The Establishment of Optimized Design Parameters for a New Gypsum Mine

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SUMMARY.- Reliable values for some of the mechanical properties of gypsum are determined and these are used as a basis for establishing the optimum working dimensions for a new mine.

I.- INTRODUCTION

Many stratified orebodies are mined by means of partial extraction methods. In this type of mining, portions of the deposit are left between roadways to act as pillars providing support both for the immediate roof and for the superincumbent strata. The provision of support for the immediate roof is particularly important from the point of view of safety - an unstable roof obviously presents a hazard to personnel below. The overall support of the superincumbent strata has a less immediate bearing upon the safety of personnel, but there may well be pressing reasons for maintaining the stability of the surface or intervening mineral deposits.

Hence, a company contemplating the initiation of partial extraction operations is faced with two mutually contradictory requirements. Firstly, mining economics require that, within a given area, as much mineral as possible be extracted during the primary mining operation, i.e., as little ore as possible should be sterilized in the form of pillars. Secondly, however, it is essential that safe working conditions be maintained; this means that a stable roof is most desirable. Furthermore, there is often the need to provide general support for the superincumbent strata. The corollary of these requirements is an array of stable pillars, i.e., ones which may contain a considerable amount of ore.

The engineer seeking to optimize the dimensions and extent of an array of pillars and roadways must, therefore, achieve a compromise between the necessity to maintain safe working conditions and the requirement to extract as much mineral as possible from a given area. It will be shown that, providing the material making up the deposit lends itself to reliable mechanical testing, and if the prevailing conditions can be defined with reasonable accuracy, then a close approach may be made to the optimum working dimensions.

II.- NOTATION

σ_t	ultimate tensile strength, p.s.i.
σ_c	ultimate compressive strength, p.s.i.
τ	shear strength, p.s.i.
E_p	secant modulus of pillar, p.s.i.
E_r	secant modulus of roof, p.s.i.
E_s	secant modulus of solid, p.s.i.
ν	Poisson's ratio
V	specimen volume, ins^3.
H	specimen or pillar height, ins.
D	specimen or pillar width or diameter, ins.
M	moment, lbs. ft.
C	pillar constant
k	coefficient of permeability, m/sec.
$F(\zeta)$	probability coefficient

III.- APPRECIATION OF CONDITIONS AT THE MINING SITE.

The new mine is situated below the extensive post-glacial plains which cover a large area of northeastern Yorkshire. These plains extend inland from the coast for a distance of about 15 miles and are an extremely prosperous farming area. The deposit of gypsum 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 lying north-east of this area. The gypsum, which is usually 16ft. thick, is extremely pure and it is found at comparatively shallow depths; at the mine site area, the overburden is only 120ft. 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, consisting of an extremely large number of thin layers of varved clays, fine silts, and sands extending down from the surface to a depth of approximately 75 ft. From the bottom of the lacustrian deposit to the seam, a distance of approximately 45ft., the strata consists of a succession of marls, weak sandstones and siltstones which, although they are inherently incompetent materials in the mining sense, are appreciably stronger than the surface material which, to all intents and purposes, may be considered to have negligible strength.

Despite the fact that the presence of the large amounts of clay in the surface deposits renders them almost completely impervious to the vertical movement of water, there are two continuous beds of fine sand at 25ft. and 60ft. which readily permit its lateral movement.

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 and in this condition they present an extremely serious hazard to any unprotected excavation to which they have access. The potential hazards associated with the sand beds necessitated the imposition of certain legal restrictions upon the mine design. These restrictions will be discussed later.

The gypsum seam itself was by far the strongest material encountered during the development of the mine. Physically, the gypsum, in excess of 98% pure, occurs as a fine-grained massive pink 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. Petrographic examination of exploratory cores confirmed that almost no impurities were present - it also indicated that the bulk of the gypsum was very fine-grained with the maximum crystal size being less than 2.0mm. Additional petrofabric analysis revealed no sign of any post-depositional deformations having taken place, and provided further confirmation of the extremely homogeneous composition of the gypsum.

