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Physical property tests of rock, centrifugal tests and the design of underground mine openings

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PHYSICAL PROPERTY TESTS OF ROCK, CENTRIFUGAL TESTS AND THE DESIGN OF UNDERGROUND MINE OPENINGS

BY

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THESIS

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FOREWORD

Proper design of stable underground mine openings is important to the mining industry today. Various methods have been utilized by investigators to approach this goal.

A review of previous investigations of each of the phases of underground mine opening stress analysis is presented.

An approximate method of designing underground mine structures based on estimation of the initial stress field, rock physical property tests, and model tests, together with preliminary field data, is presented, and a specific example given.

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INTRODUCTION

The Problem

In the past, underground mine openings have been designed according to previous experience. In many instances, differences in rock types, depths, geologic structure, and pressures were not taken into account in the design of underground mine openings. As a result, failures of mine structure, followed by accidents, sometimes occurred, since a suitable size and shape of mine opening in one mine was not always applicable in another mine with different conditions.

A universal scientific approach to the problem of stresses around mine openings has not been previously available; however, many investigators have been aware of the fact that this is one of the key factors in mine safety.

In contrast to other structural materials utilized by man, rock bodies are not entirely homogeneous nor perfectly elastic, and the rock may be interrupted by fissures and fractures, with or without bonding agents. In addition, the mine openings driven are not always simple geometric figures. For these reasons, investigations of underground mine structure problems can not be based on mathematical considerations alone, but must include the interpretation of laboratory and field data.

Statement of the Problem

The purpose of the research outlined in this paper is to present a method of design of underground mine structures based on laboratory data and the results of preliminary field studies.

This research is confined specifically to the determination of the critical dimensions pertinent to room and pillar mining, but the general approach to design of mine structures outlined is applicable to any method of mining.

Field and experimental data from one specific gypsum deposit in Indiana has been chosen as an example, and utilized in design and prediction of the behavior of a mine structure.

Importance of Study

It is universally recognized that a regular mining plan results in a higher degree of safety than a haphazard one. The development of a regular mining plan does not necessarily guarantee safety, however. In the past, this development has been treated largely as an art, and has depended primarily upon the experience and personal judgement of the persons responsible. The purpose of this research is to attempt to establish a systematic approach to the problem which will permit a reduction in the amount of previous experience and personal judgement which must be utilized in the development of a mining plan. The scientific approach to the design of underground mine openings should result in a more uniform, high degree of safety.

Safety should be given first consideration in mining operations. Since the failure of unstable ground has been directly responsible for a large percentage of injury to mine workers and damage to mine structures, it is imperative that any possibility of ground instability be removed during the mine design stage.

The economic importance of stable underground workings should not be underestimated by any designer. The achievement of continuous production with minimum cost of support and with a satisfactory factor of safety is of primary importance.

The adoption of an engineering approach to the design of the underground mine structure should lower the incidence of production slow-ups due to unstable ground conditions.

Procedure of Study

There are several methods which might be used to arrive at a criteria for design of underground openings: (1) theoretical analysis; (2) model studies; (3) field observations of mine openings under similar conditions; or (4) experiments conducted at the mine location. The performance of experiments in place in a mine is usually costly, dangerous, inconvenient, and time-consuming work, and, therefore one of the other methods of obtaining experimental data is necessary, if a study of mine stress problems is to have any practical value.

This study has been conducted in three phases: (1)

an investigation of the pertinent literature; (2) formulation of a systematic approach to the problem of the design of underground mine openings; and (3) an illustration of the proposed method based on laboratory experiments for a particular mining operation.

The literature investigation revealed that published material pertaining to the design of mine openings can be classified in three categories: the initial stress field in the earth's crust; the physical properties of mine rocks; and the effect of mine openings upon the behavior of the surrounding rock.

Information pertaining to the stress field in the earth's crust was primarily based on theoretical hypotheses and the result of geophysical investigations. Rock strength, elastic properties, and apparent specific gravity are physical properties which previous investigators have considered important from the structural point of view. A survey of the accepted testing methods, and accumulations of test results from various sources were utilized in the formulation of this paper.

Evidence has been presented in the literature that the observed physical properties of mine rock can often be improved from a structural standpoint by the use of cementation processes. The library investigation of this problem included a comparison of the strengths of different bonding agent-rock combinations.

The literature investigation consisted of a historical review of attempts of people connected with mining to explain phenomena observed, measured, and calculated related to the underground stress problem. In recent years, model testing appears in the literature as a common attempt to solve the mine structure problem.

A systematic approach to the design of underground mine openings is presented here, based on what seems to be a natural separation of the material which was presented in the literature review. This method consists of three steps: (1) the determination of initial stress in the earth's crust; (2) the determination of the physical properties of the rocks, and (3) the determination of the effect of underground mine openings. A suggested procedure for the completion of each of these steps is outlined, and, finally, the interpretation of the data obtained and its application to the underground mine structure is indicated.

A gypsum mine at Shoals, Indiana was chosen to illustrate the application of the suggested method of mine opening design. The initial earth stress was estimated using the suggested procedure. The physical properties of rock from the mine were determined in the laboratory using standardized methods, and compared to properties of rocks of different types obtained from the literature review. The effect of mine openings in this particular case was determined in the laboratory from centrifugal model tests. A suggested room and pillar mining plan for the Shoals mine is presented based on the data obtained.

Tests were also performed to determine the effect of different cementing agents upon the physical properties of fractured gypsum rock, and conclusions are presented based on this data.

Definitions of Terms

Mining Terms

Artificial Support: Stulls, post, cribs, slabs or reinforced-concrete pillars placed systematically or at irregular intervals in underground mine openings.

Cementation: Introduction of a foreign material with strong adhesive and cohesive properties into fracture planes of rock in an attempt to increase the strength of the rock mass.

Hanging Wall: Ore limit or wall-rock on the upper side of a dipping ore body; called "roof" in horizontal bedded deposits.

Footwall: Ore limit or wall-rock on the lower side of a dipping ore body; called "floor" in horizontal bedded deposits.

Stope: An underground excavation other than a development opening resulting from actual mining of of ore, whose outline is determined by the outline of the ore body.

Shaft: A vertical or inclined opening, giving access to and serving the various levels of a mine.

Pillar: Columns of country rock or ore left in place to serve as support for the underground openings.

Span: The length of the roof between two pillars.

Room and pillar method: A mining method applied primarily to horizontal bedded deposits in which openings and pillars alternate systematically throughout the mine plan

Applied Mechanics Terms

Stress: An internal force per unit area.

Strain: Unit change in dimension in a specific direction in a body. (17)

Strength: That particular combination of stresses that a material can undergo before failure either by rupture or plastic flow for a specific time and temperature condition. (17)

Elasticity: The ability of a body to retain deformation even after the removal of the previous applied forces. (46, p. 9.)

Young's Modulus of Elasticity: Within the elastic limit, stress is proportional to strain; the ratio of stress and the corresponding strain has been called Young's Modulus of Elasticity. (There are uniaxial, biaxial, and triaxial modulii of elasticity; also compressive, and tensile modulii of elasticity.) (46, p. 2.)

Poisson's ratio: If a uniform uni-directional stress is applied to a specimen which is allowed to expand freely in all directions at right angles to the direction of the applied stress, the ratio of the linear lateral (transverse) strain to the linear longitudinal strain within the elastic limit of the material is defined as Poisson's ratio. (45)

Strain Energy: The sum of all the work done on a body during a compression or extension of that body.

Miscellaneous Terms

Initial stress: Stress occurring within the earth's crust before the existence of any underground excavation.

Stress concentration: The ratio of peak stresses within a physical body, relative to the average stress within the body.

Sima: Dark rocks, including basalt and related types, of specific gravity about 2.9-3.0 and still heavier rocks of S.G. up to about 3.4.

Geosyncline: An elongated downwarping of the crust of the earth, forming a deep trough in which a great thickness of sediments has accumulated, especially in the central portion.

Interbeds: A number of layers of different kinds of rocks occurring in such a manner that they are superimposed, one on the other, irregularly, in varying thicknesses.

Mining overburden: The total amount of material, such as rock formations and sediments, overlying the top of underground mine openings.

Prototype: The original item of interest on which subsequent models are to be based. In the field of rock mechanics, the prototype is simply the mine structure itself.

Polarization: A condition of radiant energy, most noticeable in light, in which vibrations assume a definite form or direction when subjected to special influences.

Strain-gage: A gage for the purpose of unit-deformation measurement. A common form is the electrical resistance gage which detects strain by a change in grid wire resistance.

CHAPTER II

REVIEW OF THE LITERATURE

The Underground Mine Structure Problem

Stability of underground mine openings is one of the important problems in the mining industry, since failure of roof, rib or face of the underground openings may result in fatalities and work stoppage.

A detailed analysis of injury experience in coal mining published by the Bureau of Mines (88) states that coal mines in the United States in 1948 had one of the lowest injury records since 1934. A total of 54,471 fatal and non-fatal injuries occurred. Of the 999 fatal injuries at all mines, 880 occurred in underground workings; fall of roof and face caused the death of 554 men in underground workings during this period. This represented 59 percent of all fatalities at underground mines.

The Bureau of Mines indicates (87) that a total of 18,114 fatal and non-fatal injuries occurred in all coal mines in the United States in 1954. Of these, 374 fatalities occurred in deep bituminous and anthracite coal mines (342 in underground workings). Of these 342 underground injuries, 213 were the result of failures of roof, face or rib, comprising 62 percent of the total fatal accidents.

Statistical data (85) from the Bureau of Mines shows that accidents in the coal mines of the United States during 1955 and 1956 resulted in 862 men losing their lives; of these, 737 were lost underground, and 454 were fatally injured due to falls of roof or failure of rib or face; thus, failure of roof, rib or face comprised 62 percent of the fatal accidents that occurred underground.

Records (86) of the injury experience in 1954 in the metal and non-metal industries in the United States indicate that a total of 6,045 fatal and non-fatal injuries occurred during this period. Of these, 95 were fatal and 72 out of these 95 took place underground.

The Bureau of Mines (86) analysed accidents occurring underground in the iron mines of Lake Superior District in 1956, and found that 73 of the 346 were from the falls and slides of ground.

From the above data, the importance of the investigation and design of the mine structure with the aim of prevention of accidents can be seen.

Previous Approaches to the Mine Structure Problem

The problem of the stability of the underground mine structure has been approached through different methods by many investigators. These methods have been based on observations, mathematical analysis, prototype, experimentation, and model analysis—centrifugal model tests, photoelastic model analysis, etc.

Although underground stress distributions and their related structural problems have been investigated by engineers who have constructed tunnels in the civil or excavation engineering field, there has not been any great amount of literature published concerned with underground mine structure problems and their solution.

The first approaches to the mine structure problem were on a purely empirical, cut-and-try basis. The design of the mine was initially based on a mining plan which had proven successful under seemingly similar conditions. If this were not successful, more conservative designs were attempted, until an underground mine structure was arrived at which stood up for the required period of time. This empirical method did not furnish any guarantee of a margin of safety for the mining plan.

The first semi-theoretical approach to the mine structure problem may be traced back to the early 1880's. (5) This study and several others which followed were based upon the observed behavior of mine openings. (5) (18) (29) As a result, conflicting conclusions were drawn. No attempt to explain the observations with regard to the earth's stress field or the physical properties of the rocks involved was made at this time. A number of investigators in more recent times attempted analysis of mine structures from a purely mathematical viewpoint. (1) (5) (8) In most cases they confined themselves to the treatment of rock as an elastic medium and mine openings as smooth geometric figures.

As recently as 1951, investigators (24) became aware that the variation in the earth's stress field may be important from the mine structure viewpoint, and approximations, such as hydrostatic, unidirectional, etc., have been used to approximate the earth stress field at different depths.

In addition to empirical and mathematical approaches, model analysis, such as photoelastic, centrifugal and other model analogies were adapted to this problem. This approach has become increasingly popular during the recent years. (2) (3) (4) (9) (10) (24)

The most recent tool which has been applied to the Solution of underground stress problems is soil mechanics. (22) Physical properties of rock are expressed in terms previously used for soil masses.

The Initial Stress Field

The stress in the earth's crust before any excavation or mining operation may be defined as the initial stress field.

L. A. Panek suggested that the initial stress field, regardless of how it was induced, whether by dynamic or static processes, may be separated into two components according to the direction of stress; a vertical stress field and a horizontal stress field. The former is primarily compression, with or without shearing stress, and the latter is composed of compression, tension or shear. (72) (24)

A large portion of the stresses which occur in the earth's crust were assumed to be of unknown origin, (65) (67) or were derived from a complex combination of different sources. However, an approximation based on practical observations and theoretical hypothesis could supply sufficiently accurate data for design purposes in most cases.

J. T. Wilson indicated that the vertical stress field is primarily determined by the effect of gravity upon the mass of rock overlying a particular point in the earth's crust. In addition, vertical stresses are induced by intrusions and extrusions of magma, doming intrusions of salt, and glaciation processes. (76) (74) There were many instances of vertical stress fields in the earth's crust for which there was no good explanation.

The most common explanation for the horizontal stress field within the earth's crust was that it is a result of the lateral restraint supplied by adjoining rock. (50) (65) (69) As a result of the geotectonic processes, fractures occurred, to a varying degree, within the earth's crust. Due to the variation of rock elastic properties with confinement, fractures were more easily produced at shallow depths. (70) As a consequence, lateral restraint was reduced, and the net horizontal stress became relatively small or non-existent at shallow depths. (24) (66) At extreme depths, the combination of high vertical and horizontal pressures resulted in plastic deformation of the rock and subsequent increase of the horizontal stress field to a theoretical maximum equal to the applied vertical stress field (the so called hydrostatic state). (5) (24) (77)

Horizontal stress fields were thought to be induced by large scale geotectonic processes as well as the sources described above. Theories based on contraction of the earth, of subcrustal convection currents, of atomic heating, of formation of denser minerals, or of chemical processes have been proposed at different times. (77) (75) (76) Each can explain a specific observed phenomenon, yet none provides conclusive evidence of completely universal application.

It was believed by many people that the contraction theory was capable of explaining more of the data observed than any of the other theories mentioned above. (74) (76)

Several hypotheses were recently presented to explain the presence of stress fields of continental proportions within the earth's crust. S. Paige (74) summarized the most generally accepted viewpoints on this subject. He assumed that the kinetic energy of the earth's revolution is the source of instability within the earth's crust. He also stated that the physical and chemical instability of the simatic crust has caused deformation and transformation of the earth's crust, and that the unbalance of light continental segments and heavy ocean basins has released forces in a chain reaction that continually deforms and transforms the surface of the earth. The deformations resulting from these two sources induce horizontal and vertical stresses within the outer layer of the earth's crust. Furthermore, a hydrostatic stress field at relatively shallow depths may result from the temperature-pressure gradient within the earth. The dioritic-granitic intrusions derived from deep geosynclinal depressions, as well as the continual additions of basaltic rock during epeirogeny, are all possible sources of induced stresses in the earth's crust. Finally, he emphasized that most of the forces that have been invoked in the past are losing strength with time.

Mathematical explanation for the plastic buckling of the earth's crust was undertaken by F. A. Meinesz (71) to indicate the origin of geosynclines. He stated that the development of geosynclines requires continuing compressive stress. Relaxation of the crust, resulting in a decrease in stress, and a beginning of readjustment to isostatic equilibrium, takes place when the compressive stress vanishes.

F. Birch (67) considered that the principal stresses at a point in the crust can differ by no more than two times the maximum shearing stress that the material can support. The principle horizontal stress must fall within the limits $\bar{d}gh \pm 2s$ ($\bar{d}$ is the mean density, g is the acceleration of gravity, h is the depth). The mean pressure, which is one-third of the sum of the principal stresses, must be within the limits $\bar{d}gh \pm 4/3s$; where s is the maximum shearing stress the material can support.

Physical Properties of Mine Rock

There have been a large number of investigations published which were concerned with the physical properties of rock. However, the majority of these studies have been concerned with physical properties which are involved in comminution of mine rock, or which affect drilling rate, explosive efficiency, or other allied problems (44) (45) (53) (64). Relatively few papers have been published which deal with the relationship between rock physical properties and the behavior of underground mine structures.

Summing up the opinions of Obert (53), Windes (64), and Hardy, Jr., (44) (45), it is clear that prior to theoretical calculations involved in any design of a stable mine structure, physical properties of rocks of that certain area should first be determined; and should be compared with the physical properties of similar types of rocks. Therefore, handbook-like data of physical properties of rocks of all kinds obtained by standardized methods, together with macroscopic and microscopic descriptions based on a logical classification, is necessary. It is even more important to have a complete understanding of the factors affecting the physical properties of mine rocks.

At the same time, standardization of established methods of testing is of prime importance, as will be indicated in the succeeding sections.

The majority of the tabulations of rock physical properties are too generalized, do not have sufficient data for statistical analysis, do not have an adequate petrographic description, and have not followed a standardized procedure; so that an accurate correlation with other rock types is not possible. Tables of rock properties based on studies by the U. S. Bureau of Mines, and the Bureau of Reclamation are exceptional in this respect and provide a convenient source for comparison. A table condensed from these sources for a selected group of rocks is included in the following pages.

The question of the significance of the results of rock physical property tests has been raised by many critics. Obert, Windes and Duvall have stated (53) (64) that the possible reason that more use has not been made of the physical properties of rocks in determining mining methods is the large number of variables which must be considered by the investigators, making correlation of laboratory results with field data difficult. The disturbing influence of heterogeneous, fracturing, and other petrographic features upon field measurements has been mentioned. In spite of these difficulties it was felt that progress could be made by a properly oriented analysis of the physical properties of mine rocks. No attempt was made by the authors to evaluate the influence of any of these petrographic features upon physical properties of rock determined in the laboratory.

The Effect of Mine Openings on Underground Stresses

The presence of mine openings has the effect of

Table I

Compressive Strength of Rock

Type of Rock	Compressive Strength psi (average)	Standard Deviation D or Range R
Andesite (USBR)	18,710-19,150	3,360-3,510 (R)
Basalt (USBR)	24,450-31,850	9,879-16,780 (R)
Gneiss (USBR)	9,310-15,140	0 0 (R)
Granite	33,200	3.2 (D)
Granite, slightly altered (USBR)	8,250-9,400	3,820-4,820 (R)
Limestone	10,900	5.3 (D)
Limestone, reef breccia (USBR)	860-4,960	190-4,080 (R)
Marble	30,800	9.0 (D)
Sandstone (two types)	10,400 & 6,100	4.6 & 14.5 (D)
Sandstone (USBR)	8,810-12,200	7,470-5,210 (R)
Schist, biotite (USBR)	7,750-12,010	0 0 (R)
Schist, biotite-chlorite (USBR)	5,290-17,000	0 0 (R)
Schist, biotite-sillimanite (USBR)	1,160-4,930	0 0 (R)
Schist, biotite-sillimanite-quartz (USBR)	1,250-4,520	0-2,800 (R)
Slate	30,400	5.2 (D)
Tuff (USBR)	530	190 (R)

(70, p. 52) taken from U.S.B.M. Report of Invest., 4459, 1959, and U.S.B.R. Concrete Lab Rept., sp 39, 1954.

Table II

Tensile Strength of Quarried Rocks

Rock Type	Diameter Core size, Inches	Average Tensile Strength lb. per sq. in.	Standard Deviation Percent
Marble	7/8	700	20
Limestone	7/8	520	1
Granite	7/8	930	6
Sandstone (1)	7/8	105	-
Sandstone (2)	7/8	149	14

Table III

Modulus of Rupture of Quarried Rocks
(loaded perpendicular to bedding planes)

Rock Type	Modulus of Rupture lb. per sq. in.	Standard Deviation Percent
Marble	2,770	6
Limestone	1,550	9
Granite	2,900	6
Sandstone (1)	454	15
Sandstone (2)	430	26
Greenstone	6,680	14

(53) from U.S.B.M. Rept. of Im., 3891, 1946.

altering the underground stress field. The magnitude of this disturbance, and its manifestations, in the form of loads on artificial supports, failure of roof and pillar, and even more spectacularly in the form of rock bursts, has long been a question provoking the mining engineer.

Many previous investigators have tried various approaches to this problem. These methods have included: (1) semi-theoretical; (2) mathematical (based on the theories of elasticity, plasticity, and soil mechanics); (3) empirical; and (4) laboratory (such as photoelastic study, centrifugal model tests, triaxial tests, pressure vessel tests, etc.) analyses.

Previous Work on this Subject

The first semi-theoretical approach to the mine structure problem may be traced back to the early 1880's. At this time, Fayol and Rizha (5) proposed the first "dome" theory for mine openings. A spherical shell was assumed to exist surrounding a mine opening, within which the rock was assumed to be loaded by its own weight. Failure of the rock indicated that this weight was greater than the cohesive force of the rock.

In 1898, G. Kirsch (5) presented a theoretical solution of the stress distribution around a circular hole in an infinite plate under uniform stress parallel to one axis. This solution is applicable to a mine opening whose axis of symmetry is long compared to its cross-section dimensions.

In 1908, J. C. Meem (18) presented the original theory of arching. This "arch" was assumed to relieve the pressure around the excavation. He also mentioned that the greater the angle of repose of the rock, the greater the pressure exerted on supports inside the arch.

In 1923, G. S. Rice (29) presented a theory in which it was assumed that the layers of rock overlying a mine opening act as restrained beams loaded by their own weight. When failure occurs, inward shearing of both ends of each beam causes the occurrence of a dome, inside of which the rock is fractured and de-stressed.

In 1934, P. B. Bucky (2) performed centrifugal model tests to observe the effects of approximately vertical cracks on the behavior of horizontal lying roof strata of sandstone, and the effect upon roof strength of cementing and grouting existing cracks. From the results it appeared that the vertical cracks decreased the strength of the rock beam 15 percent, approximately, and increased the amount of subsidence. Restrained beams had greater strengths. Bucky proved that the cemented beams had greater strength than the original beams containing vertical cracks.

In 1935, J. R. Dinsdale (7) proposed that an egg-shaped pressure ring surrounds mine openings, and concluded that the pressure on the surface of the ribs and supports within the opening is small in comparison to the stress within the ribs a short distance from the inside of the opening.

In 1937, J. Spalding (32) established a theory of stress distribution, using the term "ring stress," based on assumptions of elastic behavior. This theory was little different from the previous work of Kirsch.

In 1941, W. H. Evans (11) developed the Voussoir beam theory to predict behavior of roof strata, and verified his theory with tests of the strength of composite brick beams.

In 1944, J. J. Reed (27) performed model tests using a pressure vessel, simulating a hydrostatic stress field. The results of his work indicated that failure generally occurred simultaneously for openings, regardless of shape, if the same loading conditions prevailed.

In 1948, R. P. Schoemaker (31) reviewed and critized Dinsdale's dome theory, Fenner's analysis of stress distribution and plastic flow, and Van Herson's granular mass theory of subsidence. His criticism was that all of these theories are based on the assumption that the rock is stressed within its elastic range.

In 1948, W. I. Duvall (9) (10) presented an extensive study of single and multiple openings of various shapes based on mathematical and photoelastic analysis. Diagrams included illustrate variation of the stress distribution due to shapes of openings.

In 1949, L. A. Panek (23) performed centrifugal model studies, obtaining safe roof spans, and safe and economical arch structures.

In 1949, W. R. McCutchen (17) presented examples of the effect of static and dynamic loadings on rock masses, including the stress distributions around a circular, cylindrical, single opening.

In 1949, H. K. Van Poollen (35) presented the results of photoelastic model analysis of single, variously shaped openings, and pointed out the importance of wall support for the stability of mine structures.

In 1949, J. Spaulding (33) compiled existing theories and assumptions of stress concentrations around mine openings with particular attention to arching and doming. He gave a description of the design of mine plans to avoid high stress concentrations. A rather complete discussion of the theoretical and practical effects of geologic factors in relation to underground mine openings was also given.

In 1951, L. A. Panek (24) described the limiting conditions of the earth's stress field, and obtained stress concentrations for variously shaped single and multiple openings in a uniform stress field, based upon photoelastic experiments. A considerable quantity of laboratory data was given. Comparisons between the behavior of drifts of different shapes under various stress field conditions were also given.

In 1954, R. G. K. Morrison (21) described the development of a stress zone, fracture zone, and dome around an underground excavation. The area, dip, time, and rock strength were assumed to affect the size, shape and stability of the dome.

In 1954, C. T. Holland (13) presented a discussion concerning stress distribution which emphasized pillar design. A rather complete consideration of factors which influence the stress distribution in pillars and the design of pillars was included.

In 1955, R. D. Caudle and G. B. Clark (5) applied mathematical solutions to simple and restrained beams under three loading conditions, and the similarity of the results was pointed out. The applications by various investigators of the theory of elasticity to the analysis of mine openings of different shapes, and to single and multiple openings was summarized.

In 1955, R. G. K. Morrison and P. G. Coates (22) applied soil mechanics to the rock failure problems. Stress distribution around mine openings, vertical shafts and within the rock mass were discussed.

In 1956, S. Boshkov (1) presented a mathematical analysis of the underground mine stress problem which was entirely based on the theory of elasticity. This was an extension of Dr. Ko Suzuki's development of the stress distribution around elliptical openings as a function of inclination. Included was a summary of Duvall's work, with particular emphasis on the advantage of the photoelastic method. Boshkov also mentioned consideration of rock as a soil, and introduced the concept of deformation as a function of time, showing necessity for the evaluation of a safety factor based on time. He suggested that determination of constants of discrepancy between pure theory and practical results related to mine opening stress analysis should be carried out.

In 1956, Roux, Denkhaus, and Leeman (30) presented a mathematical, photoelastic model analysis and also developed de-stressing practice. The purpose of the investigations were to approach the problems of rockburst in the Witwatersrand goldfield though these various methods.

In 1956, J. J. Reed (28) performed experiments concerned with re-distribution of stress concentrations about mine openings. He found that the high skin-stress can be reduced by means of a wedging action which increases compressive stress at depth. This study was a continuation of his earlier pressure vessel studies (27).

In 1959, R. H. Merrill (19) performed empirical work for the design of underground mine openings in an oil-shale mine in Colorado. Equations based on various theories and assumptions were used to calculate allowable roof span and sag of the roof and verified in the field.

In 1959, A. V. Corlett and L. L. Emery brought forth the idea of prestress phenomena as applied to rock beams. They suggested that the mechanics of consolidation develop a stress component perpendicular to the direction of compaction, especially in sedimentation. This may be a source of prestress. They stated that when the rock beam was loaded to design limits, the lower chord retained some compression and the whole beam depth was effective in compression, therefore, compressive failure of mine openings can be represented by tests of prestressed experimental models. They also mentioned that redistribution of unilateral stress around an opening is an important factor. It was claimed that no rock is actually a Hooke Solid, rather most are St. Venant Solids. Mathematical analysis and photoelastic measurements were performed on representative models of mine openings.

Methods of Investigation

There are three methods which are commonly used by investigators: (1) mathematical considerations, consisting of calculations based on either theories of elasticity or plasticity or the application of soil mechanics; (2) empirical treatments; and (3) model analysis, which includes the photoelastic method, centrifugal method, pressure method, etc.

Mathematical considerations.

Theory of elasticity. Problems concerning the distribution of stresses are often solved by the classical theory of elasticity. There are, however, many limitations and assumptions which should be considered when the theory of elasticity is applied to the problems of the effect of underground mine openings on stresses. These assumptions are as follows: (59) (5)

(24) (47) (17).

