Stresses Induced by Mining Operations at Mount Charlotte

By

W. E. Bamford, B.E., A.M.Aus.I.M.M.

(Senior Lecturer in Rock Mechanics, University of Melbourne)

SUMMARY.- Mount Charlotte is a mechanized underground mine, extracting low-grade gold ore, at Kalgoorlie, Western Australia. Following a "bump", or partial brittle failure of a rib pillar, a quick series of rough calculations, utilising published elastic theory stress concentration factors, was carried out to determine whether the overstressing of this pillar could have been predicted, and, if successful, whether this rock mechanics approach could be used to suggest modifications to the mining methods. The previously-measured high horizontal stress field was assumed to be consistent with Hast's measurements, and calculations showed that the stress induced in the rib pillar after partial removal of the crown pillar was of the same order as the rock mass strength, so rendering it logical that failure would ensue. Another rib pillar was shown to be in a slightly less highly stressed state, and a prediction that it would be safe until its planned extraction was proved to be correct. A more refined recalculation is in progress.

I.- INTRODUCTION

Gold Mines of Kalgoorlie Ltd.(GMK) operates a large underground mine at Mount Charlotte, on the edge of the town of Kalgoorlie, Western Australia. The orebody is almost vertical, about 700 feet long (azimuth 142°True), varying in plan width from 100 to 200 feet. The host rock is a massive quartz dolerite greenstone of Archaean age, and the gold ore occurs associated with pyrite mineralization and quartz veins. The ore grade is approx. 3 penny-weights per ton (approx. 4.5 grams per metric ton).

The mining method adopted in A Block (0 to 500 feet below the surface) was cut and fill, with transverse support pillars. This method was designed with the aid of the Snowy Mountains Hydro-Electric Authority (S.M.A.), in 1963 (Ref.1,2). In B Block (500 to 900 feet below the surface) the mining method is sub-level open stoping, with open stopes 300 feet high, 100 to 180 feet wide and 80 to 180 feet long, separated by rib pillars 80 to 90 feet thick (in plan). Open stopes are filled with broken ore by blasting adjoining rib- and crown-pillars. As the broken ore is drawn off at the 860 ft. and 900 ft. levels, a plug of dry fill from A Block, continuously replenished by dumping into an old open-cut at the outcrop, moves down on top of the subsiding ore.

Fuller descriptions are given in Refs.3 and 4.

II.- FAILURE OF RIB PILLAR B2

At 5.23 a.m. on 1st April 1970 a seismic event was recorded on the Kalgoorlie seismograph of the Bureau of Mineral Resources. Gregson (Ref.5) states that this event had relative amplitude 15 (= approx. Richter magnitude 0.7) and strain release 18×10^6 (ergs). The "bump" was heard by many persons over a wide area of the Kalgoorlie-Boulder district. Later that morning mining engineer R.A.Tastula carried out inspections and noted (Ref.6):

(a) On the 600 level cracking and scaling were evident in the P.B.2 Rib Pillar cross cut and in the west wall of the stripping drive Approx. 10-15 tons of rock had fallen in this area. The rails had been moved 6 inches off line.

(b) On the 700 level similar cracking had occurred and about 20 tons had fallen.

(c) On the 800 level cracking had occurred in several places along the P.B.2 Rib Pillar cross-cut and also on the N.E. and S.W. corners of the pillar.

(d) On the 900 level major scaling had taken place along the walls of eastern millholes in P.B.2 and S.B.2. On the western side, only the most southern millholes, No.5, had shown signs of movement.

From these observations he concluded that the pillar had been subjected to excessive stress from the walls towards the centre.

