SOME FACTORS IN THE USE OF
ROOF BOLTS FOR MINE ROOF CONTROL

by

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I

INTRODUCTION

Before the advent of the present day man equilibrium conditions had been established in many phases of nature. One of the phases which had reached this equilibrium condition was the coal and mineral deposits. As they stand, the stresses acting on coal and mineral deposits are in equilibrium, - any underground mining operation upsets this equilibrium. Shocks due to the unbalancing of stresses oftentime cause catastrophic roof-falls which kill and injure miners and damage equipment. Attempts are made to control these unbalanced stresses by choosing the proper method of mining and suitable method of roof support.

About 20 years ago, the St. Joseph Lead Company in Missouri, in an attempt to get away from the awkward, bulky and expensive method of supporting roof by timber or steel and concrete, tried a new method of roof support.(17) The new method was named "roof-bolting". As the name implies, the lower exposed strata of the roof are bolted to the stronger less exposed strata above the mine opening. This new method has been used more and more in the past three years. At the present time over 200 companies are using this method of roof support.(16)

The roof bolting method of roof support has presented many problems which have to be solved before definite procedure for its use can be established. Some of these problems are:

1. How long should the bolt be.

2. At what angle should the bolt be inserted.

3. What should be the longitudinal and transverse center-to-center distance between the bolts.

4. What stress should be applied to the bolt when installed.

5. How often should the bolt be retightened.

This thesis will be an attempt to solve some of these problems.

II

THE REVIEW OF LITERATURE

Originally it was thought that the only force acting on underground rock was a vertical one.(11) Within the past century it has been proven that the forces are acting in all directions. For this discussion the forces will be resolved into their vertical and horizontal components.

The magnitude of all the forces are proportional to the depth and to the density of the rock. In addition to these, the magnitude of the horizontal components are affected by Poisson's ratio which is not a fixed value for a particular type of rock but is a function of the depth.(5) (11) (See Appendix 1) When an entry is opened force equilibrium is upset and the stress concentrations build up. The magnitude of the stress concentration is affected by the cross-sectional shape of the entry. The rectangular cross-section, the usual shape of entries, causes the largest stress concentrations — not only do stresses build up in the side walls but they build up exceedingly high in the corners. The magnitude of the maximum stress does not depend on the cross-sectional area of the opening but it is a function of the relationship between the width and the height of the opening.(11)

When an entry is cut, vertical stresses build up to a maximum value directly above the side walls. From this maximum the stresses decrease in a direction normal to the entry until a point is reached where the vertical stress is equal to the equilibrium value (see Fig. 1).

Prior to the opening of an entry the horizontal forces are compressive forces. As soon as the entry is opened, the horizontal forces immediately above and below the entry change to tensile forces.

The locus of the points where the horizontal forces change from tension to compression is a vertical ellipse proscribed about the cross-section of the entry, with its minor axis in the direction of the horizontal center line of the cross-section* (see Fig. 2, and Appendix II).

Rock is weak in tension and the exposed horizontal layer soon develops cracks along the cleavage planes. These cracks progress upward through the horizontal layers of the rock to the point where the tensile force changes to a compressive force (the top of the ellipse). After cracking, the only support the cracked layers have is the bend with the adjacent layer. As the cracks progress through successive layers, the stress on the bonds increases until a point is reached when the bond is no longer able to support the weight of hanging rock. At this point the mass of rock in the center of

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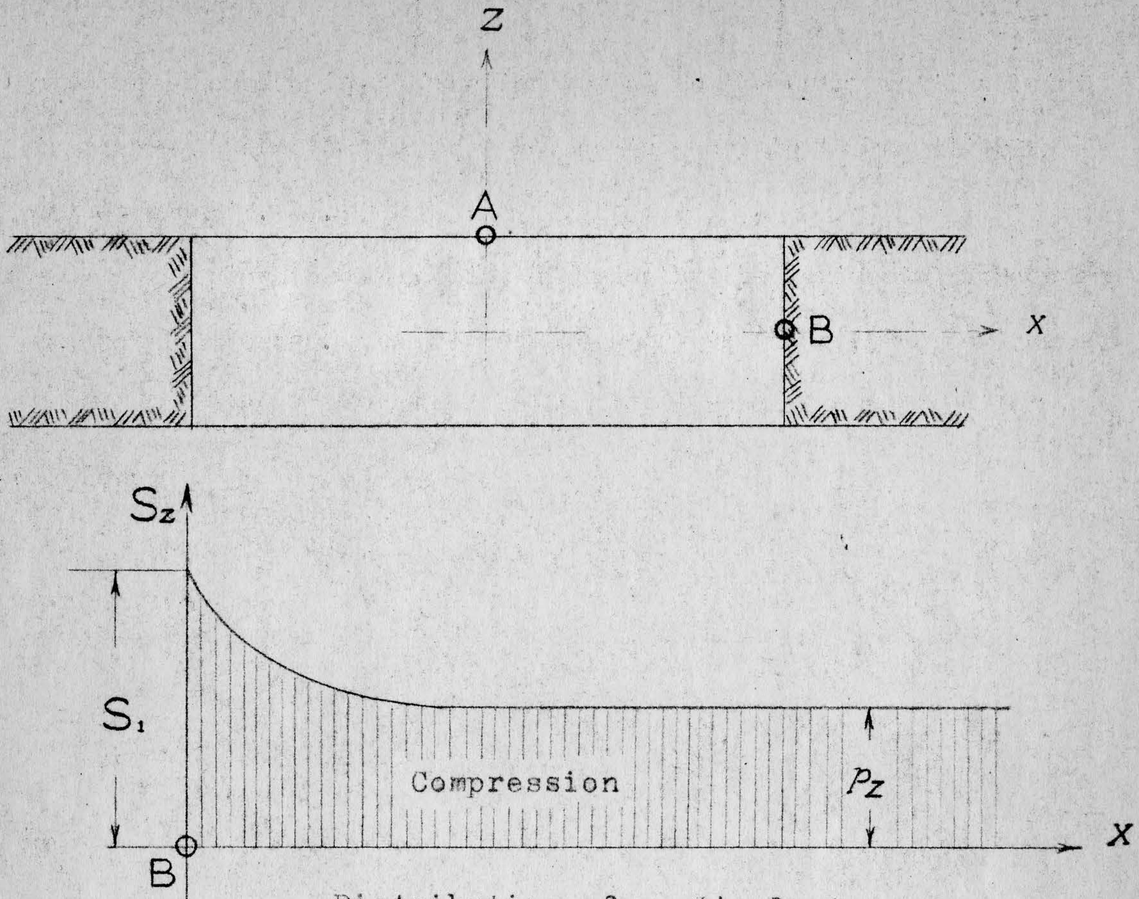
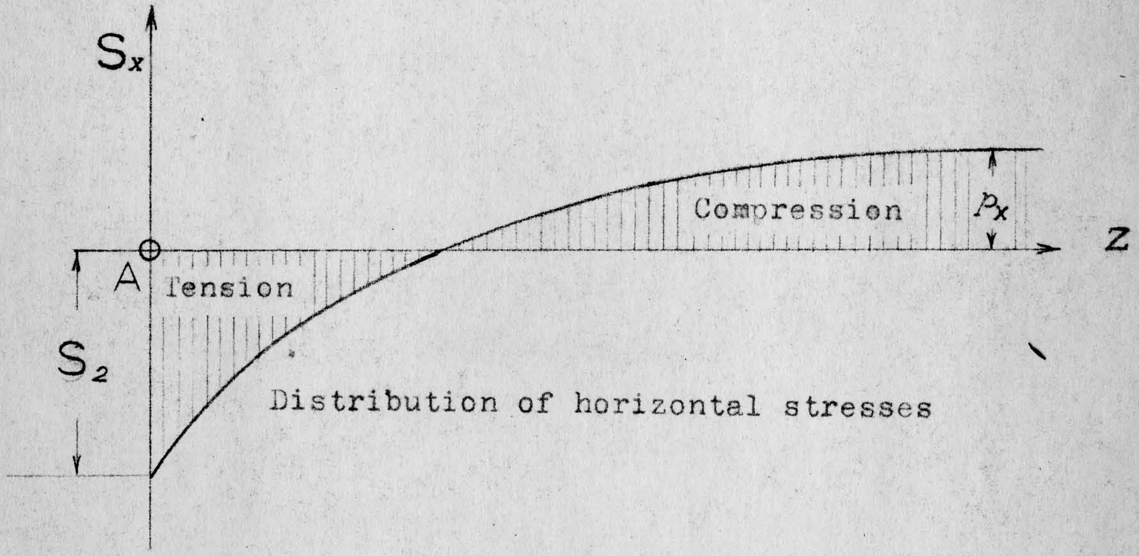
* When an entry is opened in the vicinity of old mining the shape of the ellipse may be distorted.



Distribution of vertical stresses



Distribution of horizontal stresses

Fig. I. Stress distribution in a mine opening.

the roof falls. If the cracks have not progressed to the point of zero tension the above process is repeated over and over again until the top of the ellipse has been reached (see Fig. 3).

For many years roof control was accomplished by timbering. The timbering method of roof control incorporates beams across the roof of the opening. The roof beams (cross bars) are supported at each end in many ways, all of which are variations of the vertical upright support. The timbering method of roof control, while adequate when properly installed, has many disadvantages in modern mining methods. Some of those disadvantages are:

1. The cross bars are designed to hold the roof in place, not to apply an upward force that preserves the bond between horizontal layers.

2. Moving machinery may knock out some of the supports allowing sections of the roof to fall in.(17)

3. The entry must be cut wide enough to allow room for the end supports. This additional width increases the ratio of the width to the height, thus increasing the magnitude of the horizontal tensile stresses.

4. The end supports and beams offer resistance to the flow of air for ventilation.(8)

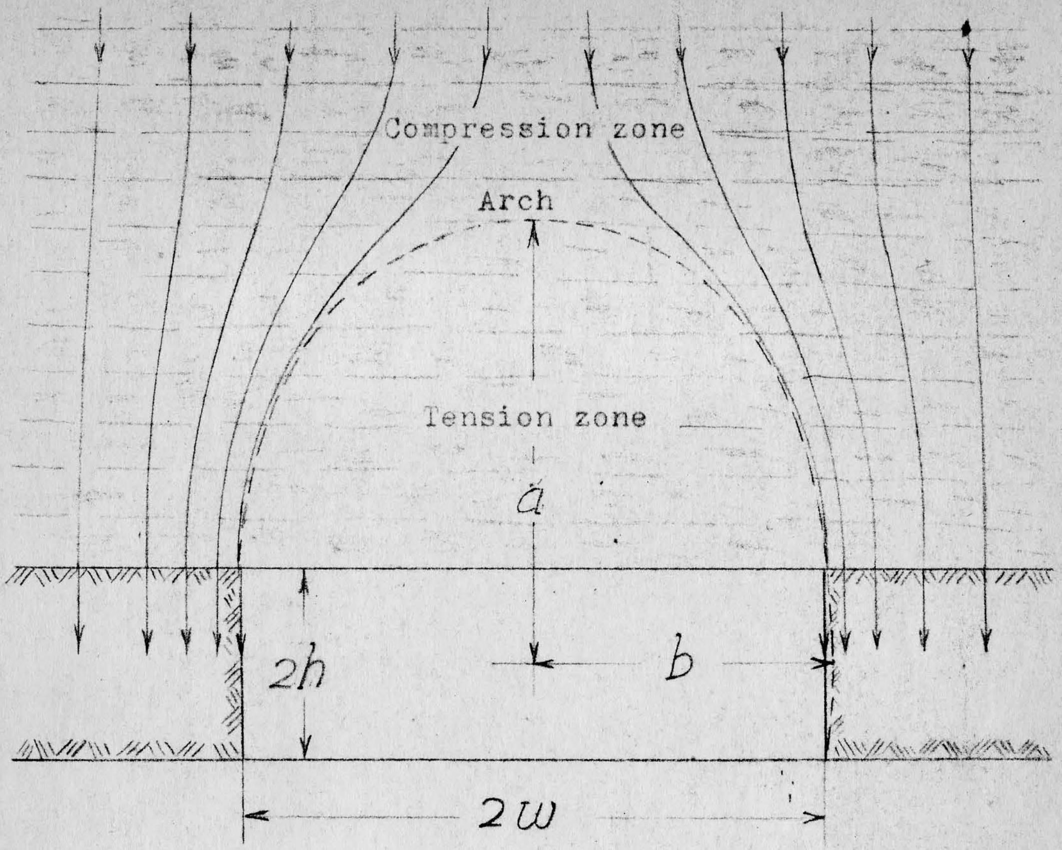


Fig.2. Stress ellipse and compressive stress distribution.

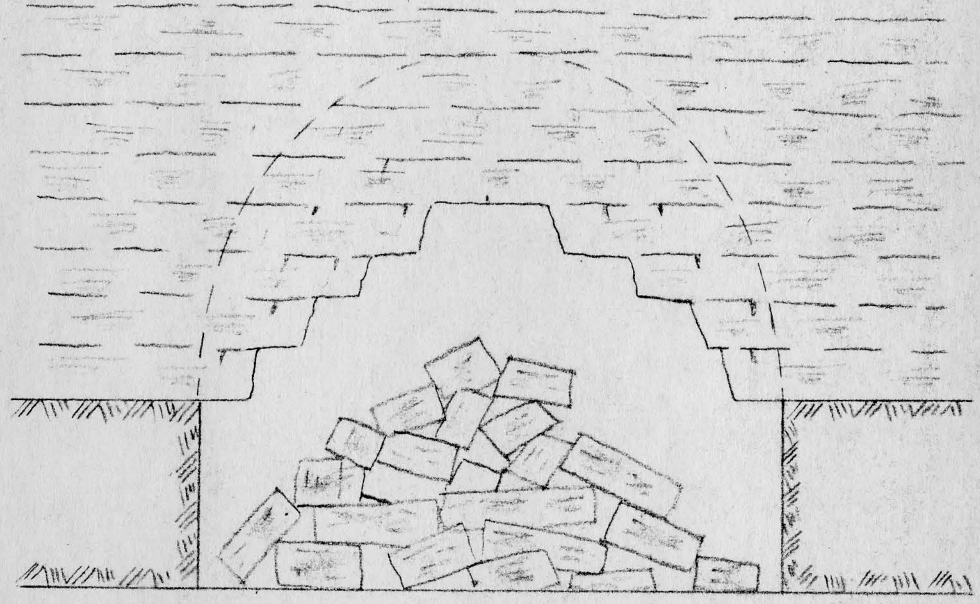


Fig.3. Fallen strata.