(1) The rock must be assumed as a constituently homogeneous and mechanically isotropic mass.

(2) There should be no fractures or any geologic discontinuities in the stressed rock body. In other words, the compatibility requirement in the mathematical solution should be satisfied.

(3) The boundary conditions in the mathematical treatment should also be satisfied. The stresses within the rock body must be in equilibrium with the external loads.

(4) The applicable range is only within the elastic limit. When inside this range, the rock should behave as a perfectly elastic material.

(5) The boundary of the opening should not be very irregular. That is to say, that the mine opening should be a definite geometric shape.

(6) The length of the mine opening should be much greater than the cross-section; and the length of the opening is assumed to be horizontal. These assumptions are not necessary if the more difficult three-dimensional stress analysis is attempted.

<u>Theory of plasticity</u>. According to definition, a deformation becomes plastic when the stress in the material exceeds the yield point, and complete recovery

is not observed after release of the stress. (57)

Whenever a portion of the interior stress of a material is relaxed under the influence of the exterior disturbances (especially thermal agitation) or due to discontinuous movements within the rock (such as are postulated in the current dislocation theories), the equilibrium is disturbed. As a consequence the exterior applied stress will cause readjustments of the dimensions of material until the interior stresses have regained their original value, or equilibrium is restored. This process will be repeated continually if the relaxation continues. Consequently, a state of flow will be reached. (57) (46)

From the above mentioned plastic flow phenomenon, it is evident that time is an important factor in the elastic-plastic deformation processes. Therefore, a deformation, which, when executed quickly, is perfectly elastic, may become plastic if it is executed slowly.

Deformations in geologic processes in which static loads are applied over a long period of time are similar to the deformations of rock due to stresses induced by introduction of underground mine openings. These deformations may not be very far from those predicted by assumption of plastic flow as influenced by the time factor. (37)

It is evident from short term static tests that rock behaves plastically below the yield point, even though the tests require only a few minutes. Therefore, the plastic deformation of rock cannot be ignored. An approach to the investigation of the effect of underground mine openings on stress distribution based on the theory of plasticity deserves further study. (37) (57) (46)

Empirical methods. Empirical methods are primarily a result of observation and measurement in place, of deformations and the other behavior of prototype mine openings, before or after the failure of the surrounding rock. This method is often used to extend hypotheses obtained from previous mathematical and model analysis. As an independent method, mathematical equations are often developed which satisfy the observed data. (the original theories of Fayol and Rizha were a result of empirical analysis.)

Empirical methods alone do not provide overall information as to the effect of underground mine openings on stress. The shortcoming of this type of approach is the tendency of individual empirical hypotheses to have only limited geographical application. Use of empirical methods to check the results of other methods, however, is valuable. (36) (19)

Methods of empirical analysis vary from place to place, depending on the nature of the individual problem. Description in detail is beyond the scope of this section.

Model analysis. Mathematical methods may be difficult or impossible to apply analytically to the behavior of some underground openings because of the irregular shape of the openings and the complex nature of the surrounding rock, such as heterogeneity, discontinuities and anisotropic properties. Therefore, investigators must find other methods of analysis. Previous experiences in other structural engineering fields, have shown model analysis to be a most valuable method of investigation since it provides information concerning major aspects of design, quickly, conveniently and inexpensively. (2) (3) (25) (27).

The common methods of model analysis applied to the study of underground openings include photoelastic, centrifugal and pressure vessel methods.

<u>Photoelastic methods</u>. The principle feature of photoelasticity is the utilization of the optical property of plastic plate to become doubly refracting when subjected to external load.

Similar to the cross-section of the prototype mine openings, openings are made in plates of plastic which are transparent and optically isotropic before load is applied. This plate is loaded externally with a scale representation of the prototype load. Polarized light is passed perpendicularly through the plate, and the change in the optical properties of the plate due to the external loading is observed.

Two common methods for producing polarized light for the use in photoelasticity are (1) the use of an artificial polaroid material; and (2) the use of a nicol prism. The plastic model is placed between two sheets of polaroid material; the first is called the polarizer, and the second, the analyzer. Because different magnitudes of the principle stresses occur inside the plate, the optical properties of the plate become different along two mutually perpendicular directions, and vary from point to point. The polarized light from the polarizer will be broken into two systems of transverse waves, one of which has been retarded relative to the other by the presence of the model. The light will then be passed through the analyzer, and upon emerging from the analyzer, will consist of a transverse vibration in one plane. The intensity of the light transmitted will be zero where the principal stresses are parallel to the axis of the polarizer. In addition, the intensity of the light will vary sinusoidally with a change in magnitude of the difference of the principle stresses. Photos can be taken; isoclinics can be determined; and from this data, the complete stress distribution in a body can be obtained.

Centrifugal method. The principles of similitude require that if a model is to be made of a body loaded by its own weight, the body forces acting on the model must be increased in direct proportion to the model scale constant, all other factors being held constant. The principle of centrifugal model analysis is to apply centrifugal forces to a model by means of a specially designed centrifuge to simulate underground loadings on the prototype, thus satisfying the similitude equations. (3).

In cases such as the investigation of the deflections of the component beds in a nonhomogeneous prototype, as in a stratified mine roof, it is convenient to apply centrifugal body forces to the test model.

The centrifugal force developed in a centrifuge is equal to mv^2/r, where r is the distance from center of rotation to center of gravity of model, m is the mass, v is the linear velocity. Gravitational force acting on a stationary mass is equal to mg. The ratio of the centrifugal force to the gravitational force is:
(25)

$$k = \frac{mv^2/r}{mg} = \frac{v^2}{gr} = \frac{4\pi^2 rn^2}{g}$$

This ratio is called the model ratio (by Bucky) (2) when r is in feet, n is equal to RPS, and g is in feet per sec.^{2}. Thus, the model represents a prototype of the same material which has dimensions k times those of the model.

Centrifugal model testing has long been used as a method of stress analysis for the purpose of investigation of the proper span of roof. A systematic method of experimental study in this field can be traced back to the early research projects performed by Professor P. B. Bucky of Columbia University starting in 1933.

The results from these pioneer works encouraged later investigators to pursue this particular phase of mine model analysis. Improvements in the original apparatus utilized by Professor Bucky have consisted of an increase in the effective diameter, resulting in a more uniform effective weight of specimen; larger specimen holders, enabling the use of bigger, and more complex models; continuously variable D. C. motors with accurately controlled speeds; and the introduction of strain measuring equipment permitting a nondestructive method of testing.

<u>Pressure vessel method</u>. The main purpose of this method is to obtain a hydrostatic condition by means of pressure transmission through a special hydraulic fluid inside a closed, sealed vessel. The model, consisting of a cylindrical mass, with a model of a mine opening extending along its axis of symmetry is placed in a proper position in the vessel where it can be loaded hydrostatically. (27) (28) (45) (44) (41). Visual observation of the opening deformation may be made through special view ports, or strain gages may be attached to the specimen for strain measurement.

This method has the advantage of inexpensive cost of installation of the equipment. The disadvantages of this method are: (1) the apparatus is limited to hydrostatic stress fields; (2) model size is limited; and (3) it is difficult to develop sufficiently large pressures within such a vessel to cause openings in strong rock to fail.

<u>Stress Concentrations Due to Underground Mine Openings</u>.

The effect of mine openings on underground stress is a subject worthy of investigation. Fundamental questions, such as: What happens to the initial stress field when an underground excavation is introduced? What is the effect of the size of openings upon the stress and strain distribution? Which shape of opening has least stress for a particular condition? What are the magnitudes and distribution of stresses surrounding the boundary of openings within a rock mass? What is the difference in stress distribution between single and multiple openings and what is the mutual effect of multiple openings? Assumptions, equations, and hypotheses have already been established by investigators covering these important topics, as well as many others. A condensed description is given in the following pages.

The initial stress equilibrium of the earth's crust is disturbed when an opening is introduced within the rock mass. This can be explained by the phenomenon of force transmission; that is, the discontinuance of linear force transmission and disturbance of stress equilibrium due to the existence of an opening may cause a change in direction of the induced stresses from radial to tangential in order that the rock mass may achieve a new state of equilibrium. It follows that surrounding the boundary of this opening, on all sides, the rock is in a greater state of stress than it was prior to the existence of the opening; e.g., greater than when it was in the state of the initial equilibrium. (33)

The extra stress, which is in addition to the original stress within the rock body, has been termed ring stress, concentrated stress, or stress concentration by previous investigators. The magnitude of this extra stress varies from different geometric shapes of the openings. Spaulding indicated that the tangential ring stress around a circular hole at a distance r from the center of an opening of radius a is $(a^2/r^2)Q$. (33, p. 17) Three important facts are revealed by this equation: (1) the maximum concentrated stress theoretically occurs in the boundary (skin) of the opening, (2) the concentrated stresses die away as the distance from the center of the opening increases, and (3) the maximum concentrated stress within the rock body surrounding the boundary of an opening is independent of the diameter of that opening; in other words, the size of the opening does not affect the magnitude of the concentrated stress. (33) (21).

As to the size of underground mine opening, Panek (24) added a further statement. When the depth from the topographic surface to the top of an underground opening in a homogeneous rock is greater than about twice the long cross-sectional dimension of the opening, the tangential stress on the boundary of the opening is practically independent of the size of the opening.

The stresses around an opening of a shallow depth are generally tensile in the roof (top) and the floor (bottom); and compressive in the ribs (sides). However, when the lateral pre-existing earth stress is greater than about one-half the vertical stress, compressive tangential stress occurs on the entire boundary of an opening. The ratio of lateral to vertical pre-existing stress is determines whether tensile stress shall occur in the roof and floor. (9) (24).

The ratio of the short cross-sectional dimension of an opening parallel to the larger initial earth stress, to that perpendicular to the initial stress, is the most important factor affecting the critical compressive stress, which increases when the ratio is decreased, (for example, the critical compressive stress is three times the vertical pre-existing stress when the ratio is 1; increasing to nine times when the ratio is 0.25 in the case of elliptical openings in a unidirectional stress field.

In referring to the effect of shape of an opening, Panek (24) stated that for elastic materials an opening of circular cross-section will induce the smallest critical stress under conditions of hydrostatic pressure. In contrast to this, Reed mentioned that there was not much difference in stress induced between a circular and other opening shapes for the same hydrostatic condition, according to the results of his experiments.

McCutchen (17) suggested that the determination of the directions of the principal planes of stress within underground mine openings may best be performed by employing Mohr's stress diagram to a sufficient number of points around the opening. From these principal stress planes, the shear or tensile planes of failure may be located. He found that the fracture planes occur at an angle of 68° from the maximum-principal-stress plane, for the rocks he studied. Spaulding mentioned that the effect of differential stress in rock is a tendency to form cracks parallel to the direction of tangential stress near the boundary of an opening.

The cracks may be entirely independent of the rock formation. (33) (9). (See Figure 1-a.)

Elastic theory indicates that the maximum concentrated stresses occur in the skin rock at the boundary of an opening. As a matter of fact, the stresses in the skin rock never reach this value; conversely, they are nearer zero. This reduction is due to the effect of a natural stress relief action which occurs during the mining operation primarily due to blasting, prior fracturing in the rock and small amounts of plastic flow. Thus, a loose, or semi-loose zone surrounding the boundary of the opening may be developed. The thickness of this zone is not the same in every place. Spaulding has stated that one foot thickness is reasonable in many cases. He also indicated that a zone of influence exists around a mine opening beyond which the extra stress due to the opening's presence can be disregarded. The zone of influence is an important consideration in the case of multiple openings. (21).

A series of various geometrical shapes has been chosen by many investigators to represent the possible effects of different mine openings mathematically and experimentally. These studies can be separated into two groups: (1) single opening, and (2) multiple opening. The most common shape studied in both of the two groups are horizontal cylindrical openings with circular, ellitical, ovaloidal and rectangular cross-sections.

<u>Stresses around various shaped, single openings.</u> A large percentage of the investigations within the field

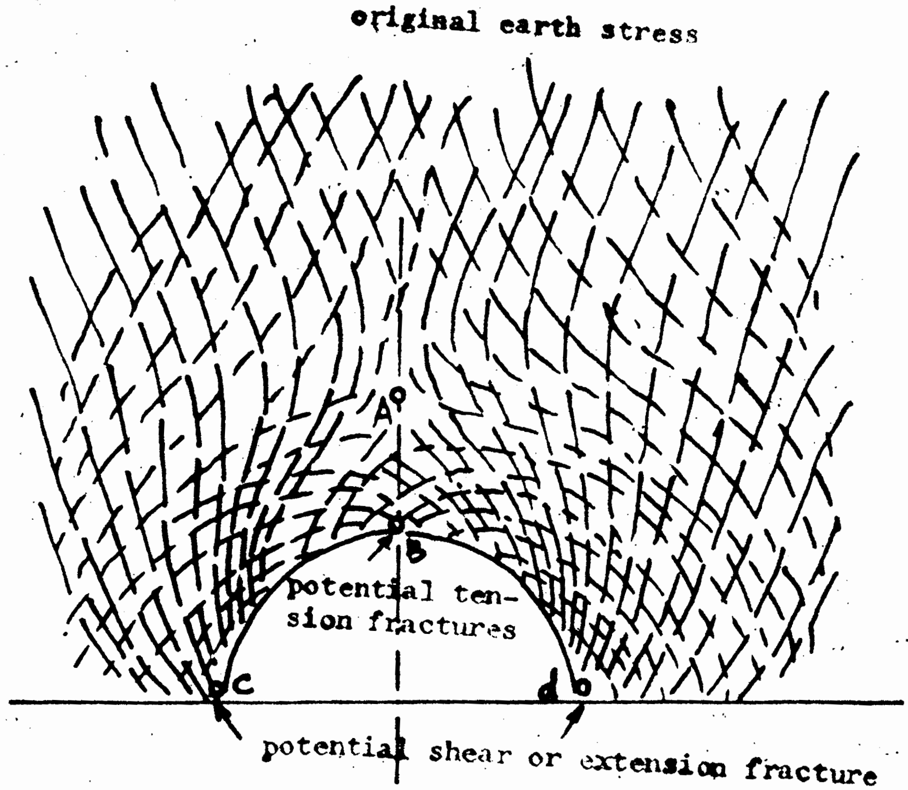


Fig. 1a Pattern of Shear Weakness Planes Surrounding a Circular Tunnel. (after W.R. McCutchen)

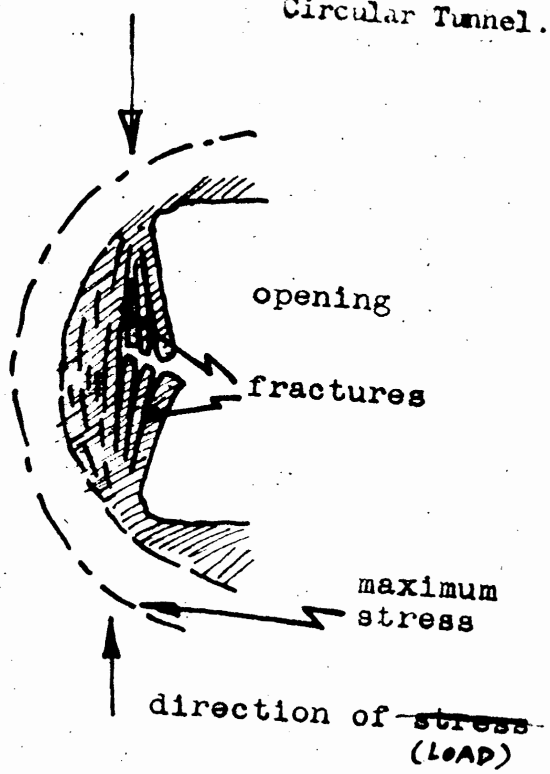


Fig. 1b . Fractures Parallel to The Direction of ~~Stress~~ (LOAD).

(after J. Spalding)

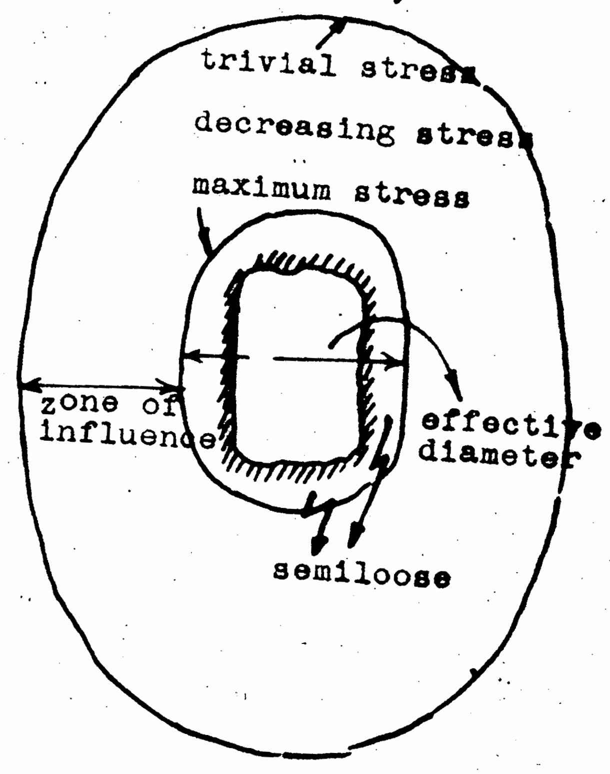


Fig.1c. Zones Around an Underground Mine Opening.

(after J. Spalding)

Fig. 2a Theoretical and Actual Distribution of Concentrated Stresses in an Underground Mine Opening .

(After J. Spalding)

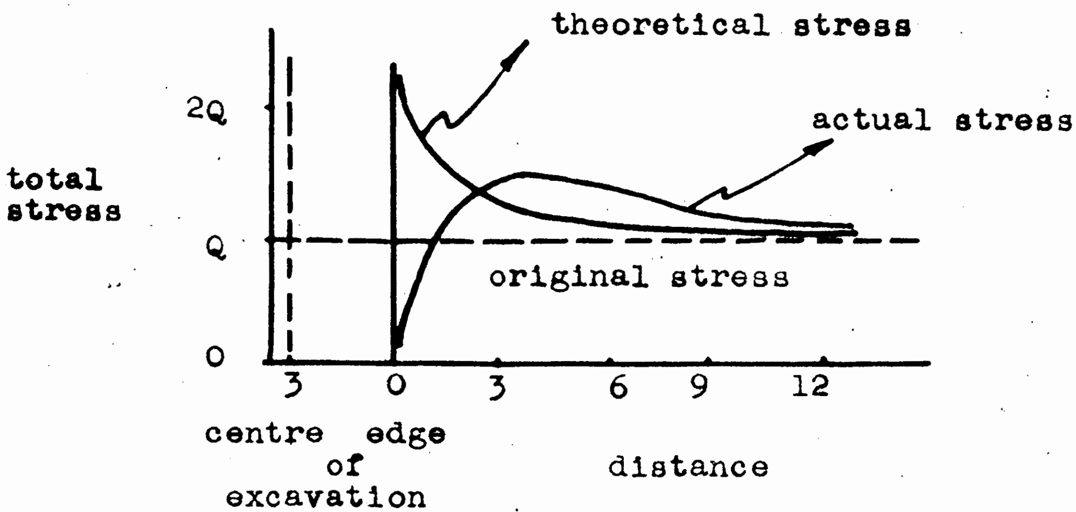
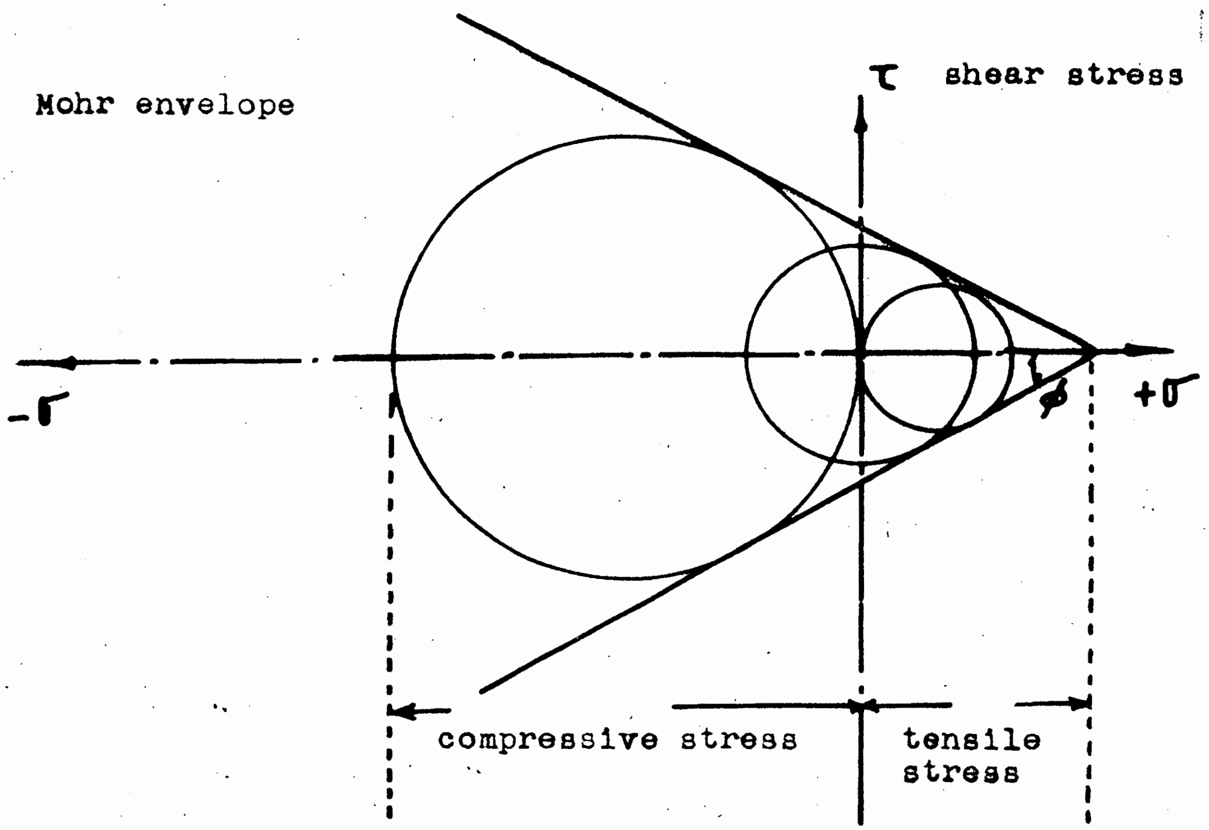


Fig. 2b Mohr Enveloping Curve . (uniaxial, unequal tensile and compressive stresses at failure)



of rock mechanics have been concerned with the effect of various opening shapes on the magnitude of stress concentration, with the objective of finding the optimum shape for the design of mine openings; e.g. the smallest possible induced stress concentration around the opening. (See Figures 5, 6 and 7).

<u>Circular cross section</u>. Mindlin presented a rather complete analysis of stresses around a horizontal cylindrical hole of circular cross-section in a semi-infinite elastic solid based on the theory of elasticity. Panek (24) made a further development and application of Mindlin's work. The important phenomena pointed out for a single, circular opening were: (1) the stress concentrations due to the three simplified initial stress fields are independent of the size of the hole, and are also independent of the elastic moduli of the material; and (2) the roof and floor of the opening are in tension; the ribs are in compression except for the case where the horizontal pressure within the earth is greater than one-third of the vertical pressure, when compressive stress exists around the whole boundary. (See Figures 3 and 4).