Later that month the author was invited to make an inspection. The pattern of cracking and fracturing within Rib Pillar B2 was strongly suggestive of failure due to compression along the East-West axis of the pillar. In the central pillar axis drives the walls typically showed cracking and slabbing parallel to the side walls, with some vertical tensile cracking in the roofs. This could indicate that the major stresses were either vertical or East-West. However, in transverse (North-South) drives, slabbing and spalling parallel to the roof and vertical tensile cracking normal to the walls indicated that the major stress was acting in an East-West direction. (See Fig.1)

A survey pin in the centre of the pillar on the 800 level was re-surveyed, to give an indication

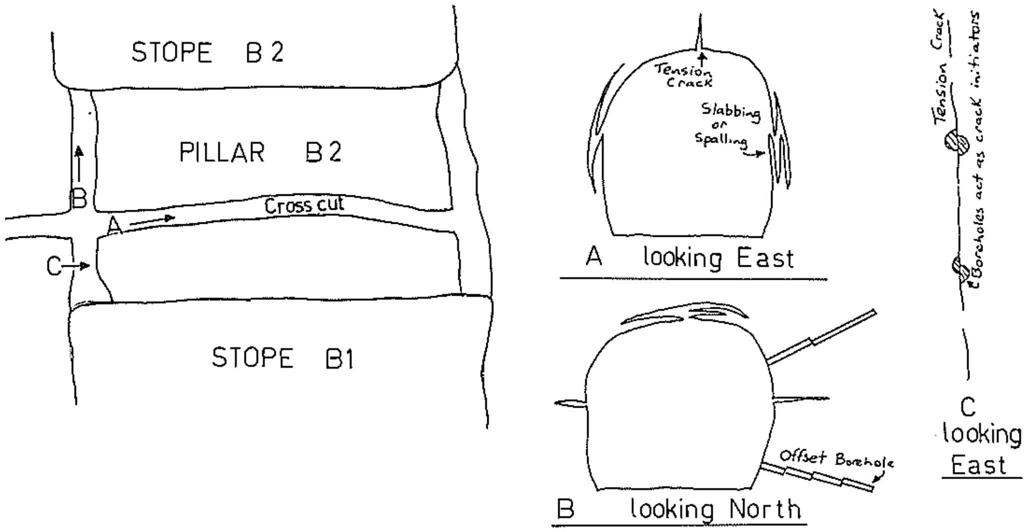

FIG.1. SKETCHES OF TYPICAL FAILURE PHENOMENA IN RIB PILLAR B.2

of the movements.

December 1969	9274.80N	R.L.=408.05
24th April 1970	9274.68N	R.L.=408.07
Movement =	0.12foot North	0.02 foot up

This movement is more consistent with a horizontal East-West compression, causing slight lateral (Northwards and upwards) buckling than with a vertical compression.

III.- ESTIMATION OF VIRGIN FIELD STRESSES

The virgin rock stresses measured by the SMA (Ref.1) on the 300 and 500 levels were of the order of 2500 lb/in,^{2}(p.s.i.) along the long axis of the orebody (henceforth referred to as North), 1500 p.s.i. across the orebody (henceforth referred to as East), and 1500 p.s.i. vertical. The sum of the horizontal stresses is consistent with the relationship postulated by Hast (Ref.7)

$\sigma_1 + \sigma_2 = 191 + 0.99\ H$ kgf/cm^2 (with H in meters) or

$\sigma_1 + \sigma_2 = 2716 + 4.36\ H$ p.s.i. (with H in feet).

At a depth of 400 feet Hast's equation indicates that the sum of principal stresses is approx.4,400 p.s.i., compared with the measured 4,000 p.s.i. (if the 300 and 500 level results, which show no increasing or decreasing trend with depth, are averaged and assumed to be valid for a depth of 400 feet). Stresses at any other depth were inferred as shown in Fig.2, with the ratio of σN to σE assumed constant at 5:3, and σV increasing as γH.

