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An appraisal of the pillar support system at the St. Helena gold mine

Hazell, A. A.

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FIG 49

Relationship Between Closure and Stress

2476 Top Stope Pillar

There was no difference in the rate of movement during the period in which the internal stope pillars were removed.

2436 Stope test results are given in graphical form in FIGS. 46 and 47. Closure at the pillar stations is plotted against date and distance of the stope face away from the pillar centre.

It can be seen in FIG. 46 that closure near the stope face is rapid, and 0.5 ins. took place at Station A.1 within 48 hours even though the stope face had only advanced a few feet. By the time the pillar was developed, the closure at the first stations, A.1 and A.2, had reached ½ ins. and 1½ ins. The rate of closure decreased as the stope face advanced further from the pillar. The final amount of closure after twelve months was 2¾ins. at Station A.1 and 3¼ ins. at Station A.2 - the stope face was 140 ft. away from the pillar centre, and the strike span was 600 ft.

2426 Stope test results for No. 1 Pillar are given in FIG. 51 and these show that a closure of 0.7 ins. occurred in two months. A similar amount of closure occurred in 2436 Stope during the two months after installation of the stations.

6.2.4 Comparison with Laboratory Results:

Due to the pillar removal a closure of 1¾ ins. occurred in 2323 Stope at the mid-span, but Resistance Analogue results indicate that 3.88 ins. of closure should occur at mid-span if a solid pillar is removed, (FIG. 23). It was deduced from this that pillar failure had previously occurred in 2323 Stope.

In 2436 Stope, by the time the stope face had reached its strike limit, a maximum closure of 3.2 ins. had occurred at Pillar "A" and 3.4 ins. at Pillar "B". The removal of the remnants between the stopes and the removal of the internal pillars, could be expected to result in a final closure at Stations "A" and "B" of at least 4 ins. This amount of closure

is / .....