<u>Rectangular cross sections</u>. Both Duvall (9) and Panek (23) (24) have performed considerable of photoelastic work concerned with the stresses induced in rectangular openings. Their results indicated that: (1) in the case of the unidirectional stress field, tensile tangential stress exists at the top and bottom boundaries of rectangular openings; the critical points being at the midpoints of the top and the bottom; (2)

Fig. 3

RADIAL STRESS DISTRIBUTION-CIRCULAR OPENING

(After W.I. Duvall)

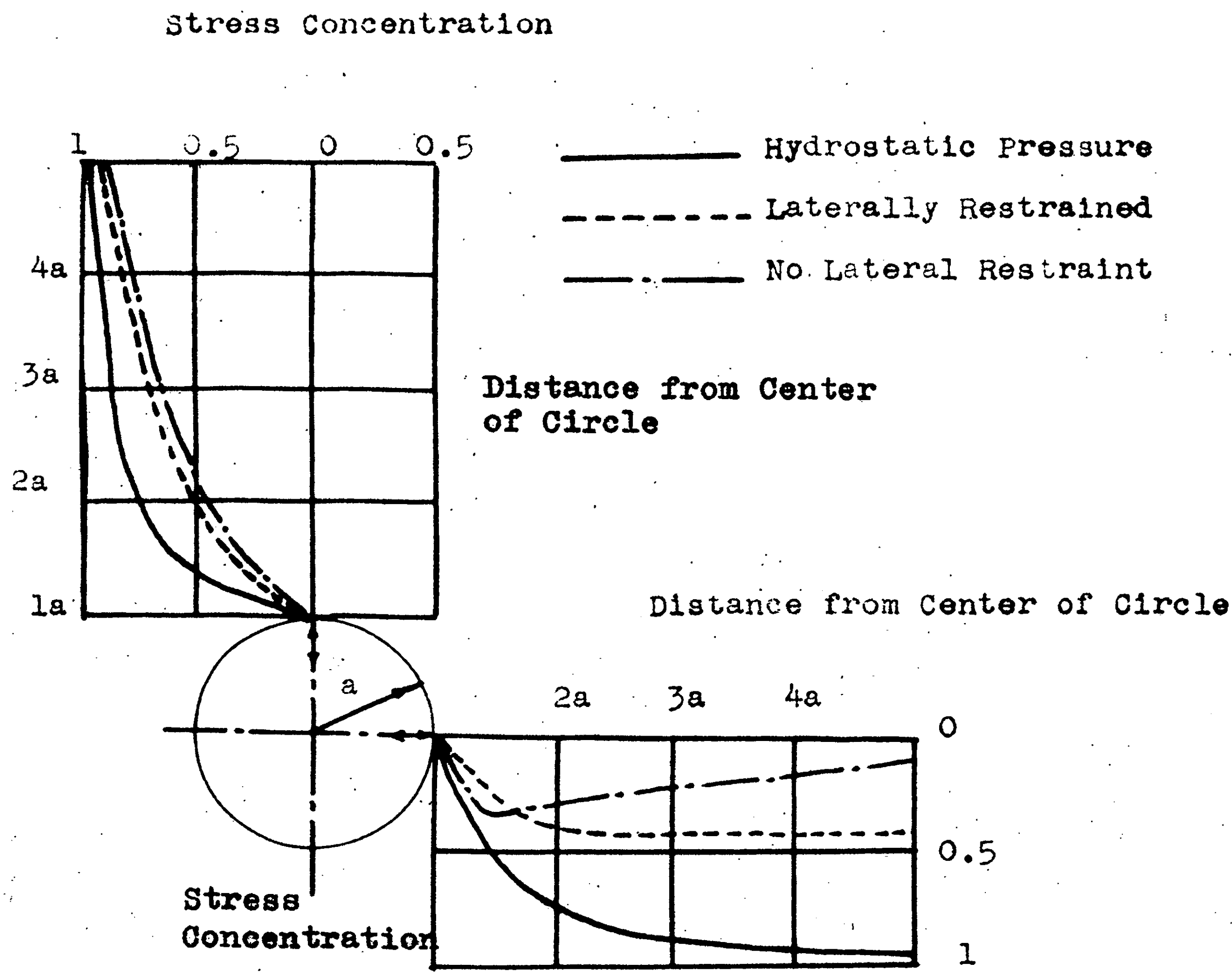
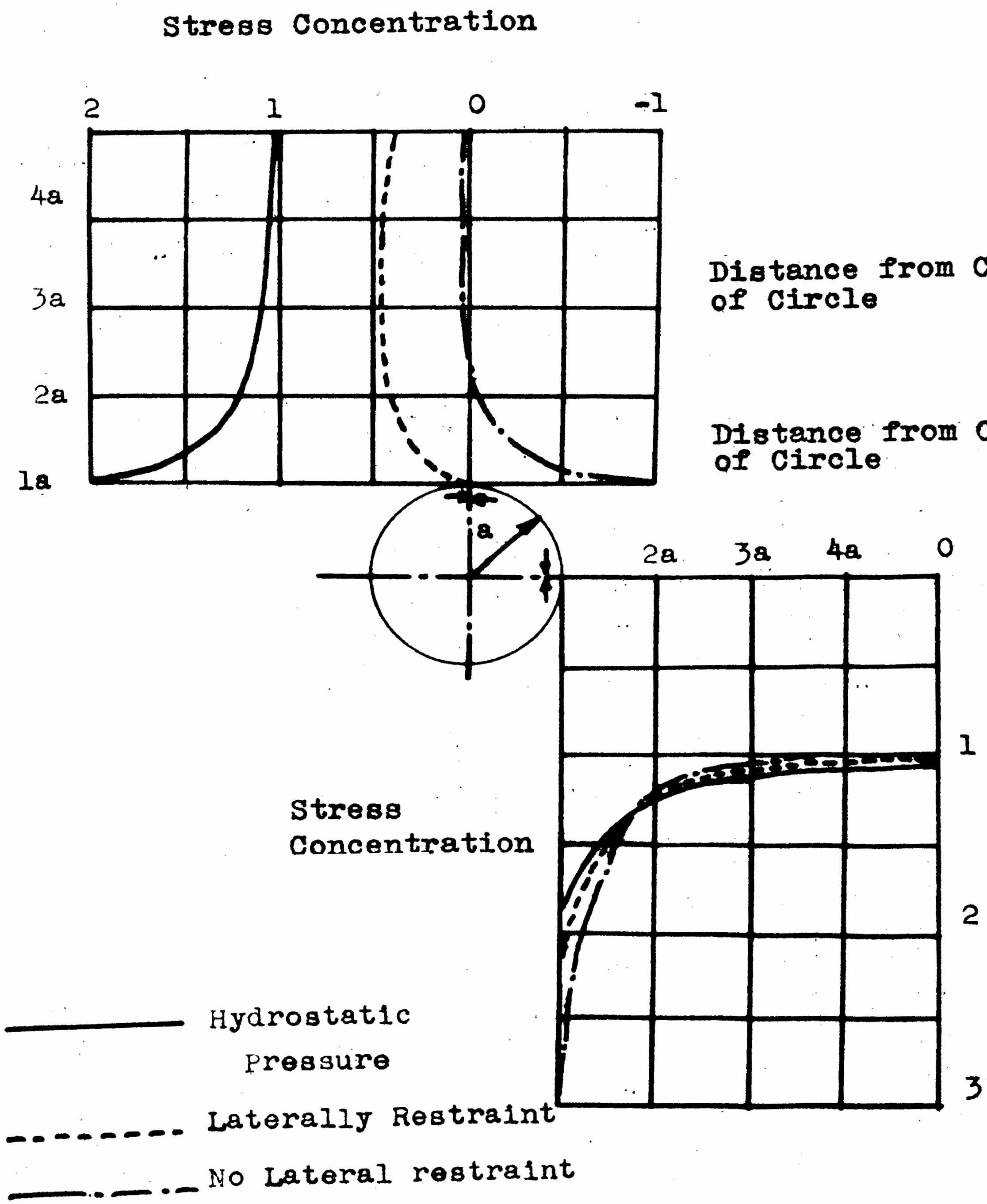


Fig. 4

TANGENTIAL STRESS DISTRIBUTION-CIRCULAR OPENING



(After W.I. Duvall)

the critical compressive stresses occur at the intersections of the fillets and ribs; (3) if the horizontal initial stress is greater than half of the vertical initial stress, the stresses on the boundary of the rectangular opening become entirely compressive; (4) the critical compressive stress increases linearly with increasing opening width to height ratio; (5) the smaller the radius of curvature of the fillet, the closer the critical compressive stress is located related to the center of the fillet, and the greater its magnitude.

Elliptical cross sections. Panek obtained data concerning critical values of tangential stress for an opening with an elliptical boundary by means of photoelasticity. Some of these data are summarized in the following table. Based on the data from the following table, and from the experimental results of Inglis and Duvall, Caudle (5) summarized these facts for an elliptical opening as follows: (1) under a hydrostatic stress field, the whole boundary of the ellipse is in compression. Critical stresses always occur on the major axis, while minimum stresses occur mainly on the minor axis, regardless of the angle of inclination of the major axis with the horizontal; (2) in the case of greater initial vertical pressure, than horizontal, critical compressive stresses occur at the sides, and critical tensile stresses occur at the top and bottom of the opening; (3) for a fixed initial horizontal stress, the critical compressive stress is the greatest when the angle of inclination of the major axis with the horizontal is zero, and is the smallest when at 90 degrees; (4) if the long dimension of the opening is nearly horizontal, the critical compressive stress increases

Fig. 5

EFFECT OF SHAPE OF OPENING ON

COMPRESSIVE STRESS CONCENTRATION

--Unidirectional Stress Field--

(Courtesy of Prof. R.D. Caudle)

After W.I. Duvall

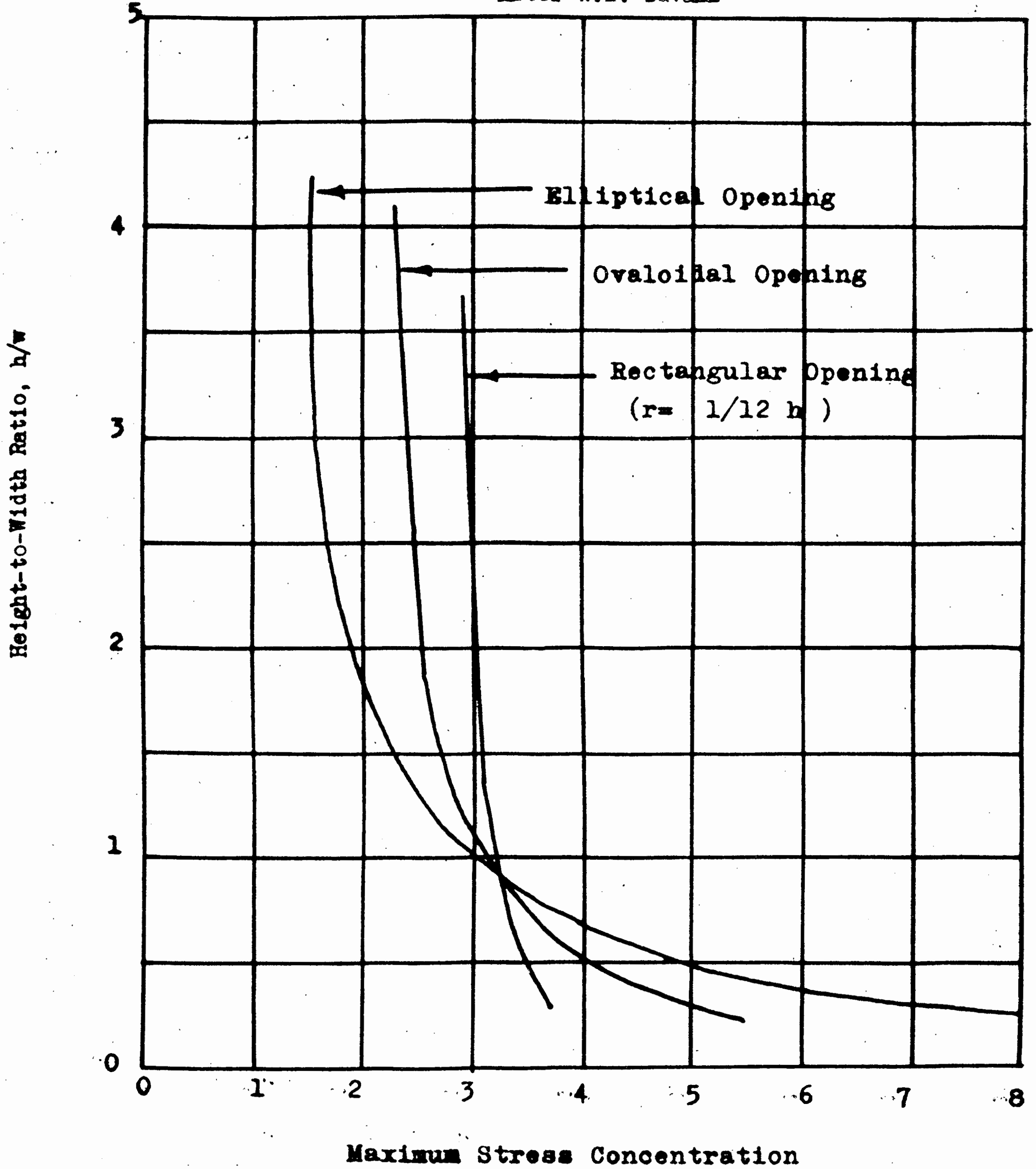


Fig. 6

EFFECT OF SHAPE OF OPENING ON

COMPRESSIVE STRESS CONCENTRATION

--Laterally Restrained--

(Courtesy of Prof. R.D. Caudle)

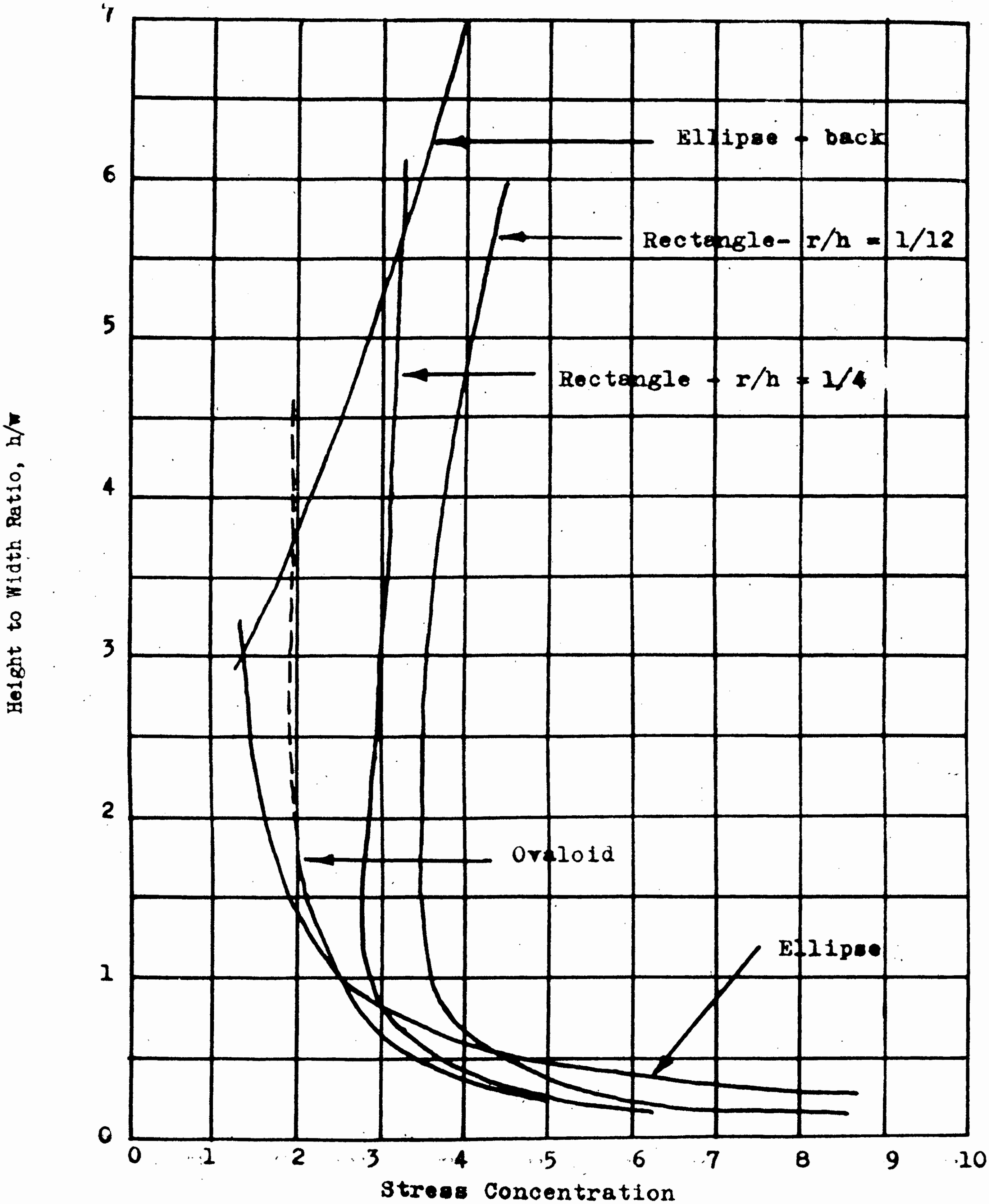


Fig. 7

EFFECT OF SHAPE OF OPENING ON

COMPRESSIVE STRESS CONCENTRATION

-- Hydrostatic Pressure --

(Courtesy of Prof. R.D. Caudle)

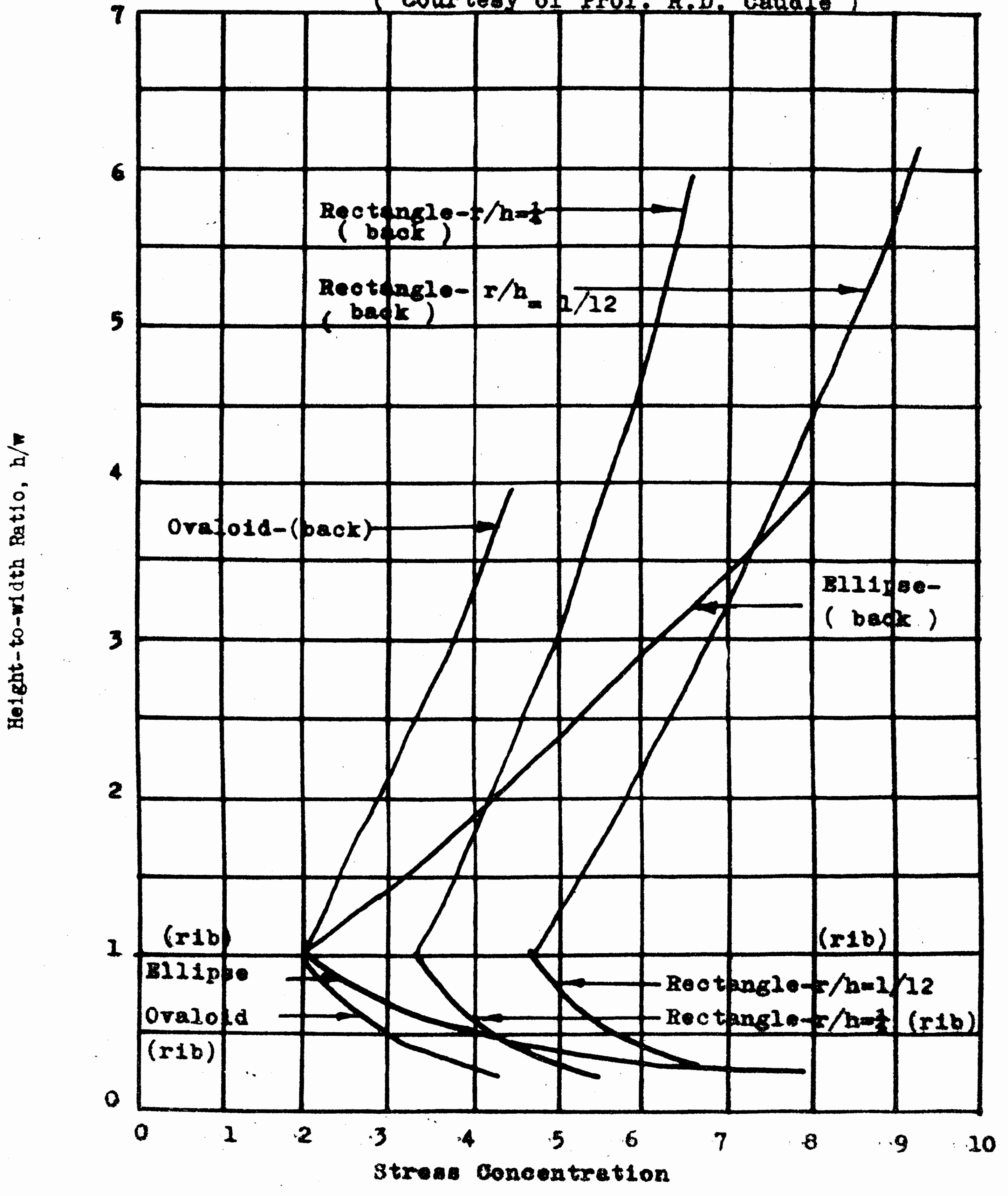
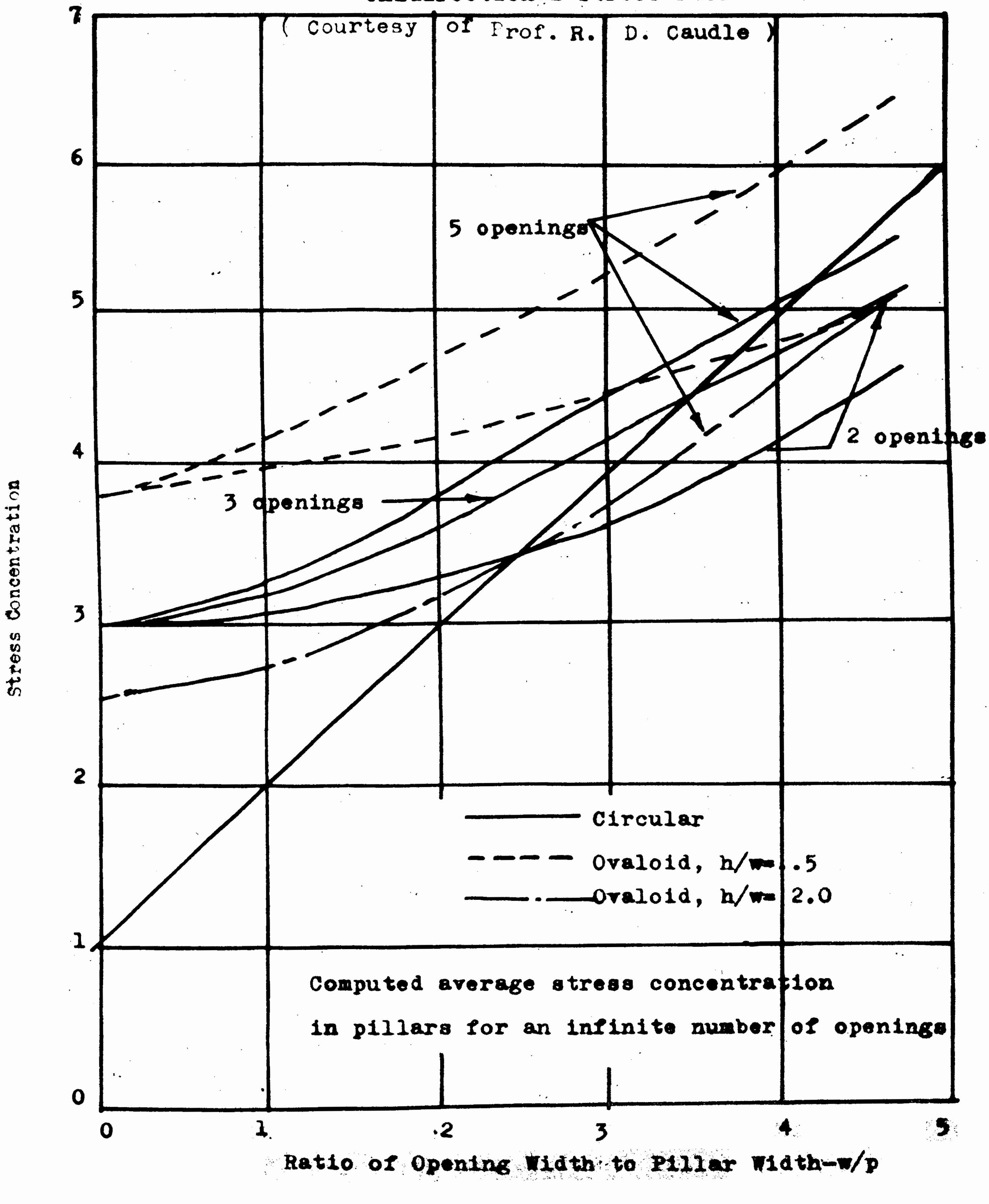


Fig. 8

STRESS CONCENTRATION VARIATION DUE TO

OPENING WIDTH TO PILLAR WIDTH RATIO

-- Unidirectional stress Field --



linearly with an increasing ratio of the major to minor axis.

Ovaloidal cross sections. An ovaloidal opening is a square with two semi-circles erected on opposite sides. Important facts concerning the stress distribution associated with an ovaloid, obtained by various investigators are as follows: (1) based on photoelastic tests, Duvall (9) stated that the maximum stress concentration at the ovaloidal boundary increases without limit as the height-to-width ratio decreases; (2) under a unidirectional loading condition, the stress on the ends of the vertical axis is a tensile stress roughly equal in magnitude to the applied stress; (3) under an initial hydrostatic condition, the stress concentrations are all compressive, in spite of the major to minor axis ratio. Caudle applied Martin Greenspan's mathematical analysis of a plate with an ovaloid-shape hole to an actual mine opening.

Stresses around various shaped multiple openings. Openings which are close enough together to have their zone of influences overlap are known as multiple openings. Distribution and magnitude of stresses for multiple openings are different from those determined for a one opening case. However, as in the case of the single opening, stresses are a function of the shapes of the openings.

A general summary of the effect of multiple openings in a homogeneous, elastic medium observed by previous authors (9) (10) (5) (23) (24) (8) as follows:

1. Stress concentrations induced by more than one opening are generally greater than those induced by a single opening of similar geometrical shape.

2. Stress concentrations increase with increasing number of openings up to a limit of about five openings. Above this point, the stress concentration will be more or less independent of the number of the openings.

3. The average stress concentration in the pillars between openings will increase rapidly (faster than the maximum stress concentration) as the opening-pillar width ratio increases.

4. Maximum stress concentration in pillars often occurs near the surface of the rock in the ribs.

5. The central pillars in a group of pillars, usually bears the highest stress.

6. If the width-height ratio is greater than one, high critical pillar stress can usually be avoided by decreasing the long cross-sectional dimension of the opening, but not by increasing the pillar width.

7. If pillar width-height ratio is less than one, increasing pillar width will avoid a high critical pillar stress.

8. As in the case of the single opening in a unidirectional stress field, tensile stress which is

equal to the magnitude of the applied external compressive stress is often induced in the top and bottom of the openings; while in a case of lateral restraint, these tensile stresses disappear.

(Figure (8) summarizes the effects of multiple openings of various shapes obtained by previous investigators.

<u>Theories Concerned with Design of Roof-span and Pillars Horizontal Mine Openings.</u>

The maximum safe roof span, size of pillars, and optimum extraction ratio are important factors in the design of room and pillar type underground openings. In theoretical and experimental investigations for the distribution and magnitude of the stress concentrations around the openings, the geologic structure, and the physical properties of the rock body itself play an important part in determining the previously mentioned design factors.

In the usual calculations for roof-span, assumptions are normally based on one of two theories. They are the plate theory and the beam theory as follows: (19) (36) (34).

<u>Plate theory.</u> This theory is based on the assumption that mine roof can be approximated by a uniform, horizontal, self-gravitationally-loaded slab which is clamped on all edges. It is assumed that the length and width of the slab are approximately equal.

The maximum tensile stress (σmax) in this slab is (19,p.16):

$$\sigma max = \frac{6B\rho a^2}{t} \text{ - - - - - - - } (1)$$

The maximum sag (Smax) in this slab is:

$$Smax = \frac{C\rho a^4}{Et^2} \text{ - - - - - - - } (2)$$

Where a = shorter horizontal dimension of the plate
b = longer horizontal dimension of the plate
B = constant depending on Poisson's ratio and b/a
C = constant depending on Poisson's ratio and b/a
E = Young's modulus of Elasticity
ρ = density of the rock
t = thickness of the slab

Beam theory. This theory is based on the assumption that mine roof may be approximated by a homogeneous, horizontal lying, self-gravitationally-loaded beam with restrained ends. The length of the room must be more than two times its width and the thickness of the beam must be small in comparison to its length for application of the equations developed. The maximum tensile stress (σmax) in the beam is: (19, p. 13)

$$\sigma max = \frac{\rho L^2}{2t} \text{ - - - - - - - - } (3)$$

The maximum shear stress (τmax) in the beam is:

$$\tau max = \frac{3\rho L}{4} \text{ - - - - - - - } (4)$$

The maximum deflection (Dmax) of the beam is:

$$Dmax = \frac{\rho L^4}{32Et^2} \quad \text{-------- (5)}$$

Where E = Young's modulus of elasticity
L = length of span
ρ = density of the beam
t = thickness of the beam

<u>Calculation of safe roof-span.</u> A safe roof span may be determined by application of the above mentioned theories. Obert and Duvall presented a method of calculation (19). From equation (1):

$$\sigma_{max} = \frac{6B\rho a^2}{t} \quad \text{------ (3)}$$

hence

$$A_s = \sqrt{\frac{T_a t}{6\rho BF}} \quad \text{------ (6)}$$

where T_a = the modulus of rupture of the roof rock
F = safety factor
t = thickness of the plate
ρ = density of the rock
B = constant depending on the ratio of b/a and Poisson's ratio.
A_s = the safe length of t he shorter hor. dim.

Merrill (19) performed a calculation, which was based on the beam theory, to determine the safe roof-span as follows:

From equation (3)

$$\sigma_{max} = \frac{\rho L^2}{2t} \text{ - - - - - - - - } (3)$$

the value of the modulus of rupture of the roof rock T_a can be substituted for the tensile stress max, hence the safe roof-span L_s can be obtained: (19, p. 17)

$$L_s = \sqrt{\frac{2T_a t_{max}}{\rho F}} \text{ - - - - - - - - } (7)$$

where T_a = the modulus of rupture of the roof
t = thickness of the beam
ρ = density of the rock
F = safety factor

These methods result in theoretical determinations of the safe roof-span based on the requirements of absence of major geologic defects and homogeneous, horizontal strata.

Pillar loads. According to Duvall, a simple equation for the relationship between pillar width, opening width, average stress and loads supported by the pillar is as follows: (5) p. 28)

$$P_p = \sigma_y t \quad (L_o + L_p)$$

σ_y = initial vertical stress field at a distance from row of holes

L_o = width of opening

L_p = width of pillar

t = thickness of plate

Or

$$\sigma_p = \frac{\sigma_y \, (L_o + L_p)}{L_p}$$

The above equation is based on the assumption that the longitudinal axes of the openings are large compared to their cross sectional dimensions, and that any material mined by cross cuts is negligible.

