

At the time the instrumentation scheme was first proposed in February 1969, the mine was being developed by means of the 5 parallel 15 ft. wide, 10 ft. high headings, being driven to the East, forming 65 ft. long x 25 ft. wide pillars. The position of the mine workings at this time is shown in Fig.37. The overall width of the development workings was approximately 175 ft. and whilst, for a vertical depth of cover of 120 ft., this width does not provide ideal conditions for

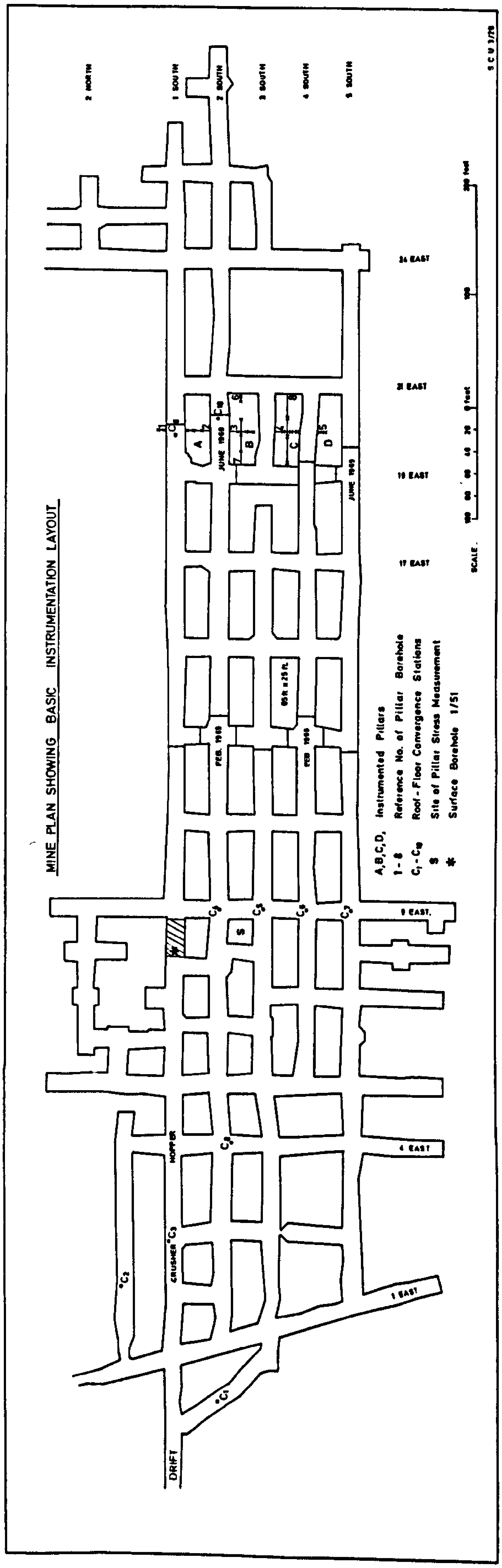

MINE PLAN SHOWING BASIC INSTRUMENTATION LAYOUT
A,B,C,D, Instrumented Pillars
1 - 8 Reference No. of Pillar Borehole
C_1 - C_{10} Roof - Floor Convergence Stations
S Site of Pillar Stress Measurement
* Surface Borehole 1/51
SCALE.
2 NORTH
1 SOUTH
2 SOUTH
3 SOUTH
4 SOUTH
5 SOUTH
1 EAST
4 EAST
9 EAST.
17 EAST
19 EAST
21 EAST
24 EAST
DRIFT
CRUSHER
HOPPER
JUNE 1968
JUNE 1969
FEB. 1968
FEB 1969

Fig. 37

strata deformation observations, it was the maximum panel width expected during the development of the mine. For this reason, coupled with the fact that the pillars were fairly large, the amount of detectable deformation, pillar deformation in particular, was expected to be fairly small.

The main factor that had to be considered in choosing a suitable site for pillar instrumentation was that a natural barrier was proposed between cross-cuts 13 and 15 East in headings 2, 3 and 4 South. The pillars between cross-cuts 17 and 19 East were therefore chosen for instrumentation, since they would be sufficiently remote from the effects of this barrier. It became known later, however, that this barrier was not to be formed but the proposed instrumentation site could not be changed in time.

On this basis, an instrumentation scheme was submitted to record the deformation taking place in selected parts of the mine. Basically, the scheme was divided into two parts, namely :-

(i) Roof - Floor Convergence.

(ii) Lateral Deformation of the Pillars.

4.3.1 <u>Roof - Floor Convergence.</u>

The installation of a number of vertical convergence stations was proposed in various parts of the development workings, to provide information regarding vertical movement throughout the mine. The siting of these stations was as follows :-

a) A total of 4 convergence stations was proposed along the line of 9 East cross-cut. These were to be installed at the centre of the intersections referred to on the plan, Fig.37, as :-

9 East - 2 South.
9 East - 3 South.
9 East - 4 South.
9 East - 5 South.

This line of convergence stations did, in fact, approximately correspond to a proposed line of subsidence measuring stations on the surface that were to be installed by the mine owners.

b) A single convergence station was proposed near the drift bottom to monitor any vertical convergence which might occur in a section of the mine regarded as completely stable. This particular roadway was the main transport roadway in and out of the mine.

c) The roadway containing the crusher and the hopper had been excavated to a depth of 30 ft. in places, and had penetrated through the gypsum seam into the underlying marl. Between the crusher and the hopper, the roadway sloped upwards to the usual roadway height of 10 ft., and it was proposed that a convergence station should be installed at this point.

d) A definite sag of the immediate roof had been indicated at the roadway intersection referred to on the plan, Fig.37, as 4 East - 2 South, and a convergence station was to be installed at this intersection to confirm this.

e) The siting of these random vertical convergence stations was completed by a proposed convergence station in the return ventilation roadway, 1 North, leading to the shaft.

These convergence stations were to be quite simple, consisting of a roof bracket and a floor extensometer boss-anchor station, the actual measurement being carried out between suitably punched holes in a stainless steel tape attached to the roof bracket and the extensometer fixed on the floor station.

When these convergence stations were first proposed, the nature of the floor with regard to the anchorage of the floor station was not known. For this reason, it was originally proposed that a 12 in. long bolt, attached to a modified B.J.B. anchor shell, be used to anchor the floor station. When the first stations were installed, however, it was realised that such anchorage was not really necessary due to the competent and solid nature of the floor, and all subsequent floor stations consisted of a short rawlbolt as shown in Fig.32. Once installed, these floor stations were to be linked to the underground datum by means of a regular levelling survey.

In addition to these scattered convergence stations, it was proposed that a large number of vertical convergence stations would be installed in the roadways surrounding the instrumented pillars. For various reasons however, described later, only two were installed.

4.3.2 Pillar Instrumentation.

In order to obtain continuous deformation measurements across the selected pillars, steel wires and anchors were to be installed in horizontal boreholes drilled through the whole width of the pillars, at approximately the middle of the pillar sides. Since the amount of lateral deformation was not expected to be very large, it was essential that the boreholes should be drilled and instrumented as soon as possible during the formation of each pillar. This was to enable a large proportion of the deformation, due to the re-distribution of the load caused by the extraction of the seam, to be recorded. Thus, as soon as the side of a pillar, in which it is proposed to drill a hole, was excavated, the borehole should be drilled for the length or width of the proposed pillar. The 1.11/16th in.B.J.B. end anchor - Fig.34 - with wire attached, would then be installed in the hole, just short of the proposed pillar edge. The intermediate anchors with their wires attached can then be positioned at various distances from the mouth of the hole. The proposed relative positions of these anchors, only three in each hole to minimise wire interference, were 8 ft. (2.438 m.), 16 ft.(4.877 m), and 23 ft. (7.01 m.). Finally, an anchor is installed at the mouth of the borehole to which all measurements are related.

It was proposed originally, that the 4 pillars between cross-cuts 17 and 19 East would be instrumented in this manner, the boreholes being drilled through the shorter width of the pillars. In addition, boreholes were to be drilled and instrumented through the length of the two centre pillars. In this case, the anchor distances were to be 21 ft. (6.40 m.), 42 ft. (12.80 m.), and 63 ft. (19.20 m.).

It was also proposed to install anchors and wires in boreholes drilled into the solid ribsides of roadways 1 South and 5 South. The length of these boreholes were to be dependent on the drilling equipment and time available.

For various reasons, which will be discussed later, it was not possible to install this pillar instrumentation scheme in full, though most of that proposed, was in fact carried out. Also, the site of instrumentation had to be changed to the set of pillars between cross-cuts 19 and 21 East. The revised pillar instrumentation scheme is shown in Figs. 37 and 38.

4.4 The Installation of the Equipment Underground.

The installation of the equipment took almost 9 months to complete. Because of this, if for no other reason, it is felt necessary to comment on the difficulties encountered and the reasons for the revising of the instrumentation scheme.

4.4.1 Roof - Floor Convergence.

A total of 10 roof-floor convergence stations were eventually installed at pre-selected sites in the mine. The siting of these stations is shown in Fig.37, and denoted by the letter C followed by the station reference number, i.e. Stations C1 - C10.

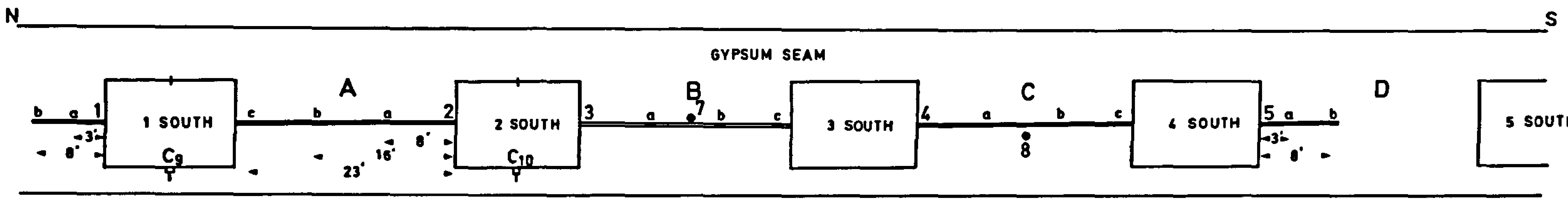
The convergence stations C1 - C8 were installed by the 13th March 1969, without any real difficulty. Measurements were taken at these stations at regular intervals following this. The convergence station C9 and C10 were part of a series of convergence stations that were installed around the instrumented pillars, the remainder of which, for various reasons to be described in the following section, were not installed. These two convergence stations were installed on the 26th August 1969. Unfortunately, within a very short time, station C10 was destroyed before any significant amount of measurements could be obtained, so that no results are presented for this station.

All the floor stations were linked by levelling at regular intervals to determine any change in floor level at each station, the actual levelling being carried out by surveyors employed by the mine owners.

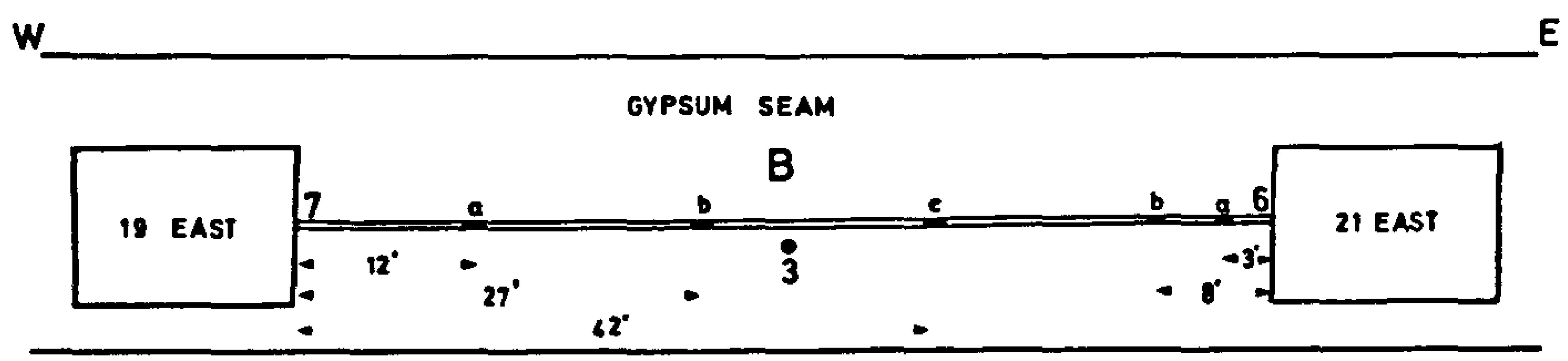
4.4.2 Instrumentation of Pillars.

As mentioned previously, the set of pillars between crosscuts 17 and 19 East were originally chosen as the site for the measurement of pillar lateral deformation. Unfortunately, when this set of pillars was about to be formed, geological difficulties were encountered in roadways 3 and 4 South, which resulted in a temporary suspension of work in these two headings. Also, the drilling machine which was to be used to drill these boreholes was not available, and by the time it was, the roadways 1 and 2 South, and to some extent 5 South, had been advanced well beyond this set of pillars. On the 22nd May 1969, working

N-S SECTION THROUGH INSTRUMENTED PILLARS A, B, C, AND D.



W-E SECTION THROUGH INSTRUMENTED PILLAR B.



W-E SECTION THROUGH INSTRUMENTED PILLAR C.

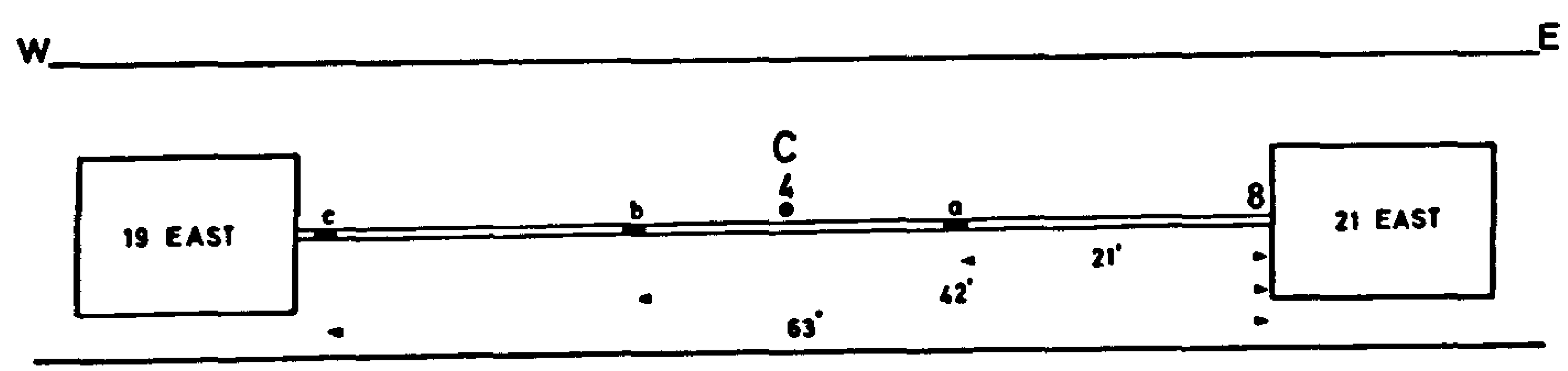


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FIG. 38

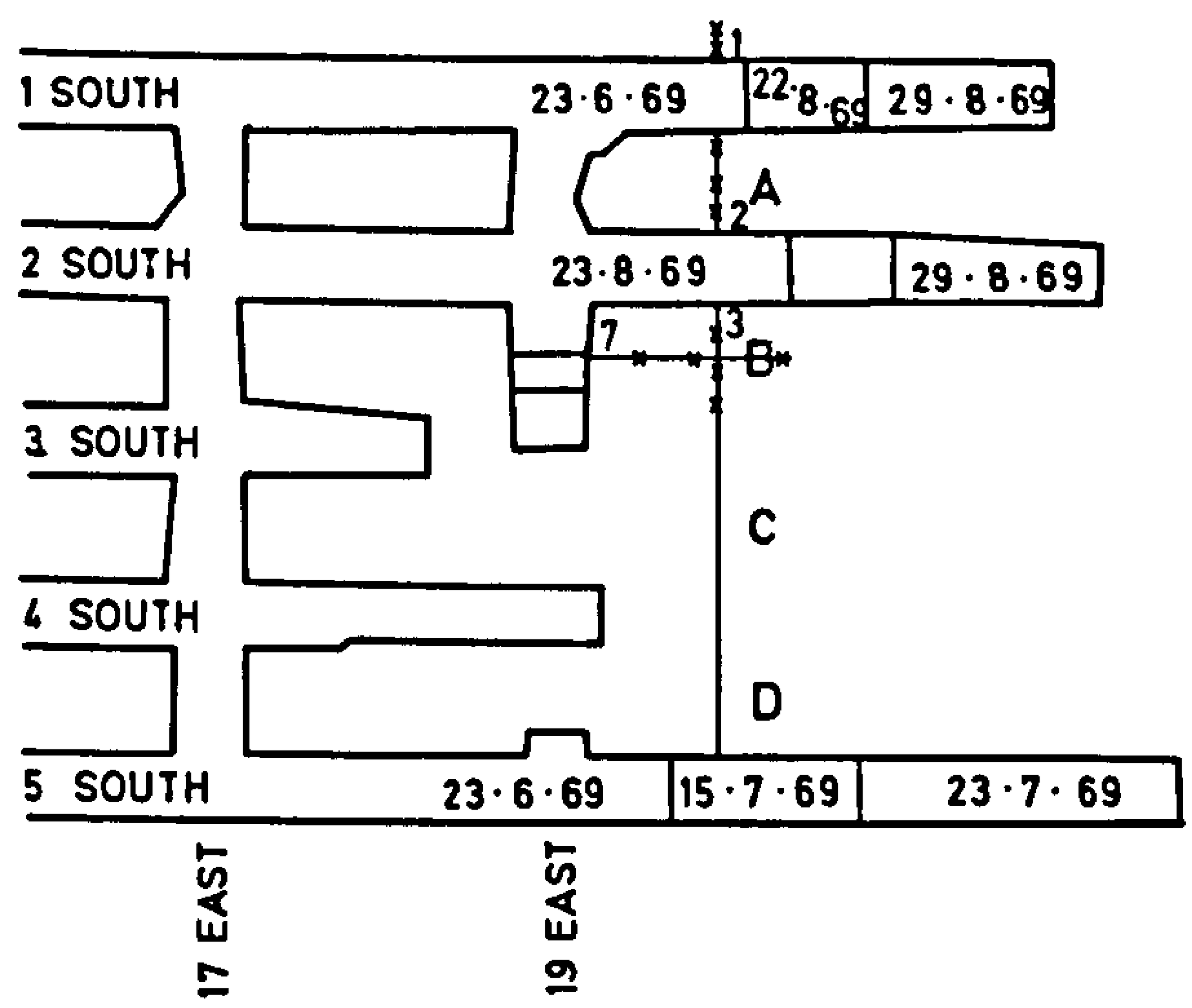
of these headings was temporarily suspended, the position of the working faces being shown in Fig.37. The set of pillars between cross-cuts 17 and 19 East had been virtually formed and no boreholes for instrumentation had been drilled.

The problem at this time therefore, was whether or not to proceed with the instrumentation as planned since a large proportion of the probable pillar deformation would have already taken place. It was decided to take advantage of the fact that the roadways 1 and 2 South, were far enough advanced for some boreholes to be drilled and partly instrumented, in the pillars between cross-cuts 19 and 21 East, before work resumed in these headings. The intention therefore, was to use the same instrumentation scheme proposed for the pillars between cross-cuts 17 and 19 East, but in the next set of pillars between cross-cuts 19 and 21 East, referred to hereafter as pillars A, B, C and D as shown in Fig.37.

A borehole, 1.11/16 in. diameter, was drilled through the whole width of pillar A from roadway 2 South. Another borehole of the same diameter was then drilled N - S from the same position passing through the proposed pillars B, C and D. It was thought that the benefit gained by being able to instrument, at the earliest possible opportunity, each part of this long borehole when the various roadways intersected it, offset the risk, due to any change in working level, that the various roadways may not intersect the borehole at the correct height.

The boreholes referred to on the plans as numbers 2 and 3, in pillars A and B respectively, were then fitted with steel wires fixed to the mechanical anchors shown in Fig. 34. The actual arrangement of the anchors is shown in the pillar section drawing, Fig.38. The first measurement was taken on these boreholes with the Mk.I Extensometer, on the 27th June 1968. No

EXTRACTION PLAN (i)



EXTRACTION PLAN (ii)

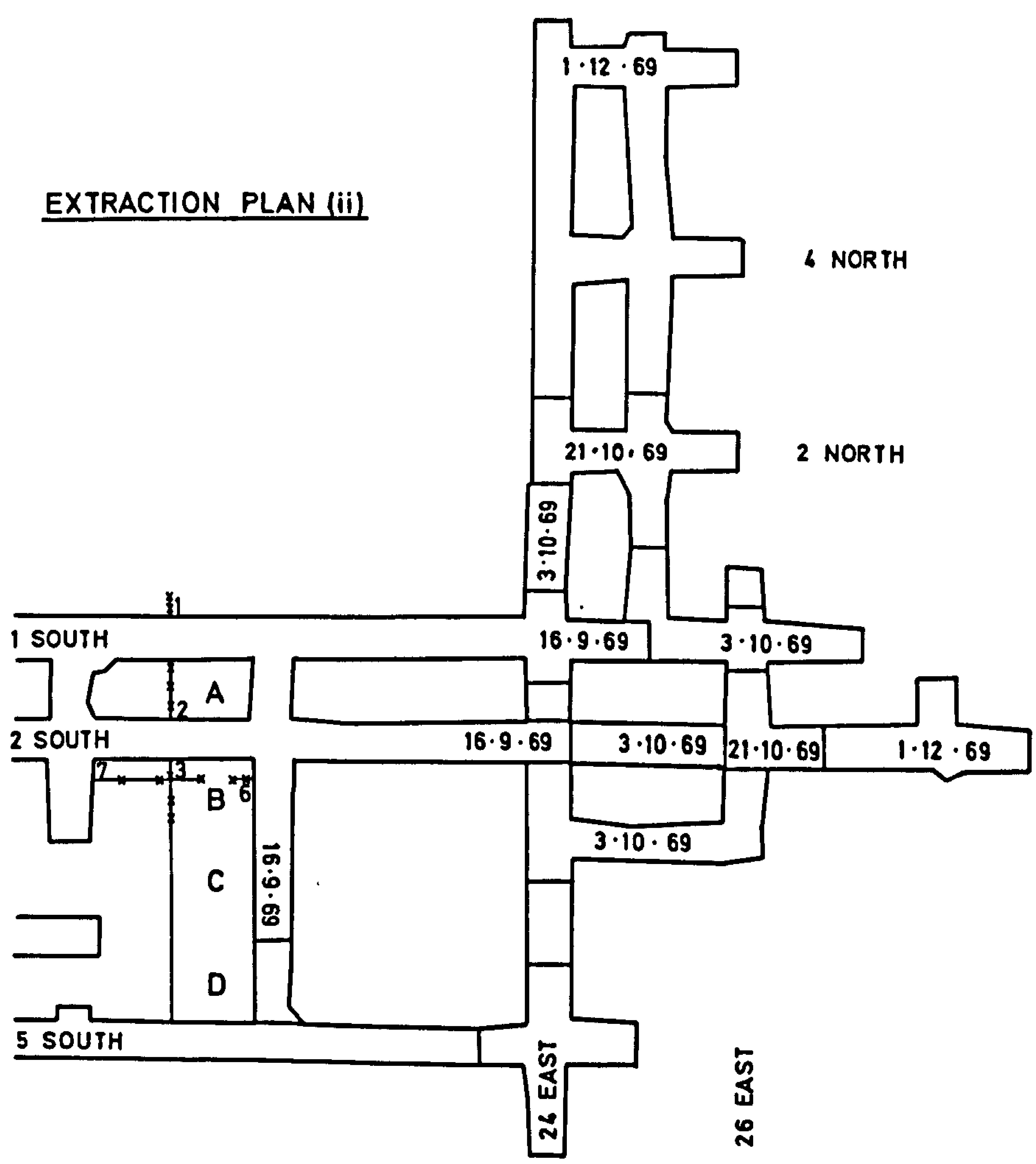


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FIG. 39

convergence stations could be installed at this time since both roadways were extensively flooded, roadway 1 South in particular.

Following this, A Dosco Mining Machine was used to extend roadway 5 South for a distance of 140 ft. before being removed. It had been hoped that this roadway would have intersected the long borehole, but the roadway was inclined downward somewhat, and this together with the fact that the Machine did not form the ususal roadway height, meant that the borehole passed above the roadway.

On the 18th August 1969, the usual procedure of drilling and blasting was resumed in headings 1 and 2 South. The extraction plans shown in Figs.39 and 40, show the position of the working faces on those days that measurements were taken. Due to the unavailability of the large drilling machine, a smaller machine had to be used to drill borehole 1 in the solid ribside. This borehole was drilled and instrumented on the 29th August 1969, to a depth of 8 ft., the anchors being placed at a distance of 7 ft. 6 in. (2.286 m.) and 3 ft. (0.914 m.) from the mouth of the hole. At the same time, the two convergence stations, C9 and C10, were installed in roadways 1 and 2 South, together with borehole 7 in pillar B. Due to the length of this borehole, it was found to be impossible to place an anchor at the required distance of 63 ft. from the mouth of the hole, due to the force required to set the mechanical anchor at this distance. For this reason, the anchors were sited at distances of 12 ft. (3.66 m.), 27 ft. (8.23 m.), and 42 ft (12.80 m.) from the mouth of the borehole, as shown in Fig.38. To partly offset this, a short borehole, referred to as borehole 6, was drilled with the small drilling machine from cross-cut 21 East in pillar B, along the line of borehole 7. Anchors were installed in this hole at depths of 7 ft. 6 in. and 3 ft., as in borehole 1.

DEC. 1st.-9th. 1969.

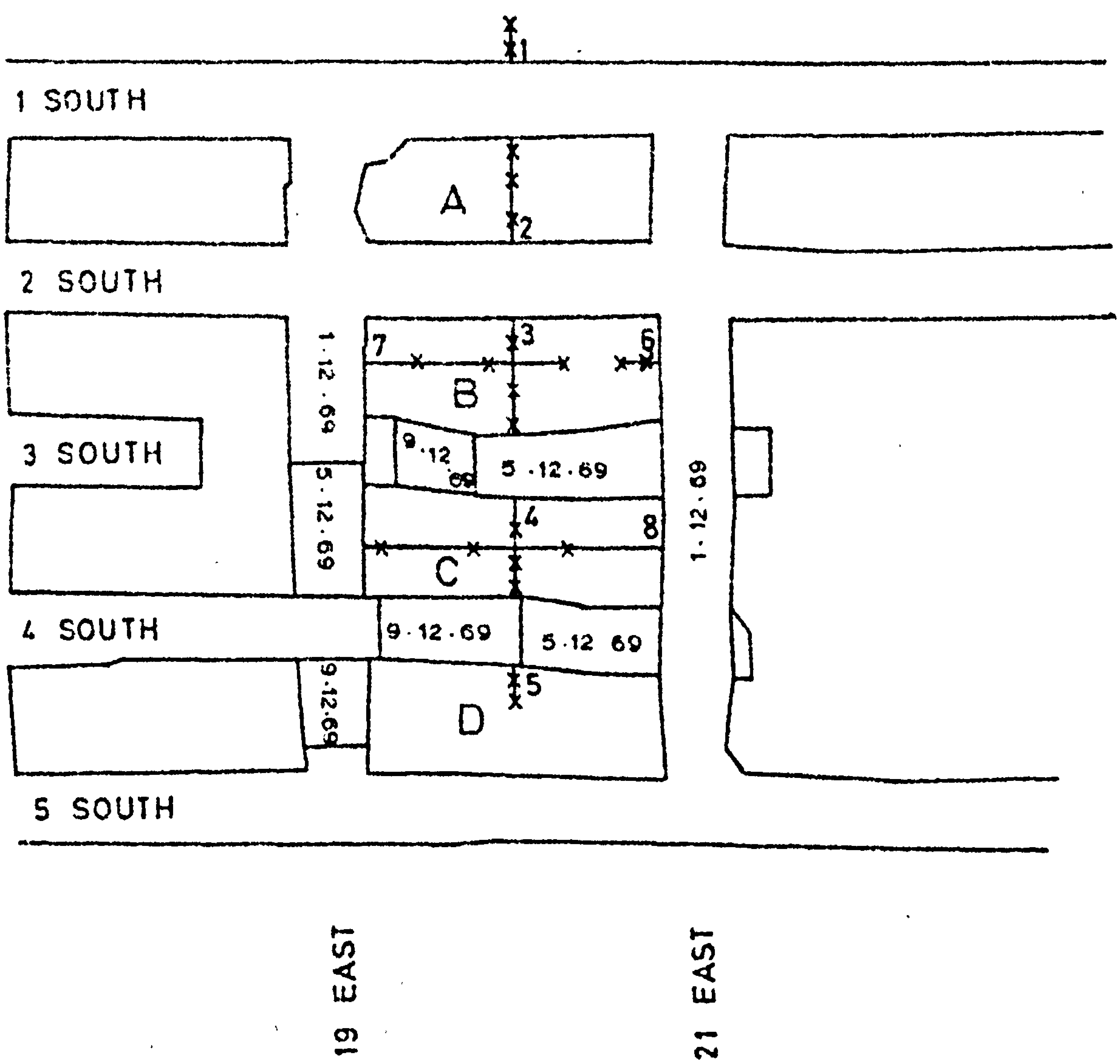

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FIG. 40

It will be noticed from the plans of extraction shown in Figs.39 and 40, that the pillars B, C and D were not immediately formed. A two heading mining system was used leaving roadways 3, 4 and 5 South unworked. Cross-cut 21 East was formed leaving a large pillar made up of the proposed smaller pillars B, C and D. The roadways 1 and 2 South were driven for some distance eastwards, measurements being taken on the instrumented boreholes throughout, before finally turning to the North. The cross-cut 24 East was then driven to the South to form up with roadway 5 South, Figs.39 and 40.

It was not until the 1st December 1969, that work restarted on forming pillars B, C and D. By this time, boreholes 1, 2, 3, 6 and 7 had been instrumented, and measurements subsequently taken for more than 5 months in the case of boreholes 2 and 3. Since the distance had been found to be too great for mechanical anchors in borehole 7, it was decided that in the case of borehole 8 in pillar C, a hydraulic anchor of the type shown in Fig.35. would be used. This required a borehole of 2¼ in. diameter. The borehole was drilled from 21 East in pillar C, the hydraulic anchor being installed at a distance of 63 ft.(19.20 m.) from the mouth of the hole. Further mechanical anchors were then installed at distances of 42 ft. (12.80 m.) and 21 ft. (6.40 m.) from the mouth of the hole. This was completed before the block was worked.

The pillars B, C and D were completely formed between the 1st and 9th December 1969, as shown in Fig.40. When it became possible, boreholes 4 and 5 were instrumented. Borehole 4 was in fact, part of the long borehole that had been drilled previously, whilst borehole 5 was a short borehole drilled and instrumented in the same way as boreholes 1 and 6, since that part of the long borehole passing through pillar D was found to be too high.

No further convergence stations were installed around these pillars for a number of reasons. Only 8 days were taken to form the pillars and the convergence stations could not be installed during this time due to the machinery being used. Also, once these pillars were formed no further extraction would take place in the near vicinity. The working face had advanced to some distance away and very little convergence could have been related to the working of these or any surrounding pillars. In addition to this, a very large amount of water was collecting in these roadways thus preventing firstly, the installation of the floor station, and secondly, the obtaining of subsequent measurements. For these reasons therefore, it was decided not to install any further convergence stations in this area.

5. ANALYSIS OF THE RESULTS OBTAINED FROM THE 'IN SITU' INVESTIGATIONS.

This section is concerned with the analysis and discussion of the various 'in situ' deformation measurements carried out at the Sherburn mine. Since the measurements were obtained in a number of different circumstances, the results obtained will be discussed under three headings :-

(i) The Surface Borehole Measurements.

(ii) Roof - Floor Convergence Measurements.

(iii) Pillar Lateral Deformation Measurements.

It will be noticed that the deformation measurements obtained are shown as a function of time. With regard to the vertical convergence and pillar lateral deformation measurements, the latter in particular, they should also be compared with the extraction plans shown in Figs.39 and 40, to give a relationship between the deformation recorded and the pattern of extraction.

5.1 The Surface Borehole Measurements.

The apparent movement of the surface overlying the mine workings up to July 1968 was so insignificant that no attempt was made to present it graphically. In the first 8 months of observations some very small deformations occurred, apparently at random, and then the readings became constant and remained so until July 1968 when underground working recommenced. In the following months, approximately weekly measurements were taken up to the 6th December 1968, when the borehole was finally sealed. The results obtained are presented graphically in Fig.41 from the 20th September 1968. Because there was no stable reference anchor installed in the borehole, the results are presented as the movement of the 'mouth of hole' station, or the surface, relative to the anchor installed in the seam.

The most significant fact concerning this measured deformation was the small magnitude of the apparent surface movement, though the fact that the borehole anchor was never undermined whilst measurements were being taken should be taken into consideration. In the immediate 4 - 6 weeks following the resumption of underground working in July 1968, no measurable deformation occurred in this borehole. As the development headings were being formed around the base of the borehole however, some relative movement became apparent in the form of a slight lowering of the surface at this point. This continued up to the beginning of November 1968, at which time there was a sudden upward movement of the surface relative to the borehole anchor, greater than the total downward movement of the surface that had been measured up to that time. At the time this actual measurement was taken, the surface surrounding the borehole was flooded to a depth of several feet following several days of torrential rain. It is conceivable therefore,that the influence of this ground water could cause the surface to rise since the majority of soils display volume changes as their moisture content changes. In addition to this, the hydrostatic forces associated with changes in the level of the water table could possibly result in the uplift of the surface.

Four days following this actual measurement when the flood water had disappeared, it was found that the surface had again subsided

MOVEMENT OF THE SURFACE RELATIVE TO THE ANCHOR.

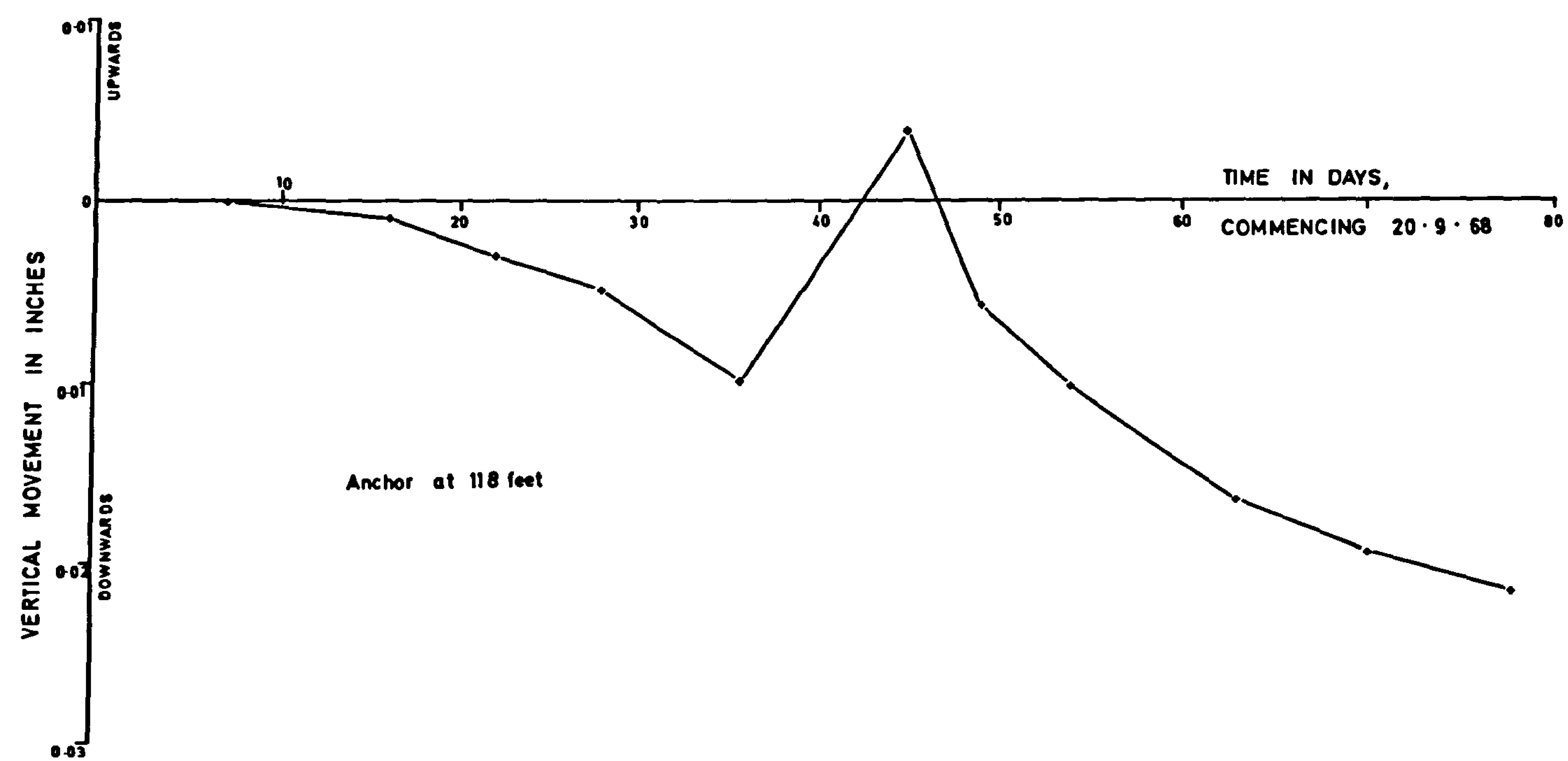

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FIG. 41

relative to the borehole anchor. In the following weeks this downward movement continued as extraction proceeded, until the final reading was taken on the 6th December 1968, at which stage it was found that the surface point had moved downwards relative to the borehole anchor a distance of 0.022 ins.

This observed vertical movement is so small that doubts could be raised about its actual occurrence in view of the conditions prevailing at the measuring site, though the trend of the measurements indicated that what in fact was measured did occur. The limits of the accuracy of the Mk.II Extensometer have already been mentioned; under good conditions the instrument should give readings which agree to within ± 0.002 in., the actual readings taken at the time of each measurement in fact agreeing to within ± 0.001 in. Though this accuracy tends to be reduced by the presence of additional measuring tapes in the borehole, since only one tape was installed in the borehole, no loss of accuracy was expected in this respect.

If the influence of variations of temperature on the behaviour of the measuring equipment is considered, it could have a marked effect on the reliability of the results. A temperature change of 1° F. over the length of the borehole would cause the length of the tape to change by 0.026 in., a distance greater than the total movement observed. Usually however, the effect of temperature changes may be safely ignored as the variation of temperature at the surface has virtually no effect on the borehole temperature which is stabilised by the surrounding strata.

The factor which could possibly mask the actual surface movement due to the mining operations is the variation in the amount of water present in the overlying strata. Some phenomena related to this has already been mentioned and in addition, the surface at Sherburn is only 25 ft. above sea level and as it is completely waterlogged, it is not unlikely that there could be some slight height variation which is attributable to tidal effects

Hence, while it is not suggested that the surface movement observed at Sherburn should be considered as due to these factors, in view of the low magnitude of the movement recorded they should be noted in any conclusions that may be drawn from the results. It should also be remembered however, that the borehole was not that between the surface and the roof beam above the roadway as was first intended, but the relative movement above the edge of the workings or the solid ribside. In any event, the possible surface movement measured at this point must be considered as being insignificant, certainly it does not justify drawing conclusions regarding the stability of the strata overlying the actual workings.

5.2 Roof - Floor Convergence Measurements.

As mentioned previously, a total of 10 convergence stations were installed, two of which, C9 and C10, were to be part of a series surrounding the instrumented pillars. The stations C1 - C8 were all installed by the 13th March 1969, the first measurement being taken on that date, whilst the convergence stations C9 and C10 were installed on the 29th August 1969, C10 subsequently being destroyed. The amount of convergence recorded at stations C1 - C9, with the exception of station C6, is shown in Figs.42 - 49, the convergence being shown in both mms. and ins.

The results obtained from these convergence stations are rather interesting because in almost every case they show an almost perfect linear relationship with the passage of time. There is some variation from this in a few instances for which possible explanations can be given.

It will be noticed from Figs.42 - 44, that the convergence recorded at stations C1 (5.66 mm), C2 (2.89 mm) and C3 (4.42 mm) after 445 days is lower than that recorded at the other stations, though it is increasing at an almost constant rate. No mining has taken place in the vicinity of these three stations following their installation which may explain why there has been no sudden change in the

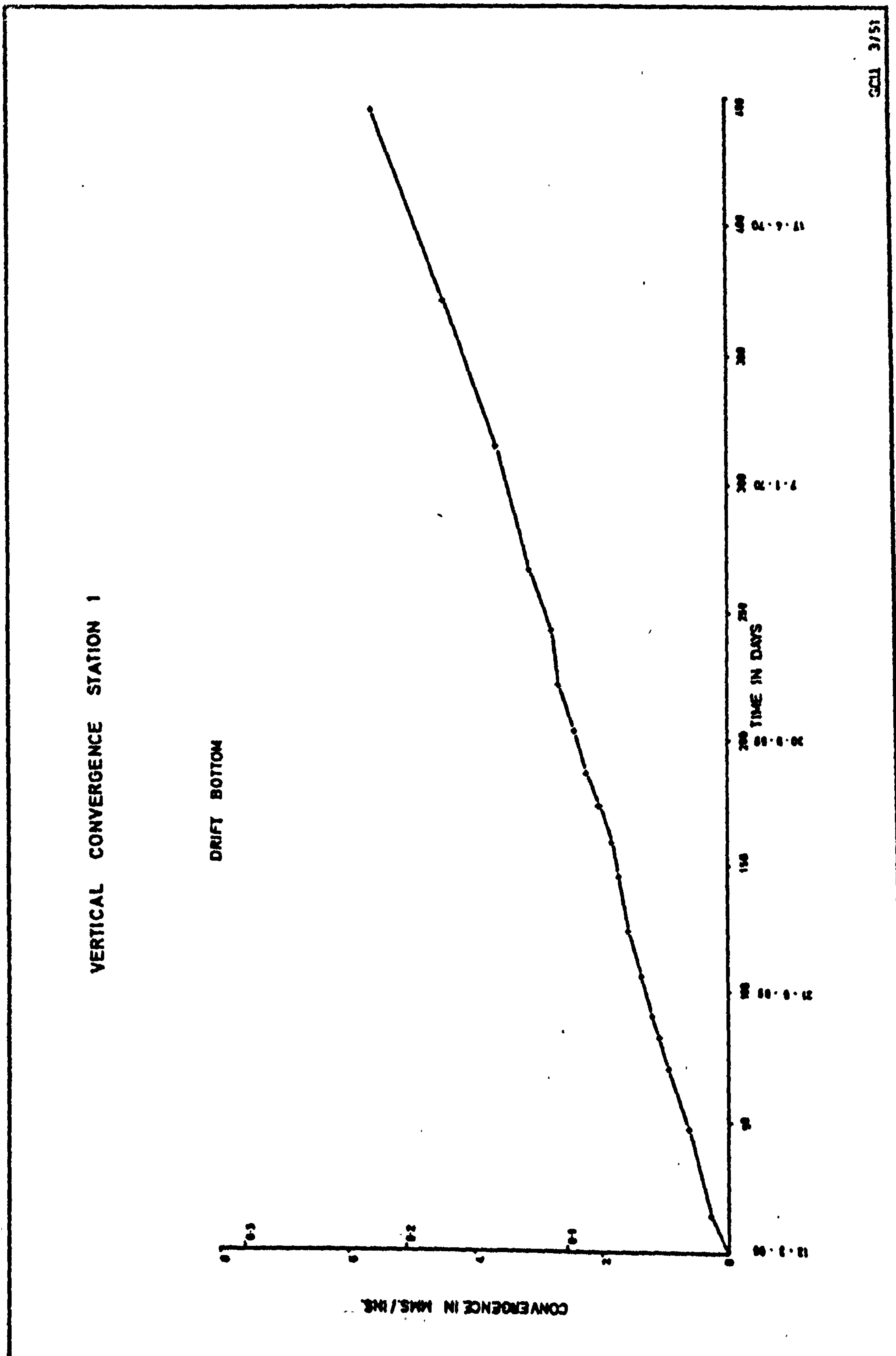
VERTICAL CONVERGENCE STATION 1
DRIFT BOTTOM
TIME IN DAYS
CONVERGENCE IN MMS./INS.

Fig. 42

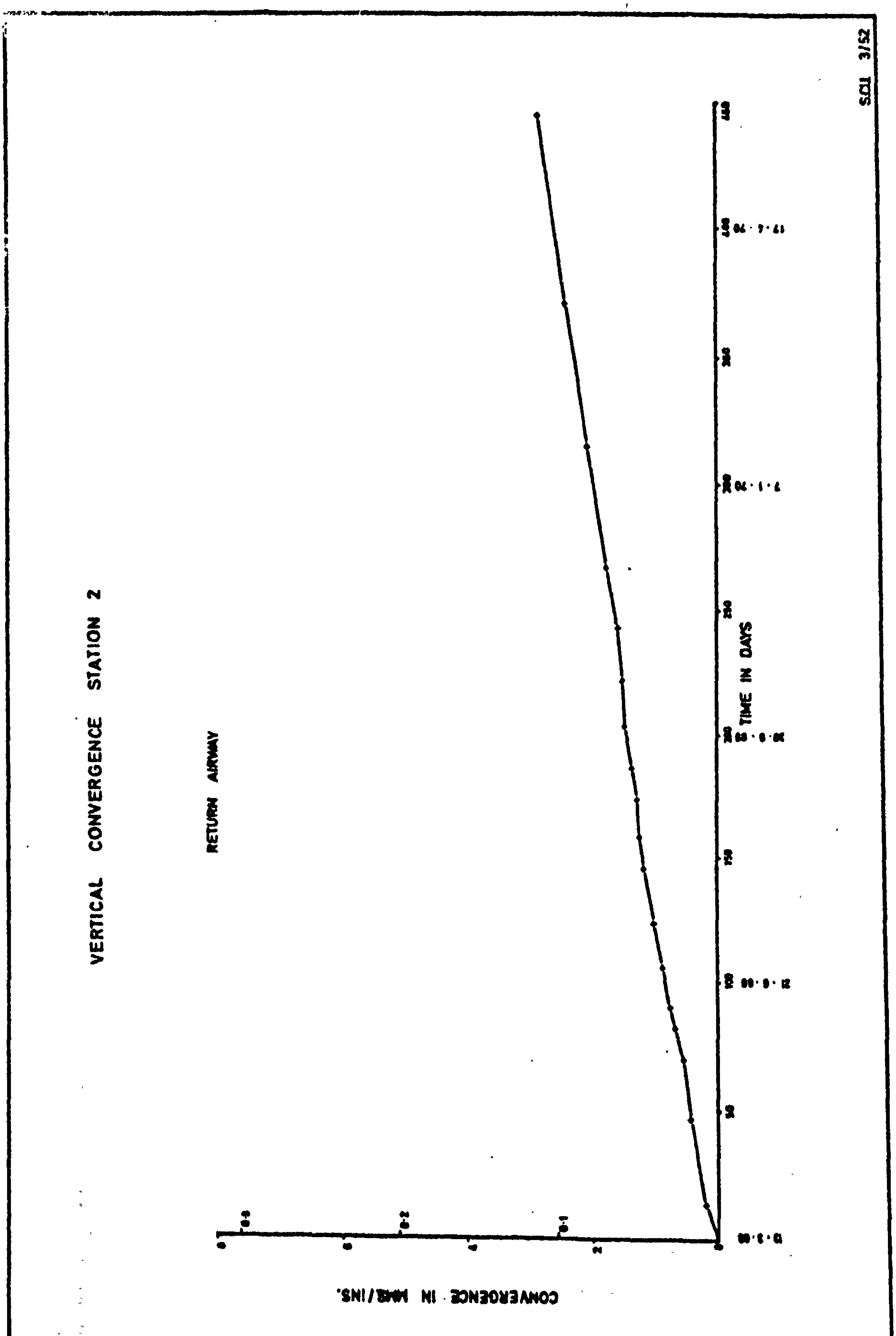
VERTICAL CONVERGENCE STATION 2
RETURN AIRWAY
TIME IN DAYS
CONVERGENCE IN MMS/INS.
SCI1 3/52

Fig. 43

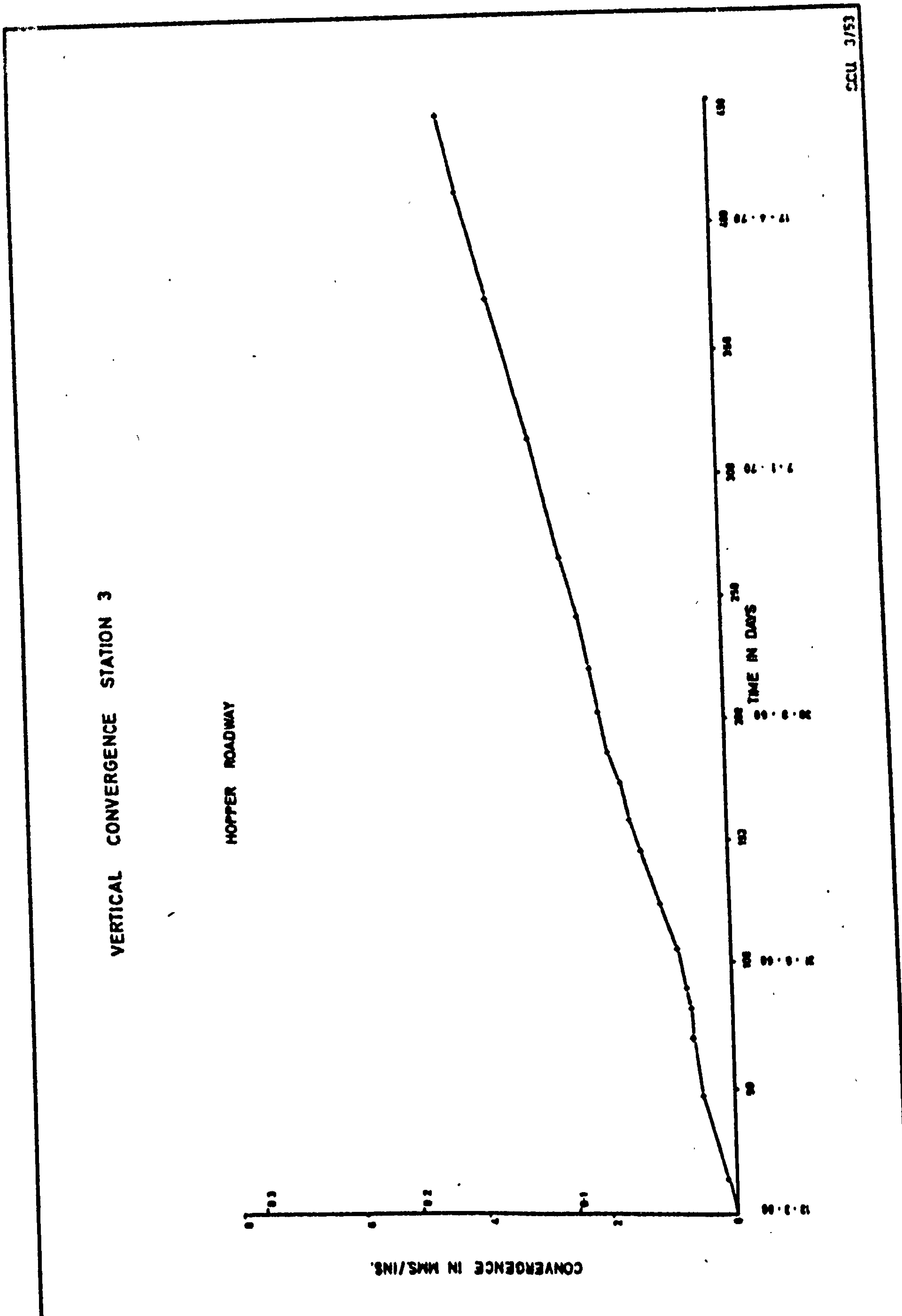
VERTICAL CONVERGENCE STATION 3
HOPPER ROADWAY
TIME IN DAYS
CONVERGENCE IN MMS/INS.
CCU 3/53

Fig. 44

rate of convergence in the time following this installation. The lower total convergence recorded at C1, C2 and C3 when compared with the other convergence stations, may be accounted for by examining their position on the mine plan, Fig.37. The stations C1 and C2 are situated at the centre of the roadways that are adjacent to a solid ribside. The station C3 could not be placed in the middle of the roadway because of the conveyor between the hopper and the crusher. The roof-floor convergence therefore, at the centre of this roadway between the hopper and the crusher, can possibly be expected to be higher than that recorded by C3.

The convergence recorded by stations C4, C5 and C7, situated at the centre of roadway intersections, was much higher than at any of the other stations. This is shown in Figs.45 - 47. These stations have at various times been difficult to obtain measurements from due to the large amount of water that has accumulated in this area. In fact, there have been only two measurements taken on C6 since it was first installed in March 1969.

It will be noticed from Fig.45, that the early behaviour of the convergence recorded at C4 was unusual. In the first 50 days following installation, station C4 recorded very little convergence. Then there was a sudden increase in the amount of convergence but unfortunately, there was a large time gap between measurements at this time, due mainly to the fact that C4 is sited on the main transport roadway, and therefore difficult to obtain a measurement from. During this time however, the adjacent roadway 1 South, was formed beneath the surface borehole 1/51 mentioned previously, resulting in the isolation of the pillar adjacent to C4. This is shown in Fig.37. Whilst this does not account for the fact that very little convergence was recorded prior to this, C1 and C2 also adjacent to solid ribsides recorded some relatively considerable convergence during the same period of time, it could account for the sudden increase in convergence rate. Following this, the convergence recorded at C4 increased

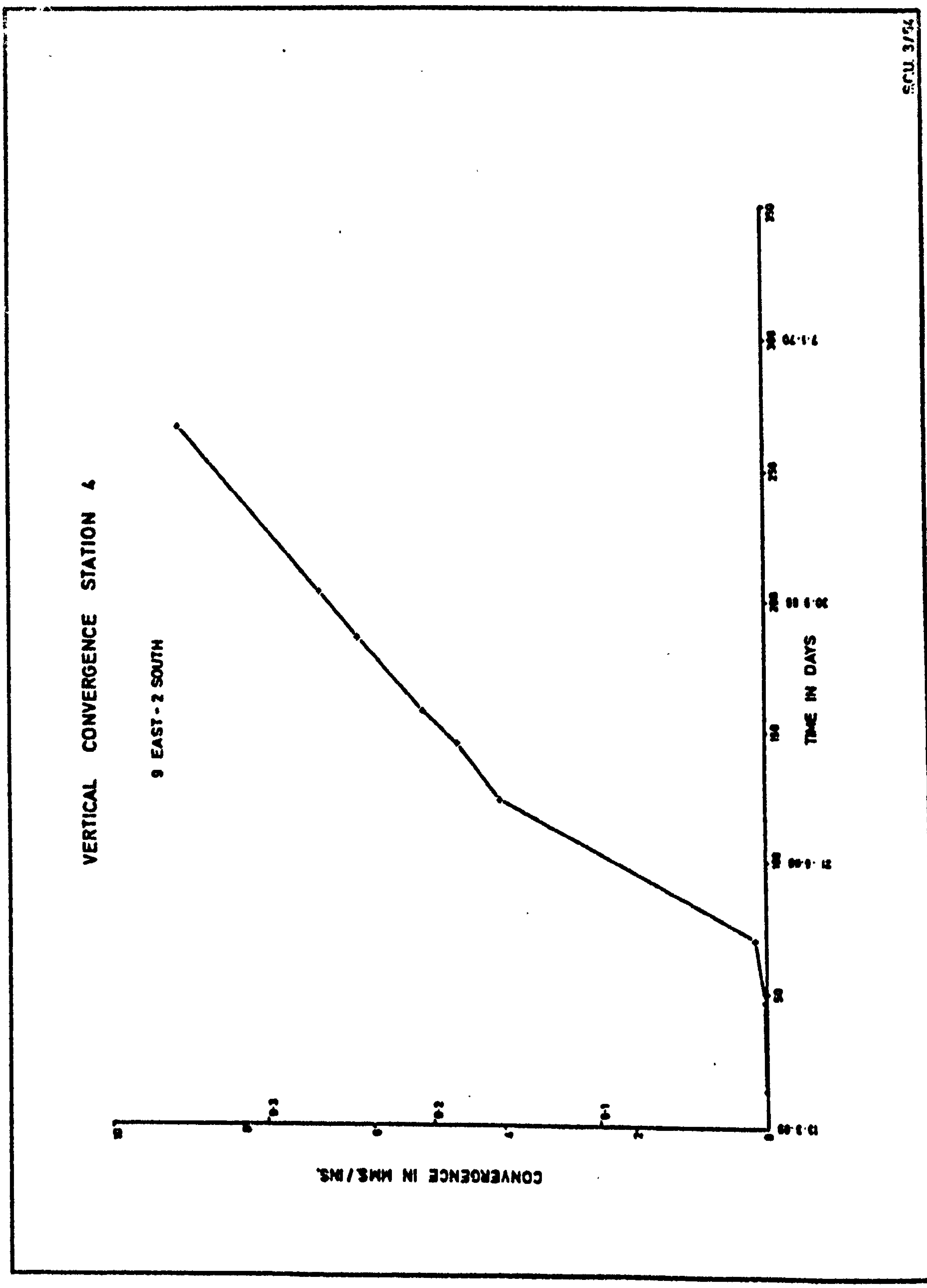
VERTICAL CONVERGENCE STATION 4
9 EAST - 2 SOUTH
TIME IN DAYS
CONVERGENCE IN MMS/INS.