Below the gypsum lay a thick deposit of competent dolomite. In most places this was in intimate contact with the gypsum, but in a few areas from 6 - 9 in. of marl occurred between the two evaporites.

The majority of the problems which exist at the new 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. Knowledge of this unstable structure caused the Inspector of Mines to impose certain arbitrary restrictions on the workings within the seam. These included:

(1) Bord and pillar workings should be used, i.e., the workings would consist of two sets of mutually perpendicular drifts forming an array of square or rectangular pillars.

(2) 6ft. of gypsum should be left between the roof of the workings and the gypsum/marl contact.

(3) The pillars should have a safety factor of 8 to 1.

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 detectable ground movement at the surface. The reason for the imposition of this condition was again connected with the nature of the upper strata. The lacustrian material at the surface covers a large area and this plain is valuable farming land. Because of its extent, 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 mining 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 bord and pillar mining was employed, the overlying strata should be disturbed as little as possible so that there would 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.

Preliminary boreholes which had passed through the seam and penetrated the underlying dolomite had indicated that below this lay an aquifer with an artesian head of approximately 170ft. However, as the dolomite had a minimum proven thickness of 15ft., the presence of this aquifer was not felt to present any hazard.

Thus, the initial conclusions were that the mine workings would be situated in the lower two-thirds of the gypsum seam, there would be a 6ft. thick roof layer and, whatever working dimensions were chosen, there should be no detectable subsidence at the surface. Furthermore, the mine workings would have to be stable on a long-term basis.

As the deposit lay at a depth of 120ft., it was assumed that the gypsum would be subjected to a mean inherent vertical stress of 120 p.s.i. This meant that, even with an extraction rate of 80%, the mean vertical pillar stress would only be 600 p.s.i. Therefore, it appeared that the load bearing capacity of the individual mine pillars was probably not going to be the most important factor in the design of the mining layout.

With the conclusion concerning the strength of the pillars in mind, attention was turned to a preliminary examination of the other basic structural component in the proposed layout, i.e., the roof span. The most probable limitations on the final working dimensions would be those imposed by the maximum tensile stress induced in the roof.

An additional limitation on the overall design could be imposed by the magnitude of the shear stresses which would be induced in the roof adjacent to the edges of the mining panel. It was felt desirable to examine these, together with the tensile stresses due to overall panel roof deflections, in order to establish overall working dimensions.

The mechanical properties of the gypsum which would play important parts in the former limitations would probably be:

(1) The ultimate tensile strength of the gypsum - this would effectively limit the maximum tensile stresses which might be induced in the roof

(2) The shear strength - shear stresses would be induced at the panel edges.

(3) Young's Modulus and Poisson's Ratio - pillar deformation would control overall roof deformation and, hence, the magnitude of the tensile stresses

(4) Time dependent deformation properties - long term pillar stability was of paramount importance

(5) The permeability characteristics of the gypsum would have a bearing upon the build-up of hydro-

-static pressure above the roof layer.

In addition, a testing programme was initiated to establish relevant values for the uniaxial and triaxial compressive strength of the gypsum.

IV.- LABORATORY INVESTIGATIONS.

The high degree of homogeneity and the continuous nature of the gypsum, revealed by the preliminary study of numerous cores, has already been commented upon. The exploratory holes provided approximately 220ft. of 4in. diameter gypsum core, taken perpendicular to the plane of the deposit. In addition, large quantities of material became available when the exploratory shaft was completed. This gypsum took the form of either blocks, weighing up to 2500lb. cut from the face of the development heading, or very large amounts of BX core from horizontal holes within the seam.

Thus, at the start of the laboratory phase, the following samples were available:-

(1) 220ft. of 4in. diameter core - vertical

(2) large amounts of BX core - horizontal

(3) massive blocks up to 36in. x 36in. x 24in.

It was apparent that, since the pillars would be formed in the lower part of the seam, there was little point in carrying out extensive tensile strength determinations in this region. Similarly, there appeared to be no reason to carry out innumerable uniaxial compressive strength determinations in the roof horizon, other than for obtaining data concerning the homogeneity of the gypsum. However, it was decided that it would be instructive to establish whether or not there was a variation in tensile strength along the vertical section. The results of these tests showed that, with the exception of a 2ft. thick portion at the top of the deposit, where some leaching may have taken place, there was no significant variation in tensile strength down the vertical section.