In the case of a room and pillar method, where pillars are of equal dimensional cross-section, the average load supported by the pillar is given by the equation

$$P_p = \sigma_y \, (L_o + L_p)^2$$

After the consideration of the factor of safety, the average stress in the pillar becomes:

$$\sigma_p = \frac{\sigma_{max}}{F} = \frac{\sigma_y \, (L_o + L_p)^2}{L_p^2}$$

Where σ_{max} = ultimate compressive strength

σ_p = average stress in pillar.

Theories of Failure Applied to Rock (16) (57) (60) (58)

In the preceding section on physical properties of rock, a number of useful physical properties were described. The majority of these properties were derived from simple uniaxial tests. Most of the loading conditions underground are tri-axial. It is, therefore, necessary to predict the failure of rocks under these conditions from the laboratory physical property data. For this reason some of the more widely accepted theories of failure are considered here.

Solid materials are classified by their behavior when subjected to static or impact loads as elastic, viscous or plastic. The point of failure of most structural materials is generally considered to be the condition under which they first lose their elastic properties. The failure of a load-resisting member usually consists of either plastic deformation or brittle fracture. There have been six major theories advanced to predict the point of inelastic action. In each theory, the factor held responsible for limiting the maximum applied load, is assumed equal in the magnitude to that same factor determined in a simple uniaxial test.

<u>Maximum Stress Theory</u> (Rankin's theory)

The maximum principal stress in a material determines the point of failure regardless of the magnitude of the other principal stresses applied. This theory has received some acceptance as an explanation of the behavior of brittle materials, and, therefore, has widespread application in mining. A special note must be added that this theory does not apply in the instance where mine rock failure is plastic, a case which will be considered in more detail later.

<u>Maximum</u> <u>Strain</u> <u>Theory</u> (St. Venant's theory)

The maximum normal elastic extension of which the material is capable determines the point of failure. One may express the principal strains in terms of the principal stresses according to Hook's law. This is a widely accepted theory which is applied to the brittle failure of materials, such as rock.

<u>Maximum</u> <u>Shear</u> <u>Theory</u> (Coulomb's theory)

The maximum shear stress theory is based on the premise that the point of inelastic action of a material occurs when the maximum shearing stress at a point within the body exceeds a critical value. This theory is applied frequently to explain the behavior of ductile materials.

<u>Maximum</u> <u>Strain</u> <u>Energy</u> <u>Theory</u> (Beltrami & Haigh theory)

This theory is based on the assumption that there is a limitation to the amount of energy that a body can absorb and remain in the elastic state. This limit determines the points of failure. In reference to this theory, Nadai states "the total elastic energy stored in a material before it reaches the plastic state can have no significance as a limiting condition . . . . since under high hydrostatic pressure large amounts of elastic energy may be stored without causing either fracture or permanent deformation.

Maximum Distortion Energy Theory

This theory, as defined by some current authors, states that inelastic action at any point in a body under any combination of stresses begins only when the strain energy of distortion per unit volume absorbed at the point is equal to the strain energy of distortion absorbed per unit volume at any point in a bar stressed to the elastic limit under a state of uniaxial stress.

Mohr's Envelope (Theory of Strength)

If a number of compressive tests are performed, each with a different degree of confinement, the data at the point of failure may plotted as a series of Mohr's circles.

A point on the envelope to these circles represents the magnitude of normal and shear stresses on the plane of failure in the material, for a particular ultimate loading. Mohr's envelope may be analysed in several ways. The shearing strength of a material may be shown equal to

$$\tau = C + \sigma \tan\phi$$

where C = shearing strength of the material with zero normal stress

σ = normal stress on the failure plane

ϕ = the angle of internal friction

The maximum compressive strength of a material using this method is found to be (p17)

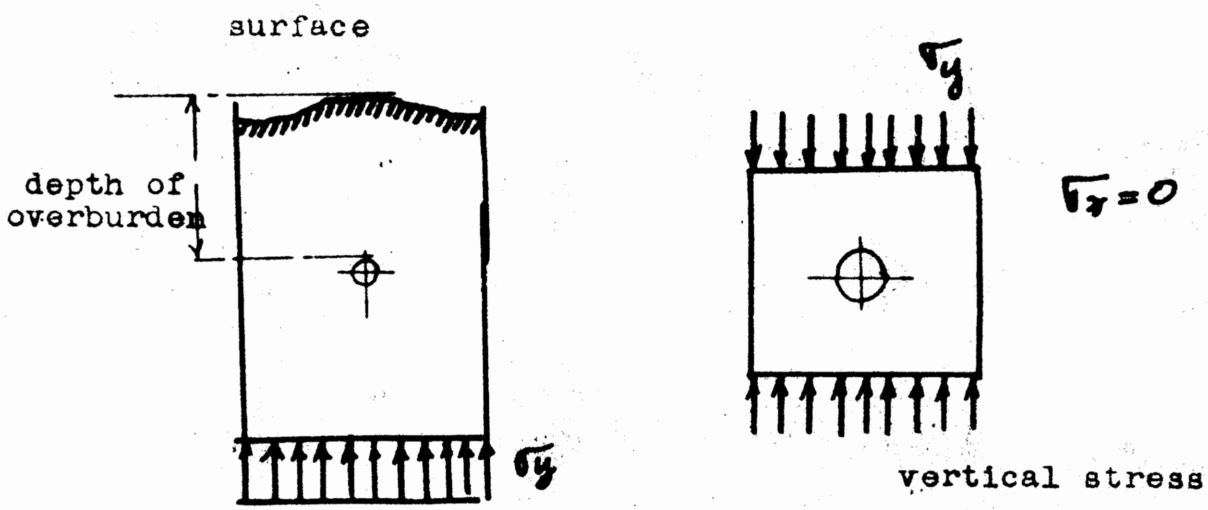
$$\sigma 1 = \sigma 2\ (\tan^2(45^\circ + \phi/2) + 2c \tan\ (45^\circ + \phi/2)$$

where σ_1 = maximum compressive strength

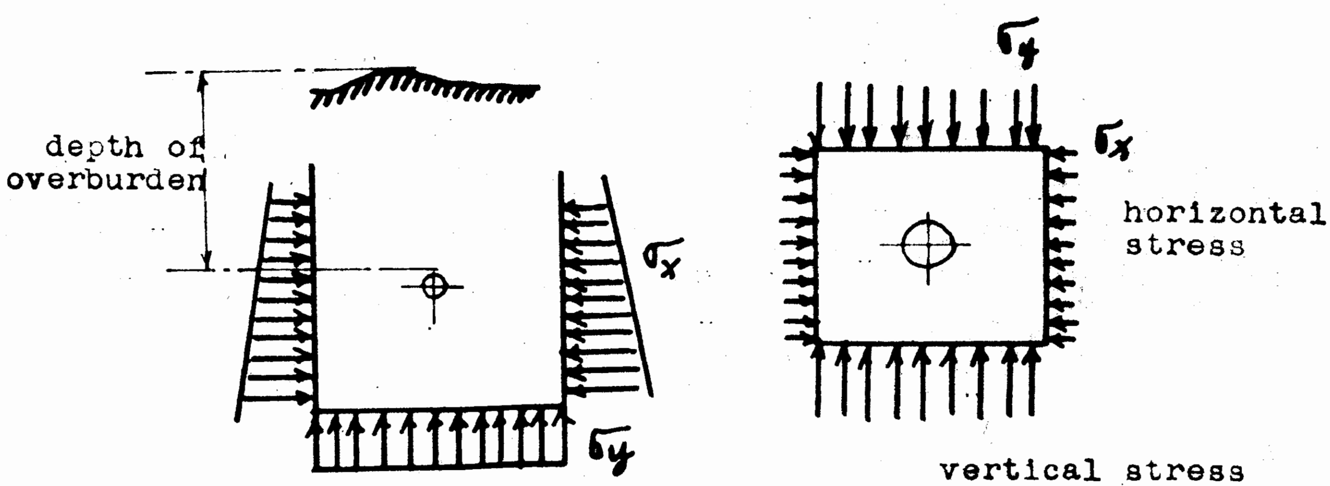
σ_2 = confining pressure

Examples of Mohr's envelopes for a selected group of rocks are given in (Figure 26).

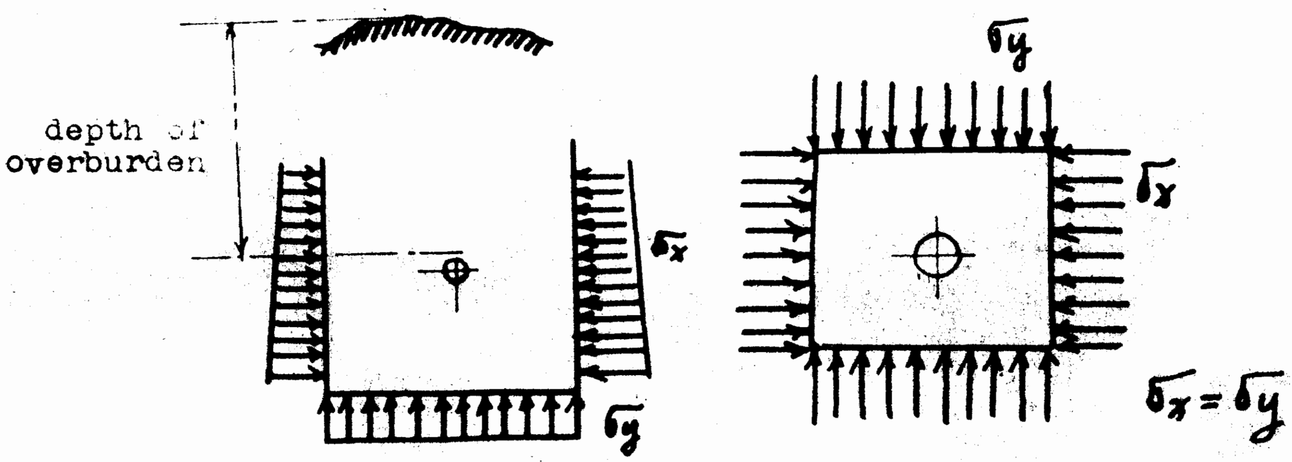
The use of Mohr's envelope to predict rock failure under triaxial loading is the most widely accepted method at the present time. (17) (58).



(a) Case 1 : No lateral restraint



(b) Case 2 : Laterally restrained



(c) Case 3 : Hydrostatic pressure

Fig. 9. Three Assumed Cases of Pre-existing Stress in the Earth

CHAPTER III

SYSTEMATIC PROCEDURE FOR THE PRELIMINARY DETERMINATION OF A MINE STRUCTURE

A systematic procedure for the preliminary determination of an underground mine structure is outlined in the following pages. This portion of the paper has been so arranged that a person interested in utilizing this material for design purposes need not refer to other chapters. Material has been arranged in the sequence which a logical design procedure must follow. The chapter has been subdivided into five sections; namely, estimation of the pre-existing stress field; choice and determination of pertinent physical properties of rock; determination of the behavior of the mine roof strata by use of the centrifuge; calculation of pillar loads; and the application of data obtained in the design of a mine opening.

Estimation of Pre-existing Stress Field

The determination of the pre-existing stress field is necessary in the design of underground mine openings, as well as for any underground excavations such as tunnel driving, just as the determination of loading conditions is necessary for the design of surface structures.

A preliminary estimation of the pre-existing underground stress field should be performed before the initiation of mining operations. For this purpose, the simplification of earth stress fields into three categories as suggested by Panek, is sufficiently accurate for most cases.

On a horizontal plane below the ground surface it may be assumed there exists only a uniform vertical pressure (σy) acting normal to the plane, which is equal to the weight of the overlying materials; that is, the product of the depth (d) and density (w) of the overlying materials.

$$\sigma y = -wd$$

In combination with the stress acting on the horizontal plane, one of three possible states of stress may be assumed to exist on the vertical plane: (5) (24).

<u>Unidirectional</u> <u>case</u>. At a shallow depth, no (σx) pressure acts normal to the vertical plane. This condition may exist at any depth where open fracture systems are found.

$$\sigma x = o$$
$$\sigma y = wd$$

<u>Lateral</u> <u>restraint</u> <u>case</u>. This condition is assumed to exist at intermediate depths where no horizontal strain can occur in rock since confinement exists on all sides.

$$\sigma x = \frac{\mu}{1-\mu} \quad y$$

$$\sigma y = -wd$$

(μ is Poisson's ratio)

The presence of ground water in open fracture systems can result in horizontal pressures approximately equal to those given for the laterally restrained case.

Hydrostatic case. At extreme depths, a hydrostatic condition is assumed to exist, normal stresses in all directions being equal.

$$\sigma_y = -wd$$
$$\sigma_x = -wd$$
$$\sigma_y = \sigma_x$$

The three initial stress field conditions outlined above, can not be specifically positioned according to depth. A rough range may be taken as unidirectional 0-1,000 feet; lateral restraint 200-10,000 feet; and hydrostatic pressure 5,000 ft. and deeper. The preliminary estimates should be conservatively applied where there is an overlap in the possible stress field at that depth. (See Figure 9.)

The approximate horizontal stress field given by the equations above, may be greater than that actually in existence in cases where fractures, faults and other geological discontinuities or topographic depressions occur. On the other hand, the actual horizontal stress may be greater than that estimated, in cases where the mine opening is located in the neighborhood of higher topography or near a regional high stress zone.

The choice of the preliminary stress field should be verified as soon as possible after mining operations have commenced. The only means of determining the initial stress field underground is by measuring the response of a particular mine opening, and then deducing the stress field.

At the present time, there is only one method of measuring the response of the rock surrounding a mine opening to an external stress field which has any evidence of success. This is the flat-jack method, (50) first described by M. E. Tincelin in 1953. In this method, a slot is cut into the wall of a mine opening of known shape (Figure 10a), which results in expansion of adjoining rock into the slot due to the surrounding stresses. Pressure is applied to the walls of the slot by means of a special hydraulic jack inserted in the slot, to restore it to its original shape. The indicated jack pressure is assumed equal to the stress acting normal to the slot, before the slot was cut. The initial stress field at that point is then calculated by dividing the stress determined by means of the flat jack by the stress concentration factor associated with the particular shape of opening in which the test was performed.

The determination of the pre-existing stress field has been previously considered by a number of investigators. (5) (24) (50). The United States Bureau of Reclamation (47) (49) performed some tests of this nature in a tunnel at Boulder Canyon in 1932, and also in tunnels at Prospect Mountain, Colorado and at Gorge, Washington in 1957. The Australian underground hydro-electric power plant in the Snowy Mountains, (50) Australia, is a good example of successful measurement of the initial earth stress field.

Choice and Determination of Pertinent Physical Properties of Rock

Of all the physical properties of rock; elastic properties, specific gravity, and rock strengths have the closest connection to the behavior of underground mine openings.

The magnitude of pre-existing stresses depend upon Poisson's ratio and specific gravity of the rock. The spacing and sizing of mine pillars usually depend upon compressive strength and Poisson's ratio. In many cases, the design of the safe length of span of the roof may be restricted by a limitation in tensile strength or modulus of rupture of the rock. The deformation of rocks is commonly predicted on the basis of their elastic properties. Therefore, ultimate compressive strength, ultimate tensile strength, modulus of elasticity, Poisson's ratio, and modulus of rupture, are the pertinent physical properties of rock from the standpoint of design of stable underground mine openings.

A brief description of the above mentioned physical properties of rock and the standardized methods for their determination is given here.

Standardized Procedures

The first step of each of these physical property tests is the preparation of specimens. Rock specimens are usually prepared in either a cubic or cylindrical shape with different dimension ratios, depending on the kind of tests. Equi-dimensional cubes have been used by the U. S. Bureau of Mines as standardized specimens. Cylindrical specimens obtained from drill cores are somewhat more practical. For this reason, all of the standardized tests described herein are based on test specimens cut from cylindrical drill cores.

The ultimate compressive strength. The average compressive stress at which a specimen under a uni-axial compressive load fails is known as the ultimate compressive strength.

$$\sigma_c = \frac{P_c}{A}$$

where σ_c is the ultimate compressive stress

P_c is the ultimate compressive load

A is the original cross-sectional area

Specimens for compressive strength tests should be cut from diamond drill core with a length-to-diameter ratio of approximately 1.8. This ratio is sufficiently large to minimize end effects, while at the same time being sufficiently short to avoid column effects. The ends of the specimens should be ground smooth and parallel.

After preparation of the specimens, measurement of their dimensions should be obtained to an accuracy of $\pm$ 0.001 in. The specimen is then placed under the center of the compressive loading head of a standard universal testing machine. The loading head must have a platen mounted in a ball and socket to permit equalization of the compressive load.

An attempt should be made to maintain a uniform rate of load. A commonly accepted figure is 100 lb./in ^{2}/sec. The specimen should be loaded continuously until failure occurs. The measurement of strain simultaneously with loads can provide data for the determination of the modulus of elasticity. (44) (45) (53) (64).

<u>The ultimate tensile strength</u>. The ultimate tensile strength is the maximum average tensile stress on the plane of failure of the specimen, where other principal stresses on the plane are negligible.

According to the nature of rock, the tensile strength is a fraction of the compressive strength of the same rock (one-tenth to one-fortieth). A large portion of the falls of mine roof are probably caused by locally exceeding the tensile strength of the roof. The determination of tensile strength accurately is important for fundamental work in the design of underground mine openings.

There are two primary methods of determining tensile strength: the direct and indirect methods. The direct tensile test is the classical test in which a rock specimen is gripped at opposite ends in a universal testing machine and subjected to a tensile load. In this type of test difficulties due to clamping stresses and induced bending are encountered. The indirect tension test has been chosen as the standardized test method since (1) no special loading heads are required, (2) the indirect tensile test is relatively simple, and (3) the test results in slightly greater tensile strengths than the direct test which, is believed, are a result of a method of testing which has small inherent error. The specimen is prepared from a cylindrical drill core section as in the case of the compressive strength specimens. The length of a specimen is approximately equal to its diameter. Special preparation of the ends of the specimen is not necessary.

The specimen is placed between the compressive platens of the stationary and the loading heads of a universal testing machine with its axis of symmetry horizontal. The common rate of load is 50 lb./sec. The specimen should be loaded until failure occurs. A single fracture plane should occur parallel to the direction of applied loading on a diametral plane. A uniform tensile stress is developed perpendicular to this plane. In addition, a compressive stress is developed parallel to the plane. However, the tensile strength of rock is so small compared to its compressive strength that the effect of the compressive stress may neglected.

The application of a uniform loading along the specimen length is important, otherwise the specimen will be subject to strong shear stresses due to variation in the normal loading. Fractures other than the one described above indicate the occurrences of an incorrect loading condition, or an anomalous weakness within the material. (44) (45).

The tensile stress developed by the indirect method can be calculated as follows:

$$\sigma_t = \frac{2 P_c}{\pi DL}$$

Where σ_t is the ultimate tensile stress

P_c is the ultimate compressive load

D is the diameter of the specimen

L is the length of the specimen

<u>Modulus of elasticity</u>. The modulus of elasticity is the ratio of stress and its corresponding strain within the elasticity limit under a uniaxial compressive or tensile loading.

The modulus of elasticity in compression is not necessarily equal to the tensile modulus. In general, the compressive modulus has wider usage in rock mechanics. The compressive modulus of elasticity may be determined from data obtained while performing an ultimate compressive strength test.

The basis for any determination of Young's Modulus of elasticity is the preparation of a curve of stress versus strain. It is therefore necessary to measure deformation of the specimen at regular loading intervals.

The measurement of compressive strain can be best performed by the use of strain gages bonded to the specimen or indirectly by the use of a compressometer attached to the specimen.

Stress-strain curves obtained from experimental data are not always linear over the entire test range, due to the anelastic behavior of rock. The tangent modulus of elasticity is the most satisfactory modulus for correlation purposes. A tangent line should be drawn to the steepest part of the curve, at a point in the region between one-third and one-half of the ultimate strength. This results in a modulus of elasticity at the working strength of the material.

Poisson's ratio. When a material is subjected to a longitudinal tensile stress, it will elongate in a direction parallel,and decrease in cross-section perpendicular to the load. When a material is subjected to a longitudinal compressive stress, longitudinal shortening and accompanying increase in perpendicular cross-section occurs.

The ratio of the strain in a lateral direction to the longitudinal strain under the condition of uniaxial stress within the proportional limit is known as Poisson's ratio.

A recommended method of obtaining Poisson' ratio is to attach four resistance strain gages to the specimen, two parallel and two perpendicular to the direction of the long axis of the cylindrical specimen. Lateral and longitudinal strains can be obtained independently by means of a strain indicator connected to these gages, at designated intervals of compressive load. The dimensions of the specimen should be the same as for the standard compression test.

Poisson's Ratio can be expressed as:

$$\mu = \frac{\epsilon_t}{\epsilon_e}$$

Where μ is Poisson's Ratio
ϵ_t is longitudinal strain
ϵ_e is lateral strain

This can also be determined graphically from the slope of the curves of lateral strain versus longitudinal strain.

Modulus of rupture. The modulus of rupture is defined as the equivalent elastic stress in the outermost fibers of a beam at the point of failure under a bending moment.

The modulus of rupture test results in higher strength magnitudes than the ultimate tensile strength test, since the neutral axis in the rock beam tends to shift toward the beam lower extremity and the stresses in the upper and lower surfaces of the beam do not stay within the elastic limit.

The modulus of rupture can be determined from the following equation:

$$\sigma_{mr} = \frac{Mfc}{I}$$

Where σ_{mr} is the modulus of rupture
Mf is the bending moment of failure
c is the one half of the thickness of the beam
I is the moment of inertia of the cross-section with respect to a diameter perpendicular to the load.

The experimental procedure suggested here has been adapted from that utilized by the U. S. Bureau of Mines as follows:

A cylindrical specimen 6-inches in length and $1\frac{1}{4}$ inches in diameter is placed on knife edges 5 inches apart in a specially designed fixture, as shown in Figure (6). A compressive load is applied at the center of the specimen by means of a single knife edge. The compressive load should be applied at a rate of 20 lb./sec., until failure occurs.

The modulus of rupture can be calculated from the data obtained from this test as follows:

$$\sigma_{mr} = \frac{8WL}{\pi D^3}$$

Where σ_{mr} is the modulus of rupture

W is the applied load (ultimate)
L is the distance between supports (5 in.)
D is the diameter of the specimen.

Specific gravity. The specific gravity of rock can be obtained by various methods. If the volume of the specimen is known in metric units (easily determined on specimens prepared for elastic properties tests), the quickest and simplest method to be employed is as follows:

$$\rho = Wt/V$$

Where ρ is the specific gravity of rock
Wt is the weight of the specimen
V is the volume of the specimen.

Standardized Method of Determination of the Effect of Mine Openings (25)

The determination of the effect of a mine opening upon the surrounding rock generally resolves into two phases: (1) effect of the mine opening on roof behavior, and (2) effect on pillar stresses.

The Use of the Centrifuge in Determining Roof Span Behavior

The principles of centrifugal model testing have already been mentioned in the literature review in the previous section. However, an attempt to summarize the test procedure and the equipment utilized is presented briefly, hereafter.

The description of equipment included here is based on that available at the U. S. Bureau of Mines and that recently assembled at Department of Mining Engineering of Missouri School of Mines and Metallurgy.

Specimens for this test are cut from rocks in place. These specimens may represent a section of tunnel roof or a complete set of mine openings. Centrifugal forces are applied to these specimens by means of the centrifuge to simulate underground loadings.

Destructive and non-destructive tests are based on the same principle but differ slightly in test procedure in that the former does not include any means of strain measurement.

<u>Centrifugal model test equipment</u>. The centrifugal testing machine is composed of a rotor, model holder, counterweight, driving motor, and housing tank.

The rotor consists of a box-like structure which is mounted on a vertical shaft. The effective radius of rotation of the model holder has no fixed limits, and varies according to the requirements of the particular investigator. An opening through the rotor permits observation of the model. Since the maximum usable radius of rotation is fixed when the machine is built, the scale ratio is varied by controlling the speed of rotation.

The purpose of the model holder is to hold the model in the rotor while approximating the constraints placed on the prototype in nature. The holder is a rectangular frame, whose size may be varied to fit different models, within the limitations of available rotor space.

The counterweight may consist of a variable number of interchangeable metal plates of different weights, placed on the opposite side of the rotor to the model to provide a dynamic balance.

A D. C. motor is normally used as the driving motor, since a D. C. motor can be uniformly controlled over a large range in speed. A resistance box or some type of electronic control is used to vary the rotating speed.

A steel tank surrounds the rotor and carries the bearing housings within which the rotor turns. Plate-glass observer ports are necessary to permit visual access to the tank. In order to minimize the driving power required, the centrifuge is operated in a vacuum.

Accessory equipment to the centrifuge consists of a vacuum pump, balance, tachometer, stroboscope, and strain indicator.

A vacuum pump is utilized to minimize the rotor driving power by eliminating some of the air resistance to the rotating members.

A balance with lever arms equal to those of the rotor is useful for obtaining an approximate dynamic balance of the counterweight to the model, before they are placed within the tank.

A tachometer is necessary to determine the speed of the rotation of the shaft, since centrifugal forces are proportional to radial velocity squared.

A stroboscopic light, triggered by a contact on the upper end of the shaft and placed so that it illuminates the model once per revolution, permits the operator to view or photograph the model.

If strains in the model are to be measured, a strain indicator should be connected directly. Because of the rotation, a set of slip rings must be installed to connect the gages and the strain indicator. (25).

<u>Centrifugal model test procedure</u>. As the first step of the test, specimens must be prepared. There are many possible kinds of specimens, such as rock beams of various length and thickness. Since the dimensions of this kind of specimen are greater than the common specimen used in the physical property tests mentioned previously, a large rock saw is necessary.

The unit weight of the rock material being utilized should be determined at this point. In a non-destructive test, strain gages should be cemented to the specimens at the points where large strains are expected. The model should be mounted in a holder with the proper clamping forces to simulate the constraints applied to the prototype underground.

The counterweight assembly is adjusted in the balance to obtain a static moment equal to that of the holder with specimen. This is usually sufficiently accurate to provide a dynamic balance over the test speed range. Model and counterweight are placed at opposite ends of the centrifuge rotor. The vacuum pump is started to evacuate the air after the housing tank has been sealed. When the vacuum reaches the required minimum, the drive motor is started to rotate the centrifuge, and bring it up to speed slowly to avoid acceleration forces due to changes in speed. The operator may observe the image of the rotating model by means of the stroboscopic light and the plate glass view ports until failure of the model occurs. The speed and corresponding strains are recorded if the non-destructive method is employed. (25).

The model ratio (ratio of prototype dimension to model dimension) can be calculated from the speed at failure of specimen, obtained through experiments. The model ratio times the dimensions of model determines the dimensions of a prototype--thickness and length, which would fail under its own weight underground. A curve of prototype beams, whose thickness versus length is just sufficient to stand under their own weights, can be constructed from the results of a number of model tests with various thickness to length ratios. From this curve, knowing either the required thickness or length, the remaining dimension may be found. (25) (2).