Fig. 45

VERTICAL CONVERGENCE STATION 5

9 EAST - 3 SOUTH

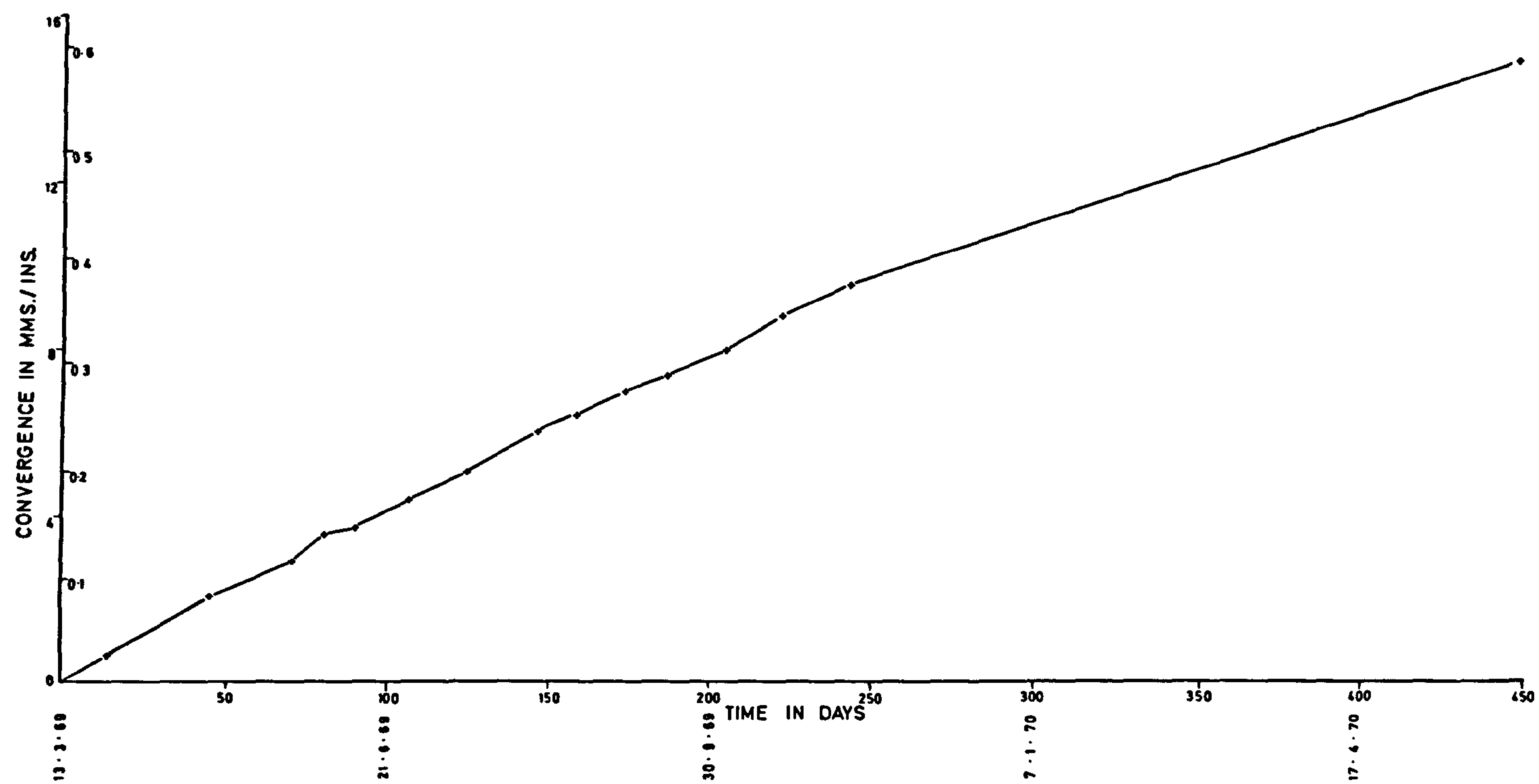

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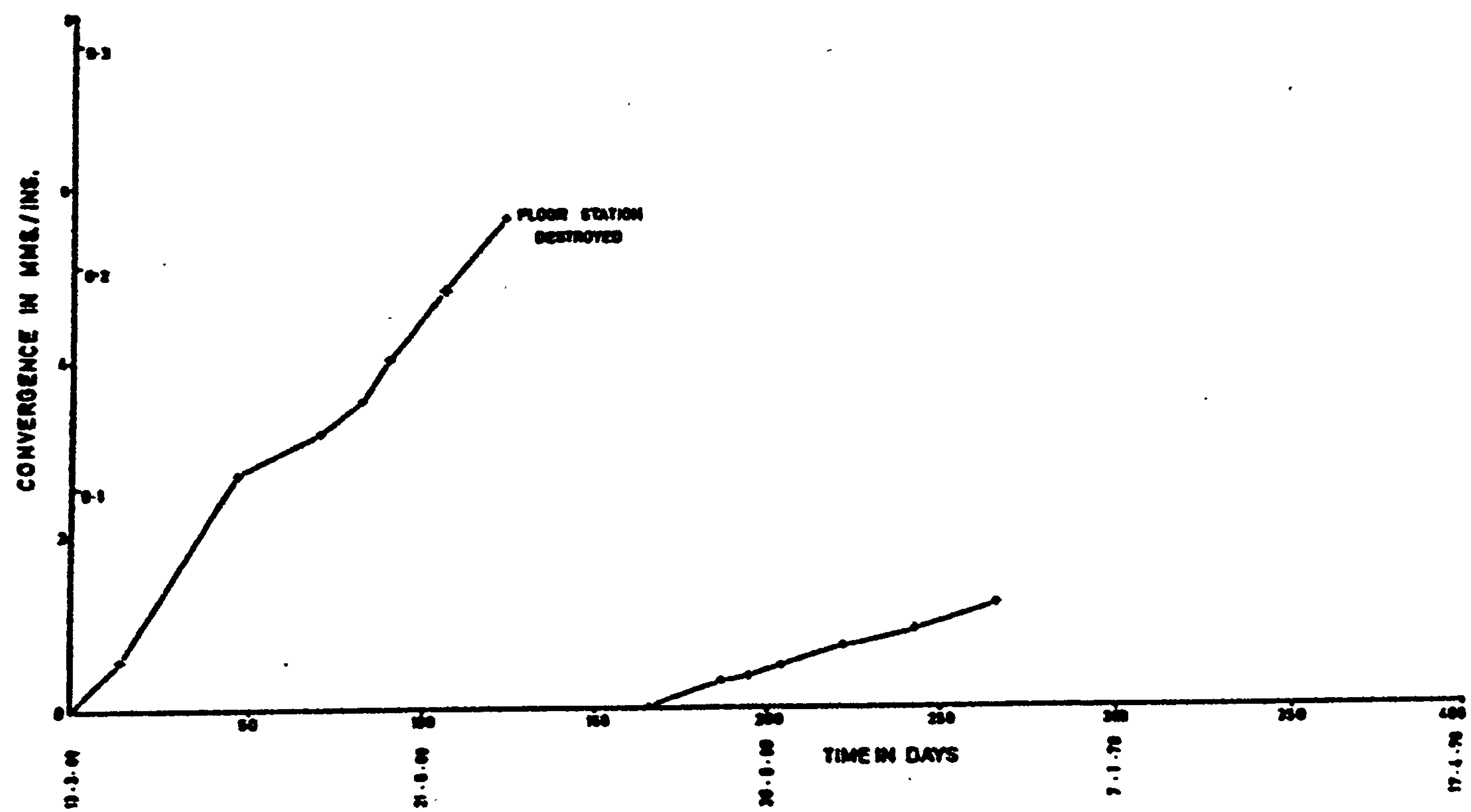
FIG. 46

VERTICAL CONVERGENCE STATION 7

9 EAST - 5 SOUTH



SCLI 3/57

Fig. 47

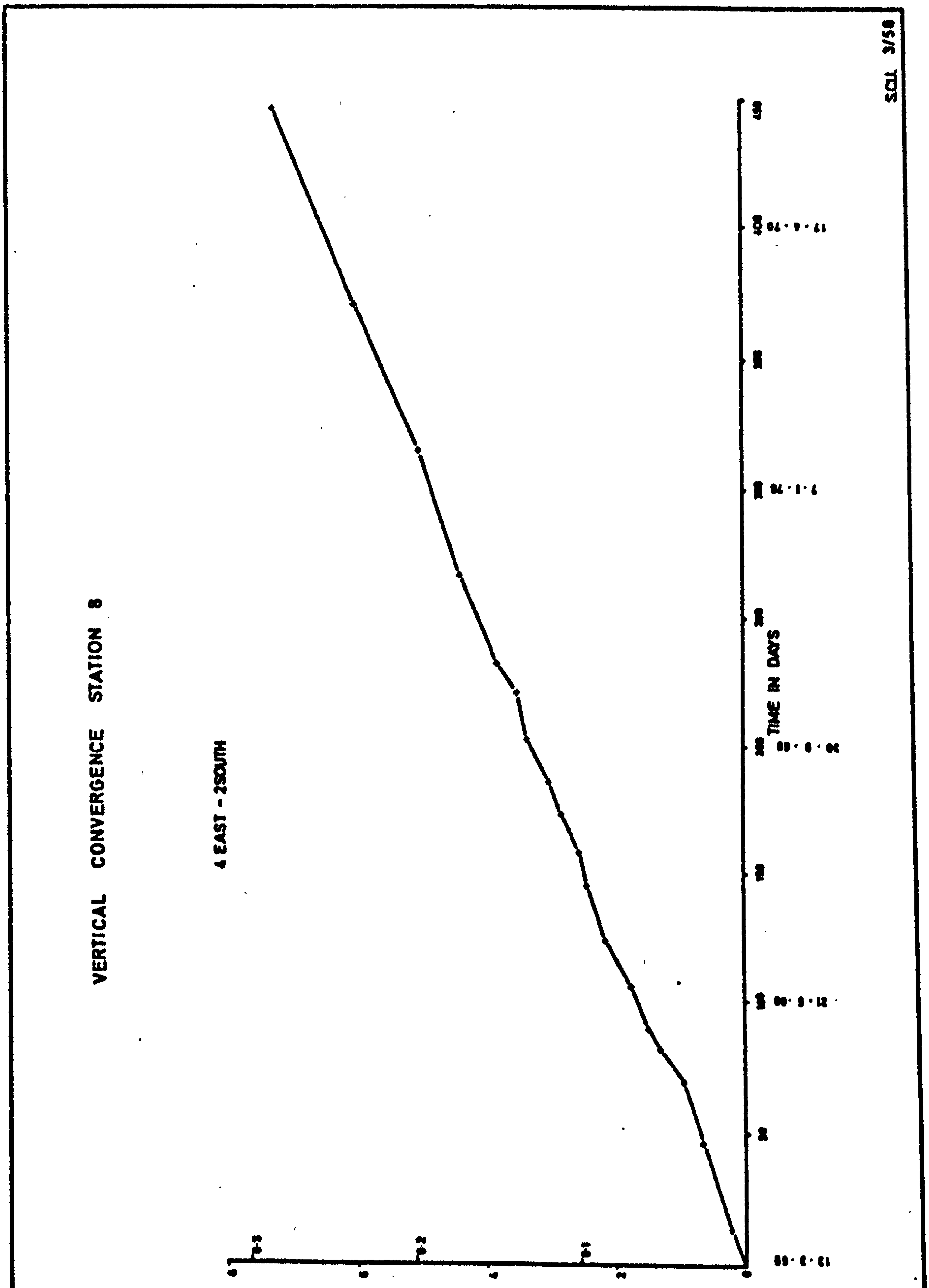

VERTICAL CONVERGENCE STATION 8
4 EAST - 2SOUTH
TIME IN DAYS
CONVERGENCE IN MMS./INS.
SCL1 3/56

Fig. 48

at a constant rate until December 1969 at which time the station was damaged by one of the mine vehicles. At this time, a total convergence of 8.99 mm. had been recorded over a period of 267 days, following installation.

The convergence recorded at C5 and C7 increased at almost the same constant rate following their installation as shown in Figs.46 and 47. Unfortunately, C7 was destroyed after 124 days when a concrete roadway was laid down, and had to be replaced. At this stage, a total convergence of 5.64 mm. had been recorded. The convergence rate at the new station C7 was found to be much lower, though still fairly constant. This lower rate of convergence may be explained by the fact that this new station had to be sited nearer to the pillar side because of the thickness of the concrete. No measurements have been possible on C7 since December 1969 because of the depth of water, though the amount of convergence that has occurred up to the present time could possibly be estimated since it was increasing at an almost constant rate prior to December 1969.

The same difficulty, due to the water, of obtaining measurements was experienced with C5, only two measurements being possible in the 6 months following December 1969. These have indicated that the rate of convergence at this point could possibly be slowing down though further measurements should be carried out to confirm or deny this. The total convergence measured at this station after 445 days (14.91 mm) is by far the highest recorded. This is understandable since it is situated at the centre of the development workings.

The convergence recorded at C8 has increased at an almost constant rate since it was installed. As may be seen from Fig.48, this station was not sited at the centre of an intersection because it was thought that it would probably be damaged by mine tractors turning for the hopper. Even so, the total convergence recorded (7.34 mm) after 445 days is higher than that recorded by C1, C2 and C3, though not so high as that recorded by C4, C5 and C7.

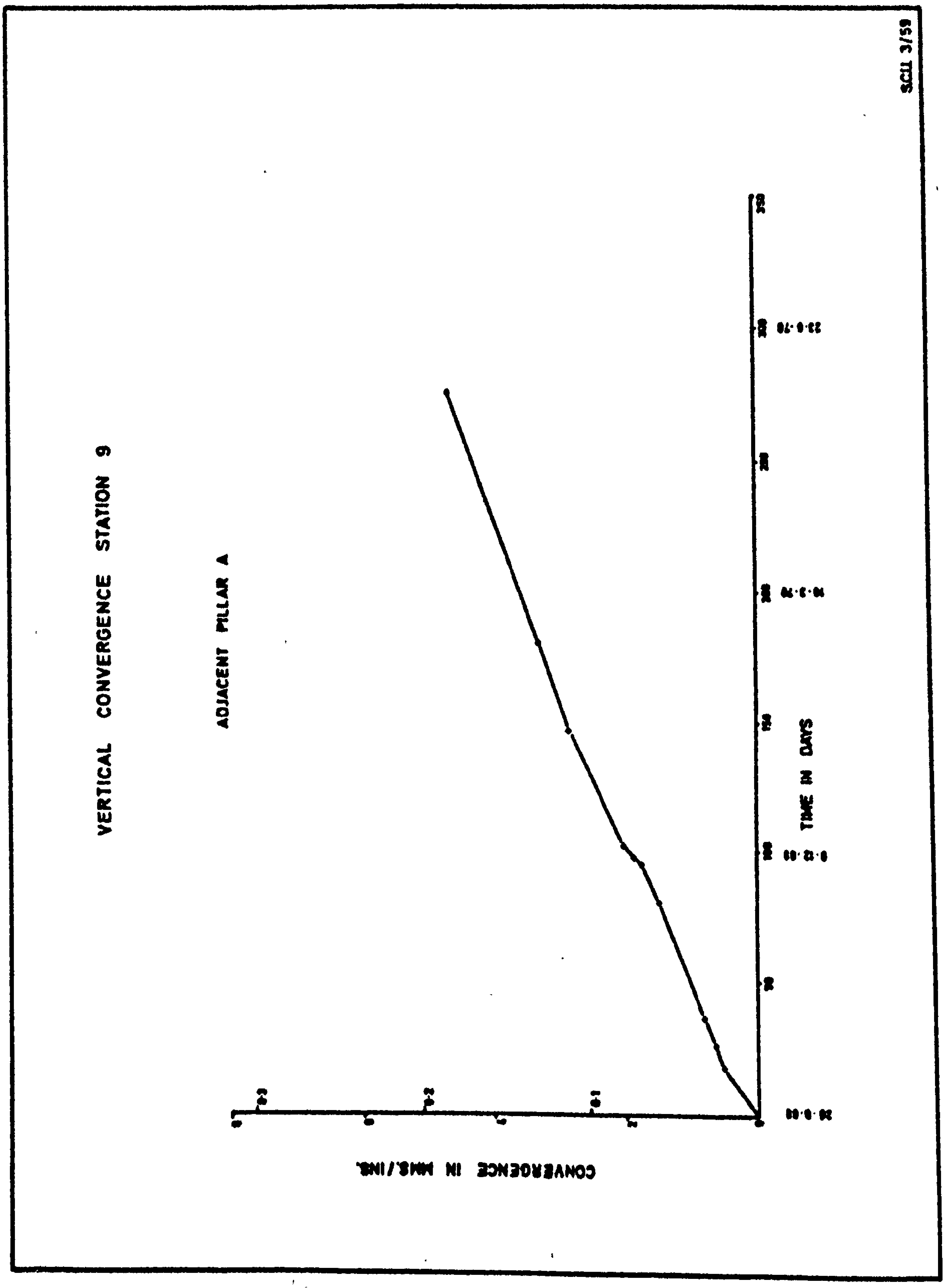

VERTICAL CONVERGENCE STATION 9
ADJACENT PILLAR A
TIME IN DAYS
CONVERGENCE IN MMS./INS.
S.C.11 3/59

Fig. 49

The total convergence recorded by station C9 is shown in Fig.49. This convergence station, situated in the middle of the roadway between the solid ribside and the instrumented pillar A, was seemingly unaffected by mining in the near vicinity, the convergence increasing at a fairly constant rate until day 96 when work resumed on the instrumented pillars adjacent to the station. At this time the convergence increased notably but settled to a constant rate again when the pillars were completely formed. Over a period of 275 days a vertical convergence of 4.66 mm. was recorded at C9.

It should be remembered however, that C9 was installed fairly soon after the roadway was formed and hence, whilst there was an unavoidable delay between the exposure of the site and the first measurement, it was by no means as long as the delay between the exposure of the site for stations C1, C2, C3 and C8 and the first measurement of approximately 4 years. It is conceivable therefore, that the total convergence at these stations in the older section of the workings is considerably higher than that recorded.

Since convergence is the algebraic sum of roof and floor movement, it was essential that any change in the level of either the floor or roof stations should be recorded. For this reason, a precise levelling survey was carried out at various intervals of time to determine any change in level of the floor stations with reference to the underground bench mark. It was found that the small differences in level recorded were within the limits of error expected with the instrument used, and this suggested that there has been no change in the level of any of the floor stations. It would seem therefore, that the deformation recorded at these convergence stations is due mostly, if not entirely, to the downward movement of the immediate roof of the workings. However, the comparison of the extensometer results with the levelling results is not strictly practical in view of the type of level used. The results from the underground levelling never reached the same degree of accuracy as the convergence measurements but since this was the most accurate level available at the time, it was felt that it would at least give some indication as to the behaviour of the floor stations.

These vertical convergence measurements can be compared with the theoretical values for the deflection of the immediate roof beam described in Section 3. In each case, the convergence recorded was much greater than that suggested by the theoretical appreciation for various load conditions used. The load condition referred to as Case 3, which was felt to be the most likely load acting on the beam where the load was assumed to be due to the weight of the beam plus the hydrostatic head of water, gave a maximum deflection value of the roof beam at the centre of the roadway of 0.51 mm (0.020 in.). This was approximately equal to 17% of the lowest convergence recorded in the mine at C2 of 2.89 mm (0.114). It would seem therefore, that either some of the various assumptions made in the theoretical appreciation were not applicable, or the roof configuration used in the calculation of that for a single layer roof beam was incorrect. With respect to this latter consideration, it is possible that the immediate roof is in fact a double layer or even a multi-layer beam where the layers are free to separate. This however, can only be determined by comprehensive 'in situ' instrumentation of the actual roof beam.

5.3 Pillar Lateral Deformation Measurements.

The usual method of plotting the results of lateral pillar deformation is to use the mouth anchor as a 'fixed' reference point. The periodic deformation that takes place between an anchor and the reference horizon has been plotted as anchor movement relative to the 'fixed' position of the mouth anchor. In addition to this, a computer program was used to calculate the results from the measured data. This program divided each borehole into bays and calculated the cumulative movement, strain, and strain rate for each bay length, the length of each bay being determined by the relative positions of the anchors in the borehole. Thus, for example, pillar A has been divided into 3 sections by the anchors in borehole 2, bay 2 - a, bay-a - b,

LATERAL BAY DEFORMATION AND STRAINS.

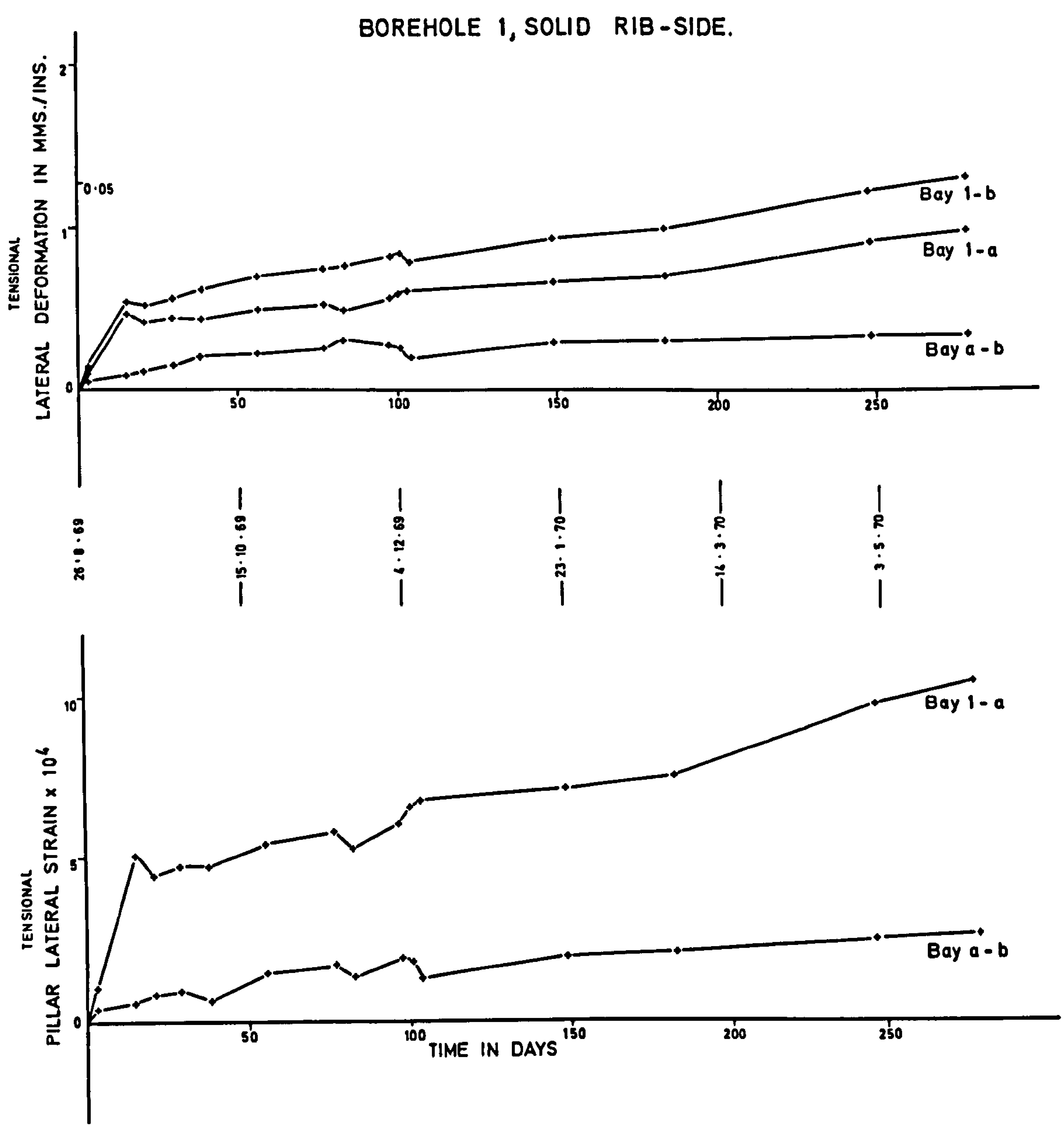


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FIG. 50

and bay b - c. These deformations are presented both in mms. and ins., and the convention of increased distance (or extension) being positive and reduced distance (or contraction) being negative, used throughout.

The induced apparent strains are presented as deformation per unit length, and though strain rates were calculated for each bay length of the various boreholes, they are not presented here since it is felt that they do not provide information which cannot be gained from the deformation and strain figures. The strain has been referred to as 'apparent' since some part of the measured deformation from which the strain is calculated is probably due to the opening or closing of joints or fractures within the seam, and not due to the deformation of a continuous mass alone. Both the deformation and strain diagrams should be studied simultaneously since in some cases the deformations being compared have been measured over different bay lengths.

The results obtained from borehole 5 were found to be unreliable and have not been used.

Borehole 1 - Solid Ribside.

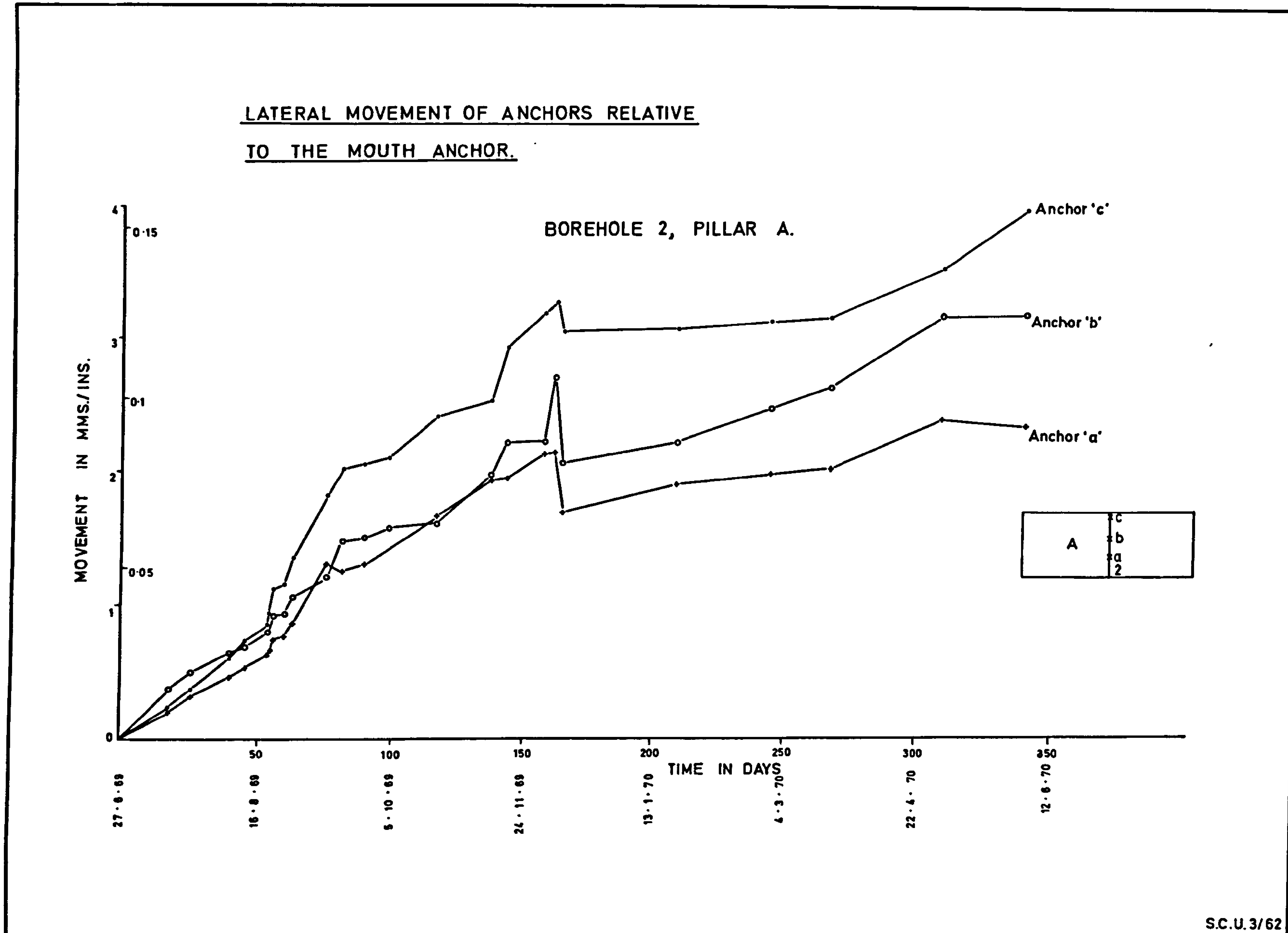
Since this was a short borehole containing only two anchors, the movement of each anchor relative to the mouth anchor has been plotted on the same diagram as the cumulative bay movement, Fig.50. The deformations occurring in this borehole are seen to be extensional. Both anchors have behaved in almost exactly the same manner, anchor 'b' having the slightly higher total movement relative to the mouth anchor after 280 days. It can be seen that this borehole was not affected to any great extent by the development of the workings. There was a fairly high initial deformation rate following its installation, but this soon became constant. The small sudden change in the region of day 96, 1st December 1969, corresponds to the formation of the adjacent instrumented pillars B, C and D. Following this, the lateral deformation continued to increase, though at a very low rate, and has continued to do so up to the present time.

The type of deformation indicated in Fig.50 is typical of instrumented boreholes that have been installed in a solid ribside, where this extensional or tensional movement is confined to an 'edge-zone'. This is also shown in the lower part of Fig.50, which depicts the lateral strains for the bay 1 - a and bay a - b. This indicates that the strain is much higher in the bay 1 - a than bay a - b. If a longer borehole could have been drilled and instrumented, the distance that this 'edge' zone extends into the ribside could have been determined.

Borehole 2 - Pillar A.

The total lateral movement of each anchor relative to the mouth anchor is shown in Fig.51, and the total lateral deformation and strains which have occurred in the respective measuring bays in Pillar A are shown in Fig.52. The overall movement of each anchor relative to the mouth anchor since installation, was extensional having values of 2.36 mm., 3.22 mm. and 4 mm. for anchors 'a', 'b'. and 'c' respectively after 339 days. It can be seen from Fig.51, that though no extraction took place around the pillar until day 53 following instrumentation there was some lateral deformation taking place which Fig.52 shows to be confined to the 'edge' zone of the pillar, bay 2 - a.

When work recommenced in headings 1 and 2 South on day 53 however, there was an immediate change in the deformation of all 3 bays. This continued up to about day 80, by which time pillar A was completely formed and the working faces had advanced to some distance away. Following this, bay 2 - a continued to deform at a fairly constant rate up to day 157, 1st December 1969, when the adjoining pillars were formed, at which time there was an immediate change in the lateral deformation of each bay, due probably to the re-distribution of the load as a result of forming pillars B, C and D. These pillars were completely formed by day 165. The tensional deformation of bay 2 - a continued to increase though at a slowly reducing rate up to



LATERAL MOVEMENT OF ANCHORS RELATIVE
TO THE MOUTH ANCHOR.
BOREHOLE 2, PILLAR A.
Anchor 'c'
Anchor 'b'
Anchor 'a'
MOVEMENT IN MMS./INS.
0
1
2
3
4
0·05
0·1
0·15
50
100
150
200
250
300
350
TIME IN DAYS
27·6·69
16·8·69
5·10·69
24·11·69
13·1·70
4·3·70
22·4·70
12·6·70
A
c
b
a
2
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FIG. 51

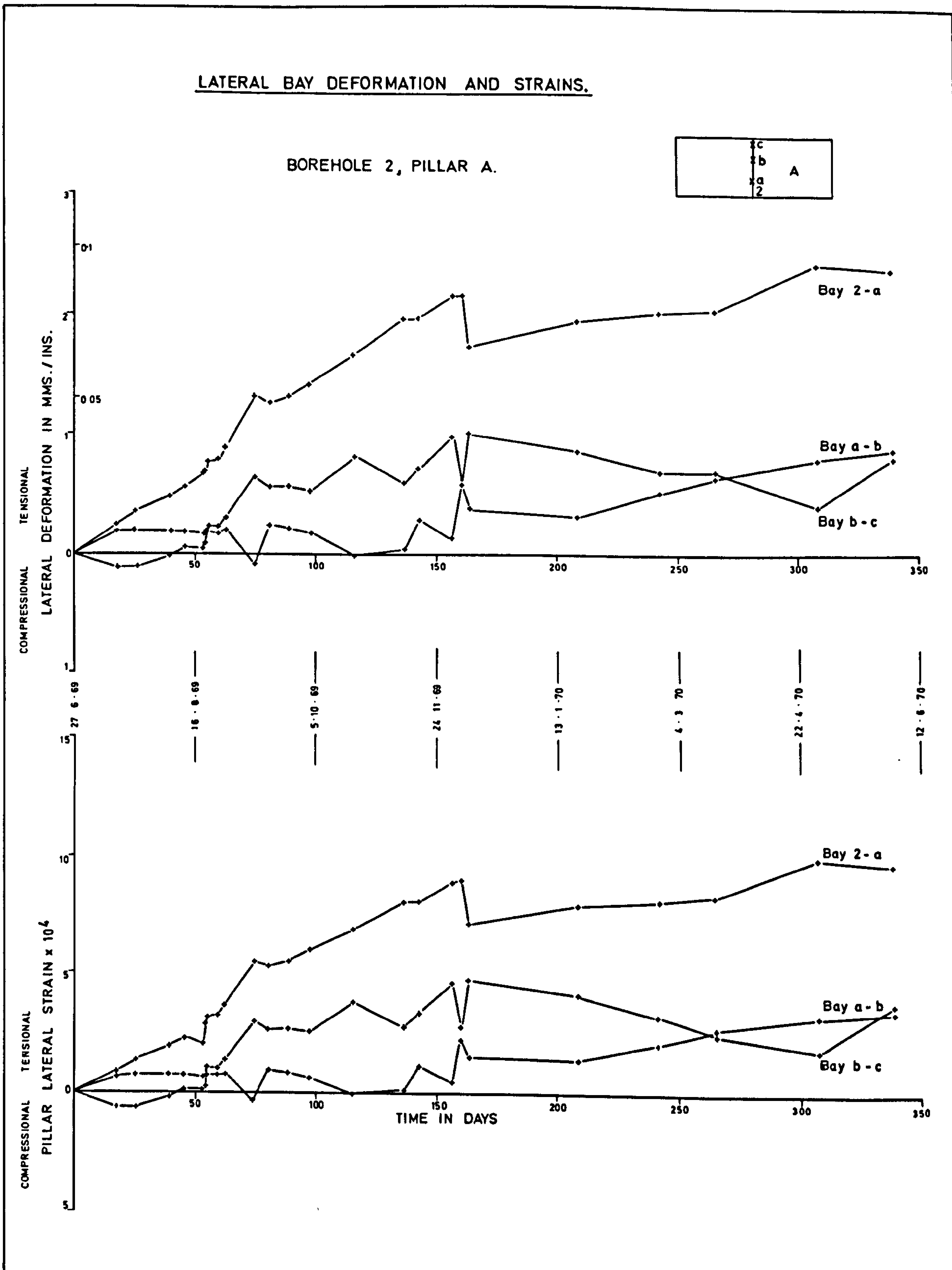

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FIG. 52

day 339, 1st June 1970, when the final measurement was taken. Over the same period of time, bays a - b and b - c deformed very little, their total tensional deformation by day 339 being almost exactly the same.

Borehole 3 - Pillar B.

This borehole was instrumented at the same time as borehole 2, pillar A, though the state of excavation was not as advanced as for pillar A. Consequently, more informative observations were expected from the measurements. The overall movement of each anchor relative to the mouth anchor, Fig.53, was higher in each case than the corresponding anchors in borehole 2, giving extensional values of 3.43 mm., 3.85 mm. and 4.40 mm. for anchors a, b and c respectively. The total lateral deformation and strains which have occurred in the respective measuring bays are shown in Fig.54.

As in the case of borehole 2, there was some lateral deformation occurring in bay 3 - a up to day 53, whilst bays a - b and b - c remained fairly constant. Between days 53 and 80, 19th August 1969 to 15th September 1969, there were quite substantial deformation changes recorded. The lateral deformation of bay 3 - a increased rapidly before tailing off when the workings advanced to some distance away. Bay a - b became increasingly compressional up to day 90 before slowly becoming tensional again and afterwards deformed similarly to bay b - c up to the time work resumed on the pillars on day 157. Bay b - c deformed very little from the date of installation up to day 157, 1st December 1969. This was probably due to the fact that pillar B, being part of a very large pillar, was acting as if it were a solid ribside.

When work resumed on forming the pillars B, C and D, the deformation of bay 3 - a increased considerably and continued to do so up to the time of the final measurement, though there are indications that this movement is tailing off. Bays a - b and b - c behaved similarly initially becoming compressional, but at the time of the

LATERAL MOVEMENT OF ANCHORS RELATIVE TO THE MOUTH ANCHOR

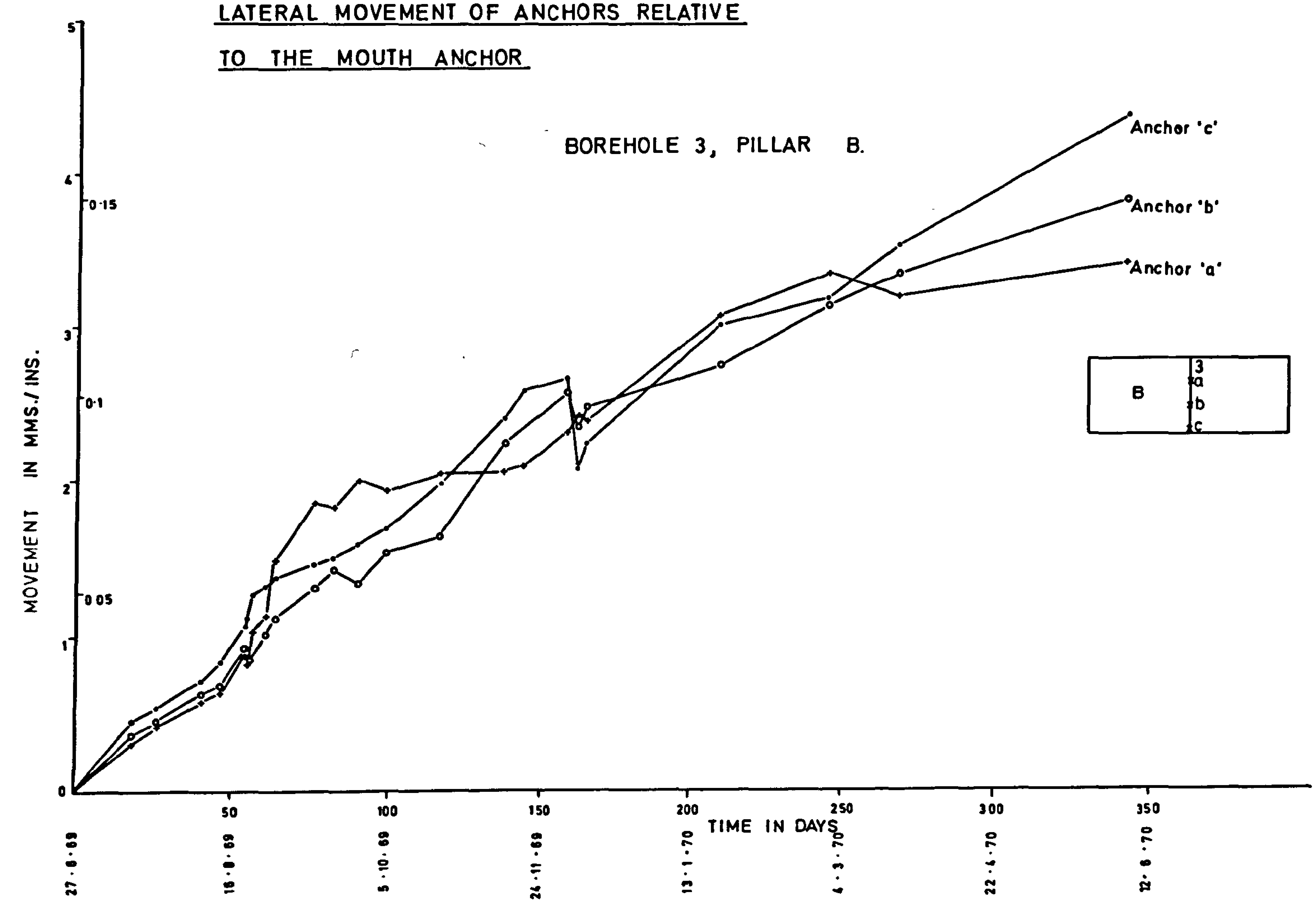

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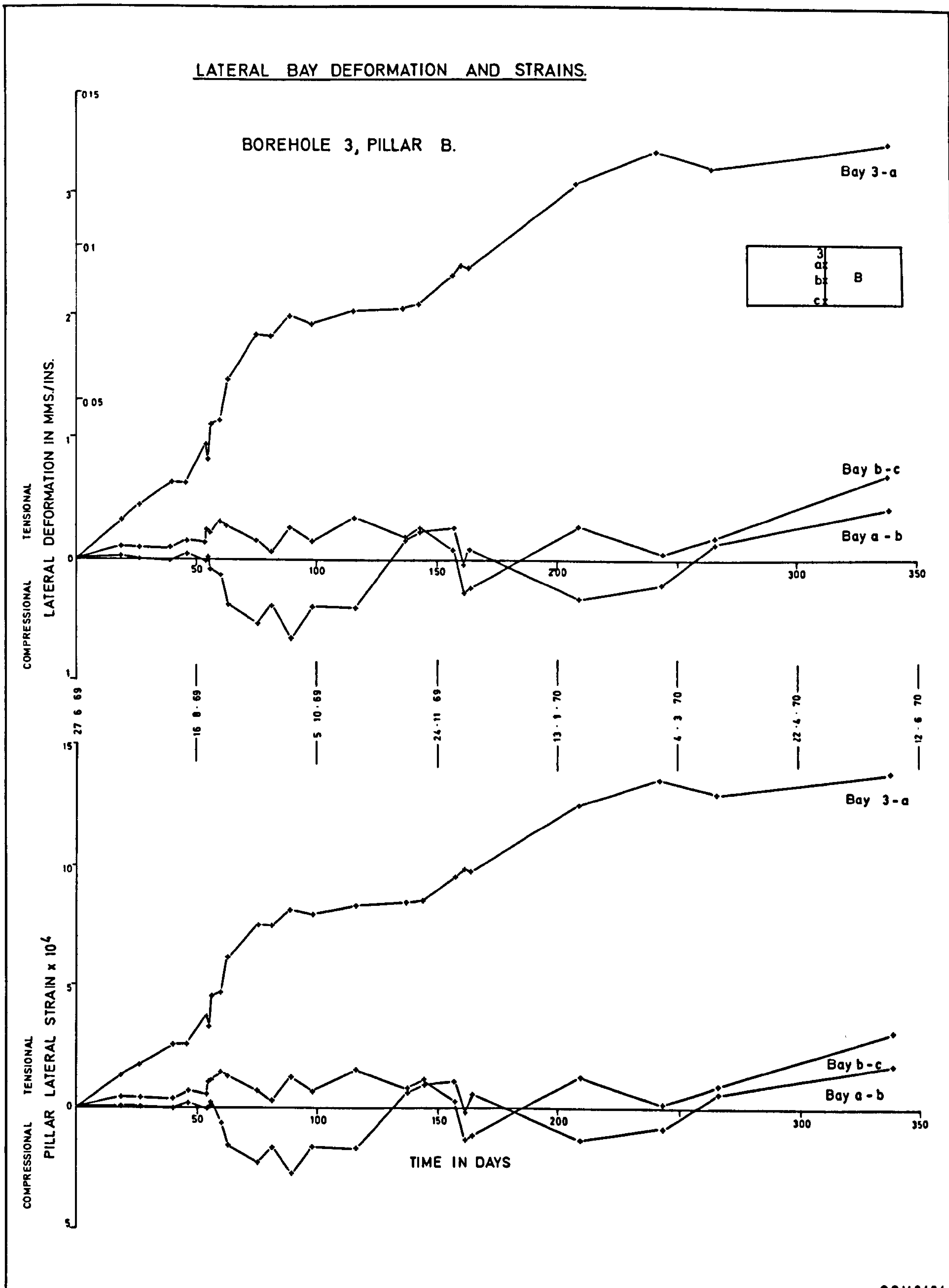
FIG. 53



S.C.U.3/64

FIG. 54

final measurement they were both tensional and becoming increasingly so.

The corresponding strain values for bay 3 - a shown in the lower part of Fig.54, are much higher than those obtained for bay 2 - a, pillar A, but the present strain values for bays a - b and b - c in both borehole 2 and borehole 3 are very similar.

Borehole 4 - Pillar C.

This borehole was instrumented on 8th December 1969, by which time pillar C was virtually formed. Therefore the lateral movement of the anchors shown in Fig.55 were not really related to either the formation of pillar C or extraction in the immediate vicinity. It can be seen that the total tensional movement of each anchor relative to the mouth anchor over a period of 175 days following the formation of pillar C, is 0.8 mm., 1.71 mm. and 2.65 mm. for anchors a. b and c respectively. The deform ation of all three bays is tensional and approximately equal, the deformation recorded in bay 4 - a approximately corresponding to that recorded over the same interval of time in bay 3 - a, pillar B, the bay lengths being equal. The deformations are confirmed by the corresponding strain values shown in Fig.56.

Borehole 6 - Pillar B.

This short borehole was drilled and instrumented on 10th September 1969. The total lateral movement of each anchor relative to the mouth anchor that has occurred since this date is shown in Fig.57, together with the deformation that has occurred in the bay a - b. As in the case of borehole 1, both anchors behaved in almost exactly the same manner. The initial movement of each anchor relative to the mouth anchor was contractional but became tensional when pillar B was completely isolated around day 85, 5th December 1969. The corresponding strains shown in Fig.57 for bays b - a and a - b indicate that though bay a - b remained tensional throughout, bay b - a changed from a quite high compressional strain to a corresponding tensional strain, the transition occurring when pillar B was completely formed.

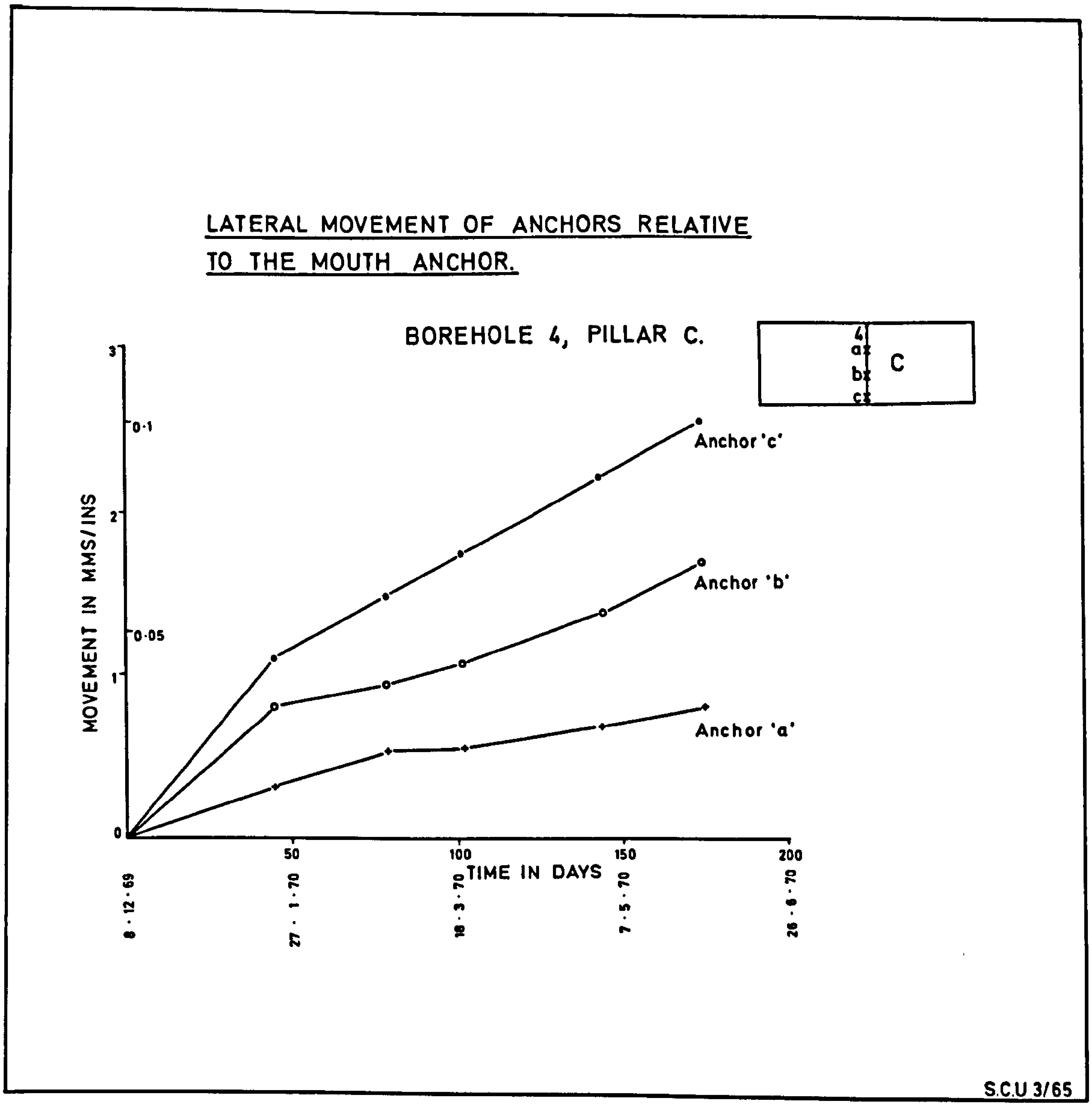


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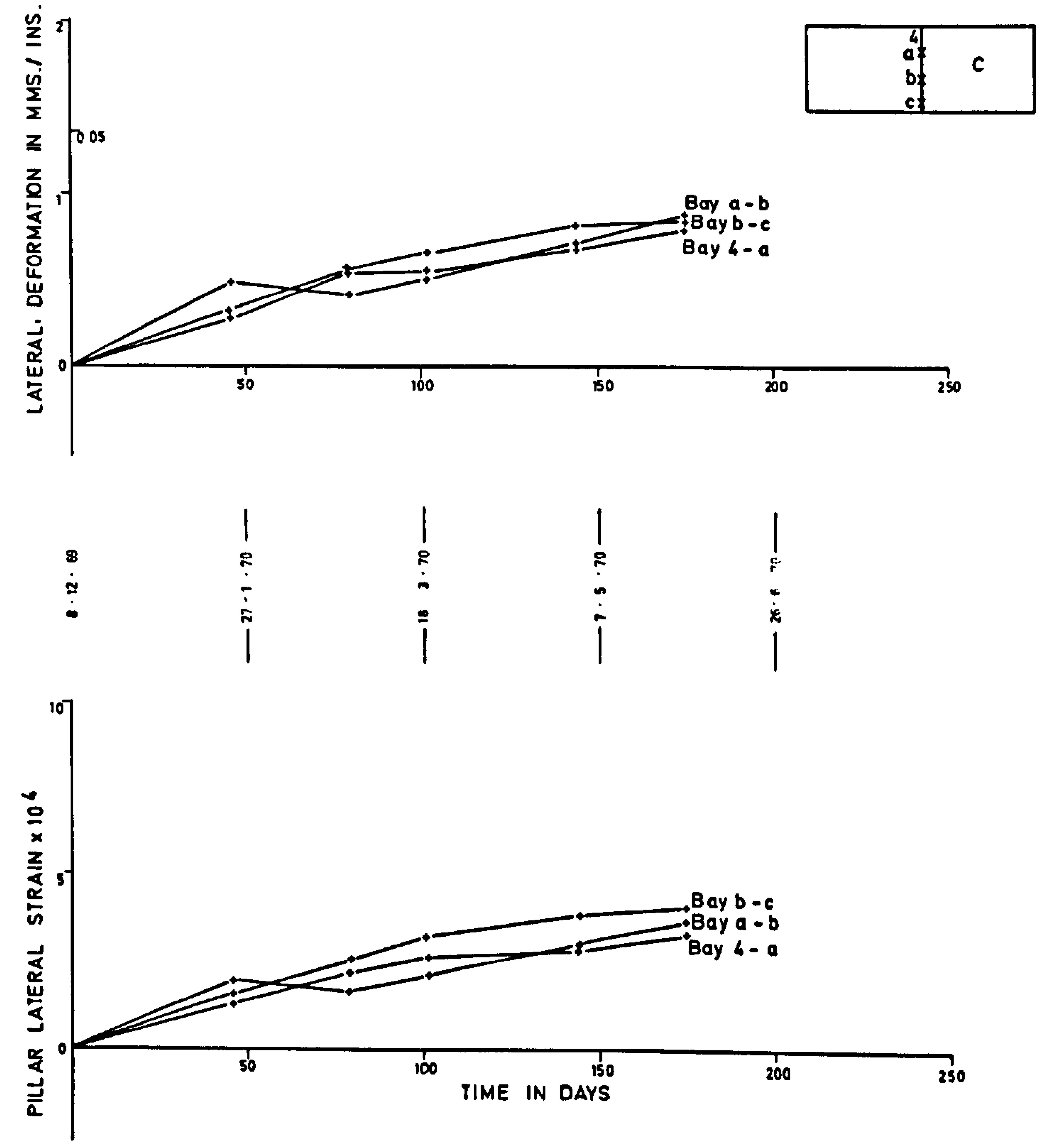
FIG. 55

LATERAL BAY DEFORMATION AND STRAINS.

BOREHOLE 4, PILLAR C.



S C.U.3/ 66

FIG. 56

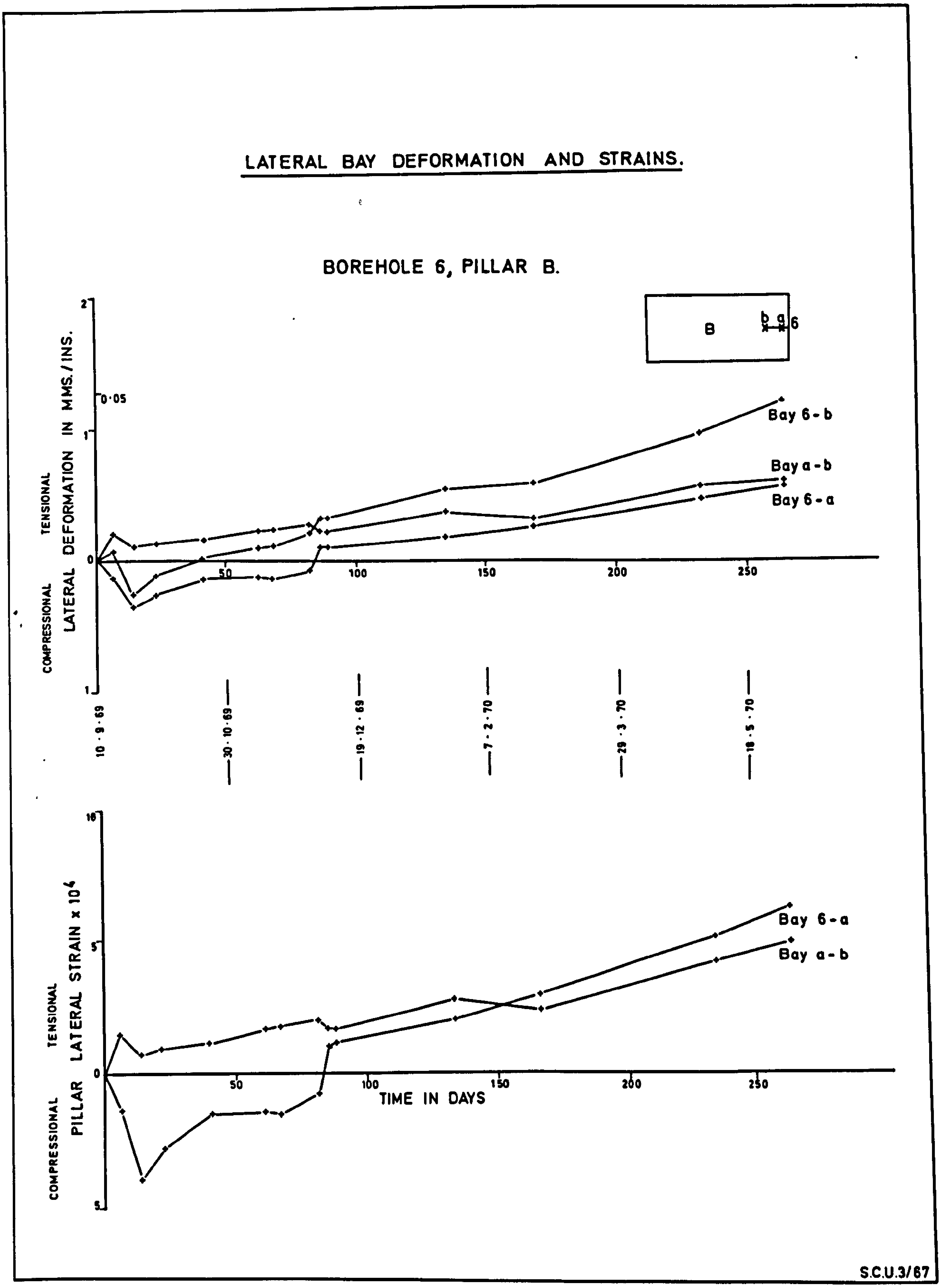

LATERAL BAY DEFORMATION AND STRAINS.
BOREHOLE 6, PILLAR B.
B
b a 6
0·05
TENSIONAL
COMPRESSIONAL
LATERAL DEFORMATION IN MMS./INS.
Bay 6 - b
Bay a - b
Bay 6 - a
10 · 9 · 69
30 · 10 · 69
19 · 12 · 69
7 · 2 · 70
29 · 3 · 70
18 · 5 · 70
PILLAR LATERAL STRAIN x 10⁴
Bay 6 - a
Bay a - b
TIME IN DAYS
S.C.U.3/67

FIG. 57

At the time the final measurement was taken the total strain value for each bay was tensional and approximately equal.

Borehole 7 - Pillar B.

This borehole was instrumented on 29th August 1969, the total movement of each anchor relative to the mouth anchor is shown in Fig.58, whilst the lateral deformation of each bay length and the corresponding strain are shown in Fig.59. Though very little extraction took place in the immediate vicinity of pillar B until day 94, 1st December 1969, there was some noticeable movement of each anchor relative to the mouth anchor, which Fig.59 shows to be confined to bay 7 - a. The lateral deformation of bay a - b has remained tensional and fairly constant throughout, seemingly unaffected by the extraction around pillar B. The same could probably be said for bay b - c, though in this case the change in deformation remained compressional throughout and fluctuated only very slightly over the first 100 days. The sudden increase in the lateral deformation of bay 7 - a at about day 94 corresponds to the forming of pillar B. Since this time the deformation has been confined to this bay, increasing at a fairly constant rate.

Borehole 8 - Pillar C.

This borehole was only instrumented on 1st December 1969, on the day work resumed on forming pillars B, C and D. In this case, the bay lengths of 21 ft. (6.4 m.) were much longer than in any other borehole. The lateral movement of each anchor relative to the mouth anchor is shown in Fig.60 and the bay deformations and strains shown in Fig.61. These show that there were considerable variations in the deformation of each bay length in the period of 8 days the pillar was being formed. In the following period of 100 days, the tensional deformation in bay 8 - a increased fairly rapidly before appearing to become constant. Over the same period of time the deformation of bay a - b remained constant and tensional, whilst bay b - c, initially compressional became tensional and attained a value approximately equal to bay a - b. In the following 100 days however, up

LATERAL MOVEMENT OF ANCHORS RELATIVE TO THE MOUTH ANCHOR.