(a) Tensile Strength Determination.

Because of the large quantities of core available, the most convenient method of obtaining tensile strength data was by means of the Brazilian Disc Test. However, it was appreciated that the '4-Point Loading Test' would more closely approach the loading conditions that the gypsum would be subjected to in the mine. Correspondingly, prismatic specimens were cut from some of the large core and tested. The results of these '4-Point Loading Tests' were compared to the results obtained from testing adjacent material by means of the Brazilian Disc Test. Two facts immediately became apparent; firstly, the results from the prisms were consistently higher than those from the discs, and secondly, as the length of the constant tensile stress region between the two loading points increased, there was a reduction in the calculated tensile strength. However, this decrease did not continue indefinitely. There appeared to be a critical length for the constant stress zone above which there was little further reduction in tensile strength. This critical length was 3.0in., i.e., approximately 35 times the maximum grain dimension in the specimen. The higher strength obtained from what is frequently described as a flexural test agrees with the results of other tests made with different materials (1). Specifically, the mean tensile strength of the supercritical prismatic specimens was greater than the mean strength of the disc specimens by a factor of 2.2.

Additional testing gave values for the indirect tensile strength of the gypsum which agreed closely with the results of the initial disc tests and had a comparatively small standard deviation. The results from two typical groups of disc specimens taken from BX cores are given below -

Group 17(ii) 77 discs	
Mean indirect tensile strength	434 p.s.i.
Standard deviation	±44 p.s.i.
Group 9(iv) 64 discs	
Mean indirect tensile strength	453 p.s.i.
Standard deviation	±53 p.s.i.

These results compare favourably with the disc results obtained initially from one of the vertical boreholes -

Boreholes 1/51 89 discs	
Mean indirect tensile strength	441 p.s.i.
Standard deviation	±47 p.s.i.

Although no evidence of anisotropy was found, great care was taken to ensure that the plane of tensile failure in the discs cut from the horizontal boreholes was parallel to that of the perpendicular boreholes, and that both these lay in the same plane as the anticipated tensile stresses in the roof.

As a result of testing 669 discs, it was felt that a reliable value for the indirect tensile strength of the gypsum had been obtained, i.e., 457 p.s.i., ±66 p.s.i. Furthermore, the results of the preliminary flexural testing suggested that the indirect tensile strength was equal to 0.46 x the flexural tensile strength. Hence, a preliminary value for the tensile strength of the gypsum was obtained, 992 p.s.i. - the standard deviation, ±66 p.s.i.

(b) Shear Strength Determination.

The shear strength of the gypsum was taken as the value of the Mohr's envelope intercept on the shear axis; this value was 1230 p.s.i. Thirty specimens with a D/H ratio of 0.5 and a diameter of 3in. were tested in a triaxial cell capable of exerting a maximum radial stress of 10,000 p.s.i. at a maximum axial stress of 150,000 p.s.i. The shape of the envelope suggested that, at the confining pressures used, the gypsum behaved as a brittle material.

(c) Young's Modulus and Poisson's Ratio Determination.

The modulus of the pillars would influence both the deformation of the roof between the pillars and the overall deformation across the entire width of the panel. It was apparent that the modulus which would be most relevant would be the secant modulus between 120 p.s.i. and 800 p.s.i., i.e., between the probable inherent vertical stress level and a stress equal to the mean pillar stress at 85% extraction. Correspondingly, the secant moduli of a large number of gypsum specimens were established between these stress levels. Two facts became apparent; firstly, there was the customary increase in modulus as the specimens were cycled - the fifth and subsequent cycles displayed no increase - and secondly, as the specimen volume increased, the D/H ratio remaining constant at 0.5, there was a decrease in secant modulus. However, in specimens with a volume of more than about 70 cu.in., there was no further decrease - hence, the secant modulus of the gypsum was taken to be this limiting value, 2.1×10^6 p.s.i. Concurrent measurements of Poisson's Ratio gave a value of 0.28.