5. Mine cars often scrape and knock out end supports whenever the entry bottom buckles upward.

6. Timbers are often damaged or blown out during blasting operations.(17)

7. Miners are sometimes crushed between mining equipment or cars and the end supports.

8. Timbers require a large storage space and are a transportation problem.(17)

To overcome some of the disadvantages of the timbering method of roof control, roof-bolting was developed.

Roof-bolting consists of drilling holes in the roof, inserting and fastening bolts securely in the holes. By tightening a nut against a bearing plate the exposed lower layer of the roof is actually bolted to the unexposed upper layers. The ideal case would be to have bolts long enough to reach above the stress ellipse. This is not always feasible with the center bolts but it can be accomplished at the sides by angling the bolts near the walls (see Fig. 7). For greater support, alternate rows of bolts in the middle section of the roof are angled backward and forward (see Fig. 4).

The bolts used in roof bolting are from two to eight feet in length and have a diameter of three-quarters or one inch.*

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* The Bureau of Mines recommends a one-inch diameter bolt.(16)

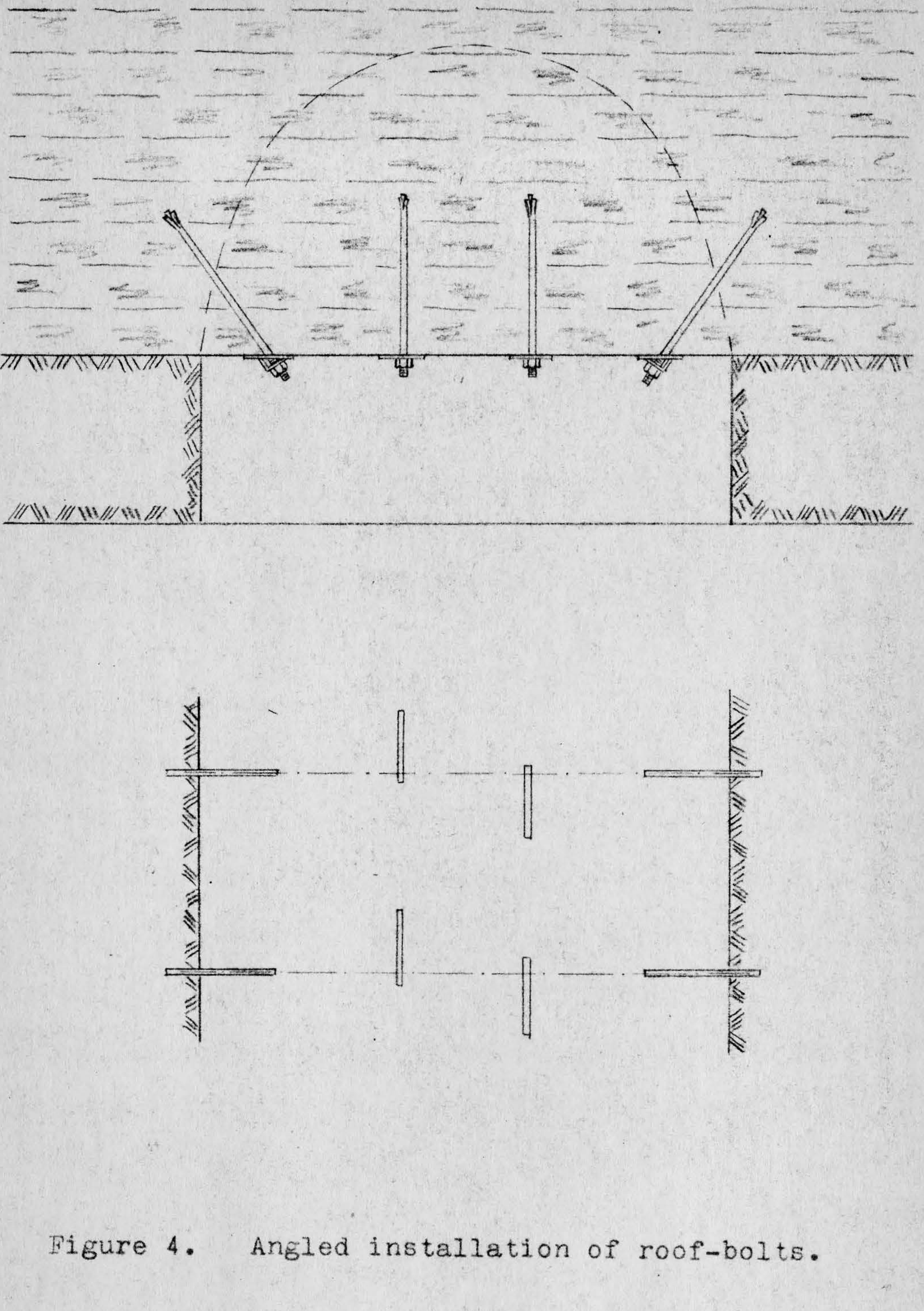
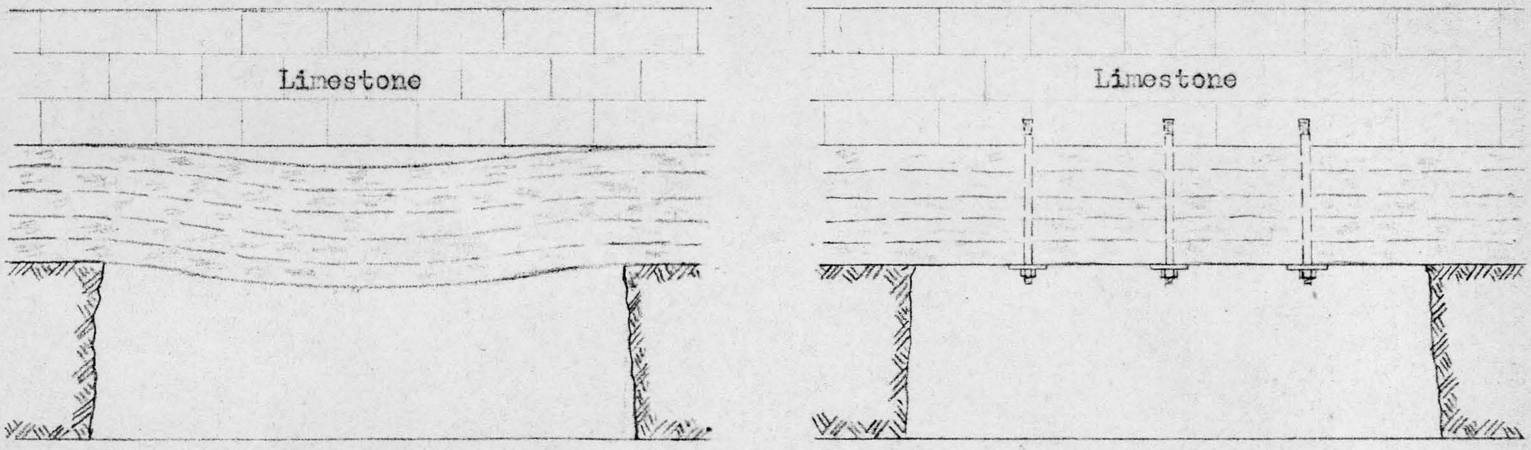


Figure 4. Angled installation of roof-bolts.

Although the bolts are not long enough to reach the stress ellipse they are long enough to reach self-supporting layers of rock(4) (see Fig. 4a). By bolting the layers together a strong beam is formed making the entire roof self-supporting(4) (see Fig. 4b).

The upper end of the bolt is usually anchored by either of two methods, split-rod with wedge or expansion shell. In the split-rod with wedge method, the end of the bolt is split and the wedge is inserted in the split. When the bolt is installed it is forced against the wedge, which seats at the top of the hole. The pressure applied to the bottom of the bolt spreads the top of the bolt and jams it against the side of the hole (see Fig. 5). In the expansion shell method the bolt is screwed into the wedge which spreads a shell as it is drawn toward the bottom (see Fig. 5). Bearing plates are used on the bottom of the bolt to prevent the nut from digging into the exposed strata and to distribute the support over a wider area. The usual bearing plate for vertical bolts has either a flat rectangular shape or a triangular shape with upward curved points(17) (see Fig. 6). Bearing plates for angled bolts have a flat surface to bear against the roof and an angled surface to bear against the nut. In some cases when the exposed layer of the roof is weak, a channel iron beam is used(18) as a continuous bearing plate across the width of the roof (see Fig. 7). This beam receives the greatest tensile stress, thus partially relieving the stress in the rock.



Room with slate unsupported

Room with slate held by rods

Figure 4a . Bolting weak beds to a strong stratum.

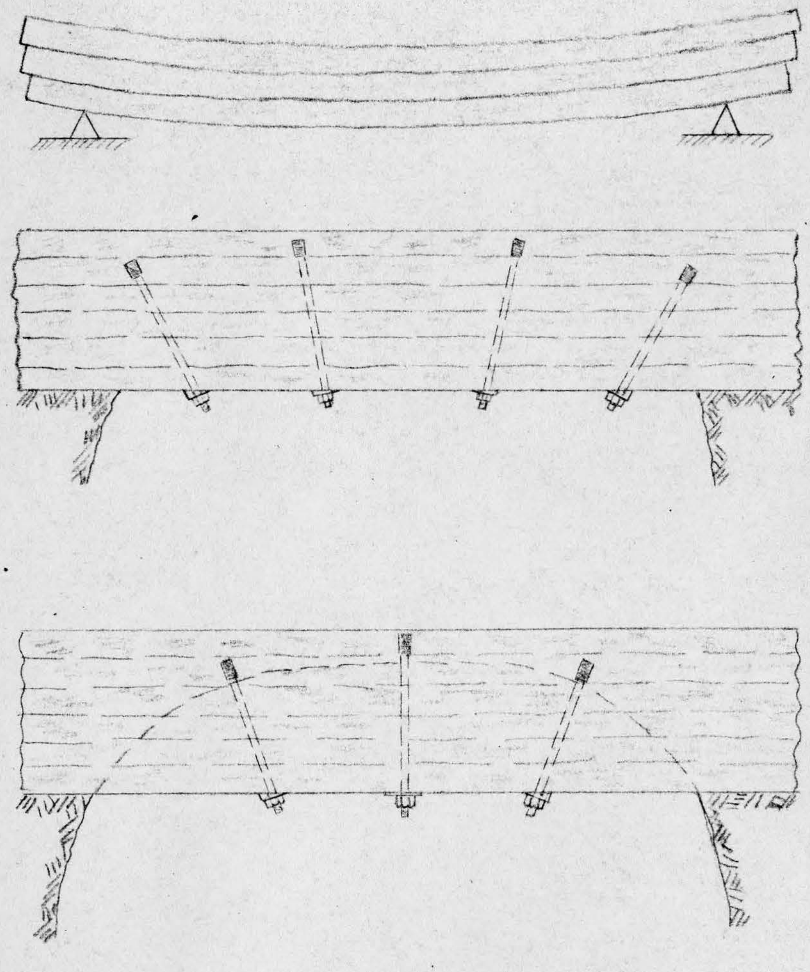


Figure 4b. Theoretical roof action and function of suspension rods.

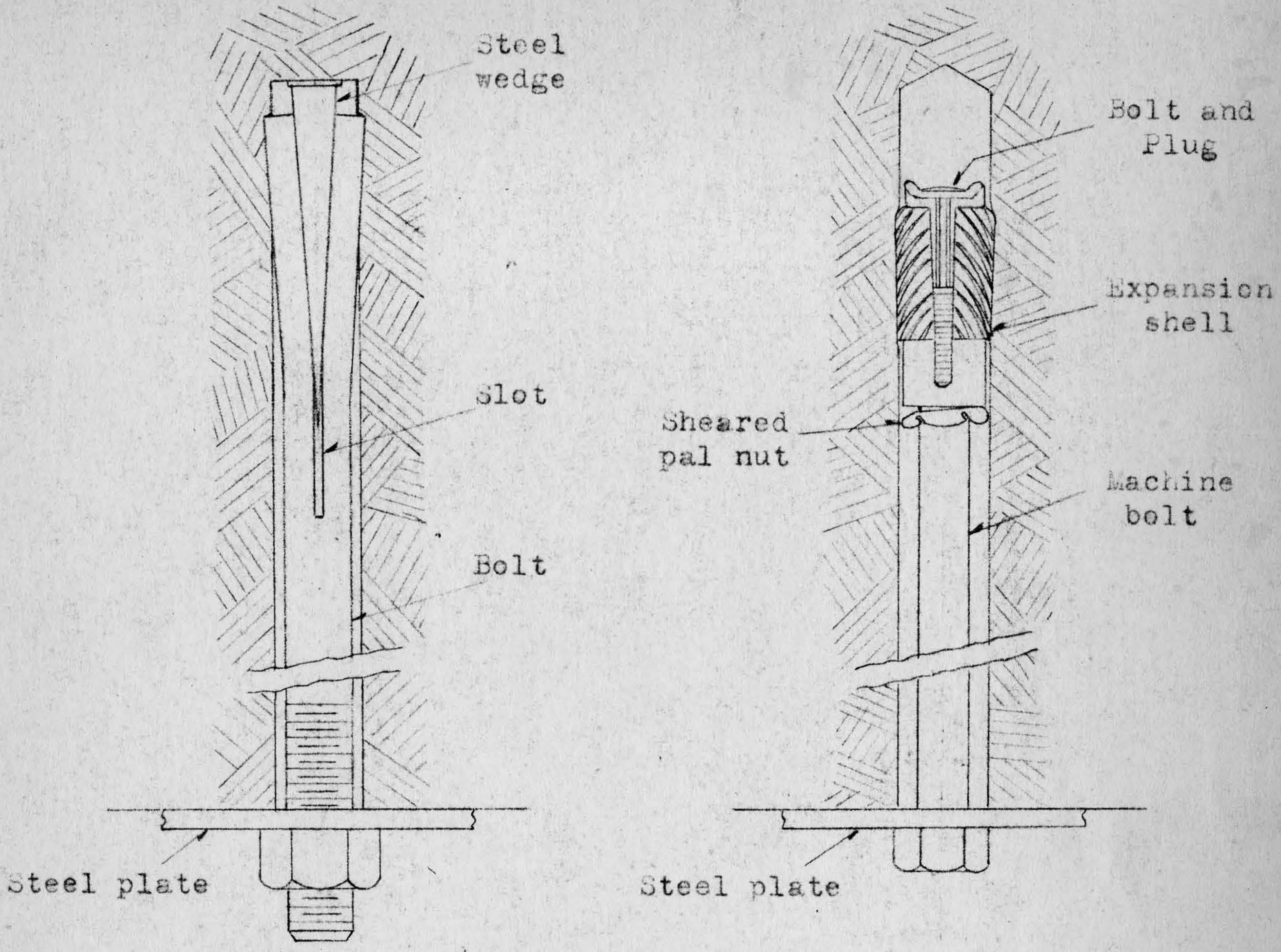


Figure 5. Types of roof-bolts.