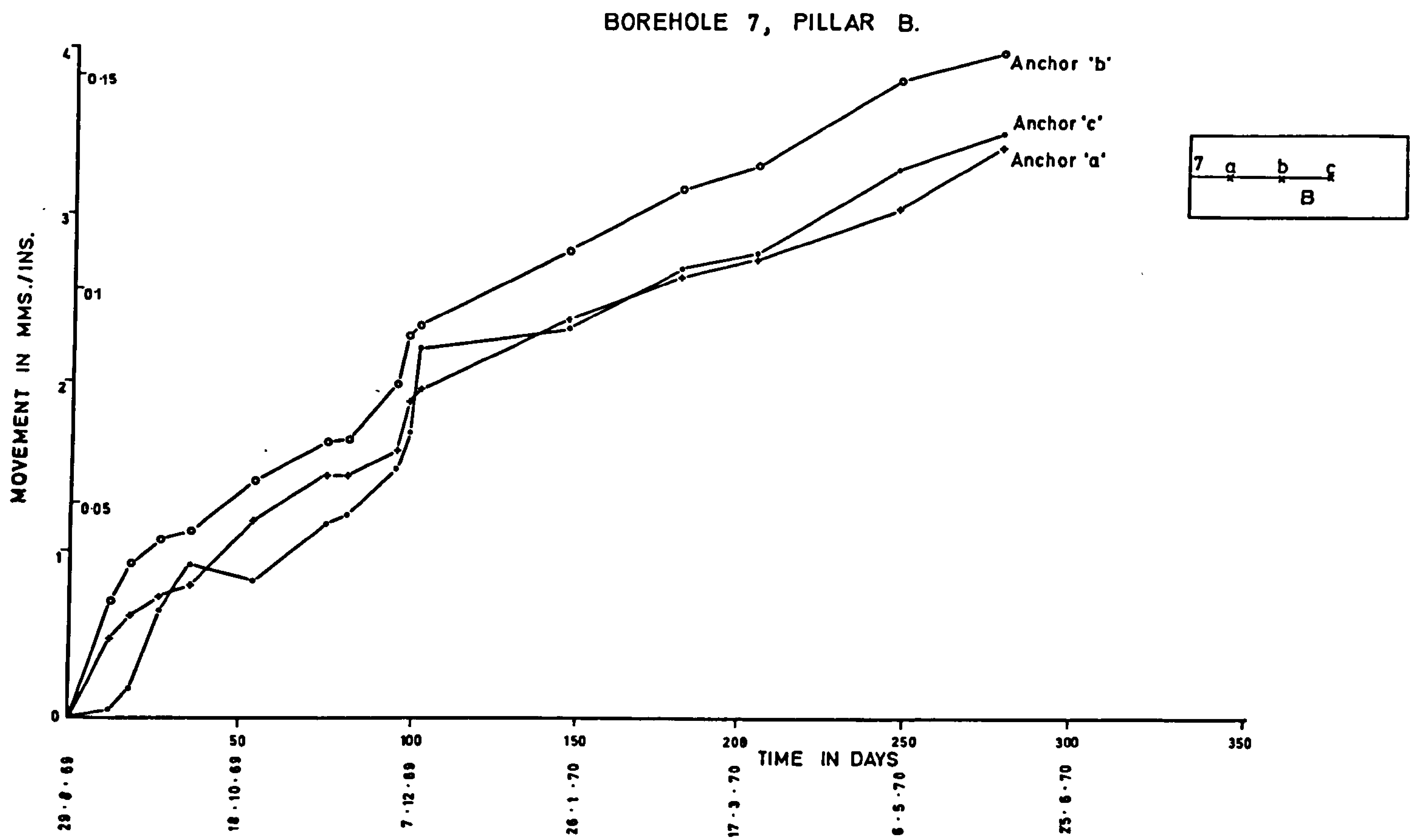


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FIG. 58

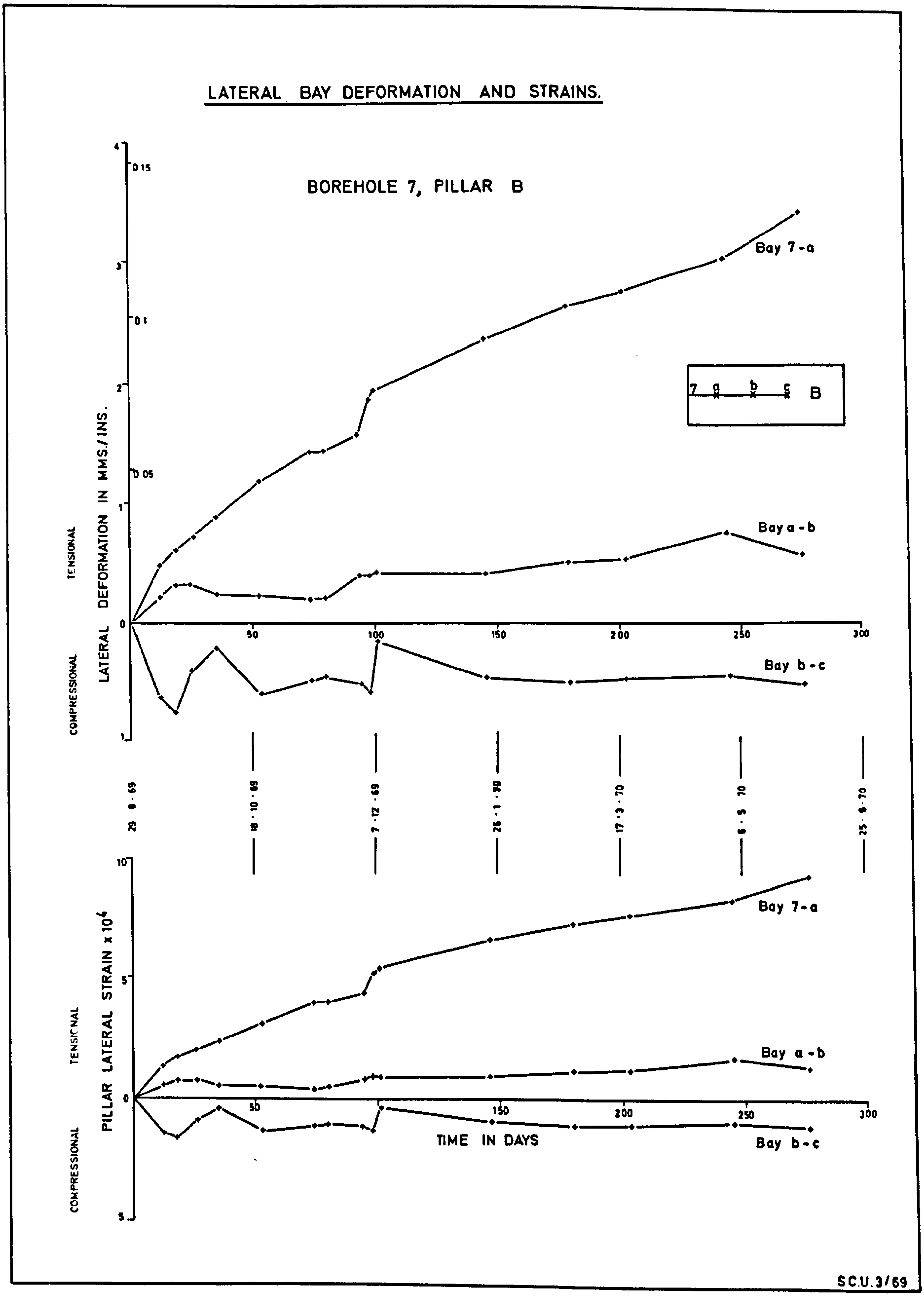
LATERAL BAY DEFORMATION AND STRAINS.
BOREHOLE 7, PILLAR B
7 a b c B
Bay 7-a
Bay a-b
Bay b-c
TENSIONAL
COMPRESSIONAL
LATERAL DEFORMATION IN MMS./INS.
PILLAR LATERAL STRAIN x 10⁴
TIME IN DAYS
29 8 69
18 · 10 · 69
7 · 12 · 69
26 · 1 · 70
17 · 3 · 70
6 · 5 70
25 · 6 · 70
0 50 100 150 200 250 300
0.15
0.1
0.05
SC.U.3/69

FIG. 59

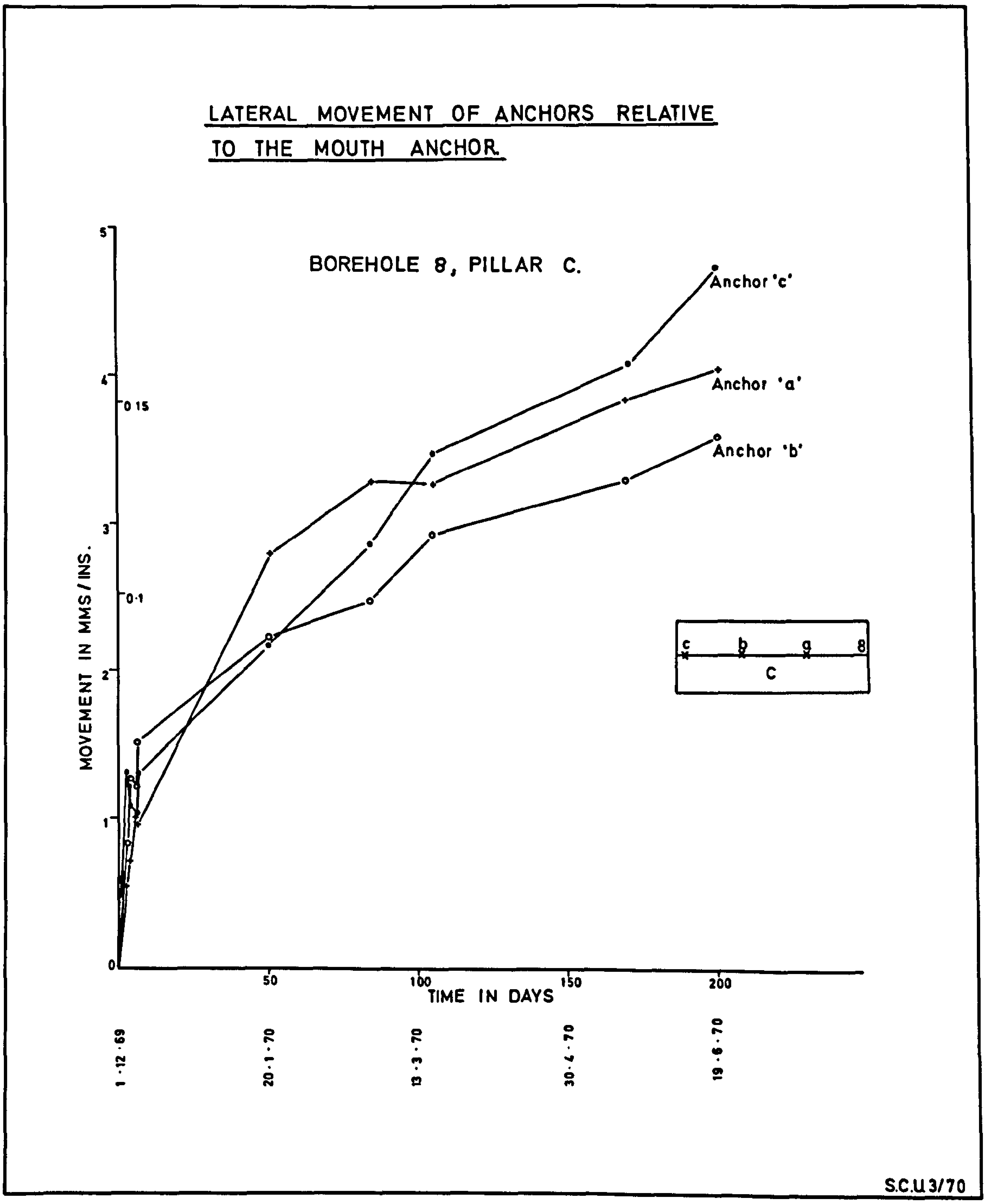

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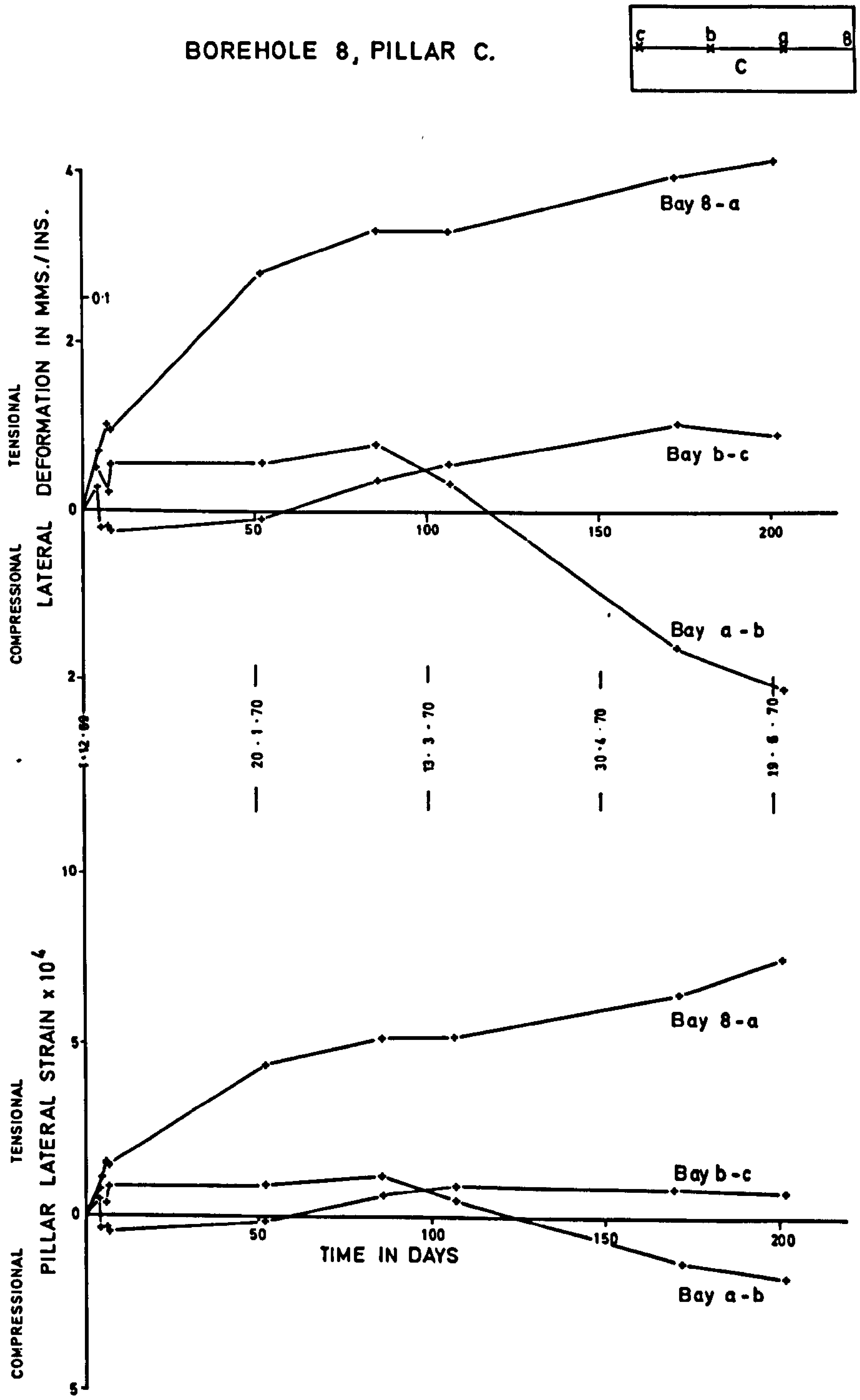
FIG. 60

LATERAL BAY DEFORMATION AND STRAINS.



S.C.U.3/71

FIG. 61

to the time of the final measurement, 20th March 1970 - 1st June 1970, The deformations occurring in bay 8 - a increased substantially, bay a - b has become increasingly compressional, and bay b - c has remained tensional and fairly constant, even though there was no working in the immediate vicinity. At the time of the final measurement, the anchors 'a', 'b' and 'c' had attained movements of 4.08 mm., 3.62 mm. and 4.77 mm. respectively, relative to the mouth anchor.

As mentioned previously, in most cases the pillars are divided laterally into 3 sections by the borehole anchors. However, the deformation or strains occurring in corresponding bays in the borehole cannot be compared directly since the boreholes were instrumented at different times. It would appear however, that most of the lateral deformation that has taken place in each pillar has been confined to the outer section of the pillar, in the bay length formed by the mouth anchor and anchor 'a'. In the case of boreholes 2, 3, 4 and 8 which passed through the whole width or length of the pillar, the centre bays a - b, deformed laterally by a relatively small amount when compared with this outer bay. The same could probably be said for the bays b - c even though this bay is situated in the same outer section of the pillar, but on the opposite side to the bay formed by the mouth anchor and anchor 'a'. This is probably due to the fact that in each case, anchor 'c' was installed at a distance of 2 ft. short of the proposed line of the pillar edge to prevent the anchor being dislodged by blasting when that side of the pillar was formed. In view of this therefore, it would appear that most of the extensional pillar lateral deformation that has been measured occurred in the extreme outer section of the pillar, probably in the outer 2 ft. section of the pillar. This is indicated to some extent by the strain values obtained in the short borehole 6, Fig.57, for the bays 6 - a and a - b. It can be seen that the strain is higher for the outer bay 6 - a than the adjoining bay a - b, the total length of the other boreholes.

It would appear from the measurements obtained from these boreholes in the period of time following the formation of the pillars, that in almost every case the deformation of the outer bay has continued to increase even though there has been no mining within 500-600 ft. (152 - 182 m.) of these pillars. In view of the continuing roadway vertical convergence recorded at the various measuring points throughout the mine, it is probable that the pillar lateral deformation that is continuing at the present time can be related to roadway convergence. This is indicated to some extent by the diagram, Fig.29, showing the calculated roof beam deflection. The deflection or sag of the beam above the pillar edge subjects the pillar to higher stresses at its extremities. The extreme outer edge of the pillar is able to move outwards quite freely, where as with increasing lateral pillar depth, the pillar material is constrained and consequently able to deform very little.

In view of the time difference involved in the instrumentation of the various boreholes, it is difficult to compare the strain or deformation per unit length values obtained between the different boreholes. In the case of pillar B, where boreholes 3 and 7 are situated perpendicular to each other, the total deformation per unit length in the outer bay section of each borehole over the same period of time is almost identical. The same can be said for the corresponding sections of boreholes 4 and 8 situated similarly in pillar C. The deformation per unit length of the outer section of borehole 2, pillar A, over the same period of time as boreholes 3 and 7 is much lower, and lower in fact than the deformation per unit length of the outer sections of boreholes 4 and 8 in pillar C that was recorded in the period of time following their installation. Borehole 2, is the only borehole in which all 3 measuring bays have achieved some measure of stability. It would appear therefore that the two central pillars, B and D, of the panel are accepting a much higher re-distributed load due to mining than the pillars closest to the solid ribside. On the other hand, pillar A was the first pillar formed and in this respect it has achieved a measure of stability some 280 days following its complete isolation.

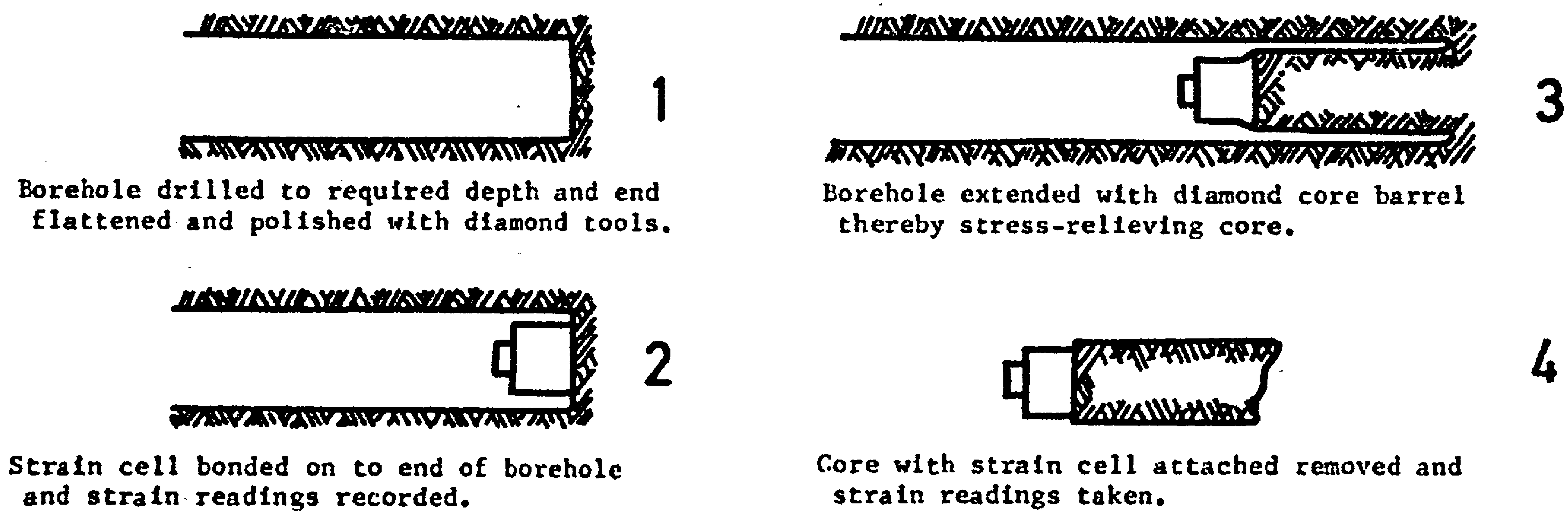
It may be noticed from the graphical results of pillar deformation that in the periods of time the actual instrumented pillars were being formed, there were quite considerable changes in the lateral deformation, sometimes extensional and sometimes contractional, over quite short periods of time. It is probable that this deformation was related to the occurrence of the jointing in the seam mentioned previously. This jointing resulted in the formation of pillars possibly consisting of a series of blocks of gypsum that moved relative to each other as the load was re-distributed. This was most pronounced in borehole 8 when pillar C was being formed, and in borehole 2 at day 161, when the adjoining pillars were being formed.

6. 'IN SITU' EXPERIMENTS CARRIED OUT TO DETERMINE THE ABSOLUTE PILLAR STRESS.

It was considered that in order to obtain a clear picture of the strata movement, the borehole extensometer results should be supplemented with some knowledge of the rock stress. 'In situ' experiments were therefore carried out to determine the absolute stress in certain pillars in the Sherburn mine.

The technique used to determine the absolute pillar stress at Sherburn has been described by Singhal in a recent thesis (45). Based on the stress relief principle, it employs the 'Doorstopper' Strain Cell which incorporates a rectangular strain gauge rosette developed by the Council of the Scientific and Industrial Research of South Africa (59). Fig.62 illustrates the four basic steps involved in determining stresses by this method.

Firstly, a borehole is drilled to the required depth and the end flattened and polished with diamond tools. Secondly, a 'doorstopper' is bonded on to the end of the borehole and strain readings recorded. In the third step the borehole is extended by means of a diamond core barrel, thereby stress relieving the core. Finally, the core with the strain cell



Four Stages in Measuring Stresses in Rock by Stress Relief

S.C.J.4/66

Fig. 62

attached is removed and strain readings taken. The difference between the strain readings before and after freeing the rock core is a measure of the stress acting at the flat end of the borehole. A brief synopsis of the theory involved in the calculation of the absolute stress values from the strain measurements is given here.

For a rectangular strain gauge rosette, let e_v, e_{45} and e_H represent the difference in the strain readings in the vertical, 45^o and horizontal directions before and after overcoring.

The principal strains (e_1 and e_2) in the rock on the end of the borehole are given by :-

$$e_{1,2} = \tfrac{1}{2}(e_H + e_v) \pm \sqrt{2\,e_{45} - (e_H + e_v)^2 + (e_H - e_v)^2}$$

The directions of e_1 and e_2 are θ_1 and θ_2 respectively, measured anti-clockwise from the horizontal (e_H) direction and are given by :-

$$\text{Tan } \theta_1 = \frac{2\,(e_1 - e_H)}{2\,e_{45} - (e_H + e_v)}$$

$$\text{Tan } \theta_2 = \frac{2\,(e_2 - e_H)}{2\,e_{45} - (e_H + e_v)}$$

The principal stresses $\sigma_1{}^1$ and $\sigma_2{}^1$ present on the bottom surface of the flat-ended borehole are :-

$$\sigma_1{}^1 = \frac{E}{1 - \gamma^2} \{(e_1 + \gamma e_2)\}$$

$$\sigma_2{}^1 = \frac{E}{1 - \gamma^2} \{(e_2 + \gamma e_1)\}$$

where E is the secant modulus of the rock on the unloading cycle.

γ is Poisson's ratio of the rock.

The stresses in the rock surrounding the borehole are given by :-

$$\sigma_1 = \frac{\sigma_1^{\,1}}{K} = \frac{1}{K}\left\{\frac{E}{1-\gamma^2}\,(e_1 + \gamma e_2)\right\}$$

$$\sigma_2 = \frac{\sigma_2^{\,1}}{K} = \frac{1}{K}\left\{\frac{E}{1-\gamma^2}\,(e_2 + \gamma e_1)\right\}$$

where K is the transverse stress concentration factor arising due to the presence of the borehole.

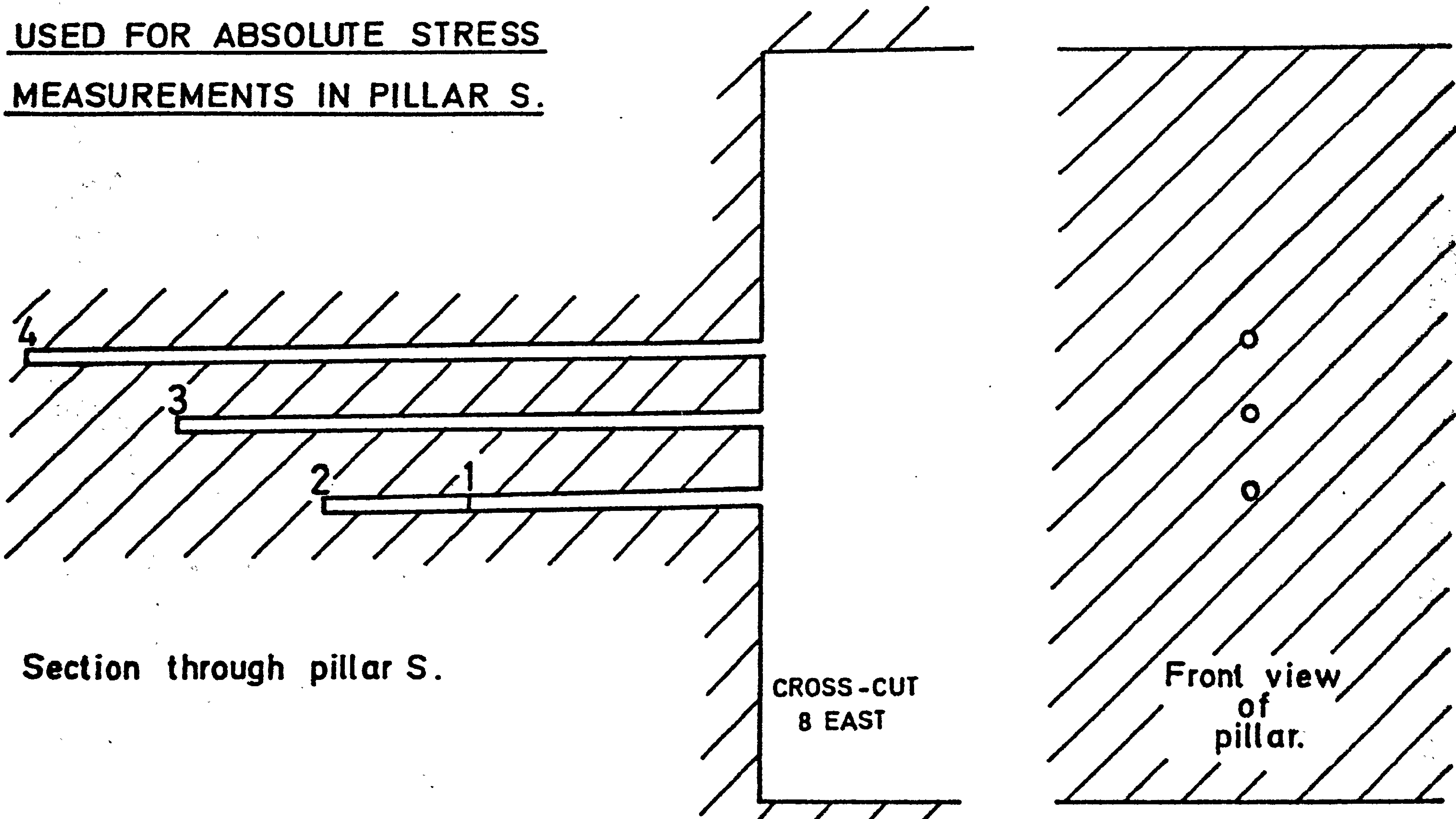
6.1 Description of the 'In Situ' Experiments Carried Out with the Doorstopper Strain Cell.

The position of the pillar in which experiments were carried out is shown on the plan, Fig.37. This pillar was chosen since it is situated at the centre of the greatest panel width in the mine at present, and is also the pillar size - 25 ft.x 25 ft. - that it is expected will be used in the main panels. The position of the holes drilled for stress measurement are shown in Fig.63. Numbers 1, 2 etc, represent the serial number of the test carried out.

First a borehole 4 ft. deep was drilled using the drilling machine as shown in Fig.64. After polishing and cleaning its end, a 'doorstopper' was bonded to the flat end using plastic steel and then left overnight to set. Next morning initial readings were taken and the 'doorstopper' was successfully overcored. However, with the tools made to retrieve the cores, it was not possible to reclaim the core intact, the 'doorstopper' was dislodged from the core, rendering the experiment abortive.

The hole was then extended to 6 ft. in depth and another 'doorstopper' bonded to its flat end. All the drilling was done dry with some considerable drilling vibration at times. In this case, overcoring was successfully carried out and the core retrieved with the aid of the tools made for this purpose. However, the core towards

DIAGRAM SHOWING POSITION OF BOREHOLES USED FOR ABSOLUTE STRESS MEASUREMENTS IN PILLAR S.



SCALE : 1 inch = 2 feet.

S.C.U.3/33

FIG. 63

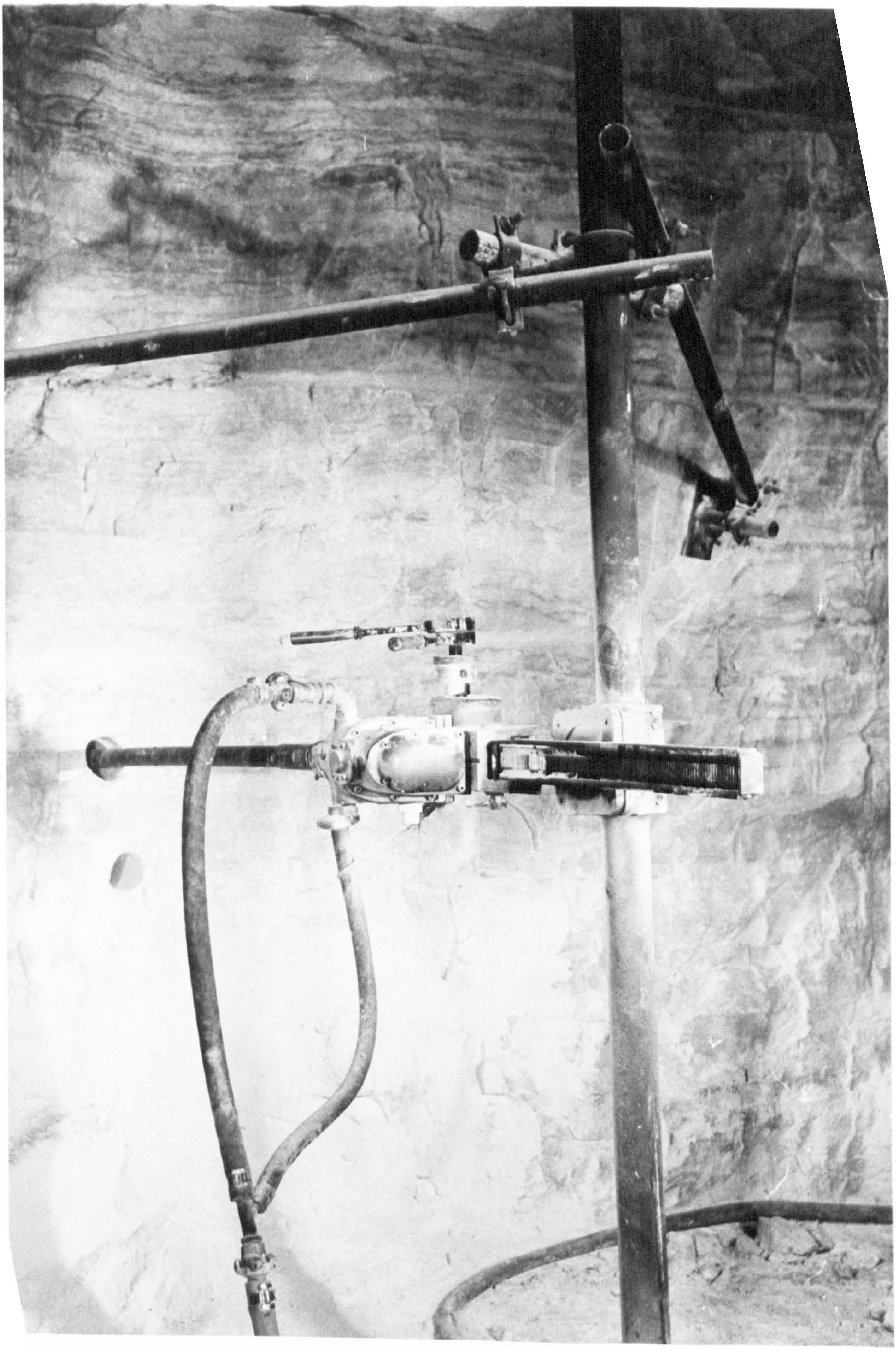
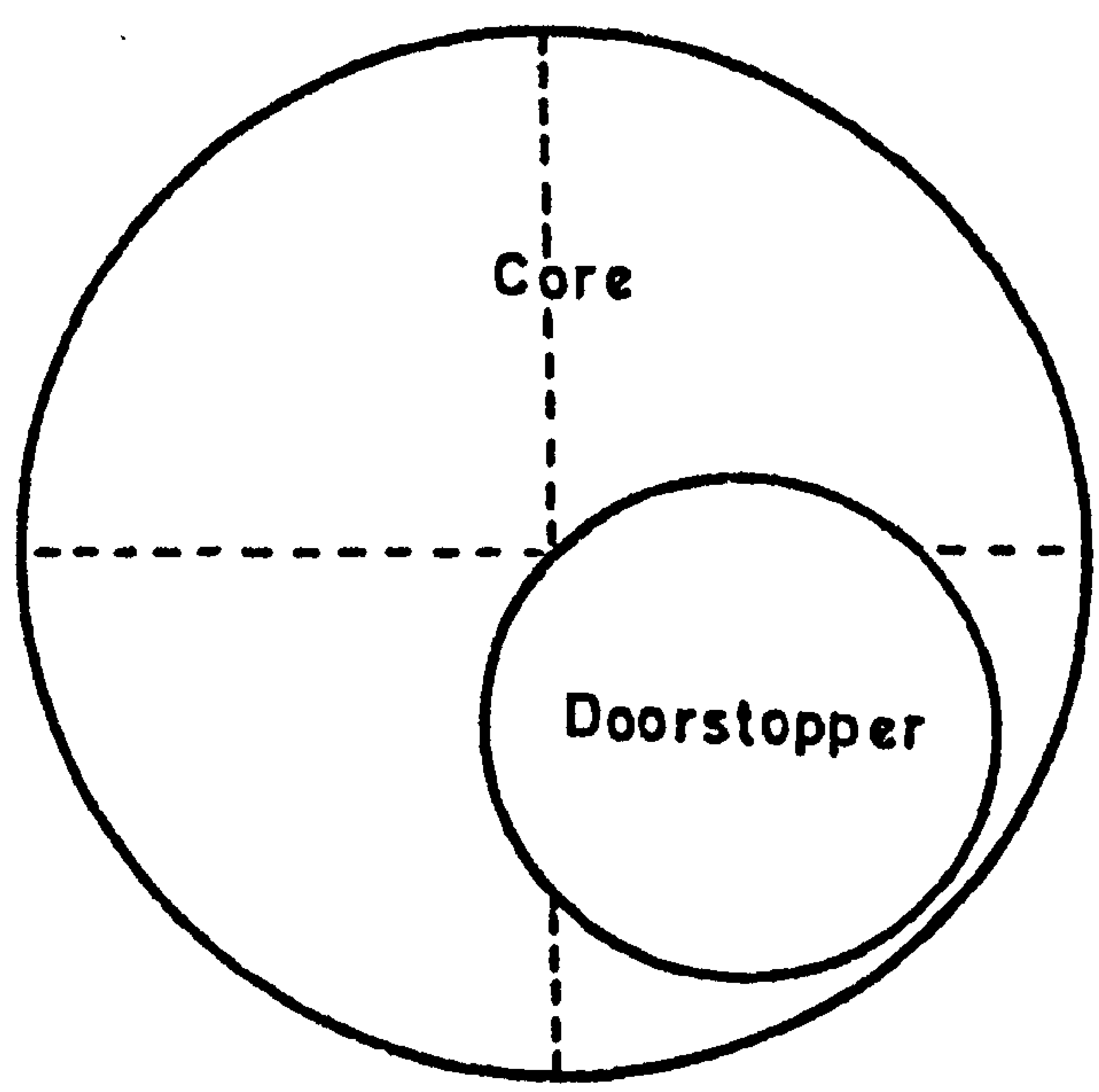


Fig. 64

POSITION OF THE DOORSTOPPER ON THE CORE RECOVERED IN EXPERIMENT No.2, BOREHOLE No.1



End-on view of core

S.C.U.3/34

FIG. 65

the opposite end to which the 'doorstopper' was bonded was somewhat tapered and the 'doorstopper' was found bonded to it in a corner, Fig.65, rather than at the centre of the flat end. The strains resulting due to stress-relief are given in Table 4.

Next, a hole 8 ft. deep was drilled. A 'doorstopper' was bonded to the flat end. However, when the installing tool was withdrawn from the borehole, the 'doorstopper' came with it. A large volume of fine dust was found sticking to the back of the plastic steel, clearly showing that the hole was not perfectly clean, and this dust had prevented the 'doorstopper' from being set to the back of the hole.

In each experiment the hole was cleared by blowing compressed air, wiping the flat end with acetone solution. Before attempting to bond the 'doorstopper', visual examination of the borehole was carried out by throwing the light from the cap lamp. Just before this particular experiment, the compressed air hose to the drill had burst and had been replaced by another of smaller length, the only one available at that time. It is possible that this shorter length of hose did not reach the bottom of the hole when clearing the hole, leaving some dust behind. In retrospect, light from the cap lamp is not considered an adequate means of judging the cleanliness of the flat end, particularly in gypsum due to the white texture when in the form of a powder. Care was therefore taken to ensure the air hose was of sufficient length.

Another attempt was made to bond a doorstopper to this hole. This attempt was successful; an 8 in. core being recovered as shown in Fig.66. The strains resulting due to stress-relief are given in Table 4.

The final test of the series was carried out in a hole 10 ft.deep. The 'doorstopper' was successfully bonded and overcored. However, with the core recovery tools in use at this depth, it was not possible to

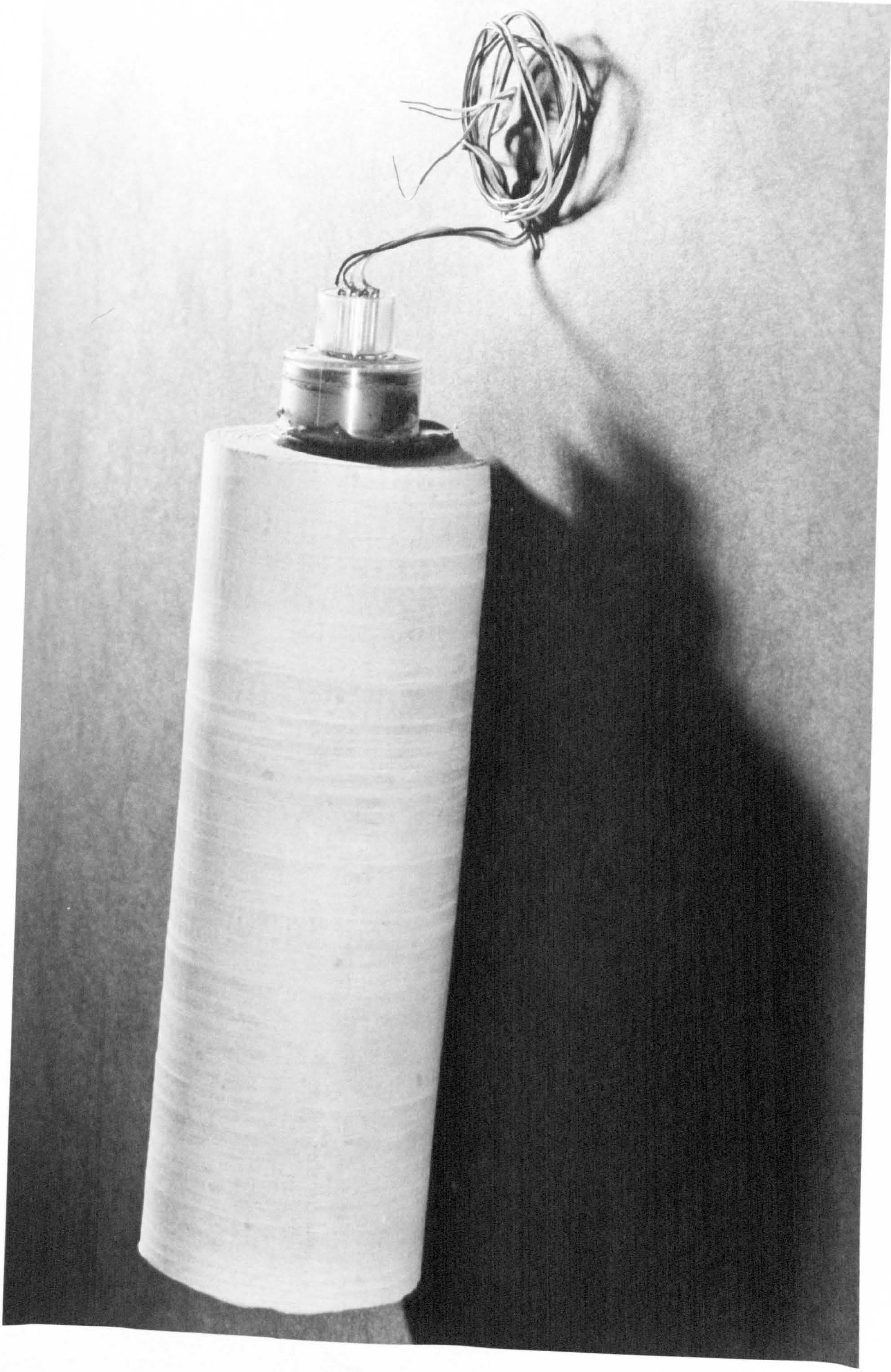


Fig. 66

reclaim the core intact. It was removed in two pieces and the 'doorstopper' was claimed separately detached from the core. The experiment was unsuccessful.

6.2 Calculations of the Results.

The tapered core recovered in Experiment No.2 could not be utilised for preparing a compressive test specimen of W/H = 0.5 to determine the secant moduli and Poisson's ratio of the gypsum. The only specimen which could be prepared for laboratory testing was from the core recovered in Experiment No.3. The core was fitted with strain gauges for measuring its lateral and longitudinal deformation under load, then placed in a testing machine and given three up and three down loading cycles. A stress/strain curve was plotted by taking the average of readings for the three cycles. From this stress/strain plot a graph showing the variation of the average secant modulus with stress was plotted. Stresses were then calculated by using the Newton's method of successive approximations, described by Singhal (45). For both successful experiments the same values of secant moduli and Poisson's ratio were used. Desired convergence was achieved at E = 1.60×10^6 lb/in^2, and Poisson's ratio = 0.4. Table 5 presents the results. These results have been calculated by assuming the transverse stress concentration factor as equal to 1.25.

6.3 Discussion of the Results.

The stress at the site of the experiments can be estimated by the equations :-

$$\frac{\sigma_H}{\sigma_v} = \frac{\gamma}{1 - \gamma} \qquad (12)$$

where σ_H = horizontal component of stress.

σ_v = vertical component of stress.

TABLE 4

Strain Recovery due to stress relief by overcoring in experiments with doorstoppers and modified installing tool at Sherburn Gypsum Mine

Experiment No.	Location of the Dcorstopper	Vertical Gauge (ev) Microstrain	Horizontal Gauge (eh) Microstrain	45^{o} Gauge (e_{45}) Microstrain
1	at 4 ft. in borehole 1	Experiment Abortive		
2	at 6 ft. in borehole 1	460	175	380
3	at 8 ft. in borehole 2	160	100	135
4	at 10 ft. in borehole 3	Experiment Abortive		

TABLE 5

Results from the Doorstopper Stress Relief Tests

Carried out at Sherburn Gypsum Mine

Experiment No.	Borehole No.	Θ_1 direction of σ_1 from the vertical	Major Principal stress at the back of the hole lb/in^2 σ_1^1	Major Principal stress in the rock S.C.F. = 1.25 lb/in^2 σ_1	Minor Principal stress at the back of the hole lb/in^2 σ_2^1	Minor Principal stress in the rock S.C.F. = 1.25
2	1	11° 50'	1001.860	800.00	654.883	532.20
3	2	4° 45'	372.093	297.674	305.116	244.00

and $\sigma_Y = \rho H \frac{1}{1 - R}$ (5) (Part 1,Section 3.1)

where ρ = average weight per unit volume of the overburden.

R = percentage extraction

H = depth of working.

By applying these equations the estimated stresses at the site of measurement are approximately 310 lb/in^2 and 210 lb/in^2 assumed vertical and horizontal respectively, and calculated by using a Poisson's ratio of 0.4.

In Experiment No.2, the major principal stress is thus 2.58 times the expected stress. Likewise, the minor stress is 2.49 times that expected. Because the 'doorstopper' was located in a corner of the circle forming the flat end of the borehole, it is possible that a stress concentration exists in that zone. No analytical solution for such a case is available but nearly equal factors (2.58 and 2.49) would seem to lend support to this view. The major principal stress is found to be acting at approximately 12° from the vertical.

In Experiment No.3, carried out at a depth of 8 ft. from the surface of the pillar, the major and minor principal stresses are calculated to be 298 lb/in^2 and 244 lb/in^2. These are in close agreement with the expected values. Major stress is found to be acting at approximately 4° 45' from the vertical. This result is considered to be most reasonable.

The magnitude of the transverse stress concentration factor assumed as 1.25 in the case of gypsum seems reasonable. Singhal (45) has shown that the magnitude of the s.c.f. depends upon the nature of the rock material. He found that in the case of rock salt it was 1.0, and in the case of sandstone it was 1.13. Due to the unavailability ov a large block of gypsum, an actual test to determine this factor has not been possible. However, since gypsum exhibits very little creep characteristics, and on the unloading cycle the stress-strain curve is completely linear, a transverse stress concentration factor of 1.25 is considered justifiable.

7. CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK.

The conclusions which may be drawn as a result of the investigations carried out to determine the state of stability of the development workings at the Sherburn-in-Elmet mine are presented in the first part of this section. The conclusions are divided into three parts :-

(i) The stability of the overlying strata.

(ii) The stability of the immediate roof of the workings.

(iii) The stability of the pillars.

The conclusions are followed by the second part of the section, containing recommendations concerning future research which could be carried out to further enhance the knowledge of the state of stability of the workings, and hence the establishment of more general design data on which to base future working dimensions. The scope for future investigations is wide and includes the direct application of some of the findings resulting from the investigations carried out by the author.

7.1 Conclusions.

7.1.1 The Stability of the Overlying Strata.

The conclusions that may be drawn from the instrumented surface borehole 1/51, with regard to the behaviour of the strata overlying the workings are limited. Had measurements been possible while the roadway immediately beneath the borehole was being formed, the effects would have been recorded and more precise conclusions drawn. It would have been possible to carry out roof-floor convergence measurements beneath this borehole using the borehole anchor as the roof station, since this anchor was accessible when the roadway was formed. Then by means of precise levelling, the corresponding floor station could have been linked to a stable datum, and it would then have been possible to relate all subsequent movement of the surface. However, since this was not possible, the conclusions drawn from the results must be considered with regard to the fact that the borehole was situated above the edge of the workings in the period of time that actual measurements were being taken.

In view of the various factors discussed in section 5.1, that could possibly have affected the actual measurements taken on the surface borehole, the possible surface movements observed at Sherburn must be considered to be extremely insignificant. If any movement does occur above the edge of the workings it must be in the order of 1/40 in., which for practical purposes is negligible.

Perhaps the most important conclusion that may be drawn from these measurements, concerns the need to establish the occurrence and amount of surface subsidence above the workings, by means of a number of surface levelling station that are being installed at the present time. The relationship between a change in the level of the water table and the amount of ground water, and a rise or fall of the surface was clearly shown, and indicated that this could affect surface levelling results to some extent.

7.1.2 The Stability of the Immediate Roof of the Workings.

The underground measurements indicated that the stability of the immediate roof strata is a factor which limits the working dimensions to a much greater extent than the factor of pillar stability. The possibility of widely varyingexternal loads on the roof beam was discussed in Section 3, and what was felt to be the actual load condition was established. This was the case of the roof beam supporting its own weight together with the full hydrostatic head of water. The amount of water entering the workings through openings and fissures in the roof beam in parts of the mine, indicated that this load condition was the most likely, even though there was no direct correlation between the calculated roof beam deflection for this load condition, and the vertical convergence recorded at the various underground measuring stations.

Calculations based on this load condition showed that the maximum longitudinal fibre stress induced in the beam at the centre of the roadway was unlikely to result in failure at this point, the tensile strength of the beam material being much higher than the induced stress. These calculations were carried out for a single layer roof beam, but in view of the considerable difference between the calculated beam deflection and the measured roof-floor convergence, it is possible that this beam configuration is incorrect, and that a more likely configuration is that of a double or even multi-layer beam, the different layers having free contact and thus able to separate. This is confirmed to some extent by an extensive visual examination of the roof beam throughout the development workings. It is on this basis therefore, that it is felt future investigations should be carried out.

The magnitude of the roof-floor convergence recorded at the various measuring stations was much higher than expected. At each station this convergence increased at a fairly constant rate and at the time of the final measurement there was little indication that this rate of convergence was decreasing. The most convergence was recorded by those stations situated at the centre of roadway intersections, the maximum of approximately 15 mm. (0.59 in.) being recorded at C5 situated in the central roadway of the five heading mining system. The measured convergence can normally be expected to be due to the combined effects of the vertical deformation of a recently formed pillar, and the deformation of the roof and floor. It is unlikely that the pillars in the older parts of the mine are still deforming vertically to any measurable extent, whilst periodic levelling of the floor stations has suggested, though not conclusively proved, that there was no change in the level of the floor stations. It has already been noted that the underground levelling procedure was not as accurate as the extensometer measurements, and for this reason, only a qualitative assessment of roof and floor movement should be made at this stage. However, it would seem likely that the convergence recorded is due almost entirely to the downward movement of the roof beam above the roadways. If this is so and this downward movement continues to increase at approximately the same rate, then failure of part or all of the immediate roof beam is inevitable at some future time.

This form of roadway convergence is unusual and is more normally associated with the mining of rock materials that exhibit time-dependent deformation or 'creep' characteristics. However, laboratory investigations indicated that Sherburn gypsum under constant compressive loading exhibited negligible time-dependent deformation. It would appear however, that the convergence recorded in the mine is very similar to creep measurements in that they are both functions of time, and in view of this, time-dependent deformation of the roof beam material should not be discounted at this stage of the investigation.

7.1.3 The Stability of the Pillars.

The 'in situ' measurements carried out to determine the state of stability of the pillars confirmed the previous conclusions of Jones (5), that not only are the pillars formed by the development workings stable, but also those which it is planned to form in the foreseeable future. The initial conclusions were based on the fact that the laboratory determined ultimate compressive strength of the gypsum forming the pillars appeared to be a great deal larger than the compressive stress that the pillars are required to support. The investigations carried out to determine the absolute pillar stress described in Section 6, tended to confirm this. The vertical compressive stress value obtained at a depth of 8 ft. from the side of a 25 ft.square pillar, was found to be 298 lb/in^2., a value in close agreement with the theoretical load due to the depth of overburden acting on the pillars. It should be emphasised that this was a single absolute pillar stress measurement obtained for a small pillar of the size proposed in future workings, surrounded by the larger development pillars, and it is therefore possible that this is not a representative stress value. However, provided the stress-relief used in this instance, is applicable to the Sherburn gypsum, and there is no reason at this stage to suggest otherwise, it gives an indication of the actual load carried by the pillars.

The pillar borehole deformation measurements, in providing a means of determining the state of stability of the pillars formed by the development workings, also indicated the effect of the planes of discontinuity present in the seam on the redistribution of the load due to mining. Though the opening and closing of these joints probably contributed to the lateral deformation recorded, the pillars were much too large for the joints to result in pillar instability.

In each borehole, most of the lateral movement recorded occurred in the extreme outer section of the pillars, probably within 2 ft. of the pillar edges. In each case, the outer bay length of the pillar boreholes continued to deform after the pillars were formed, even though there was no miningwithin several hundred feet of these pillars. This deformation can probably be related to the roof-floor convergence occurring in the surrounding roadways, that the several measuring stations throughout the mine have shown, continues after mining has ceased. The lateral deformation of the middle bay lengths of each borehole, representing the middle sections of the pillars, tended to become constant as soon as the pillars were isolated. It would seem therefore, that the middle sections of the pillars have achieved a measure of stability and there were indications at the time of the final measurement, that the extreme outer sections of the pillars were on their way to achieving this stability.

7.2 Recommendations for Future Work.

The underground measurements indicated a continuing closure of the roof and floor of the development workings. However, though the existing convergence stations indicated the magnitude of the convergence there was little indication concerning the reason for this continuing convergence. In view of this therefore, it is suggested that though further such measuring stations should be installed to provide a simple and very demonstrative control tool for monitoring the convergence, future work should include investigations designed to qualify in detail the behaviour of the roof and floor gypsum. It is suggested that this information could be obtained by means of both underground and laboratory investigations.

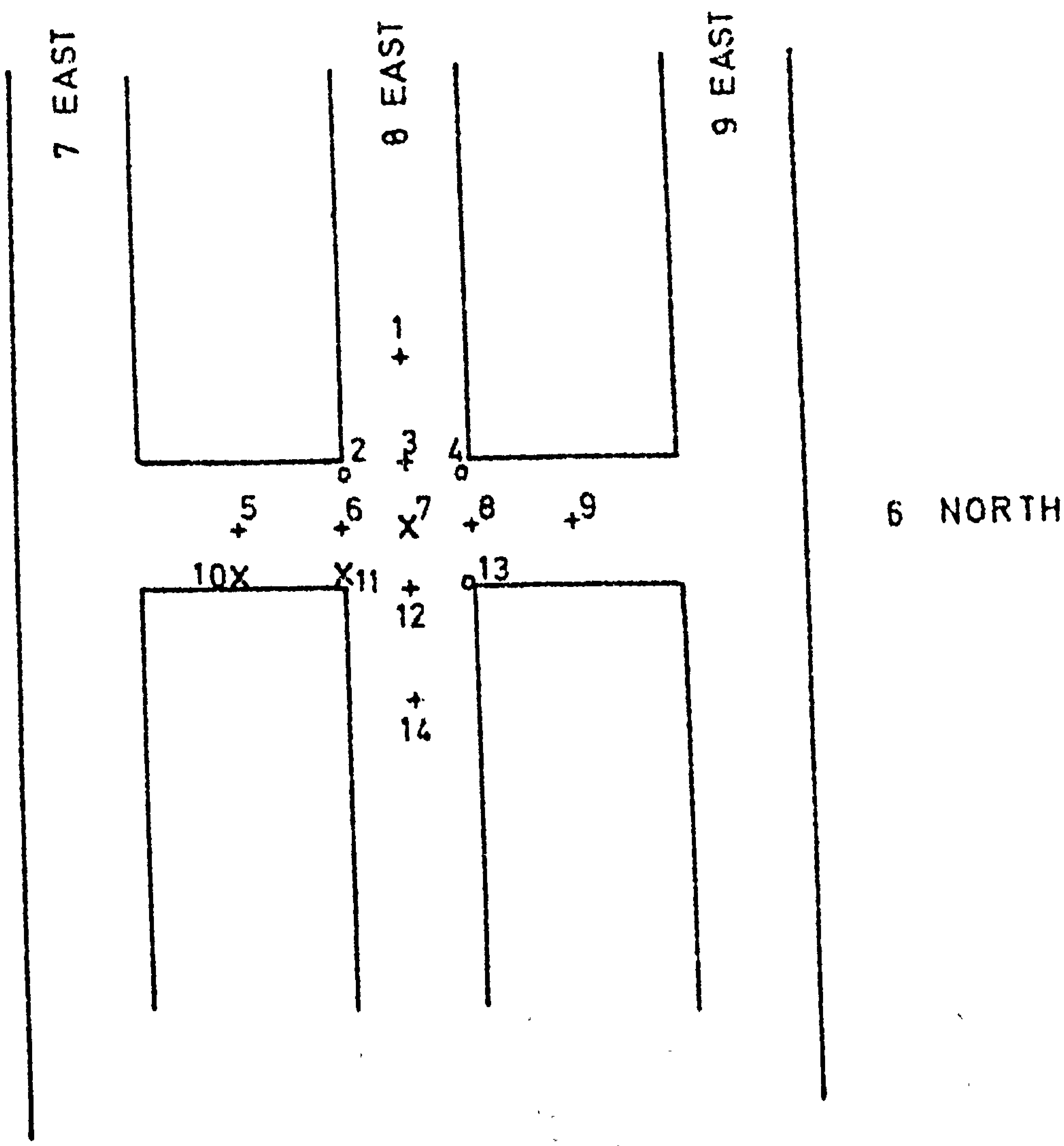
7.2.1 Underground Investigations.

An underground instrumentation scheme has been designed by the author utilising Extensometer techniques, to separate the various factors that were possibly contributing to the convergence being recorded. These factors are the downward movement of the immediate roof, bed separation occurring in the roof beam and floor lift.

It is suggested that a roadway intersection close to the working faces in the current development headings could be selected for instrumentation at a time when all preliminary arrangements for drilling and installation had been made. It was recognised that the primary need of an instrumentation scheme designed to define vertical movement within mine workings is the need for a stable datum to which all measurements could be related. In view of this and the fact that accumulative errors due to levelling over large distances could mask the actual movement recorded, a datum in the form of a long borehole anchor bolt, set at a depth of 20 ft. below the working level is proposed. This anchor bolt, consisting of three 6 ft. long sections of steel rod, with interchangeable stainless steel attachements at floor level for the taking of extensometer measurements and precise levelling, has been made and is shown in the photograph, Fig. 68.

A plan view of the instrumentation scheme is shown in Fig.67. It is proposed that 3 floor datum stations could be installed at the points referred to on the plan, Fig.67, as stations 7, 10 and 11. The use of 3 datums in close proximity would prevent any errors being introduced in the levelling procedure, the siting of the stations ensuring only one setting of the level for determining any change in level of all the various floor stations. The actual level instrument used would be of sufficient accuracy to produce precise levelling measurements. All recorded deformation could then be related to the stable datum with the required accuracy. Vertically above these floor datum stations, 6 ft. long roof boreholes are proposed with a number of borehole anchors with wires attached, fixed at various distances from the mouths of the roof boreholes. The anchor distances could probably be decided from a visual examination of the borehole prior to their installation. Roof-floor convergence measurements could then be carried out between the roof borehole and the floor datum station. The movement of

PROPOSED INSTRUMENTATION SCHEME TO DEFINE ROOF BEAM STABILITY.



1,3,5,6,8,9,12,14, – + – Combined Instrumented Roof Borehole and Convergence Station.

7,10,11, – X – Floor Datum, Instrumented Roof Borehole and Convergence Station.

2,4,13, – o – Convergence Station.

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FIG. 67

Fig. 68

any anchor could then be related to the datum anchor bolt in the floor.

The proposed stations referred to on the plan as numbers 1, 3, 5, 6, 8, 9, 12 and 14, would be of the form of a roof borehole,instrumented in a manner similar to that described above, together with a short floor convergence bolt vertically beneath the roof borehole to enable convergence measurements to be carried out. This instrumentation scheme would be completed by the installation of simple roof-floor convergence stations at the pillar corners referred to on the plan, Fig.67, as stations 2, 4 and 13. As mentioned previously, all floor stations would be linked by precise levelling to the 3 datum stations.

Prior to the installation of any instrumentation, an instrument available in the Department of Mining Engineering known as an Endoscope, could be used to examine the formation of the immediate roof. This Endoscope could be inserted into a borehole drilled at the centre of the intersection before instrumentation, to allow a visual examination of the borehole walls for signs of bed separation. Photographs could be taken at intervals of time in the event of bed separation being observed so that an assessment of the rate of deterioration of the roof beam could be made. This could be supplemented, prior to the installation of the roof borehole anchors, by an examination of the roof boreholes by means of a simple break detector in order to detect the presence of bed separation, and if so, whether or not the separation exists at a common horizon.

Preliminary arrangements were made by the author to install this instrumentation scheme at one of the centre intersections of the three development headings 7, 8 and 9 East, being driven to the North. This however, is a single intersection, and a similar instrumentation scheme should be installed at one of the multiple intersections at the corners of the large central block, which the development workings are in the process of isolating. This instrumentation scheme would provide perpendicular lines of roof-floor convergence occurring at an intersection, whilst the instrumented roof boreholes would indicate the relationship between this convergence and bed separation, if present. It should be emphasised however, that this can only be achieved by means of satisfactory datum anchor installation and precise levelling.

