CANADA

DEPARTMENT OF
ENERGY, MINES AND RESOURCES
MINES BRANCH
OTTAWA

A ROCK MECHANICS CASE HISTORY OF ELLIOT LAKE

D. F. COATES, H. U. BIELENSTEIN AND D. G. F. HEDLEY

MINING RESEARCH CENTRE

Reprinted from Canadian Journal of Earth Sciences,

Volume 10, Number 7, pp 1023-1058 July 1973 Ottawa

Reprint Series RS 127

Price: .25 cents

Available by mail from Information Canada, Ottawa, K1A 0S9
and at the following Information Canada bookshops:

HALIFAX
1683 Barrington Street

MONTREAL
640 St. Catherine Street West

OTTAWA
171 Slater Street

TORONTO
221 Yonge Street

WINNIPEG
393 Portage Avenue

VANCOUVER
800 Granville Street

or through your bookseller

Price: .25 cents Catalogue No. M38-8/127

Price subject to change without notice

Information Canada
Ottawa, 1974

Canadian Journal of Earth Sciences

Journal canadien des sciences de la terre

Published by
THE NATIONAL RESEARCH COUNCIL OF CANADA

Publié par
LE CONSEIL NATIONAL DE RECHERCHES DU CANADA

Volume 10 Number 7 July 1973

Volume 10 numéro 7 juillet 1973

A Rock Mechanics Case History of Elliot Lake

D. F. COATES,
Mining Research Centre, Mines Branch, Department of Energy, Mines, and Resources, Ottawa, Canada

H. U. BIELENSTEIN,
Western Office, Mining Research Centre, Mines Branch, Department of Energy, Mines, and Resources, Calgary, Alberta

AND

D. G. F. HEDLEY
Elliot Lake Research Laboratory, Mining Research Centre, Mines Branch, Department of Energy, Mines, and Resources, Elliot Lake, Ontario

Received February 8, 1973

Revision accepted for publication March 9, 1973

The Elliot Lake area is characterized by Proterozoic sediments containing uranium-bearing conglomerates separated by quartzite beds 10 to 100 ft (3.0 to 30.5 m) thick. The geological structure consists of a broad syncline with an east–west axis plunging about 5° W, cut by northwest trending faults, and with steeply dipping east–west extension joints. All the mines use a stope-and-pillar method of extraction with narrow rib pillars about 250 ft (76 m) long on dip and sill pillars on strike.

After the Elliot Lake Laboratory was established, detailed studies were undertaken to evaluate the methods that were available for the determination of the mechanical properties of the rock mass and its state of stress before mining. Practical studies were then made on the pillars, roofs, and abutments.

Testing techniques were improved for the rock substance and the rock mass; however, much remains to be done to be able to characterize adequately the mechanical properties of the rock mass. A novel random sampling approach produced a suite of specimens many of which included fractures, with a mean uniaxial strength that was surprisingly little lower than the mean of only the solid specimens. The dispersion of values in such a suite was, of course, quite large. Of the other tests used, Brazilian tests were found to be useful for quality control of stress determinations using a strain recovery technique.

The use of borehole pressure cells, seismic velocity, and borehole penetrometers as techniques for the determination of the mechanical properties of the rock mass remains questionable.

The tectonic history of the region was resolved; it provides an explanation for the existence of horizontal stresses greater than vertical stresses and for the major principal stress to be oriented parallel to the axis of the syncline. It was also shown that the major principal stress axis is essentially parallel to the strike of extension joint surfaces, even when the strike deviates from the predominant 090° azimuth direction.

After considerable experience with mining in these geologic conditions, which probably are more uniform than in most metal mines, the determination of stable spans of stopes and breadths of pillars can be done very well by judgment. However, for examining new layouts relatively simple theoretical analyses, particularly for the determination of stable pillar sizes, were found to provide a rational and useful basis for extrapolation.