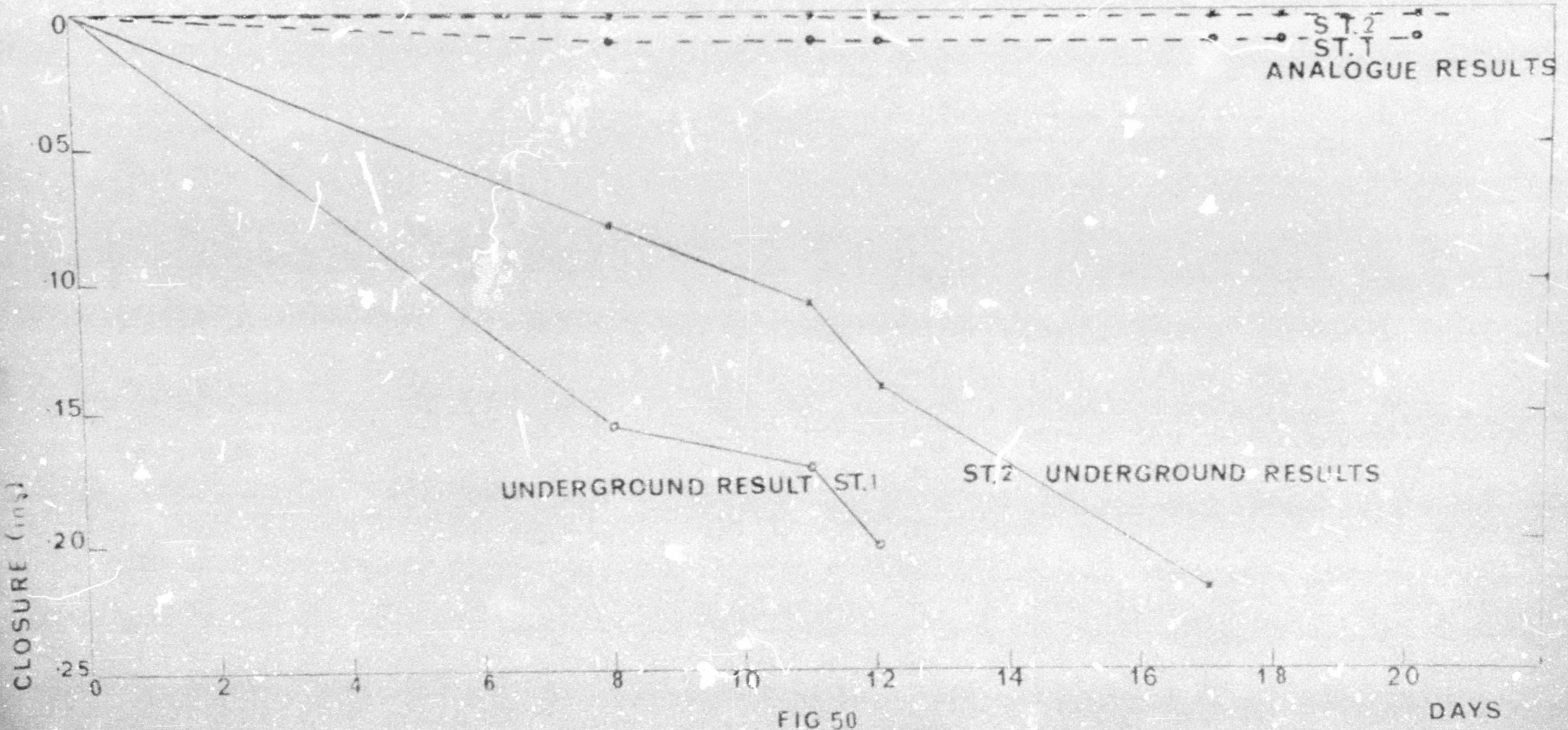


FIG 50

Comparison of Closure

Resistance Analogue Results – Underground Closure Results

2436 Stope Top North Pillar

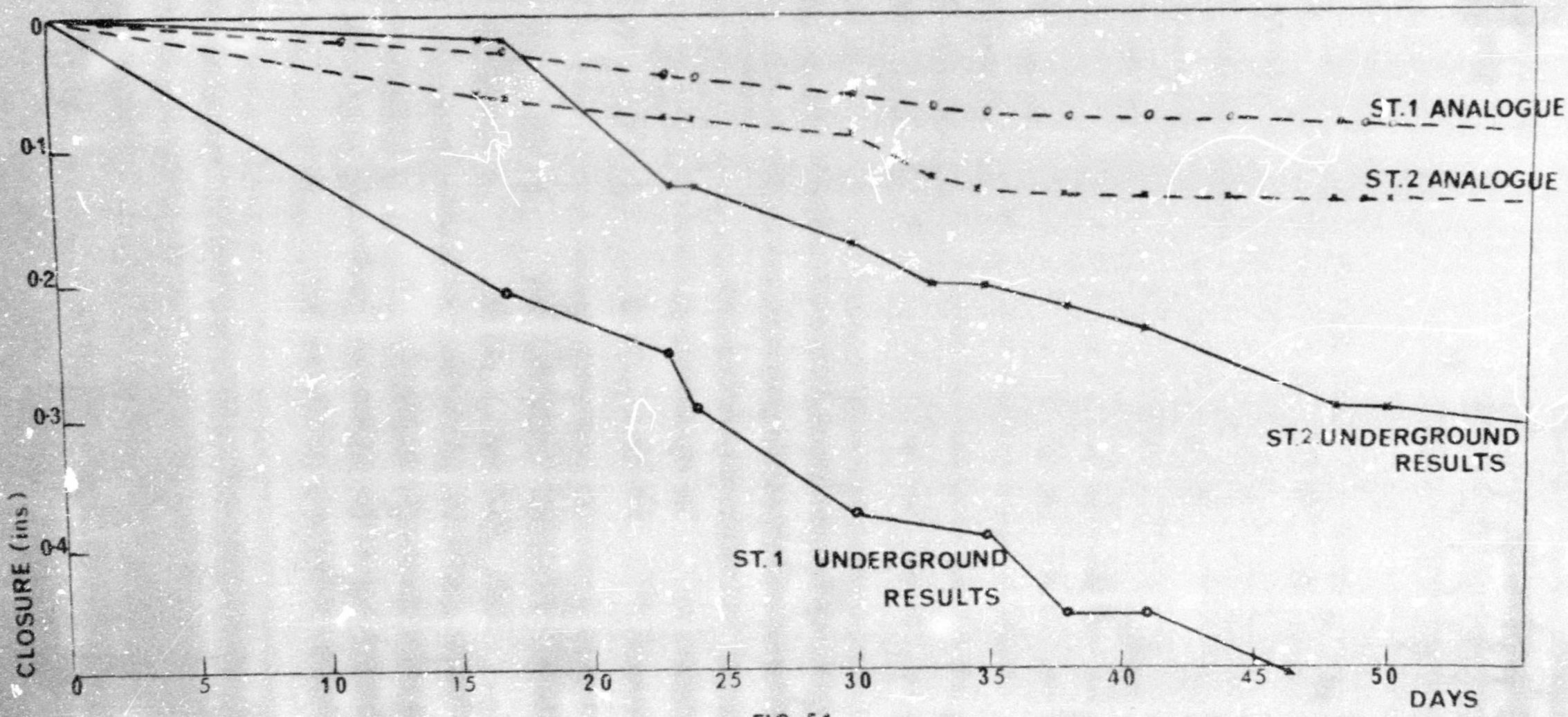


FIG 51

Comparison of Closure

Resistance Analogue Results – Underground Closure Results

2426 Top North Pillar

is more that the theoretical amount of 2.6 ins. which would occur at these positions, (deduced from FIG. 23 at a position 50 ft. away from the drive pillar). Furthermore, it can be seen from FIG. 50 that actual closure is greater than theoretical closure from the commencement of pillar development. FIG. 51 shows similar results in 2426 Stope.

It can be concluded from the above results that the immediate hangingwall and footwall did not behave elastically in any of the stopes where measurements were taken, i.e., there must have been a fracture zone around the workings.

It can be seen from FIGS. 50 and 51 that the strata adjacent to the pillar does not behave as predicted by elastic theories, and it is concluded that the pillar sides have failed. Furthermore, an increase in closure rate occurs as pillar development continues, thus implying an increase in the rate of pillar fracture.