The outer fiber stress at the supports (pillar in prototype) for a rectangular, restrained, beam loaded by gravity is theoretically (25, p. 17)

$$\sigma_{max} = \frac{WL^2_{max}}{2t_{max}}$$

Where W = weight per unit volume
tmax = thickness of beam at failure
Lmax = span of beam at failure

If equidimensional pillars are left in the mining process, the rock beam model most accurately depicts the bending effects between corners of diagonally opposite pillars, and therefore, the span of the opening at failure is equal to $L_{max}/\sqrt{2}$. Since the bending stress is proportional to L^2, the inclusion of a factor of Safety, F, and the reduction in span to account for the diagonal measurement, results in the following expression for safe span.

$$L_s = \sqrt{\frac{1}{2F}}\ L_{max}$$

Similarly, a safe thickness may be chosen, when L_s is chosen equal to L_{max} from the expression

$$t_s = 2Ft_{max}$$

Where t_s is the safe thickness

There is no data available for the selection of a factor of safety for mine roof. However, in view of the fact that most mines are not designed as permanent structures, a factor of safety of from 2 to 4 should be adequate.

The Determination of Pillar Stresses and Pillar Sizes

Preliminary pillar size can best be determined by theoretical calculation. Confirmation by model tests may prove practical in the future, although centrifugal model tests attempts to date have not been successful.

There are three possible approaches to the problem of multiple pillar size determination, assuming that pillar spacing has been determined previously (1) average pillar stress method, (2) theoretical maximum pillar stress method, and (3) a method based on the assumption that all underground openings act as ellipses regardless of their shape.

The average pillar stress method may be applied in those cases where the width of opening to pillar ratio is large (high percent recovery) or where the material surrounding an opening is fractured or deforms plastically, preventing the development of large local stress concentrations.

The theoretical maximum pillar stress is probably never achieved in nature. However, some materials exist in nature, close to the ideal homogeneous, elastic mass required for the application of this method. The resulting stress concentration factors serve as an upper limit of the possible stress factor span.

A discussion of the effect of arching upon the shape of a mine opening has been previously presented in Chapter II. Of particular interest is the concept that all underground openings tend to act as ellipses, whose height-to-width ratio depends upon the initial earth stresses. This method can only be applied in cases where sufficient fracturing and plasticity exists to permit the effective shape of the opening to change.

<u>Average pillar stress</u>. A simple equation for the stress in multiple pillars suggested by Duvall (9) can be applied if the long dimension of the opening is greater (more than 4 times) than the dimension of the cross section of the mine opening. This is:

$$\sigma_{av} = \sigma_y \left(\frac{L_o + L_p}{L_p}\right) = \sigma_y \left(1 + \frac{L_o}{L_p}\right)$$

Where σ_{av} = average stress developed in a pillar

σ_y = initial vertical stress

L_o = width of opening (span)

L_p = width of pillar (pillar size)

As applied to equidimensional room and pillar mining operation, the thickness t is equal to the sum of the width of an opening and width of a pillar, (L_o + L_p). The above equation can be rewritten as follows:

$$\sigma_{av} = \sigma_y \; (\frac{L_o + L_p}{L_p^2})^2$$

Where σ_{av} = av. stress on one pillar

After the factor of safety has been considered, the equation may be rewritten as follows:

$$\sigma_{av} = \frac{\sigma_{ult}}{F} = (\frac{L_o + L_p}{L_p^2})^2$$

Where σ_{ult} = ultimate compressive strength
F = factor of safety

Theoretical maximum pillar stress. The theoretical maximum pillar stress can be determined from the curves in Figure 8, for a limited number of opening shapes in an unidirectional stress field. To permit analysis of a large number of opening shapes, it has been assumed that the peak stress within a pillar is approximately equal to the average stress when the ratio of opening width to pillar width is greater than 5, and that the stress concentration factor varies parabolically with a change in opening-to-pillar-width ratio. These assumptions result in the equation

$$B = \alpha + K \, (\frac{L_o}{L_p})^2$$

Which may be applied in the case of openings whose lengths are much greater than their cross-sections.

And from Figure 8

$$K = \frac{6-\alpha}{25}$$

Where B is the multiple opening stress concentration factor

α is the single opening stress concentration factor

Or

$$B = \alpha + \frac{6-\alpha}{25}\left(\frac{L_o}{L_p}\right)^2$$

Thus it follows that

$$\sigma_{max} = \frac{\sigma_{ult}}{F_1} = \sigma_y B = \sigma_y \left[\alpha + \frac{6-\alpha}{25}\left(\frac{L_o}{L_p}\right)^2\right]$$

Where F_1 is the safety factor used to determine allowable maximum stress.

The safety factor (F_1) may be lower than that utilized with the average pillar stress determination since there is much less likelihood that the maximum predicted stresses will be exceeded.

<u>Pillar stress using the equivalent ellipse</u>. According to the hypothesis of arching, all openings act as ellipses in respect to the stresses developed within their zones of influence. The ratio of the vertical to the horizontal axis of the equivalent ellipse is assumed equal to the ratio of vertical to horizontal cross fields. The dimensions of the ellipse should be determined so that it is circumscribed around the original opening.

The equations developed above for openings of any shape may be used to determine the stresses about the equivalent ellipse. (12) (32).

Application of Data Obtained to the Design Of a Mine Opening

The procedure outlined in the preceding sections may be summarized as follows:

(1) A preliminary estimation of the pre-existing stress field may normally be made by classifying the situation as a case of no lateral restraint, lateral restraint, or hydrostatic pressure according to depth and geologic features. Two physical properties of the mine rock and overburden are necessary for the estimation, Poisson's ratio of the mine rock and the specific gravity of the overburden.

(2) Pertinent physical properties of the rock in place can be approximated by tests on sections of diamond drill-cores.

(3) The maximum roof span may be approximated by testing a suitable beam model in a centrifuge. The product of the effective support distance of the model beam, the scale factor obtained from centrifugal testing data and a factor of safety should provide a safe roof span. The factor of safety should vary according to the knowledge available concerning faulting, variation in bedding, etc.

(4) The determination of pillar stresses as well as pillar sizes can be obtained through theoretical calculation. Three methods have been outlined, average pillar stress, peak pillar stress and the equivalent ellipse method. The only method which can be applied to equi-dimensional pillar calculations, at the present time is the average stress method, since stress concentration factors for this type of pillar are not presently available. All three methods presented may be applied to openings whose length is several times greater than their cross-section, the choice of method being determined by the nature of the rock, i.e., fractures, plasticity, etc. Correct procedure would include a pillar calculation by each of the applicable methods, and then weighing of the results, according to the specific physical properties of the geologic material involved.

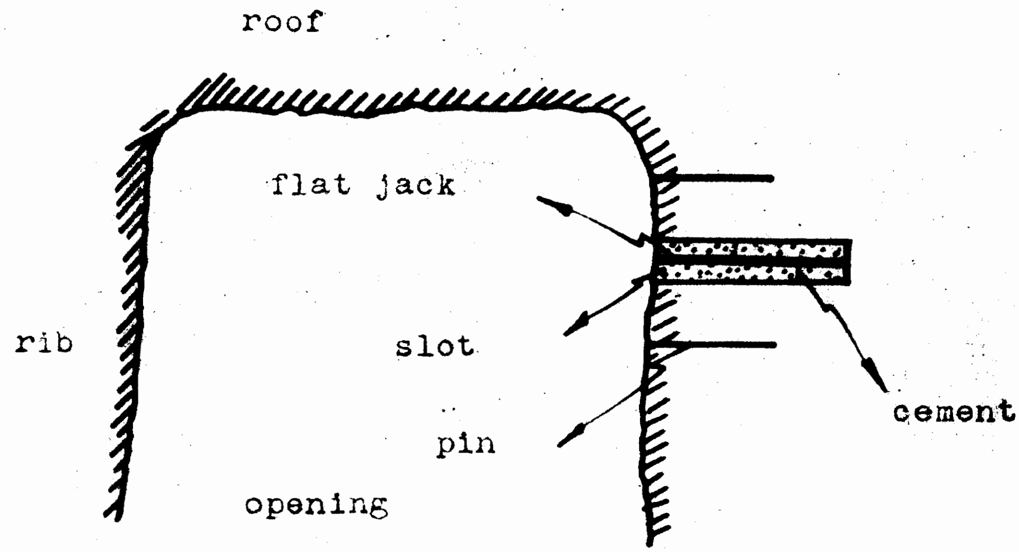


Fig. 10a. A Simplified Sketch Showing the Set up of a Flat Jack in an Underground Opening.

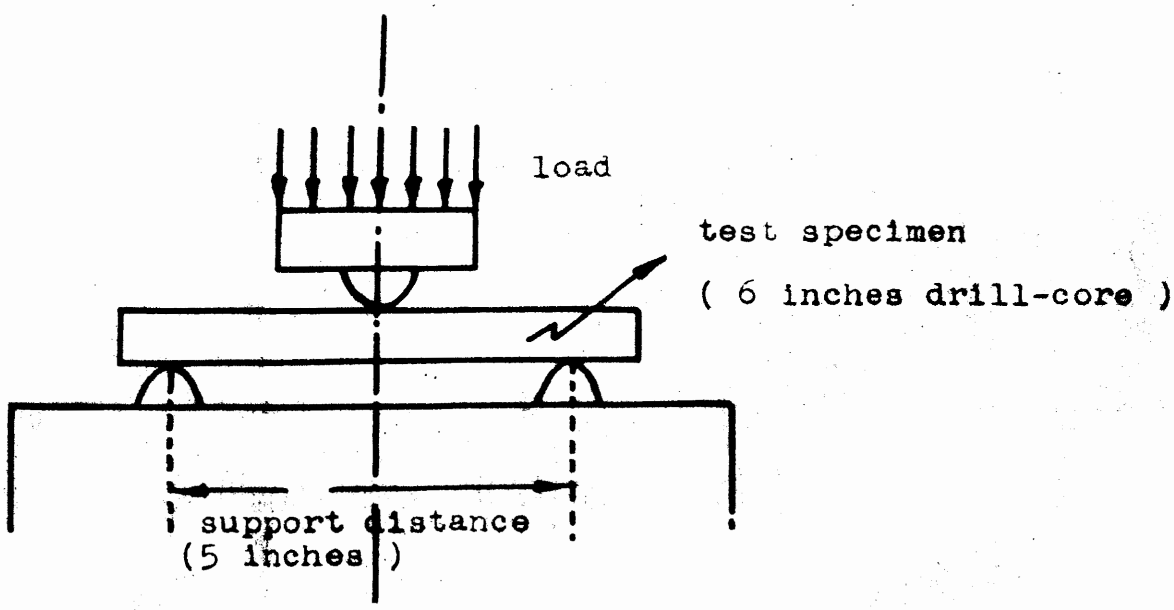


Fig. 10b. Loading Device for Drill-core Specimens in the Test of Modulus of Rupture

CHAPTER IV

ILLUSTRATIVE EXAMPLE: DESIGN OF ROOM AND PILLAR SPACING FOR A SPECIFIC GYPSUM MINE

The following example has been selected to illustrate the salient points in the standardized procedure for design of a room-and-pillar mine plan. This particular mine was placed in production in the latter part of 1959, utilizing a room-and-pillar mining method.

Geologic Field Data

The gypsum rocks which have been used as testing materials in this work were taken from the Shoals mine of the U. S. Gypsum Company. The location of this mine is at Section 26, Township 3N, Range 3W, Martin County, Indiana.

The gypsum occurs as bedded deposit. It is within the St. Louis limestone series of the Meramecian formation, Mississippian Period. The average thickness of the gypsum bed is less than 20 ft. The overlying materials are interbeds of limestone and sandstone. The depths of the overlying materials, namely, the distance from the topographic surface to the top of the gypsum bed vary from place to place due to the uneven topographic features. However, an average thickness of the overburden approximately of 400 feet is anticipated for the first twenty years of operation.

The upper part of the gypsum bed is composed of fine grained, hard and compacted, brownish mudstone; a couple of feet in thickness. Next below the mudstone, there lies a mixture of mudstone and gypsum that is approximately 50 percent gypsum. In the middle part of the gypsum bed, snow white, clear crystalline gypsum of nearly 90 percent purity occurs. In the lower part of the bed, there again occurs a mixture of mudstone and gypsum. Mudstone occurs as lenses, as spots and as irregular shapes, intermingled with gypsum.

Small fissures and fractures occur within these gypsum rocks. The contacts between mudstone and gypsum are particularly weak, although no obvious sheared zone could be seen from these rock samples.

Approximately 350 specimens were prepared for the different tests from 13 blocks of gypsum from one vertical column of the mine. Tests included the determinations of compressive strength, tensile strength, Poisson's Ratio, modulus of elasticity in compression, cementation tests and centrifugal model testings.

Estimation of the Pre-existing Stress Field

The initial stresses in the upper surface of the earth's crust, as mentioned in the previous sections were induced from various sources. Most of these stresses exist in the form of residual stress on a local or regional scale. In a particular vicinity, the stress field may vary due to uneven distribution of the weight of the overburden, discontinuities, topographic differences, the thermal gradient and nearby geotectonics.

From the geologic field data obtained, the average thickness of the overlying material over the gypsum bed is approximately 400 feet, of which, 90 feet is shale, and the rest consists of interbedded limestone, mudstone, and sandstone.

The data available also indicates that there are no obvious structural discontinuities in the area of the mine openings. From the depth of the overburden, it can be assumed that the initial stresses of this area may be considered to fit the laterally restrained case.

Based on the above considerations, the vertical stresses may be approximated from the weight and the density of the overburden theoretically as follows:

$$\sigma_y = -d \rho W_w$$

Where d = the depth of the overburden
ρ = the density of the overburden
W_w = the unit weight of water

The average density of shale may be assumed as 2.7. And, the average density of the limestone, sandstone and mudstone interbed may be assumed as 2.4.

The vertical initial stresses can then be calculated as follows:

$$\sigma_y = -d_1 \rho_1 W_w + d_2 \rho_2 W_w$$

Where ρ_1 = density of shale = 2.7

ρ_2 = density of limestone, sandstone, and mudstone interbeds = 2.4

W_w = unit weight of water (0.0361 lb./in.)

d_1 = depth of shale = 90 ft.

d_2 = depth of the limestone, sandstone, and mudstone interbeds = 310 ft.

Therefore:

$$\sigma_y = 90 \times 2.7 \times 12 \times 0.0361 + 310 \times 2.4 \times 12 \times 0.0361 = 106 + 323 = 429 \text{ p.s.i.}$$

If laterally restraint occurs, horizontal stresses should be considered, which may be approximated as follows:

$$\sigma_x = \frac{\mu}{1-\mu} \sigma_y$$

Where μ is Poisson's ratio, which is 0.2 in this case (obtained from physical property tests) (page 4.20)

$$\sigma_x = \frac{0.2}{1-0.2} (42) = 107 \text{ p.s.i.}$$

Due to the fact that geologic features are often heterogeneous, and some forces may present of uncertain origin, a deviation as great as 50 percent in the values presented above must be given consideration.

Results of Physical Property Tests

The physical property tests on gypsum rock specimens from Shoal mine, Indiana consisted of tests for compressive strength, tensile strength, Poisson's ratio, modulus of rupture and the calculation of the modulus of elasticity.

Preparation of Specimens

Thirteen $1\frac{1}{2}$-ft. cubes which were numbered from top to bottom of a gypsum bed in the above mentioned mine were obtained for these tests.

Cubes No. 1 to No. 5 were mostly medium grained, well crystallized gypsum. Cubes No. 5 to No. 12 were generally composed of mudstone lenses in a heterogeneous mixture primarily composed of gypsum. Some of these cubes contained as much as 50 percent mudstone. No. 13 was mainly composed of a very fine grained shaley limestone.

All specimens were prepared from the afore mentioned cubes by cutting cores of 1.25-inch diameter with a laboratory diamond drill-bit mounted in a drilling machine. Cores were taken in three mutually perpendicular directions from the cubes of gypsum. Cores which were cut parallel to the X-axis and Z-axis, in other words the direction of the horizontal bedding, were called horizontal cores (H). Cores which were taken parallel to the Y-axis, that is perpendicular to the bedding, were called vertical cores (V). (See Fig.11).

Seven 1-foot vertical cores were taken from each cube of gypsum. Commonly, fifteen shorter horizontal

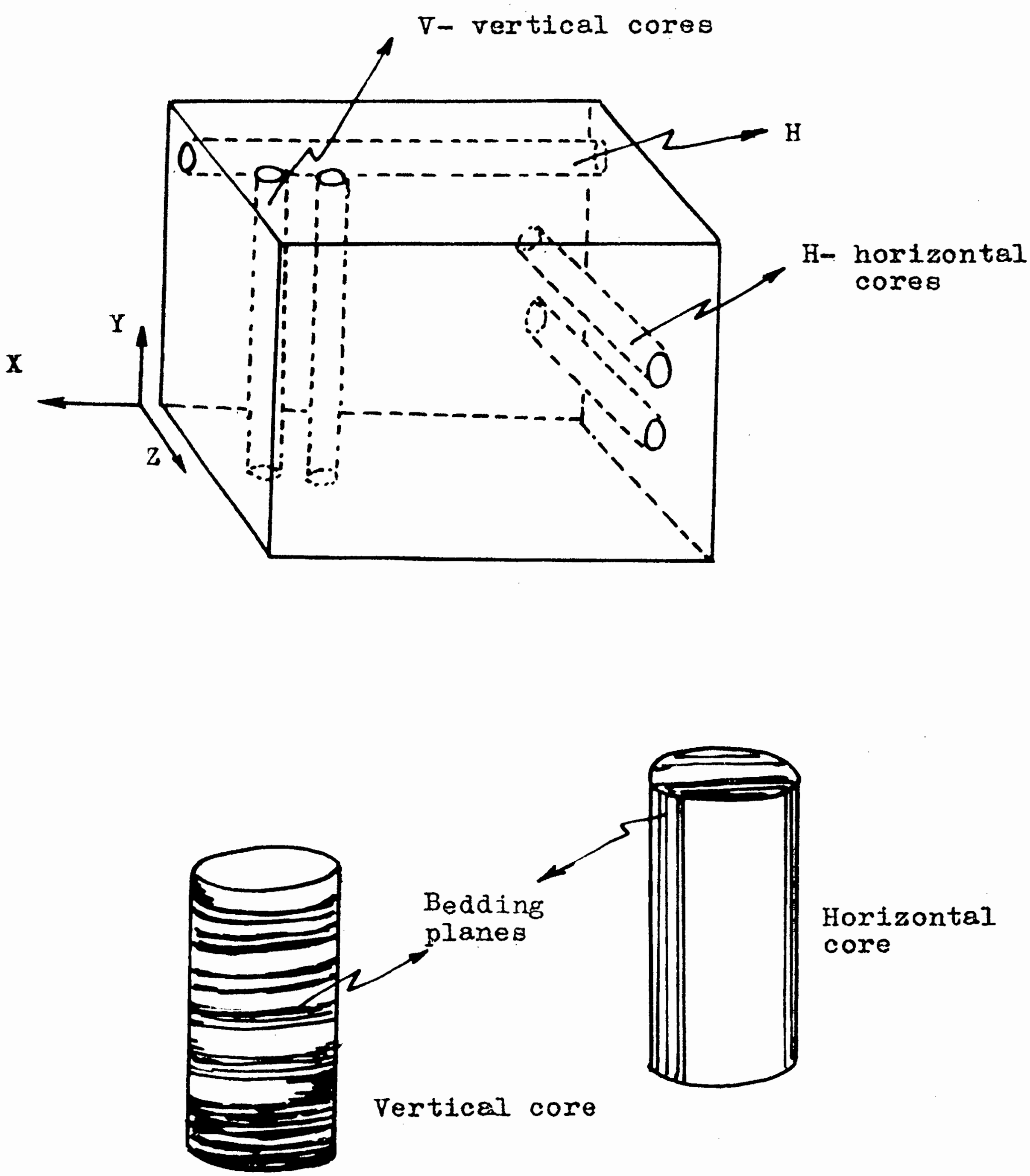


FIG. 11. Preparation of Specimens for Physical Property Tests .

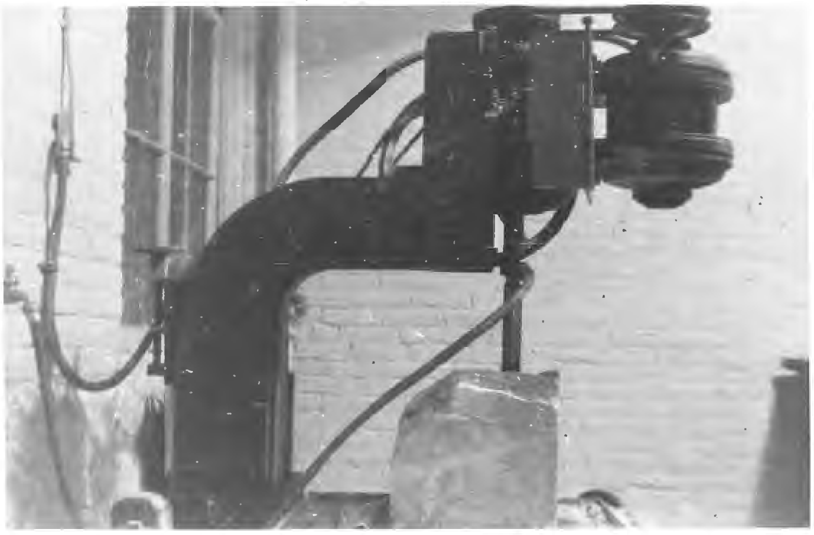


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FiG. 11 Diamond Drill.

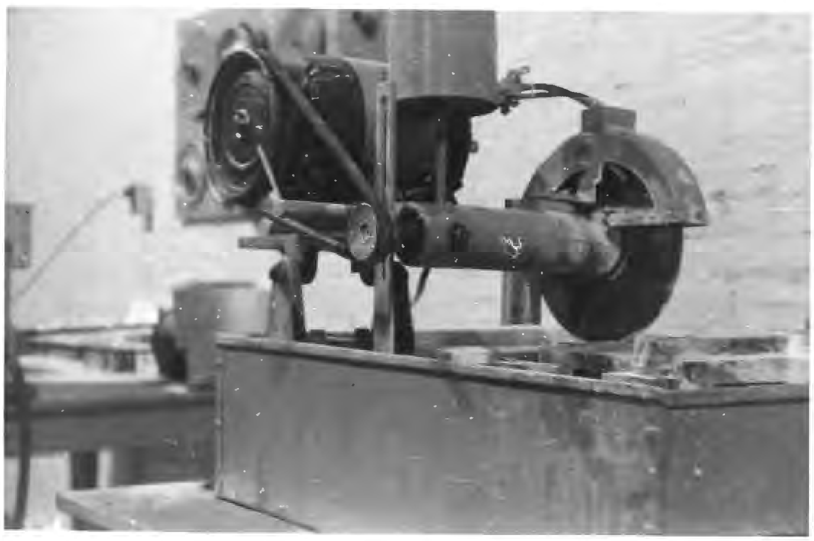

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Fig.12 Cutter.

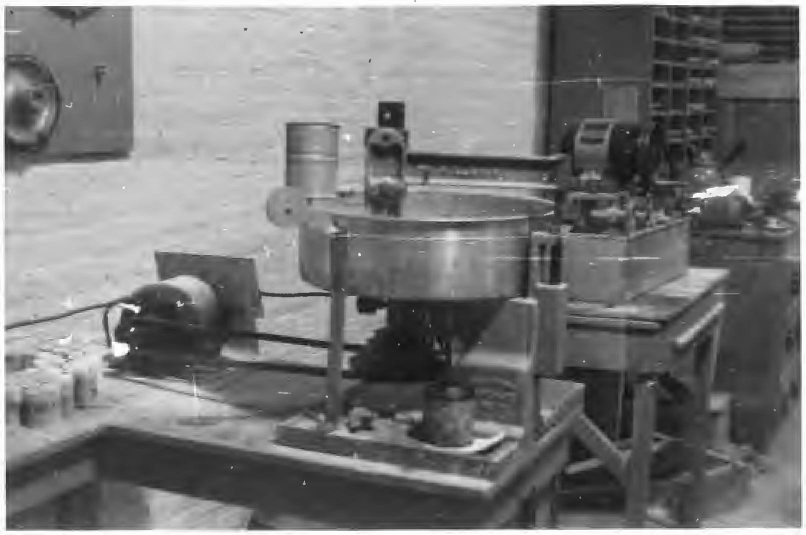

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Fig.13 Grinder.

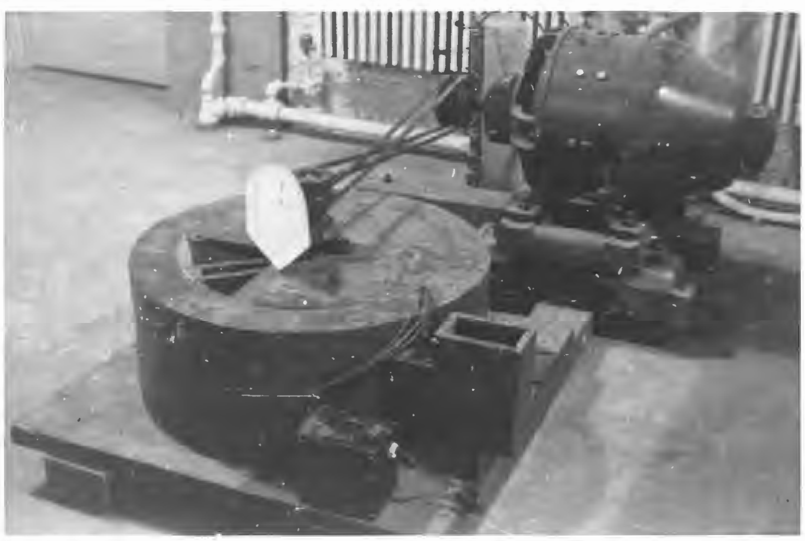

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Fig. 14 Centrifuge (small scale).

cores were taken from one half of the same cube, the other half being reserved for model tests.

The recovery of core was low in the middle part of the gypsum bed, because of weakness due to mudstone lenses, and fracture planes within the gypsum.

The long cores were then cut into many shorter cores to fit the requirements of the different physical property tests.

The specimens were then numbered using India ink. The length and diameter of all specimens were measured for tests by using 2-to-3 inch, and a 1-to- 2 inch micrometer, accurately to one thousandth of an inch.

The specimens were placed on the floor for two days in a well ventilated condition; after which they were tested for the particular property for which they were prepared.

All specimens used in these physical property tests were cylindrical. The typical dimensions of the specimens for compressive tests, tensile tests, Poisson's ratio and the modulus of rupture tests are as follows:

Compressive strength tests (length--1.25-inches, diameter--1.25-inches)
Tensile strength tests (length--1.25-inches, diameter--1.25-inches)
Poisson's ratio tests (length--2.24-inches,

diameter--1.25-inches)
Modulus of rupture (length--6-inches, diameter--1.25-inches).

Compressive Strength Tests

Cores from both the horizontal (H) and vertical (V) direction were tested to determine any an-isotropism of compressive strength.

In the compressive test, loads were applied parallel to the bedding in the horizontal core (H), and perpendicular to the bedding in the vertical core (V).

The specimen was placed between the compression platens of a 60,000 pounds Riehle Universal Testing Machine (Screw power type) for each test. The suggested rate of loading of 100 lb/psi per minute could not be achieved with this equipment. Therefore, it was necessary to halt five seconds between each unit interval of 300 or 500 lb. to approximate the desired loading rate.

Interpretation of compressive strength test data.