The stresses determined in pillars and abutment zones and the deformations of the roofs corresponded fairly well to values predicted by analytical and model techniques. The increased stress in the abutment zones extended into the solid for a relatively limited distance, which, in this relatively hard rock, seems to be related substantially to the span of the adjacent stope. All field measurements were subject to dispersion. The electrolytic analogue, which takes into account the three-dimensional aspects of the geometry of the tabular orebodies, showed that irregular mining boundaries have a distinct contribution to the variance of the pillar stresses. The finite element method was found to be flexible and useful in studying specific questions, particularly related to novel mining plans.

La région d'Elliot Lake est caractérisée par des sédiments contenant des conglomérats uranifères séparés par des lits de quartzite de 10 à 100 pi. (3 à 30.5 m) d'épaisseur. La structure géologique consiste en un grand synclinal avec des joints d'extension est–ouest à pendage abrupte, et dont l'axe est–ouest plongeant 5° ouest est coupé par des failles d'orientation nord–ouest. Toutes les mines utilisent une méthode d'extraction face d'abattage et pillier avec de minces pilliers d'environ 250 pi. (76 m) de long dans le sens du pendage et des pilliers de mur le long de la direction.

Après la création du laboratoire d'Elliot Lake, des études détaillées furent entreprises afin d'évaluer les méthodes disponibles susceptibles d'être utilisées pour déterminer les propriétés mécaniques de la masse rocheuse et son état de contrainte avant les travaux miniers. Des études pratiques furent alors effectuées sur les pilliers, toits, et aboutements.

Les techniques d'essai furent améliorées pour la matière rocheuse et la masse rocheuse; toutefois, il reste beaucoup à faire afin de pouvoir caractériser adéquatement les propriétés de la masse rocheuse. Une nouvelle méthode d'échantillonnage au hasard a donné une série de spécimens dont plusieurs contiennent des fractures et dont la résistance uniaxiale moyenne n'était, fait surprenant, que légèrement inférieure à la moyenne des échantillons solides. La dispersion des valeurs dans une telle série était, évidemment, assez grande. Parmi les autres essais utilisés, les essais brésiliens furent trouvés utiles pour le controle qualitatif des déterminations de contraintes en utilisant une technique de récupération de la déformation.

L'utilisation des cellules de pressione et de pénétromètres dans les trous de forage, et de la vélocité seismique comme technique de détermination des propriétés mécaniques de la masse rocheuse reste contestable.

L'histoire tectonique de la région fut résolue; elle a pourvu une explication à l'existence de contraintes horizontales plus intenses que les contraintes verticales et au parallélisme entre l'axe du synclinal et la contrainte majeure principale. Il a aussi été montré que l'axe de la contrainte majeure principale est essentiellement parallèle à la direction des plans des joints d'extension même lorsque cette direction dévie de son orientation prédominante d'azimuth 090°.

Après une expérience considérable avec les travaux miniers dans ce contexte géologique, qui est probablement plus uniforme que dans la plupart des mines de métal, la détermination d'étendues de surface d'abattage et de largeurs de pilliers stables peut très bien être faite au jugé. Cependant, il fut trouvé que pour examiner de nouveaux arrangements, des analyses théoriques relativement simples, particulièrement pour la détermination des dimensions de pillier stable, fournissent à l'extrapolation un fondement rationel et utile.

Les contraintes déterminées pour les zones de pilliers et d'aboutements et la déformation des toits correpondent assez bien aux valeurs prédites par les techniques analytiques et de modèle. La contrainte accrue dans les zones d'aboutement se prolonge dans le solide sur une distance relativement limitée ce qui, dans cette roche relativement dure, semble relié substantiellement a l'étendue de la face d'abattage adjacente. Toutes les mesures de terrain furent sujettes à la dispersion. L'analogue électrolitique, qui tient compte de l'aspect tridimensionnel de la géométrie des gisements tabulaires, a montré que les travaux miniers aux limites irrégulières contribuent distinctement aux variations de contraintes dans les pilliers.

Il a été trouvé que la méthode de l'élément fini est flexible et utile pour l'étude de questions spécifique, particulièrement en relation avec de nouveaux projets d'exploitation minière.