It is not possible from the results of the closure measurements to deduce how far the fractures extended into the pillar and therefore whether strength failure occurred[22]. Rupture or disintegration of a pillar does not take place.

TABLE 3

"IN-SITU" STRESS RESULTS:

WORKING PLACE	POSITION OF HOLE	DEPTH OF READING	PRINCIPAL STRESS MAGNITUDE		DIRECTION FROM VERTICAL	RATIO OF STRESSES
			Major	Minor		
18 South Drive	Perpendicular to drive face	4 ft.	2,300	101	23°	.04
		5 ft.	1,210	350	15°	.29
		8 ft.	1,000	250	10°	.25
2436 Stope	Top North Pillar	7.5 ft.	1,710	566	28°	.3
		8 ft.	2,805	926	31°	.3
		9.5 ft.	2,340	554	25°	.25
2436 Stope	Bottom South Track Cutting	6.5 ft.	7,890	2,705	19°	.3
		7.5 ft.	4,858	1,618	25°	.3
		9.0 ft.	2,500	486	18°	.15

CHANGE IN STRESS RESULTS:

WORKING PLACE	POSITION OF HOLE	DEPTH OF READING	MAXIMUM STRESS CHANGE LBS./INS.^{2}	DIRECTION
2436 Stope	Top North Pillar	12 ft.	+ 4,600	40°
	Top South Pillar	10 ft.	600	unsymmetrical
	Bottom South Drive Pillar	8 ft.	+ 7,000	+ 3°
2426 Stope	Top North Pillar	7 ft.	+ 5,000	-

7. PILLARS AS UNDERGROUND SUPPORTS

The rock mass around a small excavation in virgin ground on St. Helena Mine is assumed to be elastic. However, general observations show that as such an excavation increases in size a fracture zone develops around it.