(d) Creep Testing.

This was carried out on both cylindrical and prismatic gypsum specimens. Although the material displayed time dependent deformation characteristics, these only became apparent when the compressive stress exceeded 4000 p.s.i. At a stress of 2500 p.s.i. at 90% humidity, and with an ambient temperature of 70°F, negligible deformation occurred during 300 day creep tests. In view of this, it would appear that there is little liklihood of time dependent deformations occurring in this mine.

(e) Permeability Testing.

It was possible that the development of a hydrostatic head above the roof would depend upon the permeability of the gypsum. Although the absolute value was thought to be quite low, no leaching having extended more than 2ft. below the top of the seam, a series of permeability tests were carried out. Cylindrical specimens 4.1cm. in diameter and 5.0cm. long were bonded inside steel cylinders and these assembled cells were used for testing. The apparatus consisted of a small positive displacement pump connected to these cells and was designed to permit the application of pressures of up to 2000 p.s.i. The results of this work showed quite conclusively that the permeability of the gypsum was very low, i.e., $k = 6.13 \times 10^{-10}$ m/sec.

(f) Compressive Testing.

This programme was designed to provide data relating to the probable strength of the gypsum pillars within the mine. Since it is an established fact that the compressive strength of rock specimens varies with both volume and D/H ratio, the testing programme had to fulfil two functions:

(1) it was necessary to establish a relationship between mean specimen strength and specimen volume over as wide a range of volumes as possible

(2) the influence of variations in the D/H ratio upon the compressive strength of the specimens had to be determined.

Very briefly, the first part of the programme consisted of testing 419 specimens of D/H ratio 0.5. Ten different diameters of specimens were tested, ranging from 0.75in. to 6.0in. and, as was expected, there was a marked reduction in the ultimate compressive strength as the specimen volume increased. The mean strength fell from an initial value of approximately 7400 p.s.i., to a constant value of 5100.

After processing these results, the following expression was obtained relating ultimate compressive strength to specimen volume when the D/H ratio was 0.5 -

$$\sigma_c = 6662\, V^{-.059} \qquad (1)$$

The second part of the compressive testing programme involved the determination of the strength of 570 gypsum specimens divided into groups whose diameters varied from 1 to 8 in. and whose D/H ratios ranged from 0.4 to 3.7. In accordance with the results obtained by many other workers, there was found to be a marked increase in the compressive strength as the D/H ratio increased. This was primarily due to radial constraint on the central core inducing triaxial loading conditions. This phenomenon is responsible for the situations where mine pillars support loads considerably in excess of their mean uniaxial compressive strength (2).

Analysis of the results obtained during this phase permitted the derivation of the following expression:-

$$\sigma_c = 8340\, D^{0.27}\, H^{-0.36} \qquad (2)$$

(i) Conclusions drawn from the compressive testing.

It should be remembered that expression (1) only applies to specimens with a D/H ratio of 0.5 and that, therefore, it does not take into account the effect of triaxial loading on the central portion of the specimen. However, expression (2) caters for variations in both diameter and height and, hence, it takes into account the effect of constraint and the consequential increase in the specimens apparent mean compressive strength.

The primary function of the testing of specimens with a constant D/H ratio but variable volume was to determine the magnitude of the "size" or "scale effect" for gypsum (3) (4) (5), i.e., to establish the relationship between uniaxial strength and specimen volume. The results of this programme indicated that, although there was a fairly rapid reduction in strength as the volume increased, a limiting value was reached quite quickly. This critical volume appeared to be slightly in excess of 60cu.in. and it represented a reduction in compressive strength of approximately 40%. When the results of this constant D/H ratio testing are viewed in the light of the work of Bieniawski, it would appear justifiable to treat the gypsum as a 'hard' rock in the sense that it is fine-grained, homogeneous, and free from any discontinuities. On the basis of Bieniawski's work, it would appear that the 'in situ' compressive strength of a material is closely approximated by the strength of specimens whose minimum dimension is greater than 25 times the maximum grain or particle size (3). Applying this criterion, it would appear that a specimen of the gypsum with a minimum dimension of more than 2in. would have a compressive strength close to the limiting value. However, an examination of the graphic presentation of the compressive strength vs. specimen diameter curve for D/H ratio 0.5, Fig.1, indicates that the limiting strength is reached when the minimum dimension of the specimen exceeds 3.2in., i.e., about 40 times the maximum grain size.