[Traduit par le journal]

Introduction

Geology

The Elliot Lake area lies near the southern margin of the Canadian Shield and is characterized by Proterozoic sediments which contain uranium bearing conglomerates, 6 to 30 ft (1.8–9.1 m) thick, near the contact with the Archean basement complex. Various reefs occur, separated by quartzite beds 10 to 100 ft (3.0 to 30.5 m) thick. The geologic structure comprises a broad syncline, plunging about 5° W, cut by northwest trending faults and westerly trending extension joints. The basement rocks and the sedimentary sequence have been intruded by numerous diabase dikes and sills (Bielenstein and Eisbacher 1969, 1970).

Mining

Twelve mines have been developed in this area; the maximum depth of mining is 3500 ft (1067.5 m) (see Fig. 1). All mines use a stope and pillar method of extraction with narrow rib pillars about 250 ft (76 m) long down dip. To get into production as fast as possible, most mines initially did their development within the orebody, advancing the workings away from the shafts to the property boundaries. Develop-

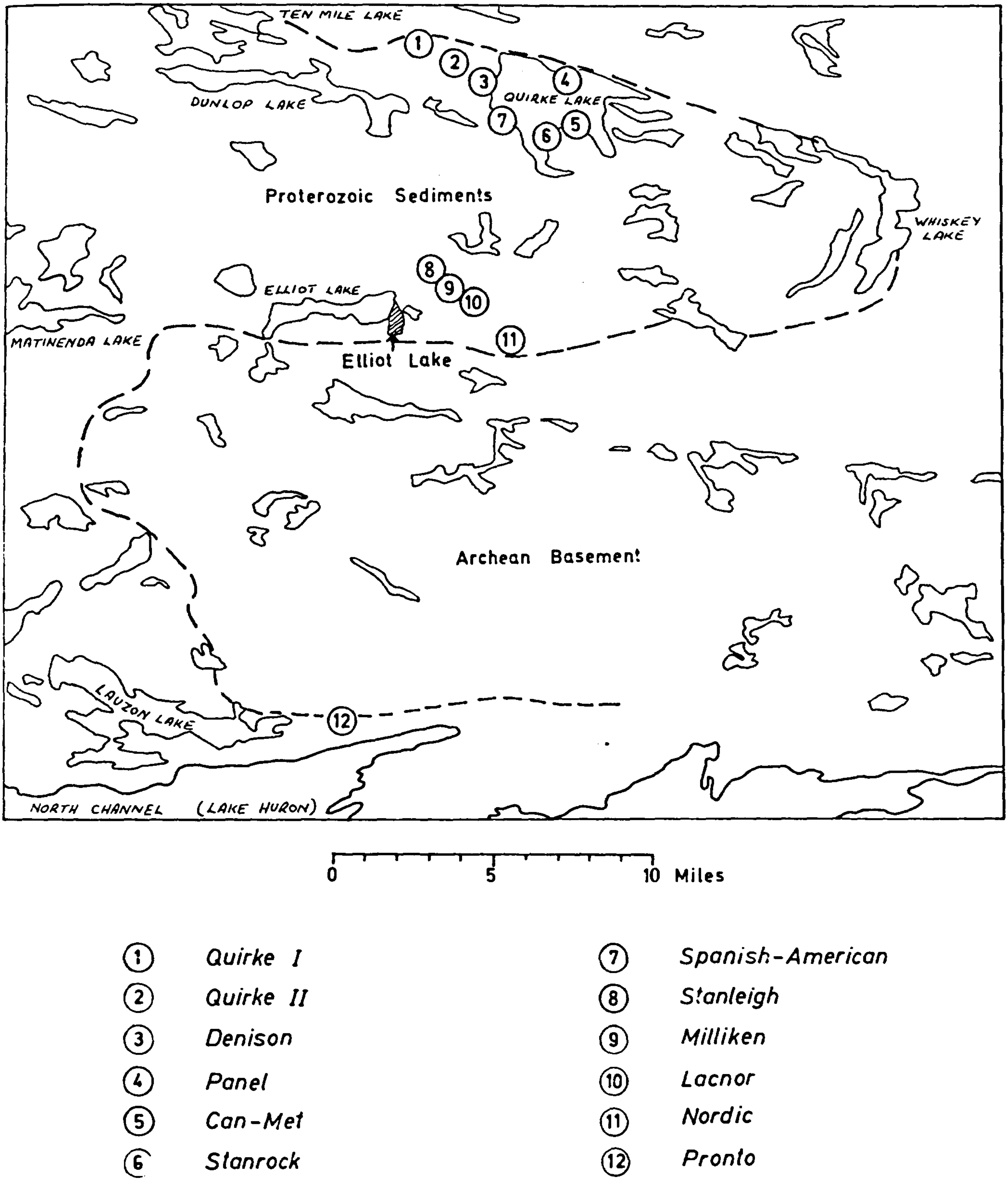


FIG. 1. Location of uranium mines in the Elliot Lake area and a trace indicating the Quirke syncline.

ment headings were driven along strike, and the rooms, or stopes, were mined on dip.

Where the orebody was greater than 10 ft (3 m) thick and the dip less than about 15°, trackless equipment was used for drilling, loading, and transporting. Approximately square pillars were left on regular or random patterns. Where the orebody was less than 10 ft (3 m) thick or the dip greater than 15°, usually scraper stoping was used. With this system, narrow rib pillars about 250 ft (76 m) long were left on dip with sill pillars on strike at the ends of the stopes (Morrison *et al.* 1961, Mamen 1956, Barrett *et al.* 1958, Airth and Olson 1958, McCutcheon and Futterer 1960).

In many mines more than one ore zone exists. If two zones are close together they are mined as one unit, otherwise they are mined separately. Where conservative stope and pillar dimensions were originally used, some of the pillars have been either removed or robbed.

Rock Mechanics