If the rock surrounding an underground excavation is elastic, a tensile stress zone will exist in the hangingwall over almost the full span, (FIGS. 4, 5 and 6). The height of this tensile stress zone is related to the span and the depth, and at a particular depth the height of the zone increases as the span increases, (FIG. 12). In Sec. 3.2 it was shown that incipient fractures are most likely to extend in a tensile stress zone. Depending on the grain structure of the rock and the magnitude of the minor stress, the fractures may extend to a length equal to the height of the excavation. Fractures around an underground excavation are initially caused by blasting, and the fractures in the hangingwall are those most likely to increase in size. These fractures may intercept bedding planes if the rock forming the hangingwall of the stope is stratified, as it is on St. Helena Mine. The beds will readily separate and form hangingwall beams or blocks. In Sec. 3.2 it was shown that the height of the fracture zone, determined from the location of seismic events, was equal to the height of the tensile zone calculated from FIG. 12.

The behaviour of stope pillars must be considered in relation to the fracture zone and the other stope supports. At depth the support system must not only control the fracture zone, but must also limit the energy release rate to a safe amount as mining proceeds. Energy release is the product of stress and closure. Closure can be minimised by limiting the span of the hangingwall or providing very high density support. In this chapter the results of laboratory and underground tests will be used to discuss pillars as a means of support.

7.1 Stope Pillars: / .....

7.1 Stope Pillars:

The results of tests on the Resistance Analogue indicate that the average stress on a newly formed stope pillar at a depth of 5,000 ft. is 12,000 lbs./ins.^{2}, (FIG. 24). When the stope face has advanced 125 ft. away, the stress on the pillar is 30,000 lbs./ins.^{2}. After the stope remnant has been mined out the stress on the pillar is 50,000 lbs./ins.^{2}. A final stress magnitude of 70,000 lbs./ins.^{2} is reached after the removal of the other stope pillars. The photo-elastic analysis made on a St. Helena Mine model shows that a considerable stress variation occurs within a stope pillar. From FIG. 21 it can be seen that the principal stress at the pillar sides is about twice that at the pillar centre, and is therefore greater than the average stress.

From these theoretical results it can be expected that as the stress is in excess of the uniaxial compressive strength, (Table 1), pillar sides will fracture before the stope reaches the remnant stage. Further cracking of the pillar may be expected as the stress increases to a final magnitude of 70,000 lbs./ins.^{2}.

Underground tests confirm that a fracture zone exists ahead of the stope face at a depth of 5,000 ft. The actual stress must be greater than the uniaxial compressive strength of the rock, and greater than the theoretical average stress of 12,000 lbs./ins.^{2} obtained on the Resistance Analogue. Pillar sides fail as the pillar is being formed, and closure tests show that a fracture zone develops around it in the hangingwall and footwall of the stope excavation. At the centre of the pillar an increase in stress of at least 8,000 lbs./ins.^{2} has been measured, and a comparison with the Resistance Analogue results showed that the solid core of the pillar was elastic, (Sec. 6.1.4). FIGS. 36 and 38 show that after the pillar is formed the actual stress increase is greater than that obtained on the Resistance Analogue. This effect is probably due to the load being carried on a smaller core of solid rock.

As mining proceeds and the stope face advances the stress on the pillar increases and further fracturing is assumed to take place,

(Sec. 6.1.4) /.....

(Sec. 6.1.4). Certainly closure measurements indicate an increase in the rate of pillar fracture (Sec. 6.2.4). At this stage rock fractures under triaxial stress conditions due to the restraint afforded by the fractured sidewall. The continuation of cracking due to further mining will depend on the stress magnitude and the constraint provided by the increasing volume of fractured rock. Theoretically, the final stress on a solid pillar may reach 70,000 lbs./ins.^{2}. Triaxial strength tests[16] indicate that a restraint of 7,000 lbs./ins.^{2} is needed to maintain a stress of 70,000 lbs./ins.^{2}, but it is doubtful whether the fractured part of the pillar would provide such a restraint. Therefore, cracking will extend completely through the pillar at a stress level of less than 70,000 lbs./ins.^{2} and strength failure may occur[22]. This stage will be reached after the stope remnant has been mined, and before any stope pillars have been removed.

During the life of a pillar barring away of the sidewalls may be carried out once or twice, but pillars have never been observed to collapse. Furthermore, removal of a pillar usually requires blasting, but it can be assumed that a fractured pillar still carries considerable load.