However, even if the critical specimen dimension is slightly greater than 3in., this is still well within the capabilities of laboratory testing and it is suggested that it is reasonable to use expression (2) as a basis for the design of the support pillars.

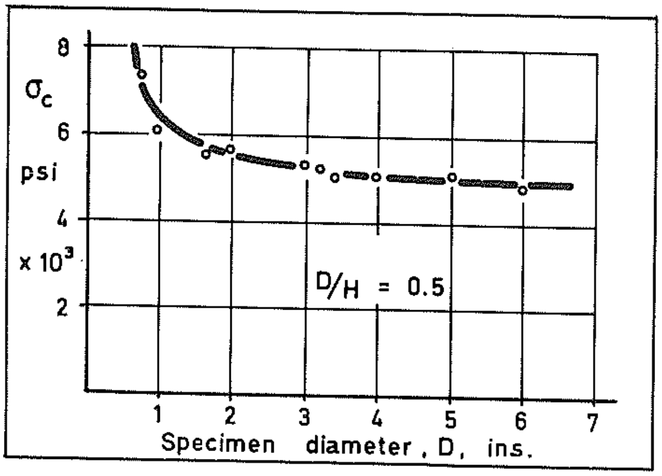

Fig.1. Relationship between σ_c and specimen volume.

The laboratory testing yielded values for all the relevant mechanical properties. The data from the testing, dealing with the 'size effect' for this gypsum, would appear to indicate that, in this case, the laboratory results may be used as the basis for design work. The relevant values are -

Ultimate tensile strength	992 p.s.i. ±66 p.s.i.
Shear strength	1230 p.s.i.
Secant modulus	120 - 800 p.s.i. 2.1×10^6 p.s.i.
Poisson's ratio	0.28
Ultimate compressive strength	$8340\ D^{0.27}\ H^{-0.36}$ p.s.i.

V. - THE ESTABLISHMENT OF WORKING DIMENSIONS.

It has already been noted that the tensile strength of the roof material will probably be the property which limits the working dimensions. In discussions with the mine operators, it had been decided that the probability of the roof failing under tension at any intersection should be maintained at less than 1%. As the results obtained during tensile testing displayed normal distribution, the value for F(ς) required to give a confidence coefficient of 99% could be derived from statistical tables (6). Thus, the value for the ultimate tensile strength which, it was suggested, would result in less than 1% of the intersection roofs failing was 836 p.s.i.

(a) The Pillar Dimensions.

Before calculating the permissible span, it was necessary to establish what the maximum percentage extraction might be, given the required safety factor of 8 and the compressive strength of the gypsum from expression (2). The nomogram - Fig.2 - has been constructed to permit the dimensions of acceptable pillars to be established for any given extraction rate. A series of pillar and span widths may then be evaluated for tensile stability by the technique referred to below. The nomogram may, therefore, be used to determine one of two things: either whether the pillar formed at a particular extraction rate with a specific span width is stable, or to establish minimum limits for the span necessary to maintain a specified extraction with pillars which fulfil the stability requirements.