After the Elliot Lake Laboratory was established by the Mines Branch, it was decided to exploit the opportunities of the location. Detailed studies were undertaken to evaluate the methods that were available for the determination of the mechanical properties of the rock mass and its state of stress before mining. These studies then provided the environmental data within which practical studies have been made on the pillars, roofs, and abutments.

Testing

Uniaxial Compression Tests

During the years immediately after the mines were opened at Elliot Lake, a considerable amount of testing was conducted at Universities (Coates *et al.* 1963). Using EX($\frac{7}{8}$-in. (2.2 cm) diameter) core from several mines mean values were obtained in the ranges shown in Table 1.

When particular studies at one mine were started, mean values were obtained for approximately 100 samples of each rock type, as shown in Table 2. The variance was much less in these than in the previous work. The diameter of the samples was 1 in. (2.54 cm), the length/diameter ratio was 2, and the machining tolerances were kept to within 0.001 in. (0.0025 cm).

The composition of the samples from the Nordic Mine varied somewhat. The quartzite had grains that varied from 0.2 to 1.5 mm, the feldspar content varied from a small amount up to 5%, and the pyrite content from 0 to 1%. The conglomerate samples contained grains that varied from 5 to 15 mm with feldspar contents up to 15%. The diabase samples contained plagioclase crystals that varied from less than 1 mm at the edges of the dike to 2 mm at the center; densities varied from 2.66 g/cm^3 at the edges to 3.01 g/cm^3 at the center; the average strength varied from 18 000 p.s.i. at the edges to 46 000 p.s.i. at the center (Parsons and Sullivan 1965).

TABLE 1. Uniaxial compression strength and deformation modulus

	Strength (10^3 p.s.i.)	Modulus of deformation (10^6 p.s.i.)
Conglomerate	10 to 36	8.6 to 13.2
Quartzite	21 to 44	9.2 to 11.2

TABLE 2. Test results for Mine I (coefficients of variation in brackets)

	Strength (10^3 p.s.i.)	Modulus of deformation (10^6 p.s.i.)
Quartzite	37.0 (15.8%)	11.6 (2.5%)
Conglomerate	31.6 (22.5%)	10.8 (3.2%)
Diabase	31.0 (35.2%)	13.5 (11.5%)

Classification testing of the samples showed that the plastic, or irrecoverable, strain for the quartzite samples was 2.9% of the total strain, for the conglomerate samples it was 1.8%, and for the diabase 0.7%. Viscous, or time dependent, strains were found to be 0 for quartzite, 1×10^{-6}/hr for the conglomerate and 1.5×10^{-6}/hr for the diabase samples (Coates and Parsons 1966).