The deduction of stresses concentrated by the large excavations presented a formidable problem. No three-dimensional method was feasible within the limitations of cost and the short time within which results were required. The earlier photoelastic studies by the SMA (Ref.2) had been limited to boundary stresses of the A Block stope, with no results that could be confidently extrapolated to B Block. The finite element method of computer analysis showed promise, but, paradoxically, was thought would probably take longer to obtain an answer (albeit possibly more correct) from, because of "debugging" and time use limitations, than a deductive or analytical method. Also, the data available on material properties and excavation shapes were not very voluminous or reliable, rendering exact methods of solution less appropriate. So, it was decided to study the two-dimensional stress concentrations around each stage of excavation, superimposing and sythesising, to arrive at the overall picture.

IV.- ESTIMATION OF STRESS CONCENTRATIONS

Following the comments of personnel who had tested rock specimens from Mt.Charlotte at the S.M.A Cooma, and the Australian National University, Canberra, and personal observations on the rock mass the assumption of perfectly elastic behaviour of the rock can be made with only slight error. Simplified versions of the shapes of the excavations were

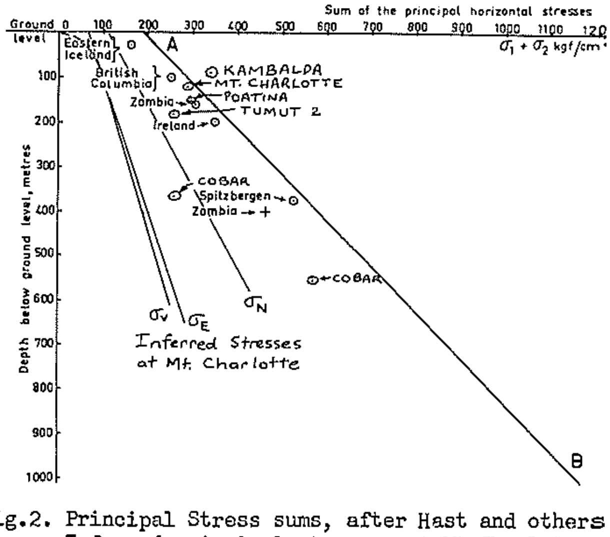

Fig.2. Principal Stress sums, after Hast and others
Inferred principal stresses at Mt.Charlotte.

assumed, and stress concentrations calculated by Savin (Ref.8) were followed.

The effect of the mined-out A Block notch, open to the surface, was approximated by taking the stress concentrations due to the lower half of a rectangular hole. Although this is obviously not the same model, a quick comparison of notch stresses (Ref.9) with hole stresses (Ref.8) indicated that they were reasonably similar, in magnitude and distribution pattern. So, an East-West cross-section through A Block, generalized as being 500 feet deep and 150 feet wide, was simulated by the lower half of a rectangle with rounded corners, 1000 feet high and 150 feet wide. (See Fig.3)

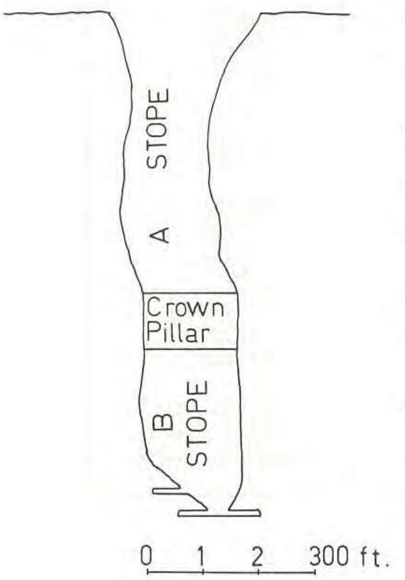

Fig. 3 Typical West-East Cross-section

In a similar fashion, in a North-South longitudinal section the effect of a notch 540 feet long and 500 feet deep on the North-South field stresses was evaluated. (See Fig.4).

Table 1 shows the calculated average stresses existing in the orebody after removal of A Block.