The theoretical stress results shown in FIG. 24 for mining with pillars, must now be modified and the stresses on the stope face and drive pillars must lie between the two comparisons. The energy release rate during mining will, therefore approach that due to mining without stope pillars (FIG. 29). Due to extensive fracturing the average stress on the stope pillar is less than the theoretical stress and no rock bursts should occur when the pillar is removed.

The stability of the hangingwall in the stopes tested is indicated by the small rate of closure and ride before pillar removal (FIGS. 43 to 47). In 2436 Stope 3 ins. of closure occurred during the year in which tests were made and in 2426 Stope the closure rate was ¼ ins. per month. A comparison with the theoretical closure, (FIGS. 50 and 51), showed that the stope excavation at a depth of 5,000 ft. below surface is surrounded by a fractured zone. At these depths a newly

formed / .....

formed pillar is mainly solid and the stability of the hangingwall fractured zone is the result of Voussoir Arches forming between the pillars, and between the pillars and the stope face (Sec.4.1). As the stope excavation increases the stress on the pillar increases and the cracks extend into the pillar. As the stress approaches 70,000 lbs./ins.^{2} cracks extend through the pillar and the load carried by the pillar decreases with increasing deformation, i.e. strength failure occurs[22]. In Sec. 3.2 it was shown that a failed pillar at a depth of 5,000 ft. below surface can support a Voussoir span in excess of the pillar spacing, and the main hangingwall loads can still be carried by the stope pillar system.

During the sweeping stage payable internal pillars are mined, usually one at a time, retreating towards the centre gully. Underground tests in 2323 Stope showed that only 1 ins of closure and ½ ins. of ride took place during the pillar removal (Sec. 6.2.2). It must be assumed that due to the slow process of pillar removal, about four months, loads are gradually transferred to drive pillars and matpacks, and therefore, hangingwall stability is maintained. In the year after pillar removal in 2325 Stope, a further ½ ins. of closure only was measured.

At depths greater than 5,500 ft. pillar cracking will be more extensive and strength failure will occur rapidly. The pillars will, therefore, shed load and the rate of loading on the matpacks will increase. This may result in matpack instability and hangingwall collapse will occur.

7.2 Drive Pillars:

Results on the Analogue Resistance machine indicate that when stoping commences the average stress on an elastic drive pillar at a depth of 5,500 ft. is 6,000 lbs./ins.^{2}. This stress increases to 8,000 lbs./ins.^{2} when the stope remnant has been removed. When stope pillar removal is completed and the stope excavation is very long on strike, the stress on the drive pillar edges is 50,000 lbs./ins.^{2},

(FIG. 27) / .....

(FIG. 27), although at the pillar centre it is still about 6,000 lbs./ins.^{2}. The average stress on a drive pillar may reach the very high level of 100,000 lbs./ins.^{2}, (FIG. 27) when the other drive pillars have been removed. The results of the photo-elastic analysis on the St. Helena stope model show that large stress concentrations occur on the stope side of the pillar. Inside the pillar and round the drive excavation, the concentration of stress is small, (except in the corners), and the magnitude is approximately one and a quarter times the overburden stress. At a depth of 5,000 ft. below surface the stress round the drive varies between 4,000 and 6,000 lbs./ins.^{2}. From these theoretical results it can be expected that the side of the drive pillar forming the stope excavation will fracture. Due to the low stresses, fractures around the drive perimeter will not extend and only small movement of the rock may be expected.

Underground observations confirm that the edge of the pillar in the stope excavation is fractured, and scaling of the sidewall often occurs. "In-situ" stress measurements in the drive pillar have been inconclusive. At a point 8 ft. inside the pillar from the stope sidewall, an increase in stress of 8,000 lbs./ins.^{2} has been measured. Around the perimeter of the drive a closure of $\frac{1}{2}$ ins. only was measured during stope pillar removal and the year following the removal. It can be concluded from laboratory and underground measurements that drive pillars are mainly solid up to the removal stage.

A solid drive pillar can support a Voussoir Arch of 420 ft. span, (Sec. 3.2), and abutments can form along the whole strike length, both at the top and bottom of the stope excavation. Therefore a solid drive pillar can be expected to stabilize hangingwall movement over a large area of the stope. Underground measurements show that the ride movement in the stope excavation is less than $\frac{1}{2}$ ins. Since the drive pillar remains solid during the stoping operation, its effect on hangingwall stability is not decreased until mining of the drive pillar commences.