It was thought that the samples in the earlier testing had failed at lower values than their true strength owing to the constraint provided to the ends of the samples by the loading platens. To minimize these effects, a system was developed as shown in Fig. 2. The lead and Teflon wafers were to provide an effective method of compensating for the slight nonparallelism between the ends of a sample (0.001 in. (0.002 cm) difference would produce in the quartzite a difference in stress of 6000 p.s.i. from one side to the other). This approach was more effective than the conventional spherically seated platens which frequently were found to bind before providing sufficient rotation. The steel plug contained the lead and eliminated the transmission of horizontal shear stress to the sample. The steel wafer minimized the constraint applied to the sample against lateral expansion. A test sample showed that the difference in strains between opposite sides was reduced from typically 420μ to 114μ by using this method (Parsons 1968).

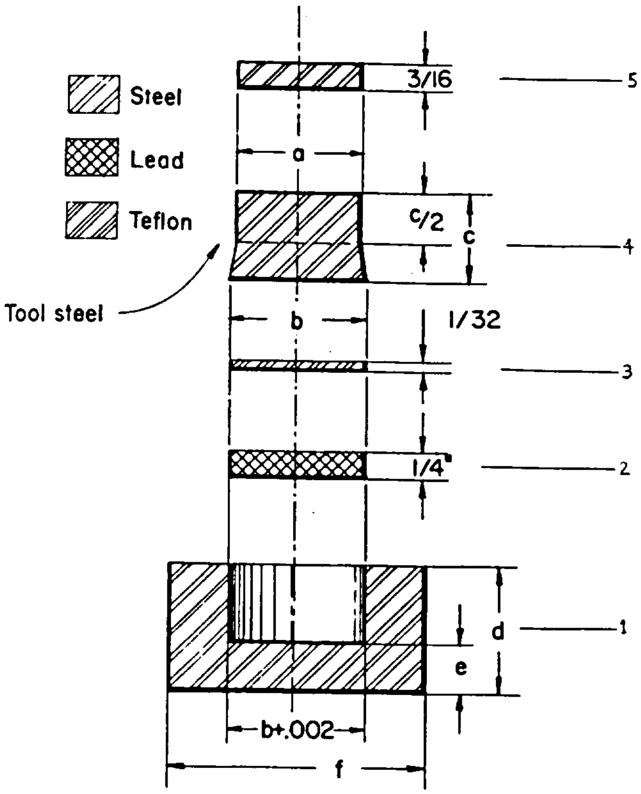


FIG. 2. Teflon-lead swivel head for compression testing of rock samples.

Studies were made, on the quartzite, of the reduction of strength with volume of the sample using the new loading heads. By extrapolating such a trend it was thought that estimates of the strength of the rock mass could be made by assuming continuity of fabric from microscopic to mesoscopic scale. In an attempt to relate strength variation to a geological property, the light source from a petrographic microscope was reduced to a diameter of 50μ for the purpose of scanning the ends of the samples. A photo-sensitive resistor was used to record the interfaces between grains and cracks that were encountered by the light (Parsons 1968). It was thought that strength could be shown to vary with the unit density of interfaces within a sample. Whereas there was some correlation between strength and the number of discontinuities per unit volume, it seemed that the volume of the sample itself was the more significant parameter to correlate with strength.

A second study was conducted on samples selected randomly from drill core even though many of these samples included fractures (Kostak and Bielenstein 1971). The samples varied in diameter from 0.5 to 9.38 in. (1.27 to 23.82 cm), had a length/diameter ratio of 2 and were oriented perpendicular to the bedding. Mean strength of the 1-in. (2.54-cm) diameter samples was higher than that of Table 2, and the coefficient of variation was much lower due to the loading heads. It was found that from 0.5 to 1 in. (1.27 to 2.54 cm) in diameter the mean strength increased and thereafter decreased with increase in size: 46 000 p.s.i. (3.9% coefficient of variation) for 1-in. (2.54 cm) diameter to 27 000 p.s.i. 9.0%) for 9.38-in. (23.82 cm) diameter. The increase in coefficient of variation with size was not in accordance with expectations; however, this was affected by the number of samples tested for each size.