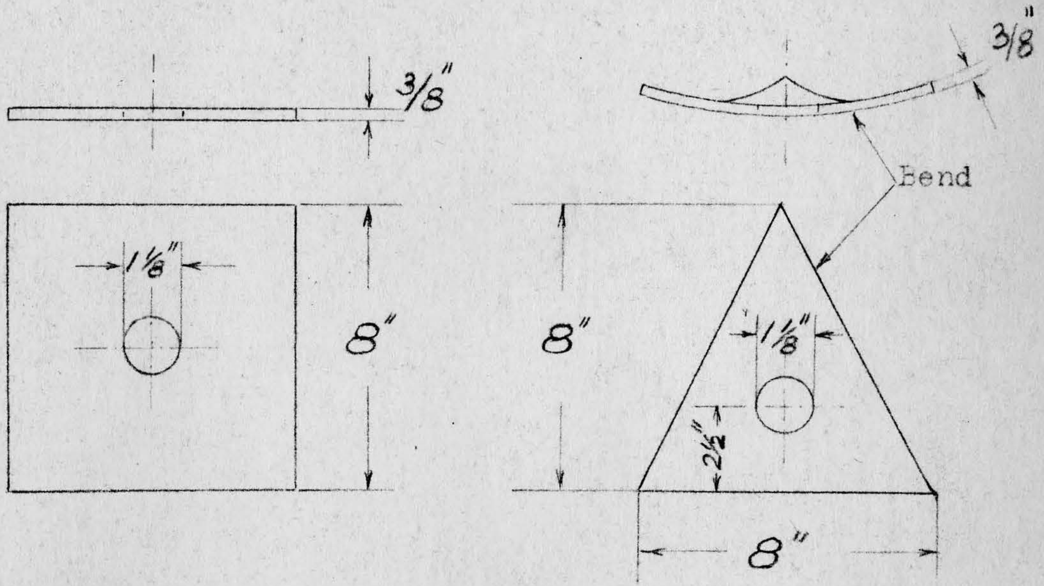


Figure 6. Bearing plates.

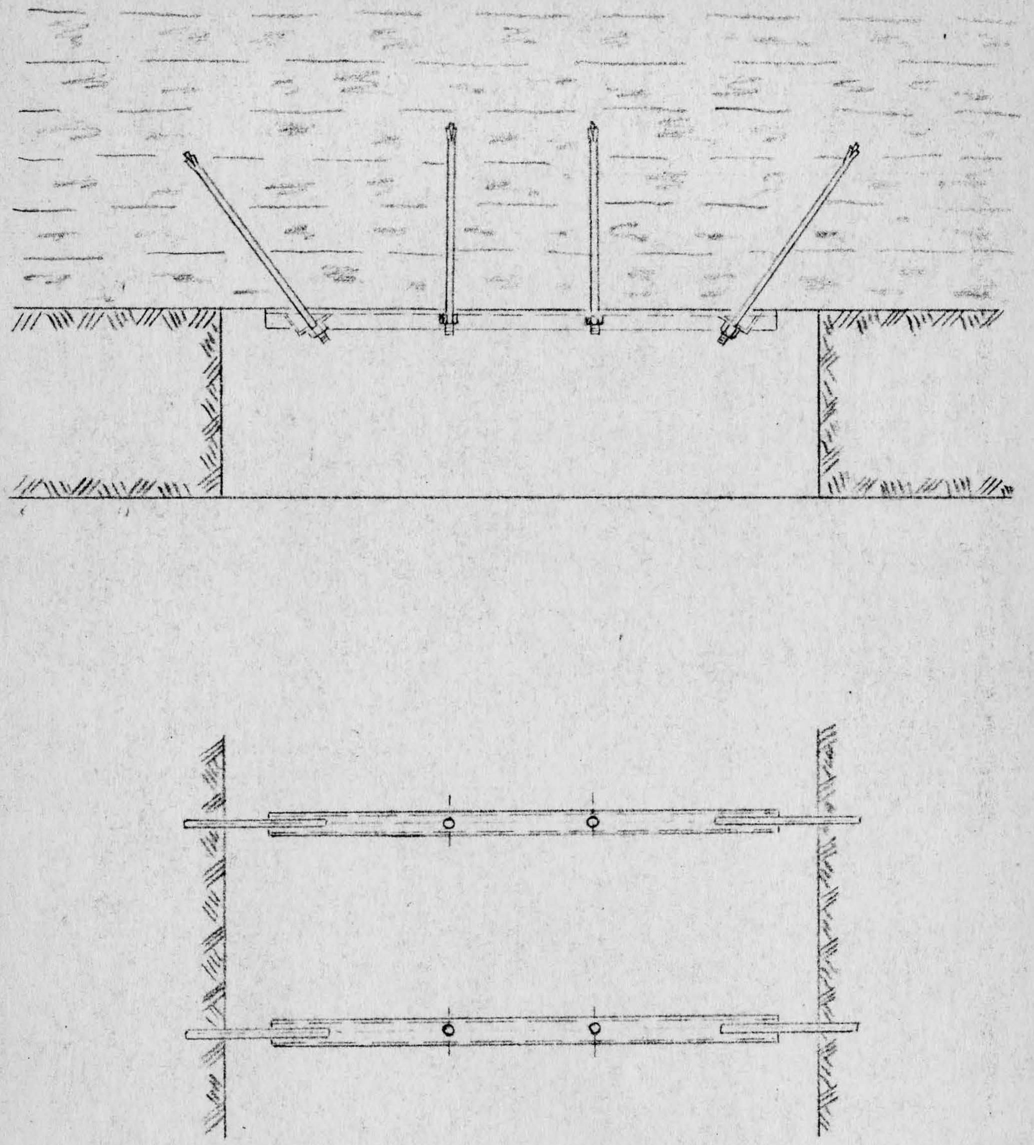


Figure 7. Installation of roof-bolts using a continuous channel iron instead of bearing plates.

The roof bolting method of roof control has overcome many of the disadvantages of the timbering method of roof control. The bolts exert an upward force which helps preserve the bond between the horizontal layers of rock. By doing away with bulky cross-bars and end support, the entry can be narrower, there are no restrictions to the flow of air for ventilation, there are no supports to hinder the movements of machinery and cars, there are no supports to be blown out when blasting, and there is a very small storage space requirement. Although the initial cost of the machinery necessary for installation of roof bolts is high, the final overall cost of the roof bolting method of roof control is cheaper than the timbering method.

III

THE INVESTIGATION

Object of Investigation

A. To determine the stress variations with time of a four foot roof-bolt, installed in a sandstone and shale roof of a recently driven mine entry.

B. To determine the anchor holding strength of sandstone and shale roofs for various depths of anchor insertion.

C. To compare the maximum holding strength of two types of bolts.

D. To get some information on roof deflections with and without roof-bolts support.

Plan of Investigation

A. To determine the stress variations of an installed roof-bolt, ten roof-bolts will be inserted in a sandstone and a shale entry roof and the change in the elongation of the bolts over a six-months period will be measured.

B. To determine the anchor holding strength of sandstone and shale roofs, roof-bolts will be anchored in each type of rock at depths ranging from 12 inches to six inches in two-inch decrements. After anchoring, an increasing downward force will be applied to the bolts until the anchor pulls through the roof or the bolt breaks. The force on the bolt at the breaking point of failure of the roof or bolt will be measured.

C. To determine the maximum holding strength of two types of bolt, three bolts of each type will be anchored in a sandstone and a shale roof. After anchoring, an increasing downward force will be applied to the bolt until the anchor slips or the bolt breaks. The force on the bolt at the point of slip or bolt failure will be measured.

D. To determine the roof deflection in a newly driven entry a 48-inch roof-bolt will be enclosed in a 44-inch hole. An extensometer will be attached to the free end of the bolt with the needle touching the roof. Extensometer dial readings will be observed over a six month period. This procedure will be followed in an entry having the conventional timber type of roof support and in one having the suspension type of roof support.

Apparatus and Material

The apparatus used in the investigation consisted of the following instruments, equipment and material.

Mine roof-bolts. Two types of mine roof-bolts were used. Both types were made of steel and had lengths of 24 and 48 inches, had nominal diameters of one inch, had rolled threads, and required a drilled hole in the roof of 1-1/4 inch diameter. One type, manufactured by the Bethlehem Steel Company, Bethlehem, Pennsylvania, has a slot cut with a torch and a flat-headed wedge (see Fig. 8). The other type, manufactured by Cleveland Cap Screws Company, Cleveland 4, Ohio, has a forged slot and wedge (see Fig. 8).

Roof plates. The roof plates used were flat square plates manufactured by the Bethlehem Steel Company, Bethlehem, Pennsylvania. The plates were eight inches square on a side and three-eighths of an inch thick (see Fig. 8).

Angle washers. The bearing plates used for 60-degree angled bolts were washers manufactured by the Bethlehem Steel Company, Bethlehem, Pennsylvania. The angle washers were 3-1/2 in. x 2-1/2 in. x 3/8 in. and 3 in. long (see Fig. 8).

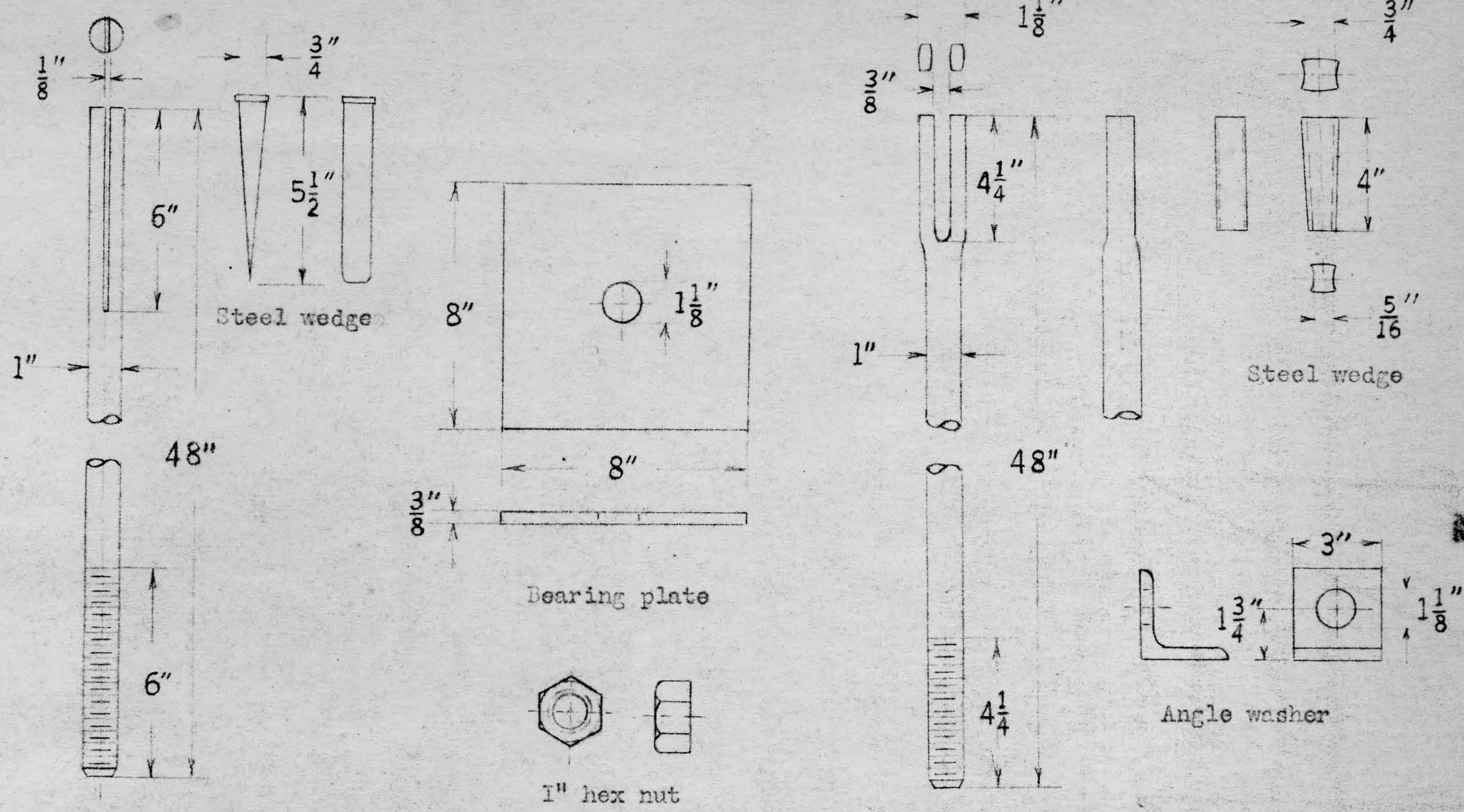
Nuts. The nuts used were heavy hex nuts manufactured by the Bethlehem Steel Company, Bethlehem, Pennsylvania (see Fig. 8).

Roof-bolt pull tester. The roof-bolt pull tester was a model BT-252 Roof Bolt Pull Tester manufactured by the Continental Gin Company, Birmingham 2, Alabama. The pull tester is composed of a hand-operated hydraulic pump and a hydraulic jack connected by a high pressure hydraulic hose. The hydraulic pump has a gage whose dial is calibrated to read the load applied to the bolt in tons (see Fig. 9).

Stoper. The stoper used was a Le Roi Cleveland Roof Bolting Stoper manufactured by the Le Roi Company, Cleveland 11, Ohio. The stoper was especially designed to drill holes in low roofs. It was operated by compressed air at a pressure of 100 pounds per square inch.

Pneumatic impact wrench. The pneumatic impact wrench used to tighten the roof-bolt nuts was a Power Vane Wrench, size 65, manufactured by the Chicago Pneumatic Pool Company, New York 17, New York.

Strain gages. The strain gages used were the electric resistance type AB-3, manufactured by the Baldwin Locomotive Company, Philadelphia 42, Pennsylvania. The AB-3, cupro-nickel wire, electric resistance gage, requires phenol-resin cement for application, is 13/16 inches long, and is covered with bakelite. The bakelite covered gage is moisture and creep resistant and is therefore used for the



Bethlehem split rod roof-bolt

Cleveland forged lug roof-bolt

Figure 8. Roof-bolts and accessories.