Extensive cracking of the stope pillar will result in

greater / .....

greater closure of the stope excavation and an increase in the rate of energy release. Therefore the energy release rate during normal stoping approaches that shown in FIG. 29 for the rate of energy release when mining without pillars. No rock bursts have been experienced at 5,500 ft. depths in any of the stopes mined out. It must therefore be assumed that the energy release rate of 8×10^6 ft. lbs. per fathom, that results from mining out the last few fathoms in the stope, is absorbed by the hangingwall fracture zone, (Sec. 3.3). Due to the solidity of a 150 ft. wide drive pillar, this rate of energy release will not be exceeded during stoping, and no rock bursts should occur.

It has been the mine's policy to remove drive pillars, and removal at a depth of 4,000 ft. over a limited area has not resulted in rock bursts. FIG. 30 shows that the removal of a pillar at 5,500 ft. in an area of 70 per cent extraction will result in an energy release rate of 10^8 ft. lbs. per fathom. This is much higher than the theoretical danger level and much greater than the energy absorption figure of 8×10^6 ft. lbs. per fathom given previously. More work is necessary to determine the maximum rate of energy absorption. Until this maximum rate is known it must be assumed that drive pillar removal at a depth of 5,500 ft., particularly in areas of large percentage extraction, will result in rockbursts.

8. CONCLUSIONS

The results of underground tests show that the stope excavation is surrounded by a fracture zone, (Sec. 6.2.3. and Sec. 6.1.4). The size of the zone was not measured, but the height of the zone was shown to be about the same as the height of the tensile zone, (Sec. 7). The height of the tensile zone in relation to span is shown in FIG. 12. In Sec. 3.2 this figure was used to determine the span at which hangingwall instability might occur. The magnitude of this span was in close agreement with the average span at which the original hangingwall collapses occurred.

Visual observations at a depth of 5,500 ft. show that stope pillar sides fracture as the pillars are formed, and these fractures increase in size as the stope excavation increases. A comparison of the underground closure with the theoretical elastic closure showed that a fracture zone exists around a stope pillar, (Sec. 6.2.3). However, stress measurements made at the pillar centre proved that the rock behaved elastically whilst the pillar was being formed. It is possible that these measurements were made in blocks of fractured rock with each block behaving elastically. In fact, underground observations and theoretical stress calculations indicate that fractures may extend into the centre of the pillar and that strength failure might occur[22]. No pillars have been observed to disintegrate and it must be assumed that pillars still have a load bearing capacity which reduces with pillar deformation. Due to the failure of the pillar its average stress level will be less than the theoretical estimate and there will be no danger of the pillar bursting during removal.

Failure of stope pillars reduces their load bearing capacity and the load shed by the pillars must be carried by the other supports. At depths greater than 5,500 ft. stope pillars will develop extensive fractures and rapid pillar failure may occur. The matpacks may not be able to carry the loads shed by the pillars and unstable hangingwall conditions will result.

Analysis of stress in mine models shows that the drive pillars

are / .....

are not highly stressed, (Sec. 5.2), and it was concluded in Sec. 7.2 that drive pillars are solid. Drive pillars carry some of the load shed by the fracturing of stope pillars, and the loading is increased further when the stope pillars are removed. Removal of stope pillars results in comparatively little closure and ride within the stope excavation, and it is concluded that these movements are small because of the solidity of the drive pillar and the stability of the system.

No rock bursts have occurred at St. Helena during the mining of stope excavations at depths less than 5,000 ft. The energy release rate, although high, must be absorbed by the hangingwall fracture zone, and it may be concluded that the present drive pillar system reduces the possibility of the occurrence of rock bursts, (Sec. 7.2). This conclusion is born out by theory, see Sec. 3.3, assuming that drive pillars are not removed.

Laboratory tests were carried out on the Resistance Analogue, Sec. 5.3. The results indicate that rock bursts would occur if drive pillars were mined out completely, or if large sections of pillar were removed in areas with a high percentage extraction.