TABLE I

CONCENTRATED STRESSES BELOW A BLOCK

Depth	S_E	S_N	S_V(lbs/sq.in.)
500 ft.	15,200	10,500	0
550	12,500	7,400	400
600	9,200	4,900	1,300
650	5,300	4,700	1,800
700	4,700	4,700	2,100
750	3,800	4,700	2,100
800	3,200	4,700	1,800
850	3,200	4,800	1,900
900	3,200	4,900	2,100

(Note: S_E = Concentrated stress in East direction,
N = North,
V = Vertical

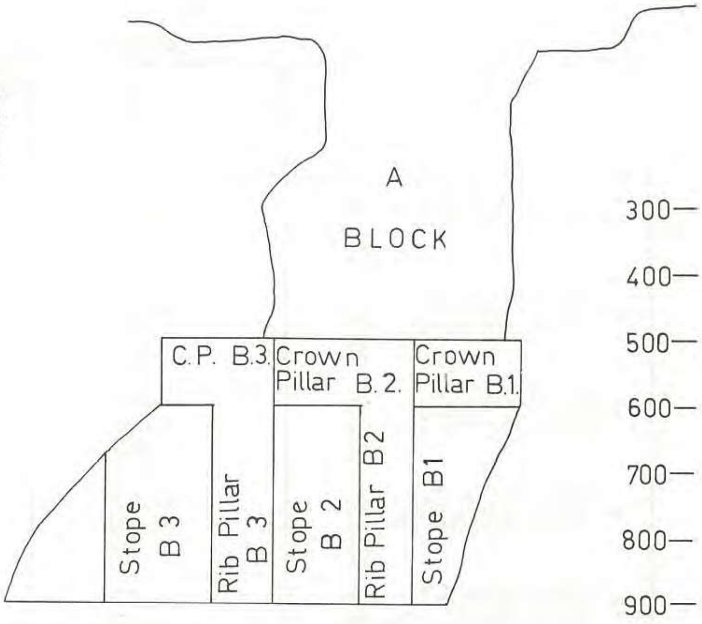

Fig.4 Typical North-South Cross-section

The next phase of excavation simulated was the opening of the stopes in B Block, from 600 to 900 feet below the surface. The effect on the average stresses in the crown pillar is shown in Table 2.

TABLE II

CONCENTRATED STRESSES IN CROWN PILLAR ABOVE B STOPES

Depth	S_E	S_N	S_V (lbs/sq.in,)
500 ft.	15,600	10,600	0
550 ft.	13,500	7,600	-1900
600 ft.	13,400	5,500	0

The next model was the plan outline of the stopes generalized as rectangles with rounded corners (See Fig.5). The stress concentration factors, in a horizontal plane, about each stope could then be calculated, and multiplied by the concentrated stresses due to the proximity of the A Block notch, to give the doubly-concentrated stresses around the B Block stopes. By the principle of superposition the stresses in the Rib Pillars could then be estimated. It was realized that while the two-dimensional model of stress concentrations across the long axis of the excavation would be valid in the central portion, near the ends the stresses would be changing. It was assumed that the zone of uniform concentrated stresses would extend to within a distance of one stope width (i.e. 120 to 160 feet) of the end of the excavation viewed in the North-South longitudinal section, and that the stresses would then undergo a transition, returning to their undisturbed value at a distance of two stope widths from the end of the excavation (c.f. St.Venant's principle).

A similar rectangle model was applied to the concentrated stresses in the crown pillar, to study the effect of the removal of Crown Pillar B.1 in December 1969, an event which was followed within 4 months by the "bump" in the adjoining pillar. The

TABLE III

DEDUCED AVERAGE STRESSES IN PILLARS (lbs/in.^{2})