1. When the percentage of gypsum was high there was no clearly pronounced difference between stresses and their corresponding strains occurring as a result of compressive loads applied to horizontal cores and those occurring in the case of vertical cores. This was most likely due to the fact that bedding planes

Table IV

Ultimate Compressive Strength of Gypsum Rock,
Shoal Mine, Indiana

	Horizontal Cores			Vertical Cores		
Block No.	Ave.Ult. Comp.Str. (psi)	Stand. Dev.	Percent Stand. Deviation	Ave.Ult. Comp.Str. (psi)	Stand. Dev.	Percent Stand. dev.
1	2247	133	5.9	2582	325	12.6
2	3138	644	20.5	2520	271	10.8
3	2175	190	8.7	1983	97	4.9
4	3261	428	13.1			
6	2510	209	8.3	2617	811	30.9
7	3176	406	12.8	3346	174	5.2
8	3324	482	14.5	2741	414	15.1
9	2620	241	9.2	2398	640	26.7
11	3166	264	8.3	3614	147	4.1
12	2826	654	23.0			

(Each value contains an average of
ten specimens)

within the gypsum were not distinct. Since the gypsum is quite massive, the physical properties tend to be mechanically isotropic.

2. When the rock mass was heterogeneous, namely where there were muddy, shaley materials inside the fissures of gypsum, the irregular mixture of gypsum and mudstone gave quite erratic elastic characteristics as shown by the stress-strain diagrams. It may be concluded from the data that the compressive strength of this type of rock is comparatively low. The occurrence of failure at rather low stresses in the fissured, cracked or otherwise weakened specimens was not uncommon.

3. There were no sounds emitted during the failures of the specimens, except in group No. 13 which was composed mainly of fine grained limestone. In specimens of group No. 13, there was an explosive burst at the failure of each specimen.

4. The failure usually occurred on a surface inclined to the direction of loading. Nearly half of the failure surfaces made an angle of 65° (25° with the principal stress plane) or greater with the horizontal, and the remaining failure surfaces made an angle with the horizontal slightly smaller than 65° (from 65° to 50°). Only a small portion of all the specimens tested show a fractured pattern of many cracks parallel to the direction of loading. These parallel cracks may have been due to uneven loading. The inclined fracture pattern indicates that failures where due to a critical combination of shearing and normal stresses (such as may be

predicted by use of Mohr's envelope.) (See Figure 15-b).

5. The average ultimate compressive stress of the tested specimens obtained from the experiments was 2725 PSI in the vertical cores.

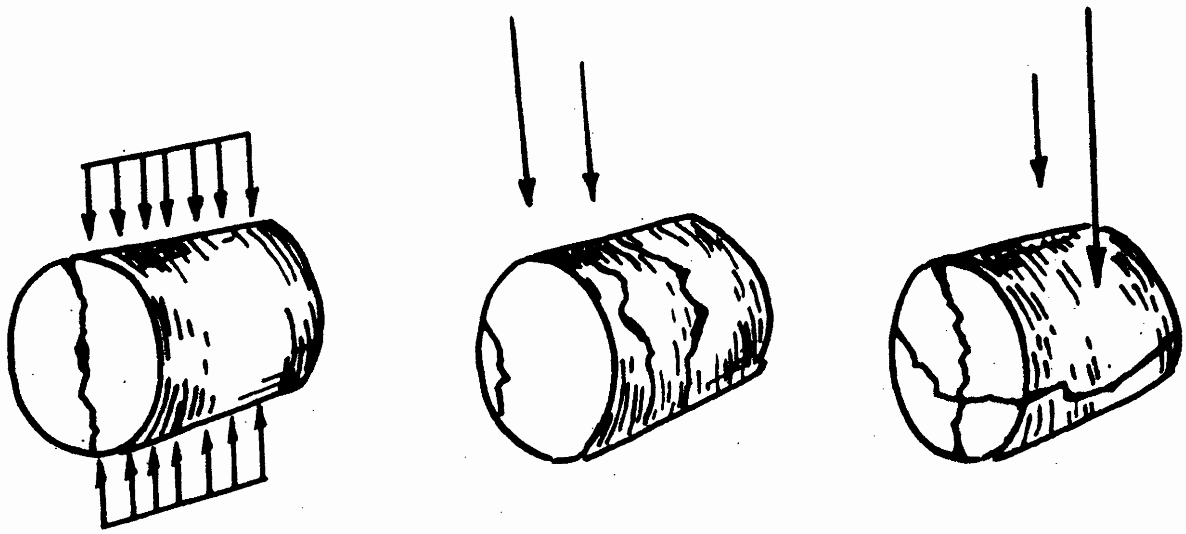
Tensile Strength Tests

Tensile tests were performed using the indirect tensile strength method which has been described in the previous section.

In these tests, the specimen was placed in the testing machine in such a manner that the direction of load was perpendicular to the axis of the cylinder. Compressive load was applied, and tensile stresses were induced. The test machine was the same as that one used for the compressive strength test. The rate of load was 100 lb./in.^{2} per second until failure occurred. The five seconds halt was also adapted to reduce the loading rate. No strain measurements were performed in these tests.

Interpretation of tensile strength test data

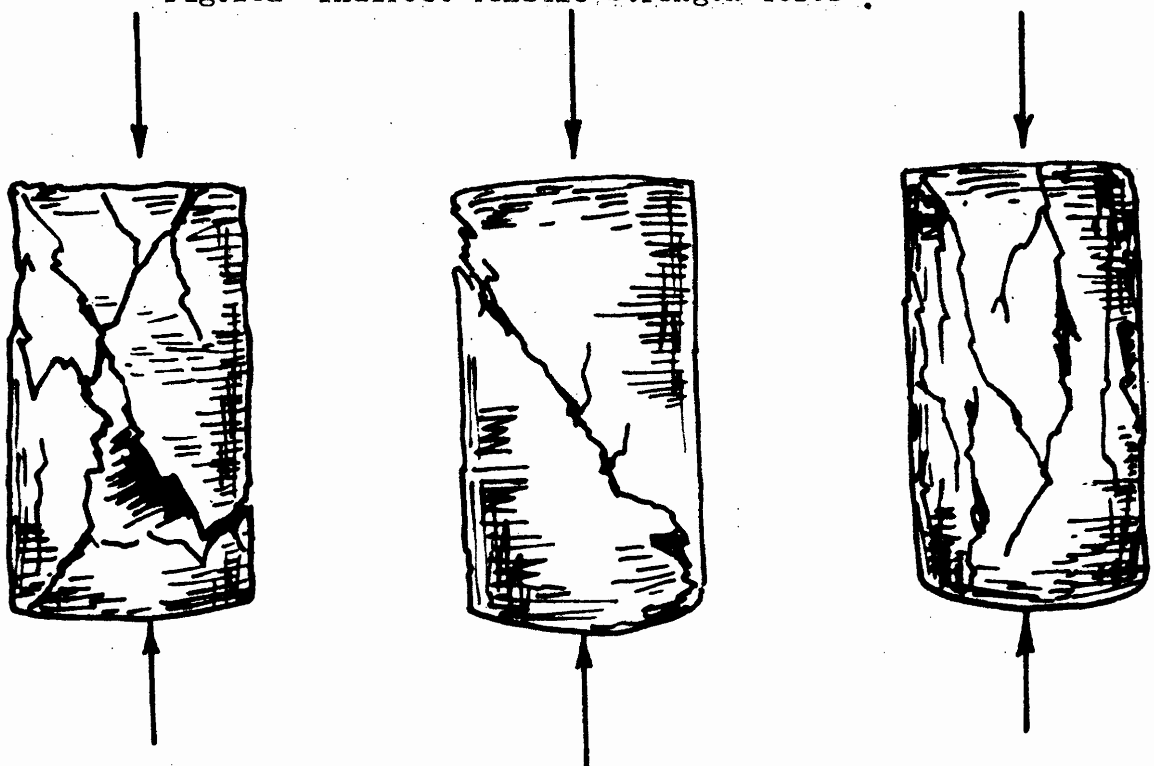
1. The average ultimate tensile stress was 399.5 PSI. (the average ultimate compressive stress was 2820 PSI). The ratio of the compressive stress to the tensile stress was 7.05 : 1.00. This value shows that the tensile strength of this rock is relatively higher in comparison to the compression strength than other rocks



(a) Typical tension crack, under uniform uniaxial loading condition.

(b) Tension cracks which are due to uneven contact uniaxial loads.

Fig.15a Patterns of Fractures in The Indirect Tensile Strength Tests.



(a) Typical compression crack under uniform uniaxial loading condition.

(b) Compression crack under uniform uniaxial loads.

(c) Cracks due to uneven contact uniaxial compressive loads.

Fig.15b Patterns of fractures in the Compressive Strength Tests.

where the ratio lies between 10:1 and 40:1.

2. Almost all specimens were broken silently except those specimens from cube No. 13.

3. More than 95 percent of the specimens showed a vertical tensile crack after failure. This crack sharply divided the specimen cylinder into two approximately equal parts, parallel to the direction of load.

4. Other patterns of fracture also existed. Specimens No. 4H-4, 6H-7 & 13H-9 developed horizontal fractures which were perpendicular to the normal tensile crack. Another fracture pattern was shown by specimens No. 1H-2, 1H-4, and 13H-13, in which planes developed separating the cylinder into two or more smaller cylinders. The causes of these latter kinds of fracture patterns may be explained by one or more than one of the following assumptions: They are due to (a) uneven loading; (b) pre-existing planes of weakness; or (c) bonding along bedding planes is weaker than the basic rock tensile strength. (See Figure 15-a).

5. The contact of the loading platens of the test machine and the specimens was always spread over a small area rather than a line contact. This is due to the soft nature of the gypsum which permits local deformations and causes some error in the determination of the true tensile strength. Since this error is approximately the same in all specimens, it has little effect on comparison of tensile strength of rocks.

Table V

Ultimate Tensile Strength of Gypsum Rock,
Shoal Mine, Indiana

	Horizontal Cores			Vertical Cores		
Block No.	Ave.Ult. Tens.Str. (psi)	Stand. Dev.	Percent Stand. Dev.	Ave.Utl. Tens.Str. (psi)	Stand. Dev.	Percent Stand. Dev.
1	491	158	32.2	367	82	22.4
2	449	99	22.1	448	38	8.6
3	335	11	3.3	278	70	25.3
5	271	40	14.5	365	38	10.6
6	346	60	17.5	393	69	17.5
7	348	50	14.5	343	84	24.5
8	325	85	26.2	342	27	7.8
9	303	64	21.1	319	84	26.4
11	412	56	13.7	358	40	11.2
13	1384	128	9.2	298	74	24.9

(Each value contains an average of ten specimens)

(See also Figures 27 and 32)

<u>Modulus of Elasticity</u>

In this work, the modulus of elasticity of gypsum was determined from data obtained from the ultimate compressive strength tests, and should, therefore, more properly be called the compressive modulus of elasticity.

The compressive strains of the gypsum specimens were determined by measuring the movement of the loading platen of the test machine at intervals of load by means of a dial gage. The strain of the specimen was determined as follows:

$$\varepsilon_L = \frac{R_1 - R_0}{L}$$

Where L is the original length of the rock specimen

R_1 is the dial reading after load P has been applied

R_0 is the original reading of the dial before load P has been applied

Curves of stresses versus the corresponding strains showing the characteristic straignt line of the compressive modulus of elasticity have been plotted using data obtained from these tests. The modulus of elasticity of gypsum was obtained by the tangent method described previously. Table VI summerizing the modulus data follows.

<u>Interpretation of modulus of elasticity data.</u>

Table VI

Compressive Modulus of Elasticity of Gypsum Rock, Shoal Mine, Indiana

	Horizontal Cores			Vertical Cores		
Block No.	Mod. of Elas. 10^6 (aver.)	Stand. Dev. 10^6	Percent Stand. Dev.	Mod. of Elas. 10^6 (ave.)	Stand. Dev. 10^6	Percent Stand. Dev.
1	.28					
2	.68	.52	77	.56	.06	11
3	.24	.08	34	.33	.11	33
4	.99	.16	16			
6	.85	.17	20	.67	.21	32
7	1.06	.24	23	.97	.25	26
8	.96	.16	16	.76	.12	15.2
9				.21	.07	36
11	1.11	.05	4.3	1.24	.17	14
12	.37	.19	53			

1. The values of modulus of elasticity obtained from these experiments varied from 0.21 x 10^6 to 1.24 x 10^6 psi. The data indicated that this elastic property of the gypsum rock varied from point to point within the bed. This may have been due to the fact that the gypsum content differed everywhere within the bed. The average value of the modulus of elasticity (both from vertical and from horizontal cores) was 0.703 x 10^6 psi.

2. The specimens which developed a higher ultimate compressive strength possessed a longer and more ideal, elastic range than those in which failures occurred at relatively lower stresses. The values of the modulus of elasticity obtained from the former groups of specimens are higher than those from the latter case.

3. In many cases, the stress-strain curves did not show a straight line portion which should be characteristics of a material within the elastic limit.

<u>Poisson's ratio</u>

Poisson's ratio of each gypsum specimen was determined by the employment of four resistance type strain gages (SR-4, A-3, strain gage, Baldwin-Lima-Hamilton Corporation) which were attached at the surface of the specimen, in such a manner that two were parallel to the specimen axis of symmetry; the other two were perpendicular to the axis. SR-4 strain gage cement was used to attach these gages to the specimen. The gages were connected to a strain gage bridge to measure the longitudinal and lateral strain of the specimen under compressive load.

Interpretation of Poisson's ratio data.

1. From the average test data, a Poisson's ratio for this particular gypsum rock is 0.2 which is slightly lower than the common value (0.25).

2. During the early period of loading, strains were irregular.

3. Above 2000 PSI, Poisson's ratio appears to increase rapidly. This is due to spalling of rock under the lateral gages, increasing the apparent lateral deformation. (See Figure 26).

Modulus of Rupture

The procedure for modulus of rupture tests followed the standardized method proposed by the U. S. Bureau of Mines.

The length of the cylindrical specimen obtained from drill core was 6 inches, and the separation between the centers of the knife edge supports was 5 inches (as shown in Fig. 10-2a) The machine used was the same universal testing machine as used in compressive tests. The specimens were loaded at the rate of 100 pounds per minute.

The modulus of rupture was calculated by the use of the following equation:

$$R = \frac{8WL}{\pi D^3}$$

Where R = modulus of rupture
W = applied load
L = span between centers of knife edges
D = diameter of specimen

Interpretation of modulus of rupture data

1. An average value of 1199.64 PSI for the modulus of rupture of this particular gypsum rock was obtained on the basis of 25 specimens. Magnitude of percentage of standard deviation is 22 percent.

2. Those existing specimens which had visible weaknesses developed slightly lower than average values of modulus of rupture.

Results of Centrifugal Model Tests

More than twenty rock beams were taken from the upper and the middle parts of the gypsum bed for the destructive centrifugal tests. The work was performed using the centrifuge of the Department of Mining Engineering at Missouri School of Mines and Metallurgy.

Equipment for the Tests

The equipment used for the centrifugal tests consisted of a centrifuge and its accessories. The centrifuge was made up of a housing tank, a rotor, model holder and counterweight, a D. C. motor, stroboscopic light and its connection. The accessories included a balance and a vacuum pump. (See Figures 16 and 17).

The housing tank was made of steel. The diameter of the tank was 90 inches. Two observation ports were on opposite sides of the tank. An opening in the cover of the tank permitted a person to enter the tank. (See Fig. 21).

The rotor was a rectangular box-like structure made of alluminum-alloy plate. The height of the rotor was 33 inches and its length was 86 inches. The overall length of the vertical shaft was $74\frac{3}{4}$ inches. The diameter of the shaft at its largest section was 4 inches. The shaft bearings were bolted to the structural steel channels at the top and bottom of the tank. (See Fig. 22).

The model holder and counterweight were made of steel. The support distance of the model holder was fixed.

A 10 h.p. D.C. motor with a maximum speed of 1750 R.P.M. was installed outside the housing tank.

The stroboscopic light was triggered by a contact on the uppermost end of the shaft. A pair of mirrors were set up to permit viewing the image of the specimen inside the rotor.

A one h.p. vacuum pump with an adjustable outlet valve was connected directly to the housing tank.

Procedure of the Test

The apparatus for measuring strain was not installed at the time these tests were performed, therefore, all experiments were destructive tests. The procedure of these tests consisted of the following steps: (1) Preparation of specimens--the gypsum rocks were cut into rectangular beams 12 inches in length, 2 inches in width approximately; their thickness was varied with different tests. (2) Mounting model--Both ends of the specimen beam were clamped by screws in the model holder. The model holder with specimen was weighed and recorded. (3) Assembling counterweight--Metal plates were added to the counterweight until the weight was the same as the weight of model holder and specimen. (4) The loaded specimen holder and counterweight were placed separately in the two ends of the rotor of the centrifuge. (5) The entrance of the housing tank was sealed tight to prevent air leaks. (6) The vacuum pump was turned on to evacuate the air inside the tank. (7) The stroboscopic light was turned on. (8) The D.C. motor was started and the rotor slowly brought up to speed. (9) The performance was observed through the glass window of the housing tank by means of a pair of deflected mirrors. (10) The first occurrence of a crack was observed and the speed at that point recorded. (11) The rotation of the centrifuge was continued until the specimen had completely failed, and the speed at that point was recorded. (12) The vacuum pump was turned off and air was admitted to aid the electronic braking system in bringing the rotor to a stop. (13) The broken specimen was removed from the centrifuge, and the crack pattern and the specific gravity of the specimen were recorded.

<u>Interpretation of the Tests Data</u>

Data obtained from these tests and their interpretation are presented in the following pages.

The speed of rotation at failure ranged from 250 R.P.M. to 560 R.P.M. for gypsum beams approximately one inch in thickness. The average speed obtained from seven tests was 432 R.P.M.

The speed of rotation of the centrifuge in R.P.M. at the point of failure was linearly proportional to the thickness of the specimen beams, since the effective length of the beams had already been fixed at a specific value (8.02 inches). Namely, the thicker the beam, the higher the R.P.M. obtained at failure.

The R.P.M. at the occurrence of first fracture shows no relation to the thickness of the beam. For example the first fracture occurred at 480 R.P.M. (complete failure at 510 R.P.M.) in gypsum beam No. 1 which had an average thickness of 0.99 inches, and the first fracture occurred at 390 R.P.M. (complete failure at 630 R. P.M.) in gypsum beam No. 8 which had an average thickness of 1.51.

The differences in R.P.M. between first fracture and final failure varied from beam to beam.

In all of the tests of these gypsum beams, three

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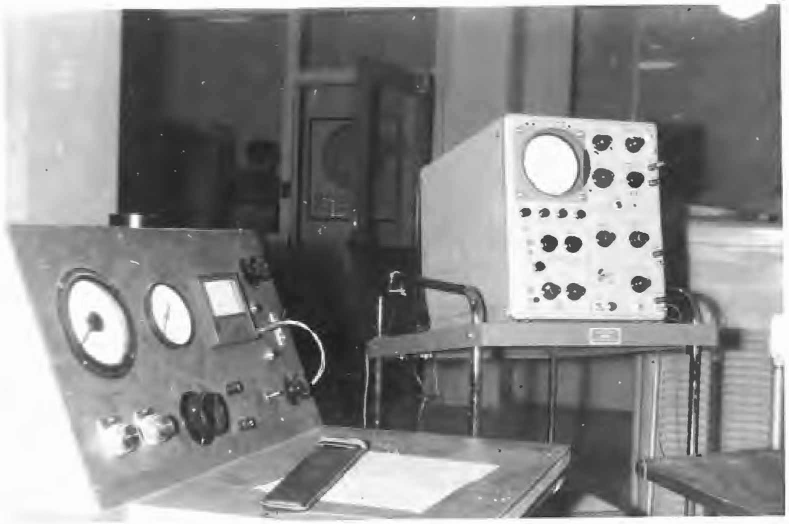

Fig. 16 Control~~lers of~~ Center for Centrifuge.

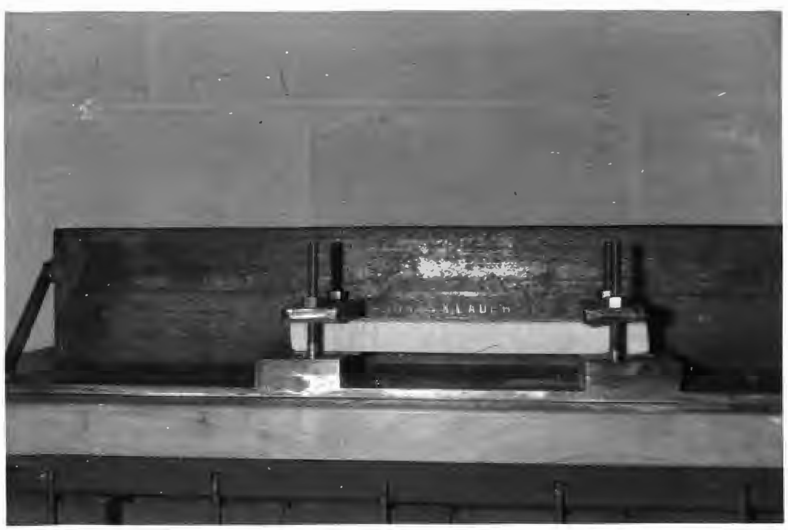

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Fig. 17 Gypsum Beam and Specimen Holder.

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Fig. 18 Broken Gypsum Beam After Test.

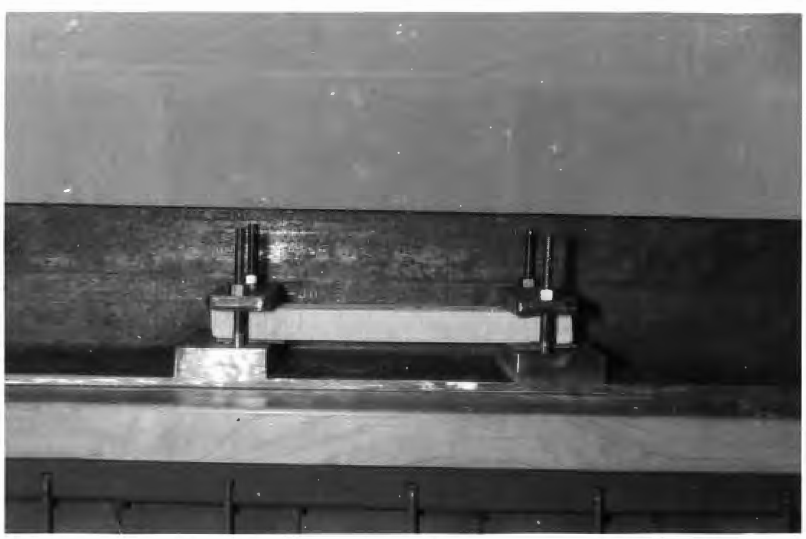

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Fig. 19 Gypsum Beam Before Test.

Fig. 20 Centrifuge Installation

Fig. 21 Observer Ports and the Entrance of the Centrifuge.

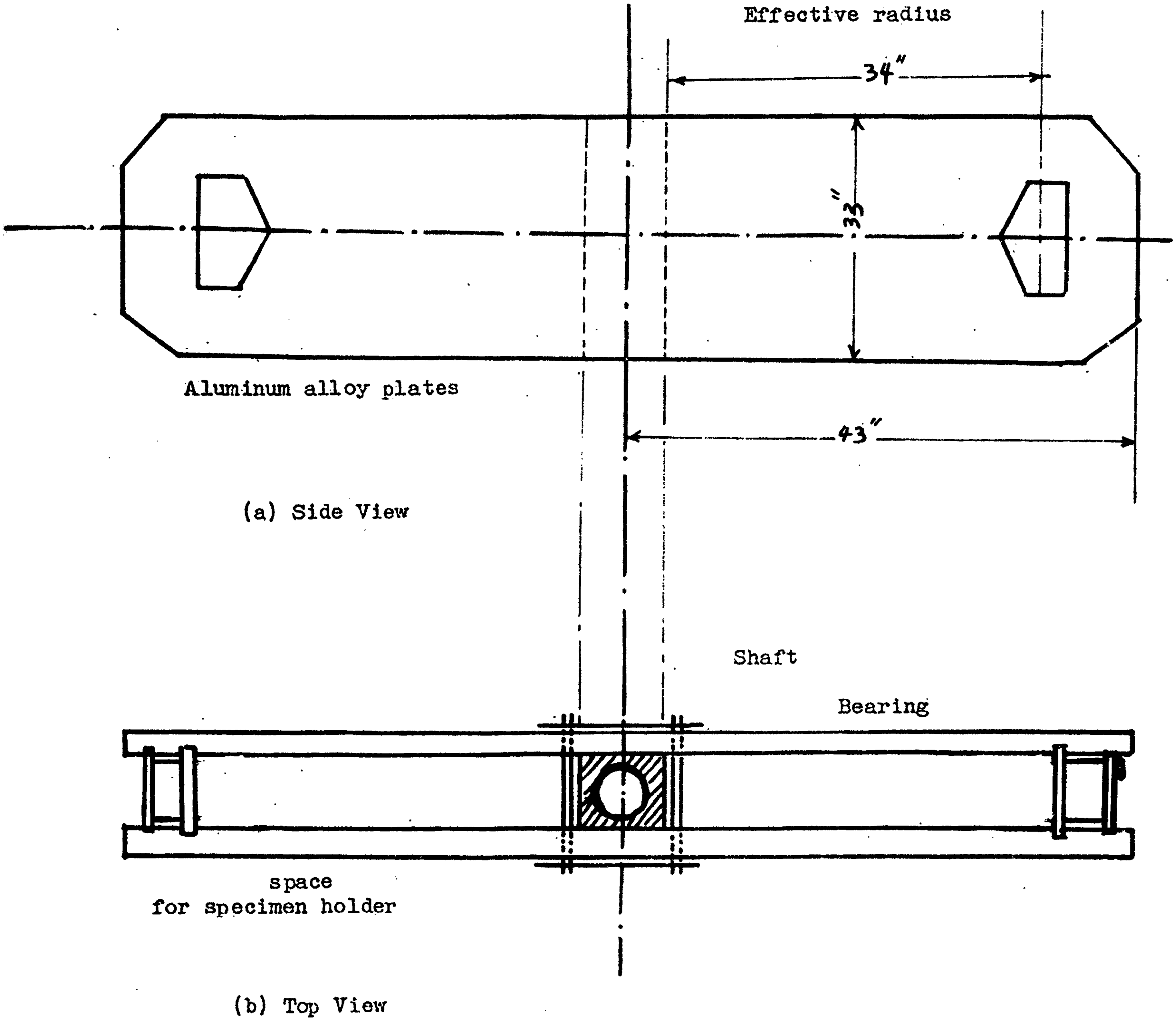


Fig. 22 Simplified Sketch of The Construction of the Rotor of the Centrifuge at Missouri School of Mines and Metallurgy.

stages were observed during loading in the centrifuge: (a) beam in uncracked condition--at this stage, there was no significant loading effect; (b) before and after the occurrence of the first fracture--at this stage, there was no apparent deflection to be seen before the occurrence of the first crack. After the appearance of the first crack, a slight deflection could be seen as the speed increased. Other cracks sometimes formed at the end of this stage. The origin of the cracks was often uncertain. Individual cracks may have been caused by tensile or shearing stresses, or a combination of those stresses. When the first crack occurred toward one side of the bean, cantilever action resulted in complete beam failure. If the first crack occurred at the middle of the beam, local crushing occurred. (c) The final failure--This stage was a continuation of the second stage in practically all of the gypsum beams. No. 5-3 gypsum beam was an exception, where the whole beam was broken suddenly along several planes, resulting in complete failure without a second stage. (See Figures 18, 19 and 24).