7.2.2 Laboratory Investigations.

The laboratory investigations that could be carried out in conjunction with the underground instrumentation scheme, should include both model tests and a theoretical appreciation of the behaviour of the roof beam. In view of the lack of understanding of the phenomena causing the continuing roof-floor convergence, it is necessary to obtain an understanding of the relationship between the failure of the roof beam and the magnitude of the induced strain in the beam at various estimated tensile stress levels in the roof beam. This knowledge could best be obtained in the laboratory by means of time-dependent strain tests of the model beam or Brazilian disc type. The laboratory technique for establishing this data is not entirely satisfactory, but measurement of the deformation of test specimens under different applied stresses would indicate the presence of a time-safe-strength of the Sherburn gypsum under tensile load conditions.

These laboratory model tests could also be supplemented by a theoretical analysis of the stability of the immediate roof beam similar to that carried out by the author, and described in Section 3. The theoretical analysis carried out by the author concerned a single layer roof beam configuration, but there was found to be little correlation between the calculated results and the underground measurements. In view of the suggested instrumentation scheme designed to determine whether or not the continuing convergence could be related to a separation of different layers making up the roof beam, the theoretical analysis could be continued by considering the roof beam as a double layer or even a multi-layer member. Assuming known dimensions of the roof beam, elastic constants determined in the laboratory, and underground measurements obtained from the proposed instrumentation scheme, it may then be possible to determine the composition of the roof beam and the nature of its deformation. Once this is known, the analysis could be extended to determine the maximum safe roof span for different beam thicknesses.

B. THE STAMPHILL MINE.

8. AN APPRECIATION OF THE STABILITY OF THE STAMPHILL MINE WORKINGS.

The Stamphill gypsum mine, situated near the village of Kirkby Thore in Westmorland, was the subject of an investigation carried out to determine the state of stability of the 'A' Bed workings. These investigations were initiated by Jones (5), who carried out laboratory strength tests on samples obtained from this bed. Since the geology and general conditions at the Stamphill mine were described in some detail by Jones (5), only a brief mention is made of this here.

The 'A' Bed gypsum occurs within the beds known as the St.Bees Shales and may be up to 120 feet in thickness, consisting for the most part of low grade gypsiferous marls. Where the gypsum content rises to a sufficiently high level, between 65 and 75% gypsum, economic mining may take place. That part of the seam which is considered to be economically worth mining is usually about 20 feet thick, although in some places this increases up to approximately 30 feet. The position of this economic zone within the 'A' Bed however, is not constant and the working horizon is moved up and down through the deposit to maintain a constant gypsum content. The countryside above the mine consists of gently rolling land with a general rise to the East, whereas the 'A' Bed deposits bear no relationship to the surface topography. Thus the thickness of overburden above the bed may vary considerably within a very short distance. Basically, the factors which can affect the stability of the mine workings are as follows :-

i) The depth of working varies considerably with a resulting variation in the load carried by the support pillars, assuming a constant pillar size throughout the mine.

ii) The working height can alter quite appreciably from one pillar to another. Thus a pillar at one place might be 24 feet square and 20 feet high, whilst an adjacent pillar with the same cross section might be 30 feet high. Such variations in the W/H ratio of the pillars means that the inherent stability of the pillar varies.

iii) The seam material can alter completely from soft marly material to tough amorphous gypsum, or to marls interspersed with satinspar. The whole deposit is a mass of thin bands of one type of gypsiferous material or another, and the occurrence or non-occurrence of one type of material has apparently little effect on the position of the economic zone. Therefore, at one place support pillars may be formed from massive amorphous material whereas a few yards away the pillars may be formed from marls containing a large number of satinspar bands.

iv) The fact that the seam is made up of a succession of thin bands means that the stability of the roof is an everpresent problem.

The optimum dimensions of a working layout which caters for these factors are limited only by the ability of the seam material to maintain itself in stable equilibrium in and around the proposed workings, and it is only after the determination of the mechanical properties of the seam material that a preliminary assessment of the state of stability of the workings can be established. Preliminary investigations carried out by Jones (5) to obtain such data, had suggested that, as at Sherburn, roof stability would most probably be a problem in certain parts of the 'A' Bed workings. At the same time, however, there was the possibility that in the deeper parts of the mine, the compressive stresses induced in the pillars, due to the weight of the superincumbent strata, might approach values which could well be considered as limiting.

The fact that the depths at which mining was carried out varied so much, i.e. about 100 ft. at the more shallow parts but down to approximately 900 ft. in other parts, tended to set up two sets of limiting factors. Obviously, in the shallow parts of the mine, there was no possibility of the bearing capacities of the pillars being exceeded at the rates of extraction which were used. In these areas the stability of the maximum roof span, i.e. the unsupported roof above roadway intersections, would limit the permissible room width. In the deeper parts of the mine, however, since the tensile stresses induced in the outermost fibres of a roof span would not depend on the depth below ground of the span, but only upon its actual thickness, it was possible thet the compressive stresses in the pillar could approach the ultimate compressive strength of the pillar material.

Of the two effective limits imposed in 'A' Bed, that of the seam compressive strength and hence the stability of the pillars, could be the most important. Whereas a roof failure would cause a local roof fall it would tend to lead to natural arching and thus stability. A pillar collapse in the deeper parts of the mine however, could give rise to cumulative load transfer and thus the possibility of a much more widespread collapse. For this reason, it was decidedthat the investigations carried out by the author would pay much greater attention to pillar stability rather than roof stability.

8.1 Synopsis of Data Obtained from the Compression Testing of the Seam Material.

The above discussion concerning the problems involved in determining the stability of the workings indicated the importance of obtaining data relating to the strength properties of the seam material, and as for the Sherburn mine, this data was obtained by Jones (5).

All the samples for compressive testing were taken either from the economic zone or its immediate vicinity as it was most unlikely that support pillars would be formed outside this zone. It was found upon visual examination of the samples, that the test specimens were made up of numerous layers of widely differing rock material and it was decided to classify these into three rock types referred to as Rock Types A, B and C. Rock Type B however, was quite difficult to obtain in reasonably sized pieces. For this reason the compressive strength data obtained from Rock Type B is not very representative. To a certain extent the same comment applied to Rock Type C. However, since these two Rock Types do not play a particularly important part within the working horizon, it was not felt to be a serious handicap to establishing representative values for the ultimate compressive strength of the general seam material.

Fine Grained/Amorphous Gypsum.	Rock Type A.
Total number of specimens	33
Mean compressive strength	3387 p.s.i.
Standard deviation	1093 p.s.i.
Crystalline Gypsum 'Daisy Band' Type.	Rock Type B.
Total number of specimens	3
Mean compressive strength	1605 p.s.i.
Standard deviation	186 p.s.i.
Crystalline Gypsum in Fine Matrix.	Rock Type C.
Total number of specimens	7
Mean compressive strength	2611 p.s.i.
Standard deviation	1120 p.s.i.
MEAN COMPRESSIVE STRENGTH OF TOTAL	3147 p.s.i.

Over the small volume range covered by an investigation into the size effect on compressive strength, there appeared to be a limiting compressive strength for the seam material of 2400 p.s.i.. Bearing in mind the predominance of Rock Type A, the strongest of the three, it would appear reasonable to suggest that the presence of the laminations of weaker material in fairly small quantities would not cause the effective limiting compressive strength of the material to be much less than it would be if the pillar consisted only of Rock Type A.

8.2 Method of Approach Used in this Investigation.

The investigations carried out by Jones (5) indicated that the rock material making up the economic zone of the 'A' Bed at the Stamphill mine, does not lend itself to the laboratory determination of its mechanical properties. For this reason it is not possible to draw any specific conclusions relating to the stability of the mine workings as a result of the laboratory work. In addition to this, the laboratory investigations indicated that the deposit does not lend itself to theoretically based idealised treatments of the type carried out at Sherburn. It became apparent therefore, that relevant information could be obtained by means of 'in situ' investigations only.

The stability of the immediate roof at Stamphill tends to resolve itself into two parts, which may be described as :-

a) The local stability of the constituents of the roof.

b) The overall stability of the immediate roof strata when in the form of a continuous beam.

The presence of certain constituents that may cause local instability within an otherwise stable roof can only be effectively tackled by roof bolting the offending areas. It should be remembered that this localised failure is not necessarily related to the length of the unsupported span, but there exists the probability that the longer the span the greater is the likelihood that a weak structure be found within it.

The laboratory investigations into the strength characteristics of the seam material together with a visual examination of the mine roof indicated factors which may greatly influence the stability of the immediate roof of the workings. These are :-

a) Large variations in the thickness of the roof beam can occur over a very short distance.

b) The type of material forming the roof of the roadway can vary along the length of the roadway, and hence the mean tensile strength of the immediate rock material will be altered.

c) There isa very wide variation in the tensile strength values which makes it almost impossible to predict with any accuracy, the ultimate tensile strength of a particular section of roof strata.

In view of this, the simplest and most practical way of achieving reasonable roof stability would beto encourage the formation of a stable natural arch to the roadway, so that the weak material would tend to support itself. A disadvantage of this type of solution is that the problem of ore dilution by the lower grade material above the working zone, would tend to preclude its application in particular situations where ore concentration was a critical factor. Where this roadway configuration has been used in the mine however, complete stability of the roadway has been obtained. This would be a more satisfactory method of maintaining roadway stability than the uncertain and impractical method of simply reducing the span or the expensive one of extensive roof bolting. Therefore, whilst the general stability of the immediate roof can obviously prove troublesome, it can be overcome if the need arose, by the formation of a semi-elliptical roof to the roadway.

In view of this, the 'in situ' investigations carried out by the author at the Stamphill mine have been confined to an investigation into the stability of the pillars in the deeper sections of the mine. As in the underground investigations carried out at Sherburn, the main basis of the investigation was the measurement of strata displacement within the pillars as they were formed, utilising borehole extensometer techniques. It was thought that the direct measurement of the lateral displacement of the pillar material under load, over extended periods of time from the formation of the pillar, could provide a means of determining stability.

In addition to the measurement of the pillar deformation, it was decided that an attempt should be made to measure the stress change in pillars as they were formed. The measurement of stress in a brittle rock material is a difficult operation, and in a rock material of a laminated nature as at Stamphill, it is even more difficult. It was decided, however, to attempt to measure stress changes, or at least to indicate the qualitative stress behaviour in the pillars instrumented for deformation measurements, as mining proceeded. The instrument used to measure these stress changes will be described in the following Section.

9. INSTRUMENTATION.

In addition to the obtaining of direct measurements of the lateral displacement of the pillar material under load by means of the borehole extensometer technique, described in section 4.1, an attempt was also made to measure the re-distribution of the pillar load by means of a hydraulic borehole stressmeter. This instrument, developed in the Mining Engineering Department, University of Newcastle upon Tyne, has been described in some detail elsewhere,(79) (80) (82) and is only briefly described here.

9.1 Description of the Stressmeters.

The stressmeters used in this investigation are of the high modulus inclusion type and are capable of measuring a 'uni-directional' deformation in a borehole. The instrument is based on the principle that the stresses set up inside a high modulus inclusion within a material of comparatively low modulus are simple in nature, and do not vary considerably with any changes which may take place in the host material. Fig. 69 shows a cut-away view of a stressmeter; two main components are shown in this figure, a steel blade on the left hand side elevation and a pressure sensitive head on the right hand side. Running through the centre of the blade is a thin rectangular slot, 0.01 in. x 1.34 in. wide, and approximately 5.5 in. long which is completely filled with a soluble oil and water mixture, with an

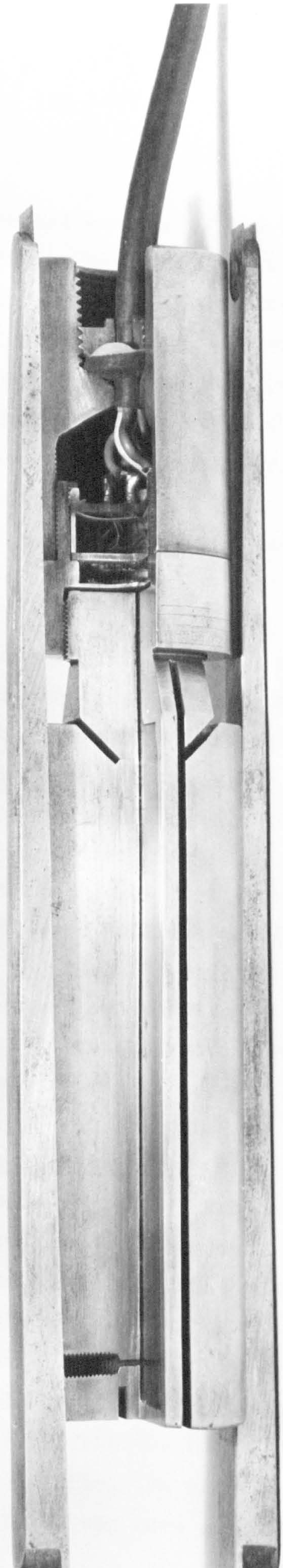


Fig. 69

outlet through the threaded section of the blade to a 50 thou. thick diaphragm machined into the head. This diaphragm thickness was chosen after considering previous stressmeter work carried out by the Department of Mining Engineering. Keyways for steel sliding wedges are machined above and below the slot. When the stressmeter is driven between the wedges, they bed themselves into the borehole walls thus making contact with the rock. To assist this contact, it is usual to coat the wedges with an adhesive such as plastic steel prior to insertion, in case the borehole walls should have been scoured during drilling.

A change in rock deformation perpendicular to the stressmeter slot due to rock stress changes, is transmitted through the wedges to the stressmeter blade. The resulting deformation causes a pressure change which strains the diaphragm. Four small 120 ohm. resistance strain gauges, forming a bridge, are positioned on the diaphragm at areas of high strain intensity. Their strain output is measured with a portable Peekel Strain Bridge and Capacitance Box. Provided that excessive deformation of the diaphragm does not take place - resulting in plastic straining of the steel - a relationship exists between the load applied to the stressmeter and the measured strain output. To determine this relationship, a number of calibrations have to be carried out on each stressmeter.

9.2 Calibration of the Stressmeters.

The stressmeter heads were first subjected to a number of calibrations designed to give an assessment of the range and accuracy of the stressmeters. Both stressmeters were then calibrated in a borehole drilled through an elastic material. A steel block was used for this, of size 18 in. x 18 in. x 6¾ in. wide, with a 1.15/16 in. diameter hole drilled through the centre of the 18 in. square face. By using a calibration block of large dimensions compared with the borehole size, the stress distribution around the stressmeter borehole will be the same as that around a similar borehole in an infinite block. The steel block was placed in a 500 ton capacity compression testing machine and a 200 ton load applied. The stressmeter wedges and keyways were coated with plastic steel and then inserted into the borehole. An initial amount of pre-stress was applied by gently driving the stressmeter between the wedges. Incremental loading and strain measurements were then carried out for a number of load cycles.

Both stressmeters gave a linear relationship between the measured strain and applied stress, showing that its behaviour was good in an elastic medium.

The final and most important calibration, should be the calibration of the stressmeters in a block of the host rock into which they are to be installed. Unfortunately, a laboratory calibration of the stressmeters to interpret field readings directly could not be obtained. The reason for this was that it was found to be impossible to obtain a calibration block from the samples taken from the mine. The laminated nature of the seam material caused the rock samples, which had been obtained from the mine with considerable difficulty, to disintegrate under the action of the cutting wheel. The stressmeters were therefore calibrated in a sandstone block and corrections made for the difference in the physical properties of the rocks. The obtaining of the calibration correction factor used is described in the following Section.

The calibration procedure for a sandstone block was almost the same as that described for a steel block. The block, 17 in. x 17 in. x 8 in. wide, was made of Springwell sandstone, a rock material whose properties were very well known. As before, plastic steel was used to bed the stressmeters with the rock. A stressmeter under calibration is shown in Fig.70.

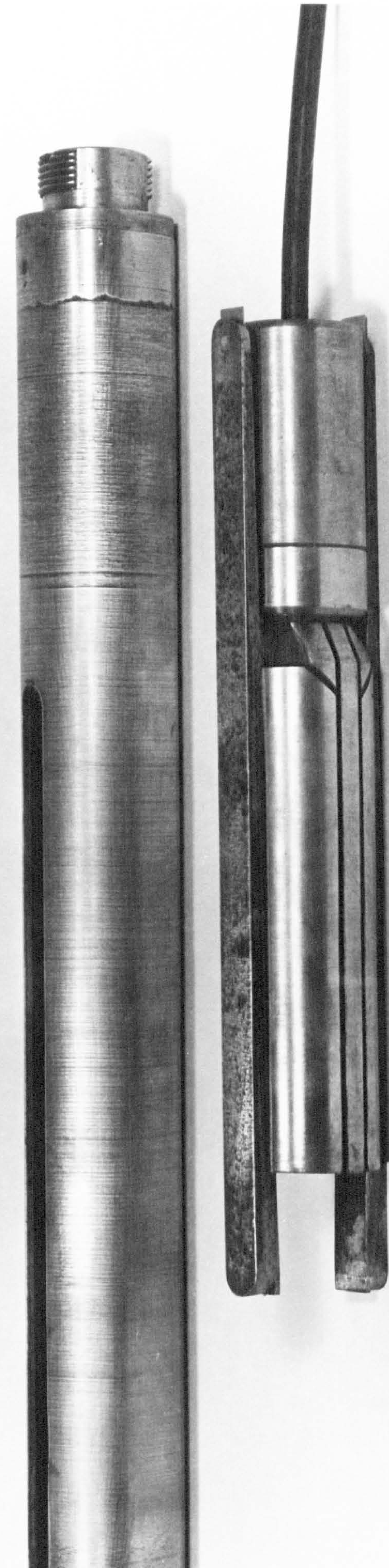


Fig. 71

It was noticed that once loading had commenced there existed a linear relationship between stress and strain, this relationship also holding good during unloading.

The sensitivity of the stressmeter is a function of the axial thrust applied to the instrument when it is forced between the wedges. An increase in axial thrust increases the area of contact and also the sensitivity of the instrument. A measure of axial thrust is given by the strain gauge bridge and is equal to the difference between the bridge output before insertion into the hole, and its output after insertion. This difference is termed the pre-set load.

A stressmeter's sensitivity can be expressed as follows :-

$$\text{Stressmeter sensitivity} = \frac{\text{Stressmeter strain output, }\mu\text{s.}}{\text{Corresponding stress increment lb.in}^{-2}}$$

The sensitivity or calibration factor of the stressmeters used, as tested in the sandstone block, are shown in Table 6, hereunder.

TABLE 6

Stressmeter Number	Sensitivity lb/in^2/10 µs.	
	First Cycle	Last Cycle
202	5.34	5.32
203	5.78	5.78

9.3 Calibration Correction Factor.

Ideally, the stressmeter should be calibrated under triaxial loading conditions in the rock into which it is to be inserted. As this was not possible, corrections were made to the readings in the following manner.

Salamon (60) gives the output of the stressmeter, r, as follows:-

$$r = \frac{a\ Ue}{1 + \omega \dfrac{Est}{E_R}} \qquad (13)$$

where a = calibration constant

Ue = borehole deformation in direction of measurement

ω = design constant of the stressmeter

Est = effective modulus of the stressmeter

E_R = Young's modulus of the rock

The values of ω and Est were obtained in a laboratory investigation by Tomlin (62) and found to equal 0.5 and 20×10^6 p.s.i. respectively. Therefore, the output, r_1, of a stressmeter under uniaxial load is given by :-

$$r_1 = \frac{a\ U_1\ E_{R_1}}{E_{R_1} + 10^7} \qquad (14)$$

where E_{R_1} = Young's modulus of calibration rock

U_1 = borehole deformation under conditions of no lateral constraint

Similarly, triaxial stress conditions in any rock are given by :-

$$r_2 = \frac{a\ U_2\ E_{R_2}}{E_{R_2} + 10^7} \qquad (15)$$

where U_2 = borehole deformation under triaxial stress conditions

E_{R_2} = Young's modulus of field rock

Therefore
$$r_1 = \frac{r_2 \, U_1 \, E_{R_1} \; (E_{R_2} + 10^7)}{U_2 \, E_{R_2} \; (E_{R_1} + 10^7)}$$

$$= r_2 \, C_1 \, \frac{U_1 \, E_{R1}}{U_2 \, E_{R2}} \qquad (16)$$

The deformation, U_1, of a borehole under uniaxial loading without constraint is given by :-

$$U_1 = \frac{\sigma_1 \, b}{E} \; (1 + 2 \cos 2\theta) \qquad (17)$$

where
σ_1 = vertical stress
E = Young's modulus of host rock
θ = orientation angle to principal stress
b = radius of borehole

For a point at the perimeter on the vertical axis :-

$$\theta = 0$$

Therefore
$$U_1 \, E_{R_1} = 3 \; \sigma_1 \; b \qquad (18)$$

Under triaxial stress conditions where

$$\sigma_2 = \sigma_3 = \sigma_1 \frac{\gamma}{1-\gamma}$$

where $\sigma_2 = \sigma_3$ = lateral stress

γ = Poisson's ratio of the host rock

U_2, is given by :-

$$U_2 = \frac{\sigma_1 \, b}{E} \; \{ (1 + \gamma) + (1 - \gamma - 2\gamma^2) \, 2 \cos 2\theta \} \qquad (19)$$

Therefore, for the same point :-

$$U_2 \, E_{R_2} = \sigma_1 b \, (3 - \gamma - 4\gamma^2) \qquad (20)$$

Then equation (16) becomes :-

$$r_1 = r_2 \, C_1 \frac{3}{(3 - \gamma - 4\gamma^2)} = r_2 \, C_1 \, C_2 \qquad (21)$$

The ratio r_1/r_2 is the calibration correction factor. The core material obtained from each stressmeter borehole was examined. Samples of the same rock were collected and subjected to laboratory tests to determine its elastic constants. The values obtained were then compared with the same values obtained by Jones (5), and the following mean values of Poisson's ratio and Young's modulus used.

Poisson's ratio	= 0.29
Young's modulus	= 2.7×10^6
Young's modulus of calibration rock	= 2.80×10^6
Correction factor, C_1	= 0.99
Correction factor, C_2	= 1.26
Calibration Correction factor	= 1.247

10. THE DESIGN OF THE UNDERGROUND INVESTIGATION AND THE INSTALLATION OF THE EQUIPMENT.

Due to the fact that the gypsum deposits bear no relationship to surface topography, the depth of the workings in the 'A' Bed gypsum varies from approximately 100 ft. to 900 ft. Therefore, assuming a constant pillar size the load on the support pillars varies throughout the mine. The working dimensions are usually 16 ft. wide roadways leaving 24 ft. square pillars, with the working height varying between 10 ft. and 30 ft. depending on the quality of the seam.

INSTRUMENTATION SITE AT THE STAMPHILL MINE

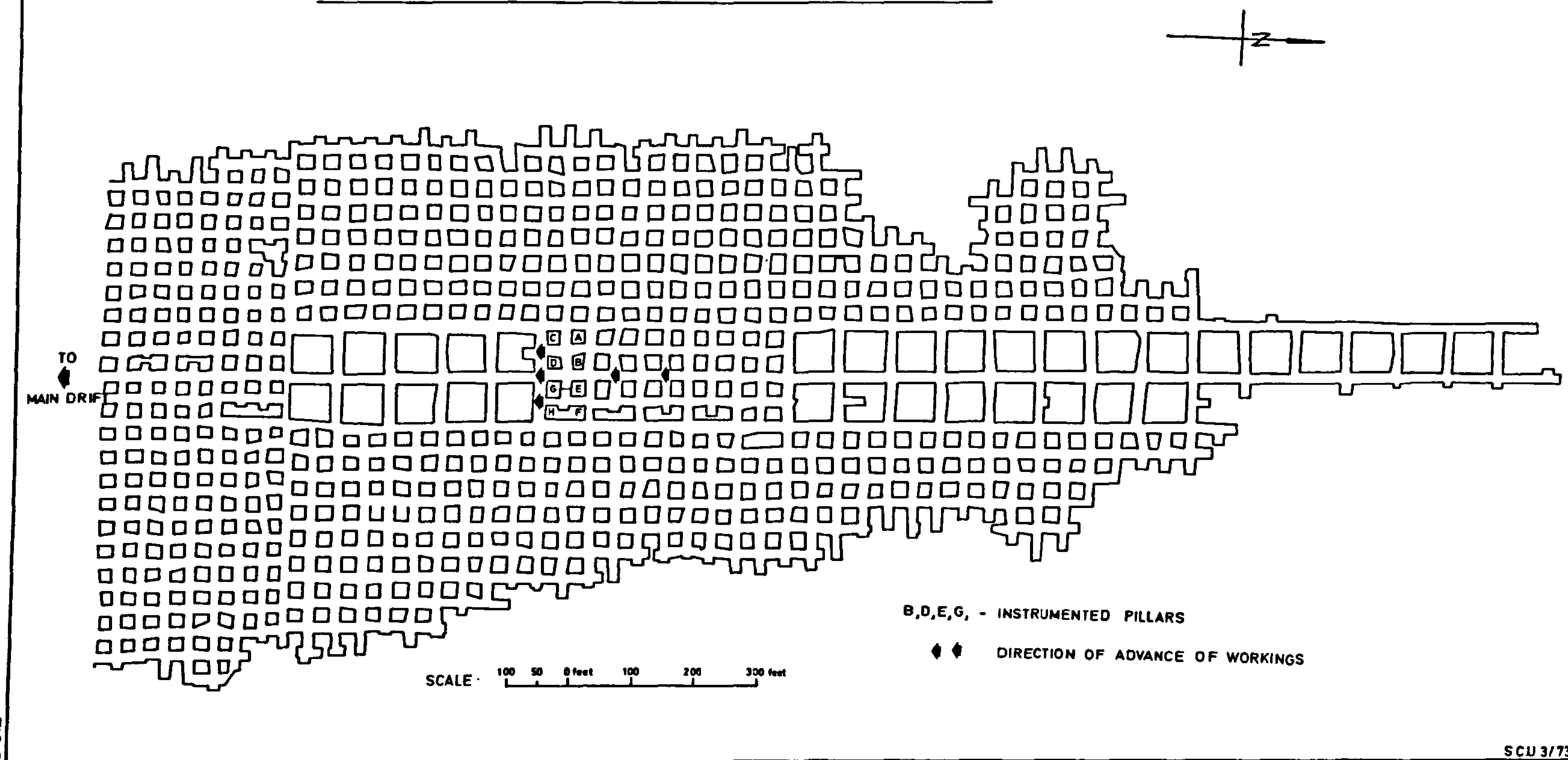


FIG. 72

In selecting a site, therefore, for the installation of instruments in suitable pillars, the fact that there was a variation both in the depth of cover and the W/H ratio of the pillars was considered. As a result, it was decided that the instrumentation site chosen should be in the deepest section of the mine, in an area where the full effect of the cover load would be transmitted to the instrumented pillars. It would then be possible to infer from the measurements obtained, the state of stability of the instrumented pillars, and with the assistance of the already available laboratory data, the state of stability of the pillars having the same dimensions as the instrumented pillars in the more shallow parts of the mine.

In July 1969, when the instrumentation scheme was first proposed, there were no workings being developed in the deeper section of the mine. There were, however, a number of large pillars, 64 ft. square, about 3/4 mile E.N.E. of the Stamphill drift. These pillars had originally been left to protect the main conveyor roadway to that district, and were situated at a depth of approximately 700 ft. from the surface in the approximate centre of a worked area between 700 and 800 ft. wide, as shown in Fig.72. Partial extraction of these large pillars was being undertaken by driving 16 ft. wide and 10 ft. - 12 ft. high headings from the middle of each pillar side, thus dividing each large pillar into four smaller pillars 24 ft. square. Then using the broken rock material as a working platform, the roadway height was being increased up to between 20 and 30 ft. high, depending on the quality of the seam material at this horizon. An arched roof was being formed to the roadways, resulting in almost complete roof stability in the area with the exception of some localised spalling.

It was therefore decided that the underground pillar instrumentation should be carried out utilising these large pillars which were ideally situated in the deeper section of the mine. Measurements taken from instruments placed in some of the resulting smaller pillars prior to mining, would indicate the effect of the re-distributed load due to mining on the stability of the instrumented pillars. This instrumentation would also provide an opportunity for determining the effects on the pillar stability, of increasing the height of the pillars.

10.1 The Instrumentation Scheme.

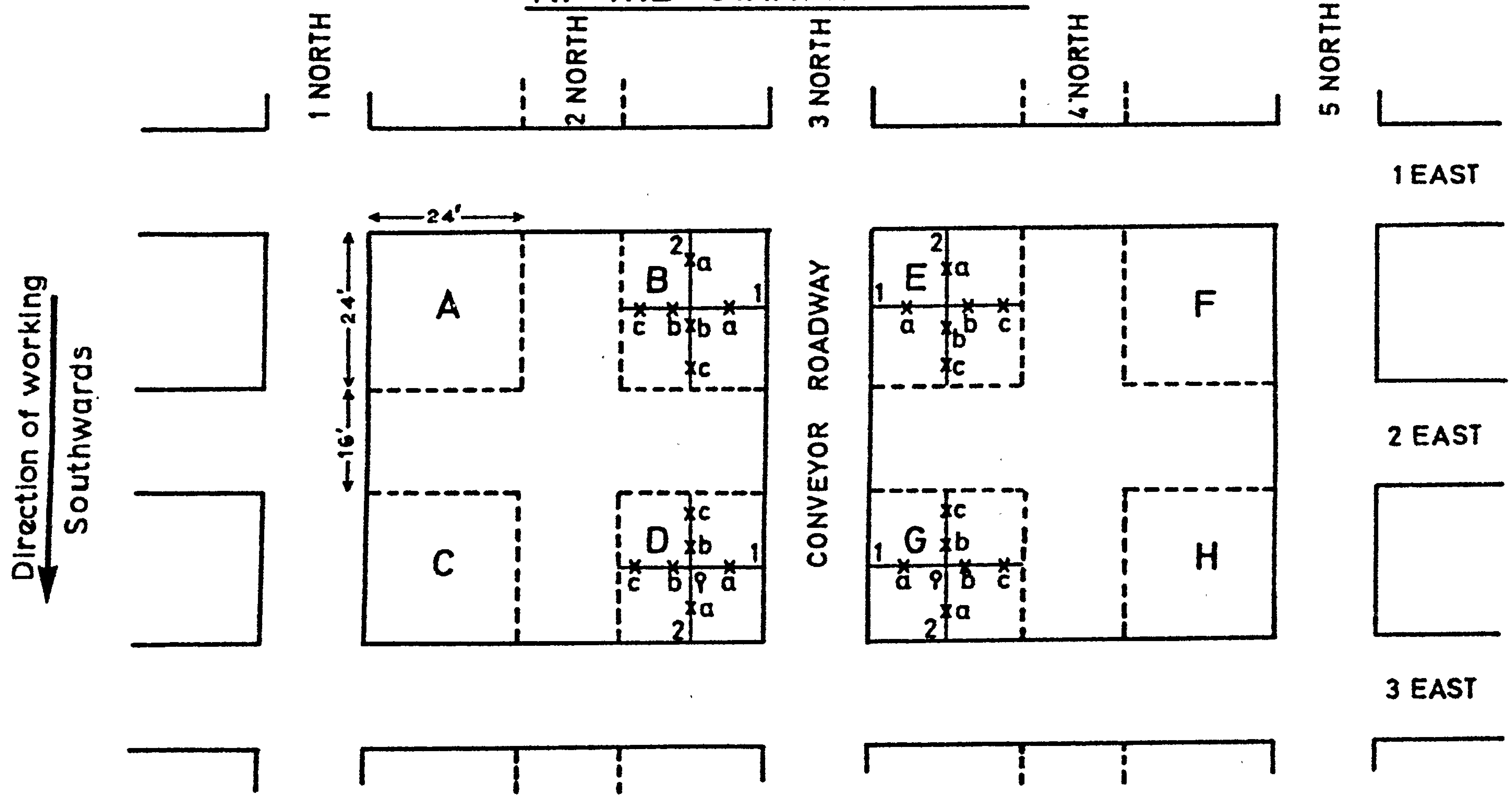
The objective of the instrumentation scheme installed, was to determine the effect of the re-distribution of the load due to the partial extraction of the large pillars, and hence the stability of the resulting smaller pillars. The instrumentation scheme described here was designed to record the effect of this re-distribution of the load both in the form of pillar lateral deformation and pillar stress behaviour as mining proceeded.

The site of the instrumentation is shown in Fig.72. It will be noticed that the direction of advance of the 'splitting' of the large pillars was towards the main Stamphill drift, i.e. from North to South. The two large pillars indicated were chosen for 'in situ' observations mainly on the basis that they could be prepared for instrumentation well in advance of the date they were due to be worked.

For ease of reference, the pillars to be instrumented and the surrounding roadways were numbered as shown in Figs.72 and 73. The small pillars formed by the 'splitting' of the two larger pillars were referred to as A, B and C, etc. up to H, and the three parallel roadways forming the two large pillars were referred to as 1 North, 3 North and 5 North, whilst the two proposed roadways 'splitting' the two large pillars were referred to as 2 and 4 North. Hence the roadways connecting 1 North and 3 North were referred to as 1 East, 2 East and 3 East.

The roadway 1 North had already been formed to its full working height of approximately 25 ft. with an arched roof, as also had roadway 5 North. In addition to this, roadway 5 North was

PLAN SHOWING THE BASIC INSTRUMENTATION LAYOUT AT THE STAMPHILL MINE



D_1 and D_2-Boreholes in Pillar D. o- Stressmeter.

x Position of Anchors in Borehole.

S.C.U.3/74

FIG. 73

approximately 8 ft. lower than the roadways it intersected, 1 East and 3 East. This was due to the fact that there had been a change in the working horizon at this point due to a variation in the gypsum content of the seam. The roadways 1 East, 3 East and 3 North were only 10 ft. - 12 ft. in height with a minimum of a further 10 ft to be removed from the roof as the pillars were being formed.

10.1.1 Pillar Lateral Deformation.

As at Sherburn, it was proposed that pillar lateral deformation measurements be carried out, and this would form the major part of the instrumentation scheme. It was planned to instrument the small pillars formed by the partial extraction of the two large pillars. The planned location of the deformation measuring instruments is shown in Fig.73. Borehole anchors with steel wires attached were to be installed in horizontal boreholes drilled through the proposed width of these small pillars, at the middle of the proposed pillar sides. To record the largest possible total lateral deformation of these pillars, the boreholes were to be drilled and instrumented before extraction of the large pillars commenced. In order to obtain the maximum amount of information from this scheme, it was proposed that the instrumentation be concentrated in the four centre pillars B, E, D and G.

Since the pillar height was to be increased from 10 ft. to between 20 and 25 ft., the boreholes were to be drilled as high up the pillar as was possible so that the borehole would be situated in the mid-section of the final pillar height. The proposed relative positions of these anchors, not more than three in each borehole to minimise wire interference, was 6 ft. (1.83 m.), 15 ft. (4.57 m.) and 20 ft. (6.09 m.) respectively from the mouth of each borehole. This is shown in Fig.73 together with the reference number for each anchor. The Mk.1 Extensometer, described previously, was to be used to carry out all measurements.

In addition to the measurement of the pillar lateral deformation, it was originally proposed that roof-floor convergence stations would also be installed around the instrumented pillars. In view of the fact that the roof of the roadways was to be removed, these stations could not be installed prior to mining, whilst the excessive height and the unavailability of a means of access to the high roof after mining, meant that this type of measurement was impractical.

In order to supplement the pillar deformation measurements it was decided to measure the stress changes induced in the proposed small pillars D and G situated on either side of the central conveyor roadway 3 North. In order to measure this transference of the load as a result of mining, two hydraulic stressmeters referred to as 202 and 203, were to be installed in boreholes at the centre of the proposed pillars D and G. In view of the very slow advance of the working of these pillars it was felt that a continuous recording of the output of these stressmeters was not necessary and it was decided that measurements would be taken at fairly regular intervals using the Peekel Strain Bridge.

10.2 Installation of the Equipment.

The installation of the measuring equipment was carried out without alteration to the original instrumentation scheme. Boreholes, 1.11/16 in. diameter were drilled in the proposed small pillars B, D, E and G as planned, at the centre of the small pillar sides, to a depth of 25 ft. The height of the mouth of these boreholes above the floor was as follows :-

		Height of borehole
Pillar B :	Borehole 1	9 ft.
	Borehole 2	7 ft.
Pillar D :	Borehole 1	8 ft. 6 in.
	Borehole 2	8 ft.

		Height of borehole
Pillar E :	Borehole 1	8 ft.
	Borehole 2	9 ft.
Pillar G :	Borehole 1	8 ft.
	Borehole 2	7 ft.

These boreholes were instrumented on 12.9.69. The end anchors with wires attached were inserted at a depth of 20 ft. from the mouth of the borehole. This was 4 ft.short of the proposed edge of the pillars, but an examination of pillars already formed indicated that in some instances the pillars were only between 20 and 21 ft. wide. Also, it was felt that by leaving the anchor a few feet short of the pillar edge would prevent it being dislodged by the blasting. Following this, two intermediate anchors were installed in each borehole at distances of 15 ft. and 6 ft. from the mouth of the hole respectively. Instrumentation was completed with the installation of the borehole mouth anchors.

A total of 8 boreholes were instrumented in this way, three anchors in each borehole. For ease of reference, the boreholes in each pillar running parallel to the central conveyor roadway are referred to as borehole 2, and those running perpendicular to the central conveyor roadway are referred to as borehole 1. To prevent the mouth of hole anchors being damaged by blasting and fall of rock material from the roof, the mouth of the borehole was countersunk into the pillar sides for several inches, so that the mouth anchor did not protrude beyond the line of the pillar side.

The stressmeters were installed some weeks later. Stressmeter 203 was installed in pillar D on 28.10.69, whilst stressmeter 202 was installed in pillar G on 4.11.69. The boreholes in which the stressmeters had been positioned were drilled using a 2.1/4 in. diameter rotary wing bit to within 3 ft. of the proposed installation position at the approximate centres of the small pillars D and G.

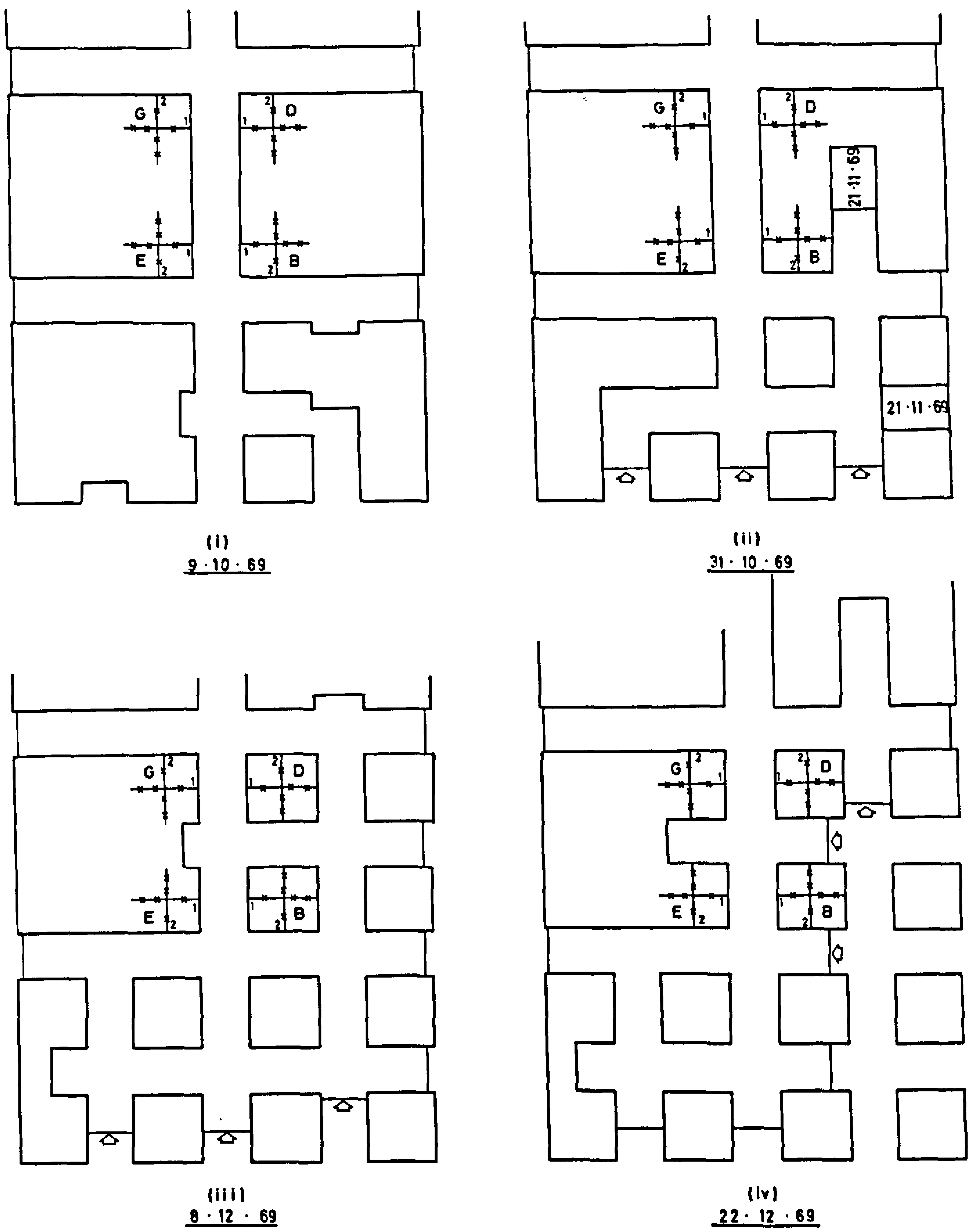
Beyond this point, a 1.15/16 in. diameter diamond core barrel was used to provide an accurate and smooth walled borehole for the stressmeter. The stressmeter to be installed, was encased in a steel case or sleeve shown in Fig.71, with two slots machined along its length to allow a direct contact between the rock material and the wedges. On the rear end of the sleeve is an adaptor, connected by a quadrant thread to a hydraulic ram. Prior to installation the wedges were coated with fast setting plastic steel. Then the stressmeter unit was inserted at its correct orientation in the borehole, and pushed by means of extension rods to its position in the centre of the pillar, 12 ft. from the pillar edge. Hydraulic pressure was then applied to the ram, pushing the stressmeter between the two key wedges within the borehole. When the desired pre-stress was reached the pressure was maintained for several minutes, during which time the plastic steel was squeezed from between the wedges and the borehole wall contact, until the pre-stress level became reasonably constant. The initial reading was then taken on the stressmeter. This procedure was repeated for the second stressmeter. Following their installation, care was taken to ensure that the electrical leads from the stressmeters were adequately protected to prevent them being damaged by the mining operations.

The complete instrumentation scheme was installed as planned, the initial readings being taken on thepillar boreholes on 12.9.69, and the initial stressmeter readings taken on 28.10.69 and 4.11.69 for stressmeters 203 and 202 respectively. Subsequent measurements were taken at fairly regular intervals. In some instances however, there were quite long intervals between consecutive readings due to the fact that working in this area was temporarily suspended.

11. ANALYSIS OF RESULTS.

A plan showing the pattern of extraction at the times measurements were taken is shown in Figs.74 and 75. These plans indicate the position of the working headings being driven through the large pillars on various

EXTRACTION PLAN



POSITION OF 'ROOFING' OPERATIONS.

S.C.U.3/75

FIG. 74

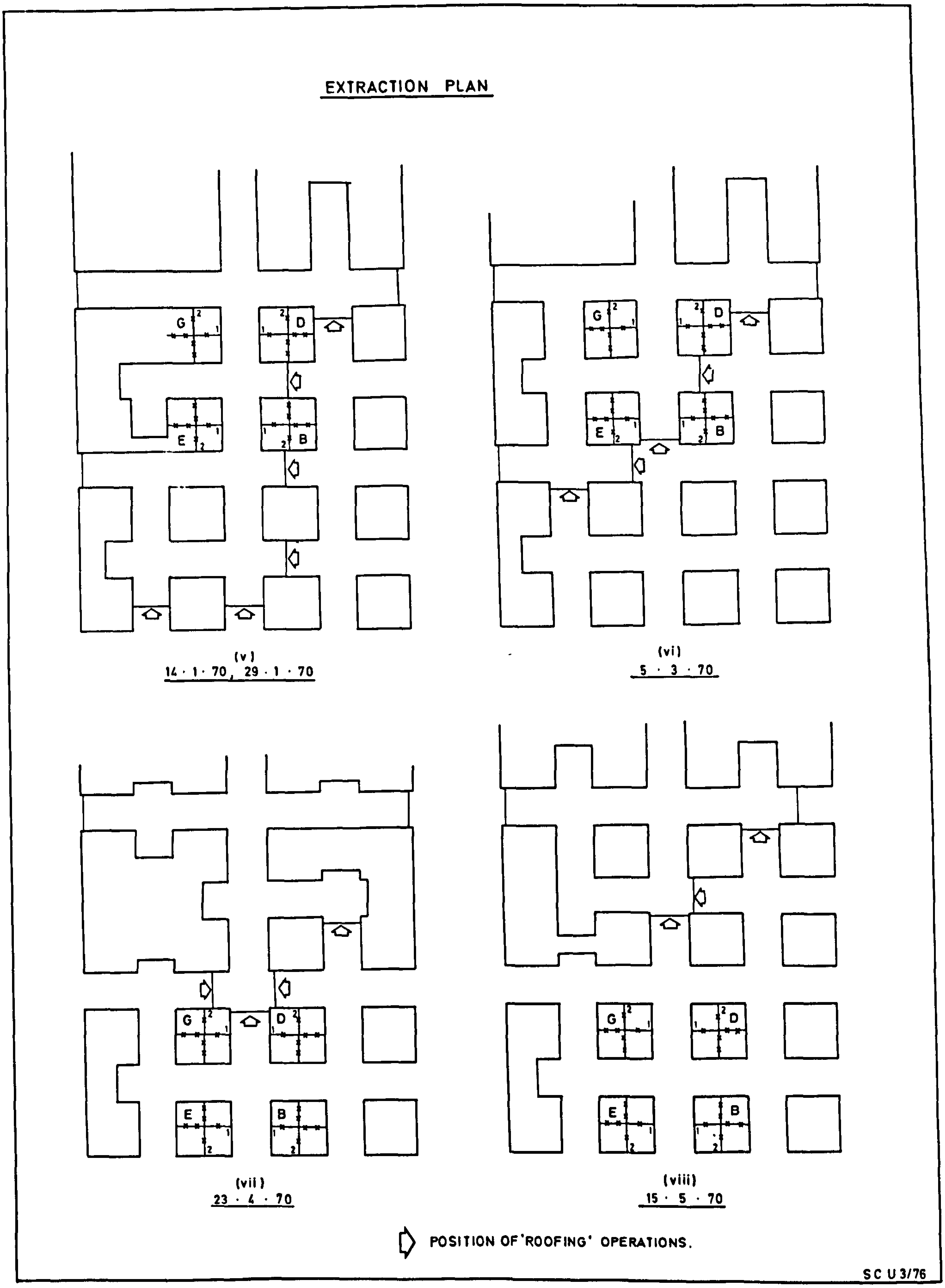

EXTRACTION PLAN
G
D
E
B
(v)
14 · 1 · 70, 29 · 1 · 70
(vi)
5 · 3 · 70
(vii)
23 · 4 · 70
(viii)
15 · 5 · 70
POSITION OF 'ROOFING' OPERATIONS.
SC U 3/76

Fig. 75

days, together with those parts where secondary roof extraction had been carried out. The plans of the workings should be compared with the results obtained from the two stressmeters and the pillar lateral deformation measurements.

11.1 Interpretation of the Stressmeter Results.

Measurements were obtained from both stressmeter 202, pillar G, and stressmeter 203, pillar D, over a period of approximately 220 days. The method of calibrating the stressmeters and the various adjustments made to the calibration factor to allow for the fact that the calibration was carried out in a different rock material to that underground, have been dealt with in Section 10. The corrected calibration factors of the stressmeters have been used to convert the actual stressmeter output readings in micro-strains, into pounds per square inch. The results derived from the two stressmeters are shown in Fig. 76 on a time basis. These results indicate the change in the load acting on the pillar and not the absolute pressure. This has been done by assuming that the load acting on the pillar at the point of measurement, prior to mining was zero, the stressmeter output indicated on the same day being the pre-set load of the stressmeter. The results shown therefore, are not absolute but comparative. As mentioned previously, stressmeter 202 was installed 7 days after stressmeter 203, but the results are presented in Fig.76 on the same time basis for ease of comparison.

At the time stressmeter 203 was installed in pillar D, work had just commenced on forming the roadway 2 North from roadway 1 East in the large pillar containing the 4 smaller pillars A, B, C and D, and by the time stressmeter 202 was installed in pillar G, this heading had advanced a distance of 15 ft. into the large pillar. Immediately following their installation there was an increase in load recorded by both stressmeters, but on 21.11.69, both stressmeters showed a reduction in output. The fall in load recorded by stressmeter 203 in pillar D was less pronounced, due probably to the fact that it was

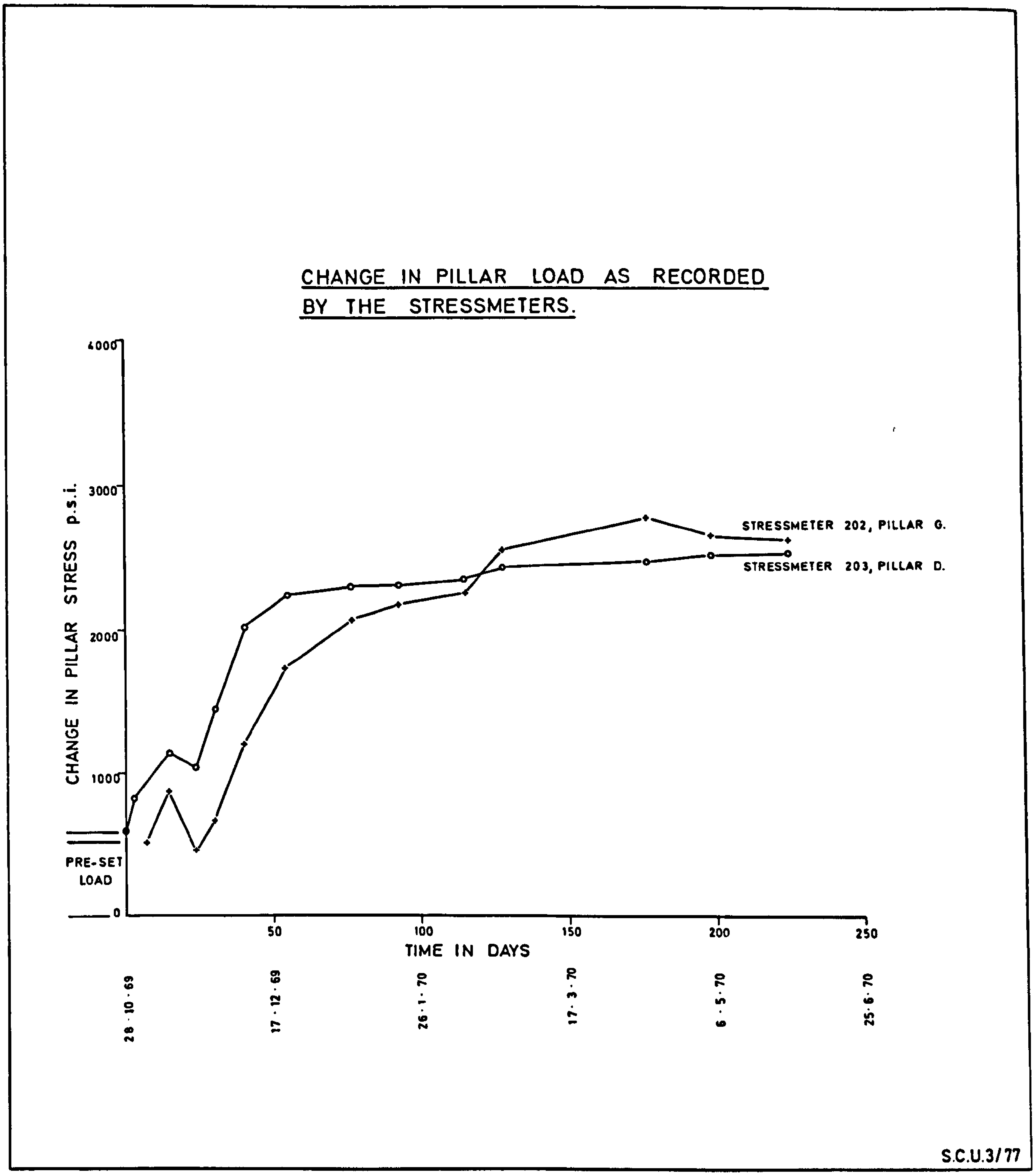

CHANGE IN PILLAR LOAD AS RECORDED BY THE STRESSMETERS.
CHANGE IN PILLAR STRESS p.s.i.
4000
3000
2000
1000
0
PRE-SET LOAD
50
100
150
200
250
TIME IN DAYS
28 · 10 · 69
17 · 12 · 69
26 · 1 · 70
17 · 3 · 70
6 · 5 · 70
25 · 6 · 70
STRESSMETER 202, PILLAR G.
STRESSMETER 203, PILLAR D.
S.C.U.3/77

FIG. 76

adjacent to the roadway being formed and thus more quickly affected by load transfer. The reason for this reduction in the output of both stressmeters was not readily apparent, but a similar phenomena was reported by Johnson (82) in conjunction with the measurement and movement of the front abutment load on a longwall face.

As mining proceeded, both stressmeters showed an immediate rise in load and again stressmeter 203 was more pronounced. This was understandable since the small pillar D was in the process of being formed, and was finally isolated in early December 1969. By 8.12.69, there had been an increase in the load acting on pillar D as recorded by stressmeter 203, of 1425 p.s.i. On the same date stressmeter 202 had recorded an increase in load of 674 p.s.i. even though the extraction of this large pillar had not commenced.

Following this, the output of both stressmeters continued to increase, but by now stressmeter 202 was increasing at the higher rate as extraction of the other large pillar proceeded. On 14.1.70 extraction in this area was temporarily suspended, by which time the output from stressmeter 203 was virtually constant, whilst stressmeter 202 in the partly formed pillar G was still increasing, having recorded an increase in load of 1549 p.s.i. since installation. The extent of the extraction at this time is shown in Fig.74.

Working in this area recommenced in February 1970 and by 5.3.70 pillar G was completely isolated, this being indicated by a slight increase in the output of both stressmeters at this time, being more pronounced in stressmeter 202. In the following weeks up to 5.6.70, during which time 'roofing' was carried out around the instrumented pillars and extraction proceeded on the adjoining large pillars to the South, the output from stressmeter 203 remained virtually constant whereas stressmeter 202 initially continued to show a slight increase in load followed by a slight reduction in output. At the time of the final measurement on 5.6.70, when extraction around the instrumented pillars had been completed, stressmeter 203 in pillar D had recorded a total increase in load of 1920 p.s.i., whereas stressmeter 202 in pillar G had recorded a total increase in load of 2093 p.s.i.

It will be noticed from the extraction plan Fig.75, that only pillars E and G were completely formed from one of the large pillars. This was due to the fact mentioned previously, that roadway 5 North was situated at a much lower horizon than the other roadways, and it was thought too dangerous for the machinery to form a roadway between the proposed pillars F and H.

The stressmeter results are both demonstrative and qualitative in illustrating the build up in load on that part of the large pillars that eventually formed the small pillars D and G. In considering these results it would appear reasonable to assume that the load originally carried by the large pillars would be re-distributed onto the resulting small pillars. The approximate theoretical load acting on the large pillars prior to the 'splitting' of these pillars can be calculated using equation (2) given in Part I, Section 3.1 of this thesis. For a cover load, dependent solely on the depth of cover, of 700 p.s.i., this suggests a pillar load of 1094 p.s.i. If the same equation is now used for determining the load acting on the small pillars, this suggests a pillar load after the large pillars were 'split' of 1944 p.s.i. This gives a theoretical re-distribution of the load as a result of mining of only 850 p.s.i., much lower than the change in load recorded by both stressmeters. Both stressmeters in fact, recorded an increase in load in close agreement with the theoretical load acting on the small pillars.

In view of the almost identical behaviour of the two stressmeters and the similar change in load recorded, it is thought unlikely that the sensitivity of the stressmeters could have been changed by the conditions inside the borehole. This section of the mine was completely dry so that possible errors introduced by a difference between a dry calibration block in the laboratory and an underground borehole containing water, were extremely unlikely.

A fact that should be considered is that the small pillars, D and G, were originally part of the outside section of very large pillars, which were sustaining comparatively low theoretical loads in proportion to their size. The stressmeters were installed at a lateral depth of 12 ft. into each of the large pillars as mentioned previously, at a distance of 20 ft. from the pillar centre. It is suggested therefore, that the section of the large pillar into which the stressmeter was inserted in each case was carrying little, if any, of the total load acting on the pillar due to the depth of overburden, this load being carried by a central load bearing core in each of the large pillars. If this was so, and it would seem a plausible explanation under the circumstances, the increase in load recorded by both stressmeters is a measure of the actual or absolute stress level acting on the resulting small pillars.

The fact that both stressmeters continue to function and show little or no change, indicates that the re-distribution of the load due to mining has been completed. There was in fact, a very close relationship between the complete isolation of each of the pillars and the time the corresponding stressmeter outputs became constant, there being little, if any, increase in pillar load once the pillars were formed.

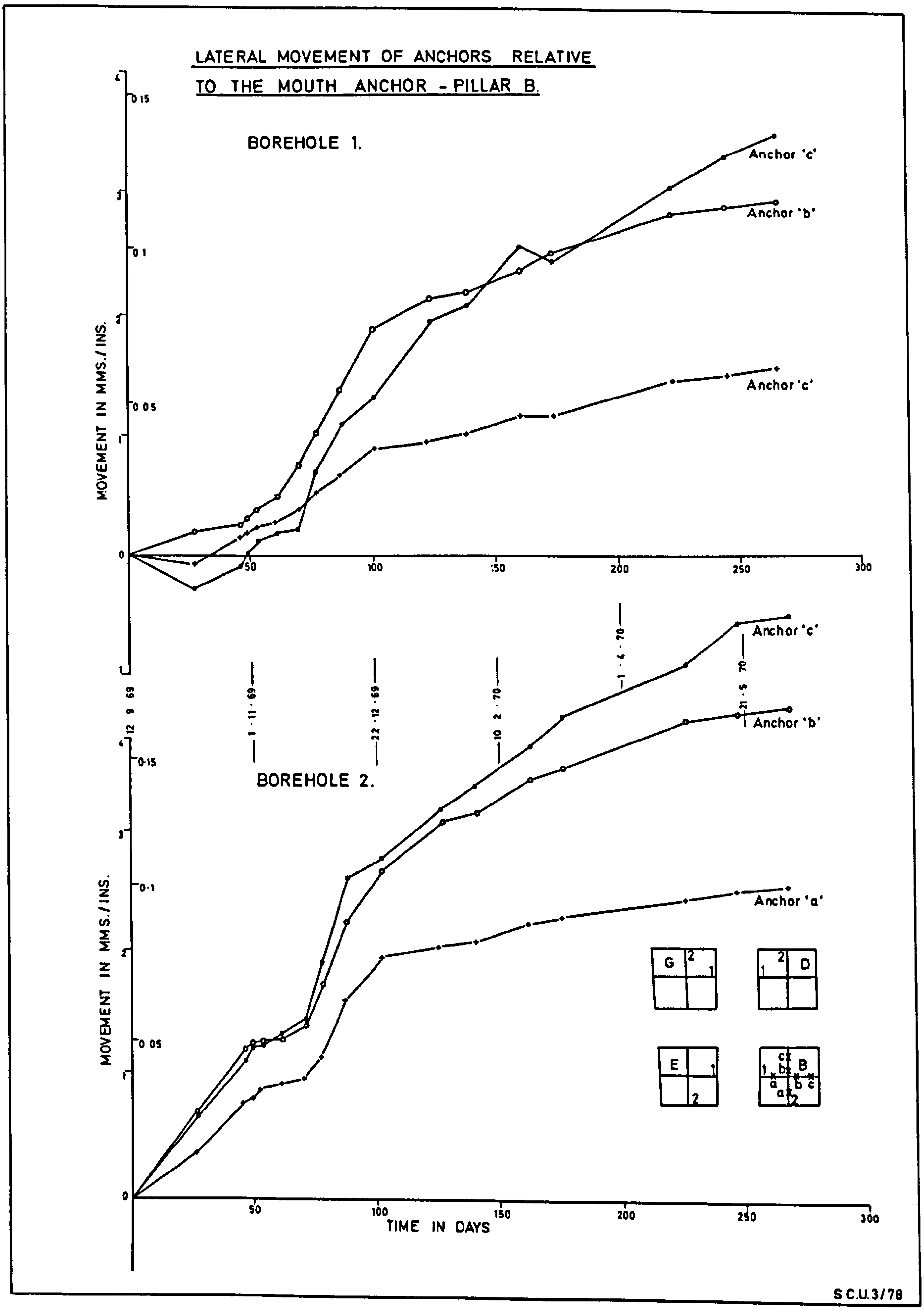
11.2 Pillar Lateral Deformation Measurements.