		Stage of Mining			
		A After Removal of A Block	B After Opening of B Block Stopes	C After Firing of Crown Pillar B.1	D After Firing of Crown Pillar B.2
1. Crown Pillar B.1 Depth 500-600ft.	S_E	10,200	11,700	-	-
	S_N	6,700	6,900	-	-
	S_V	600	-100	-	-
2. Rib Pillar B.2 Depth 600-800ft.	S_E	4,800	10,100	17,700	-
	S_N	4,700	1,000	1,000	
	S_V	1,900	3,700	4,600	-
3. Rib Pillar B.2 Depth 500-600 ft.	S_E	12,800	12,800	18,000	-
	S_N	7,600	7,600	2,700	-
	S_V	300	300	20	-
4. Crown Pillar B.2 Depth 500-600 ft.	S_E	12,800	14,300	15,200	-
	S_N	7,600	7,900	5,600	-
	S_V	300	-600	-600	-
5. Rib Pillar B.3 Depth 600-800 ft.	S_E	4,200	11,300	16,700	14,900
	S_N	4,400	1,600	2,000	2,100
	S_V	1,900	3,700	4,700	4,700
6. Rib Pillar B.3 Depth 500-600 ft.	S_E	10,100	10,100	13,800	16,700
	S_N	6,700	6,700	6,900	3,000
	S_V	600	600	600	400
7. Crown Pillar B.3 Depth 500-600 ft.	S_E	3,800	4,200	Probably little changed from previous stage	10,700
	S_N	3,300	3,900		5,000
	S_V	1,400	1,200		1,200

variation of stresses with height in the rib pillars after the removal of A Block was plotted, the average stress in the crown pillar at this stage compared with that after the removal of Crown Pillar B.1 to indicate a further proportional increase, and the stress concentration factors at various depths within the pillar similarly proportionately adjusted, to give an indication of the rib pillar stresses after the partial removal of the crown pillar.

The effect of further removal of Crown Pillar B.2 was also modelled, to attempt to predict whether its removal, in July 1970, would be followed by catastrophic failure, or whether the adjoining Pillar 2 would remain intact until the scheduled firing in December 1970.

The final results of much manipulation and calculation, the deduced progressive changes in stresses during mining, are shown in Table 3. Each stress quoted is the average over the specified portion of a pillar. The apparent differences in stresses between contiguous blocks of rock do not reflect sudden stepwise changes, but rather gradual variations over tens of feet.

V.- ESTIMATION OF ROCK MASS STRENGTH

It is of interest to observe in Table III that the stresses in Units 4 and 3 (Crown Pillar B.2 and Rib Pillar B.2 from 500 to 600 feet) at Stage C (after firing of Crown Pillar B.1) are somewhat higher than those acting on any other Unit at any stage. As the "bump" took place within Units 4 and 3 at Stage C,

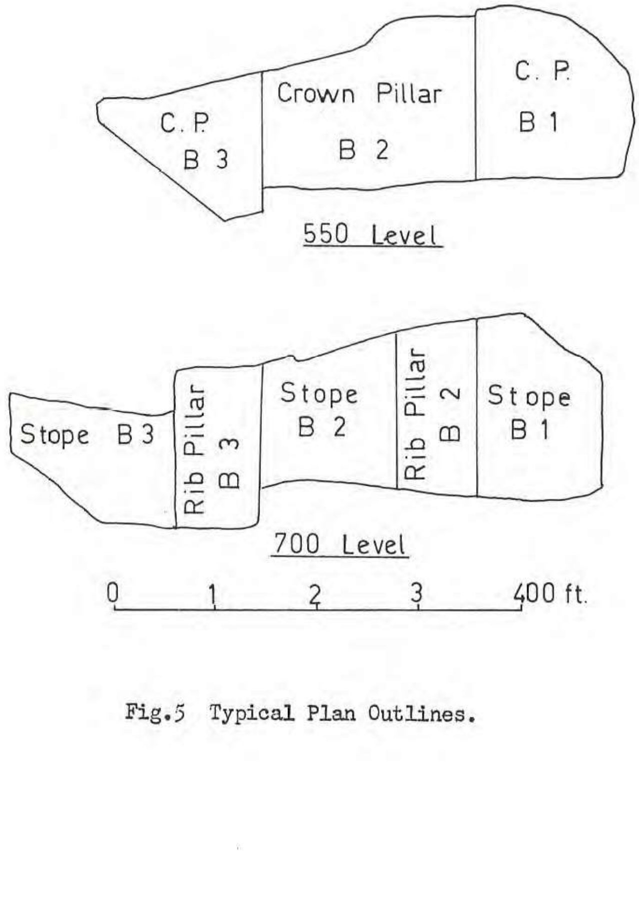

Fig.5 Typical Plan Outlines.