The dimensions of the model beams used in these tests are given in Table VII. The model scale factor, equal to the number of "g's" developed at failure, was calculated from the equation on page 2.24 for each of the specimens. The prototype dimensions were then determined. Prototype dimensions from each beam test were plotted on log-log paper to obtain a length versus thickness relationship of the beams at failure, as shown in Figure 23.

Table VII

Centrifugal Tests of Gypsum Beams

Model no.	R.P.M.	Effective Radius (inches)	Scale Factor	Av. Model Dimensions (inches)		Prototype Dimensions (feet)	
				Thick-ness	Sup-port Dis.	Thick-ness	Support Dis.
2-1 [1]	510	34	250.06	0.99	8.02	20.62	167.50
2-3	635	34	388.00	1.17	8.02	37.80	259.00
2-4	600	34	346.20	1.17	8.02	33.86	230.05
2-5 [2]	525	34	265.7	1.20	8.02	26.57	177.57
2-8 [3]	630	34	383.00	1.51	8.02	48.20	250.58
5-3	560	34	302.31	1.013	8.02	25.52	202.04
5-1	250	34	62.66	1.080	8.02	5.64	41.87
6-3	360	34	124.41	0.866	8.02	8.97	83.14
6-2	400	34	154.24	0.900	8.02	11.56	103.08
5-2 [4]	450	34	195.00	1.02	8.02	16.60	130.55
6-4	525	34	265.7	0.9	8.02	19.92	177.57

Note: Unit weight of these blocks, $W = 0.054$ lb./in.^{3}

[1] First crack occurred at 480 R.P.M.
[2] First crack occurred at 350 R.P.M.
[3] First crack occurred at 390 R.P.M.
[4] First crack occurred at 250 R.P.M.

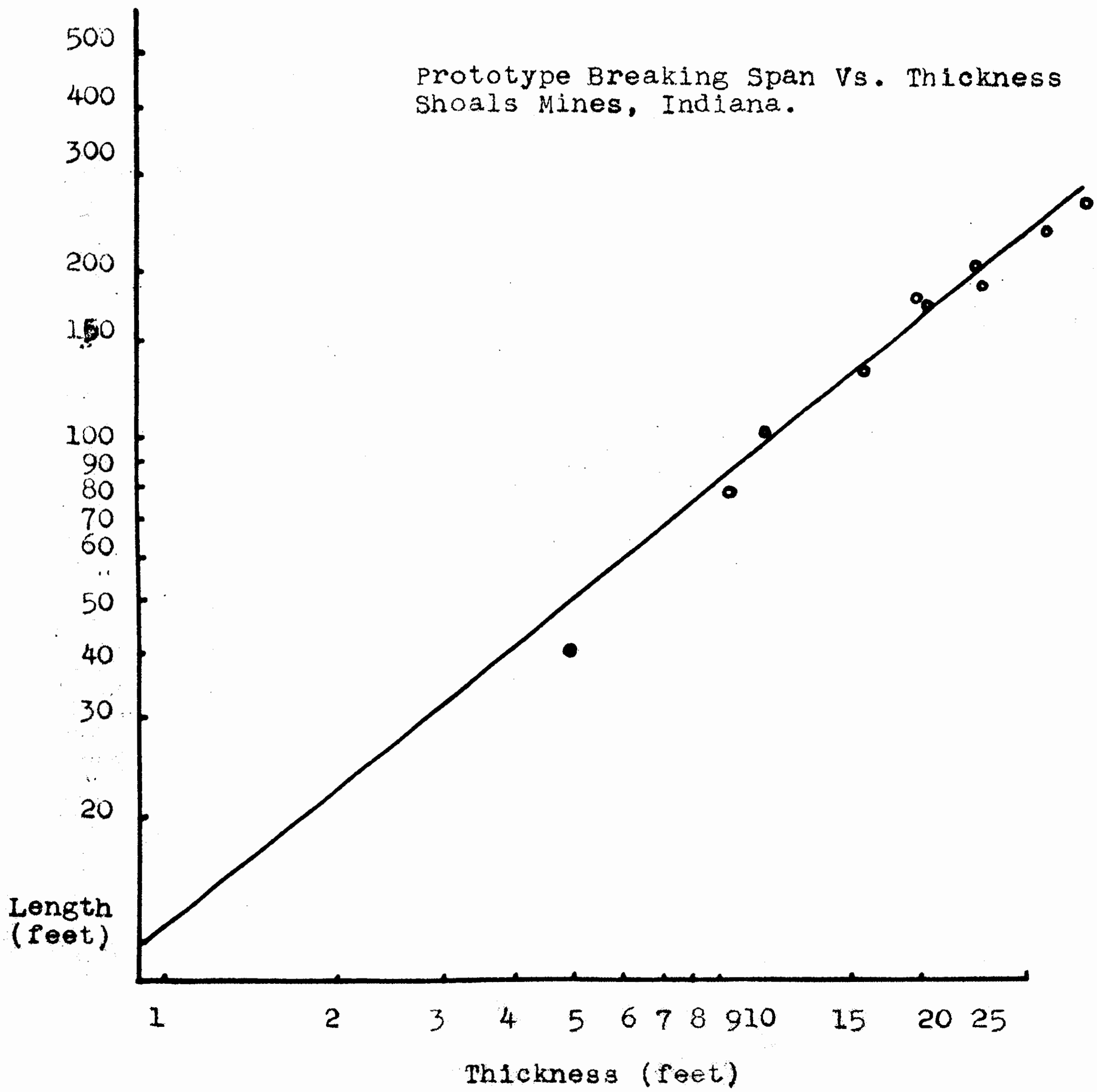
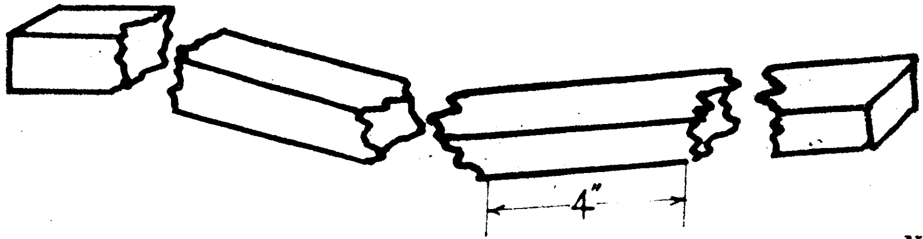


Fig. 23 Length-thickness Curve of Gypsum Beams From Centrifugal Model Tests.



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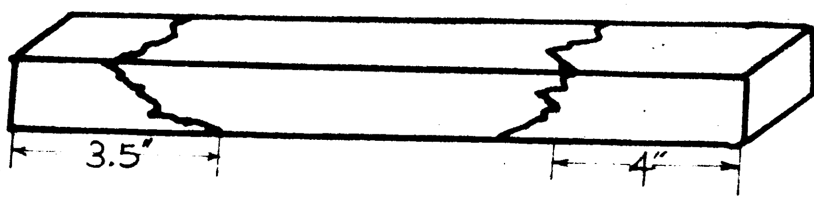

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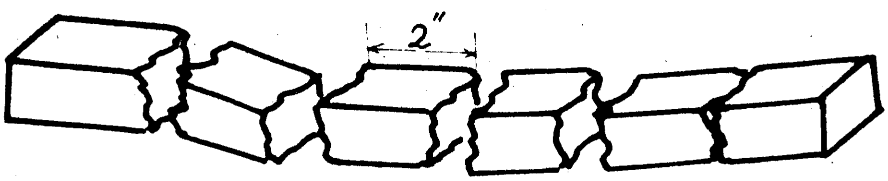

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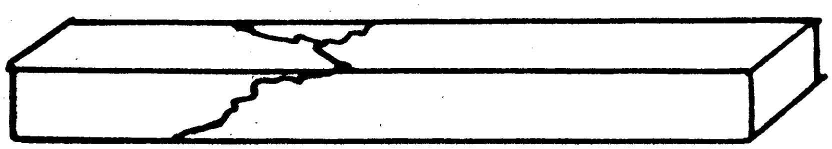

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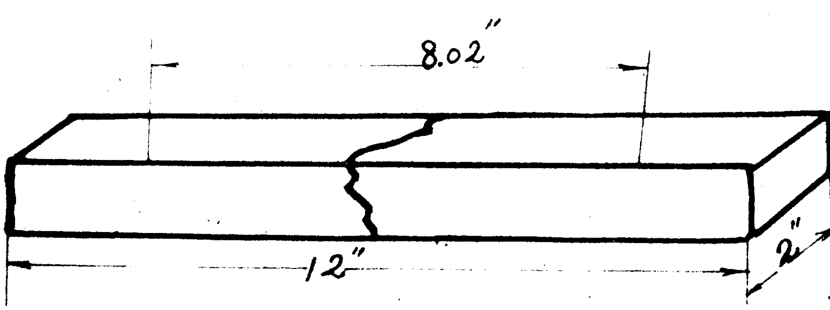


No. 6-3

Fig. 24. Typical Patterns of Failure of Gypsum Beams.

Preliminary Room and Pillar Spacings

Safe Roof Span

In general, the determination of roof span depends mainly upon the thickness of the competent roof rock immediately above the mine opening, which can either be the result of natural bedding or may be achieved artificially by roof bolting.

It was assumed that a thickness of competent roof in the gypsum mine of 7.5 feet was the maximum thickness which could be expected to be maintained over the entire mine. The corresponding maximum length of the span of the roof was then determined from the length-thickness curve (Figure 23) to be 70 feet. A factor of safety of two was chosen for determination of a safe roof span. Such a small value was chosen for two reasons: the use of the diagonal distance between pillars as a design criteria, and the assumption of a thickness of competent bedding of only 7.5 feet are both conservative. The safe span was determined from the maximum span of 70 feet, by the relationship

$$L_s \text{ (pooted)} = \frac{L_{max}}{\sqrt{2F}} = \frac{70}{\sqrt{2x2}} = 35 \text{ ft.}$$

This value was checked by use of the theoretical beam equation as follows:

$$L_s \text{ (theoretical)} = \sqrt{\frac{T_a t_{max}}{\rho F}} = \sqrt{\frac{(1199x144)7.5}{(2.3x62.4)2}} = 67 \text{ ft.}$$

Where T_a, the modulus of rupture = 1199 psi

t_{max}, the thickness of roof beam = 7.5 ft.

ρ, the density of gypsum = 2.3x62.4 lb/ft.^{3}

The discrepency between the centrifugal test result and that of the theoretical calculation serves as an indication of the necessity for the centrifugal test. The difference between the results is most likely due to the fact that weaknesses in relatively large rock beams tested in the centrifuge are more likely to produce failure because peak bending stresses exist over a considerable part of the span of the beam; whereas, the theoretical results are based on a modulus of rupture test which is relatively insensitive to weakness within the rock specimen since the maximum bending stress decreases rapidly on either side of the central load.

Safe Pillar Size

The safe pillar size may be determined, provided that the safe roof span, the initial earth stress, and the ultimate compressive strength of the pillar rock have been determined.

An ultimate compressive strength of the gypsum in the pillars of 2000 psi was conservatively chosen for design purposes because this is the lowest average value of strength of any of the gypsum blocks tested.

In calculating the average pillar stress, a safety factor of 2.0 was chosen. This value is relatively low compared to some other fields of structural design; however, it should not result in any unconservatism since the effect of any fractures or faults in pillars has only negligible effect on their strength. In addition, there is probably an increase in relative strength of large pillars, over test specimens due to confinement toward the center of the pillar (13).

The average pillar stress may be determined from the following equation

$$\sigma_{av} = \frac{\sigma_{ult}}{F} = \sigma_y \frac{(L_o + L_p)^2}{L_p^2}$$

Where $\sigma_{ult.}$, the ultimate compressive stress = 2000 psi

F, the factor of safety=2

σ_y, the initial earth stress = 429 psi (p. 4.4)

L_o, the safe span 35 ft. (p. 4.32)

L_p, width of pillar

The safe pillar width, based on the average pillar stress was determined by solving the above equation for L_p

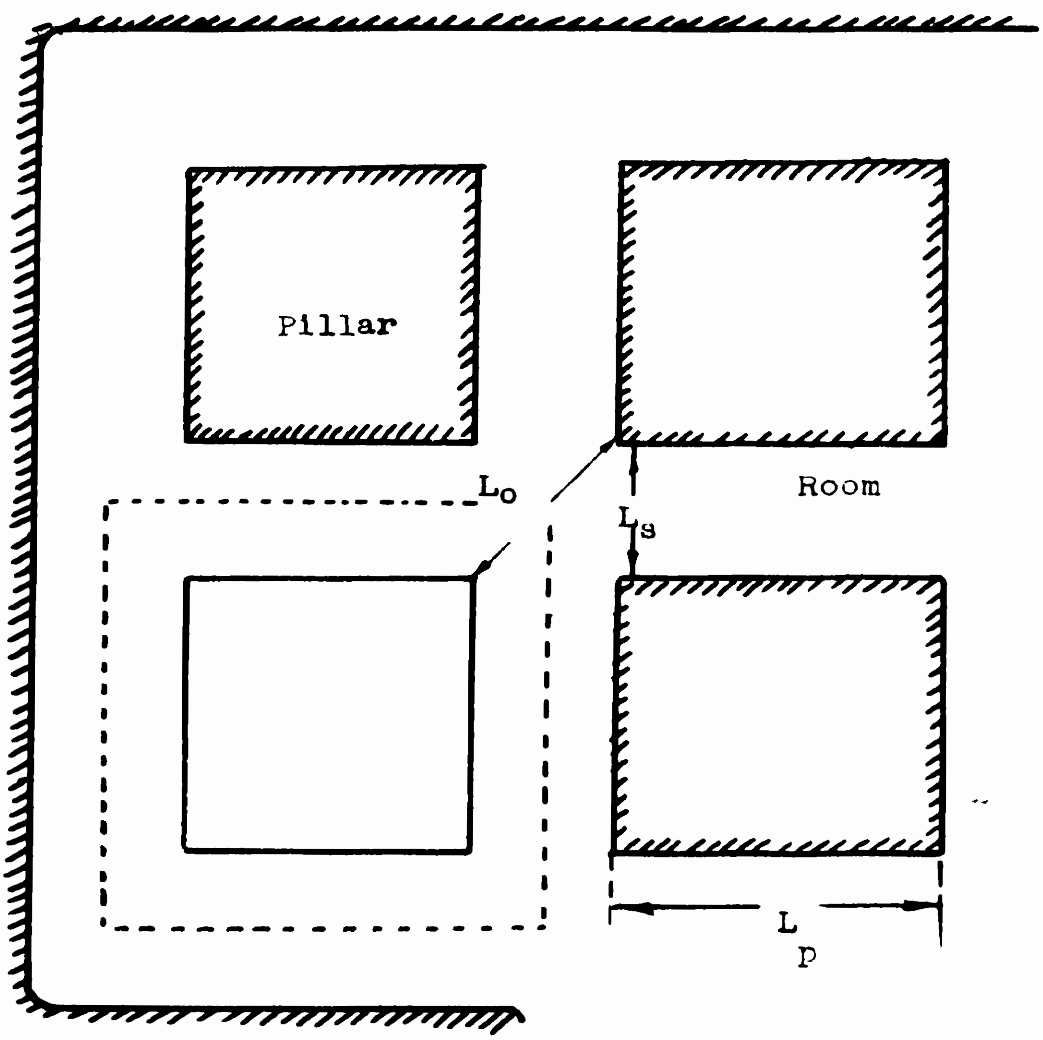
$$L_p = \frac{(F\,\sigma_y)^{\frac{1}{2}}}{\sigma_{ult}^{\frac{1}{2}} - (F\sigma_y)^{\frac{1}{2}}}\, L_o$$

$$= \frac{(2x429)^{\frac{1}{2}}}{(2000)^{\frac{1}{2}} - (2x429)^{\frac{1}{2}}}\,(35) = 64 \text{ ft.}$$

The example chosen has resulted in a room-and-pillar mining method with roof span L_s of 35-feet, and pillar width of 64-feet, as shown in Figure 25. A 58 percent recovery would be realized initially using these dimensions.

Fig. 25

ROOM AND PILLAR S.ACING



CHAPTER V

Summary

A systematic approach to the design of underground mine openings lessens the possibility of serious accidents, while at the same time increasing productivity. An attempt was made in this paper to set forth a standardized procedure by which the design of a room and pillar system of mining could be systematized.

There are three basic steps in the design of a stable underground mine structure: the estimation of the pre-existing earth's stresses; the determination of pertinent physical properties of mine rocks; and the determination of the magnitude and distribution of the stresses which result from the introduction of underground mine openings.

The pre-existing stress in the earth's crust can be estimated both theoretically and experimentally. There are three different conditions which may be used to approximate the pre-existing stress theoretically. They are the unidirectional, lateral restraint, and hydrostatic cases. Variations in the initial stress field may occur due to local and regional geologic conditions.

A more precise method of determining the earth stress field, after mining has commenced, involves the use of a flat-jack to obtain experimental stress measurements in an underground mine opening.

The properties which are of primary importance in the design of underground mine openings are rock strength, elastic constants and specific gravity; since the size and spacing of mine pillars depend upon the compressive strength, modulus of rupture, specific gravity, and Poisson's ratio of the rock; the deformation of rock is usually close related to the elastic properties of the rock; and the magnitude of pre-existing stresses in most cases depends upon specific gravity and Poisson's ratio. Comparison of data on physical properties of similar types of rocks can provide reference information.

Standardization test methods for physical properties of mine rock is necessary for the purpose of obtaining accurate data which may be correlated with the work performed by other investigators. Ultimate compressive strength, ultimate tensile strength, modulus of elasticity, Poisson's ratio, modulus of rupture, and specific gravity are the pertinent physical properties of mine rocks.

The existence of underground mine openings creates disturbances in the pre-existing underground stress field. These disturbances occur as ~~in~~ variations in magnitude, direction, and distribution, corresponding to the shape and number of openings. Mathematical analysis based on theories of elasticity, plasticity and soil mechanics; empirical methods; and model analysis such as photoelastic study, and centrifugal study are the common approaches to the investigation of the effect of mine openings on underground stresses.

A specific example of the application of the standardized procedure for the design of room and pillar mine workings was given.

A gypsum deposit at Shoals, Indiana was chosen for this purpose. The results are as follows:

The pre-existing earth stresses were estimated to be: (1) vertical initial stress (σ_y) = 429 psi, and (2) horizontal initial stress (σ_x) = 107 psi.

The pertinent physical properties were determined (as a result of more than 300 individual tests) to be: (1) average ultimate compressive strength = 2820 psi (range from 2,000 to 3,500 psi); (2) average ultimate tensile strength = 399 psi; (3) average compressive modulus of elasticity = 0.703×10^6 psi; (4) Poisson's ratio = 0.2; (5) modulus of rupture = 1199 psi; and (6) specific gravity = 2.3.

The centrifugal model tests for gypsum beams (eight inches in span, two inches in width, with a various thicknesses) resulted in the range of speed at failure from a 250 to 560 R.P.M. Several different patterns of failure occurred.

Safe roof span was determined by use of the centrifugal data and checked theoretically. Pillar size was determined by the use of experimental data. The analysis resulted in a room-and-pillar mining plan with a 35 foot roof span and 64 feet square pillars (see page 4.35); and a low percentage of recovery (58 percent), this was due to the relatively low tensile and compressive strength of the gypsum rock being mined.

Conclusions

A number of basic conclusions may be drawn from this study of the design of stable underground mine openings. These conceptions are based mainly upon the considerable amount of literature reviewed and interpretation of the experimental data obtained.

(1) The standardized procedure approaching the design of stable underground mine openings in three major steps, i.e., the estimation of pre-existing earth's stress, the performance of pertinent physical property tests, and centrifugal model analysis augmented with theoretical calculation,offers the only possibility at the present time for determination of critical dimensions prior to the mining operations. Such a procedure might result in change in choice of a projected mining method without the costly consequences sometimes associated with change at a later stage. Therefore, the use of this method is recommended for use by those investigators who can profit by use of a rapid, inexpensive preliminary design method.

(2) Due to the uncertainty of the real condition of initial earth stress at any unexcavated location, the assumption and estimation of pre-existing earth stress should be handled with great care. Data from the Snowy mountain underground hydroelectric power station in

Australia (50) was determined from experimental data to be more twice as great as the horizontal stress which was estimated. However, such an error was due to the peculiar mountain building forces in the region. The estimation of the pre-existing earth stress in the gypsum mine chosen as an example was also on a theoretical basis; some error may be expected.

(3) The strength of a rock is partially an indication of its ability to store elastic energy which may serve as a potential source of rock burst. In the particular gypsum deposit used as an illustrative example, the low value of compressive and tensile strengths of the gypsum rock rule out the possibility of a rock burst.

(4) The result of centrifugal tests on gypsum beams, as shown in the log-log diagram in (Figure 23), indicate a slope of value up to 0.89 which shows the weak nature of the rock, and a higher factor of safety should be used.

(5) Since the percentage of recovery (58 percent), as has been calculated in Chapter IV for the illustrative example represents a relatively low value, Equidimensional room-and-pillar mining methods may not be practical for this particular type of rock at such a great depth. The choice of some other type of mining method should also be given consideration.

(6) Data obtained from centrifugal tests can be directly applied to the prediction of a safe roof-span

provided that the thickness of the roof stratum is known. The occurrence of failures and their patterns can also be observed if the similarity between prototype and model is exact. It is especially suitable for the dimensional determinations for room-and-pillar mining methods. The method is not new in this field; Professor Bucky first suggested the procedure in 1934. The centrifuge at the Missouri School of Mines is the largest of its kind in the world at the present time. It is capable of testing rock models weighing up to 200 pounds at 2000 "g's". More complex problems concerned with the stability of underground mine openings may be solved by means of the utilization of this big centrifuge.

Recommendations

1. There is no doubt that the data obtained from rock specimens in laboratory are somewhat different from the real characteristics of the rock in place. This is mainly due to the physical and geological heterogeneous nature of rock and the effect blasting, cutting, and natural stress release has in changing rock characteristics. In addition this may be partially due to the complex pattern of external loads and stress distribution. For these reasons, a considerable amount of deviation between laboratory and field measurements is to be expected. A practical and low cost method of determining physical properties in place underground is necessary before any refinement in precision of the mine structural design problem is feasible.

2. Investigations of strength, stress distribution, and modes of failures of concrete under various conditions and the effect of various aggregates, etc., by many civil engineers provide excellent information which may be comparable to the results of physical property tests of mine rock. Concrete itself is also a kind of aggregate of minerals; the phenomena and mechanisms of its structural behavior should be similar and may be applicable to rock tests.

3. Additional foreign materials in rock may change the physical properties of rock, for example, change on moisture content in rock may slightly change some physical properties of the rock such as rock strength. An attempt has been made to test gypsum specimens after saturation with water. The volume of the specimen increases although the surface layer of the specimen has been disolved by the water. No extensive data was obtained, since the observations were made in connection with specific gravity tests, where this factor was merely an annoying occurrence. Subsequent tests were performed dry to avoid this complication. Further study of this phenomena should prove interesting.

Cementation in fractured rock is another fertile field for investigation, including the technique of grouting, and the search for high strength bonding materials for specific types of rocks. For instance, hydrostone gave a stronger bonding force than portland cement in the cementation tests performed with fractured gypsum. This information was found from the investigation of cementation carried out in this study.

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Cementation Tests

Cementation, as a means of strengthening rock around an underground opening has been discussed for a long period of time. Much experimental work has been carried out. But, the problem of selecting materials with high bonding forces to reinforce the tensile strength of rock through cementation of the fractures in the rock seems to be a most difficult task.

The method of applying the principles of cementation to practical underground problems is in the form of grouting. The process of cementation is the same as in foundation grouting processes used in the field of civil engineering. Different pumping pressures are required under different conditions.

There are many kinds of materials which have been tested as bonding reagents for the purpose of obtaining a higher bonded tensile strength of the fractured zone of rock by investigators, some of which are:

Portland cement (various types)
Portland cement and Berylex
Portland cement and calcium chlorite
Portland cement and embecco
Portland cement and silex
Cement mortar
Sodium silicate
Weld crete
Polyester
Epoxy

Hydrostone

The use of chemicals as a cementing material is a most satisfactory procedure, however, some chemicals such as epoxy's are too expensive to use for a full scale mining operation. Cementation can be performed to best advantage where the underground openings are in a highly fractured zone or under a lossened roof. The degree of fracture and the permeability of the rock are a most important factor affecting grouting in practice. Lowering the permeability and degree of fracture will lower the effectiveness of the grout. In many cases, cementation has been used to reinforce the stability of a roof which has been partly bolted. Compression tests are necessary if the fractures occur in pillars, and tensile tests if the cementation is applied to the roof if the optimum cementing agent is to be determined.

Current and past information concerning cementation and the process in grouting indicate a promising future.

Some tests were performed by the author to determine the effect of cementation on the tensile and compressive strength of gypsum cores by using the broken gypsum cores from previous tests. The cementing materials which were tried were portland cement, hydrostone and polyester.

<u>Conclusions on Cementation Tests</u>

1. From the experimental data obtained, hydrostone shows a comparatively high bonding force which is approximately the same as the original strength of the rock. It is larger by four times than that of portland cement. Polyester shows quite promising data as well.

2. From the observations of the experimental procedures, the water-cement ratio has some effect on the bonding force. If the water content is insufficient, there will be no effective cementation. If the water content is too high, the bonding force will be very weak. The best water-cement ratio appears to be 9:2.5 (volumetric) approximately.