The results of the pillar lateral deformation measurements are presented in a similar manner to those obtained from the Sherburn mine described previously. The lateral movement of each anchor relative to the 'fixed' position of the mouth anchor is shown for each borehole, together with the bay deformations and corresponding strain. The deformation is presented in both mms. and ins. and the convention of increased distance (extension) being positive and reduced distance (contraction) being negative, used throughout. In order to provide some direct comparison between the results obtained from two perpendicular boreholes situated in the same pillar, the total lateral movement of each anchor relative to the mouth anchor for borehole 1 is presented in the same diagram as the same measurements for borehole 2 in the same pillar.

PILLAR B - Boreholes B1 and B2.

Pillar B was the first pillar formed. The total lateral movement of each anchor in both boreholes is shown in Fig.77, and the total bay deformation and corresponding strains shown in Figs. 78 and 79, for borehole B1 and B2 respectively. The total movement of each anchor relative to the mouth anchor in B1 is seen to be extensional, showing values of 1.56 mm. (0.062 in.), 2.94 mm. (0.115 in.) and 3.50 mm. (0.138 in.), and for B2 is also seen to be extensional, having values of 2.62 mm. (0.102 in.), 4.16 mm.(0.163 in.) and 4.99 mm. (0.195 in.) for anchors 'a', 'b' and 'c' respectively in each case.

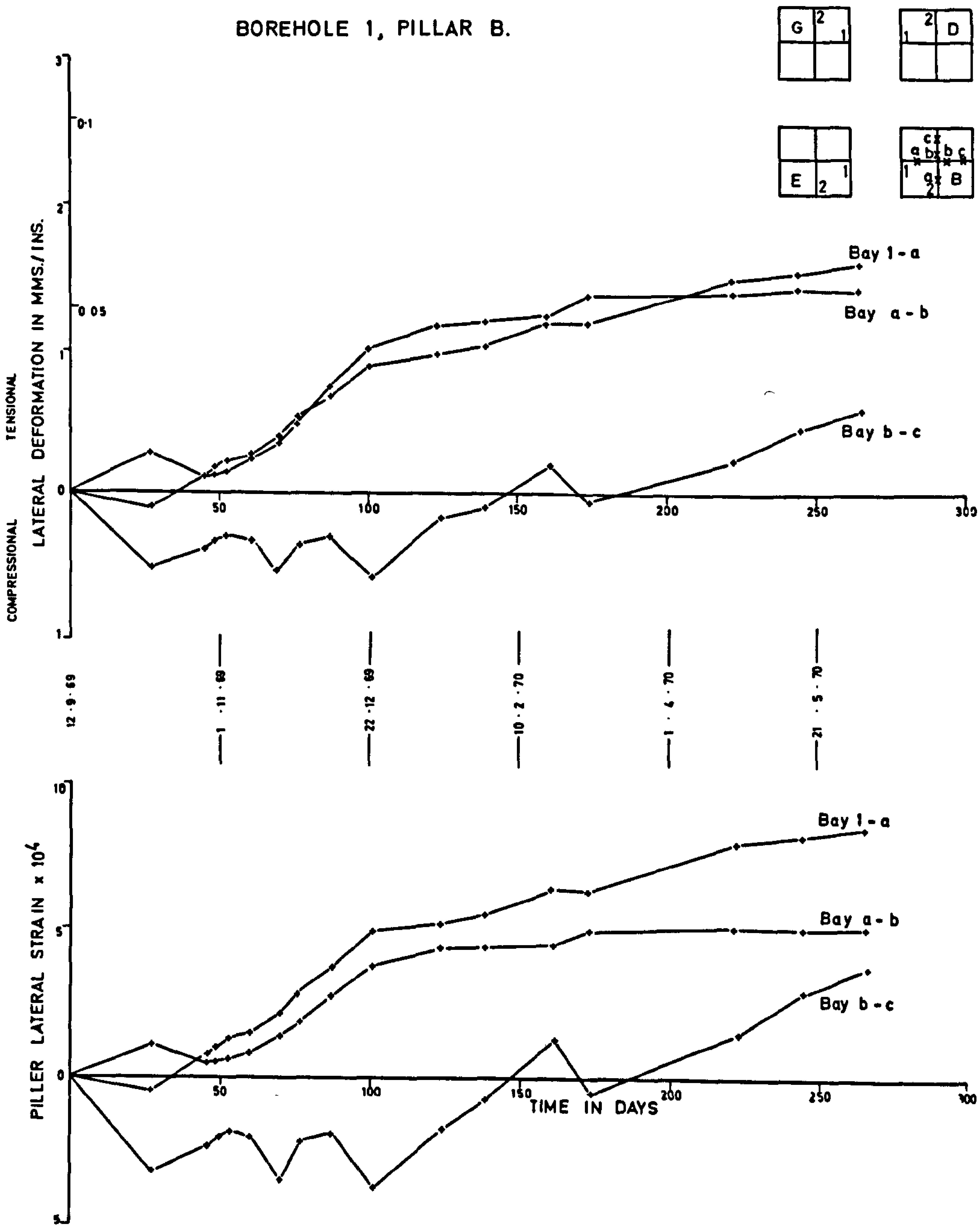
It will be noticed that in the period of approximately 40 days following instrumentation up to the day work began on forming pillar B, the movement of each anchor recorded in B2 was considerably higher than the movement of the corresponding anchors in B1. When work began on forming pillar B on approximately day 40, 22.10.69, there was an immediate change in the deformations occurring in B1, but this was not so apparent in B2. This may be explained by the fact that roadway 2 North was formed first, thus forming the pillar side through which B1 passed. This pillar was completely formed by approximately day 80, 1.12.69, but there was no apparent decrease in the deformation of the pillar. This was due to the fact that the height of roadway 2 North was being increased by 'roofing' and by day 101, 22.12.69, approximately half of the sides of the



S.C.U.3/78

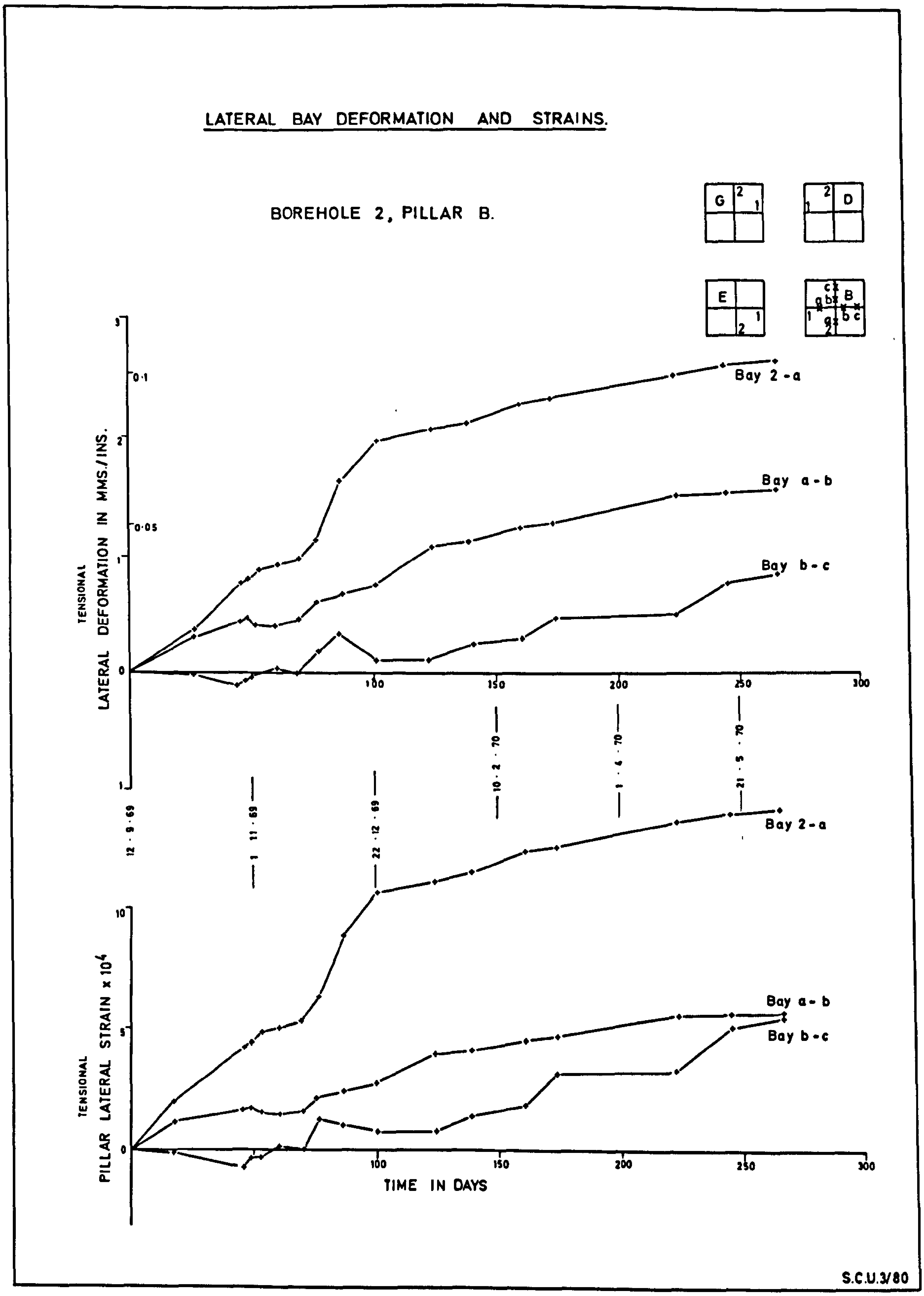
FIG. 77

LATERAL BAY DEFORMATION AND STRAINS.



S.C.U.3/79

FIG. 78



S.C.U.3/80

FIG. 79

completely formed pillar B had been increased to a height of 25 ft. It was at approximately this time that 'roofing' operations around this pillar temporarily ceased, and this is indicated in the bay deformations recorded in both boreholes in the following period of approximately 80 days up to the middle of March 1970, when 'roofing' was completed around this pillar.

Following the completion of extraction around this pillar and up to the time the final measurement was taken on 5.6.70, the deformation of each bay increased at a steadily decreasing rate with the notable exception in each borehole of bay b - c. At the time of the final measurement, the deformation occurring in bay a - b in both boreholes had virtually ceased, whilst bay b - c was becoming increasingly extensional. The deformation occurring in the outer bay lengths in each borehole, bays 1 - a and 2 - a was still increasing, though apparently at a decreasing rate.

PILLAR D - Boreholes D1 and D2.

Pillar D was the pillar formed immediately after pillar B. The total lateral movement of the anchors in each borehole is shown in Fig.80, and the total bay deformations and corresponding strains shown in Figs. 81 and 82 for borehole 1 and 2 respectively. The total movement of each anchor relative to the mouth anchor is seen to be extensional for both boreholes, having values of 4.32 mm. (0.169 in.), 5.43 mm. (0.214 in.) and 7.0 mm. (0.275 in.) for D1, and values of 0.98 mm. (0.039 in.), 2.74 mm. (0.107 in.) and 3.45 mm. (0.136 in.) for D2, for anchors 'a', 'b' and 'c' respectively.

In the period of time between day 40, 22.10.69, and day 80, 1.12.69, when pillar D was in the process of being formed, there were quite considerable increases in the movement of the anchors situated in D1, but relatively little in D2, though this may be partly explained by the fact that roadway 2 North was formed before pillars B and D were 'split'. When 'roofing' operations reached

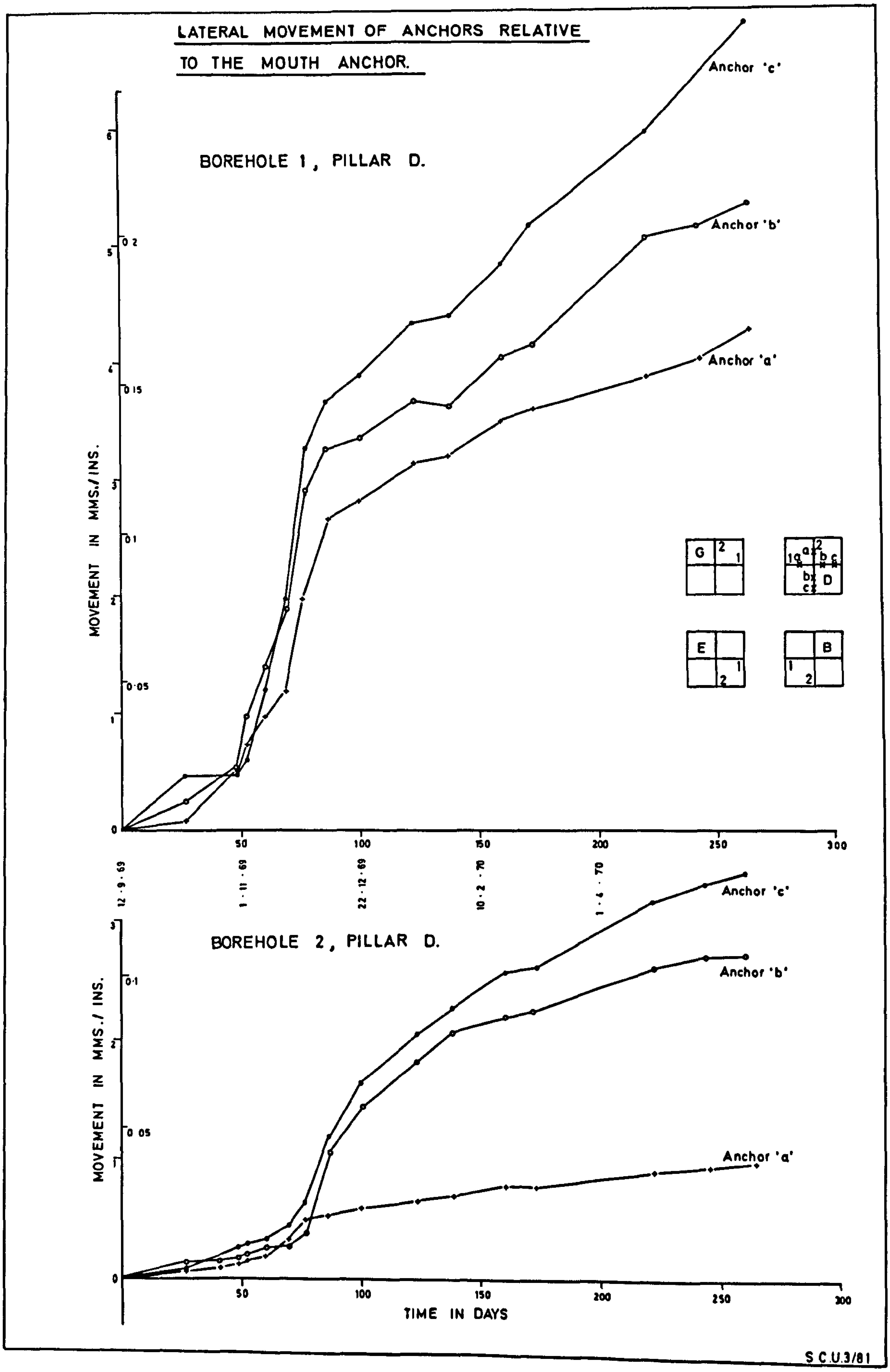
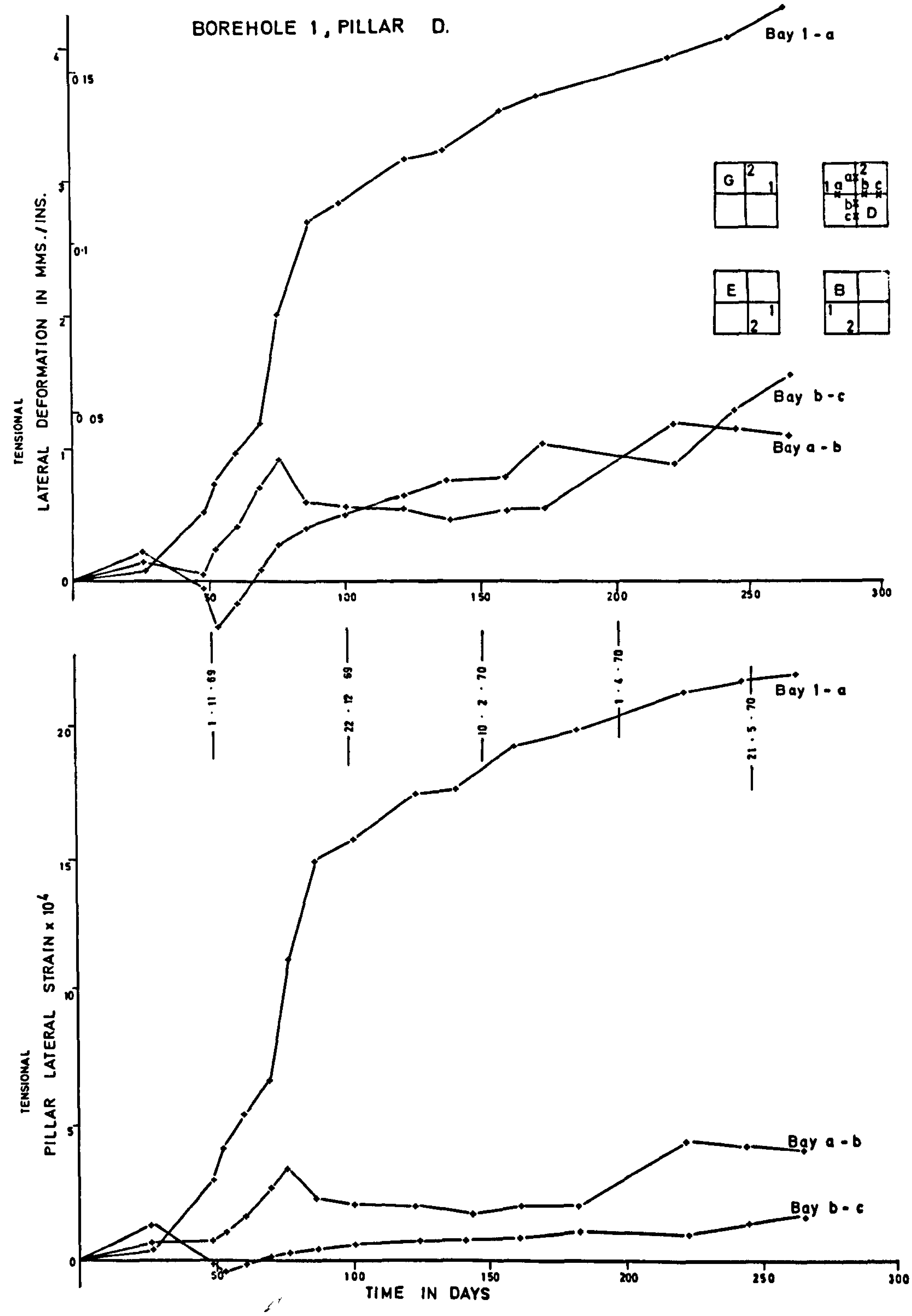

LATERAL MOVEMENT OF ANCHORS RELATIVE TO THE MOUTH ANCHOR.
BOREHOLE 1, PILLAR D.
Anchor 'c'
Anchor 'b'
Anchor 'a'
MOVEMENT IN MMS./INS.
12 · 9 · 69
1 · 11 · 69
22 · 12 · 69
10 · 2 · 70
1 · 4 · 70
BOREHOLE 2, PILLAR D.
Anchor 'c'
Anchor 'b'
Anchor 'a'
MOVEMENT IN MMS./INS.
TIME IN DAYS
S.C.U.3/81

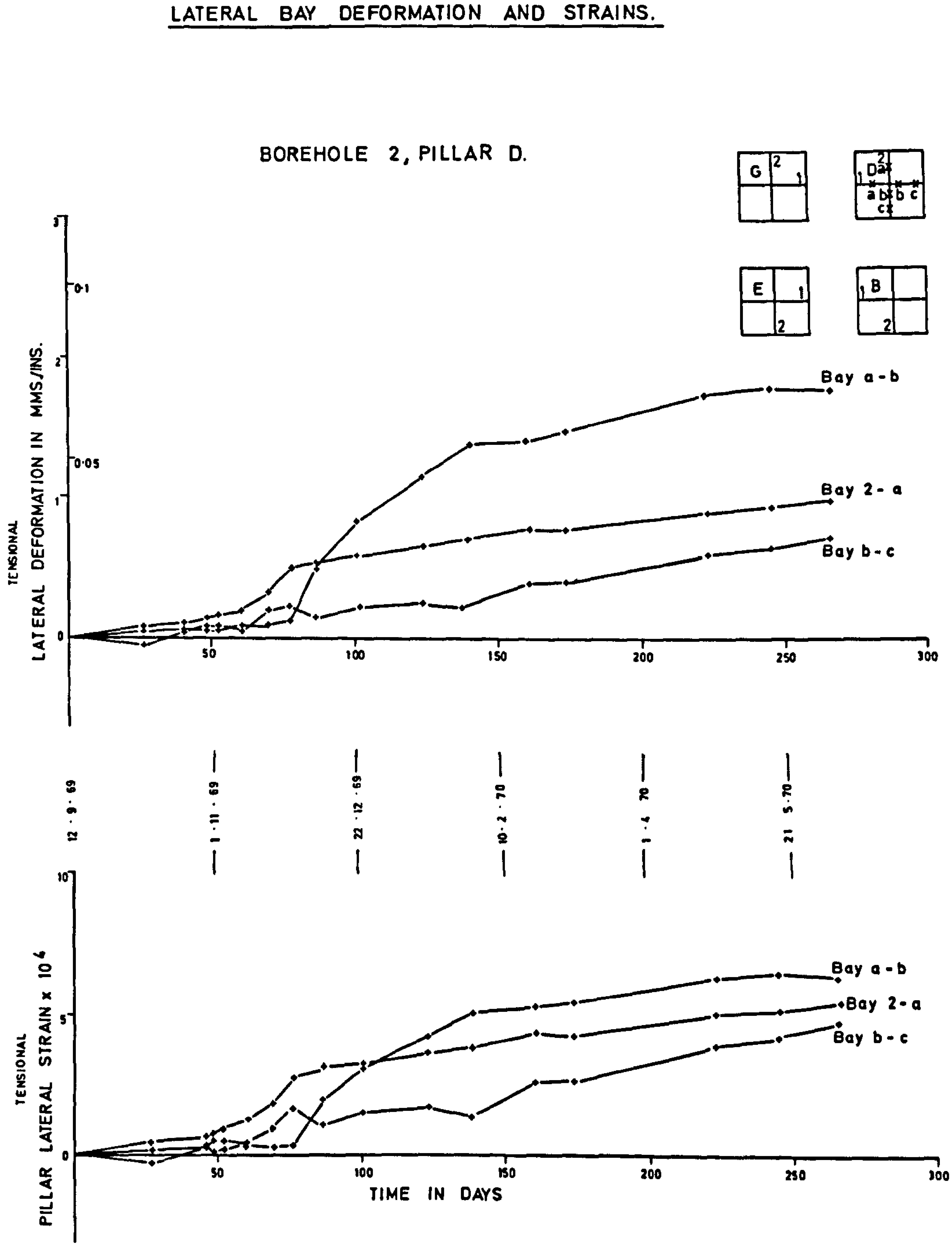
Fig. 80

LATERAL BAY DEFORMATION AND STRAINS.



SC.U3/82

FIG. 81



SC.U.3/83

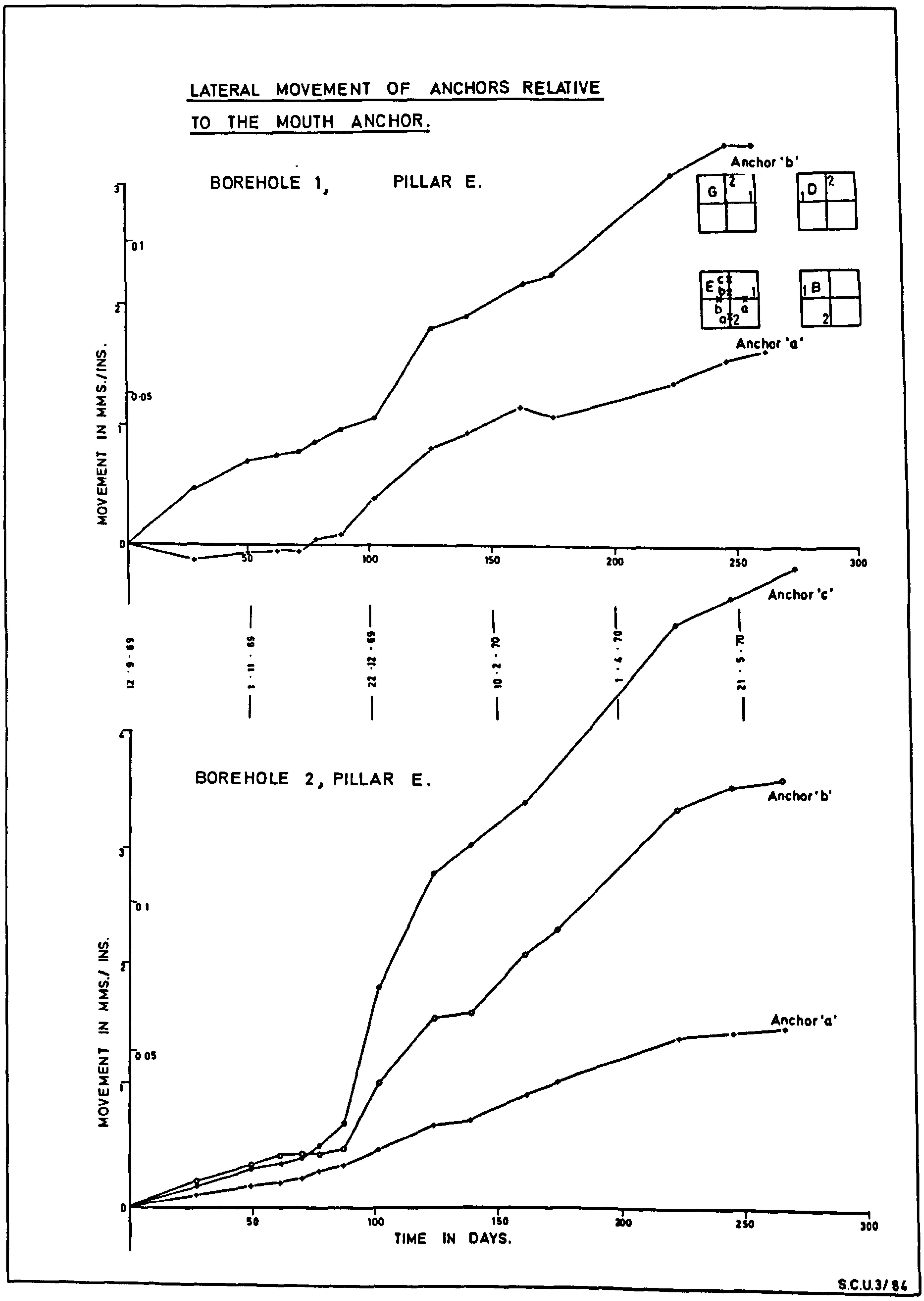
FIG. 82

pillar D on approximately day 101, 22.12.69, work concerned with this operation ceased and did not recommence until approximately day 174, 5.3.70. It will be noticed that over this period of time when no extraction was taking place immediately around pillar D, the bay deformation rates in D1 decreased slightly, whilst bay deformation rates in D2 also decreased slightly, though in this case bay a - b only decreased over the period of time between days 140 - 174.

When 'roofing' operations resumed there was an immediate increase in the deformation, this being most noticeable in the movement of the anchors in D1. This pillar was completely 'roofed' to a height of approximately 25 ft. by day 223, 23.4.70. At the time the final measurement was taken, there was no definite sign that bay deformations were becoming constant, though there were indications that bay a - b in both boreholes was tending towards this. In borehole D1, bay 1 - a was steadily increasing as was the bay b - c. In borehole D2, the deformation of bay 2 - a was also increasing at a constant rate as was bay b - c.

PILLAR E - Boreholes E1 and E2.

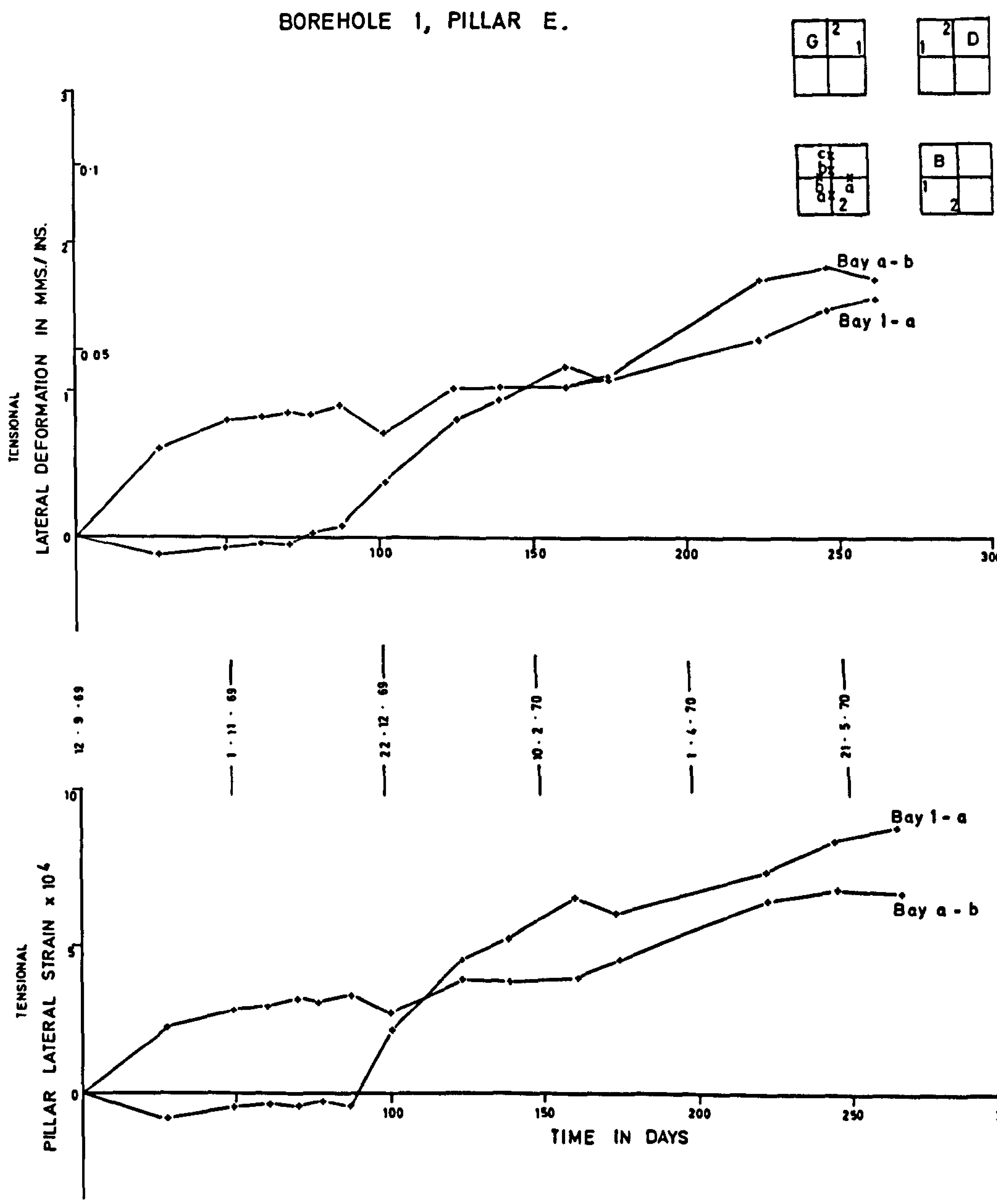
Pillar E was the third pillar formed. The total lateral movement of the anchors in each borehole is shown in Fig.83, and the total bay deformation and corresponding strains shown in Figs. 84 and 85 for boreholes E1 and E2 respectively. The total movement of each anchor relative to the mouth anchor is seen to be extensional for both boreholes, having values of 1.60 mm. (0.063 in.) and 3.45 mm. (0.136 in.) for anchors 'a' and 'b' respectively in E1, and 1.50 mm. (0.059 in.), 3.64 mm. (0.103 in.) and 5.40 mm. (0.212 in.) for anchors 'a', 'b' and 'c' respectively in E2. The results from anchor 'c' in borehole E1 have not been used since it was thought that the wire from this anchor became entangled with anchor 'b' during installation.



S.C.U.3/84

FIG. 83

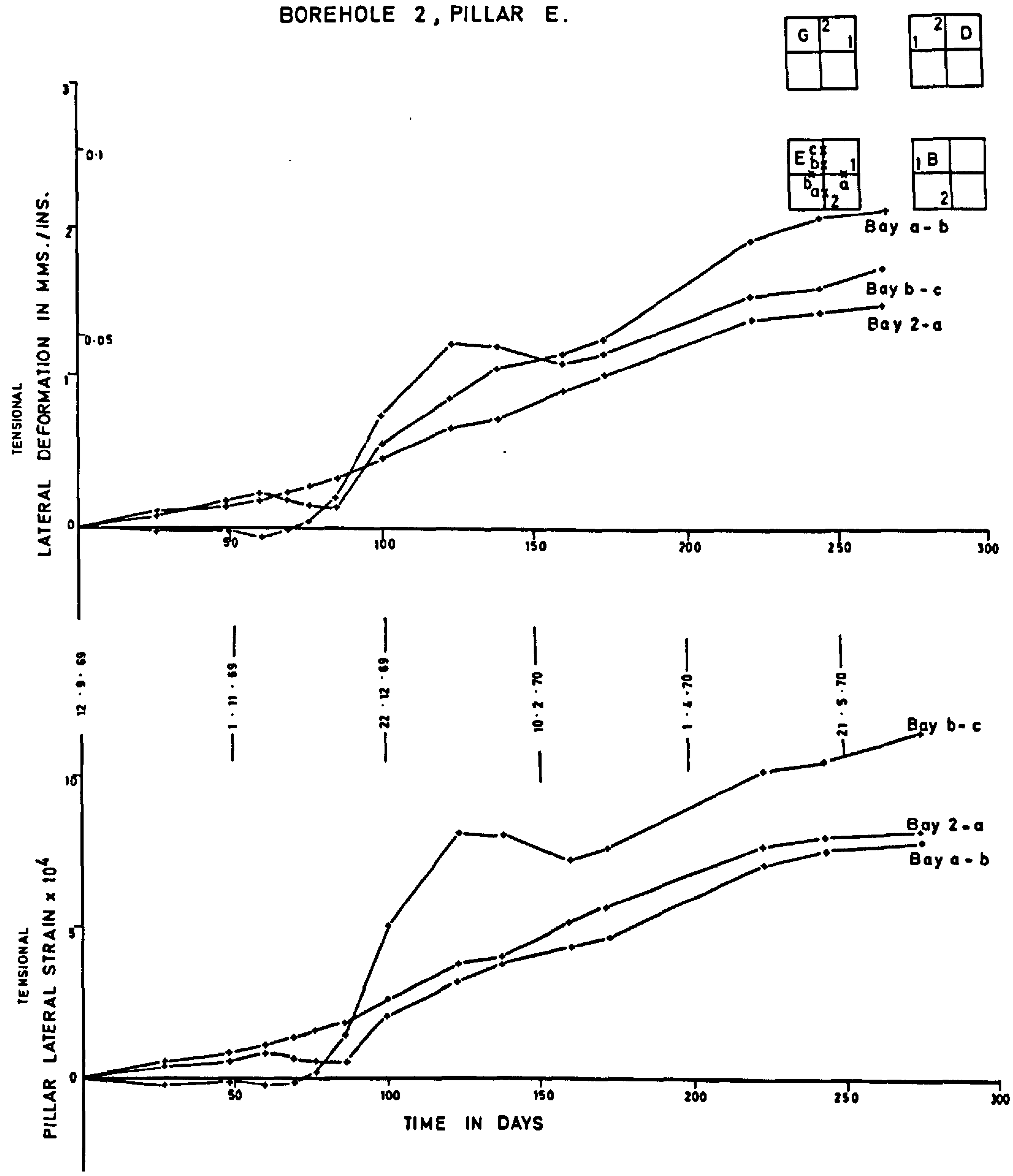
LATERAL BAY DEFORMATION AND STRAINS.



S.C.U 3/85

FIG. 84

LATERAL BAY DEFORMATION AND STRAINS



SC.U3/86

FIG. 85

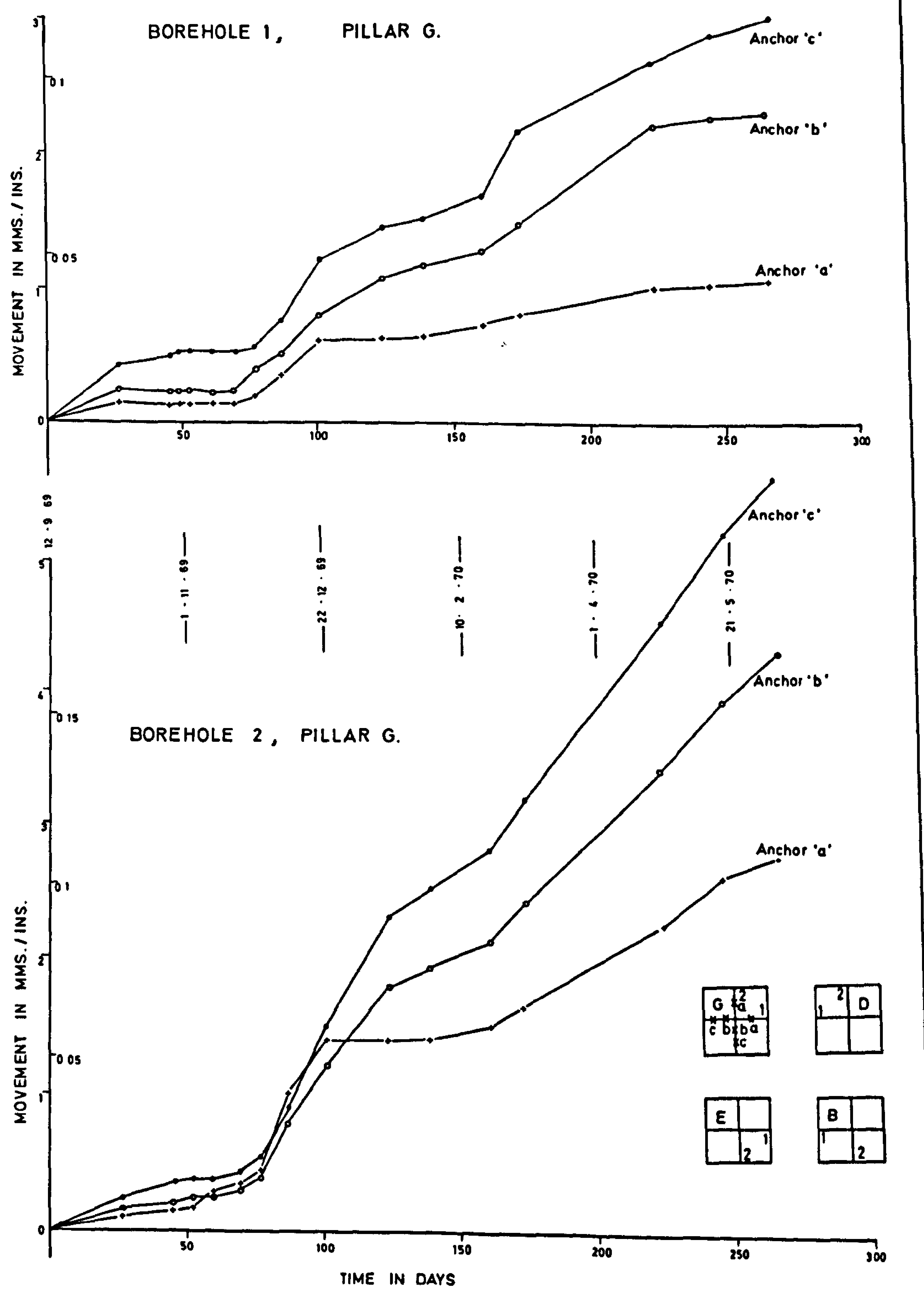
Work commenced on the formation of this pillar on day 87, 8.12.69, this being indicated by the deformation occurring in the respective measuring bays in both boreholes up to this time. However, before the pillar was completely formed, work was temporarily suspended and the pillar was not isolated completely until early February 1970 on approximately day 150. Almost immediately following its complete isolation the roadways surrounding the pillar were 'roofed' and this was completed by approximately day 190. The deformation occurring in the measuring bays in each borehole continued to increase following this. At the time the final measurement was taken there were indications that the deformation occurring in bay a - b, in borehole E1 and bay a - b,in borehole E2 was becoming constant, but the deformation of the remainder of the measuring bays was still increasing.

PILLAR G - Boreholes G1 and G2.

This was the final pillar to be formed. The total lateral movement of the anchors in each borehole is shown in Fig.86, and the total bay deformations and corresponding strains shown in Figs. 87 and 88 for boreholes G1 and G2 respectively. The total movement of each anchor is seen to be extensional for both boreholes, having values of 1.06 mm. (0.041 in.), 2.30 mm. (0.091 in.) and 3.03 mm. (0.119 in.) for D1, and 2.78 mm. (0.109 in.), 4.30 mm. (0.169 in.) and 5.62 mm. (0.220 in.) in D2, for anchors 'a', 'b' and 'c' respectively.

As for pillar E, work commenced on the formation of pillar G on day 87, 8.12.69, but was temporarily suspended between approximately days 120 and 150. This is indicated by a definite 'step' in the lateral movement of the anchors shown in Fig.86. When work resumed on forming this pillar there was an increase in the deformation rate occurring in both boreholes. The pillar was completely isolated by day 161, 20.2.70, and the surrounding roadways 'roofed'

LATERAL MOVEMENT OF ANCHORS RELATIVE TO THE MOUTH ANCHOR.

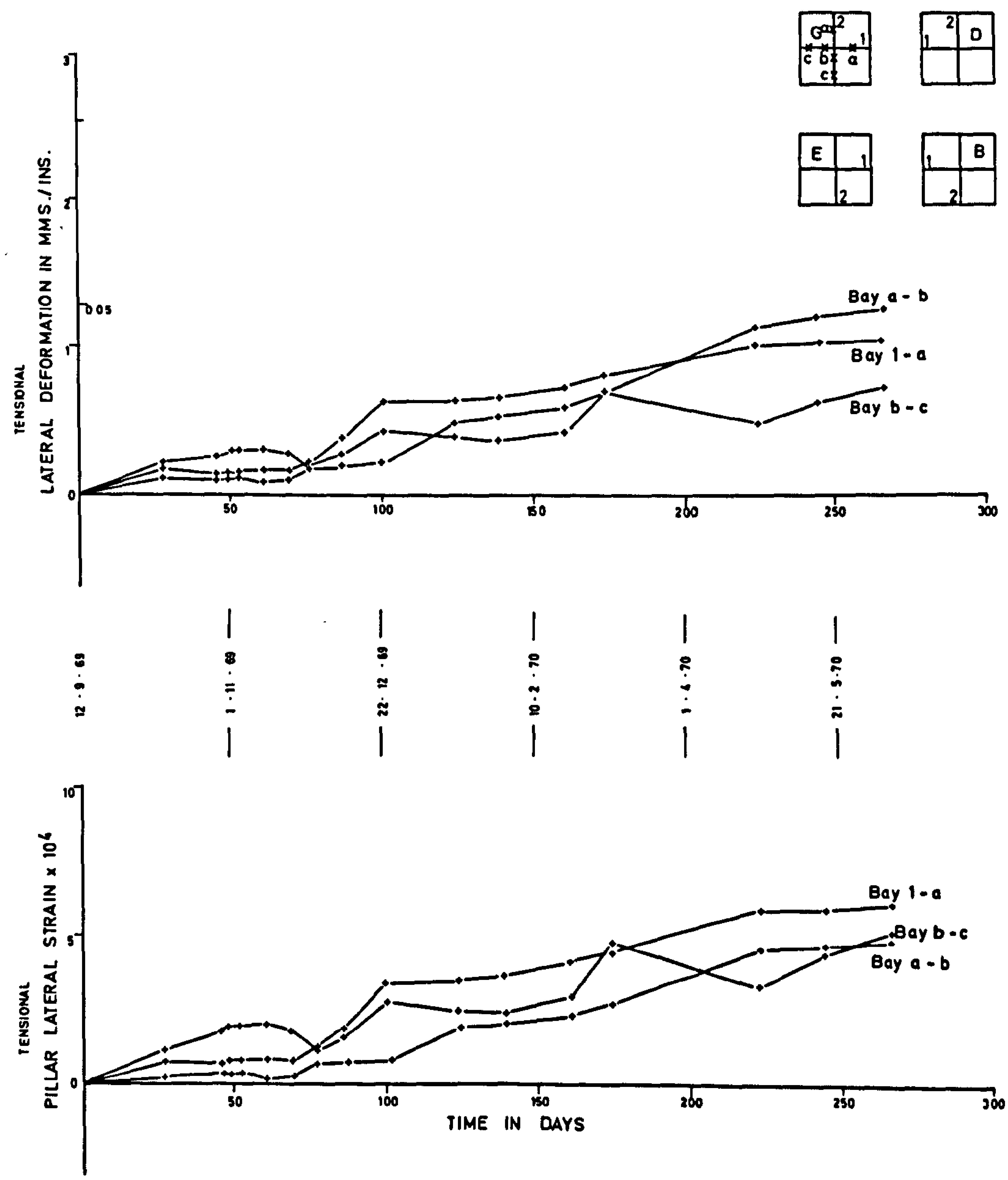


SCU3/87

FIG. 86

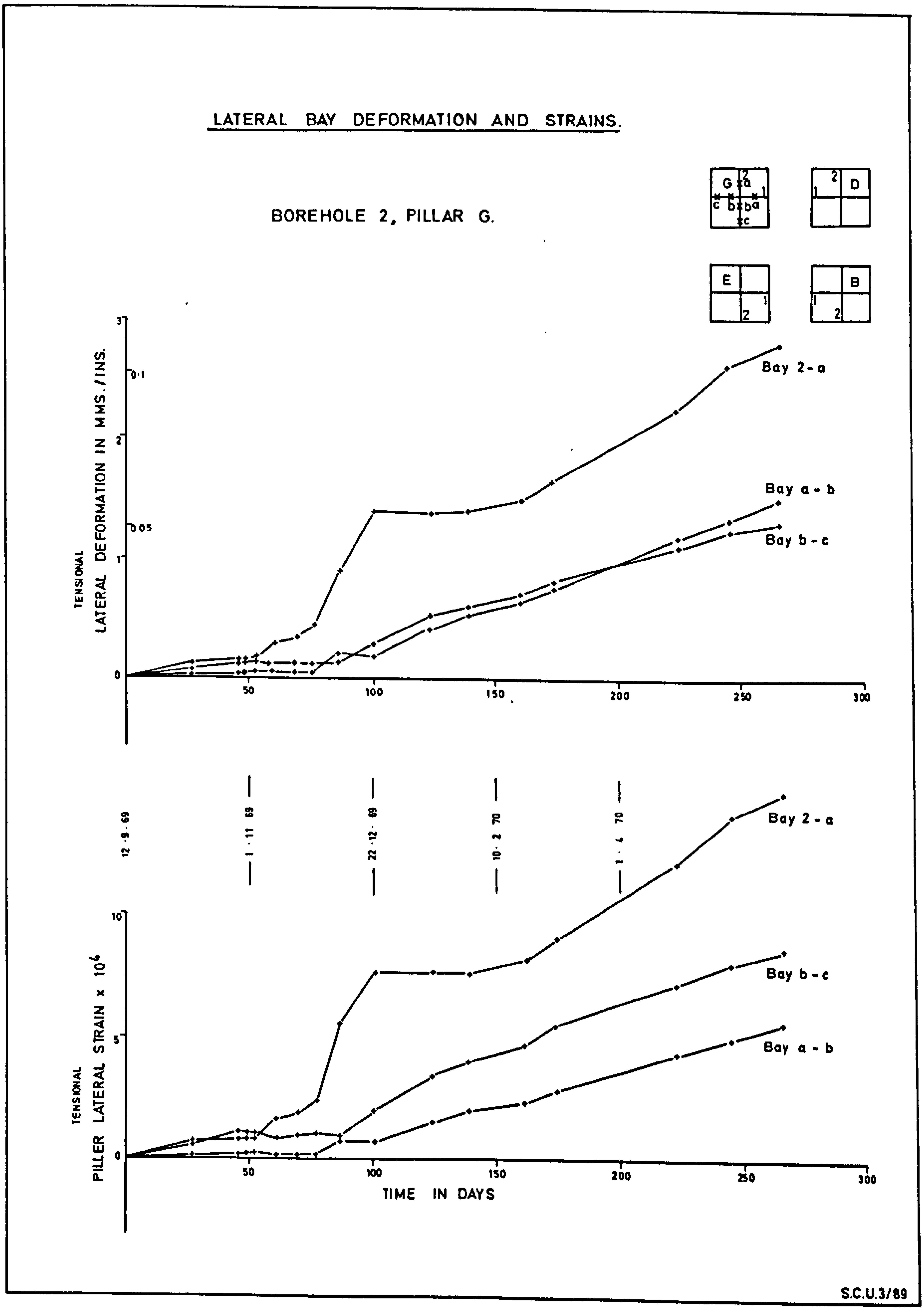
LATERAL BAY DEFORMATION AND STRAINS.

BOREHOLE 1, PILLAR G.



S.C.U.3/88

FIG. 87



S.C.U.3/89

FIG. 88

by day 223, 23.4.70. At the time the final measurement was taken on 5.6.70, the deformation of each bay was still increasing, this being most pronounced in borehole G2.

On considering the descriptions of the individual pillar deformations, a number of significant factors became evident :-

(i) The pillars were subject to a lateral deformation which took the form of an extension.

(ii) The overall magnitude of the lateral deformation of each of the pillars was fairly significant.

(iii) The lateral deformation was not confined to an outer edge zone of the pillars.

(iv) The lateral deformation continued to increase after pillar isolation as a result of 'roofing' operations.

The pillar lateral deformation was obviously related to the changing load conditions as support was removed and load transferred. There was a very close correlation between the increase in load acting on the pillars D and G as recorded by the stressmeters, Fig.76, and the lateral deformation of these two pillars as shown in Figs.80 and 86. It may also be noticed that with the exception of pillar D, the lateral deformation of the pillars was greater in the direction in which the workings advanced, i.e. from North-South, than in the direction perpendicular to this advance. The notable exception of pillar D to this fact, and the magnitude of the deformation recorded in borehole D1 in particular, was so pronounced that some consideration should be given to the possibility that at some time the mouth reference anchor may have been displaced slightly. This difference in the magnitude of the pillar deformation with direction of advance, may possibly be related to the occurrence of a front abutment load which moved forward as the workings advanced from one pair of large pillars to another.

As previously mentioned, the pillars were divided laterally into three sections of different lengths by the borehole anchors, and the deformation occurring in these three sections can be compared by means of the calculated strains. These indicate that for the most part, the induced strain was only slightly higher in the outer section of the pillars than in the middle sections formed by anchors 'a', 'b' and 'c'. The effect of this can best be illustrated by comparing these results with similar results obtained for the Sherburn mine. At Sherburn, only the extreme outer sections of the pillars were affected by mining, whereas at Stamphill with much smaller pillars, almost the whole pillar was affected by the mining.

There was a definite effect recorded by increasing the height of the pillars, quite large extensional movements being recorded, this not being entirely confined to the outer sections of the pillars. Following this 'roofing' operation, which decreased the W/H ratio of the pillars from approximately 2 - 1, the deformation of most of the different pillar sections continued to increase, though at the time of the final measurement, the deformation of bay a - b in each borehole in most of the pillars, representing the central core of the pillar, had virtually ceased. The deformations occurring in the other two bay lengths however, continued to increase in most cases after completion of work around these pillars, indicating that this continuing lateral deformation occurred in a fairly wide outer section of the pillars.

12. CONCLUSIONS AND RECOMMENDATIONS RELATING TO THE STAMPHILL MINE.

The laboratory investigations carried out by Jones (5), indicated that the rock material making up the economic zone of the 'A' Bed gypsum did not lend itself to the determination of a representative value for its compressive strength, and for this reason, specific conclusions relating to the stability of the mine pillars based mainly on such a value are not possible. However, a mean compressive strength value of

3146 p.s.i. was obtained, and over the small volume range covered by the laboratory testing, there appeared to be a limiting ultimate compressive strength for the seam material of approximately 2400 p.s.i. At a depth of 900 ft., approximately the maximum depth of the present workings, the theoretical pillar load is 2500 p.s.i., a value less than the mean compressive strength of the seam material but slightly greater than the limiting compressive strength value.

It may be suggested that the load conditions which exist within the mine pillars are most probably much closer to compressive triaxial conditions, and it is quite possible that the effective limiting compressive strength of the seam material forming the pillars is somewhat greater than the laboratory determined value of 2400 p.s.i. This supposition is based on the fact that the pillar material when constrained will undergo an increase in strength. This phenomena is displayed in the laboratory in the testing of specimens of varying W/H ratio where parts of the specimens undergo compressive triaxial loading, and it is logical to assume that the same effect occurs within the mine. The W/H ratio of the laboratory specimens used to establish the compressive strength of the seam material was 0.5, whereas the W/H ratio of the instrumented pillars in particular, varied between 1 and 2, ratios which should produce an increase in the compressive strength of the material forming the central core of the pillars when compared with the laboratory specimens. The deformation measuring instrumentation scheme installed in pillars at a depth of 700 ft. from the surface, provided an indication of the magnitude and distribution of this constraint within the pillars.

Each pillar was divided into three sections by the borehole anchors and it was evident that during the formation of the pillars, there was no indication of any centre section of the pillar which was under complete constraint. The re-distribution of the load due to mining resulted in an extensional deformation of all three bay lengths. In some cases, the deformation per unit length of the centre bay a - b, was comparable with the deformation per unit length of the outer bay lengths. However, at the time of the final measurement, there were indications that the change in deformation of the centre bay length a - b, in some pillars, was tending to become constant. This suggested that there could be a central core of the pillar, approximately 6 ft.wide, which was undergoing some measure of constraint. Even if this is so, and further measurements could confirm this, the actual width of the constrained section of the pillars in relation to the width and height of the pillars, is not very great, and it is unlikely that this could lead to any significant increase in the effective limiting compressive strength of the seam material.

The effect on pillar stability of increasing the height of the pillars and thereby decreasing the W/H ratio, is not immediately obvious from the deformation measurements due to the temporary suspension of mining around the instrumented pillars, which coincided with the time when most of these pillars were about to undergo 'roofing' operations. However, it may be said that there was an increase in the deformation rate when these operations were being carried out, and it is suggested that increasing the height of the pillars contributed to some considerable extent to the fact that the lateral deformation of the pillars has continued to increase following their deformation.

It would seem therefore, that the actual stability of the existing pillars in the deeper sections of the Stamphill mine is something to be viewed with some concern. The theoretical load acting on the pillars must be very close to the actual limiting compressive strength of the seam material forming the pillars in the deepest section of the mine. Also, the variation in the W/H ratio of the pillars apparently results in the formation of unstable pillars at a depth of 700 ft. With regard to this latter factor, it should be noted that due to the need to extract that section of the seam where the gypsum content is sufficiently high, it has resulted in the formation of pillars which are much higher than the instrumented pillars. In some instances, this has resulted in the formation of pillars which are as much as 50 ft. high, if not higher, though their cross-section remains the same throughout. It may be concluded therefore, that those workings below a depth of approximately 700 ft. from the surface, may be regarded as being of an unstable nature, and it is possible that pillars at a lesser depth than this are also unstable.

Measurements should continue on these instrumented pillars as a control function for the monitoring of further pillar lateral deformation, thus ultimately defining the actual state of stability of the pillars. However, before precise data can be provided for the design of future stable workings, further investigations should be carried out to determine a more reliable compressive strength value, together with the effect on pillar strength of the variation in W/H ratio. In view of the laminated nature of the seam material, it is felt that the only way of obtaining a representative compressive strength value would be by means of a programme of 'in situ' testing of large specimens, similar to that described by Bieniawski (4). Laboratory investigations could also be carried out, but should be inclined towards the determination of the effect of the laminated structure on the load bearing capacity of the pillars. Only after further investigations of this type it may be possible to provide design data that may be used for the practical and efficient design of future workings, possibly in the form of design nomographs of the type described in Part 1, Section 3.4 of this thesis.

C. THE NEWBIGGIN MINE.

13. DESCRIPTION OF THE GENERAL CONDITIONS AT THE NEWBIGGIN MINE.

The Newbiggin gypsum mine, like the Stamphill mine, is situated near the village of Kirkby Thore in Westmorland, though it is sited in a higher stratal horizon than the Stamphill mine. Whereas at the Stamphill mine, production is now almost entirely confined to the 'A' Bed, the 'B' Bed reserves in the mine being virtually exhausted, production at the Newbiggin mine is confined entirely to the 'B' Bed gypsum. The 'B' Bed, of total thickness of approximately 12 ft., is composed of varying thicknesses of gypsum and anhydrite depending on its position in relation to the outcrop. The general dip of the strata is about 1 in. 8 to the East.

Production from the Newbiggin mine was originally confined to the quarrying of 'B' Bed, but this was supplemented by underground mining of the same bed when the quarry face reached a point beyond which the thickness of overburden became too great for large scale economic quarrying. A room and pillar method of mining is used with 16 ft. wide roadways and 21 ft. square pillars. A 10 ft. seam section is removed in this way, leaving about 2 ft. of the seam to form the immediate roof. Only a relatively small area of 'B' Bed, within 500 ft. of the outcrop being quarried, has been extracted to date.

The workable reserves of this high quality 'B' Bed gypsum however, are dwindling, and attention is being turned to the possible underground mining of a previously unworked seam of the gypsum at Newbiggin, referred to as the 'C' Bed. This gypsum bed lies some 12 - 14 ft. above the 'B' Bed. A section of this bed has been revealed by a short development drift driven from the 'B' Bed workings. This showed the 'C' Bed at this point to be made of approximately 2.1/2 ft. of anhydrite at the base overlain by 5 ft. of gypsum. An extensive surface borehole survey carried out previously however, indicated that this was not representative of the bed as a whole.

The inter-seam strata consists of fairly soft red marl with gypsum spars, and similar material is to be found from immediately above the seam to the surface, between 80 and 100 ft. above. It is expected that the nature of strata overlying the 'C' Bed could materially affect the possible working of this bed. In addition to this, it is expected that mining could be further complicated by the necessity to superimpose the pillars in the two seams, by the probable requirement that gypsum be left to form the immediate roof and floor of the workings, and by the presence of water which is already in evidence in the initial development drift.