At the same time, single impact testing for comminution research was being conducted on the quartzite. These studies also showed an increase in strength with an increase in size to 1 in. (2.54 cm) (Tervo and Lyall 1968). Assuming that the decrease in strength could be extrapolated to the size of the pillars, the predicted strength of the average pillar would be 23 000 p.s.i.

A special study of the dispersion of strength values for samples 2.14 in. (5.43 cm) in diameter showed that if the solid cores were separated from the fractured cores, two normal distribution curves could be fitted to the histograms as shown in Figs. 3, 4, and 6. The distribution of all samples could thus be represented by a composite curve combining the normal distributions. The coefficient of variation of the solid samples was 14.8% (with a mean of 34 700 p.s.i.), and for the broken samples the coefficient of variation was 43.6% (with a mean value of 20 900 p.s.i.). Although the composite population was not well represented by a normal curve, the computed coefficient of variation was 27.7% (with a mean value of 31 400 p.s.i.).

The modulus of deformation was also decreased with increase in size of sample. For example, with a diameter of 2.14 in. (5.43 cm) the mean value was 13.0 $\times$ 10^6 p.s.i. with a coefficient of variation of 26.8%. With a diameter of 9.38 in. (23.82 cm) the mean value was 7.1 $\times$ 10^6 p.s.i. with a coefficient of variation of 8.9%.

To examine the possibility of genetic stresses having been locked into the quartzite, the elastic strain recovery was measured on the end holes placed in 5-in. (12.7-cm) core. No evidence of locked-in stresses was found from