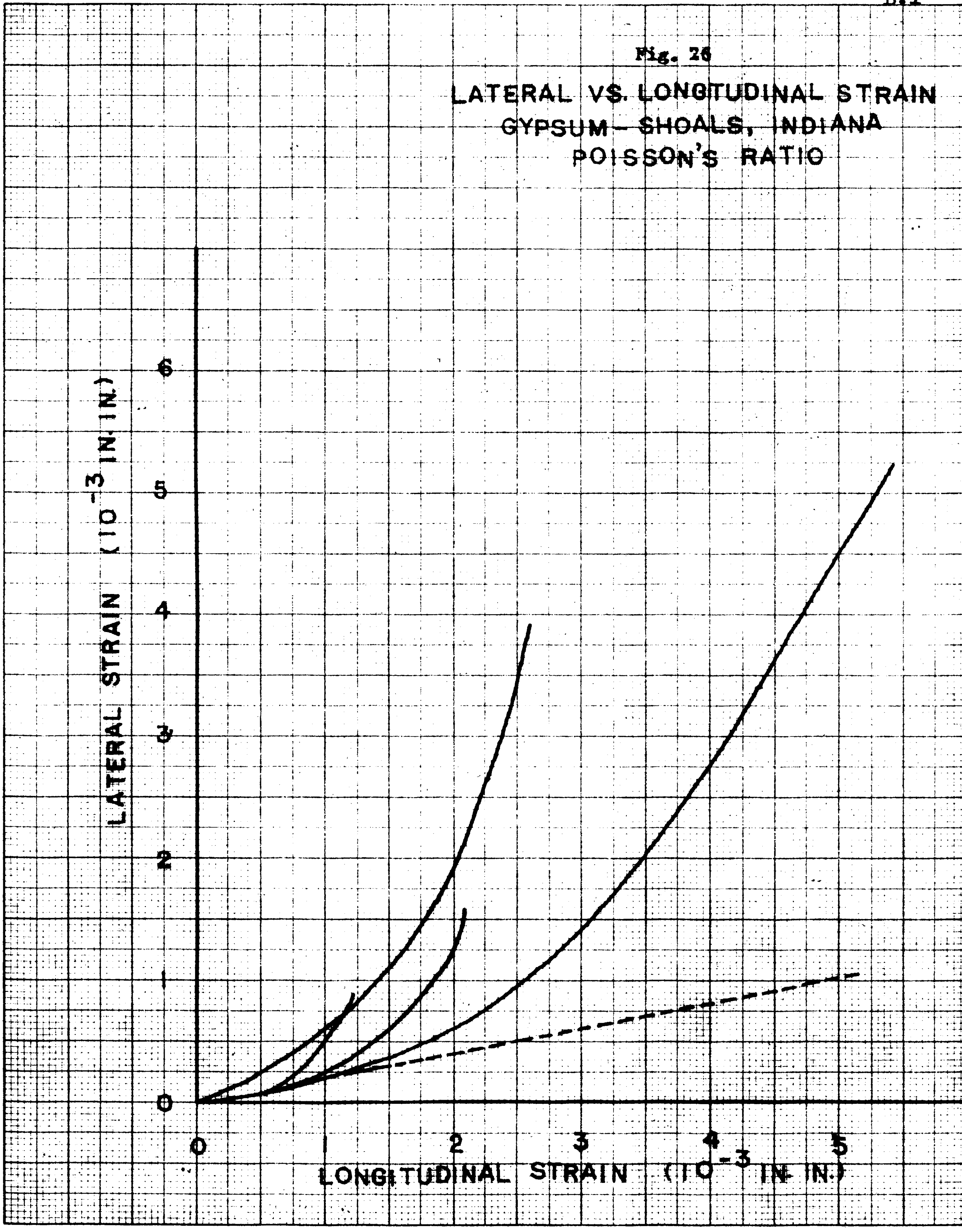
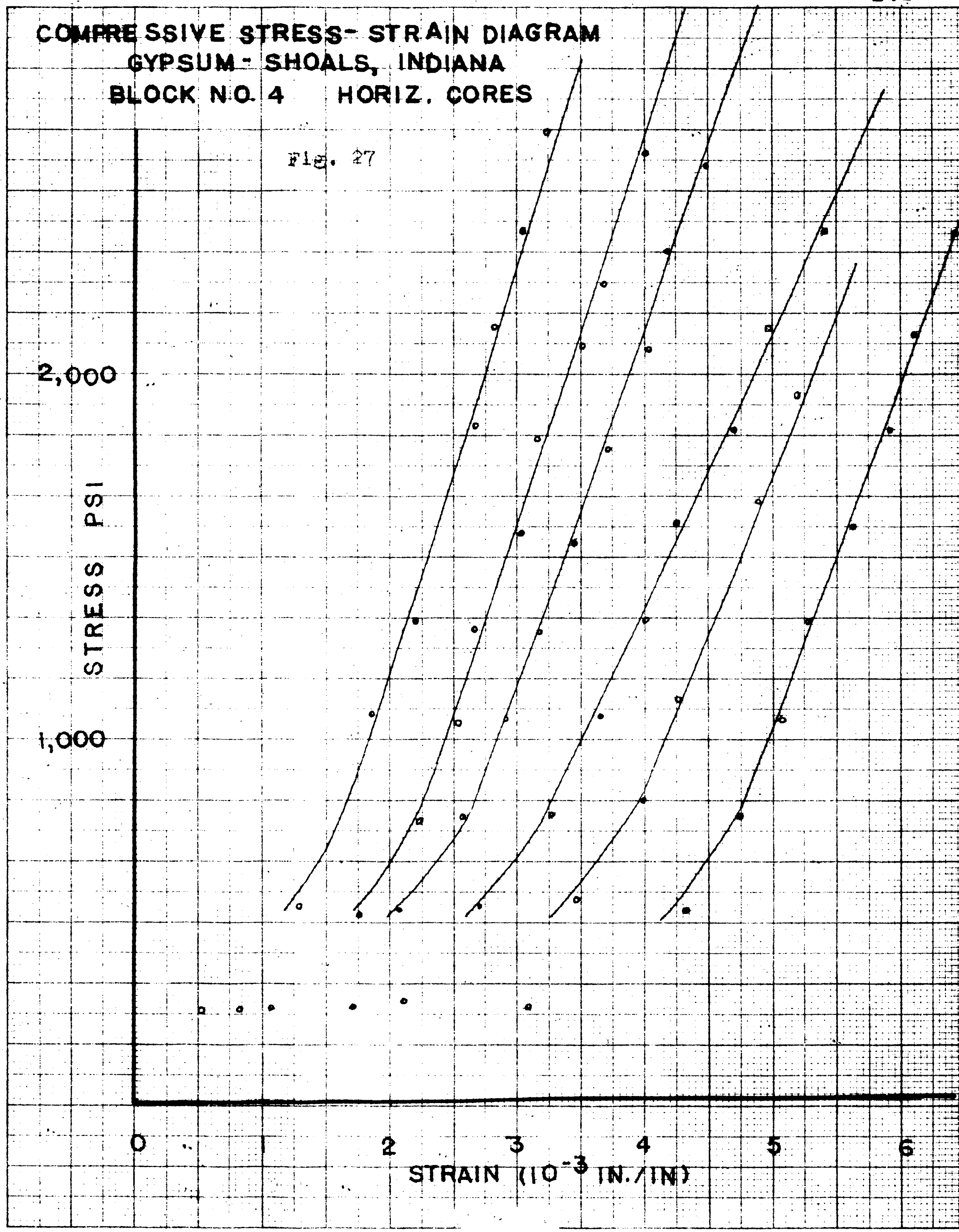


Fig. 26
LATERAL VS. LONGTUDINAL STRAIN
GYPSUM - SHOALS, INDIANA
POISSON'S RATIO
LATERAL STRAIN (10^{-3} IN. IN.)
0
1
2
3
4
5
6
0
2
3
4
5
LONGITUDINAL STRAIN (10^{-3} IN. IN.)


COMPRESSIVE STRESS- STRAIN DIAGRAM
GYPSUM - SHOALS, INDIANA
BLOCK NO. 4 HORIZ. CORES
Fig. 27
2,000
1,000
STRESS PSI
0
1
2
3
4
5
6
STRAIN (10^{-3} IN./IN)

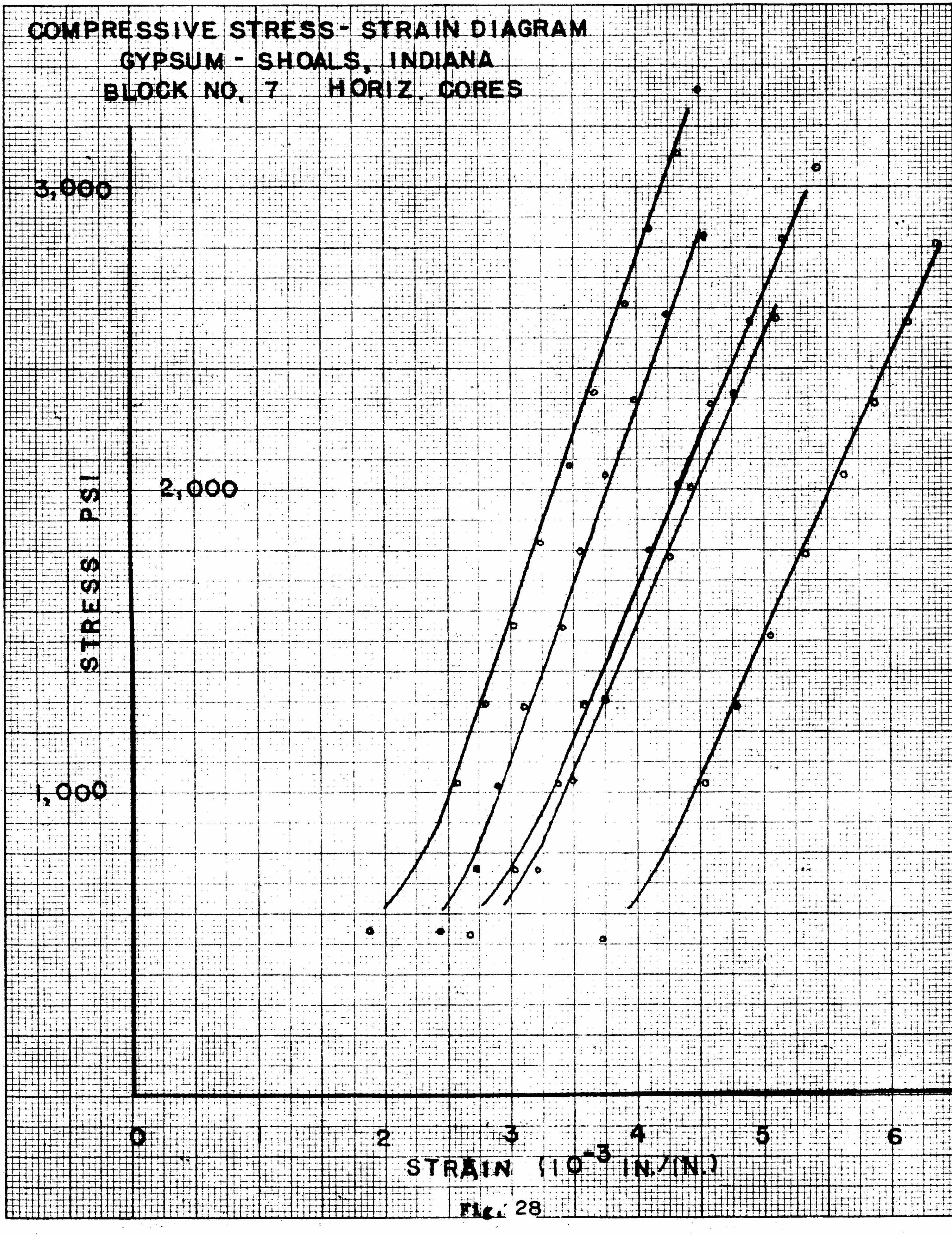

COMPRESSIVE STRESS - STRAIN DIAGRAM
GYPSUM - SHOALS, INDIANA
BLOCK NO. 7 HORIZ. CORES
STRESS PSI
3,000
2,000
1,000
0
2
3
4
5
6
STRAIN (10^{-3} IN./IN.)

Fig. 28

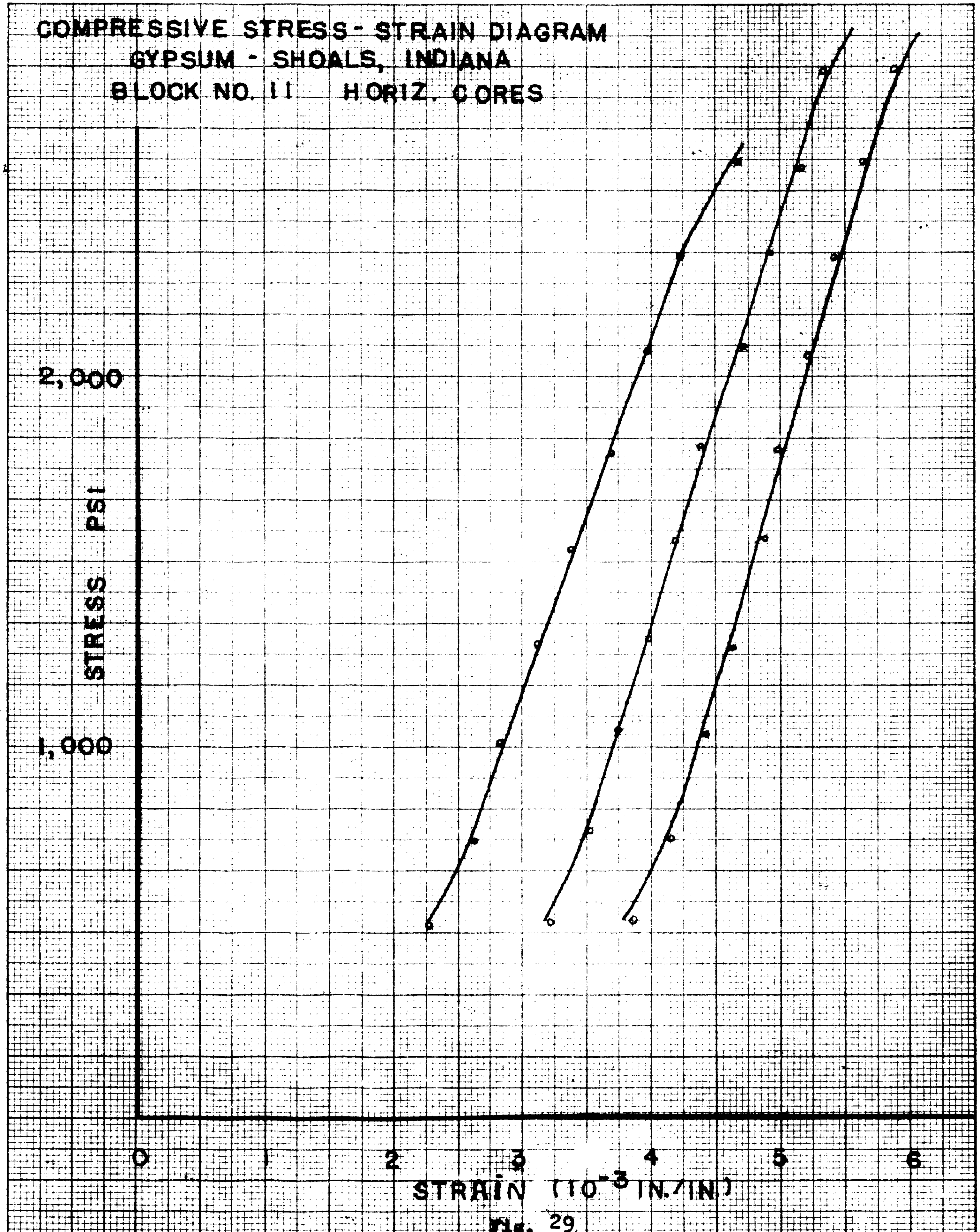

COMPRESSIVE STRESS - STRAIN DIAGRAM
GYPSUM - SHOALS, INDIANA
BLOCK NO. 11 HORIZ. CORES
2,000
1,000
STRESS PSI
0
1
2
3
4
5
6
STRAIN (10^{-3} IN./IN.)

Fig. 29

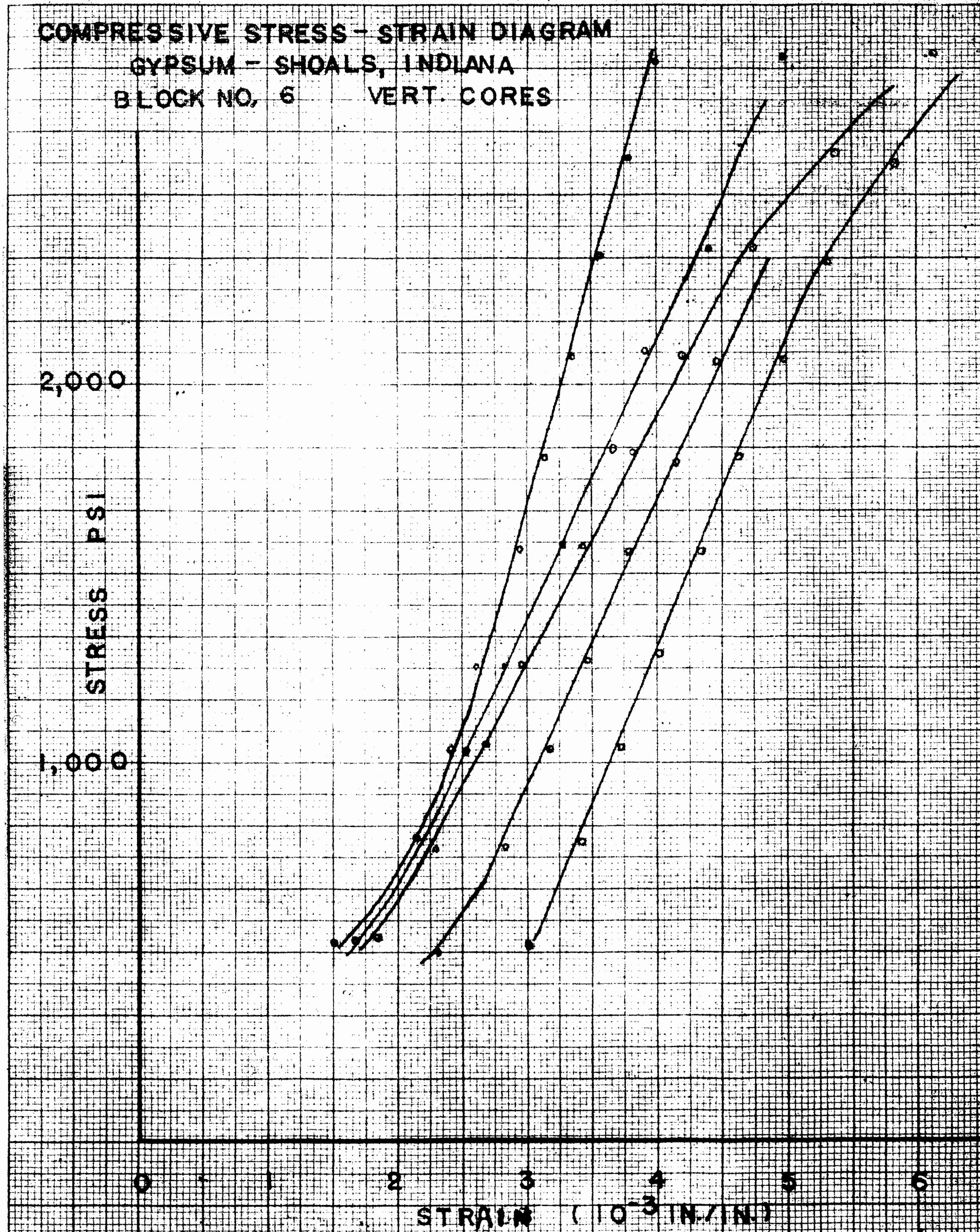
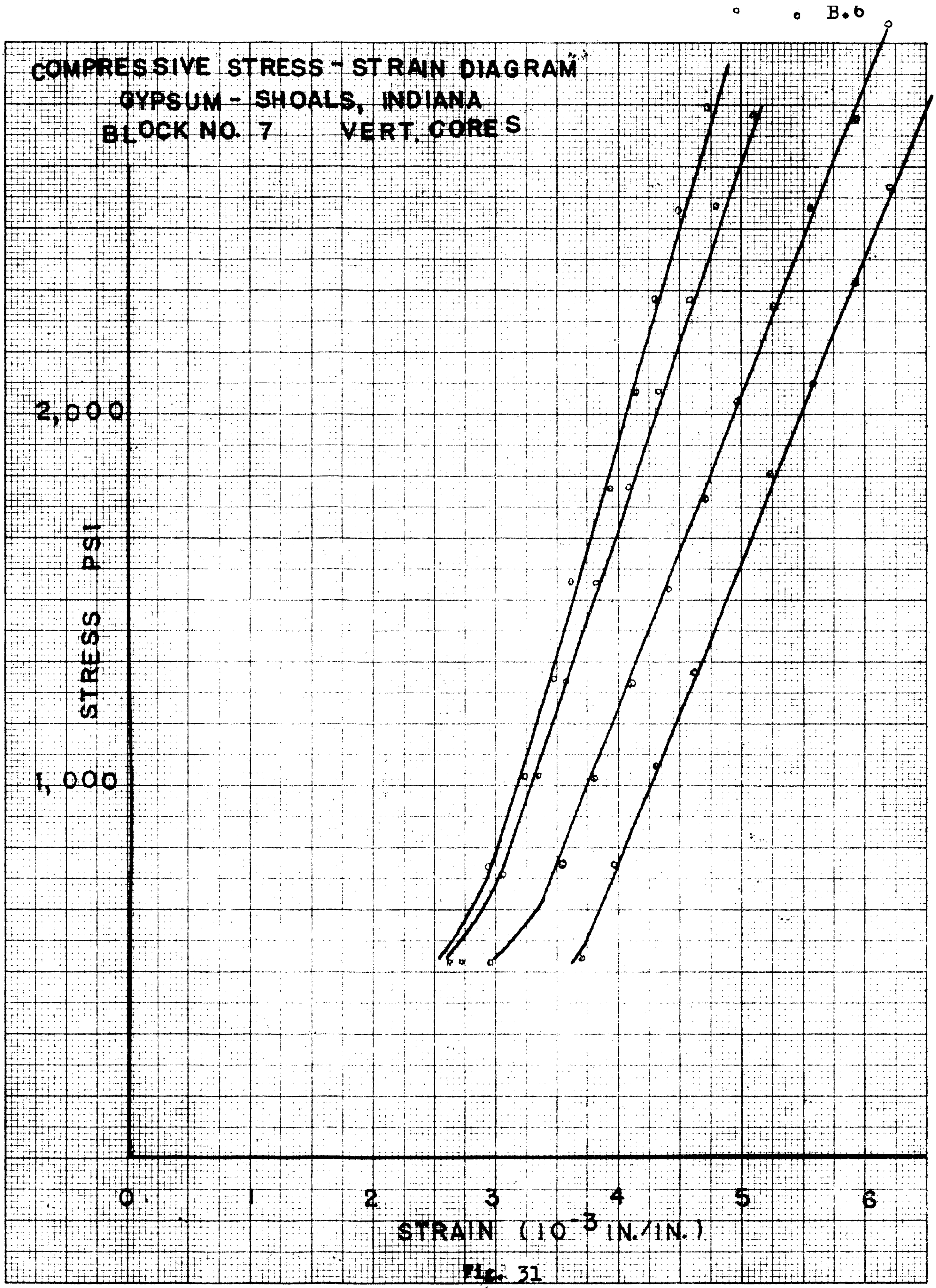

COMPRESSIVE STRESS - STRAIN DIAGRAM
GYPSUM - SHOALS, INDIANA
BLOCK NO. 6 VERT. CORES
2,000
1,000
STRESS PSI
0
1
2
3
4
5
6
STRAIN (10^{-3} IN./IN.)

Fig. 30



B.6
COMPRESSIVE STRESS - STRAIN DIAGRAM
GYPSUM - SHOALS, INDIANA
BLOCK NO. 7 VERT. CORES
2,000
1,000
STRESS PSI
0
1
2
3
4
5
6
STRAIN (10^{-3} IN./IN.)

Fig. 31

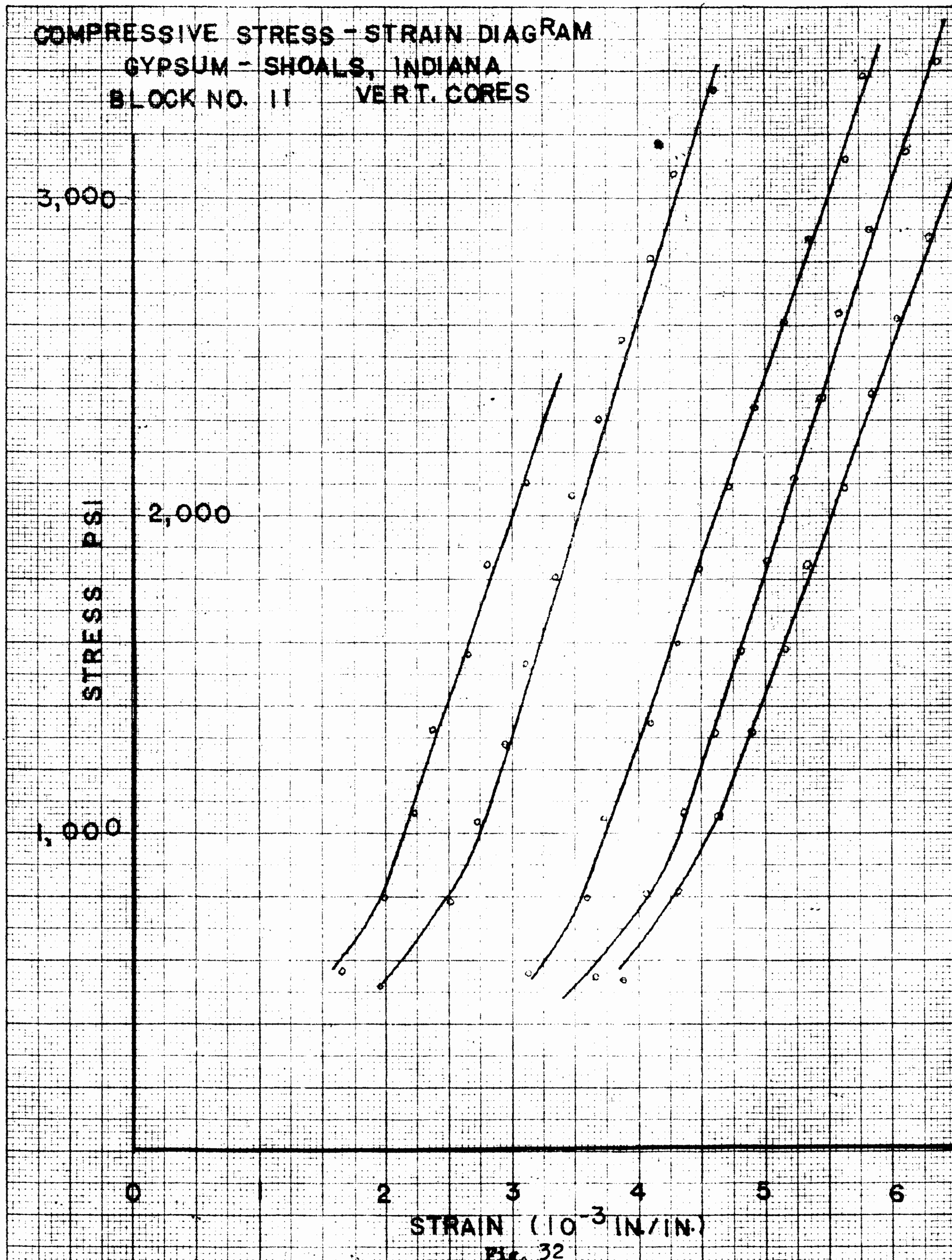

COMPRESSIVE STRESS - STRAIN DIAGRAM
GYPSUM - SHOALS, INDIANA
BLOCK NO. 11 VERT. CORES
3,000
2,000
1,000
STRESS PSI
0
1
2
3
4
5
6
STRAIN (10^{-3} IN/IN)

Fig. 32

VITA

Samuel Shu Mou Chan, the eldest son of General and Mrs. Chinan Chan, was born on June 11, 1934, at Hainan Island, China.

His college education began at the Taiwan College of Engineering, Taiwan, China, in September, 1953. He received a B.S. degree in Mining and Metallurgical Engineering in June, 1957, from the same college, whose name has since been changed to Taiwan Provincial Cheng Kung University.

He came to the United States and enrolled for graduate study in the Department of Mining Engineering at the Missouri School of Mines and Metallurgy in February, 1958. He became a Ph. D. candidate with a major in geology in September, 1959.

He is a student member of the Chinese Institute of Mining and Metallurgical Engineers; American Institute of Mining, Metallurgical and Petroleum Engineers; Society of Sigma Xi; Geological Society of America; Mineralogical Society of America; and Geochemical Society of America.