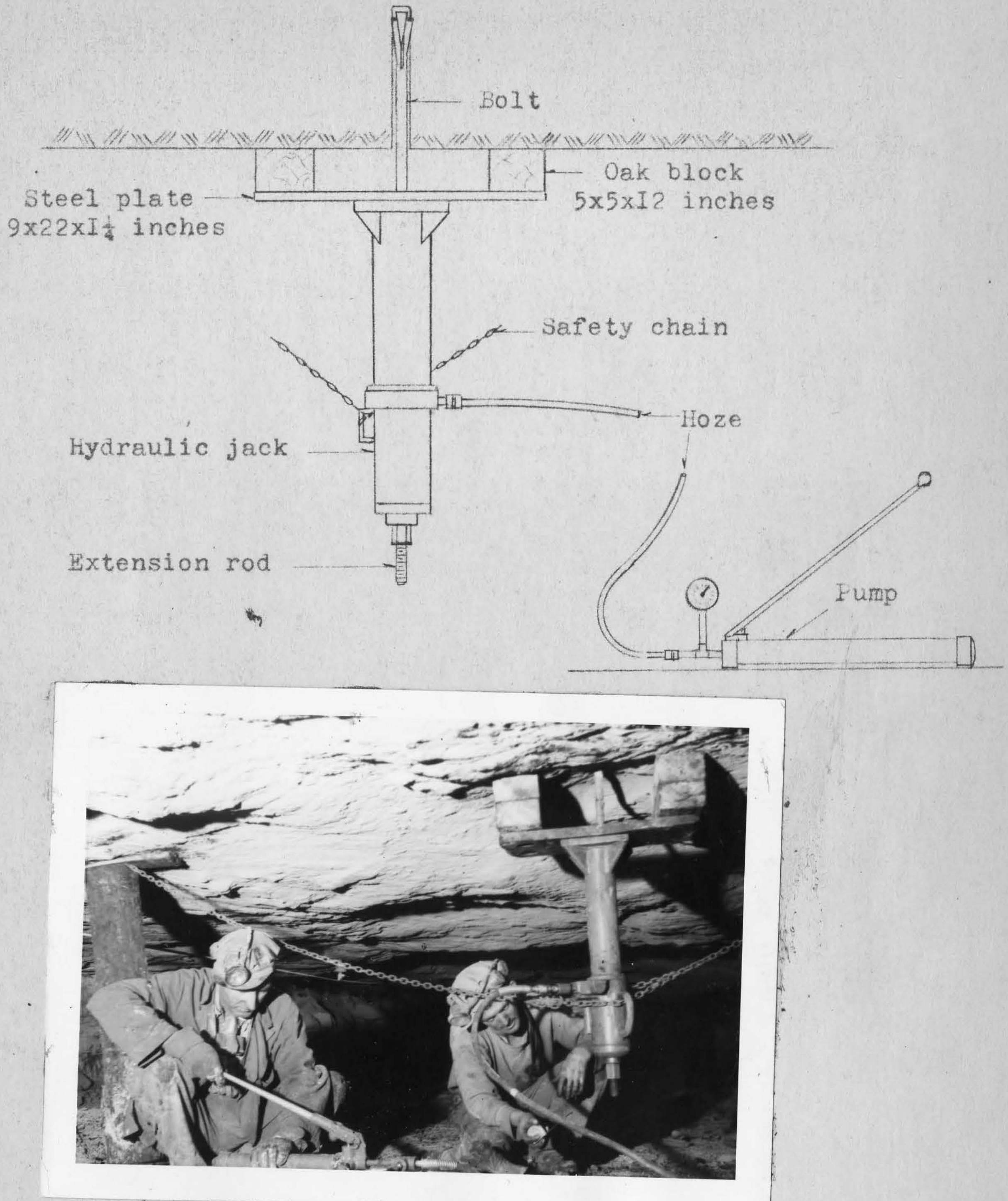


Figure 9. Installation of a Roof Bolt Pull Tester.

test covering a long period of time. The AB-3 type of gage has a resistance of 120 ohms and a gage factor of 2.09 (see Fig. 10 for method of installation).

Strain indicator. The strain indicator used was a model K portable strain indicator made by Baldwin Locomotive Works Company, Philadelphia 42, Pennsylvania. The model K portable strain indicator is housed in a metal box 9 in. x 6 in. x 12 in. and weighs about 25 pounds.

By means of the Wheatstone-Bridge and a calibrated dial the change in resistance of the strain gage wire can be read directly as the change in elongation in micro-inches per inch of length. The range of the model K indicator is 12,000 micro-inches; the dial is marked off in 20 micro-inches per inch increments which can be approximated to plus or minus two micro-inches per inch (see Fig. 11).

Extensometer. The extensometer used in this investigation to measure the sag of the top rock, was an Ames 262 Extensometer manufactured by B. C. Ames Company, Waltham, Massachusetts. The Ames 262 extensometer is accurate to 0.001 inches and has a maximum range of 0.5 inches (see Fig. 12).

Torqometer. The torqometer used in this investigation to find the relationship between the load and the torque exerted on the nut was a Trade TQ-1003 Mark, Serial N. 3355 L, made by Snap On Tools

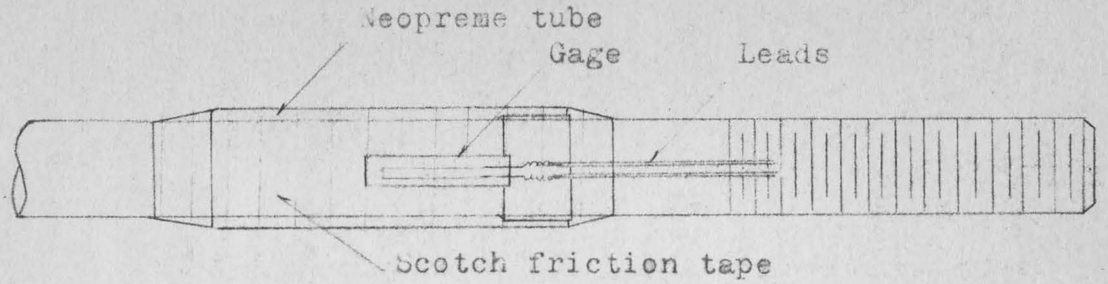


Figure 10. Method of installation of SR-4, type AB-3 strain gage.

Figure 11. Taking a reading with the SR-4 strain indicator.

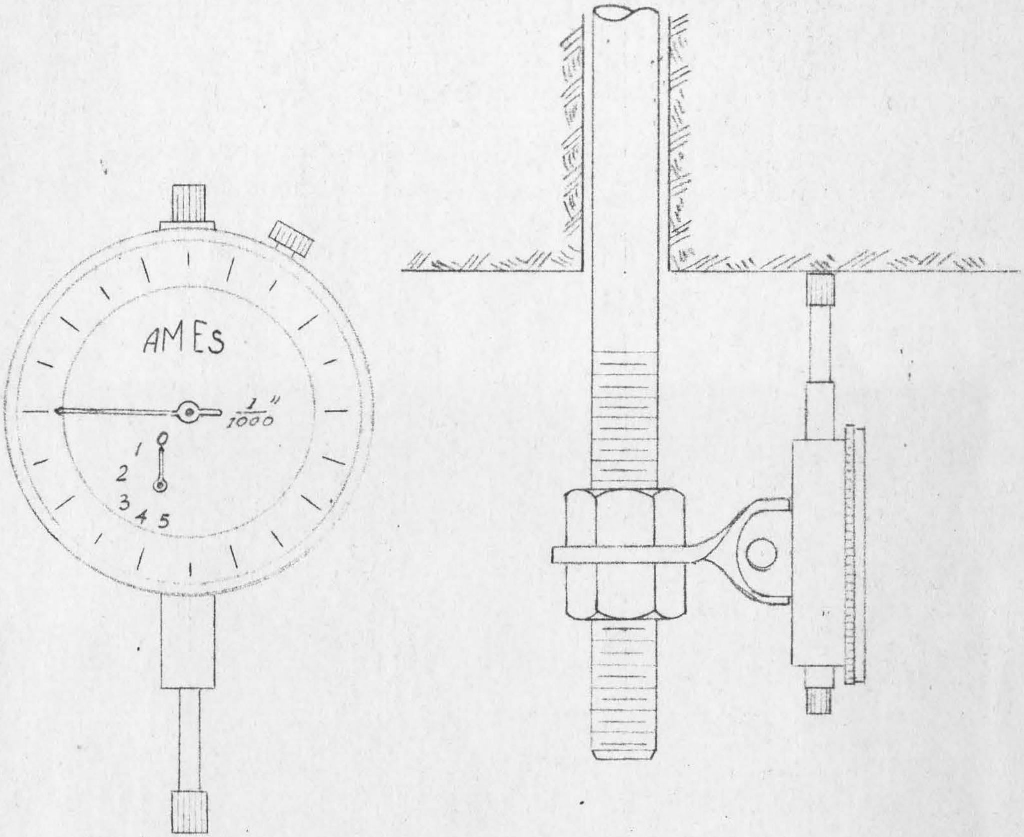


Figure I2. Ames extensometer and method of installation.

Corporation, Kenosha, Wisconsin. The dial on the torqometer read from zero to 1000 ft-lb in 20 ft-lb increments.

Method of Procedure

A. Determination of the stress variations in an installed roof-bolt.

The SR-4 type AB-3 strain gages were applied to roof bolts (see Appendix III) one foot from the bottom in accordance with the instructions furnished by the Eddystone Division of the Baldwin Locomotive Works. Part of the installation instructions are listed below:

1. Prepare surface to which gage is to be cemented.
    a. Remove scale or rust and roughen surface with medium grade emery paper.
    b. Clean carefully with absorbent cotton or clean cloths dipped in acetone. Repeat until all traces of grease or oil are removed.
2. Clean underside of gage with acetone.
3. Apply thin coat of cement to surface to which gage is to be cemented.
4. Place previously cleaned gage in desired location. Place over it a piece of cellophane somewhat larger than the gage area. Over this place a neoprene or felt pad (1/8 in. thick).
5. Place a metal plate over the pad and apply, by means of a C-clamp, a clamping load of 100 pounds per square inch.
6. Bake the clamped gage for one hour at 140° F., two hours at 175° F., and two hours at 250° F.

Rubber tubes* one inch long and three-quarters of an inch in diameter were pulled over the ends of the bolts and slid to a position immediately below the installed gages. The rubber tubes insulated the soldered connections of the three foot extension leads and the gage leads from the bolts. The rubber tubes two inches long and one inch in diameter were pulled over the ends of the bolts and slid to a position covering the strain gage and lead connections. The ends of these tubes were cemented to the bolts and then covered with Scotch electric tape. The outside rubber tube prevented moisture from reaching the strain gages** and also protected them during installation (see Fig. 10).

Immediately after a face-blasting four holes 47 inches deep and one and one-quarter inches in diameter were drilled in the roof one foot from the new face (see Fig. 14). The two holes nearest the side walls of the entry were drilled at an angle of thirty degrees away from the vertical towards the closer wall. The center holes were drilled in a vertical direction. Wedges were lightly tapped into

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* Manufactured by Fisher Scientific Company, Pittsburgh, Pa.

** A short section of a roof-bolt with a SR-4, AB-3 type strain gage mounted on it was immersed in water and allowed to remain in the water for two weeks. As daily readings were the same for the entire period, this method of protecting the gage from moisture was used instead of the one proscribed by the manufacturer.

the split end of the prepared four foot roof-bolts and the roof-bolts were inserted in the holes. By means of a drilling stoper applied to the bottom end of the bolts, the bolts were forced upwards on the wedges which butted on the bottom of the holes. As the bolts rode up on the wedges the ends of the bolts were forced apart, thus forming an anchor (see Fig. 5). Shallow grooves were scraped in the roof in a radial direction from the bolts. The strain gage leads were held in these grooves while the bearing plates were forced against the roof by means of nuts screwed on the bottom ends of the bolts. A pneumatic wrench was used to tighten the nuts.

Before the bearing plates were slipped over the ends of the bolts, the SR-4, model K indicator was connected to the strain gage leads and zero readings were observed and recorded. As soon as the nuts were tightened against the bearing plates the indicator was again connected to the leads of strain gages and initial readings were observed and recorded. Similarly, readings were observed and recorded for every three to six days for one and one-half months. Final readings were made four months later (see Tables I, II, and III).

B. Determination of the anchor holding strength of sandstone and shale roofs at various depths of insertion.

A 1-1/2 inch hole was drilled in the center of a 1-1/4 inch steel plate, 22 inches long and nine inches wide. The steel plate was slipped over the protruding end of a roof bolt that was installed in the center of an entry; oak blocks five inches on the sides and nine inches long were inserted between the ends of the steel plate and the roof. A hydraulic jack was attached to the free end of the bolt and a pulling force was gradually applied (see Fig. 9). The force at which the bolt pulled out of the roof or at which the bolt broke was observed and recorded. In an entry having a sandstone roof this procedure was followed for bolts installed at depths of 23, 10, 8, 6, and 4 inches (see Table 4). In an entry having a shale roof this procedure was followed for bolts installed at depths of 23, 13, 10, and 8 inches (see Table 4).

C. Determination of the holding strength of two types of bolts.

A 47-inch hole, 1-1/4 inches in diameter, was drilled in a mine entry roof. A roof-bolt was driven into the hole until the wedge spread the ends thus forming an anchor. A bearing plate was slipped over the end of the bolt and the extension rod of the hydraulic jack was attached to the free end of the bolt. An increasing downward pull was applied to the bolt until the bolt slipped or broke. The pull force at which the bolt slipped or broke was observed and recorded. This procedure was followed using three split-rod roof-bolts made by Bethlehem Steel Company, Bethlehem, Pennsylvania, and three forged lug roof-bolts made by the Cleveland Cap Screws Company, Cleveland 4, Ohio, in a shale and in a sandstone roof.

D. Determination of the roof sag in a mine entry.

The roof sag of two entries was measured over a period of six months. The roof of one entry was supported by roof-bolts; the roof of the other entry was supported by timbers. In the entry supported by roof-bolts, the supporting roof-bolts were installed with a center-to-center distance of 60 inches. A 48-inch bolt was anchored in a 44-inch hole drilled in the center of the roof midway between two rows of supporting roof-bolts. In the entry supported by timbers, the cross bars were installed with a center-to-center distance of 60 inches. A 48-inch roof bolt was anchored in a 44-inch hole drilled in the center of the roof midway between two cross bars.

A step plate was attached to the free end of the bolt. An extensometer was mounted on the horizontal part of the plate (see Fig. 12) and the initial reading of the extensometer dial was observed and recorded when the needle touched the roof. Dial readings of the extensometer which measured the sag in the lower 44 inches of the roof were observed and recorded once a week for six weeks, and finally, after six months.