8.1 Recommendations:

From these conclusions, recommendations can be made for the support system at No. 8 Shaft on St. Helena Mine. Stoping at this shaft will extend to a depth of 6,500 ft. The dip of the Basal Reef, as indicated by the shaft intersection, will be 40° possibly decreasing to 30° towards the President Brand boundary. The stoping width will be 36 ins. or less in the steeper stopes, since the values lie mainly on the reef contact.

It is recommended that stoping be carried out by the method shown in FIG. 18, in which the stope face advances on strike in the direction of the long axes of the panel. The distance L between the stope centre lines can be determined approximately from FIG. 16 and it is suggested that the maximum length of stope back be 800 ft. with a barrier pillar of 150 ft. wide on dip, (Sec. 7.2 and Sec. 3.3).

This / .....

This general method of mining should be combined with a low stoping width. The strike barrier pillar will have a stabilizing effect on the stope hangingwall, which, due to the steepness of the stope, will tend to move down dip. Since the barrier pillar must be maintained at its full width, the main haulage will have to be developed in the footwall quartzites.

The internal stope support method can now be determined, assuming a maximum span of 800 ft. on dip. It must be assumed that the internal pillar at greater depths will be highly fractured. An increase in the size of this pillar to reduce the effect of fracturing is not recommended, since it may then present a rock burst hazard. The use of internal stope pillars as means of support should therefore cease and stressed composite matpacks spaced at a smaller interval is recommended. A calculation for the spacing of 2 ft. 8 ins. matpacks, (3 ft. 0 ins. high composed of alternate layers of timber and concrete), is given in APPENDIX IV. It is assumed in the calculation that planes of weakness occur at certain intervals in the hangingwall up to a height of at least 100 ft. above the reef plane. The height of the tensile zone for a span of 800 ft. at a depth of 6,500 ft. can be calculated from FIG. 12 and it is assumed that the hangingwall beds will separate at this height, and will form a zone of separated rock similar in shape to the tensile zone. It is further assumed that the weight of this rock will be carried by matpacks, pre-loaded to 50 tons and that no self-supporting rock arches exist. This is the worst possible loading condition that can occur. The matpack spacing is calculated to be 8.0 ft. for a 2 ft. 8 ins. matpack 3 ft. 0 ins. high and it should be composed of alternate layers of timber and concrete.

PHOTO-ELASTIC ANALYSIS

A. The relationship between principal stress difference, model thickness, material constant and fringe order for a birefringent material is :-

$$\frac{p - q}{2} = \frac{NF}{2t}$$

Where p is the major principal stress

q is the minor principal stress

N is the fringe order

F is the material constant (83.5 lbs./ins.^{2}/fringe)

t is the material thickness (.25 ins.).

Along the edge where the load is applied the minor principal stress is zero and the major principal stress is due to the applied load.

$$\therefore \quad \frac{p - q}{2} = \frac{p}{2} = \frac{620 \text{ lbs.}}{\frac{1}{4} \text{ ins.} \times 9 \text{ ins} \times 2} = \frac{N\ 83.5}{2 \times \frac{1}{4}}$$

$$\therefore \quad N \frac{620}{83.5 \times 9} = \frac{620}{751.5} = .83$$

Therefore,, the fringe order at this edge is = .83 and is called the nominal fringe order. At any other point in the model where 'q' is not zero, then if 'p' is the major principal stress due to the applied load, (i.e. the overburden stress),

$$\frac{p - q}{p_1} = \frac{N}{.83}$$

$$\therefore \quad p - q = 1.2 \quad p_1 \ N$$

and the principal stress difference at any point can be obtained directly from the fringe order there.

B. At the intersection of isochromatic fringes and equipotential lines obtained from the resistance paper, then

$$p - q = 1.21p_1N, \text{ and } p + q = 1.21p_1N_2$$

$$\therefore \quad p = \frac{N + N_2}{2} \quad 1.21p_1 \text{ and } q = \frac{N_2 - N}{2} \quad 1.21p_1$$

Thus / .....

Thus separation of the principal stresses is effected. Since actual intersection of fringes and equipotential lines is not frequent, contours can only be drawn after interpolation has produced a sufficient number of points on the curve.

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