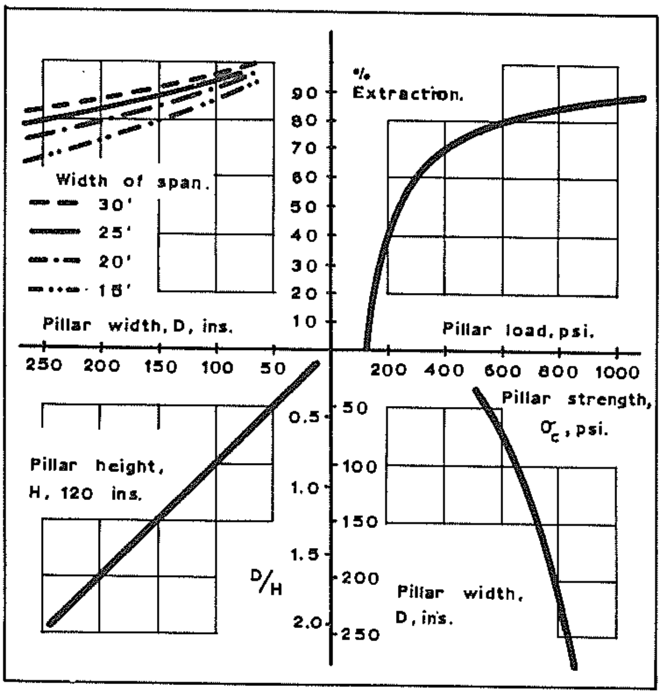

Fig.2. Relationship between percentage extraction and working dimensions.

As an example, the nomogram will be used to determine whether the pillar necessary to maintain an 80% extraction rate, with 20ft. wide rooms, will meet the stability requirements. The 80% extraction intercept on the 20ft. room width curve gives a pillar width of 15ft. on the horizontal axis. The pillar will be 10ft. high, hence the D/H ratio on the vertical axis is seen to be 1.5. The permissible load on a 15ft. wide pillar is 755 p.s.i., a value lying well to the right of the theoretical load and, therefore, the pillar is deemed to be stable.

(b) The Width of the Roadways.

Various procedures exist for calculating the magnitude of the tensile stresses induced in the roof. In all of them, certain limiting assumptions are made in order to reduce the problem to one of manageable proportions. The most satisfactory technique appears to be the one described by Wright, Ratti and Wang (7), in which they assume that the roof behaves as an elastic plate upon elastic supports. Briefly, the assumptions which are made are that the roof is elastic, and only subjected to forces acting normal to its plane. Furthermore, the roof is assumed to be thin, uniform, and only slightly deflected. Finally, the entire layout is presumed to be symmetrical. It is felt that these assumptions are valid in this situation, where the roof and pillars form a regular and elastic system. Based upon these assumptions, the techniques described in Wright's paper (7) are used to establish values for the maximum tensile stresses induced in roofs of various widths. In this way, the maximum roadway width may be determined using previously established or specified data.

Wright et. al. introduced the concept of the Pillar Constant, C, a measure of the rigidity of the mining layout, which influences the size of the bending moments in the roof. The Pillar Constant, the distributed load on the roof, and the factor a, the sum of the half-widths of the rooms and pillars, together determine the magnitude of the bending moments. Hence, if a value may be obtained for the distributed load, and for various combinations of pillar and room half-widths, it is possible to calculate the magnitude of the tensile stresses in the roof. Fig.2 is designed to give a range of possible pillar and room widths for specific extraction rates and, from these, using a value for the distributed load, the maximum tensile stresses can be obtained.

At this mine, where the rigidity of the overlying strata was greater than that of the gypsum roof, the overlying material was not expected to contribute to its loading. However, due to the nature of the sediments and to the very low permeability of the gypsum, it was anticipated that the full hydrostatic head would be developed at the gypsum/sediment contact. The value of this was taken to be 52 p.s.i., i.e., 120ft., and thus the total distributed load on the 6ft. thick roof was 58 p.s.i.

Based on the figure of 58 p.s.i. for the distributed load on the roof, and using the data from Fig.2, the following dimensions are suggested as being those which will ensure maximum extraction and yet meet the stability requirements for the pillars and roof:-

Pillar width	17ft.
Roadway width	25.5ft.
Extraction	84%

(c) Panel Width.

Tincelin and Sinou (8) and Höfer and Menzel (9) describe methods which may be used to calculate the tensile and shear stresses induced in the strata above the working horizon. These stresses are induced by the deformation of the strata due to the general lack of rigidity of the workings, rather than to the inter-pillar deformations affecting the immediate roof. Unfortunately, these techniques require an accurate knowledge of the mechanical properties of the overlying beds to give reliable results. In the case of this mine, assumptions had to be made about these properties, and this gave rise to inaccuracies in the calculated values of the maximum shear and tensile stresses. Assuming that the pillars supporting the roof act as a uniform elastic medium with a modulus equal to one-fifth of the solid seam, and that the gypsum roof and overlying sediments behave as separate layers, the following data concerning the magnitude and location of the stresses were calculated.