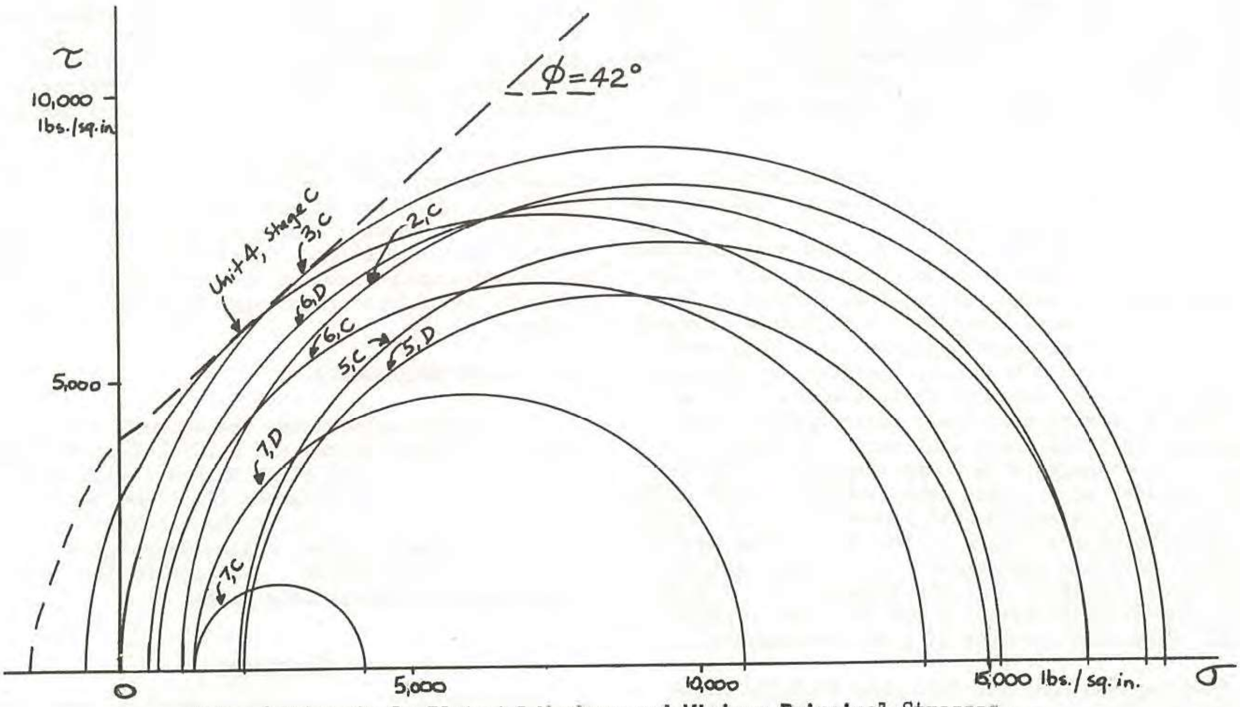

Fig. 6 Mohr Circle Plots of Maximum and Minimum Principal Stresses.

this can be regarded as a large-scale in-situ compression test to failure, and the inferred strength used as a guide in checking stability of other parts of the mine.

Mohr circle plots of the maximum and minimum stresses for Units 4 and 3 at failure (15,200 and -600; 18,000 and 20 lbs/sq.in. respectively) indicate that a straight line with apparent cohesion =4,000 lbs/sq.in., apparent angle of shearing resistance ϕ = 42° can be drawn tangent to them, and this can be taken as indicating an in-situ failure criterion for the rock mass (See Fig.6). Mohr circles for the worst stress conditions in each of the other Units can be drawn, and the apparent factors of safety deduced.