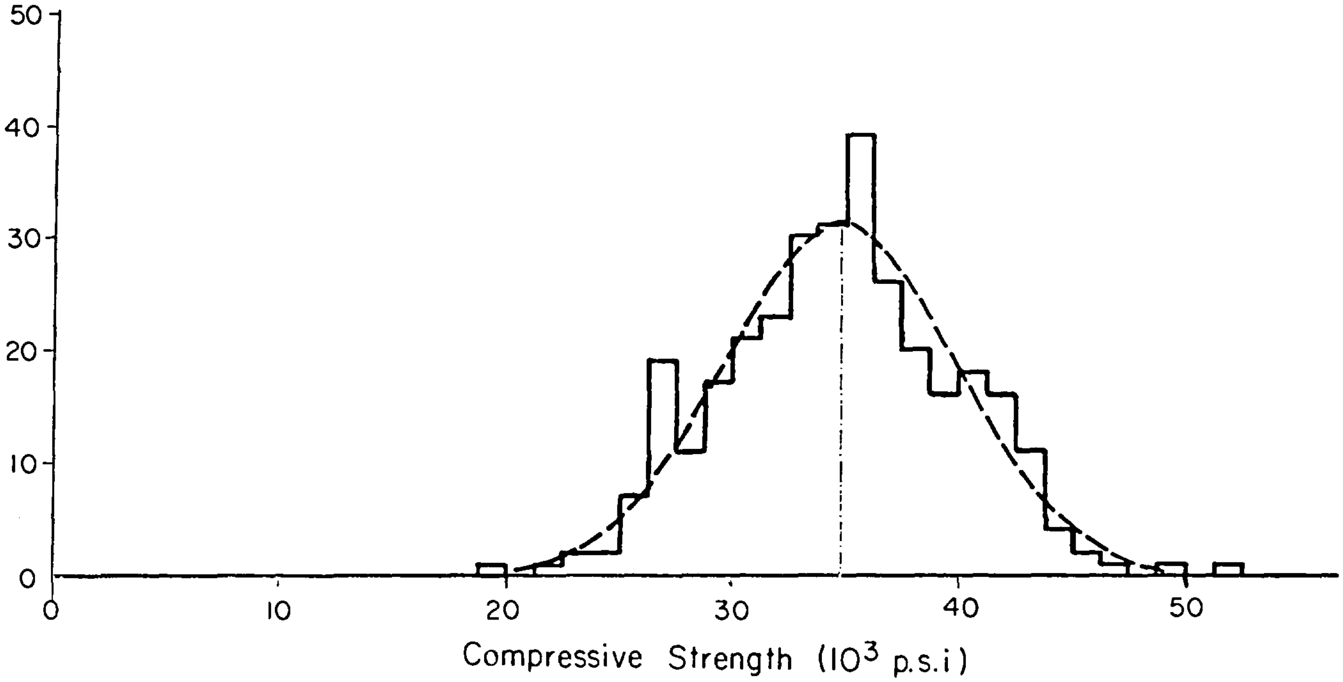


FIG. 3. Histogram of uniaxial compressive strength. Sound specimens only, D = 2.136 in. (5.42 cm).

these tests; however, the 5-in. (12.7-cm) core had been out of the ground for five years before the tests were conducted (Herget 1970).

Twenty-one 6-in. (15.2-cm) holes had been drilled for stress determinations in pillars. Flaking had occurred at the sides of a number of these holes. Using the determined stresses, the stress concentrations around the 6 in. (15.2 cm) holes were calculated and compared with the uniaxial compression strengths obtained on 1-in. (2.54-cm) cores. It was found that where failure had occurred in the holes, the mean ratio of the calculated maximum stress/strength ratio was 0.85 to indicate a mean *in situ* strength of less than 32 000 p.s.i. The mean ratio for the cases where no failure had occurred was 0.78. The indicated mean *in situ* strength corresponded to a reduction from the mean strength of the 1-in. (2.54 cm) diameter of the holes (Coates 1966*a*).

Brazilian Tests

By applying a force in the direction of the diameter of a cylinder of rock, horizontal tensile stresses, σ_x, and vertical stresses, σ_y, are created. By measuring the strains at the center, equations can be derived for the determination of the elastic properties of the rock substance. Figure 5 shows the variation with the length of the gauge of average vertical strain that would be measured over a finite length. Figure 6 shows similar information for the horizontal strain. From these curves, it was judged that strain gauges should be within a distance of 0.14r from the center of the disc.

From a total of 100 tests for all rock types, the range of values shown in Table 3 was obtained for the elastic properties of the disc (Yu 1967). The values of modulii of deformation showed greater dispersion than in the uniaxial testing of Table 2.

The primary use of the Brazilian test at this time was for quality control in the strain-relief measurements. The cores of rock, with the strain cells cemented to their ends, were taken from the mine to the laboratory and subjected to a non-destructive Brazilian test. Two loading cycles were used to determine if there was any insignificant permanent strain at the end of each cycle (*i.e.* $>20\mu$). In addition, the load-strain curves were examined for lack of linearity as shown in Fig. 7. Finally, elastic properties of the disc were calculated from the strain readings to determine if reasonable values were obtained.

In a second study making use of the Brazilian test, samples were obtained from three orthogonal holes oriented normal to the bedding, east–west and north–south (Parsons and Everell 1966). Discs from each of the holes were sub-