As soon as a preliminary reconnaissance had been carried out at the Newbiggin mine it became apparent that the inherent mechanical properties of the 'C' Bed gypsum and the surrounding strata could possibly limit the permissible mining dimensions to some considerable extent. For this reason, laboratory investigations were instituted in order to determine the values of those mechanical properties which, it was felt, would have particular relevance as far as their effect on the feasibility of any proposed mining layout. The mechanical properties which were considered to be relevant, and which consequently became the subject of the investigation were :-

a) Ultimate tensile strength.

b) Ultimate compressive strength.

It was felt that once representative values had been obtained for these properties, it would be possible to assess the problems involved in the mining of this seam.

14. THE DETERMINATION OF THE MECHANICAL PROPERTIES OF THE SEAM MATERIAL AND SURROUNDING STRATA.

In order to provide the necessary samples for laboratory testing, an exploratory borehole, reference number 629/K.T., was drilled from the surface, passing through the two gypsum beds. This borehole was sited to the East of the 'B' Bed underground workings and just South of the Newbiggin railway station. Core samples of the strata, 3 in. in diameter, were obtained from a depth of 78 ft. 6 in. to 131 ft. from the surface, i.e. from 16 ft. 6 in. above 'C' Bed to 3 ft. below the 'B' Bed at this point. Unfortunately, about 4 ft. 6 in. of the core lying immediately above the 'C' Bed was destroyed during the drilling operation. Since this represented that part of the strata that would have a great bearing on the mining of 'C' Bed, another borehole, reference number 629 A/K.T. was drilled very close to the original borehole. In this case, only that part of the strata from 11 ft. above the 'C' Bed to 3 ft. below it was cored.

BOREHOLE 629/K.T.

					ft.	ins.
Core 1	78' 6"	-	84'	Recovered	3	6
Core 2	84'	-	88' 6"	"	4	2
Core 3	88' 6"	-	91'	"	2	4
Core 4	91'	-	99'	"	4	4
Core 5	99'	-	104'	"	5	0
Core 6	104'	-	109'	"	5	0
Core 7	109'	-	117'	"	7	6
Core 8	117'	-	124'	"	6	0
Core 9	124'	-	131'	"	6	0
				TOTAL	44'	10"

44 ft. 10 in. recovered out of total core length of 52 ft. 6 in.

BOREHOLE 629 A/K.T.

					ft.	ins.
Core 1	84' 6"	-	89' 6"	Recovered	4	3
Core 2	89' 6"	-	94' 6"	"	4	8
Core 3	94' 6"	-	99'	"	4	6
Core 4	99'	-	106'	"	6	9
				TOTAL	20'	2"

20 ft. 2 in. recovered out of total core length of 21 ft. 6 in.

A full geological description of the cores is not included here, but this is given where applicable when describing the laboratory results obtained. A typical strata section has been drawn from the information obtained from the cores and this is shown in Fig.89.

It was decided to use the greater part of the cores from both boreholes for ultimate tensile strength determinations since it is this factor which most frequently limits the extent of an underground excavation of the type planned under the geological conditions present. The proposed workings will be quite shallow and the compressive pillar loads, therefore, quite low. A small number of uniaxial compressive strength tests were also carried out, the specimens being taken from various horizons in both boreholes.

14.1 The Determination of the Tensile Strength.

The most convenient tensile test specimen obtainable from the cores was a disc and therefore, the Brazilian Disc Test was used to determine the ultimate tensile strength values. A great deal of literature exists relating to tensile testing in general and to the Brazilian Disc Test in particular; the following references are fairly representative of the work which has been carried out, (85) (86) (50).

Almost the entire set of cores obtained from both boreholes, with the exception of a number of compressive test specimens, were made into discs of 3 in. diameter and 1.1/2 in. thickness. The end faces of the discs were ground flat before crushing diametrically between parallel steel platens in a 25 ton hydraulic testing machine, at a loading rate of between 15 - 75 p.s.i./min.(50). The failure strength was then calculated from the usual Brazilian formula :-

$$\sigma_T = \frac{2\,P}{\Pi d\,t}$$

where σ_T = tensile strength of the specimen

P = failure load

d = diameter of disc

t = thickness of disc

Each test specimen has been referred to depending on which borehole and which core it was obtained from, the numbers increasing with depth,

e.g.	Borehole 629A, Core 3.	C3A-S1,	C3A-S2,etc.
	Borehole 629, Core 5.	C5-S1,	C5-S2, etc.

The vertical variation of the tensile strength of the core material obtained from both boreholes, is shown in Fig.89. Also shown in this diagram is the relationship between the tensile strengths of the rock material and the position of the various strata layers, in particular the top and bottom of 'C' Bed. From this, it is possible to visualise how critical a factor is the relationship between the mining horizon and the immediate roof of the proposed 'C' Bed workings.

14.1.1 The Strata Above 'C' Bed.

As mentioned previously, the immediate 5 ft. of core material above 'C' Bed in borehole 629/K.T. was destroyed during the drilling operation. However, a number of tensile test specimens were made from the cores obtained for this section of strata from borehole 629 A/K.T., though with some difficulty. The results obtained are shown in Table 7.

This gives a mean tensile strength for the immediate 5 ft. of strata overlying 'C' Bed of 90 p.s.i.

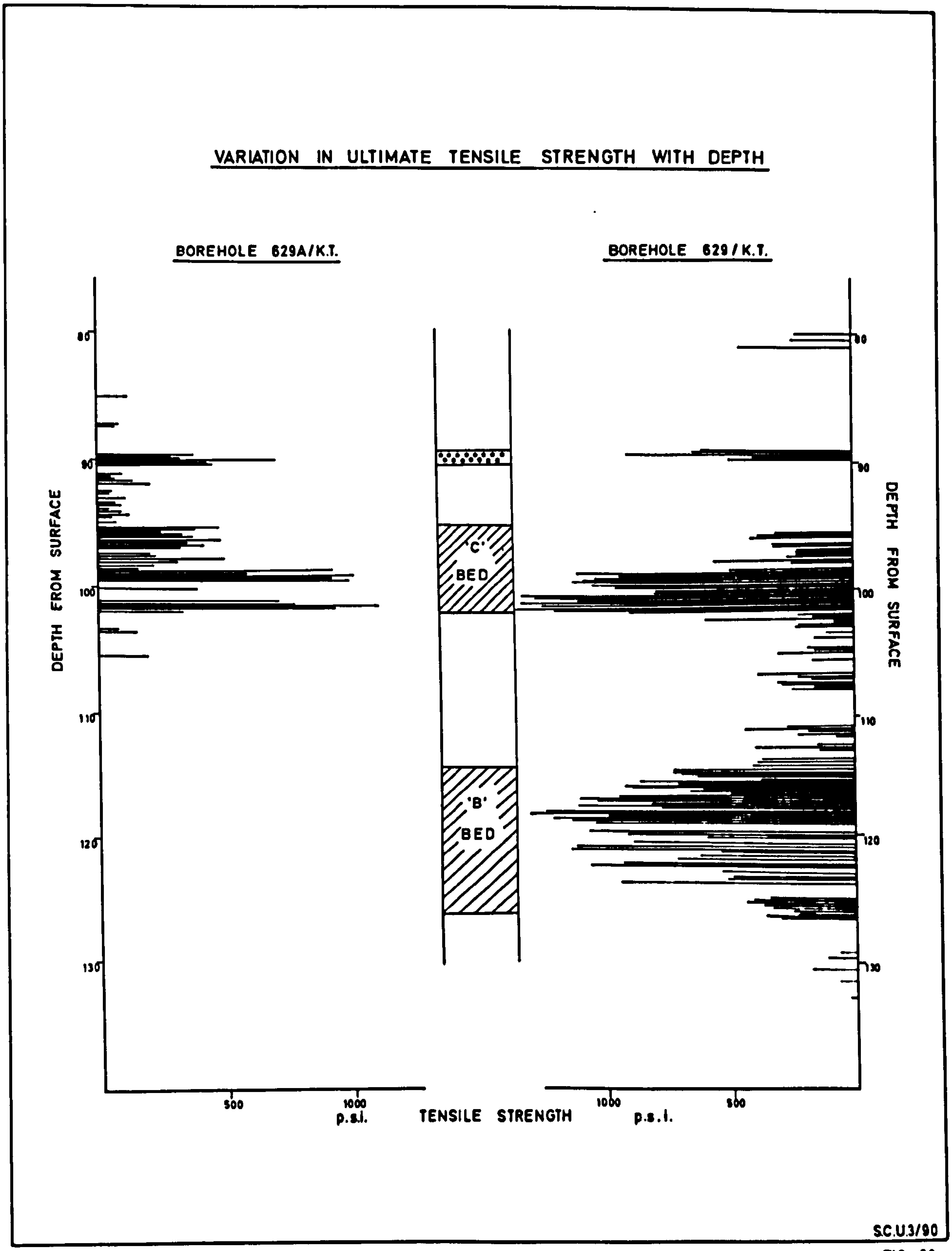

VARIATION IN ULTIMATE TENSILE STRENGTH WITH DEPTH
BOREHOLE 629A/K.T.
BOREHOLE 629/K.T.
DEPTH FROM SURFACE
80
90
100
110
120
130
'C' BED
'B' BED
500
1000
p.s.i.
TENSILE STRENGTH
1000
500
p.s.i.
DEPTH FROM SURFACE
S.C.U.3/90

FIG. 89

TABLE 7

Sample Ref.No.	Tensile Strength p.s.i.	Distance of Specimen Above Roof of 'C' Bed.
C2A - S8	105	49 ins.
C2A - S9	60	47 ins.
C2A - S10	63	45 ins.
C2A - S11	145	43 ins.
C2A - S12	215	40 ins.
C2A - S13	60	35 ins.
C2A - S14	51	31 ins.
C2A - S15	117	27 ins.
C2A - S16	75	22 ins.
C2A - S17	87	20 ins.
C2A - S18	50	17 ins.
C2A - S19	98	14 ins.
C3A - S1	123	10 ins.
C3A - S2	65	8 ins.
C3A - S3	43	6 ins.
C3A - S4	80	4 ins.

TABLE 8

BOREHOLE 629/K.T.

Sample Ref. No.	Tensile Strength p.s.i.
C3 - S1	599
C3 - S2	630
C3 - S3	893
C3 - S4	383
C3 - S5	494

Mean Tensile Strength = 600 p.s.i.

BOREHOLE 629 A/K.T.

Sample Ref. No.	Tensile Strength p.s.i.
C2A - S1	388
C2A - S2	290
C2A - S3	331
C2A - S4	700
C2A - S5	434
C2A - S6	455
C2A - S7	336

Mean Tensile Strength = 419 p.s.i.

These extremely low values for the tensile strength of specimens obtained from this section of strata was, in fact, to be expected from a visual examination of the core samples forming this section. Some of the specimens tested are shown in the photograph, Fig. 90. It consists of a badly jointed soft red marl, the joints being caused by varying thicknesses of gypsum spars.

Immediately overlying this, however, was found a relatively high tensile strength layer, about 15 in. thick, consisting of grey gypsiferous marls. The tensile strength values obtained from the testing of this material are shown in Table 8.

Above this relatively strong layer, the badly jointed red marl continued. The cores from this part were very badly broken allowing only 3 tensile test specimens to be made.

These gave tensile stress values of 76 p.s.i., 93 p.s.i. and 126 p.s.i. respectively, very similar values to those obtained from the immediate 5 ft. of strata overlying the 'C' Bed. Approximately 14 ft. above the top of the 'C' Bed, grey marl was found interspaced with the red marl and this produced much higher tensile strength values.

Three specimens were made from this section, giving tensile strength values of 448 p.s.i., 237 p.s.i. and 223 p.s.i. respectively.

Fig. 90

14.1.2 <u>The 'C' Bed Gypsum.</u>

The vertical variation of the tensile strength through the seam section is shown in Fig.89. The test specimens obtained from borehole 629A/K.T. cores gave tensile strength values between 209 - 1128 p.s.i., whilst those obtained from borehole 629/K.T. gave tensile strength values between 181 - 1552 p.s.i. Some of the specimens tested from 'C' Bed are shown in the photograph, Fig.91. The tensile strength values obtained, for each borehole, are shown in Table 9, together with the position of each test specimen in the seam in relation to the top of the seam.

The lower tensile strength values may be related to those test specimens in which the gypsum contained a great deal of solution cavities which undoubtedly affected their tensile strength. These cavities, however, were mostly found in the middle section of the seam where the tensile strength is not an important factor. The higher tensile strength specimens were mainly found in the bottom half of the seam and invariably consisted of dark grey anhydrite.

14.1.3 <u>The Inter-Seam Strata.</u>

The cores obtained from borehole 629/K.T. representing the strata between the two seams, indicated that the seams were approximately 12 ft. apart at this point. This inter-seam strata was very similar in appearance to the strata immediately above 'C' Bed, though the gypsum spars in the red marl appeared to be thicker and more frequent. In addition, the red marl appeared to be more compact than that present above 'C' Bed. This probably accounts for the fact that even though some specimens gave very low tensile strength values comparable with those obtained from the strata above 'C' Bed, the majority gave much higher tensile strengths. The results obtained are shown in Table 10, whilst a photograph of some of the broken specimens is shown in Fig. 92.

TABLE 9

TENSILE TESTS CARRIED OUT ON DISCS TAKEN FROM 'C' BED

SEAM SECTION - CORES FROM BOTH BOREHOLES

DISC DIAMETER 3" ; DISC THICKNESS 1½"

Borehole 629A/K.T.			Borehole 629/K.T.		
Sample Ref. No.	Tensile Str. p.s.i.	Distance from Top of Seam	Sample Ref. No.	Tensile Str. p.s.i.	Distance from Top of Seam
C3A - S5	485	1"	C4 - S1	304	7"
C3A - S6	390	3"	C4 - S2	374	9"
C3A - S7	260	5"	C4 - S3	402	12"
C3A - S8	339	7"	C4 - S5	320	19"
C3A - S9	380	9"	C4 - S6	304	21"
C3A - S10	309	11"	C4 - S7	215	23"
C3A - S11	494	13"	C4 - S8	225	25"
C3A - S12	363	15"	C4 - S9	209	27"
C3A - S13	432	17"	C4 - S10	253	28"
C3A - S14	336	19"	C4 - S11	181	30"
C3A - S15	269	21"	C4 - S12	550	32"
C3A - S17	212	27"	C4 - S13	241	34"
C3A - S18	233	29"	C4 - S14	494	40"
C3A - S19	508	31"	C5 - S1	1096	45"
C3A - S20	320	33"	C5 - S2	931	47"
C3A - S21	209	35"	C5 - S3	1020	49"
C3A - S22	231	37"	C5 - S4	1074	51"
C3A - S23	260	39"	C5 - S5	1115	53"
C4A - S1	942	42"	C5 - S6	1033	55"
C4A - S2	596	44"	C5 - S7	944	57"
C4A - S3	1014	46"	C5 - S8	786	59"
C4A - S4	935	48"	C5 - S9	792	61"
C4A - S5	1007	50"	C5 - S10	1321	63"
C4A - S6	320	52"	C5 - S11	1261	65"
C4A - S8	399	60"	C5 - S12	1096	67"
C4A - S10	725	73"	C5 - S13	1334	69"
C4A - S11	749	75"	C5 - S14	1245	71"
C4A - S12	1128	77"	C5 - S15	1552	73"
C4A - S13	944	79"	C5 - S16	1179	75"
C4A - S14	350	81"	C5 - S17	897	80"

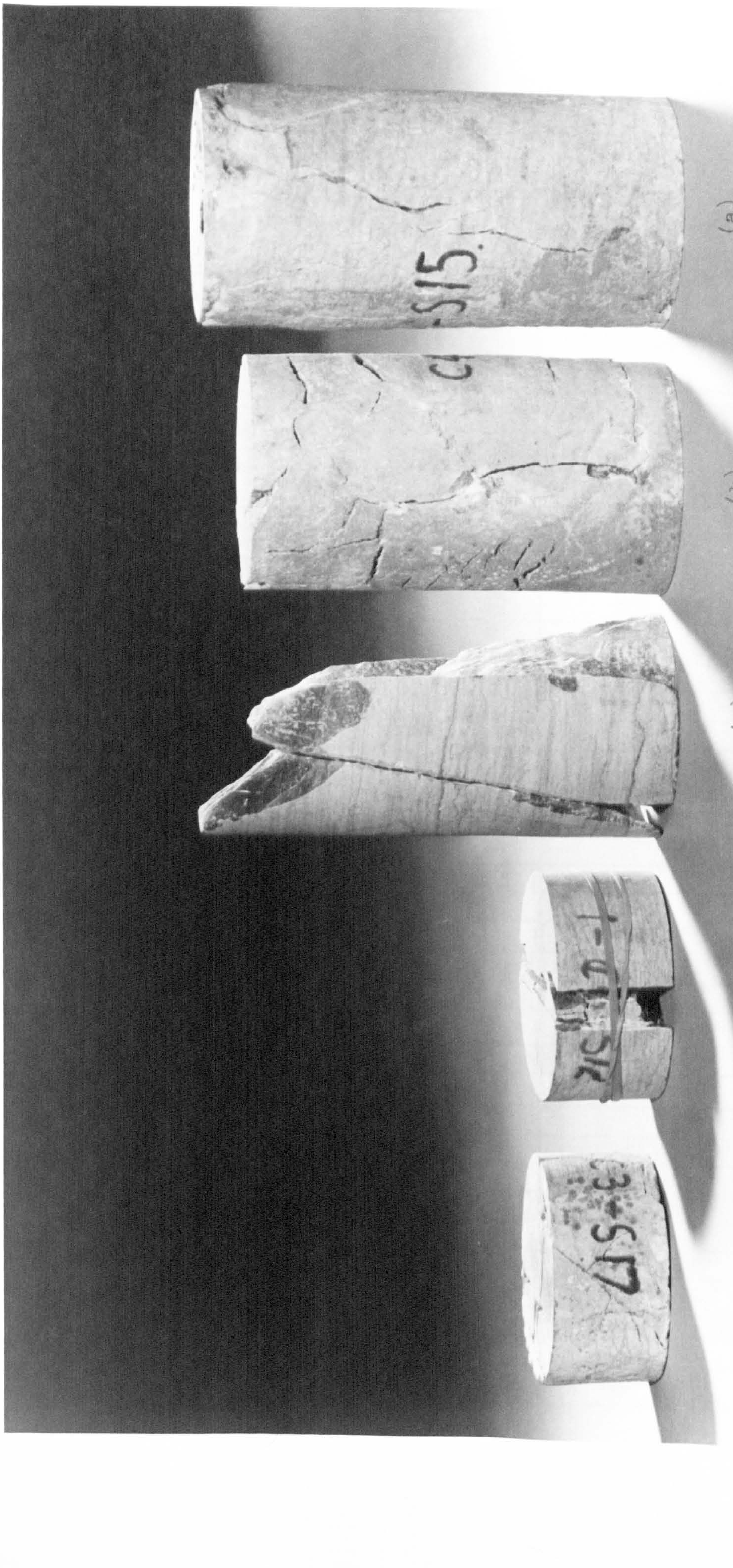


Fig. 91

TABLE 10.

TENSILE TESTS CARRIED OUT ON DISCS TAKEN FROM THE STRATA BETWEEN 'C' BED AND 'B' BED - BOREHOLE 629/K.T.

DISC DIAMETER 3" ; DISC THICKNESS 1½"

Sample Ref.No.	Tensile Strength p.s.i.	Distance below base of 'C' Bed - ins.
C5 - S19	217	1"
C5 - S20	157	3"
C5 - S21	586	6"
C5 - S22	70	9"
C5 - S23	212	11"
C5 - S24	222	13"
C5 - S26	101	20"
C6 - S1	148	24"
C6 - S2	177	33"
C6 - S3	150	35"
C6 - S4	295	37"
C6 - S5	164	43"
C6 - S6	372	58"
C6 - S7	220	61"
C6 - S8	160	63"
C6 - S9	198	65"
C6 - S10	177	67"
C6 - S11	139	69"
C6 - S12	236	71"
C6 - S13	78	73"
C7 - S1	258	109"
C7 - S2	436	111"
C7 - S3	163	113"
C7 - S4	223	115"
C7 - S5	63	118"
C7 - S7	139	126"
C7 - S8	390	128"
C7 - S9	112	130"
C7 - S10	355	140"
C7 - S11	382	142"
C7 - S12	396	TOP OF 'B' BED 144"
C7 - S13	716	-
C7 - S14	719	-

Fig. 92

TABLE 11.

TENSILE TESTS CARRIED OUT ON DISCS TAKEN FROM 'B' BED SEAM SECTION CORES FROM BOREHOLE 629/K.T.

DISC DIAMETER 3" ; DISC THICKNESS 1½"

Sample Ref.No.	Tensile Str. p.s.i.	Distance from top of Seam
C7 - S12	396	Top of Seam
C7 - S12a	716	3"
C7 - S13	680	5"
C7 - S14	719	7"
C7 - S15	618	9"
C7 - S16	364	11"
C7 - S17	849	13"
C7 - S18	691	15"
C7 - S19	906	17"
C7 - S20	871	19"
C8 - S1	596	29"
C8 - S2	652	31"
C8 - S3	494	32"
C8 - S4	938	34"
C8 - S5	1087	36"
C8 - S6	1017	38"
C8 - S7	802	41"
C8 - S8	1093	43"
C8 - S9	760	45"
C8 - S10	1223	47"
C8 - S11	1299	49"
C8 - S12	982	51"
C8 - S13	1194	53"
C8 - S14	1112	55"
C8 - S16	1011	63"
C8 - S17	1058	65"
C8 - S18	900	67"
C8 - S19	523	69"
C8 - S20	897	73"
C8 - S21	1109	76"
C8 - S22	1118	78"
C8 - S23	510	81"
C8 - S24	618	83"
C8 - S25	700	85"
C8 - S26	912	87"
C8 - S28	1058	90"
C8 - S29	526	95"

Sample Ref.No.	Tensile Str. p.s.i.	Distance from top of Seam
C9 - S1	437	101"
C9 - S2	519	103
C9 - S3	944	105"
C9 - S6	339	127"
C9 - S7	459	129"
C9 - S8	462	133"
C9 - S9	361	135"
C9 - S10	333	137"
C9 - S11	253	139"
C9 - S12	247	141"
C9 - S13	386	143"
C9 - S14	317	Bottom of Seam

14.1.4 <u>The 'B' Bed Gypsum.</u>

The entire section of the core material representing the 'B' Bed obtained from borehole 629/K.T., with the exception of 5 compressive specimens, was made into tensile disc specimens The tensile strength values obtained are shown in Table 11.

It can be seen from these results that the lower strength values were obtained from the upper 2 ft. of the seam, which is left to form the immediate roof, and the basal 1 ft. of the seam. The upper 2 ft. of the seam gave a mean tensile strength of 671 p.s.i., which is still a fairly high tensile strength value.

These results indicated the obvious geological differences between the two seams. The 'B' Bed contained a great deal of anhydrite which gave extremely high tensile strength values. This was also apparent in the 'C' Bed strength values, but not so numerous.

The immediate strata beneath 'B' Bed was very similar to that found above and below 'C' Bed. Those specimens tested from this section of the strata gave tensile strength values between 41 - 101 p.s.i., comparable with the strata above 'C', Bed.

14.2 <u>The Determination of the Compressive Strength.</u>

Only a small number of uniaxial compressive strength tests were carried out, the specimens being taken from various horizons in both boreholes. In each case, the test specimens were of 3 in. diameter and 6 in. high, being cut from the 3 in. diameter core. The end faces of each speciman were ground flat and parallel to within 0.002 in. before testing to failure between ground steel platens at a constant loading rate of between 10 - 20 p.s.i./sec. Each specimen has been referred to in the same manner as the tensile disc specimens. The results obtained are shown in Table 12.

TABLE 12

COMPRESSIVE STRENGTH DATA OBTAINED FROM CYLINDRICAL SPECIMENS, 3 in. DIAMETER, 6 in. LONG, MADE FROM CORES OBTAINED FROM BOTH BOREHOLES.

Sample Ref. No.	Compressive Strength p.s.i.	Position of Specimen Tested
C2A - S15a	737	Red Marl. Approx.24 in.above 'C' Bed.
C4 - S4	3,691	'C' Bed gypsum. 15 in.below top of seam.
C3A - S16	2,882	'C' Bed gypsum, 24 in. below top of seam.
C4A - S7	2,645	'C' Bed gypsum. 55 in.below top of seam.
C5 - S25	1,239	Inter-seam strata. 16 in.below base of 'C' Bed.
C6 - S14	1,860	Inter-seam strata. 80 in.below base of 'C' Bed.
C7 - S6	1,987	Inter-seam strata. 122 in.below base of 'C' Bed.
C7 - S21	16,731	'B' Bed anhydrite. 21 in.below top of seam.
C8 - S15	15,468	'B' Bed anhydrite. 58 in.below top of seam
C9 - S4	6,452	'B' Bed gypsum. 110 in.below top of seam.
C9 - S5	5,323	'B' Bed gypsum. 122 in.below top of seam.

Some of the fractured specimens are shown in the photographs, Figs. 90 - 92.

The single specimen obtained from the strata above the seam gave the very low compressive strength of 737 p.s.i., which confirms any conclusions that may be drawn from the low tensile strength values obtained for this section of strata.

Four test specimens were made from the 'C' Bed material, three of which were gypsum, whilst the fourth was anhydrite from the lower section of the seam. The gypsum specimens, C3A - S16 and C4A - S7 contained many solution cavities which probably accounts for their relatively low compressive strength values of 2882 p.s.i. and 2645 p.s.i. respectively. These specimens are shown in Fig.91, referred to as (a) and (b). The anhydrite test specimen gave the relatively high compressive strength value of 14,192 p.s.i., this specimen also being shown, (c), in Fig. 91.

The three specimens obtained from the inter-seam strata are shown in Fig.92. These confirmed the tensile strength values obtained from the same section of strata by giving fairly low compressive strength values of 1239 p.s.i., 1860 p.s.i., and 1987 p.s.i. respectively. The failure of these specimens was obviously influenced by the occurrence of gypsum spars in the specimens.

Four specimens were made from the 'B' Bed core material, two of which were anhydrite and the other two gypsum. The two anhydrite specimens taken from the upper half of the seam gave relatively high compressive strength values of 16,731 p.s.i. and 15,468 p.s.i. The two gypsum specimens from the lower half of the seam gave compressive strength values of 6,452 p.s.i. and 5,323 p.s.i. These strength values indicate the vertical variation in the material forming the seam.

15. CONCLUSIONS AND RECOMMENDATIONS RELATING TO THE NEWBIGGIN MINE.

The laboratory rock testing programme, though relatively short, clearly indicated the stability problems that may be encountered if the 'C' Bed gypsum is mined. The strength values obtained suggested that roof control of the 'C' Bed workings could be a critical factor in assessing the workability of the seam. To a lesser extent, it also indicated the importance of the need to directly superimpose the pillars in the two seams due to the low strength values obtained for the inter-seam strata. This latter factor could possibly limit, to some extent, the amount of machinery used, and hence affect the method of mining.

The vertical variation of the tensile strength of the seam material and surrounding strata, illustrated in Fig.89, indicated how critical a factor was the relationship between the mining horizon or the section of the seam extracted, and the immediate roof and floor of the proposed workings. The 5 ft. of strata immediately above 'C' Bed gave extremely low tensile and compressive strength values suggesting that the mining dimensions, and subsequently the percentage extraction, could be influenced to some considerable extent by this section of the strata. However, the investigations revealed the presence of a 15 in. thick bed of strong grey gypsiferous marl on top of this 5 ft. of weak red marl immediately above the seam. This bed could possibly be utilised by strata control measures such as roof bolting, that will probably have to be introduced to ensure complete stability of the immediate roof strata. With regard to this factor, it is probable that a section of the upper part of the seam would have to be left to form the immediate roof of the workings. Roof bolting could then be used to tie together the gypsum roof and the bed of strong grey marl to form a composite beam, thus enabling the formation of much wider roadways. If this method of roof control is to be used, and it would seem to be the most attractive, a number of factors would have to be considered.

An extensive borehole survey had been carried out in the past, and this had indicated that the thickness and composition of 'C' Bed varied, but there had been no indication of the presence of the bed of strong grey marl throughout the area above the seam, obviously this borehole survey being more concerned with proving the existence of the seams. However, since this marl bed could simplify the strata control measures, and more important, probably allow an increase in the width of the roadways, its presence should be confirmed throughout the area to be mined. This could be done by drilling boreholes from the underlying 'B' Bed workings and obtaining core samples of the overlying strata. It is felt that the presence of this bed would greatly increase the efficiency of any roof bolting method of artificial support that may be used. If this bed is not present, a much larger number of roof bolts would be needed, with a subsequent increase in the cost of support, since the irregular bands of gypsum spars in the marl could result in blocks of this red marl falling out of the roof if insufficient roof bolts were installed.

A further factor that would have to be considered would be the thickness of gypsum left to form the immediate roof of the 'C' Bed workings. In view of the thickness of the seam and the probable need to leave some part of the seam to form the immediate floor, the thickness of this roof beam would obviously need to be kept to a minimum. In the 'B' Bed workings, a 12 in. thick gypsum roof beam was formed and this had ensured roof stability without the need of roof bolting. The laboratory investigations showed that the mean tensile strength of the upper 12 in. of the 'B' Bed was much higher than the mean tensile strength of the upper 12 in. of the 'C' Bed. However, the introduction of an additional support in the form of roof bolts, should result in a beam of sufficient strength to ensure roof stability in the 'C' Bed workings. In all probability, assuming that a 12 in. thick gypsum roof beam is formed, this would mean that the working height would be restricted to a maximum slightly less than 6 ft. One of the main practical difficulties would probably be in maintaining a 12 in. gypsum beam since there appears to be no natural parting in this upper part of the seam.

The inter-seam strata also gave fairly low strength values. It was very similar in appearance to the strata above the seam, but the red marl appeared to be slightly harder than the red marl above 'C' Bed, probably accounting for the slightly higher strength values obtained. The initial development drift from the 'B' Bed workings to 'C' Bed confirmed the borehole information that the seams were approximately 12 ft. apart, and also indicated the large scale effect of the irregular bands of gypsum spars in this inter-seam strata. The failure of a large number of the test specimens obtained from the core material representing the inter-seam strata, was affected by the presence of gypsum spars and invariably gave low strength values. Those specimens consisting almost entirely of red marl gave much higher strength values. The initial development showed that these gypsum spars gave a block nature to the inter-seam strata, suggesting that if failure occurred, it would most probably be in the form of blocks becoming detached from the surrounding strata. However, the probable working height of 'C' Bed precludes the use of heavy machinery of the type used in 'B' Bed. Therefore, in all probability, provided the pillars are directly superimposed, there should be no excessive loading of the inter-seam strata, and it should therefore remain stable.

In view of the nature of the strata surrounding the two gypsum beds, it is probable that the most economic and safe method of mining 'C' Bed will be developed through practical experience as 'C' Bed is extracted. The initial development should proceed using the minimum possible roof span, and deformation measuring instruments installed, both in the boreholes in the overlying strata and also within the workings themselves. The efficiency of any roof bolting method of artificial support could also be easily determined. The room width could then be progressively widened, bearing in mind the fact that the maximum roof span in room and pillar workings exists at roadway intersections.

D. THE BRIGHTLING MINE.

16. DESCRIPTION OF THE GENERAL CONDITIONS AT THE BRIGHTLING MINE.

The Brightling gypsum mine, Sussex, was unusual in that two steeply inclined seams were being worked simultaneously in close proximity to each other. Expansion of the mine workings had necessitated the working of both seams on rather steep gradients, and this, together with the fact that a serious pillar collapse occurred in the lower seam workings at a nearby gypsum mine under similar circumstances, suggested that a closer appreciation of the state of stability of the mine workings was needed. A description of the geology of the area is given here since it is felt that the geological structure, and to some extent the stratigraphy of the area, may have some influence on the problem.

16.1 The Geology of the Mining Area.

The geology of the area surrounding the Brightling mine is very well known, due partly to the gypsum mines working in the area, but also due to the very large number of exploratory boreholes that have been sunk in the district in the search for gypsum, coal, and in recent years, oil and gas.

The gypsiferous seams in the area are contained in the basal part of the Purbeck Beds of Sussex. These beds, of total thickness of approximately 400 ft., have been exposed in three inliers by the erosion of sharp 'en-echelon' folds that traverse this part of the Central Weald of Sussex. The position of these anticlines is shown in Fig.93. The gypsum mines in the area are located in the anticlinal Mountfield and Brightling - Heathfield inliers, both of which are bounded by faults. The complete strata section shown in Fig.94, has been designated the type section of the area. A more detailed section of the gypsiferous beds of the Lower Purbeck at the Brightling Mine, is shown in Fig.95. This shows the gypsum seams to be approximately 400 ft. from the surface at this point. The total thickness of the gypsiferous beds varies but may be taken as between 60 and 70 ft., although much of this is comprised of shale above and below the gypsum seams proper. The base of these gypsum beds, which is also

GEOLOGICAL MAP OF THE PURBECK INLIERS

OF THE CENTRAL WEALD

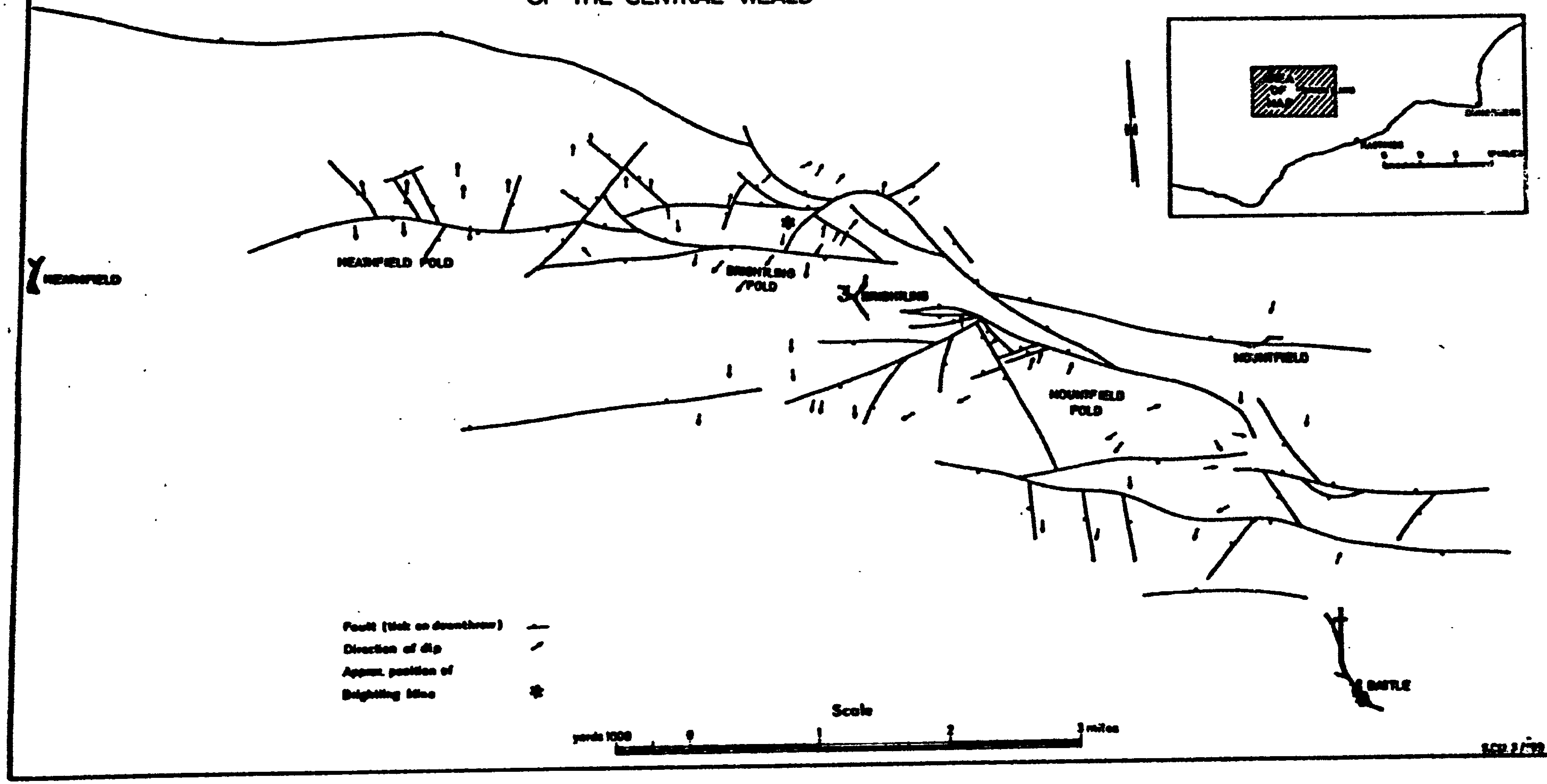
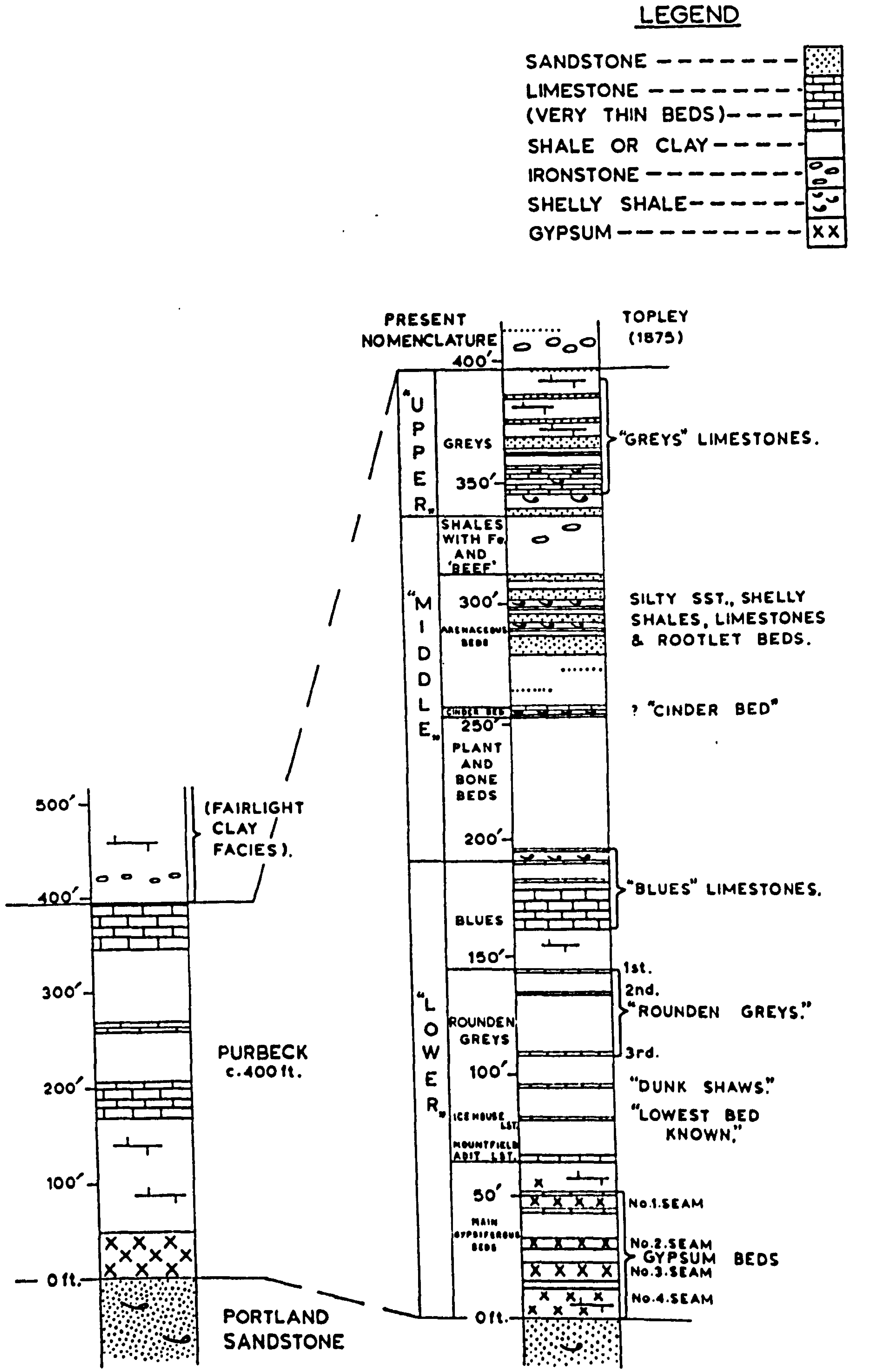


Fig. 93



General succession of Purbeck and Hastings Beds at Mountfield, Sussex.

Fig. 94

TYPICAL STRATA SECTION

BRIGHTLING AREA

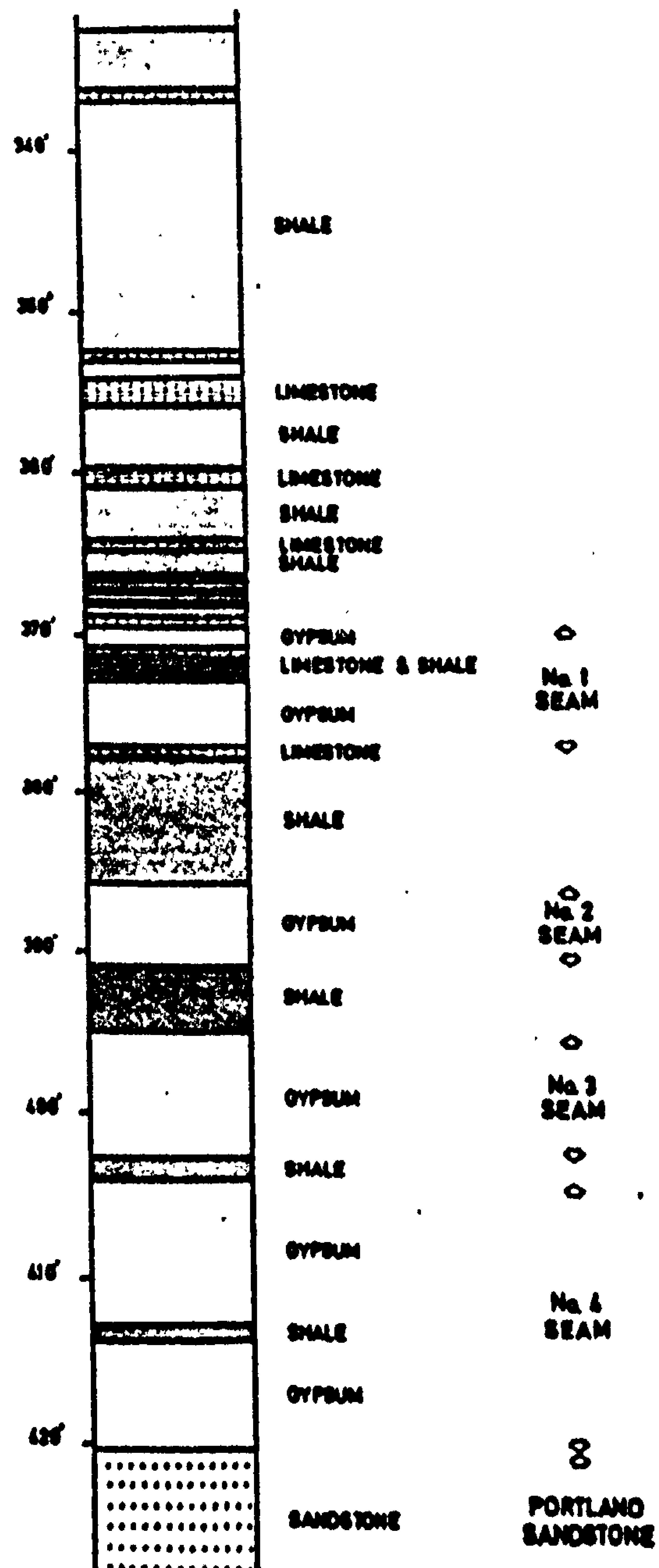


Fig. 95

the base of the Purbeck, is marked by the top of the water bearing Portland Sandstone.

There are four main gypsum seams, numbered here from the top downwards in accordance with local mining practice. The basal or No.4 Seam is worked at Brightling, and may be taken as approximately 15 ft. in thickness, although this may vary slightly from place to place. The gypsum in the bottom of the seam is separated from the Portland Sandstone by 6 in. of gypsiferous silty limestone. The remainder of the seam is very finely banded with limestone and minute shale layers. The Nos. 2 and 3 Seams, 5 ft. and 7 ft. thick respectively, are not worked at Brightling. The No.1 Seam however, is worked at Brightling, and is about 7 ft. thick. It has a basal limestone about 8 in. thick known as the 'bottom rock' by the gypsum miners. The seam is roofed by a laminated current-bedded calcareous siltstone about 7 in. thick. The strata between the seams consists mainly of shale with some thin bands of limestone. Traces of free oil and bitumen have been noted within the mine workings and in boreholes through the seams.

Three faulted anticlines occur 'en-echelon' in the area, together with their complementary synclines. These are the Heathfield, Brightling and Mountfield anticlines as shown in Fig.93. The Brightling anticline is in fact twofolds with a saddle between. The Mountfield and Brightling anticlines have several features in common. The northern flank of each is steeper (11° to 23°) than the southern flank (7° to 10°). Also, the eastern pitch of each is steeper than the western. The axis of folding changes slightly from one fold to another so that while the Mountfield anticline is ESE - WNW, the Brightling and Heathfield anticlines are E - W.

All the folds are strongly faulted, the largest faults running parallel to the folds and forming boundaries to the Purbeck inliers. The faults are shown in Fig.93. The Brightling anticline has a major fault to the North and the South throwing up to 500 ft. north and 200 ft. South respectively. The western nose of the Brightling fold has other smaller faults. A thrust fault is also present in this fold and it has been suggested that it has its surface expression in the Brightling Northern boundary fault. It has also been suggested by Howitt (86), that the thrust fault and the fold developed at the sa same time and that the faults have arisen as a result of the relaxation of the forces causing the folding and thrust fault. This assumes the direction of thrusting at Brightling to be towards the steeper northern flank of the fold. The structure of the Purbeck inliers therefore, and in particular the structure of the Brightling anticline, appears to be mainly a result of tectonic forces from the South and a partial relaxation of those forces.

16.2 Method of Mining.

The Brightling Gypsum Mine is situated on the crest of the Brightling anticline with gypsum being mined at a depth of approximately 400 ft. from the surface. The mine workings are situated in Nos. 1 and 4 Seams, with a room and pillar method of mining used, giving a 75% area extraction of both seams. The distance between the floor of No. 1 Seam and the roof of No.4 Seam is approximately 30 ft. The pillars in the upper or No.1 Seam are normally 20 ft.square and 7 ft. high, whilst those in the lower or No.4 Seam are also normally 20 ft. square but 10 ft. high. The roadways in both seams are formed 20 ft. wide at all times. Though both seams are mined simultaneously, a panel in No.4 Seam is worked in advance of the corresponding panel in No.1 Seam vertically above.

A 10 ft. section of No.4 Seam is extracted leaving several feet of gypsum to form the immediate roof of the seam. This thickness of roof beam varies but is normally 3 ft. The floor of the seam is usually quite wet due to the water bearing Sandstone immediately beneath the seam.

On the crest of the anticline where the seams are level, the pillars in the two seams were directly superimposed. The crest, where level working conditions were possible however, was limited to the East and West of the anticline by faulting, and this has necessitated the working of both seams on the rather steeply inclined northern limb of the Brightling anticline. It is anticipated that the maximum dip the workings will attain in the near future will be 1 in 5 due North, though some parts of the northern limb of the anticline attain a dip angle much greater than this.

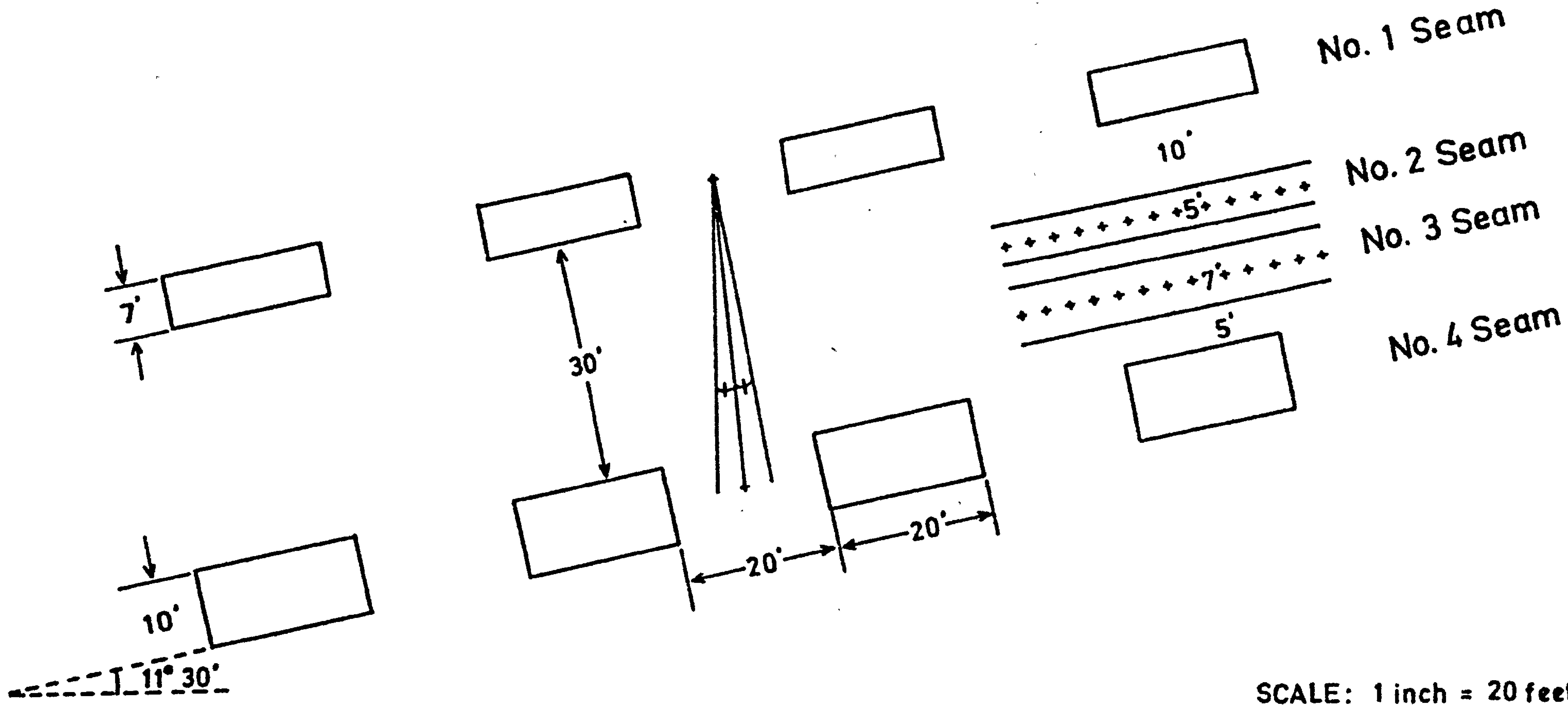
Because of this, a compromise was reached in the relative pillar layout between the seams. The lower seam pillars were given an advance of a few feet to the dip on the corresponding pillars in the seam above. A section through the workings is shown in Fig. 96. The displacement of the lower seam pillars relative to those in the seam above was calculated by bisecting the angle between the normal and the perpendicular drawn at the centre of a pillar in the upper seam. It is not known whether this pillar layout provides a satisfactory state of stability in the long term since it was arrived at by geometrical means only, rathr than using a criterion based on pillar design considerations.

16.3 Method of Approach Used in this Investigation

The stability problems introduced by the combined effect of working an inclined duplex seam system have already been discussed in some detail in an earlier section; Part 1 Section 4. It is proposed here to discuss only those additional factors which it is felt have some bearing on the stability of the workings at the Brightling mine, and to suggest ways in which these may be investigated.

The Brightling mine is situated at the centre of an area that has been subjected to geological forces of unknown magnitude. Some consideration therefore, should be given to the estimation and effect of the magnitude of the lateral primitive stress component. This

SECTION THROUGH THE BRIGHTLING MINE WORKINGS



SCALE: 1 inch = 20 feet

S.C.U.3/153

FIG. 96

lateral component of the primitive stressfield acting on the pillars in an inclined seam would tend to increase the value of the shearing stresses appearing in the pillars, as compared with the shearing stresses in pillars in a level seam. The shear stresses are not considerable when compared with the mean stresses normal to the stratification plane, but under certain unfavourable conditions these stresses could result in pillar instability. The fact that the workings are directly superimposed and only 30 ft. apart, together with the influence of the angle of dip of the workings on the stress distribution both in the pillars and the adjacent strata should also be considered.

It was decided that the method of approach to the problem should be primarily by laboratory experimentation and theoretical studies. An underground investigation could then be subsequently initiated if it was felt necessary. The main objectives of the approach used, therefore, were as follows :-

(i) To determine the mechanical characteristics of the rock material making up Nos. 1 and 4 Seams.

(ii) To determine the magnitude of the stress effects, and their depth of penetration from the working of one seam only.

(iii) To determine the effect of the variation of the forces acting on a pillar formed in an inclined seam when the inclination of the seam and the magnitude of the lateral primitive stress vary.

A mathematical and laboratory model study were carried out to investigate (ii) and (iii) above. These model studies were designed in such a way that they were complementary to each other. It was hoped that definite conclusions could be reached from these investigations that would suggest the best possible relative position of the pillars in the two seams which would provide a satisfactory state of pillar stability. From the evidence available prior to the initiation of these investigations, it would appear that the pillars of which the state of stability is being questioned are those formed in No.4 Seam. It was the lower seam pillars that collapsed at a neighbouring mine. This investigation however, has not been conducted with the intention of determining the state of stability of No.4 Seam pillars only, though it is somewhat biased towards that end. It was hoped that a clearer indication of the problem would be given following the laboratory determination of the mechanical characteristics of the gypsum from both seams.

17. THE MECHANICAL PROPERTIES OF BRIGHTLING GYPSUM.

In an investigation of the stability of an underground opening, a careful study of the mechanical behaviour of the surrounding rock materials is one of the most important requirements. For this reason, laboratory investigations were initiated in order to determine the value of those mechanical properties of the seam material which it was felt, would have particular relevance as far as their effect on the stability of the mine pillars and surrounding rooms was concernd. These properties were :-

(i) Ultimate tensile strength.

(ii) Ultimate uniaxial compressive strength.

(iii) Triaxial and ultimate shear strength.

(iv) Various elastic constants.

The rock specimens on which these mechanical property tests were carried out, were obtained from block samples taken from the working faces of Nos. 1 and 4 Seams in the Brightling Mine. These blocks were of irregular shape and size, each block sample representing a certain horizon in the appropriate seam, including the strata forming the roof and floor of each seam.

Eight block samples were obtained from No.1 Seam. Those taken from the upper part of the seam contained a great deal of calcareous shale.

Eleven block samples were taken from No.4 Seam. These had a distinctly different appearance to those obtained from No.1 Seam, the gypsum tending to be of a finer grain size. Those samples from the mid-part of No.4 Seam contained a large number of white fibrous gypsum, or satinspar bands. The samples from the upper part of this seam appeared to be of a fairly uniform granular structure, containing only a few narrow satinspar bands, while those from the base of the seam contained large quantities of shale. The block samples have been referred to by the letter S, followed by the seam number; this in turn is followed by the sample reference number determining its horizon in the seam. Thus block samples S1-1 and S1-8 denote the floor and roof sample respectively from No.1 Seam.

Extreme difficulty was experienced in obtaining laboratory test specimens from some of these blocks, due partly to their irregular shape, and also partly to the presence of narrow bands of shale which broke up the specimens when they were being made. In view of this, it was decided to concentrate on obtaining test specimens from those parts of each seam which it was thought would have the most influence on the stability of the workings, i.e. the roof, the floor and the mid-section of the seam

17.1 Determination of the Tensile Strength.

The nature of the sample is usually the determining factor in the choice of the method of measuring tensile strength in the laboratory. In this case, in order to obtain compressive test specimens from the block samples, the blocks were cored with the result that the most convenient tensile test specimen obtainable was a disc. Discs of 3 in. diameter and 1 in. width were prepared from three sections in each seam, These were the sections around the roof and floor of the seams, and a section at approximately the mid-height of each seam. The block samples were cored perpendicular to the bedding so that the disc specimens obtained were aligned in such a way that the determined tensile strength value was on the same plane as the seam, i.e. in the plane in which tensile stresses can be expected. The Brazilian Disc Test was used to determine the ultimate tensile strength of the gypsum specimens.

17.1.1 Results Obtained For No.1 Seam Gypsum.

A total of 20 disc test specimens were obtained from the block samples referred to as S1-1, S1-5, S1-7 and S1-8, the samples S1-1 and S1-8 being taken as representing the floor and roof of the seam respectively. The tensile strength values obtained from the testing of these discs are shown in Table 13.

It can be seen from Table 13 that there is little variation in the mean tensile strength of each group of discs obtained from the blocks, though there was some scatter in the individual tensile strength values of the specimens in each group. This type of scatter however, is commonly found in the results of tensile strength determinations and has been commented on by other investigators.

The disc specimens tested from the floor sample gave a fairly wide range of tensile strengths, between 356 - 584 p.s.i., and a mean value of 486 p.s.i. Those taken from blocks S1-7 and S1-8 showed very little scatter in the results and gave almost identical mean tensile strength values. This was not to be expected from a visual examination of the blocks. The block S1-8 contained a great deal of calcareous shale whilst block S1-7 appeared to be made up mainly of large crystals of gypsum and anhydrite. These strength values indicate that the part of the seam of which the tensile strength is most important, viz. the roof and floor seem to be as strong in tension as the actual seam material.

17.1.2 Results Obtained for No.4 Seam Gypsum.

In this case 19 disc specimens were tested, being obtained from the block samples referred to as S4-1, S4-4, S4-5, S4-9 and S4-10, with sample S4-1 representing the floor of the seam as before. A sample of the roof material, S4-11 was received but this sample contained a particularly wide shale band which prevented the block being cored. Block samples S4-9 and S4-10 were very similar in appearance to S4-11, but should not be considered as being representative of the actual roof material, though the thickness of the seam left

TABLE 13

Sample Reference Number	No. of Specimens	Tensile Strength Range p.s.i.	Mean Ultimate Tensile Str. p.s.i.
S1 - 8	4	500 - 568	517
S1 - 7	6	453 - 542	503
S1 - 5	5	413 - 597	489
S1 - 1	5	356 - 584	486

TABLE 14

Sample Reference Number	No. of Specimens	Tensile Strength Range p.s.i.	Mean Ultimate Tensile Str.p.s.i.
S4 - 10	4	350 - 373	362
S4 - 9	5	355 - 402	393
S4 - 4	1	-	413
S4 - 5	4	311 - 478	414
S4 - 1	5	320 - 414	365

to form the roof of the workings does vary. The results of these tensile strength tests are shown in Table 14.

When these results are compared with those obtained from the testing of No.1 Seam blocks, it can be seen that the mean tensile strength value calculated for each block sample is approximately 25% lower than those obtained for the No.1 Seam block samples.

The discs obtained from the mid-seam block samples of No.4 Seam have a slightly higher mean value than those from the floor and near the roof of the same seam, though there is a much wider tensile strength range for these mid-seam samples. The comparatively low tensile strength of 311 p.s.i.for a disc from block S4-5, the other three discs from the same block giving values greater than 400 p.s.i., may indicate possible weak layers in the seam at this level and should be compared with compressive strength values of specimens from the same horizon.

Disc specimens taken from the floor sample S4-1 gave a mean tensile strength value of 365 p.s.i. which was almost identical to the mean value obtained for block sample S4-10 of 362 p.s.i., though there was much less scatter in the individual strength values of discs from S4-10.