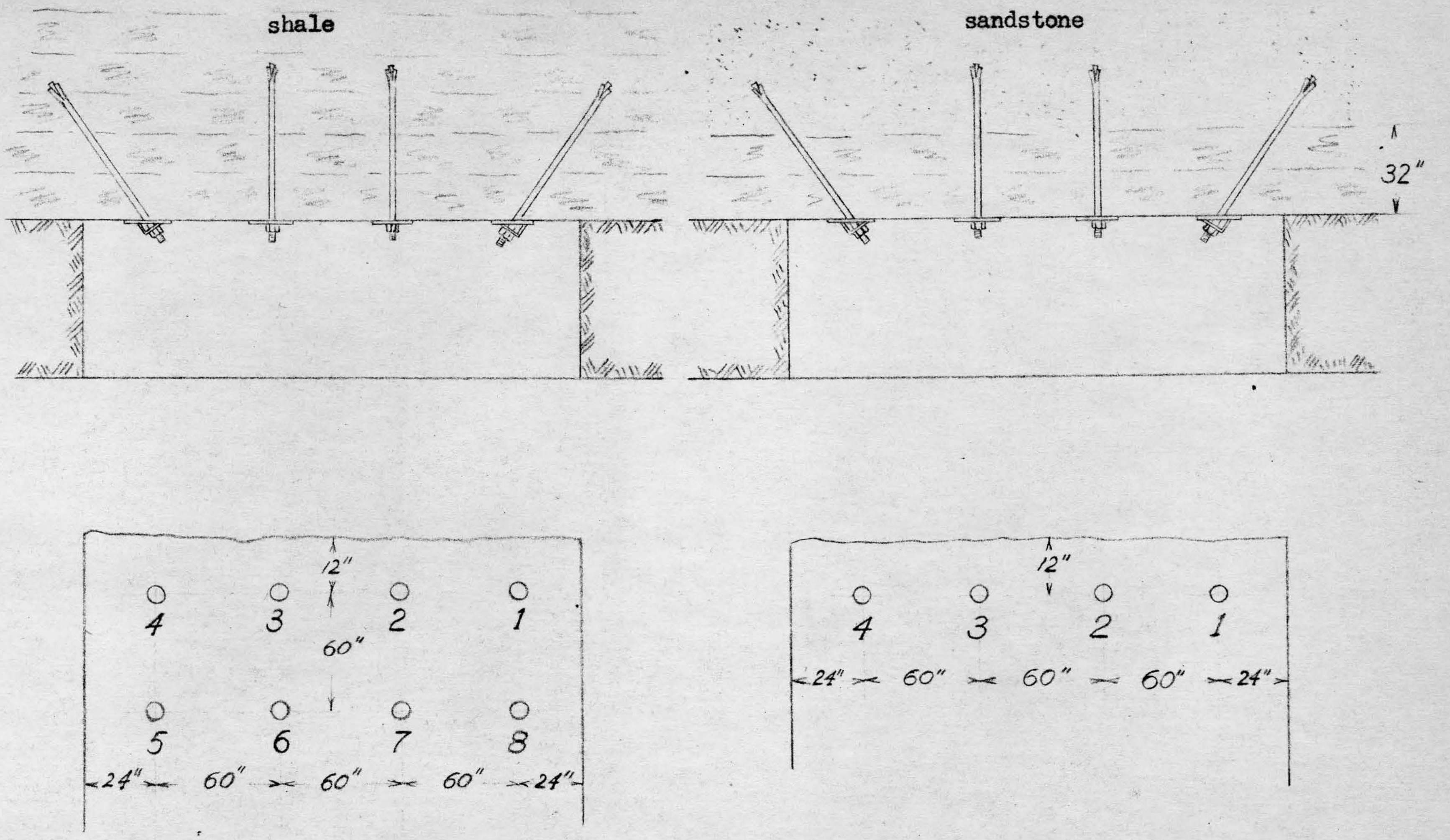
Data and Results

In order for the reader to better understand the results of the tests, a brief summary of each test is herein included.

A. Determination of the stress variations in an installed roof-bolt.

The stress variations in an installed roof-bolt can be calculated if the changes in elongation of the bolt are known. The changes in the elongation of the bolt were measured by attaching a SR-4 strain gage to a roof-bolt prior to its installation. After installation, the changes in elongation were read on SR-4 strain indicator attached to the strain gage. The strain indicator read elongation in micro-inches per inch of length. By subtracting the zero reading of the strain gage (reading obtained after the bolt was anchored but before the nut was tightened) from the strain indicator reading at the end of a period of time, the change in elongation over that period of time will be obtained. Multiplying the change of elongation in micro-inches per inch by the modulus of elasticity (30×10^6 for steel) will give the stress in pounds per square inch in the bolt at the time the reading was taken. The load on the bolt was found by multiplying the stress in the bolt by the cross-sectional area of the bolt in square inches at the point where the strain gage was glued to the bolt.

Fig. 13. Photographs showing the characteristic fracture of roof-bolts.



Bolts anchored in shale

Bolts anchored in sandstone

Figure I4. Distribution of Roof-bolts.

Example: Bolt number 1 in shale roof.

Initial reading	5450	
Zero reading	4950	
Elongation		500 micro-inches = 500×10^{-6} inches
Modulus of elasticity	30×10^{6} psi	$\times\ 30 \times 10^{6}$
Stress in roof-bolt		15000 psi
Cross-sectional area of bolt	0.645 sq in	x 0.645
Load on bolt		9,675 lb.

To find the load on the bolt at any time substitute the strain gage reading observed at that time for the "Initial reading" in the above example and proceed exactly as above example.

The loads on the bolts were calculated for each gage reading observed and listed in Tables I, II, and III. Tables I and II are the results for two rows of roof-bolts installed in a shale roof; Table III is the results of a single row of roof-bolts installed in a sandstone roof.

The first space in the "SR-4" column of all tables shows the "zero reading" and the "initial reading" for each bolt. Prior to taking the final readings on March 25, 1951, the dummy gage against which the active gage was balanced to eliminate changes in temperature, was damaged and it was necessary to make a new dummy gage. The "zero reading" using the new dummy gage was obtained by removing the nut from the bolt after the last reading was observed and balancing the strain gage indicator with no load on the bolt. The last space in the column shows the final gage reading while the bolt was in tension and also the gage reading after the nut was loosened and all tensioned removed from the bolt.

The gage leads on No. 3 and No. 5 bolts installed in shale were damaged and it was impossible to obtain final readings. The variations in stress in each bolt are presented in curve form in Fig. 15, 16, and 17. For ease of comparison, the curves for each row of bolts are plotted on the same set of axes.

TABLE I

Stress variations in a row of roof-bolts installed in a shale roof

Date	Bolt No. 1			Bolt No. 2		
	SR-4	Micro-inches	Tension in pounds	SR-4	Micro-inches	Tension in pounds
Oct. 25, 1950	4950-5450	500	9,675	4530-5160	630	12,190
Oct. 26	5500	550	10,642	5150	620	11,997
Oct. 31	5500	550	10,642	5150	620	11,997
Nov. 2	5550	600	11,610	5200	670	12,964
Nov. 6	5700	750	14,512	5330	800	15,480
Nov. 13	5800	850	15,480	5440	910	17,608
Nov. 20	5970	1020	19,737	5395	865	16,737
Nov. 27	5900	950	18,382	5400	870	16,834
March 25, 1951	8170-7300	870	16,834	7500-6780	720	13,932

Date	Bolt No. 3			Bolt No. 4		
	SR-4	Micro-inches	Tension in pounds	SR-4	Micro-inches	Tension in pounds
Oct. 25, 1950	4520-5650	1130	21,865	4620-5100	480	9,288
Oct. 26	5400	880	17,028	4950	330	6,385
Oct. 31	5520	1000	19,350	4750	130	2,515
Nov. 2	5600	1080	20,898	4800	180	3,483
Nov. 6	5750	1200	23,220	4950	330	6,385
Nov. 13	5760	1240	23,994	5030	410	7,933
Nov. 20	5830	1310	25,348	4965	345	6,675
Nov. 27	5845	1325	25,606	4950	330	6,385
March 25, 1951	—	—	—	7740-7420	320	6,192

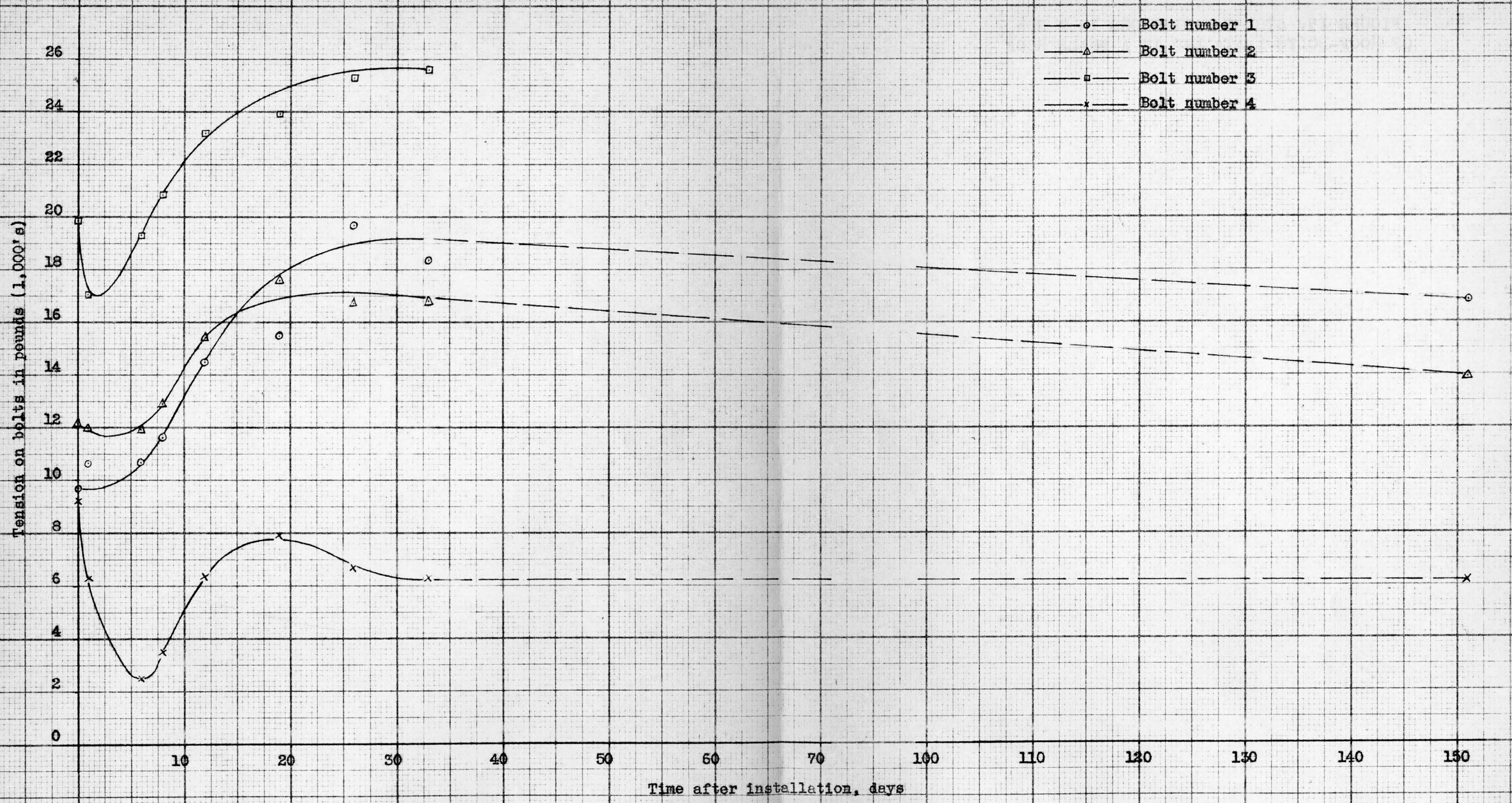


FIGURE 15. STRESS VARIATIONS IN A ROW OF ROOF-BOLTS INSTALLED IN A SHALE ROOF

TABLE II

Stress variations in a row of roof-bolts installed in a shale roof

Date	Bolt No. 5 SR-4	Bolt No. 5 Micro-inches	Bolt No. 5 Tension in pounds	Bolt No. 6 SR-4	Bolt No. 6 Micro-inches	Bolt No. 6 Tension in pounds
Oct. 25, 1950	6020-6550	530	10,255	5920-6690	770	14,899
Oct. 26	6450	380	7,353	6550	630	12,190
Oct. 31	6400	380	7,351	6340	420	8,127
Nov. 2	6400	550	10,642	6500	580	11,228
Nov. 6	6570	550	10,642	6600	680	13,158
Nov. 13	6600	580	11,223	6720	800	15,480
Nov. 20	7100	1080	20,893	6885	965	18,672
Nov. 27	6700	680	13,158	6850	920	17,995
March 25, 1951	—	—	—	9760-8930	830	16,060

Date	Bolt No. 7 SR-4	Bolt No. 7 Micro-inches	Bolt No. 7 Tension in pounds	Bolt No. 8 SR-4	Bolt No. 8 Micro-inches	Bolt No. 8 Tension in pounds
Oct. 25, 1950	4380-5070	690	13,351	5680-6500	820	15,867
Oct. 26	5000	620	11,997	6300	620	11,997
Oct. 31	5100	730	14,125	6250	570	11.029
Nov. 2	5280	900	17,415	6280	600	11,610
Nov. 6	5410	1030	19,930	6345	665	12,609
Nov. 13	5360	980	18,963	6370	690	13,351
Nov. 20	5380	1000	19,350	6310	630	12,190
Nov. 27	5400	1020	19,737	6350	700	13,545
March 25, 1951	7370-6320	1050	20,317	9150-8520	630	12,190