TABLE IV

APPARENT FACTORS OF SAFETY

Unit	Stage <u>C</u>	Stage <u>D</u>	(Terminology as in Table III)
2	1.19		
3	1.00		
4	1.00		
5	1.47	1.36	
6	1.30	1.11	
7	4.9	1.82	

With these figures available,it becomes quite apparent that when Units 3 and 4 failed, the next most highly stressed Unit, 2 (Factor of Safety 1.19) should be the most likely Unit to fail as load was shed onto it by the partially failed Units, so leading to failure of Units 2, 3 and 4 - the entire Pillar B.2. It is also reasonable that Unit 6, the most highly-stressed portion of Pillar B3, with a factor of safety of 1.30 should stand safely, without danger of failure, for a few more months, from the April prediction until the July firing of Pillar B2, as in fact happened.

When Stage D was reached, in July 1970, the likelihood of a catastrophic failure of Pillar B3 before it was due to be fired in December was of interest to the mine management. Although other commitments delayed the analysis, the author was able to give the opinion in August that if a factor of safety safety of 1.00 implied failure 3½ months after loading, a factor of safety of 1.11 (for Unit 6) should allow the required 5 months' standing time before failure. An additional strengthening factor appeared to be the fact that the contiguous Unit 5 had a substantially higher factor of safety. No failure had been observed before Pillar B3 was successfully blasted,in December 1970.

Another interesting sidelight to the above analysis was provided by the blasting of an undercut to Pillar B3, from the 860 to the 900 feet level, several weeks before the main pillar blast, to improve fragmentation and throw into the adjacent open stopes. As long as it was thought that the pillars transmitted mainly vertical stresses it was unthinkable to undercut them without introducing the likelihood of total collapse. When the implications of the above analysis were pointed out, that the pillars were mainly transmitting horizontal (East-West) stresses, it became evident than an undercut would not greatly affect their stability. In the event, the undercut was successfully accomplished, and the fragmentation of Pillar B3 was much better than that of Pillar B2.

VI.- LABORATORY PREDICTION OF ROCK MASS STRENGTH

Before uncritically accepting the results of an in-situ strength test, they should be compared with any available laboratory test results. As part of the 1963 investigations (Refs.1 and 2) the S.M.A.

carried out unconfined compressive and tensile strength tests on BX rock core samples supplied by G.M.K. from Mt.Charlotte. The mean compressive strength was found to be 26,800 lbs/sq.in., with a standard deviation of 6,200 p.s.i.. The mean tensile strength was 2,130, with a standard deviation of 800 p.s.i.

Before the results of tests on cylindrical pieces of rock 1 5/8 inch diameter by 1.9 to 3.8 inches long can be applied to the strength of a rectangular prism of rock 400 feet by 150 feet by 90 feet some allowance for scale factors must be made. Several authors in the past have suggested scaling-down factors as high as 100 i.e. rock mass strength = 0.01 sample strength. More recently it has been suggested that such scale factors may be valid for soft, yielding, or intensely jointed rocks, but for hard brittle rocks, such as quartzite or norite much lower scale factors apply. Bieniawski (Ref.10) shows that hard rock samples decrease in strength with increasing size until they reach the size of a 5 inch cube, whereupon they suffer no reduction in strength with increasing size. Applying his results for norite to the case of the Mount Charlotte dolerite greenstone (a somewhat similar rock) it can be shown that the strength of a 5 inch cube - and by implication, a 100 foot cube - should be 0.77 times the strength of a BX core sample.

This implies that the rock mass strength parameters are: Unconfined compressive strength =20,800, Standard Deviation = 4,800 p.s.i.; Tensile strength = 1640, Standard Deviation = 620 p.s.i. The observations, shown on Fig.6, are of an unconfined compressive strength of about 18,000 p.s.i. If this is regarded as the result of a precise computation of stresses, it implies that failure took place at a stress 0.58 standard deviations below the mean strength, which in its turn implies a 28% chance of failure, and renders the eventual failure quite logical. On the other hand, the computations were in fact not precise, and for the actual stresses to be within 10 to 20% of the computed stresses is all that could reasonably be expected of them.