17.2 Determination of the Uniaxial Compressive Strength.

The most commonly determined property of a rock material used as a measure of the stability of an underground opening formed in the material to be tested, is its uniaxial compressive strength. The laboratory test used to determine this value is based on the assumption that the uniaxial compressive strength of laboratory samples is representative of the rock mass. This strength value, however, is only meaningful where the boundary loading in the lateral direction is zero, and a complete comprehension of the behaviour of a rock material can only be obtained by further testing in a triaxial state of stress. The results obtained from the triaxial compressive strength tests are described in the following section 17.3.

Uniaxial compressive strength tests were carried out on cylindrical test specimens cored from the block samples. So that the results obtained could be correlated more easily, the same size test specimen was used for both uniaxial and triaxial testing. This meant that the test specimens needed to be 2.15/16 in. diameter and W/H ratio = 0.5, the reasons for this will be described later.

It is quite usual to find during an investigation into the compressive strength of a rock material that the apparent strength increases as the W/H ratio increases. The reason for this was discussed in an earlier part of this thesis. In this investigation however, insufficient test specimens could be made from the block samples available to determine the extent the uniaxial compressive strength of the seam material varied as the W/H ratio changed. Since there was an obvious geological difference between the block samples obtained from any one seam, it was thought preferable to carry out individual compressive strength tests on a uniform size test specimen so that the results could be compared directly between the block samples. A test specimen of W/H ratio = 0.5 was therefore used for all uniaxial compression testing carried out on the Brightling gypsum.

17.2.1 Preparation and Testing of the Specimens.

Cylindrical test specimens, 2.15/16 in. diameter were cored from the block samples. Because of the required specimen size, the number obtainable from any one block sample was limited. The blocks were examined before coring to determine the direction of bedding and were cored normal, or as near to normal to the bedding as was possible depending on the shape and size of each block. A diamond tipped coring bit was used throughout. Water was tried as a lubricant but with little success, the diamond edges of the coring bit becoming clogged with fine gypsum cuttings and mud, which reduced the rate of coring as well as producing a poor surface finish. Paraffin was therefore used as the drilling lubricant for cooling the bit and for flushing out the chips.

The cylindrical specimens were then cut to a height equal to twice the diameter and the end surfaces ground plane and parallel to within 0.002 in.

All specimens were then tested to failure by loading between ground steel platens in a hydraulic testing machine, at a loading rate of between 10 - 20 p.s.i./sec. as recommended by the I.S.R.M. (50). In addition to obtaining data on the uniaxial compressive strength of the seam material, information relating to certain other mechanical properties of the seam material were obtained. This data will be described later together with a more detailed description of that particular investigation.

17.2.2. Results Obtained for No.1 Seam Gypsum.

Five compressive test specimens of No.1 Seam gypsum were tested, these being taken from the blocks S1-1, S1-5 and S1-8. The uniaxial compressive strength of each speciman tested is shown in Table 15 for zero confining pressure. The fractured specimens are shown in the photographs, Figs. 99 - 101.

It can be seen that the results varied over a fairly small range only, the five specimens giving compressive strength values between 4763 p.s.i. and 5110 p.s.i. This gave a mean uniaxial compressive strength for the seam as a whole, of 4950 p.s.i. This uniformity in the results reflects, to some extent, the tensile strength values obtained for the same material. The lowest strength value of 4763 p.s.i. was obtained for the specimen taken from the floor sample, S1-1.

TABLE 15

RESULTS OF COMPRESSIVE STRENGTH DETERMINATIONS CARRIED OUT ON CYLINDRICAL SPECIMENS 2.937" DIAMETER, 5.875" LONG, CUT FROM BLOCKS REMOVED FROM THE WORKING ZONES AT BRIGHTLING.

SEAM 1

Location	Confining Pressure p.s.i.	Ultimate Compressive Strength p.s.i.	Mean Compressive Strength p.s.i.
S1 - 1		4763	
		4942	
S1 - 5	0	5012	4950
		5110	
S1 - 8		4929	
S1 - 1		9239	
S1 - 5	1500	8965	9318
S1 - 8		9660	
S1 - 1		11016	
S1 - 5	2500	11214	11523
S1 - 8		12339	

TABLE 16

RESULTS OF COMPRESSIVE STRENGTH DETERMINATIONS CARRIED OUT ON CYLINDRICAL SPECIMENS 2.937" DIAMETER, 5.875" LONG, CUT FROM BLOCKS REMOVED FROM THE WORKING ZONES AT BRIGHTLING.

SEAM 4

Location	Confining Pressure p.s.i.	Ultimate Compressive Strength p.s.i.	Mean Compressive Strength p.s.i.
S4 - 1		3854	
S4 - 4	0	4168	
S4 - 5		2812	3894
S4 - 10		4747	
S4 - 1		8849	
S4 - 5	1500	8138	8849
S4 - 10		9560	
S4 - 1		11297	
S4 - 5	2500	10255	11165
S4 - 10		11942	
S4 - 5	3000	11380	

17.2.3 Results Obtained for No.4 Seam Gypsum.

The four specimens tested in uniaxial compression were taken from each of samples S4-1, S4-4, S4-5 and S4-10. The strength values obtained are shown in Table 16 for zero confining pressure. The fractured specimens are shown in the photographs, Figs. 102 - 104.

In this case, the strength values varied over a wider range giving values between 2812 p.s.i. and 4747 p.s.i.. The specimen from block S4-5 gave the lowest compressive strength of 2812 p.s.i., the failure of this specimen obviously being influenced by the occurrence of a number of satinspar bands in the specimen. This can be seen in the photograph, Fig.103. A tensile disc specimen taken from the same block had already given a relatively low tensile strength value. This is further evidence which could confirm the presence of a weak layer in the seam at this level and must be looked for in the results of the triaxial compression testing.

The compressive strength of the specimen taken from the floor sample S4-1 of 3854 p.s.i., was lower than that obtained for the specimens taken from the blocks S4-4 and S4-10 of 4168 and 4747 p.s.i. respectively.

The mean uniaxial compressive strength of the seam as a whole, calculated from these four strength values, was only 3894 p.s.i., which is approximately 25% lower than the same value obtained for No.1 Seam. This is comparable with the difference between the mean tensile strength values calculated for each seam.

17.3 Determination of the Triaxial Compressive Strength and Shear Strength.

While the results obtained from uniaxial compression testing are useful, it is generally accepted that triaxial testing is the best

10,000 p.s.i. TRIAXIAL PRESSURE CELL

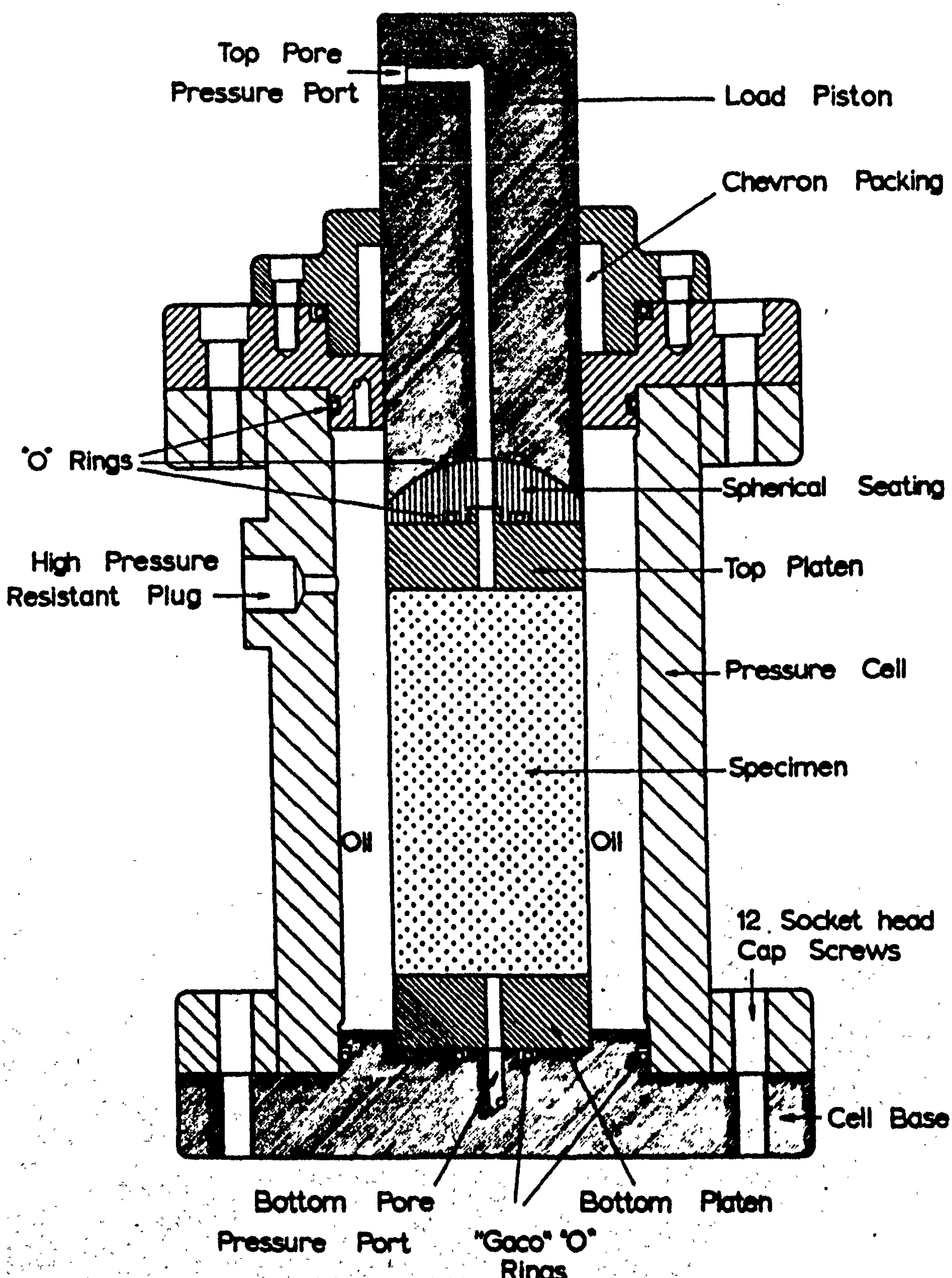


Fig. 97

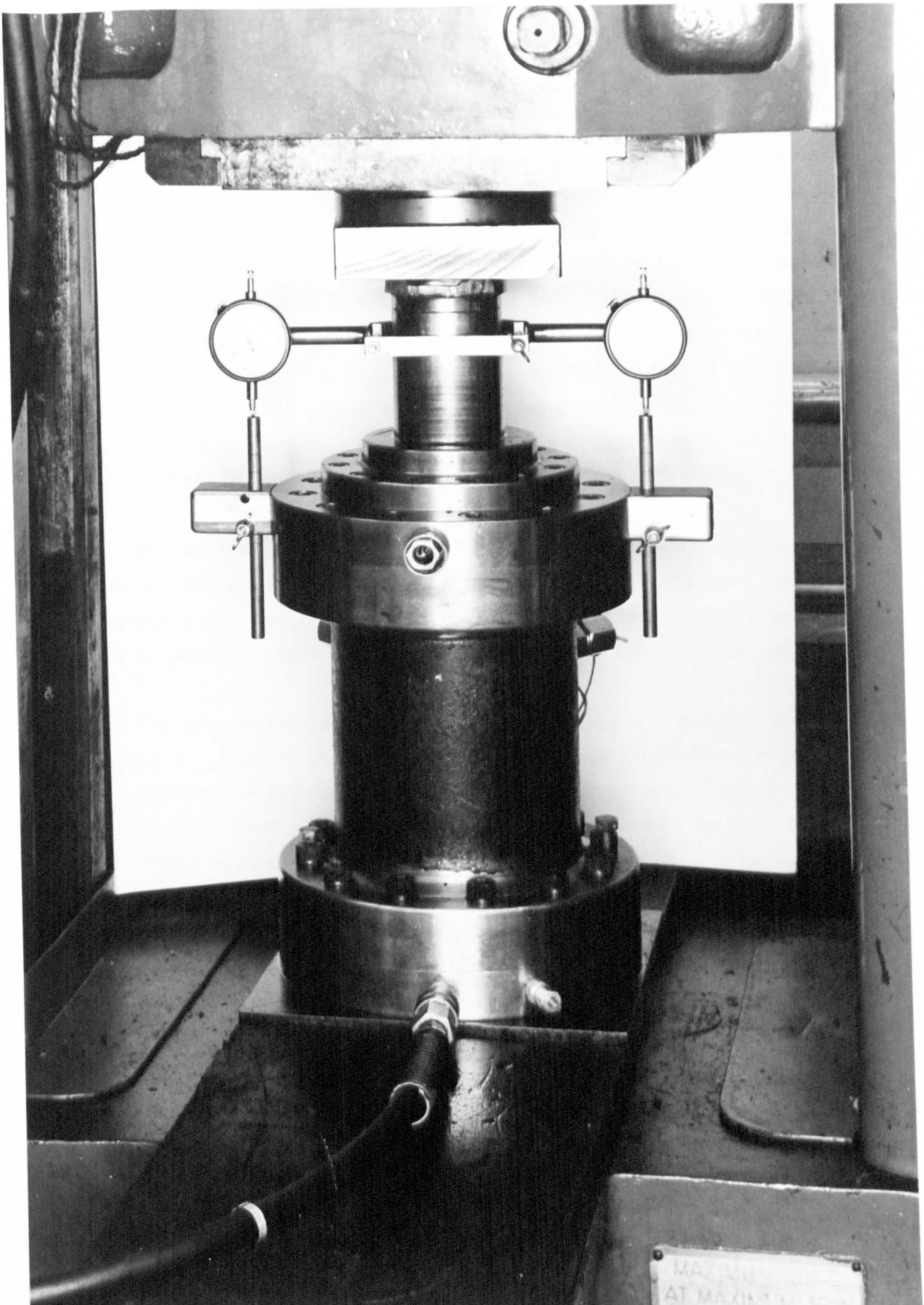


Fig. 98

available method of investigating the mechanical behaviour of rock material in close simulation to 'in situ' conditions. This test is carried out in an enclosed cell, in this case a cell specially designed for the triaxial testing of evaporite rocks. This cell was described in some detail by Buzdar (49). A section through the triaxial cell is shown in Fig.97. The test specimen is subjected to a uniform lateral pressure, ideally corresponding to the boundary conditions that can reasonably be expected under field conditions, and then loaded axially to failure. This triaxial cell was designed for a specimen size of 2.15/16 in. diameter and W/H ratio of 0.5.

Triaxial tests carried out for several values of confining pressure are generally presented by a Mohr-Stress diagram. Such diagrams have been obtained in this investigation and have been used to obtain a value for the shear strength of the seam material.

The preparation and size of specimen used has already been described. Each specimen was encased between rigid steel platens in a protective rubber sleeve. This was to protect the specimen from the oil used to apply the confining pressure. Because of the remarkably high ductility of the gypsum, the rubber sleeve was pierced in some cases after failure, and the specimens became soaked with oil. The axial load was applied at a loading rate of 10 - 20 p.s.i./Sec. by a 100 ton hydraulic testing machine, the lateral confining pressure being kept constant at a pre-determined value. A photograph of the triaxial cell under test is shown in Fig.98.

17.3.1 <u>Results obtained for No.1 Seam Gypsum.</u>

A total of six specimens were tested, two specimens being taken from each of blocks S1-1, S1-5 and S1-8, as were the uniaxial compression test specimens. One specimen from each block sample was tested to failure at 1500 p.s.i. and 2500 p.s.i. confining pressure. The strength results obtained are

Fig. 99

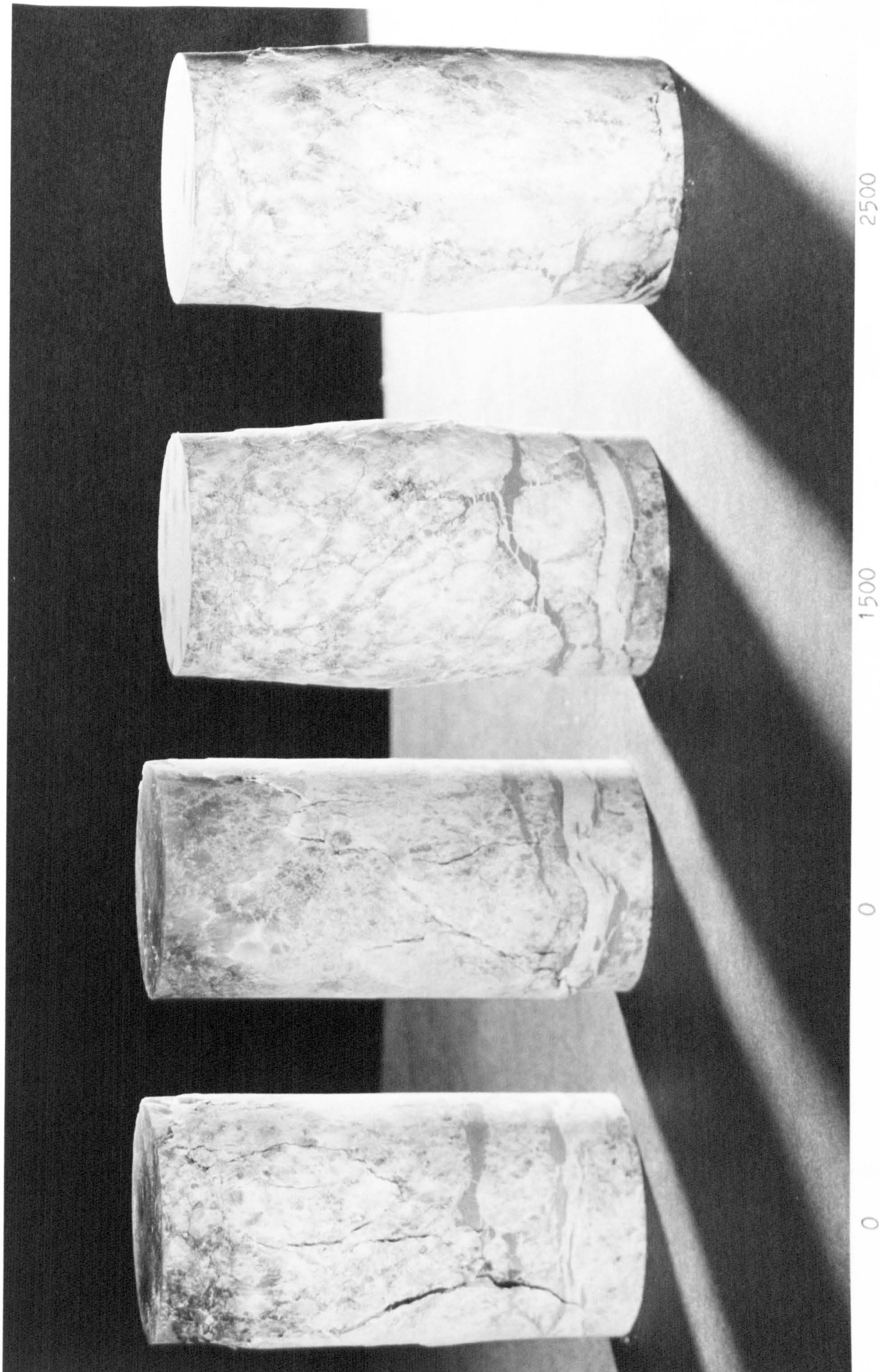


Fig. 100

Fig. 101

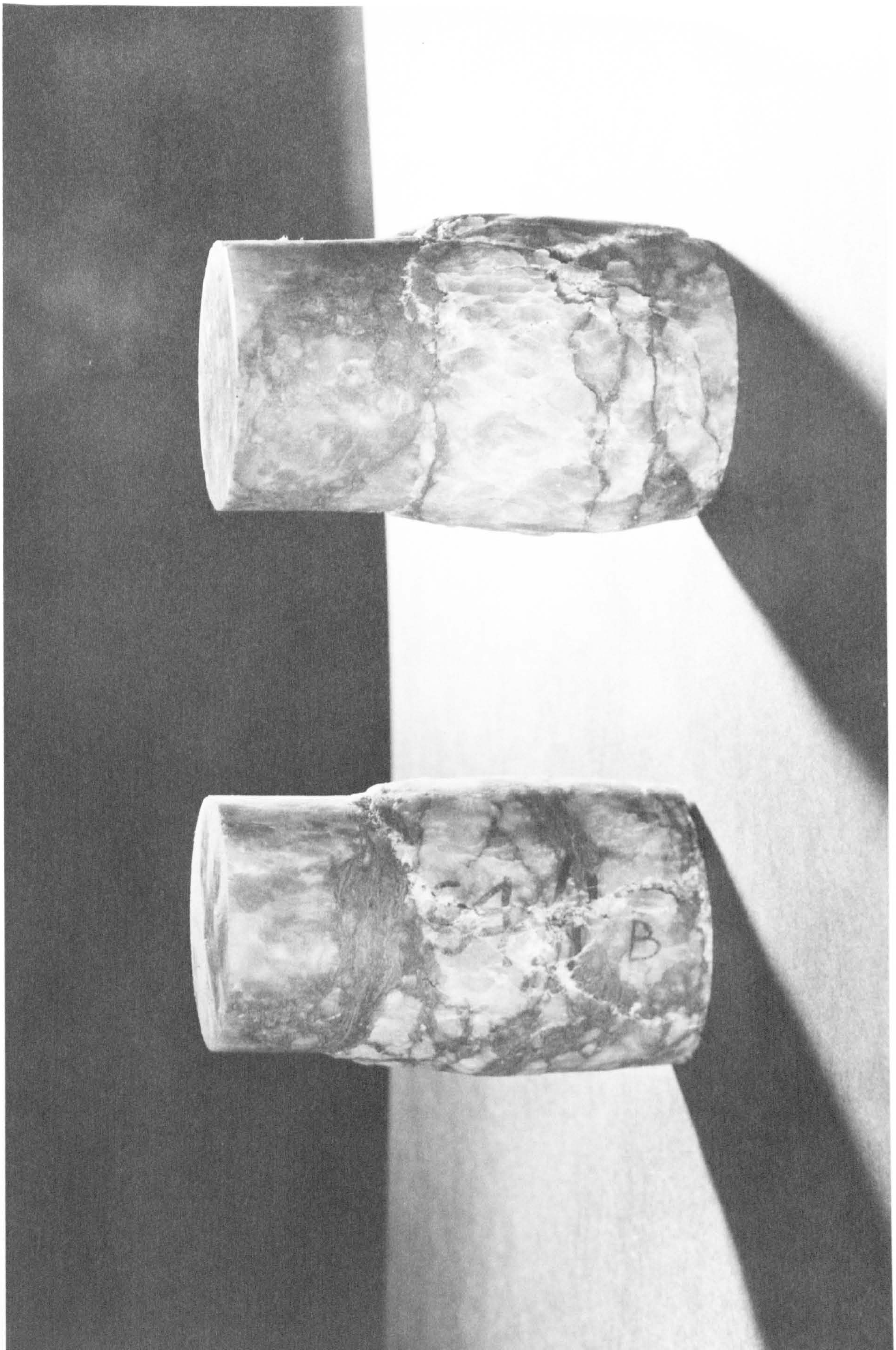
tabulated in Table 15. The strength data for the different confining pressures has been expressed in terms of a Mohr Stress Diagram, one diagram drawn for each block sample as shown in Fig.105. Photographs of the fractured specimens are shown in Figs.99 - 101.

It can be seen that there was a considerable increase in the compressive strength of the seam material as the confining pressure increased from 0 to 2500 p.s.i.. The state of stress existing in each specimen just before failure is depicted in the Mohr stress diagrams by a Mohr Stress circle. A Mohr envelope curve has been drawn as a tangent to these circles and the intercept of the envelope with the shear axis has given a value for the shear strength of each block sample. From this, a mean shear strength for No.1 Seam material was calculated to be 1360 p.s.i.

The Mohr envelopes drawn for the individual blocks S1-1, S1-5 and S1-8, indicate that whilst the blocks S1-5 and S1-8 have virtually straight line envelopes, the envelope drawn for block S1-1 tends to be more curvilinear. This would seem to suggest that the material at the floor level in the seam is much more ductile than the remainder of the seam material. This is further indicated by the photographs of the fractured specimens. The amount of plastic deformation of the specimen S1-1 in Fig.99 due to the moderately high confining pressure of 2500 p.s.i., is clearly shown, whilst those specimens from blocks S1-5 and S1-8, Figs.100 and 101, show very little similar deformation.

17.3.2 Results Obtained for No.4 Seam Gypsum.

Seven specimens were tested from No.4 Seam samples at up to 3000 p.s.i. confining pressure. Two specimens from each of the samples S4-1 and S4-10 were tested at 1500 p.s.i. and



2500

1500

Fig. 102

Fig. 103

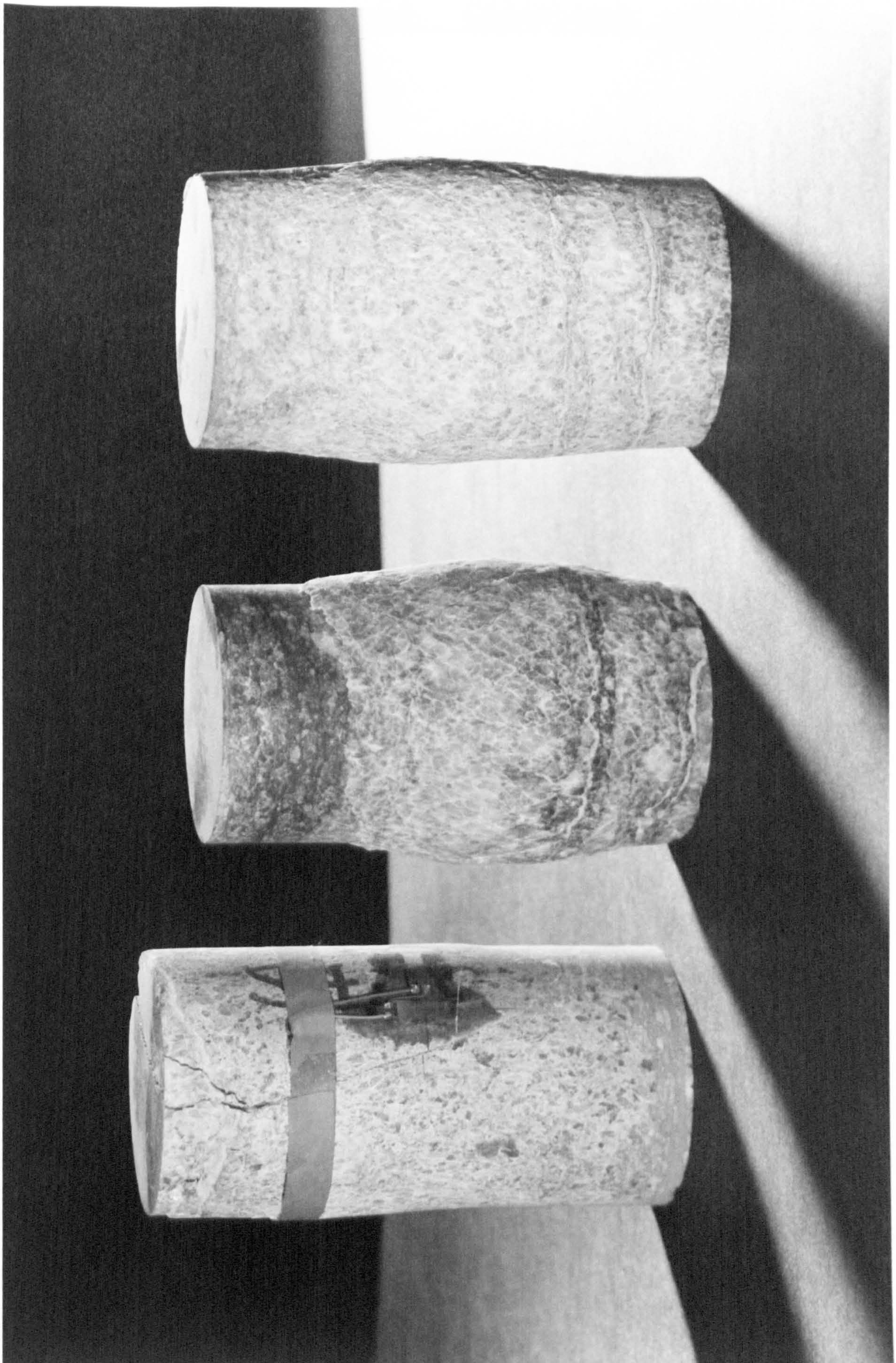
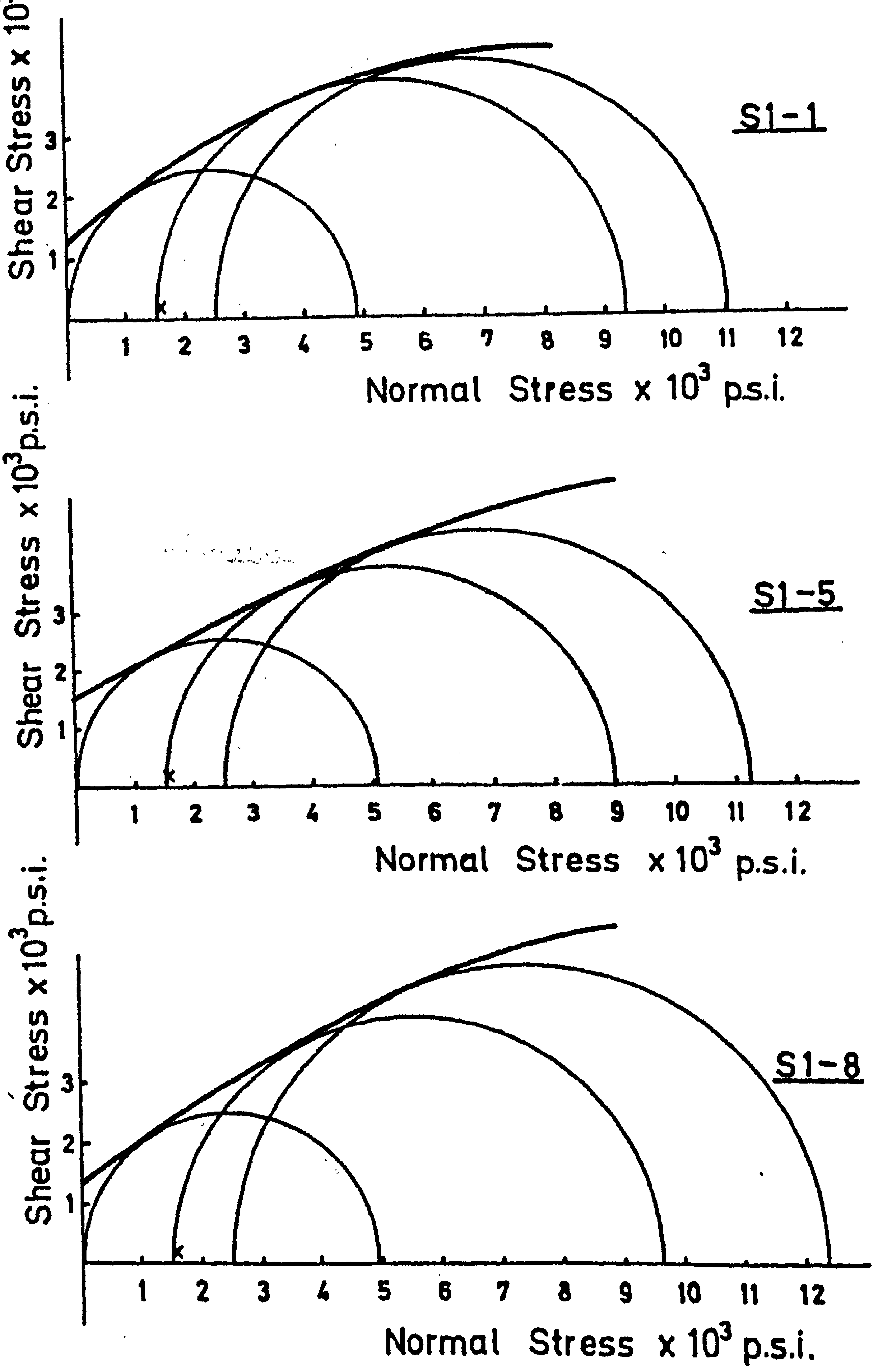


Fig. 104

MOHR'S STRESS DIAGRAMS - SEAM 1



X - Calculated pillar stresses

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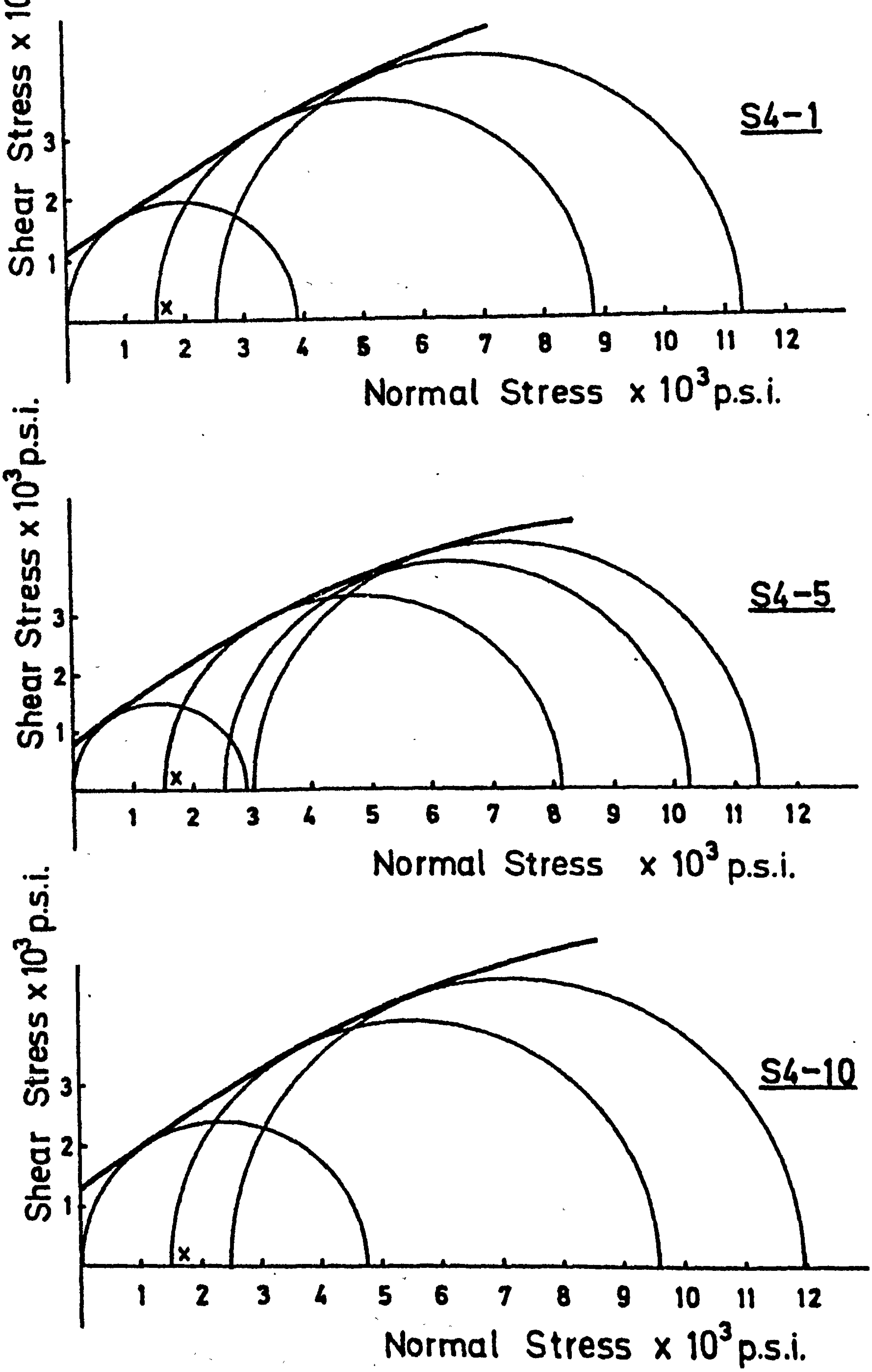
FIG. 105

MOHR'S STRESS DIAGRAMS - SEAM 4



X - Calculated pillar stresses

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FIG. 106

2500 p.s.i. confining pressure. Three specimens from sample S4-5 were tested at 1500 p.s.i. and 3000 p.s.i. confining pressure. The strength results obtained are shown in Table 16. A Mohr-stress diagram has been drawn from each block as for the No.1 Seam samples, and shown in Fig.106. Photographs of the fractured specimens are shown in Figs.102 - 104.

It can be seen from the results that throughout the range of confining pressures used, the specimens obtained from block sample S4-5, taken from the mid-height of No.4 Seam, failed at a lower load than other specimens from the same seam. This fact was also suggested by the tensile strength results described in section 17.1.

A Mohr envelope curve has been drawn for each block sample giving a mean value for the shear strength for No.4 Seam material of 1070 p.s.i. The block samples gave virtually straight line envelopes though that drawn for sample S4-5 tended to be slightly curvilinear, due probably to the presence of the white satinspar bands which obviously influenced the failure of these specimens. This can be seen in the photograph Fig.103. The photographs, Figs.102 and 104, show the fractured specimens from sample S4-1 and S4-10 respectively. These three photographs indicate clearly the variation in the seam material and also the ductile nature of the material at what are after all, relatively low confining pressures.

17.4 Determination of the Elastic Constants of Brightling Gypsum.

A number of tests were carried out to obtain information concerning the stress-strain relationship of Brightling gypsum. From this relationship, values for the various Young's moduli and Poisson's ratio of the seam material were obtained. It was hoped that these values, in particular the values of Poisson's ratio, could be used in the theoretical work carried out.

A number of specimens were tested from both seams to determine these constants, the same specimens in fact being used to determine the uniaxial compressive strength values. In order to determine the values for Poisson's ratio, the specimens were fitted with two pairs of electrical resistance strain gauges at the mid-height of the specimen, each pair set 180° apart. These gauges were arranged so that they measured the specimen deformation parallel and normal to the direction of loading. The specimen was then loaded uniaxially to a pre-determined stress and then unloaded. This cycle was repeated three times. At instants during the loading and unloading stages the magnitude of stress was noted and the longitudinal and lateral deformation of the specimen measured. In addition, the longitudinal deformation of the specimen was measured using a dial gauge. A stress-strain curve was drawn using both strain gauge and dial gauge deformation results. A value for the Poisson's ratio was obtained from the strain gauge curve, and a value for Young's modulus determined from the dial gauge deformation curve.

The range of values obtained for the modulus and Poisson's ratio of the gypsum taken from the two seams is shown below, and a mean value of each for both seams, has been calculated.

a) SEAM 1.

Five specimens taken from No.1 Seam blocks yielded values for the modulus, three of these specimens also being fitted with strain gauges to give values for Poisson's ratio for the seam material.

Young's modulus	:	Range	2.49×10^{6} - 3.05×10^{6} p.s.i.
		Mean	$2.78 \times 10.^{6}$ p.s.i.
Poisson's ratio	:	Range	0.24 - 0.29
		Mean	0.27

b) SEAM 4.

Four specimens gave values for the modulus of the seam material, three of these specimens also being fitted with strain gauges to give values for Poisson's ratio.

Young's modulus	:	Range	2.3×10^6 - 2.98×10^6 p.s.i.
		Mean	2.62×10^6 p.s.i.
Poisson's ratio	:	Range	0.29 - 0.32
		Mean	0.31

These values for the elastic constants of the Brightling gypsum were obtained from subsidiary tests carried out during the uniaxial compressive strength test. The results obtained indicated the mode of deformation of the seam material under a compressive load and gave representative values for the Young's modulus and Poisson's ratio of the gypsum. It can be seen that tha mean value of the modulus determined compares very well for the two seams, though the Poisson's ratio values were in all cases slightly higher for No.4 Seam.

18. MODEL STUDY METHODS OF INVESTIGATION INTO THE PROBLEM OF PILLAR STABILITY AT THE BRIGHTLING MINE.

In order to provide an acceptable solution to the problem, a number of laboratory methods of investigation have been used. The main reason for this approach in the simplifying assumptions which have to be introduced in both mathematical and scale model studies, to bring the problem within the realm of practicability. It was hoped that from the results of the various solutions, which should be complementary to each other, definite conclusions could be reached as to the relative positions of the pillars in the two seams. The particular approach used in this instance is one similar to that used by Trumbachev and Mel'nikov (76) who used model tests to provide a basis for determining the effect of dip angle on pillar stresses.

A common feature of the methods used in this investigation is that a criterion governing the superposition of the pillars is established which is to ensure that the strata layer between the two seams is left in the most favourable state of stability. The assumptions regarding the magnitude of the primitive stressfield due to the average cover load, are also common for each model used in the investigations. The vertical stress component of this stressfield is assumed to beinduced by the weight of the superincumbent strata. Its magnitude is taken to be 1 p.s.i./ft. depth. In general, the lateral stress components acting on an element of the rock are due to the lateral constraint of the strata compressed by the vertical stress component. The magnitude of the lateral stress components is given by the relationship :-

$$\sigma_x = \sigma_y = K\sigma_z$$

where σ_x and σ_y are the lateral stress components, σ_z is the vertical stress component, and K denotes a proportionality factor which is dependent on the properties of the rock material.

As will be seen later, the lateral components of the primitive stressfield, under normal conditions, do not influence the stability of a pillar when the seam is level or very slightly inclined. Nevertheless, when the inclination of the seam is moderately high, as in the case under consideration, the magnitude of the lateral stress components will probably have an important effect on the pillar stability. It is imperative therefore, that some consideration should be given to the estimation of the magnitude of the lateral primitive stress components.

Under ideal conditions the proportionality factor K, is expected to be in the order of 0.33. When, due to the existence of fractures in the rock, the lateral constraint is absent, the magnitude of the horizontal stress may be equal to zero, or to a small fraction of the vertical stress component. If on the other hand the rock material is capable of plastic deformation, the horizontal stress components may have the same magnitude as the vertical stress component. When the possibility of the existence of tectonic forces is considered, the assessment of the initial state of stress in the rock is more difficult. If the folded and faulted state of the strata in the mining area of Brightling is considered, then the choice of the magnitude of the primitive stress components becomes even more arbitrary.

In order to overcome these difficulties both the mathematical and the model studies have been carried out assuming a number of cases in which the magnitude of the lateral primitive stress component varies between 0 and 0.5 σ_z. The analysis has been carried out for several of these values, including that value of K determined from the laboratory rock testing, and from the trend of these solutions the effects of the variation of the lateral stress component have been deduced.

The first approach described is an analytical appreciation of the problem using a simple mathematical model. The variation of the forces acting on a pillar formed in an inclined seam perpendicular to the stratification have been considered when the inclination of the seam and the magnitude of the lateral primitive stress vary. Then the criterion of the superposition is discussed and the mathematical relationships are deduced. Also, the possible normal and shear stress components acting on the pillars have been calculated and compared with the compressive and shear strength values of the rock material obtained in the laboratory, Finally, the calculated results are analysed.

The second method of investigation consists of a two-dimensional photoelastic model study. This model has been used to study the influence of the angle of dip of the workings and the variation of the lateral stress component upon the stressed condition of the pillars and surrounding rock. The principles governing the making of the model, the method of analysis and the results obtained are described and discussed.

18.1 An Appreciation of the Pillar Stability by Means of a Mathematical Model Study.

The distribution of stress in a pillar formed in a level seam is rather complex. A number of factors such as the geometry of the pillar, the inter-effect of the mechanical properties of the seam and the surrounding rock materials, and the constraining effect of the roof and floor of the pillar, makes the mathematical analysis of the pillar stability intractable. In general, all these factors, which have been discussed in some detail elsewhere, are ignored and by introducing the assumption that the pillars uniformly support the entire load of the rock overlying both the pillars and excavated area, an average pillar stress is calculated and used in the pillar stability considerations. The average stress, in this case, is dependent upon the magnitude of the vertical primitive stress component and the extraction ratio. When the stability of pillars in an inclined seam is considered, the additional loading due to the lateral primitive stress component in the direction of the inclination, is also taken into account. Although the mathematical description of the problem employed is far from being precise, it gives some indication of the influence of the seam inclination on the external forces expected to act on the pillars.

Consider first the forces acting in a plane section through one of the square pillars in No. 1 Seam as shown in Fig. 107. It is assumed that the principal primitive stresses are vertical and horizontal and that the horizontal components are equal in all directions. The vertical primitive stress is due to gravitation while the horizontal components are a constant proportion K of the vertical component.

It follows from the equilibrium of the triangular roof element that :-

$$A_1 \ \sigma_z \ \cos \alpha = A_2 \ \sigma_V \qquad (22)$$

$$A_1 \ \sigma_z \ \sin \alpha = A_2 \ \sigma_H \qquad (23)$$

THE FORCES ACTING ON A PILLAR IN AN INCLINED SEAM

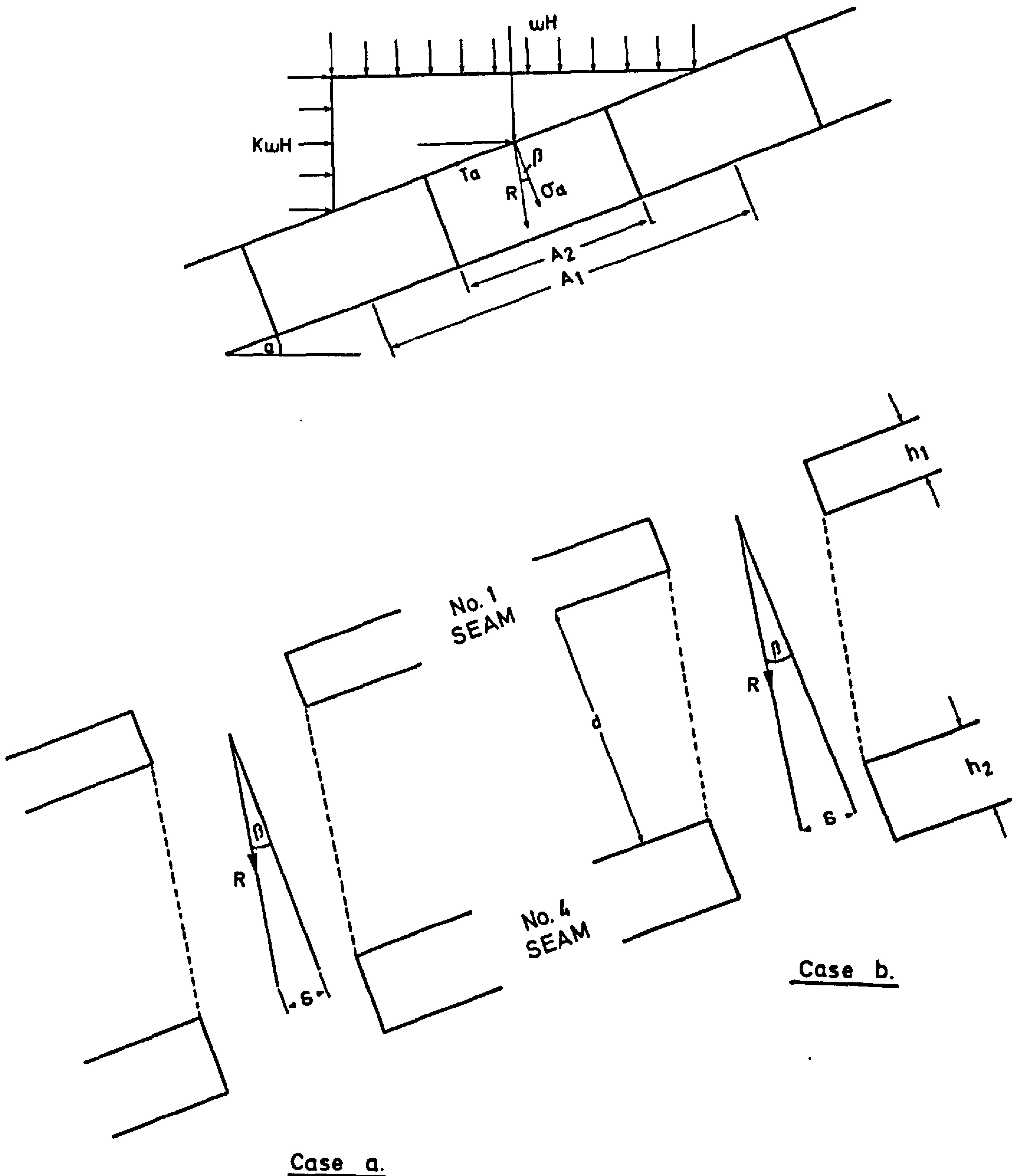

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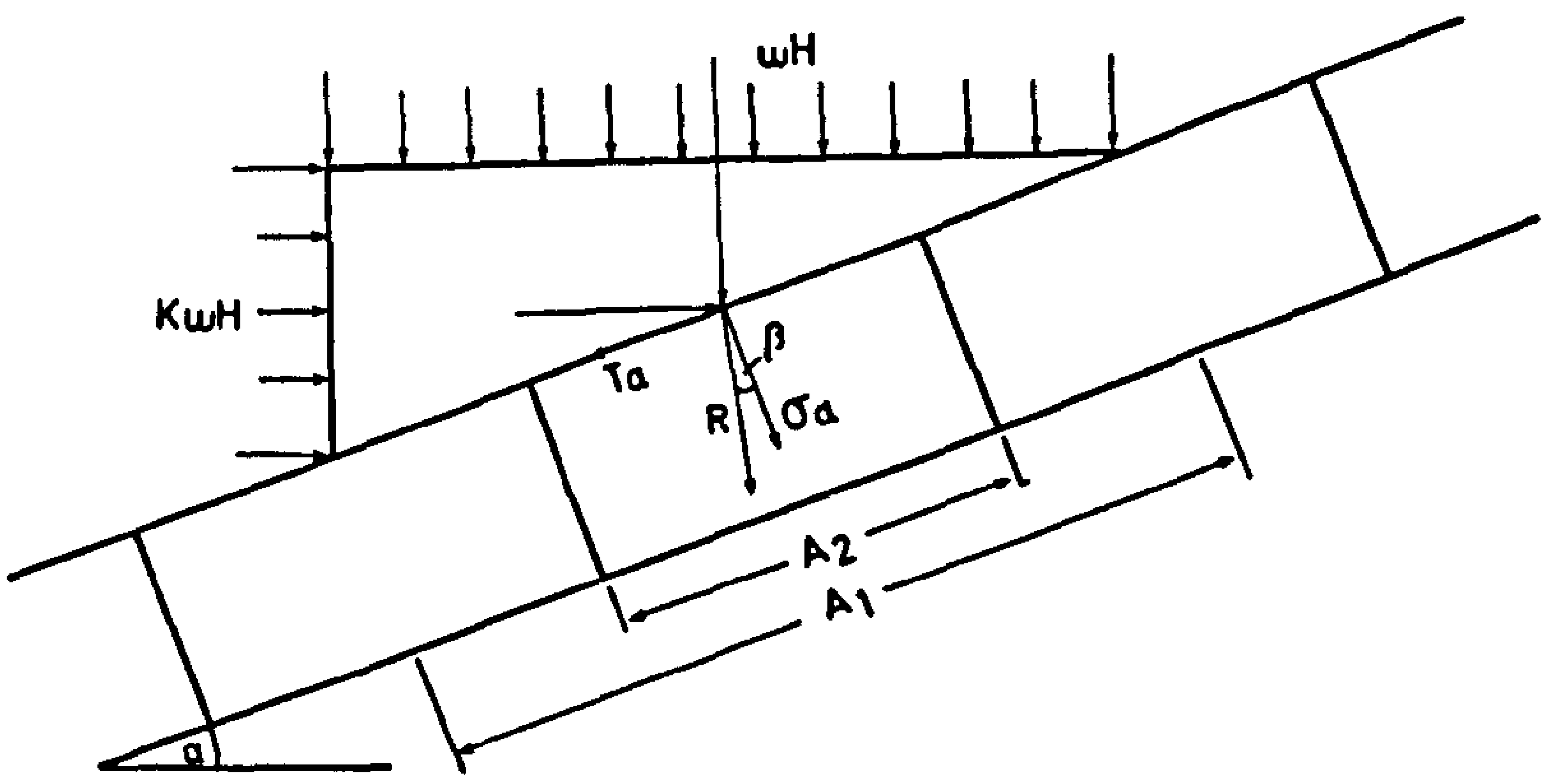
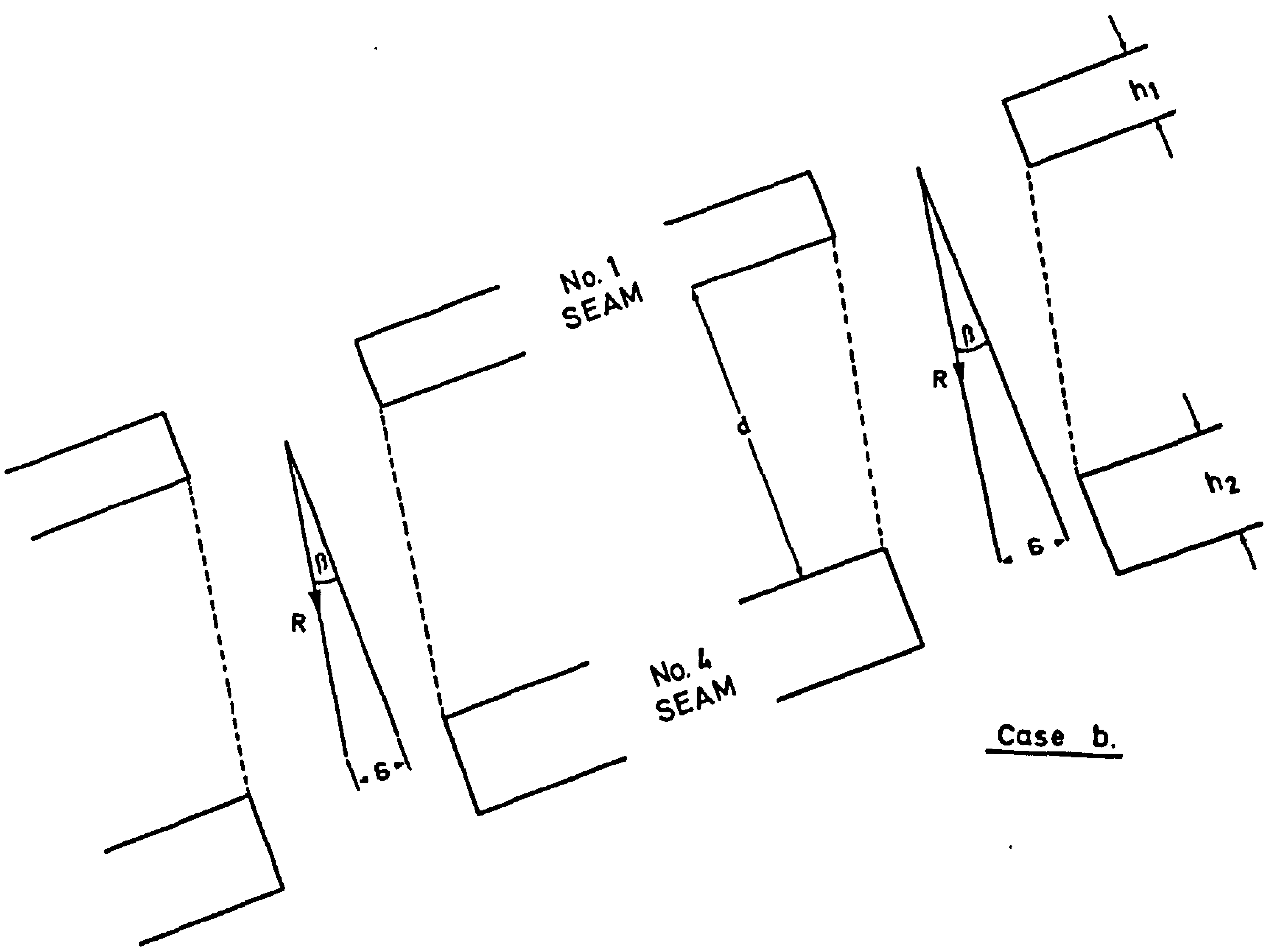
FIG. 107

THE FORCES ACTING ON A PILLAR IN AN INCLINED SEAM





Case a.

Case b.

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FIG. 107

A_1 and A_2 denote linear distances in two dimensions as shown in Fig.107, but in a broader interpretation they represent surface areas.

σ_z is the vertical stress component assumed to be induced by the weight of the superincumbent strata.

σ_V and σ_H are the vertical and horizontal stress values at the point shown on the pillar, caused by the weight of the superincumbent strata and the lateral forces acting on the pillar respectively.

α is the inclination of the seam.

K is the proportionality factor of the vertical to lateral primitive stress components.

After resolving into components in the plane of the roof and normal to it, the resultant stress components are :-

$$\sigma\alpha = \frac{A_1}{A_2} \sigma_z (\cos^2 \alpha + K \sin^2 \alpha) \qquad (24)$$

$$\tau\alpha = \frac{1}{2} \frac{A_1}{A_2} (1 - K) \sin 2\alpha \qquad (25)$$

where $\sigma\alpha$ and $\tau\alpha$ are the normal and shear stress components expected to act on the contact surfaces of the roof element and the pillar if both of these could be assumed to be rigid bodies in contact. It can be seen that both $\sigma\alpha$ and $\tau\alpha$ are functions of K and α .

These two stress components given by equations 24 and 25 can now be replaced by their resultant which is assumed to act at an angle β from the normal as shown in Fig.107. The value of β

is given by the expression :-

$$\beta \quad = \quad \arctan \frac{\frac{1}{2}\,(\,1 - K\,)\,\sin 2\,\alpha}{\cos^2 \alpha \;+ K \sin^2 \alpha} \qquad (26)$$

The pillar therefore, is subjected to a compressive stress field, the direction of which is determined by the angle β . Hence the load due to the superincumbent strata can be assumed to be transferred by the pillar to the underlying floor. This stressfield in the floor is redistributed to the inter-seam strata and subsequently transferred to the pillars in the lower seam.

It is the aim of this investigation to determine the best relative positions of the pillars in the two seams so that the resultant stress be transferred as directly as possible through the pillars in the lower seam to the floor. To effect this direct transference of load, the pillars in the lower seam should be formed in such a way that they would be in the direct line of the resultant stressfield, the direction of which is determined by the angle β . The distance by which the pillars in the bottom seam have to be displaced, relative to the pillars in the top seam, may be calculated in one of the following two ways :-

Case (a) By using the method shown in Fig.107 where the points in the pillar centres at the floor level of the upper seam and the roof level of the lower seam, are brought in line with the direction of the resultant stress. Using the angle β , the distance of advance of the lower pillars towards the dip, referred to as δ , can be calculated.

Case (b) When the pillar heights are also considered in the calculations, the pillars are superimposed so that the direction of the resultant stress goes through the pillars centres as shown in Fig.107.

The mathematical expressions for δ for these two cases are :-

Case (a) $$\delta = d \frac{\frac{1}{2} (1 - K) \sin 2 \alpha}{\cos^2 \quad + K \sin^2 \alpha} \qquad (27)$$

Case (b) $$\delta = (\frac{h_1 \quad + h_2}{2} + d) \frac{\frac{1}{2} (1 - K) \sin 2 \alpha}{\cos^2 \alpha + K \sin^2 \alpha} \qquad (28)$$

In the above equations h_1 and h_2 represent the working height of the two seams, and d denotes the distance between the seams.

It is now possible therefore, to calculate the displacement of the lower seam pillars relative to the pillars in the upper seam for the two cases described, using various values for the proportionality factor K and angle of dip of the workings α . It is also possible to calculate, using equations 24 and 25, the possible normal and shear pillar stress components for various values of K and α , and to compare these values with the compressive and shear strength values of the rock material obtained in the laboratory. The following information is used in these calculations :-

The roadway width in both seams		=	20 ft.
The pillar height in No.1 Seam	h_1	=	7 ft.
The pillar height in No.4 Seam	h_2	=	10 ft.
The pillar width in both seams		=	20 ft.
The distance between the two workings	d	=	30 ft.
The depth of No.1 Seam workings	H	=	400 ft.
Area of ground assumed to be supported by one pillar	A_1	=	$(20 + 20)^2$ ft.
Pillar cross-sectional area	A_2	=	20 x 20 ft.