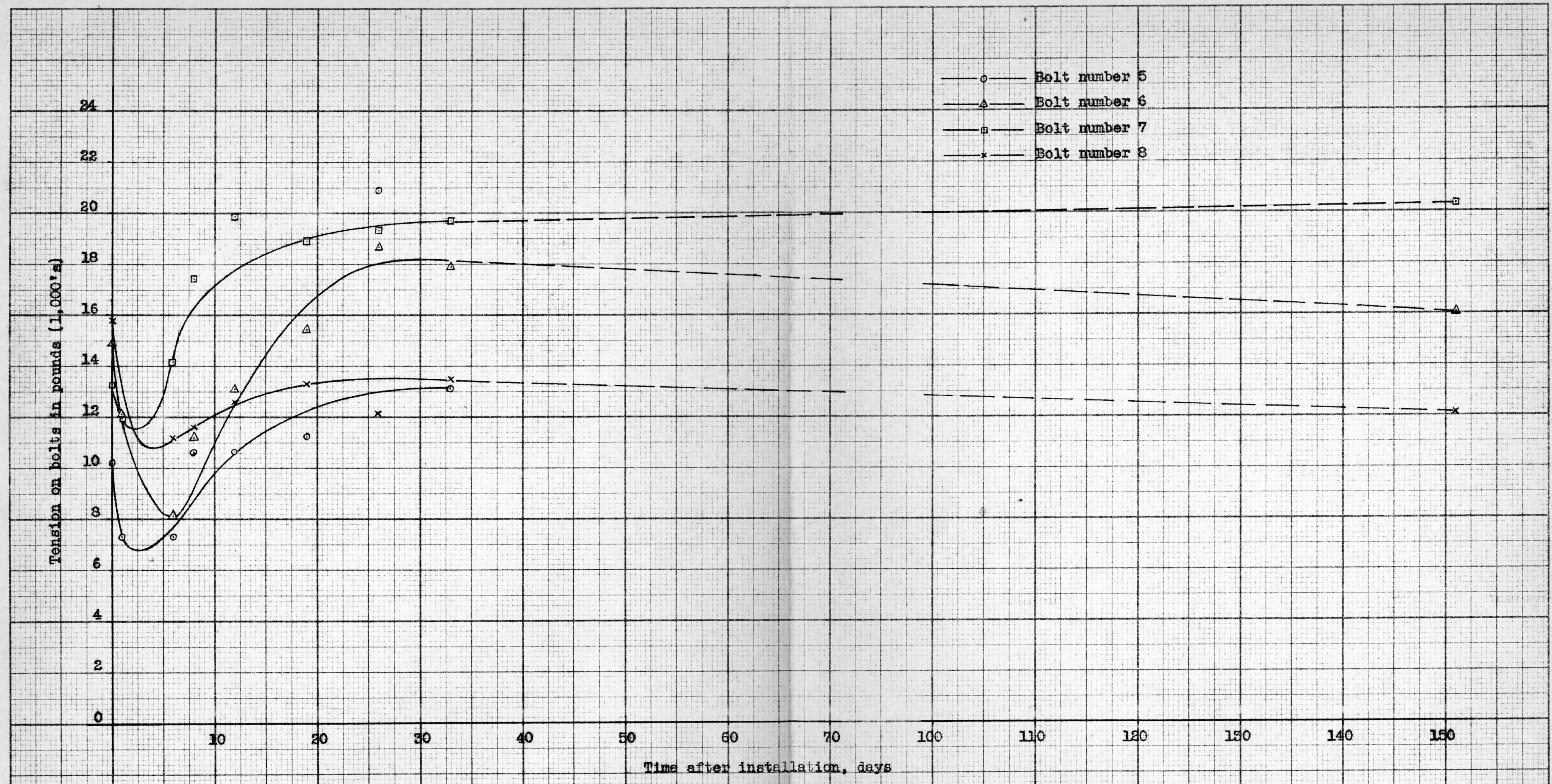


FIGURE 16. STRESS VARIATIONS IN A ROW OF ROOF-BOLTS INSTALLED IN A SHALE ROOF

TABLE III

Stress variations in a row of roof-bolts installed in a sandstone roof

Date	Bolt No. 1			Bolt No. 2		
	SR-4	Micro-inches	Tension in pounds	SR-4	Micro-inches	Tension in pounds
Oct. 16, 1950	6500-6800	300	5,805	5000-5370	370	7,095
Oct. 17	6800	300	5,805	5370	370	7,095
Oct. 18	6850	350	6,772	5430	430	8,320
Oct. 25	6950	460	8,901	5500	500	9,675
Oct. 31	6990	490	9,481	5600	600	11,610
Nov. 6	6980	480	9,288	5615	615	11,868
Nov. 13	7020	520	10,602	5590	950	18,382
Nov. 20	6950	450	8,707	5620	620	11,997
Nov. 27	6900	400	7,740	5600	600	11,610
March 25, 1951	9780-9340	440	8,514	8370-7700	620	11,997

Date	Bolt No. 3			Bolt No. 4		
	SR-4	Micro-inches	Tension in pounds	SR-4	Micro-inches	Tension in pounds
Oct. 16, 1950	6480-6900	420	8,127	7460-7840	380	7,353
Oct. 17	6950	470	9,094	7840	380	7,353
Oct. 18	6955	475	9,191	7900	440	8,514
Oct. 25	7010	530	10,255	7920	510	9,868
Oct. 31	7050	570	11.029	8030	570	11,029
Nov. 6	7020	540	10,449	8040	580	11,223
Nov. 13	7080	600	11,610	8065	605	11,706
Nov. 20	7150	670	12,964	8050	590	11,416
Nov. 27	7120	640	12,384	8100	640	13,029
March 25, 1951	9820-9250	570	11,029	1250-0700	550	10,642

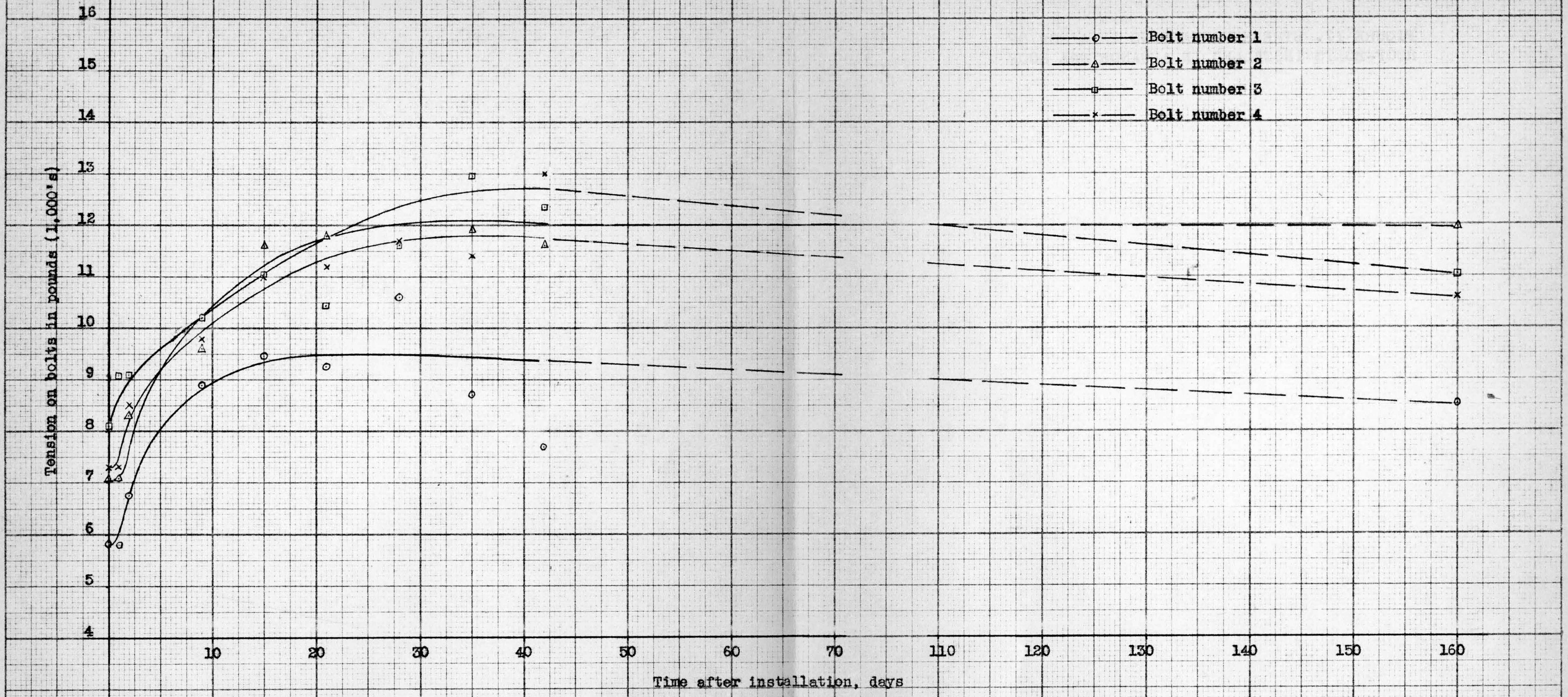


FIGURE 17. STRESS VARIATIONS IN A ROW OF ROOF-BOLTS INSTALLED IN A SANDSTONE ROOF

B. Determination of the anchor holding strength of sandstone and shale roofs at various depths of insertion.

The anchor holding strength of a roof can be determined by anchoring a bolt in the roof and applying a steadily increasing load to the bolt until the bolt slips through the roof or the bolt breaks. The anchor holding strength of a shale and a sandstone roof was determined in this manner. The shale roof tested had a fine grain structure and was relatively strong for a shale roof. Roof-bolts of 24-inch, 16-inch, and 13-inch lengths were anchored in 23-inch, 10-inch, and 8-inch holes. Steadily increasing loads were applied to the bolts (see Fig. 9) and the load at which the bolt slipped through the roof or broke was observed and recorded. Table IV is a tabulation of the length of bolt used, depth of hole, type of roof, maximum load at failure, and type of failure.

In the shale roof, the failure by slip seemed to follow the same pattern. The load on the bolt at which slip occurred remained constant during the slipping until approximately six inches of bolt remained in the hole. From this point until approximately three inches of bolt remained in the hole the load dropped to between 10 and 12 tons. As the last two or three inches of the bolt was pulled through the roof, the roof failed with a cone-shaped fracture with a twelve to sixteen inch base.

In all cases of bolt failure, the fracture occurred at the base of the fork. Bethlehem steel split-rod roof-bolts were used for all tests. The 17-inch and 13-inch bolts were cut to length from standard 24-inch roof-bolts and the slot was cut to a depth of six inches with a motor-driven saw.

TABLE IV

Anchor holding strength of mine entry roofs

Test No.	Type of bolt	Length of bolt in in.	Depth of hole in in.	Type of roof	Maximum load in tons	Type of failure
1	Bethlehem	24	23	Shale	21	Rod broke
2	"	24	13	"	18	Slip
3	"	24	10	"	15.5	"
4	"	24	8	"	17	"
5	"	13	8	"	16	"
6	"	13	8	"	15	"
7	"	24	23	Sandstone	20	"
8	"	16	10	"	21	Rod broke
9	"	24	8	"	19.5	"
10	"	16	8	"	19.5	"
11	"	16	6	"	22	"
12	"	13	6	"	17.5	Slip
13	"	13	6	"	19	Rod broke
14	"	16	5	"	18	"

C. Determination of the holding strength of two types of bolts.

The holding strength of roof-bolts can be obtained by anchoring the bolts in the roof and applying a load to the bolt until failure occurs. Bethlehem Steel split-rod roof-bolts and Cleveland forged lug roof-bolts were anchored in shale and sandstone roofs and loaded until the bolts failed. Forty-eight inch bolts anchored in 47-inch holes were used for all tests. Table V is a tabulation of the type of bolt, length of bolt, depth of hole, type of roof-rock, load at failure, and type of failure.

For these tests special care was taken to assure proper anchoring of the bolts. For the Bethlehem Steel split-rod type, the bolt is anchored properly when all except the top inch of the wedge is driven into the bolt. For the Cleveland forged lug type, the bolt is properly enclosed when all of the wedge is driven into the bolt (see Fig. 8 for dimensions of lugs and wedges).

The Bethlehem Steel split-rod bolts broke at the base of the fork. The Cleveland forged lug bolts broke at the base of the fork in some cases and below the fork in other cases.

TABLE V

Holding strength of roof-bolts

Test No.	Type of bolt*	Length of bolt in in.	Depth of hole in in.	Type of roof	Maximum load in tons	Type of failure
1	Bethlehem	48	47	Shale	15	Slip
2	"	48	47	Shale	20	Rod broke
3	"	48	47	Shale	17	Slip
4	"	48	47	Sandstone	21	Rod broke
5	"	48	47	Sandstone	20.5	"
6	"	48	47	Sandstone	22	"
7	Cleveland	48	46.5	Shale	17	"
8	"	48	47	Shale	18	"
9	"	48	47	Shale	16.5	"
10	"	48	47	Sandstone	18	"
11	"	48	47	Sandstone	17.5	"
12	"	48	47	Sandstone	17	"

* Both types of mine roof-bolts had a nominal diameter of one inch (see Fig. 8), actual diameter 0.907 inch

D. Determination of the roof sag in mine entries having different types of roof support.

The roof sag in a mine entry can be determined by attaching an extensometer to the free end of an anchored bolt and measuring the movement of the roof over a period of time. This was done in parallel entries approximately 50 feet apart. One entry was supported with roof-bolts; the other entry was supported with timbers. The center-to-center distance between rows of roof-bolts was the same as the center-to-center distance between cross bars in the timbered entry. Forty-eight inch bolts were anchored in 44-inch holes drilled in the shale roof, midway between cross bars. Extensometers were attached to the free end of the bolt and initial reading was observed and recorded. Subsequent readings were observed and recorded over a period of six months.

Table VI is a tabulation of the extensometer dial readings and the amount of roof sag on the days that readings were observed. By November 2, 1950 the roof sag in the entry supported with timbers had reached the limit of the extensometer and it was necessary to lower the bracket supporting the extensometer one-half inch in order to take further readings.

TABLE VI

Roof deformation in mine entries

Suspension roof support

Date	Oct. 18 1950	Oct. 25	Oct. 26	Oct. 31	Nov. 2	Nov. 6	Nov. 13	Nov. 20	Nov. 27	Mar. 10 1951	Mar. 25 1951
Dial reading	186	218	226	256	279	311	329	353	361	375	370
0.001 inch	0	32	40	70	93	125	143	167	175	189	184 (?)

Conventional timber support

Date	Oct. 9 1950	Oct. 18	Oct. 25	Oct. 26	Oct. 31	Nov. 2	Nov. 6	Nov. 13	Nov. 20	Nov. 27	Mar. 10 1951	Mar. 25 1951
Dial reading	0	197	408	430	483	500*	78	120	120	150	310	340
0.001 inch	0	197	408	430	483	500	578	620	620	650	800	830

* Bracket plate of extensometer was lowered and the zero reading was adjusted.