The predicted tensile strength is -1640 p.s.i., while the maximum tensile stress predicted, in Crown Pillar B2, was of the order of -600 p.s.i.. It appears logical then that the pillar should have suffered compressive failure in Stage B rather than tensile failure in Stage C.

VII.- CONCLUSIONS

The rock mechanics approach to determining the stability of mine structures is :(a) deduce virgin stress field, (b) determine stress concentration factors around excavation shapes, (c) predict concentrated stress magnitudes, (d) compare these with rock mass strength to determine factors of safety. The validity of this approach was demonstrated satisfactorily at Mt.Charlotte, by being able, by back analysis, to explain a pillar failure, and by forward prediction, to predict that another pillar would not fail.

It is now being used to study and modify the mining method planned for extracting the deeper C Block (from 900 to 1150 feet below the surface). A preliminary suggestion is that it may be almost impossible to avoid overstressing pillars of economic size in the higher stress field conditions prevailing at the greater depths, and that the orebody may be more easily mined as one large open stope, with de-stressing of critical parts of the barren wall rock to reduce risk of catastrophic failures. This approach has to be modified because of the vagaries of ore grade distribution, resulting in the desirability of leaving one pillar of very low grade ore unmined.

A more detailed (and hopefully, more precise) investigation, with the aid of finite element programs developed by Mr.A.G.Bennet, a Ph.D. candidate in the Melbourne University Mining Department, is underway at the time of writing. Comparisons of the results of this investigation and the more elementary study forming the bulk of this paper will be presented when appropriate.

VIII.- ACKNOWLEDGMENTS

I wish to acknowledge the support and encouragement of Mr.Clive Annear and Mr.Dick Tastula (respectively Manager and Assistant Manager, Mt.Charlotte Project), Mr.Gordon Anderson (Resident Manager, Gold Mines of Kalgoorlie) and Mr. Bob Nichols (Senior Mining Engineer, Western Mining Corporation). I am grateful to Gold Mines Kalgoorlie Ltd. for permission to publish this paper.

REFERENCES

1. BOWLING, A.J. - Rock Stress Measurements in the Mt. Charlotte Mine Kalgoorlie, Western Australia. Snowy Mountains Hydro-Electric Authority, Scientific Services Division, Engineering Physics Rept. No. S.E. 56, August 1963.

2. BOWLING, A.J. - A Photoelastic Investigation For The Design Of The Mt. Charlotte Stope, Kalgoorlie, Western Australia. Snowy Mountains Hydro-Electric Authority, Scientific Services Division, Engineering Physics Report No. S.E. 57, November 1963.

3. SIMPSON, R.G. - Operations at Mt. Charlotte. Eighth Commonwealth Mining and Metallurgical Congress, 1965 - Paper 125.

4. ANON. - Mt. Charlotte. An open cut gold mine underground. Australian Miner October 20, 1969.

5. GREGSON, P.J. - Letter from Mundaring Geophysical Observatory, W.A., to the Resident Manager, G.M.K. 10th April 1970.

6. TASTULA, R.A. - Report to the Manager, Mt. Charlotte Project. 2nd April 1970.

7. HAST, N. - The State of Stress In The Upper Part Of The Eerth's Crust. Tectonophysics, Vol. 8, 1969, p. 169-211.

8. SAVIN, G.N. - Stress Concentration Around Holes New York, Pergamon, 1961.

9. STURGUL, J.R. amd SCHEIDEGGER, A.E. - Tectonic Stresses in the Vicinity of a Wall. Felsmechanik Vol. V, No. 2-3, 1967, p. 137-149.

10. BIENIAWSKI, Z.T. - The Compressive Strength of Hard Rock. Tydskrif vir Natuurwetenskappe, Vol. 8 No. 3, Sept. 1968, p. 163-182.