The vertical stress component $\sigma_z = \omega H$

where ω = volumetric load/foot depth of strata. (assumed to be unity in this calculation).

Poisson's ratio for No.1 Seam = 0.27

Poisson's ratio for No.4 Seam = 0.31

Angle of inclination of workings at Brightling = 1 in. 5 or $11\frac{1}{2}^{\circ}$

By submitting the relevant data in the various equations it is possible to calculate, firstly the relative pillar displacement for the two cases, and secondly the pillar normal and shear stress components.

18.1.1 Determination of the Relative Pillar Displacement.

The two cases in which the pillars in the bottom seam are displaced relative to those in the top seam have already been described. The calculations carried out and shown here are for one set of circumstances only, that of the angle of dip of the working at 11° 30', and a value for K, the proportionality factor, of 0.41, which has been calculated by taking a value for Poisson's ratio of 0.29, the mean of that determined in the laboratory for the two seams.

Case (a) This case assumes that the direction of the resultant stress passes through the middle of the pillar at the floor level of No.1 Seam, and the middle of the pillar at the roof level of No.4 Seam. This is shown in Fig. 107.

Using the expression 27. The relative pillar displacement

$$\delta = d\,\frac{\frac{1}{2}(1 - K)\sin 2\alpha}{\cos^2\alpha + K\sin^2\alpha} \qquad (27)$$

$$= 30 \frac{\frac{1}{2} (1 - 0.41) \sin 23^{\circ}}{\cos^2 11^{\circ} 30' + 0.41 \sin^2 11^{\circ} 30'}$$

$$= \underline{\underline{3.5 \text{ feet.}}}$$

Case (b) In this case it is assumed that the direction of the resultant stress goes through the centre of the pillars in both seams as shown in Fig.107.

Using the expression 28. The relative pillar displacement

$$\delta = \left(\frac{h_1 + h_2}{2} + d\right) \frac{\frac{1}{2} (1 - K) \sin 2\alpha}{\cos^2 \alpha + K \sin^2 \alpha} \qquad (28)$$

$$= \left(\frac{7 + 10}{2} + 30\right) \frac{\frac{1}{2} (1 - 0.41) \sin 23^{\circ}}{\cos^2 11^{\circ} 30' + 0.41 \sin^2 11^{\circ} 30'}$$

$$= \underline{\underline{4.54 \text{ feet.}}}$$

In both these cases, the calculations have been carried out for the value of the angle of dip of the workings that exists at Brightling, and for a value of the proportionality factor that has been determined from the laboratory testing of the seam material from the mine. To show the full effect on the relative pillar displacement of varying these two factors, the calculations, for each case, have been carried out for values of the proportionality factor between 0 and 1 for each angle of dip of the workings of 5°, 11° $30'$ and 15°.

The relative pillar displacements for this full range of values are shown in Table 17, and are presented graphically in Figs. 108 a and b. It can be seen from these results that in both cases, as the seam inclination rises, the relative pillar

TABLE 17

K	$\alpha = 5^0$		$\alpha = 11^0\ 30'$		$\alpha = 15^0$	
	Case (a) δ ft	Case (b) δ ft	Case (a) δ ft	Case (b) δ ft	Case (a) δ ft	Case (b) δ ft
0	2.6	3.4	6.1	7.8	8	11
$\frac{1}{4}$	2	2.5	4.5	5.8	5.9	7.5
$\frac{1}{3}$	1.8	2.2	4	5.1	5.2	6.6
0.41	1.5	2	3.5	4.5	4.6	5.9
$\frac{1}{2}$	1.3	1.6	3	3.8	3.9	5
$\frac{2}{3}$	0.9	1	2	2.3	2.6	3
$\frac{3}{4}$	0.6	0.8	1.5	1.8	1.9	2.3
1	0	0	0	0	0	0

CALCULATED RELATIVE PILLAR DISPLACEMENT.

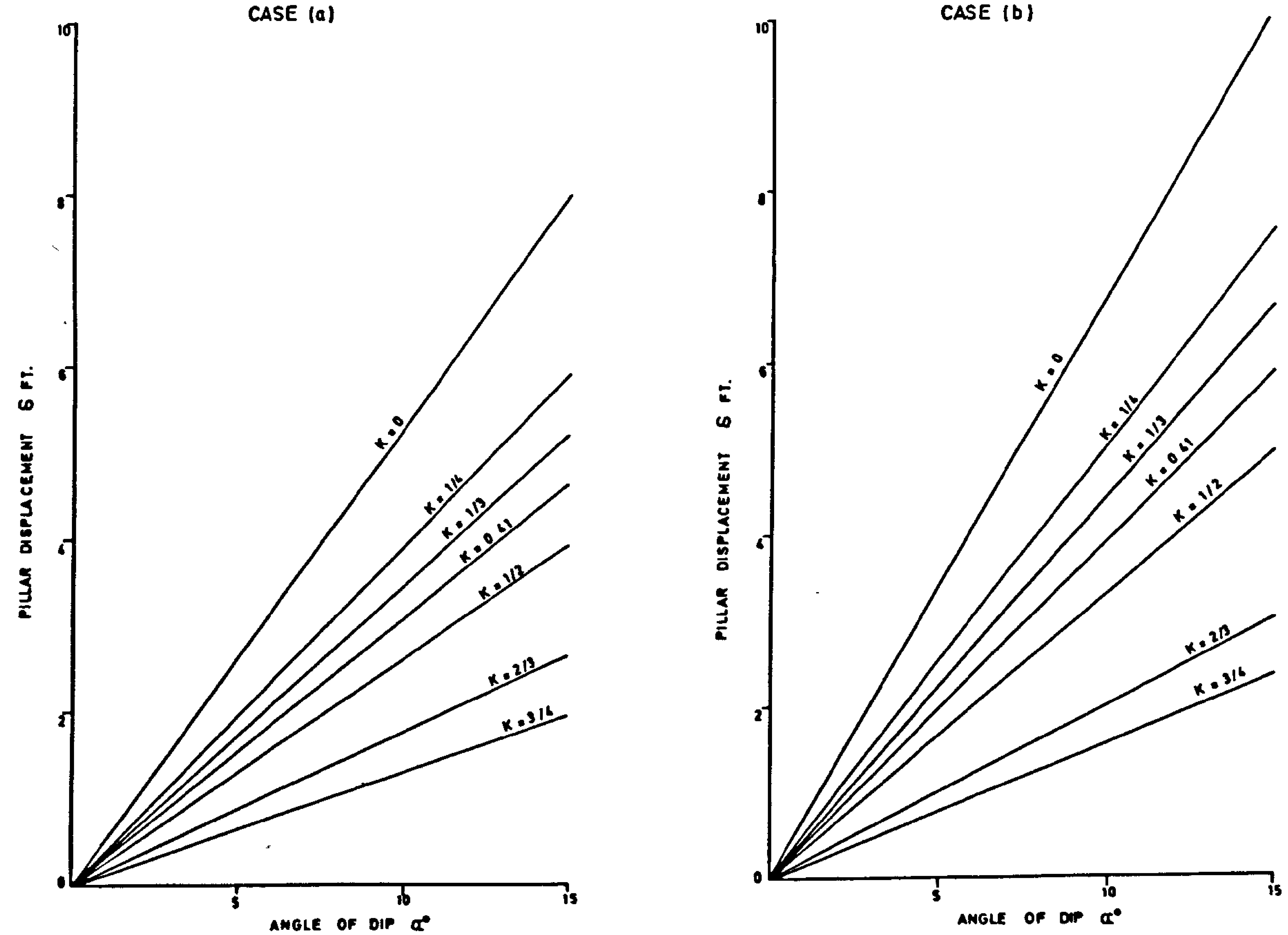

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FIG. 108

displacement becomes more dependent on the value used for the proportionality factor K. This is shown clearly in the diagrams. If the values obtained for case (b) are taken at an inclination of 5^{o} and a value for K of zero, i.e. no lateral loading, the relative pillar displacement calculated is 3.4 feet, whereas for the same inclination but a value for K of $\frac{1}{2}$, the pillar displacement is 1.6 feet. If however one considers the same value for K, but at a seam inclination of 15^{o}, the relative pillar displacements calculated are 11 feet and 5 feet respectively.

A probable value for the proportionality factor K can be expected to be in the range of $\frac{1}{4}$ - $\frac{1}{2}$. It can be seen that for the seam inclination of $11^{o}30'$ which exists at Brightling, the values for the relative displacement of the pillars for this range of values of K, are fairly close to each other. For practical purposes any value between 3 feet and 5.8 feet may be selected for the downhill displacement of the pillars in the lower seam. If the value of K calculated from the information determined in the laboratory is used, then pillar displacements of 3.5 and 4.5 feet are suggested for case (a) and case (b) respectively.

18.1.2 Determination of the Pillar Normal and Shear Stress Components.

The following equations for the normal and shear stress components acting on the pillar were obtained in section 3.1.

$$\sigma_{\alpha} = \frac{A_1}{A_2} \sigma_z \left(\cos^2\alpha + K\sin^2\alpha\right) \qquad (24)$$

$$T_{\alpha} = \frac{\sigma_z}{2} (1 - K) \frac{A_1}{A_2} \sin^2\alpha \qquad (25)$$

where

σ_α and τ_α are the normal and shear stress components expected to act on the contact surfaces of the roof element and the pillar, if both of these are assumed to be rigid bodies in contact

$\sigma_z = \omega H$ the vertical stress component assumed to be induced by the weight of the superincumbent strata.

These equations have been solved for values of the proportionality factor K between 0 and 1 for each angle of dip of the workings of 5°, $11^{\circ}30'$ and 15°. In this case, the depth of workings are assumed to be 400 ft. and 430 ft. for No. 1 Seam and No. 4 Seam respectively. The calculated stress values obtained are shown in Tables 18 and 19.

In horizontal workings no shear stresses are set up at the roof and floor level of the pillar and consequently the pillars are subjected to normal stresses only. It can be seen from these results that the calculated pillar normal stress component remains fairly constant for all values of K and α, but the shear stress component acting on the pillar increases considerably with the increase in seam inclination but decreases to zero as the value for K increases. If however the stress values calculated for the seam inclination at Brightling are considered for a probable value for K in the range $\frac{1}{4}$ - $\frac{1}{2}$, then it can be seen that the magnitude of the shear stress components is not significant when compared with the corresponding pillar normal stresses. For both seams, the calculated pillar shear stress values vary approximately between one seventh and one tenth the values for the pillar

TABLE 18

No. 1 Seam

K	$\alpha = 5^\circ$		$\alpha = 11\frac{1}{2}^\circ$		$\alpha = 15^\circ$	
	σ_α psi	T_α psi	σ_α psi	T_α psi	σ_α psi	T_α psi
0	1587	139	1536	312	1493	400
$\frac{1}{4}$	1589	104	1552	234	1520	300
$\frac{1}{3}$	1592	93	1557	208	1528	267
0.41	1593	82	1562	185	1537	236
$\frac{1}{2}$	1595	69	1568	156	1546	200
$\frac{2}{3}$	1596	46	1578	104	1564	133
$\frac{3}{4}$	1597	35	1584	78	1573	100
1	1600	0	1600	0	1600	0

TABLE 19

No. 4 Seam

K	$\alpha = 5^\circ$		$\alpha = 11\frac{1}{2}^\circ$		$\alpha = 15^\circ$	
	σ_α psi	T_α psi	σ_α psi	T_α psi	σ_α psi	T_α psi
0	1707	149	1651	336	1605	430
$\frac{1}{4}$	1710	112	1668	252	1633	322
$\frac{1}{3}$	1711	99	1674	224	1642	286
0.41	1712	89	1679	198	1652	254
$\frac{1}{2}$	1713	75	1686	168	1662	215
$\frac{2}{3}$	1715	50	1697	112	1681	143
$\frac{3}{4}$	1717	37	1703	84	1686	107
1	1720	0	1720	0	1720	0

TABLES SHOWING THE CALCULATED PILLAR NORMAL AND SHEAR STRESS

σ_α = Normal Stress

T_α = Shear Stress

normal stress within this range of values for K.

The calculated values for the pillar normal and shear stress can be compared with the mean compressive and shear strength values obtained for each seam from the testing of the rock material in the laboratory. The mean uniaxial compressive strength and the shear strength values obtained for No. 1 Seam were 4950p.s.i. and 1360p.s.i. respectively. It can be seen from Table 19 that within the probable range of values for K ($\frac{1}{4}$ - $\frac{1}{2}$) for a seam inclination of $11^{\circ}30'$, the pillar normal stress is approximately one third the mean uniaxial compressive strength of the seam material, whilst the pillar shear stress varies approximately between one-sixth and one-tenth the mean shear strength.

It will be noticed however that the difference between the calculated pillar stresses within this range of K and the laboratory strength values for No. 4 Seam of 3894 p.s.i. and 1070 p.s.i. for the mean compressive and shear strength values respectively, are not so great. In this case the calculated pillar normal stress is slightly less than one-half the mean uniaxial compressive strength of the seam material, whilst the pillar shear stress varies approximately between one-quarter and one-sixth the mean shear strength obtained in the laboratory. This comparison becomes even closer if the laboratory strength values obtained for the individual block sample S4-5 of 2812 p.s.i. and 850 p.s.i. for the compressive and shear strength values respectively, are compared with these calculated stress values.

Strictly speaking however, the comparison should be carried out taking into consideration the simultaneous action of both the normal and shear stress components. An attempt has been made to do this using the Mohr Stress diagrams shown in Figs. 105 and 106, obtained from the laboratory testing of the seam material. The calculated state of stress in the pillar for the value of K of 0.41 is depicted by the point on the normal-shear stress co-ordinate system and thus its position can be compared to the Mohr's failure envelope. If this point was on or outside the envelope, the corresponding state of stress would result in fracture of the rock. It can be seen from the plots of the stresses that the calculated state of stress does not cause any problem as far as the laboratory strength of the tested seam materials is concerned.

It should be appreciated that the values of the pillar stress components obtained from the calculations are approximate, but the information deduced from it is helpful in a qualitative way in giving some indication as to the state of stability of the pillars. The relative displacements of the pillars in the two seams have been calculated using a greatly simplified mathematical model, and it was for this reason that further laboratory model investigations were conducted to provide greater confidence in these evaluations.

18.2 The Application of Photoelastic Techniques to the Problem of Pillar Stability

The various assumptions that needed to be used in the theoretical appreciation in order to obtain values for the relative displacement of the pillars in the two seams, suggested that additional data was required which could to some degree substantiate these calculations. It was decided that photoelastic plate model experiments, equivalent as far as possible to the mining geometry, be carried out to demonstrate the influence of the angle of inclination of the workings and the proportionality factor upon the stress distribution set up in the model. The model, made of Araldite, was a scaled representation of a chosen section through the mine workings, the relative pillar layout being determined by the theoretical calculations carried out in section 18.1.1.

18.2.1 The Preparation and Testing of the Photoelastic Model

Care in the preparation of photoelastic plate models is as important as the actual analysis, otherwise machining stresses will be introduced in the model which may obscure the data being sought. Araldite CT200 was chosen as the optically sensitive material to be used, being available in the form of sheets 12" x 12" x $\frac{7}{16}$". This material machined relatively easily producing very little permanent stress in the material.

The Araldite sheet was cut by hand,using a hack-saw, to $\frac{1}{8}$" oversize and then the sides machined square and parallel using a milling machine. Since load was to be applied both vertically and horizontally extra care was taken to ensure all four sides were perfectly flat. The model was side milled to a size 11" wide by 10$\frac{1}{4}$" high, this being the maximum model size possible for use in the loading press of the polariscope. This size of model was chosen to make the edge distance beyond any opening in the model, as great as possible. Coates (26) found that in order to produce conditions equivalent to an infinite mass the edge distances for the model should be at least three times the half width of a single opening.

A combination of end and side milling was used to prepare the model openings with a maximum incremental depth of cut of 0.010 in. Cutting depths in excess of this produced undesirable machining stresses which increased in intensity as the depth of cut increased. The openings in the model corresponding to the rooms in the upper seam were made first. These openings were scaled to 1 in. equals 20 ft. and represented 20 ft. wide rooms in a 7 ft. high seam dipping at a gradient of 1 in 5 or $11^{\circ}30'$. Four rectangular openings with corner fillets $\frac{1}{16}$" radius, were formed to represent the No.1 Seam workings. This was the minimum number of pillars to ensure a constant value for the stress concentration factor (28).

The stress distribution in the model resulting from these openings was then examined by loading the model in the polariscope. Bi-axial loading was achieved using individual hydraulic rams for the horizontal and vertical directions respectively. A vertical compressive load was gradually applied to the model by means of an hydraulic ram and motor until the load acting on the model was the same as the calculated theoretical load acting on the mine workings. The shear stress distribution, or isochromatics, produced in the model using monochromatic light, was then photographed. Then, keeping the vertical load constant, the horizontal loading was applied to the model, and photographs taken of the shear stress distribution for differing horizontal loads corresponding to values for K, the proportionality factor, of $\frac{1}{4}$, $\frac{1}{3}$, and $\frac{1}{2}$.

The model was then removed and the openings in the model corresponding to 20 ft. wide x 10 ft. high rooms

K = 0

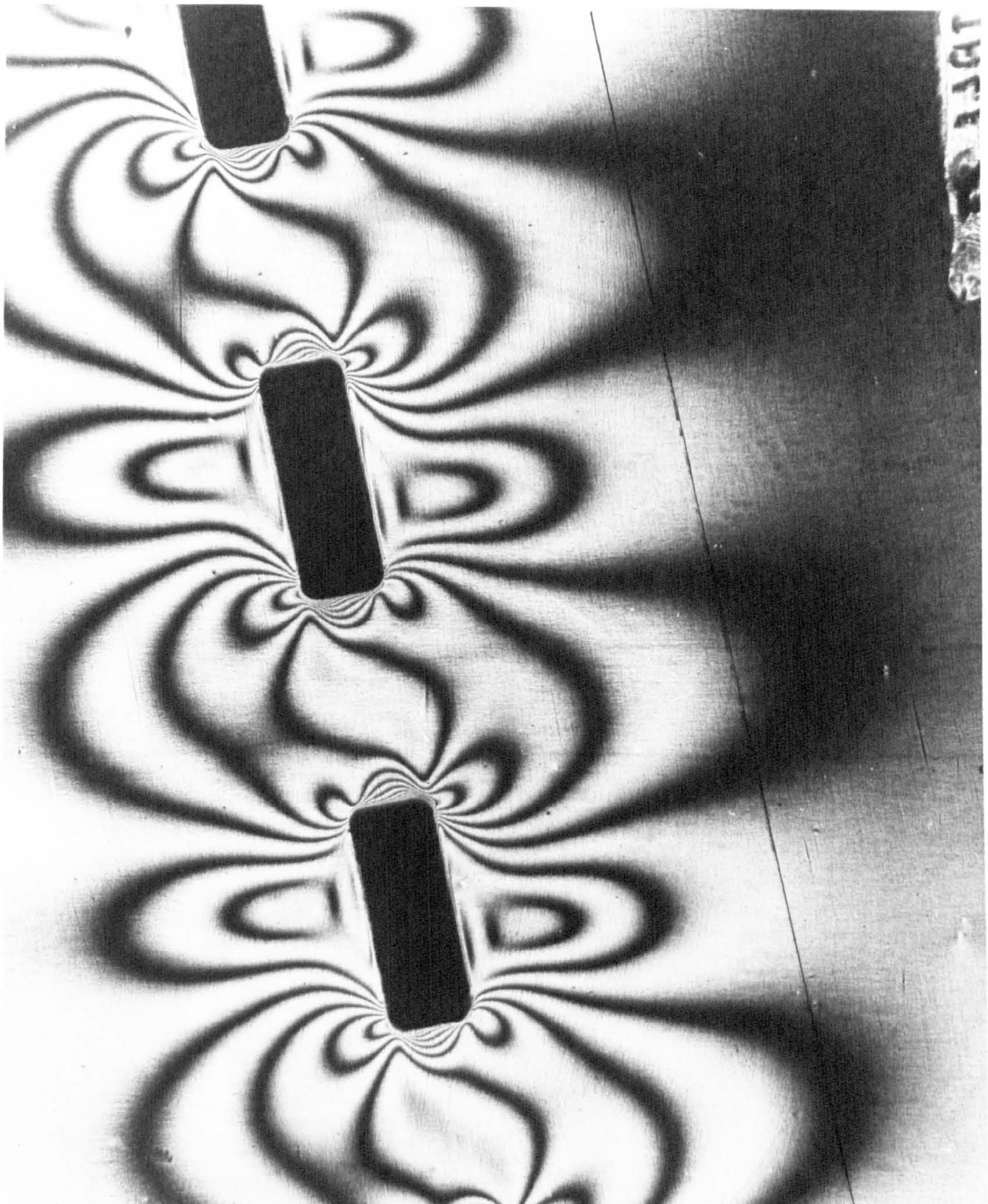


Fig. 109

K = 0.33

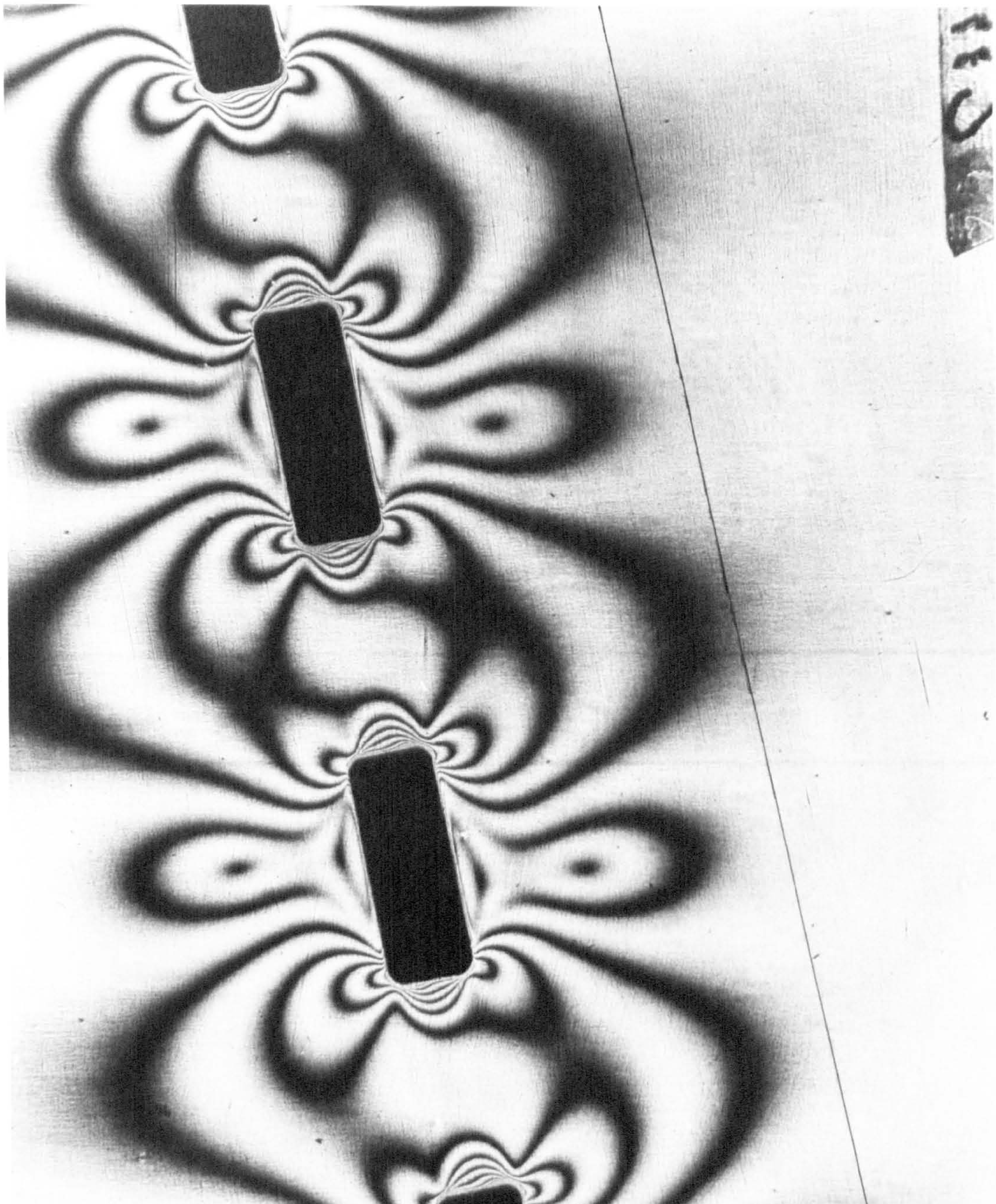


Fig. 110

K = 0.5

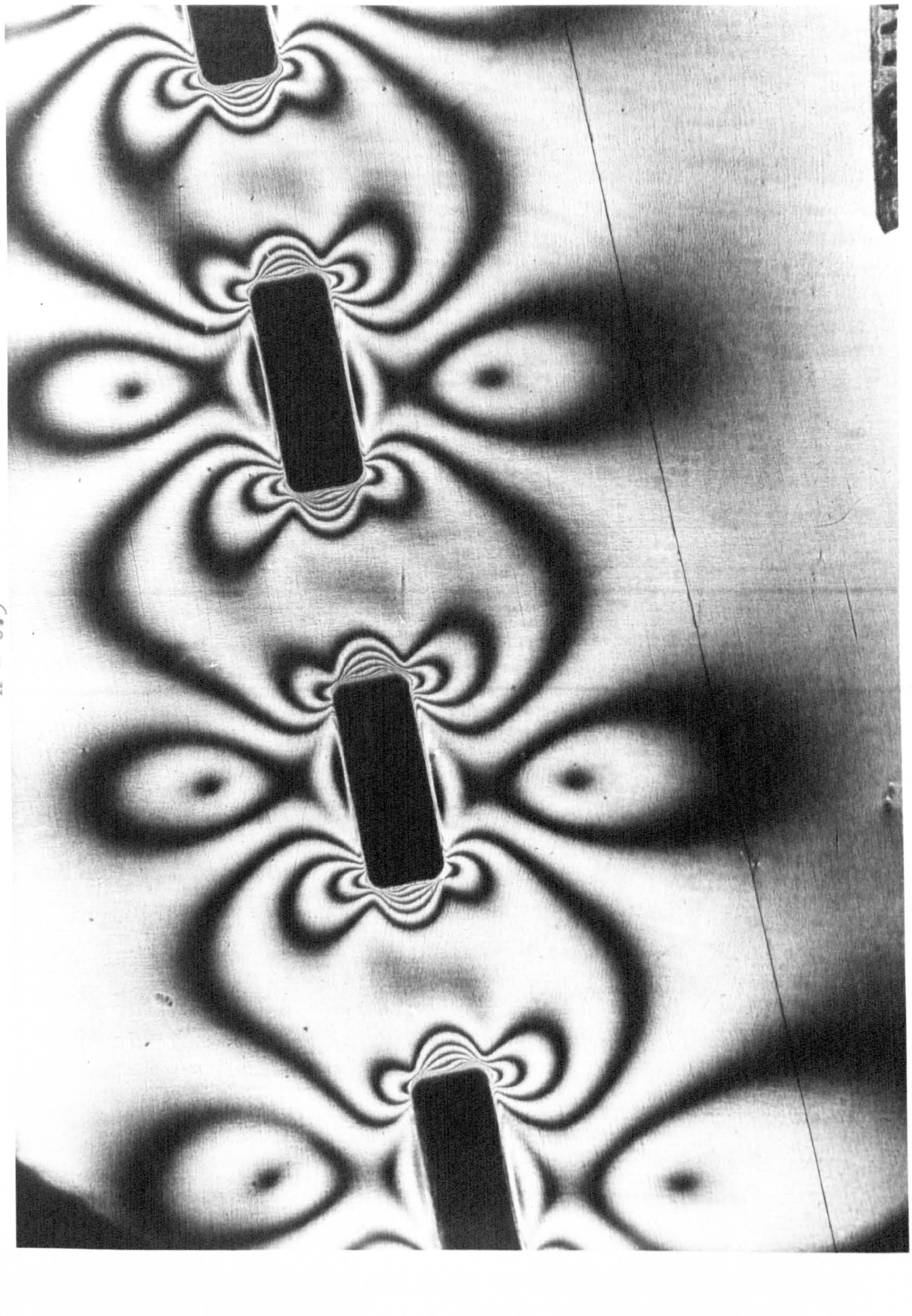
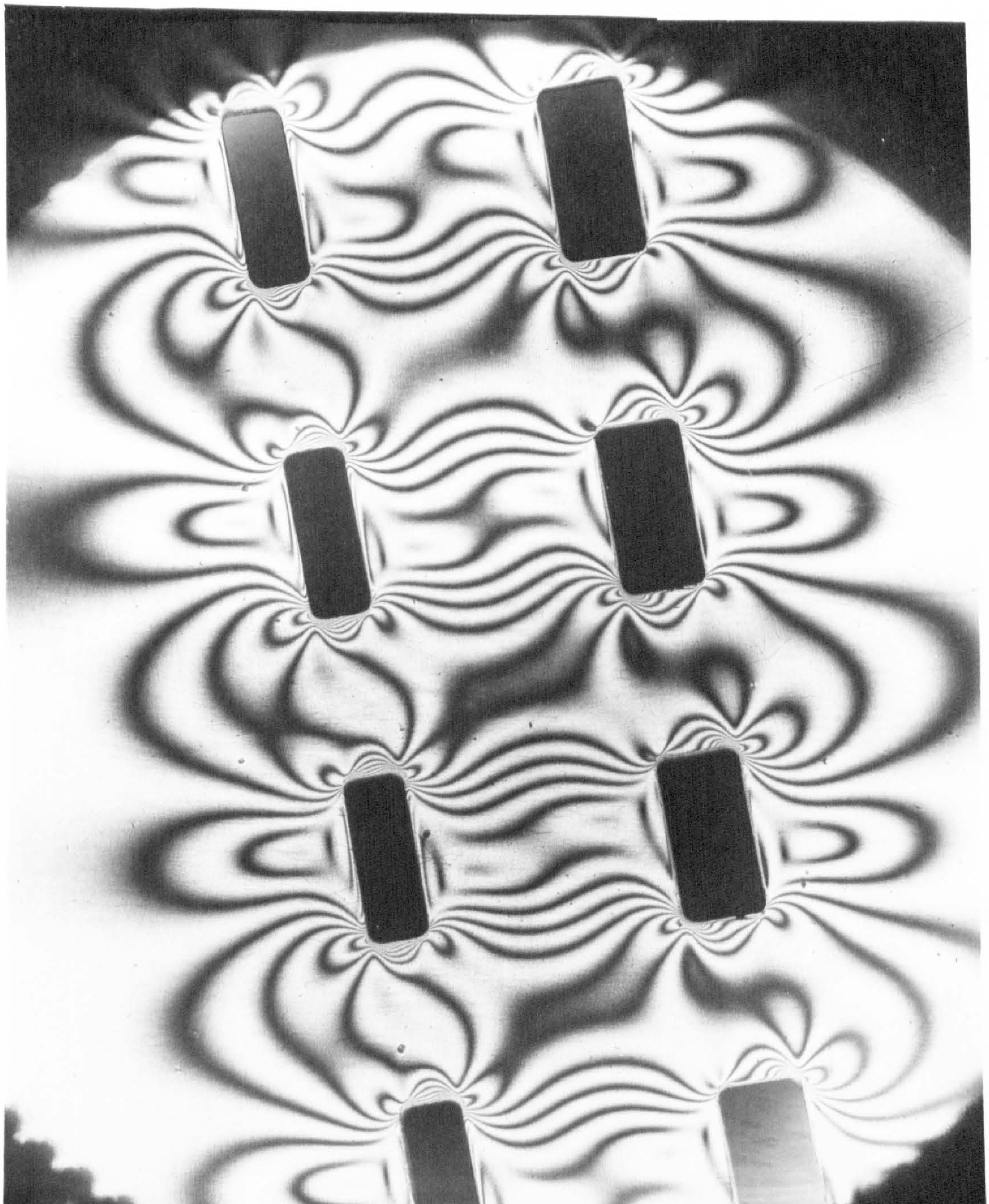


Fig. 111

in the lower seam were machined. These openings were displacedto the dip, a scaled distance of 4 ft. relative to those formed to represent the No. 1 Seam workings. This pillar displacement was within the range suggested by the calculations carried out in section 18.1.1. This, in fact, approximately corresponds to the pillar layout used at the mine prior to this investigation. The same series of photographs were then taken as before for the same values of the vertical and horizontal load.

18.2.2 <u>Discussion of the Results</u>

The isochromatics or shear stress distribution produced in the model for a constant vertical load and values for the side load of 0, $\frac{1}{3}$, and $\frac{1}{2}$ the vertical load, can be seen in the photographs, Figs. 109 - 114. The photgraphs, Figs. 109 - 111 show the shear stress distribution in that model in which only those openings representing the upper or No.1 Seam workings had been machined. The line shown in each of these photographs below the openings was marked on the model and corresponded to a scaled distance of 30 ft. below No.1 Seam, and represented the position of the roof of the lower or No. 4 Seam workings. This model, containing only the upper seam openings, was tested to determine the depth of penetration of the stress distribution due to the No.1 Seam workings, and also to enable a comparison to be made between the stress distribution in the No. 1 Seam pillars before and after the lower openings representing the No.4 Seam workings were introduced. The photographs, Figs. 112 - 114, show the shear stress distribution in the model in which both seams were represented.



K = 0

Fig. 112

K = 0.33

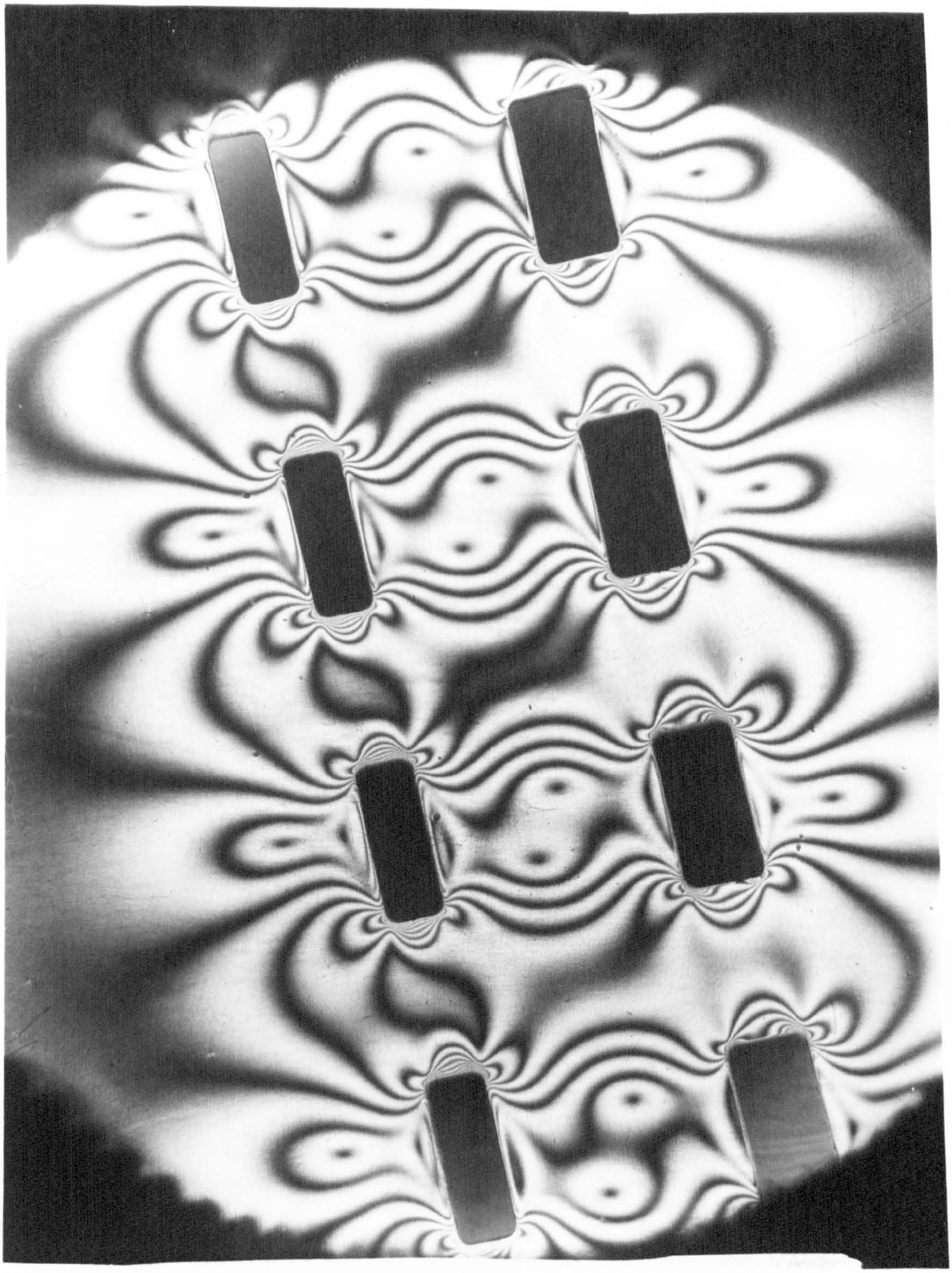


Fig. 113

K = 0.5

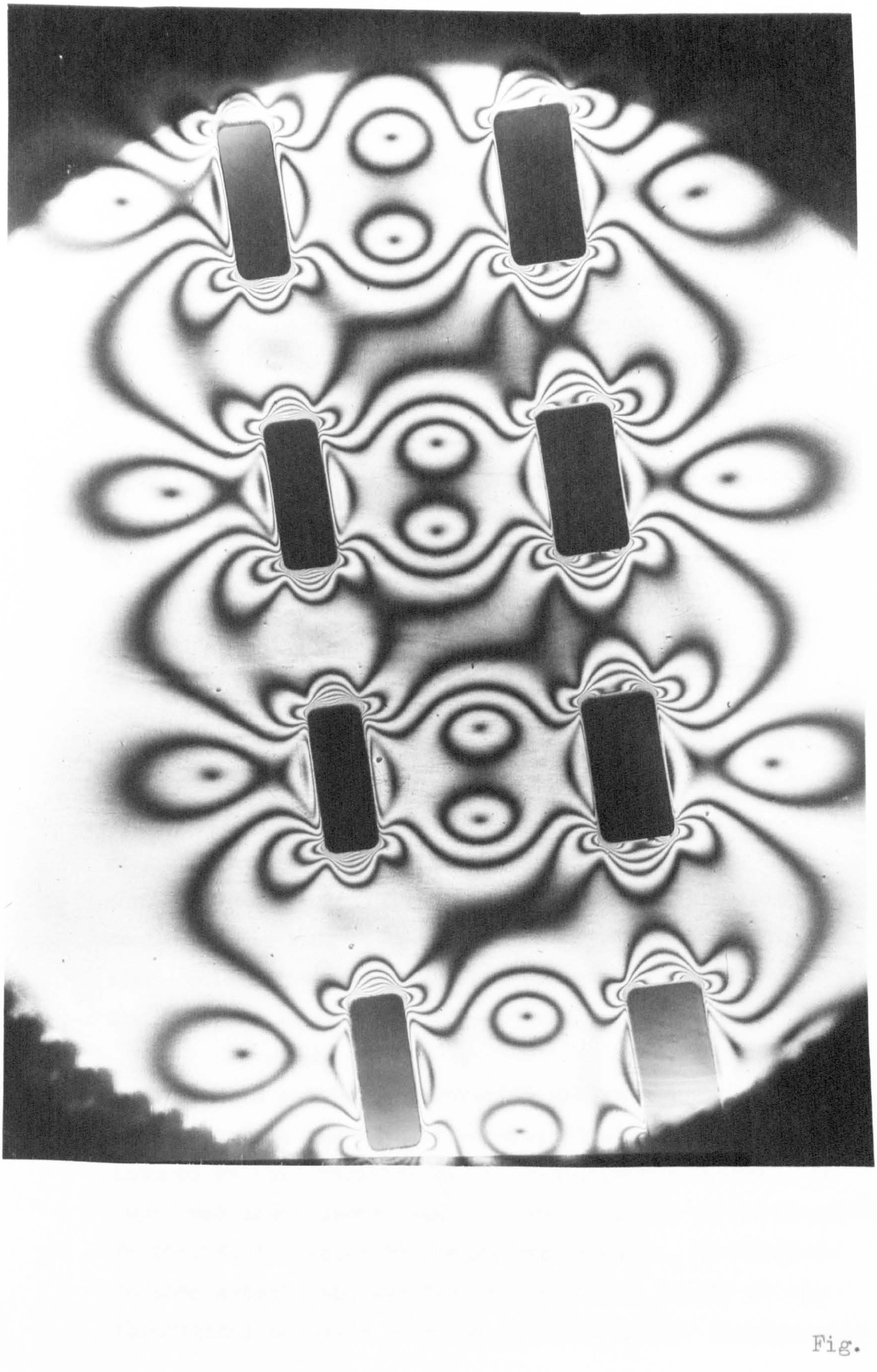


Fig. 114

It can be seen that the introduction of the lower or No.4 Seam openings has little influence on the stress distribution both above the No. 1 Seam and in the No. 1 Seam pillars themselves. The model containing the No. 1 Seam workings only also indicates the 'inferior' depth of penetration of stress effects from the upper seam when the lower seam is not worked. The effect of increasing the side loading on this model, Figs. 109 - 111, is a more localised concentration of the stress effects on the proposed position of the lower seam working. Increasing the side loading on the model in which both sets of workings are represented, Figs. 112 - 114, resulted in a better distribution of the stress effects between the seams for this relative pillar layout.

From the photographs, it would seem that the angle of dip of the workings has only a slight influence on the stress distribution in the pillars in both seams. The only noticeable effect is that the floor area on the up-dip side and near the floor on the down-dip side, is slightly underloaded in the pillars in both seams. The most loaded parts of the pillars are those at the roof on the up-dip side and near the floor on the down-dip side, this effect becoming slightly more pronounced with the increase in side loading.

This purely demonstrative analysis clearly indicates that there is no adverse stress distribution induced by this relative pillar displacement between the upper and lower seams, for each value of the proportionality factor, K. for which the model was tested. It confirms to some extent, the conclusion drawn as a result of the theoretical analysis that the relative pillar displacement used in the mine should not result in pillar instability in either seam. In fact, there were indications from this photoelastic analysis that the effect of the inclination of the seam on the state of stability of the workings is only very small, and it may even be suggested that the present angle of inclination of 11° 30' with the existing working dimensions may be considered as being fairly gentle, as far as its effect on pillar stability is concerned.

19. CONCLUSIONS AND RECOMMENDATIONS RELATING TO THE BRIGHTLING MINE

19.1 Conclusions

The conclusions that may be drawn as a result of the investigations into the state of the existing workings at the Brightling mine are fairly straightforward. The laboratory investigation carried out to determine the strength properties of rock material from both seams showed that the No. 4 Seam rock material is significantly weaker than the No. 1 Seam rock material, indicating that the stabilitiy of the No. 4 Seam pillars is the most critical individual factor in the overall stability of the duplex mining system. It was noticed that the No. 4 Seam rock material was characterised by the presence of satinspar bands of various and varying thicknesses, and these bands influenced the failure of those specimens in which they were present. This influence was most marked in those specimens taken from the approximate mid-height of the seam section where these bands were most frequent, resulting in a lower strength value being obtained for the gypsum from this section of the seam.

The theoretical analysis carried out to evaluate the possible pillar normal and shear stress compenents, for both seams for different values of the lateral primitive stress and angle of inclination of the seam, suggested that the existing pillars as individual support elements are stable. The ultimate compressive and shear strength of the gypsum forming the No. 1 Seam pillars appears to be a great deal larger than, respectively, the compressive stress that the pillars are required to bear and the shear stress induced in the pillars due to the inclination of the seam, for all possible values of the lateral primitive stress. However, the difference between the calculated stress values and the laboratory determined strength values for the No. 4 Seam gypsum are not so great. Even so, these differences are not significantly small enough to justify any conclusions other than that the No. 4 Seam pillars are stable.

The theoretical appreciation of the allowable range for the relative pillar displacement between the upper and lower seams, to the dip, has provided a measure of confidence in the existing design data. From this analysis it would appear reasonable to accept a pillar displacement in the lower seam of between 3 and 6 feet to the dip for the existing inclination of the seams. The photoelastic analysis corroborates this in showing the location of the higher shear stress values, and indicating that there is no adverse stress distribution induced by the superimposition of the workings using the present relative pillar displacement.

From the investigations carried out, it can be concluded that the present pillar displacement is quite adequate for ensuring the overall stability of the present workings since it conforms to the displacement range deduced from the existing known data.

19.2 Recommendations for Future Work

The theoretical and photoelastic model appreciation of the problem, together with the laboratory determined strength properties of the gypsum, has provided a measure of confidence in the present mine layout together with a knowledge of how the seam material deforms and fails under the action of applied forces. It should be noted however, that a number of simplifying assumptions were made in the theoretical appreciation, whilst the possible factors affecting laboratory strength testing are considerable. 'In situ' measurements can be used to provide the degree of reality that is required for a better appreciation of the range of factor of safety involved in the stabilitiy of the pillars in this duplex seam mining situation. The underground instrumentation schemes installed in the Sherburn and Stamphill mines, described in earlier sections of this thesis, have indicated how deformation measurements may be used for determining the state of stability of pillars. It is suggested therefore that 'in situ' investigations should be initiated at the Brightling mine to provide factual data concerning the state of stability of the No. 4 Seam pillars.

Bearing in mind the fact that the lower or No. 4 Seam is worked prior to the overlying No. 1 Seam, it would appear that there are two circumstances in which the No. 4 Seam pillars could be instrumented. The choice would depend on that particular part of No. 4 Seam being overmined by No. 1 Seam working in the not too distant future following the installation of the instrumentation. The possible circumstances are:

a) Instrumentation of the pillar during its formation.

b) Taking an already mined area of No. 4 Seam that is about to be overmined, and installing instruments in and around the chosen pillars.

In the former case, a much larger proportion of the deformation would be recorded. The purpose of this instrumentation would be to record deformation in and around chosen pillars in the No. 4 Seam caused by the re-distribution of the load as a result of extracting the No. 1 Seam. When finally the No. 1 seam workings are fully developed and the recorded deformation has virtually ceased, then it may be deduced that the second re-distribution of the load caused by the mining of No. 1 Seam did not induce instability in No. 4 Seam pillars.

The instrumentation scheme itself would be based on the Extensometer technique of deformation measurement, that has proved itself to be both practical and reliable in investigations carried out by the author. In addition to the installation of instruments in pillar boreholes, which would permit the measurement of horizontal strain across the pillars, and the installation of roof-floor convergence stations, the fact that the workings are inclined somewhat suggests that the lateral convergence occurring between the instrumented pillars and the surrounding pillars should be recorded. The installation of lateral convergence stations on each side of the instrumented pillars would indicate the amount of influence the angle of inclination of the seam has on the pillars. Once 'in-situ' measurements are available it should be possible to draw factual conclusions regarding the state of stability of the existing pillar layout, and in conjuction with the already available laboratory data, provide a means of assessing the stability of future mine workings that may, as a result of geological conditions, have to be worked on increased gradients.

PART 3

GENERAL CONCLUSIONS.

This thesis has been primarily concerned with the determination of the state of stability of room and pillar workings at a number of gypsum mines, each operating under quite different geological conditions. However, the results obtained from the investigations carried out by the author, and hence to some extent the conclusions, are not confined to gypsum mining, but may be of some value in most cases in which partial extraction methods of mining are used. With this in mind, an attempt was made in the first part of this thesis to define the problems involved in selecting design parameters when room and pillar methods of mining are practised. In order to judge the importance of the various problems that may be encountered, it was felt necessary to consider the opinions of others who have investigated and commented upon the factors affecting the stability of such workings. In addition, a number of fairly recent examples of collapse of room and pillar workings were briefly described to emphasise these problems, and in most cases suitable lines of research were indicated to achieve a better understanding of the mechanisms causing failure.

The methods of approach used in the investigations carried out by the author, described in the second part of the thesis, were almost entirely dependent upon the characteristics of the individual mines. At the Sherburn and Stamphill mines it was possible to carry out underground investigations to supplement previously obtained laboratory data concerning the mechanical properties of the gypsum. This combined approach has shown itself to be a fairly efficient method of assessing the state of stability of existing mine workings and for pre-assessing future design data. The scheme of instrumentation used at these two points served its intended purpose in each case in determining the character and relative magnitude of the deformations occurring in the strata forming the underground workings. In this respect, the use of the borehole extensometer strain-wire technique for the measurement of pillar lateral deformation, as a means of determining pillar stability, has proved to be extremely successful in these investigations as well as being simple and relatively inexpensive to install. It has also been shown that the extensometer technique has an important control function. In the investigations carried out at Sherburn, the application of this technique to the measurement of the vertical convergence between the roof and floor of the workings, has shown that it provides a simple and very demonstrative control tool for defining the continued stability of the roadways.

The information obtained from the pillar instrumentation scheme installed in the Stamphill mine, indicated that though the borehole strain wire technique is to be generally preferred to the use of the stressmeter as a means of determining stability, the value of the stressmeter for recording the change in pillar load as mining proceeds should not be overlooked. The stressmeters installed in the pillars at Stamphill indicated that there was a fairly close relationship between the pillar lateral deformation and the change in pillar load as recorded by the stressmeters. However, the problems involved in the calibration of the stressmeter output, and hence the direct interpretation of the results, can prove considerable, yet if this instrument is used successfully in conjunction with the extensometer technique, it may be possible to determine the stress-strain relationship of rock 'in situ'.

The attempt by the author to measure the absolute pillar stress in the Sherburn mine proved to be fairly inconclusive. The stress-relief technique of stress measurement utilising the

'Doorstopper' strain cell, appeared to be not entirely satisfactory using the equipment available. The experiments that were carried out revealed that while the stress-relief technique appeared to be quite applicable to the Sherburn gypsum, the equipment used for installing the straincell in the borehole and the subsequent removal of the core with the 'doorstopper' attached, required some modification for use in anything but the shortest of boreholes.

When for various reasons 'in-situ' investigations are not possible, as for instance in this thesis in assessing the stability of the Brightling mine workings, a method of approach based entirely on laboratory investigations can be made to provide an efficient and fairly reliable assessment of the state of stability of mine workings. An approach of this type however, should include a knowledge of the mechanical behaviour of the rock material before definite conclusions may be made.

The results obtained from the 'in-situ' investigations indicated how a pillar builds up its resistance to the re-distributed load as it is formed. In this respect a number of factors became apparent, some of which have been noted by other investigators. There was found to be a fairly close relationship between the measured roadway convergence and the lateral deformation taking place in the outer sections of the pillar. It was suggested by the results obtained from the instrumentation installed in the Sherburn mine, that the continuing lateral deformation of the outer sections of what were very large and extremely stable pillars after they were formed, could be due to a continuing closure of the surrounding roadways. It would seem therefore that roadway convergence stations should be installed wherever possible in the neighbourhood of pillar deformation measuring instrumentation schemes. It was also suggested that the re-distribution of the load due to the overlying strata on any single pillar depends to some extent on the distance of the pillar from the solid rib-sides, this rib-side probably accepting a higher proportion of the transferred load.

The information gained from the instrumentation scheme installed in pillars in the Stamphill mine showed how the stability of the pillars was affected by a change in their W/H ratio, in this case as a result of a change in the height of the pillars. The redistribution of the load acting on the pillars, as recorded by the stressmeters, ceased immediately the pillars were isolated, though the pillar lateral deformation continued to increase. With the subsequent increase in height of the pillars there was a slight increase in the deformation rate and there is very little doubt that the fact that the lateral pillar deformation has continued after working of the pillars ceased, was a direct result of this change in pillar W/H ratio.

It is unfortunate that an underground instrumentation scheme could not be installed in the Brightling mine in order to obtain factual information concerning the behaviour of the pillars in an inclined duplex seam mining system. The laboratory investigations that were carried out were designed to obtain an understanding of the state of stability of the existing workings at the mine and therefore very little information is available from this that could possibly be applied generally.

A factor that became apparent during the various investigations carried out by the author, indicated that although 'in-situ' investigations of the type described in this thesis provide factual and relevant information, a comprehensive laboratory knowledge of the strength properties of the rock material is required to obtain a complete understanding of design data. This is especially so in the case of room and pillar workings where long term stability is dependent upon the ability of the rock material to maintain itself in stable equilibrium.

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APPENDIX A

COMPUTER PROGRAM FOR A THIN SINGLE-LAYER ROOF BEAM

```
MN 04 STRESS CALC.→
begin library 1; open (20); open (30);
      writetext (30,[MN*04*STRESS*CALC.[ccc]]);
   begin real H,RHO,EU,NUU,HU,NUL,XP,PB,L,ELF,ELL,ELS,EL,
              WF,WL,WS,W,P,CU,I,K,AL,B,Y,M,MAX,AB,Z,D,MN,
              MP,WSN,WSP,MAXSN,MAXSP,N,NTEST,SUMEW,
              SUMNUW,SUMHW,EW,NUW,HW;
              integer f1,f2,f3,T1;
L060:   f1:= format([+d.ddd₁₀+ndc]); f2:= format([+d.ddd₁₀+
        ndccc]); f3:= format([+d.ddd₁₀+nd]);
        writetext (30,[STRESS*CALCULATIONS*FOR*A*THIN*
        SINGLE-LAYER*BEAM[c]]);
        copytext (20,30,[;]);
        T1:= read (20);
L100:       goto if T1 = 1 then L110 else L190;
L110:       N:= read (20); SUMEW:= 0.0; SUMNUW:= 0.0;
            SUMHW:= 0.0; NTEST:= 0.0;
L130:   EW:= read(20); NUW:= read(20); HW:= read(20);
        SUMEW:= SUMEW+(EW×HW); SUMNUW:= SUMNUW+(NUW×HW);
        SUMHW:= SUMHW+HW;  NTEST:= NTEST+1.0;
        goto if NTEST= N then L170 else L130;
L170:   EU:= SUMEW/SUMHW; NUU:= SUMNUW/SUMHW; HU:= SUMHW;
        goto L200;
L190:   HU:= read(20); EU:= read(20); NUU:= read(20);
L200:   L:= read(20); H:= read(20); NUL:= read(20);
        RHO:= read(20); ELF:= read(20); ELS:= read(20);
        ELL:= read(20); WF:= read(20); WS:= read(20);
        WL:= read(20); XP:= read(20);
L230:   PB:= H×RHO; P:= PB+XP; CU:= EU/(HU×(1.0-NUU↑2));
       :I:= (H↑3)/(12.0);
        Z:= H/L;
        AB:= P/CU;
L280:   MN:= 0.0; MP:= 0.0;
writetext (30,[[ccc]BEAM*THICKNESS*=*]); write (30,f1,H);
writetext (30,[BEAM*SPAN*=*]); write (30,f1,L);
writetext (30,[THICKNESS*/*SPAN*=*]); write (30,f1,Z);
        goto if Z > 0.2 then L330 else L350;
Writetext (30,[[c]XXX**WARNING**XXX***THICKNESS*/*SPAN
            *EXCEEDS*1/5[cc]]);
L340:   goto L350;
writetext (30,[BODY*LOAD*=*]); write (30,f1,P);
writetext (30,[ADDITIONAL*LOAD*=*]); write (30,f1,XP);
L370:   goto if T1 = 1 then L380 else L410;
writetext (30,[[c]XXX**WARNING**XXX***LAYERED*ABUTMENT*
            WEIGHTED*VALUES*FOR*MODULUS*AND* POISSONS
            *RATIO[cc]]);
L400:   goto L410;
writetext (30,[ABUTMENT*MODULUS*=*]); write (30,f1,EU);
writetext (30,[ABUTMENT*POISSONS*RATIO*=*]); write (30,f1,NUU);
writetext (30,[ABUTMENT*HEIGHT*=*]); write (30,f1,HU);
writetext (30,[ABUTMENT*DEFLECTION*=*]); write (30,f2,AB);
writetext (30,[DEF*1*IS*TOTAL*DEFLECTION[c]]);
writetext (30,[DEF*2*IS*TOTAL*DEFLECTION*LESS*ABUTMENT
                *DEFLECTION[ccc]]);
```

```
      for EL:= ELF step ELS until ELL do
      begin K: = (ELxI)/(1.0-NUL↑2);
            AL:= sqrt(sqrt(CU/(4.0xK)));
L500:       B:= ((L↑2/6.0)+(L/AL)+(1.0/AL↑2))/(ALxL+2.0);
writetext (30,[[p]XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
                    XXXXXXXXXXXXXXXXXX[ccc]]);
writetext (30,[MODULUS*=*]); write (30,f2,EL);
      for W:= WF step WS until WL do
      begin goto if W < 0.0 then L560 else L600;
L560:       Y:= (P/CU)x(1.0+(exp(ALxW))xLxALx(AL↑2xBx
                 (sin(ALxW)+cos(ALxW))-sin(ALxW)));
            M:= -0.5xPxLxALxexp(ALxW)x((B-1.0/AL↑2)xcos(ALxW)
                  -Bxsin(ALxW));
          goto L640;
L600:       Y:= (P/CU)x(AL↑4xWx((W↑3/6.0)-((W↑2xL)/3.0)-
                 ((LxW)/AL) -(L/AL↑2))+(LxBxAL↑3x((1.0+(ALxW))
                 ↑2))+1.0);
            M:= ((-PxW↑2)/2.0)+((PxL)/2.0)x(W+(1.0/AL)-(ALxB));
          goto L640;
L640:       MAX:= (Mx(H/2.0))/I; D:= Y-AB;
          goto if M < MN then L670 else L660;
          goto if M > MP then L690 else L710;
L670:       MN:= M; MAXSN:= MAX; WSN:= W;
          goto L710;
L690:       MP:= M; MAXSP:= MAX; WSP:= W;
L700:     goto L710;
writetext (30,[[10s]X-COORD*=*]); write (30,f1,W);
writetext (30,[[10s]MOMENT*=*]); write (30,f3,M);
writetext (30,[[5s]STRESS*=*]); write (30,f3,MAX);
writetext (30,[[5s]DEF*1*=*]); write (30,f3,Y);
 writetext (30,[[5s]DEF*2*=*]); write (30,f2,D);
   end;
writetext (30,[[c]MAXIMUM*NEGATIVE*MOMENT*=*]); write (30,f1
                 ,MN);
writetext (30,[MAXIMUM*STRESS*=*]); write (30,f1,MAXSN);
writetext (30,[X-COORD*=*]); write (30,f2,WSN);
writetext (30,[MAXIMUM*POSITIVE*MOMENT*=*]); write (30,f1,MP);
writetext (30,[MAXIMUM*STRESS*=*]); write (30,f1,MAXSP);
writetext (30,[X-COORD*=*]); write (30,f2,WSP);
L830:       MN:= 0.0; MP:= 0.0;
   end;
writetext (30,[END]);
   end;
            close (20); close (30);
end→
```

APPENDIX B

EXAMPLE OF THE INPUT DATA

INPUT DATA FOR PROGRAM SHOWN IN APPENDIX A.

```
COPYTEXT;
0;
+120.0; +2.8_10+6; +0.28;
+180.0; +72.0; +0.28; +0.083;
+2.0_10+6; +0.2_10+6; +3.0_10+6;
-90.0; +10.0; +90.0;
0.0;→
```

```
COPYTEXT;
0;
PILLAR HEIGHT; MODULUS; POISSONS RATIO;
BEAM LENGTH; BEAM THICKNESS; POISSONS RATIO;
                       SPECIFIC GRAVITY;
RANGE OF BEAM MODULUS;
CO-ORDINATES;
ADDITIONAL BEAM LOAD- IF ANY;→
```

N.B. Page 15, Para. 2

"uncontrolled collapse" this refers to those collapses, either pillar or roofspan, which have proved disastrous.