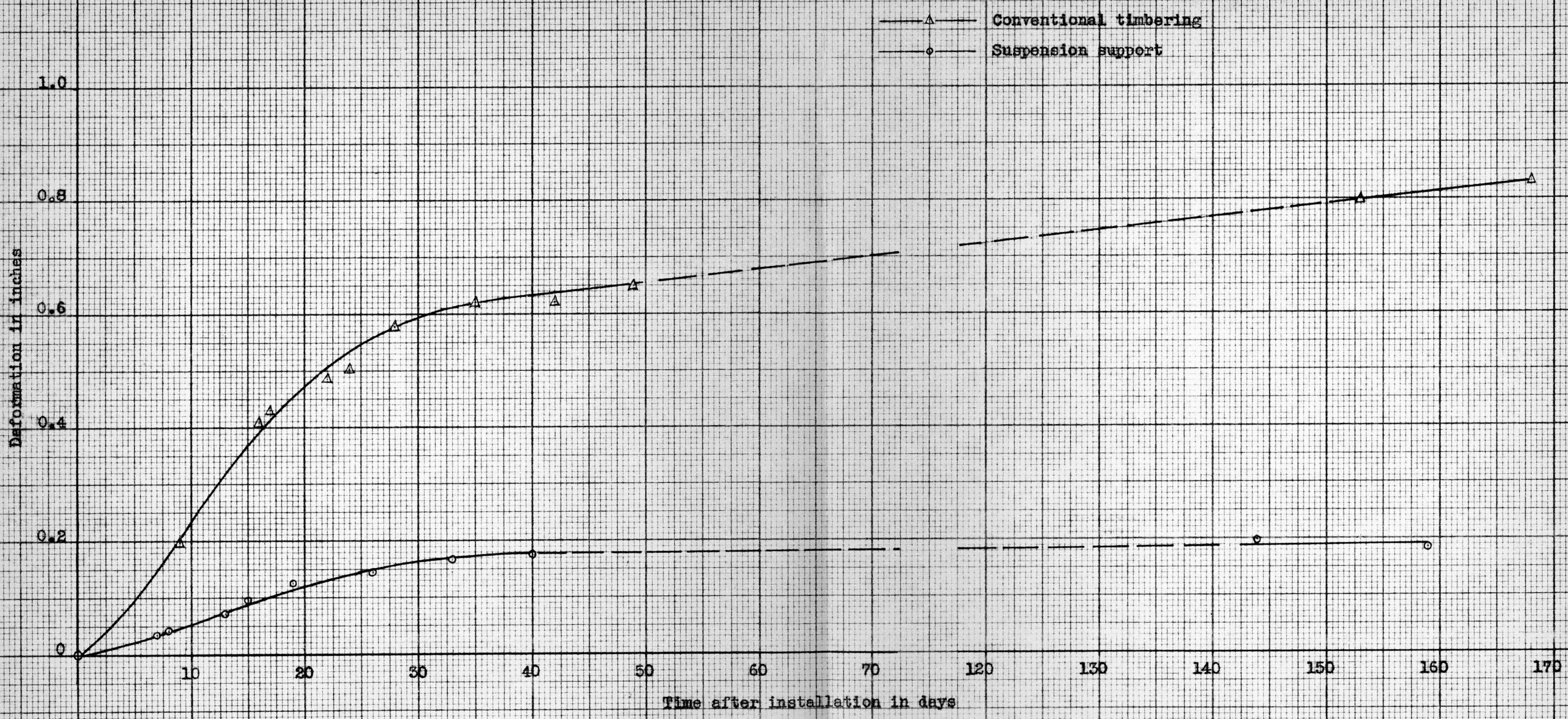


FIGURE 18. DEFORMATION OF THE ROOF IN A MINE ENTRY

IV

DISCUSSION

Discussion of Results

A. Stress variations in installed roof-bolts.

The roof-bolts installed in a mine entry having a shale roof seemed to have the same pattern of stress variation. The stress in the bolts decreased for three or four days and then increased rather rapidly for fifteen to twenty days. For the next ten days or so, there was a slight increase until a peak was reached. After this peak the stress decreased very slightly for the next five months. The final reading taken approximately six months after the initial installation was slightly higher than the initial reading.

The decrease in the stress for the first few days was due to the anchor slipping and to failure of the shale projections between the roof and the bearing plate. As the face was advanced, the roof sagged more and more, thus increasing the load on the bolts. After the face had advanced to the point where no further advances affected the roof support at the test section, the rate of roof sag gradually decreased and the stress in bolts reached a maximum. Because of plastic deformation and the effect of atmospheric conditions failure of the rock immediately surrounding the bolt anchor allowed the anchor to slip. This slip gradually decreased the stress in the bolt to a point just slightly higher than the initial stress.

The roof-bolts installed in a mine entry having a shale roof but anchorage reached a sandstone roof, also had a pattern of stress variation. The stress in the bolts increased rapidly for ten to fifteen days and then slowly for another ten to fifteen days until a maximum stress was reached. After this peak, the stress decreased over the next five months to a point slightly higher than the initial stress. Because of the good anchorage in sandstone the bolt immediately took up the roof sag without slipping. As the face advanced the stress in the bolt increased until the point was reached at which further face advances did not affect the roof at the test section. As in the shale roof, the plastic deformation and the atmospheric conditions cause a gradual failure of the rock surrounding the anchor allowing a little slip. This slip decreased the stress in the bolt.

B. Anchor holding strength of sandstone and shale roofs at various depths of bolt insertion.

The roof-bolts installed in a shale roof slipped when an increasing downward pull was applied. The exception was the bolt installed in a 23-inch hole. At this depth the anchorage was sufficiently strong to support a 21-ton load. Bolts installed in 8, 10, and 13 inch holes slipped through the drilled hole until five or six inches of the bolt remained in the hole. During this part of the slip the load on the bolt built up to a maximum of approximately 18 tons after the first inch of slip and remained at that value. As the next three or four inches of the bolt slipped through the hole the load dropped to 10 or 12 tons. When only two or three inches of bolt remained in the hole, the roof failed with a cone shaped fracture.

The ends of a bolt cut a groove in the sides of the hole as it is driven up on the wedge. The bolt is anchored when the grooves are so deep that the bolt cannot be driven further. As the load is applied the sides of grooves are compressed and there is a slight surface deformation between the bolt and the wedge. This allows some slip as the load on the bolt is increased.

As the load was increased the anchor tried to slip and pull the spread ends of the bolt through the shallower parts of the groove.

The ends of the bolt pressing against the sides of the groove exerted a compressive force on them. This increase in compression coupled with the elasticity of the rock had the effect of increasing the depth of the grooves. This allowed the anchor to slip. After the load was reached that would cause sufficient change in the rock stresses to allow slip, the slip continued with no change in load until only five or six inches of the bolt remained in the hole.

Because of the horizontal vibration of the drill, the first few inches of the hole are a little oversize. Atmospheric conditions acting on the surface layer of rock tends to weaken it.(5) All of these conditions combine to offer less resistance to the slipping of the anchor. The decrease in resistance decreases the load the bolt will support and even allows a cone-shaped section of the rock to be pulled from the roof when the last two to three inches of the bolt is pulled through the hole.

The roof-bolts installed in a sandstone roof slipped less than one inch when a load was applied to them. The load was increased until the bolt broke with no visible movement of the bolt after the initial fraction of an inch slip. Even when the bolt was installed in a five inch hole, the bolt broke instead of the anchorage failing.

Because of the greater density and hardness of the sandstone the bolt cannot be driven up on the wedge as far as it can be in a shale roof. The depth of the grooves cut in the sides of the drilled hole is sufficient to stop the bolt when there is still 3/4 inch to 1 inch of the wedge protruding above the bolt. The initial application of the load causes the sides of the anchor to compress the sides of the groove and to slightly deform the surfaces of the bolt slot and wedge. The modulus of elasticity of sandstone is less than that of shale(11) so the compressive force exerted on the sides of the grooves does not enlarge the grooves as much in sandstone as it does in shale. This reduces the slip due to the initial application of the load.

As the load was increased the sandstone offered enough resistance to prevent anchor slipping. The bolt was loaded until it broke at the base of the fork. (All bolts broke at this point due to stress concentrations at this point and the inability to apply an absolute axial load. See Fig. 13.) With only four inches of bolt anchored in a five-inch hole, the anchor holding strength of the rock was sufficient to support a load greater than the load supporting strength of the bolt.

The results of the tests on roof-bolts installed in shallow holes in a sandstone roof has a special significance. In many instances, the roof of an entry will have a layer of shale below the sandstone strata. For this condition, if the sandstone is strong enough, it is not necessary to insert the bolt more than six or eight inches into the sandstone. The anchor holding strength of sandstone for this depth of insertion is sufficient to hold any load the bolt is able to support.

C. <u>The holding strength of two types of roof bolts</u>.

Each of the two types of bolts tested had an advantage over the other one. The 48-inch Bethlehem steel split-rod roof-bolts slipped in shale at an average load of 16 tons and broke in sandstone at an average load of 21 tons. The 48-inch Cleveland forged lug roof-bolts broke in both shale and sandstone at an average load of 17 tons. The advantage of the forged lug bolt is its greater resistance to slip because of the larger area of contact between the bolt anchor and the sides of the hole. The advantage of the split-rod type is its greater load carrying strength. Both types of roof-bolts are adequate for the loads and conditions usually found in a mine entry roof.

D. Roof deformation in mine entries having different types of roof support.

The measurement of the sag of mine entry roofs brought out the basic principle of suspension roof support and illustrated its primary claim. Any sag in the roof of a mine entry increases the tension in the lower strata. Rock is weak in tension and cracks soon appear between cleavage planes. These cracks allow atmospheric air to reach the inner bonds between horizontal layers of the rock. The difference in tension between adjacent layers of rock creates a horizontal shear stress. The combination of these two actions destroys the bond between the layers of rock and allows the bottom layer to sag still more.

As each layer of rock sags the layer immediately above is subjected to the same bond destroying actions. The breaking of the bonds between horizontal layers puts more and more load on the roof supports. When the load supporting strength of the roof supports is exceeded the roof falls.

The suspension type of roof control is designed to preserve the bonds between horizontal layers of rock. The lower layers are physically bolted to the upper layers which eliminates most of the sag. By decreasing the sag, the load on the roof supports is lessened thus lessening the danger of roof failure.

Recommendations

It is recommended that:

1. An investigation be made to determine the effect the direction of the roof-bolt groove with respect to the axis of a mine entry has on the bolt anchorage.

2. An investigation be made to determine the effect of side-angled bolts on the tensile stress in the section between the bolts of a mine entry roof.

3. An investigation be made to determine the effect of atmospheric conditions on roof-bolt anchorage in a mine entry roof.

4. An investigation be made to determine the amount of roof-sag as measured by roof bolts inserted at various depths.

V

CONCLUSIONS

The results obtained during the investigation of the stress variations in roof-bolts and the behavior of shale and sandstone roofs when bolts were anchored in them led to the following conclusions:

1. SR-4 strain gages can be used to measure stresses below the elastic limit of a roof-bolt if installed according to the manufacturer's instructions and adequately protected from moisture.

2. Roof-bolts installed in sandstone do not require additional tightening after the initial installation. Roof-bolts installed in shale should be retightened on the second or third day after installation.

3. The differential roof sag in a mine entry as measured in this investigation is less with a suspension type of roof support than with conventional timbering.

4. Using a 1-1/4 inch drilled hole, good anchorage is obtained with Bethlehem split-rod roof-bolt if the wedge protrudes from 0.5 inches to 0.75 inches above the roof-bolt, when the bolt is anchored in shale and from 0.75 inches to 1.0 inch when the bolt is anchored in sandstone.

5. The sandstone above the shale in the roof of this mine afforded good anchorage if the bolt was anchored at a minimum distance of six inches in the sandstone.

6. Four-foot, one-inch nominal diameter roof-bolts installed with a center-to-center distance of 60 inches have a factor of safety of four based on the ultimate strength of the bolt.

VI

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VII

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VIII

APPENDICES

Appendix I.

The relationship between horizontal and vertical forces and values of Poisson's ratio at a point underground.

For equilibrium conditions the vertical force acting on a point underground is a function of the depth and the density of the rock. In equation form:

$$P_z = -d \cdot h$$

Where:

P_z is the equilibrium vertical force in pounds per square foot. The negative sign of the right side of the equation indicates that this force is a compressive one.

d is the density of rock in pounds per cubic foot.

h is the depth of the point in feet.

The horizontal force acting on a point underground is a function of the depth and the density of the rock and also Poisson's number. In equation form:

$$P_x = -\frac{d \cdot h}{m - 1}$$

Where: P_x is the horizontal force in pounds per square foot.

m is the particular Poisson's number for the depth of the point.

In 1948, Prof. Dr. Engr. Antoni Salustowicz(11) wrote an article that was incorporated in the book Odbudowa Gornicza. In this article, Prof. Salustowicz gave the following values of Poisson's number for depths of 0 to 2500 feet.

For sandstone : $3.5 \leqslant m \leqslant 11$

For shale and coal : $2.67 \leqslant m \leqslant 6.$

Substituting P_z for $-dh$ and different values of m in the equation for finding the horizontal force on a point not more than 2500 feet below the surface of the earth:

$$\text{For sandstone: } P_x = \frac{P_z}{3.5 \text{ to } 11 - 1}$$

$$\text{Therefore: } 0.2\ P_z \leqslant P_x \leqslant 0.4\ P_z$$

$$\text{For shale and coal: } P_x = \frac{P_z}{2.67 \text{ to } 6 - 1}$$

$$\text{Therefore: } 0.2\ P_z \leq P_x \leq 0.6\ P_z$$

As the depth of the point increases Poisson's number approaches to the value of 2. This means that the horizontal forces approach the vertical forces in magnitude at very great depths.

When a mine entry is opened the forces are no longer in equilibrium and they change in magnitude. In the case where the point in question is located in the center of the wall of a mine entry:

$$S_1 = n\ P_x - P_z \quad \text{(see Fig. 1)}$$

Where: S_1 is the non-equilibrium compressive vertical force in pounds per square foot.
n is a coefficient, the value of which depends on the ratio of the width of the entry to the height.

In the case where the point in question is located in the center of the roof of a mine entry:

$$S_2 = -P_z + n'P_x \quad \text{(see Fig. 11)}$$

In his article, Professor Salustowicz$^{(11)}$ gave the following values for n and n'.

<u>Width</u> Height	50:1	20:1	5:1	1:1	1:5	1:20	1:50
n	18.0	5.0	3.0	1.84	1.20	1.02	1.01
n'	1.01	1.02	1.20	1.84	3.0	5.0	18.0

Appendix II

Method of determining the stress ellipse of a mine entry.