The tensile stress induced in a gypsum roof rose as the panel width was increased. A maximum of 140 p.s.i. occurred when the panel was approximately 160 ft. wide, then stress decreased to a steady value of 110 p.s.i. when the panel width exceeded 220 ft. If the width was less than 220ft., the position of the maximum tensile stress was displaced toward the centre of the panel, but once the width exceeded this figure, the maximum stress was found to be consistently 70ft. from the ribside. It was apparent that the high rate of extraction was responsible for this fairly high tensile stress.

The maximum shear stress always occurred directly above the ribside and once again it was noted that the magnitude of the stress increased rapidly as the panel was opened up - reaching a maximum at about 160 ft. width and then reducing slightly to a steady value of 2100 p.s.i. when the panel width exceeded 200ft. As with the tensile stresses, the high percentage extraction envisaged was responsible for the large shear stress. It was quite apparent that the economic requirement of maintaining a high percentage extraction, and the structural requirement of lowering the induced shear stress to about 1200 p.s.i. were mutually exclusive.

VI.- CONCLUSIONS.

It is suggested that the working dimensions have been established so that the conditions which were imposed for reasons of operator safety have been satisfactorily met. Furthermore, the suggested percentage extraction will enable the mine to operate as a highly profitable unit.

The question of the optimum panel width cannot be dealt with as satisfactorily. The induced tensile stresses do not present too much of a problem as they will occur above the pillars, away from the tensile stress concentrations which occur above the pillar corners. However, the predicted shear stress clearly exceeds the experimentally determined shear strength of the gypsum, and, for practical panel widths, it is difficult to see how the possibility of shear failure above the ribside may be eliminated. A possibility in a static system would be the formation of a zone of intermediate rigidity between the main production area and the solid rib. In a dynamic system, where the extension of the mined region is an economic necessity, there are always going to be boundaries between mined and unmined regions where high shear stresses will be set up. It is possible that, in the future, a mining method will be developed which will permit the shear stresses to be maintained at a sufficiently low level, but in the absence of this, the possibility of shear failure in the roof along the panel edge will have to be accepted.

As a check upon the validity of the assumptions made regarding the modulus of the gypsum making up the pillars, and as a means of monitoring surface subsidence, borehole extensometers were used. Anchors were installed in boreholes which passed through the portions of the seam which would form pillars. Normally, four anchors would be placed in each hole, one situated about 12ft. below the proposed pillar in the underlying limestone, one at the base of the pillar, one at the top, and one 12-15ft. above the gypsum/sediment contact, the purpose of the last anchor being to establish whether or not bed separation occurred. All movements were measured relative to the bottom anchor, which was assumed to be stable, using a constant tension extensometer. The deformation predicted for the pillars in the centre of the panel was .039in. However, the average measured deformation was .027in. As the extensometer used was known to be accurate to within ±.002in., two possible reasons for the discrepancy are suggested. One, the mining had not developed to a point where the pillars were fully loaded, and thus undergone maximum deformation. Two, variations in the moisture content of the surface deposits could have caused slight heaving of the instrument stations and, thus, an apparent reduction in the amount of deformation detected. The former explanation is more likely and additional evidence in its favour was the fact that, at the last examination, the gradients of the anchor displacement curves had not become zero.

It is not suggested that this approach to the problem of establishing optimum mining dimensions may be generally employed. Previous workers have shown that the minerals found in many stratified deposits do not lend themselves to the laboratory determination of the mechanical properties. In view of this, alternative methods of dimension determination have been developed (10) (11). However, if preliminary investigations indicate that the mineral to be mined is homogeneous, isotropic and does not display extensive discontinuities, it is suggested that the type of techniques discussed here will give rise to considerable increases in the profitability of the mining operation.

VII.- ACKNOWLEDGEMENTS.

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