Roof-bolts provide better protection of the anchor in the compressive zone of forces about a mine entry. As the locus of the points of zero tension is an ellipse proscribed about a mine entry, the length of roof-bolts used should be sufficient to pass through this ellipse. The maximum tensile stress is at the highest point of any ellipse proscribed about the rectangular mine entry and in equation form is:

$$S_A = -P_z + P_x\left(1 + 2\frac{a}{b}\right) \quad (11)$$

Where: S_A is the tensile stress at the top of the ellipse in pounds per square inch
P_z is the vertical equilibrium stress in psi
a is one-half the vertical axis of the ellipse
b is one-half the horizontal axis of the ellipse (see Fig. 3)

By equating this maximum stress to zero, we can find a relationship of "a" and "b" that will give the ellipse that is the locus of the points of zero tension.

$$S_A = 0 = -P_z + P_x\left(1 + 2\,\frac{a}{b}\right)$$

$$\frac{a}{b} = \frac{1}{2}\left(\frac{P_z}{P_x} - 1\right)$$

$$P_x = \frac{P_z}{m - 1}\ ;\ \frac{P_z}{P_x} = m - 1 \quad \text{(see Appendix I)}$$

Where: m is the Poisson's number for the depth of the entry.

$$\frac{a}{b} = \frac{1}{2}(m - 2)$$

Thus the relationship of "a" to "b" is a function of Poisson's number. In order to find the value of "a" and "b" for a particular entry having a width "2w" and a height "2h" (see Fig. 3), it is necessary to have a second equation containing "a" and "b". We know that the ellipse passes through the corners of the entry;(11) therefore, the coordinates of any one of the corners will satisfy the equation of the ellipse.

The general equation of an ellipse is:

$$\frac{x^2}{b^2} + \frac{z^2}{a^2} = 1$$

Where: x is the horizontal coordinate of any point of the ellipse
z is the vertical coordinate of any point of the ellipse

Let x equal one-half the width of the entry = w

Let z equal one-half the height of the entry = h

$$\frac{w^2}{b^2} + \frac{h^2}{a^2} = 1$$

This equation can be used as the second equation containing "a" and "b".

From the equation $\frac{a}{b} = \frac{m - 2}{2}$

$$b = \frac{2a}{m - 2}$$

Substituting this value of "b"

$$\frac{w^2}{\left(\frac{2a}{m - 2}\right)^2} + \frac{h^2}{a^2} = 1$$

and $$a = \frac{1}{2}\left[w^2(m - 2)^2 + 4h^2\right]^{\frac{1}{2}}$$

Now knowing the height above the center of the entry at which the tension in the rock is zero we can determine the length of roof-bolt to use in order to have the anchor in the compressive zone. As rock has some strength in tension, it is not imperative that the anchor be in the compressive zone but near enough to it so that the tensile strength of the rock is not exceeded.

Appendix III

Stress variations in two roof-bolts having duel gages

To determine the amount of elongation of a roof-bolt that was due to the bending of the bolt, two SR-4 strain gages were attached to opposite sides of the bolts. The two bolts were installed in a shale roof and readings were observed for six weeks (see Table VII and Fig. 19). After nine days, the bolt No. 1 was stressed beyond the elastic limit of the bolt and the variations had indefinite meaning. For the first nine days the variations between the two gages mounted on each bolt were small and could be attributed to inaccuracies due to lead connections and bends in the leads, and to induction currents when the readings were observed. For this reason, and also the difficulty of moisture-proofing, two SR-4 gages on the same bolt, only one gage was installed on each bolt used during the investigation. Further justification for using only one gage on each bolt is the fact that this investigation deals with the changes in the bolt stress over a period of time and not with the actual values of the stress at any one time.

TABLE VII

Stress variations in two roof-bolts installed in shale and having duel gages

Bolt No. 1

Date	Gage A SR-4	Gage A Micro-inches	Gage A Tension in pounds	Gage B SR-4	Gage B Micro-inches	Gage B Tension in pounds
Oct. 9, 1950	3530-4370	840	16,250	5535-6450	915	17,770
Oct. 10	4320	790	15,300	6385	850	16,400
Oct. 12	4600	1070	20,750	6630	1095	21,150
Oct. 13	4800	1270	24,600	6845	1310	25,400
Oct. 18	5030	1500	29,000	7040	1505	29,100
Oct. 31	5000	1470	28,430	7060	1525	29,600
Nov. 6	5120	1590	30,800	7140	1605	31,000
Nov. 13	5220	1690	32,700	7230	1695	32,800
Nov. 20	5180	1650	31,900	7245	1710	33,200
Nov. 27	5275	1745	33,800	7310	1775	34,400

Bolt No. 2

Date	Gage A SR-4	Gage A Micro-inches	Gage A Tension in pounds	Gage B SR-4	Gage B Micro-inches	Gage B Tension in pounds
Oct. 9, 1950	5275-6000	725	14,000	6525-7475	950	18,350
Oct. 10	5910	635	12,300	7320	795	15,750
Oct. 12	6065	790	15,300	7495	970	18,600
Oct. 13	6230	965	18,700	7740	1215	23,500
Oct. 18				7750	1225	23,700
Oct. 31	Leads			7735	1210	23,400
Nov. 6	were cut			7810	1285	24,800
Nov. 13				7800	1275	24,650
Nov. 20				7925	1400	27,100
Nov. 27				8040	1515	29,400

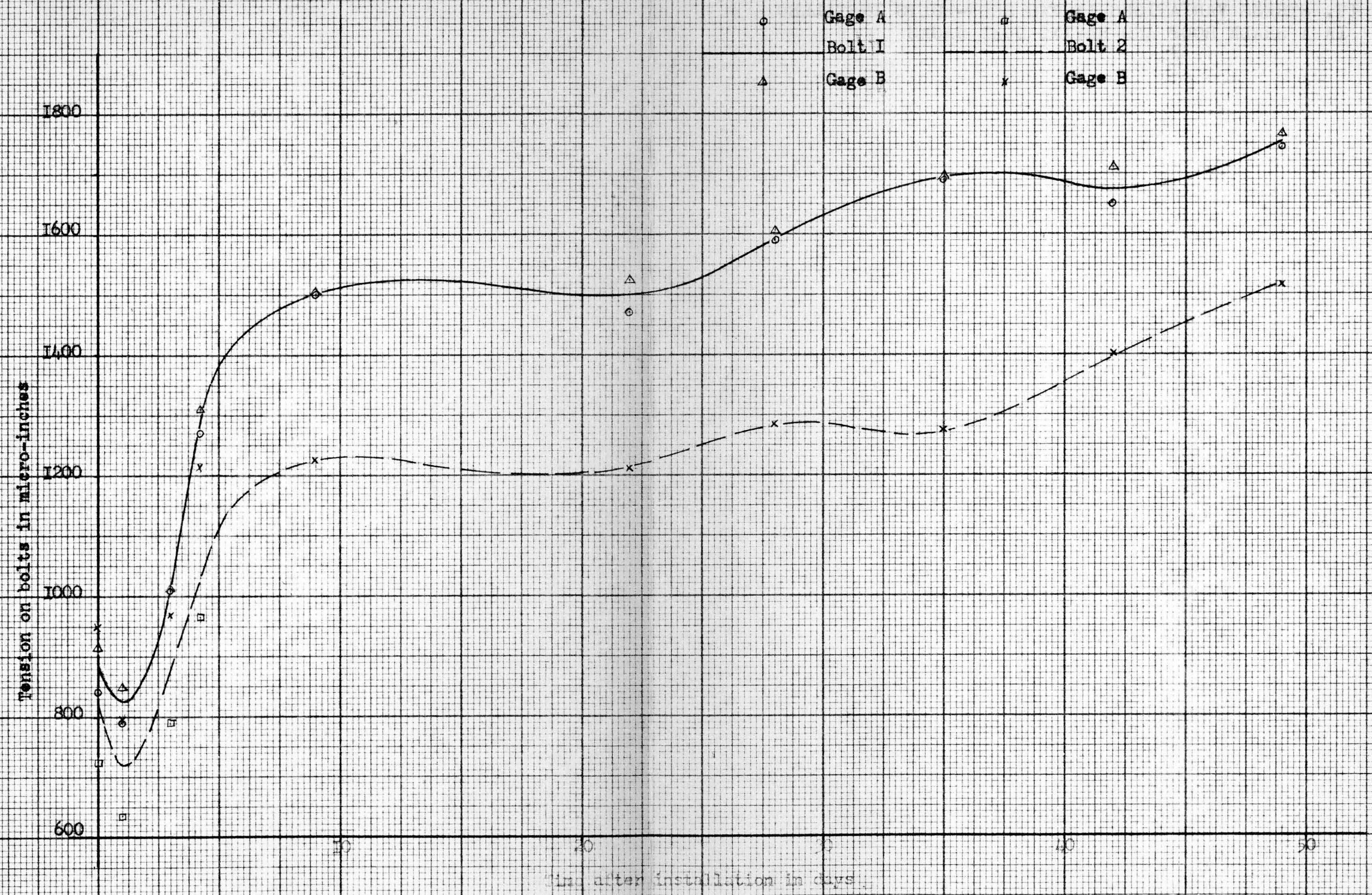

FIGURE I9. STRESS VARIATION IN TWO ROOF-BOLTS INSTALLED IN A SHALE ROOF AND HAVING DUEL GAGES

Appendix IV

Relation between torque and tension on bolt

In order to determine the relationship between the torqometer reading and the load on a bolt a 48-inch Bethlehem split-rod roof-bolt with a one-inch nominal diameter* was slipped through a five inch pipe 42-inches long. A bearing plate and a section of channel iron was slipped over the split end of the bolt and a one inch hex nut was screwed on the bolt. AB-3 SR-4 strain gage readings were observed for increments of 50 foot-pounds on the torqometer from zero to 300 ft-lb. The results of the tests performed on three different bolts are shown in Table VIII. The curve showing the relationship between the torqometer readings and load on the bolt is given in Fig. 20.

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* Actual diameter of bolt was 0.907 inch.

TABLE VIII

Relation between torque and load on a bolt*

Test No.	Torque in ft-lb	SR-4 Reading	Micro-inches	Tons
1	0	5,790	-	-
	60	5,950	160	1.55
	100	6,000	210	2.03
	150	6,160	370	3.58
	200	6,290	500	4.84
	250	6,350	560	5.60
	300	6,465	655	6.35
2	0	4,530	-	-
	50	4,635	105	1.07
	100	4,780	250	2.42
	150	4.870	340	3.30
	200	5,010	480	4.32
	250	5,140	610	5.55
	300	5,210	680	6.56
3	0	5,020	-	-
	50	5,165	145	1.4
	100	5,295	275	2.65
	150	5,370	350	3.4
	200	5,445	435	4.1
	250	5,535	515	5.0
	300	5,770	750	7.25

* Bethlehem split rock roof-bolt. (1 inch nominal diameter; actual 0.907 inch)

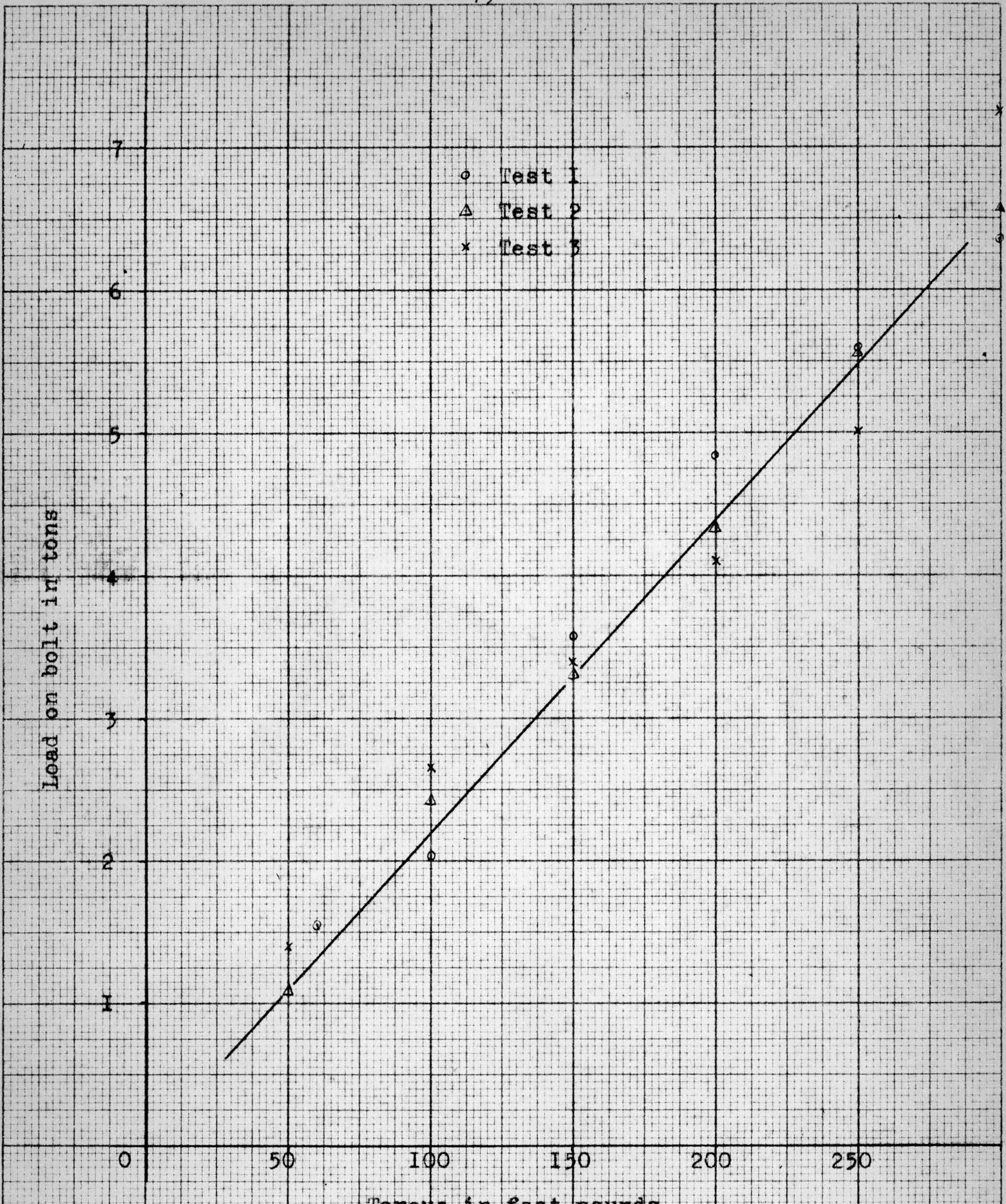

Figure 20. Relationship between torque and load on a Bethlehem split-rod roof-bolt of a one inch nominal and 0.907 actual diameter.