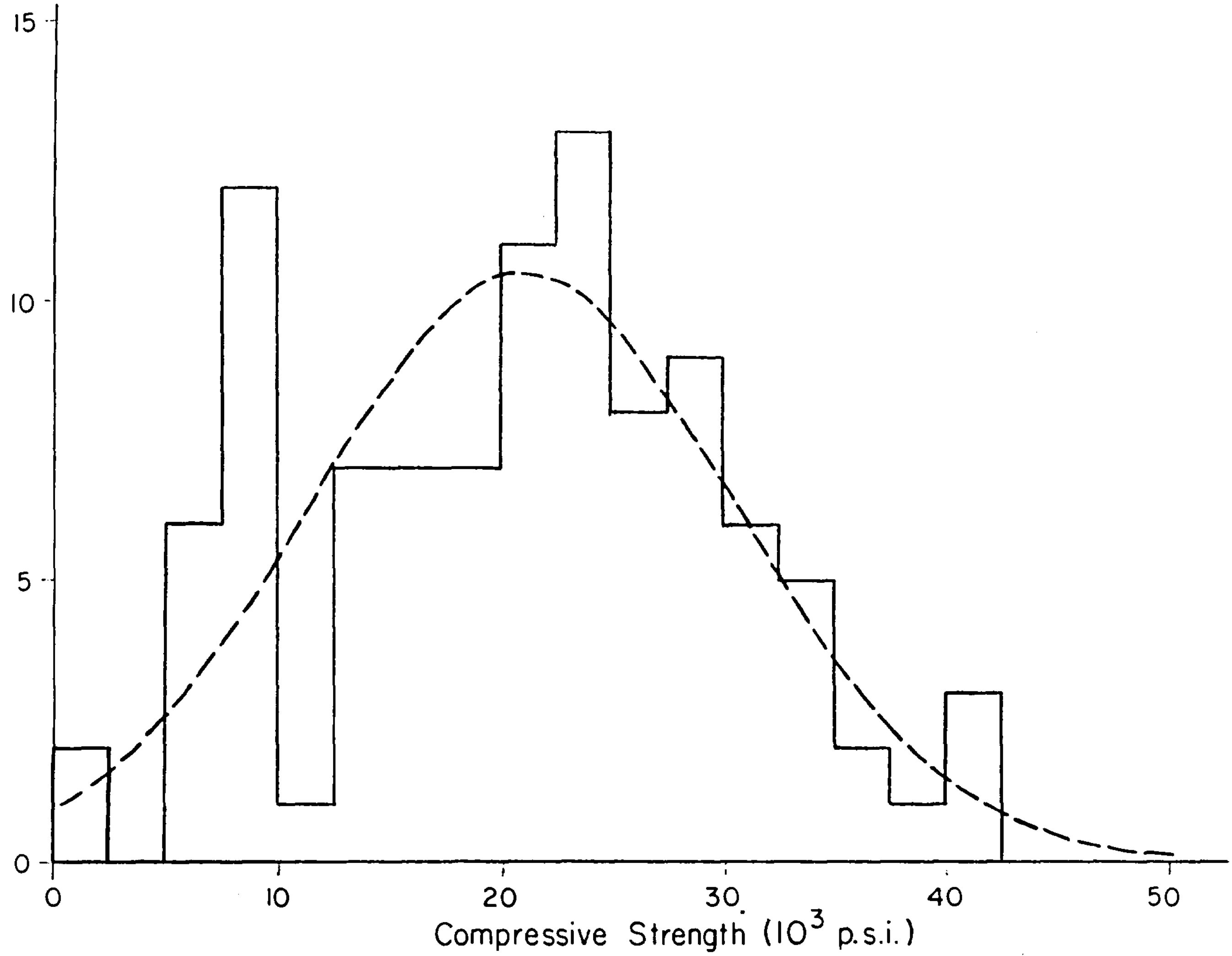


FIG. 4. Histogram of uniaxial compressive strength. Sound specimens (including broken), $D = 2.136$ in. (5.42 cm).

TABLE 3. Elastic properties determined from Brazilian Tests

	Modulus of deformation (10^6 p.s.i.)	Poisson's Ratio
Conglomerate	10.20 to 16.35	0.10 to 0.21
Quartzite	8.05 to 17.50	0.07 to 0.33
Diabase	5.15 to 18.70	0.14 to 0.33

jected to indenter tests, which consist of applying a point load to the center of the flat side of the sample. From these tests a statistical measure of the dominant direction of weakness was obtained.

Brazilian tests were then conducted on a series of samples from each hole to produce fracture in the direction of the established weakness and also in a direction perpendicular to it. The results of the studies showed that the average tensile strength of all samples was 2080 p.s.i. with a coefficient of variation of 15.8%. The average tensile strength in the direction of weakness was 1840 p.s.i. with a coefficient of variation of 6.1%. The direction of weakness coincided with the attitude of the bedding. The wide variation obtained from these tests might have been caused by the edges of the discs not being perfectly cylindrical, hence producing unequal strain, and also due to the short gauge length that would detect wider variations that could occur at the scale of the grains.

Borehole Pressure Cell

In examining methods for obtaining the mechanical properties of the rock mass, trials were conducted on a borehole pressure cell (Panek *et al.* 1964). The test is based on the theory that a uniform pressure inside a borehole will cause movement of the walls of the hole

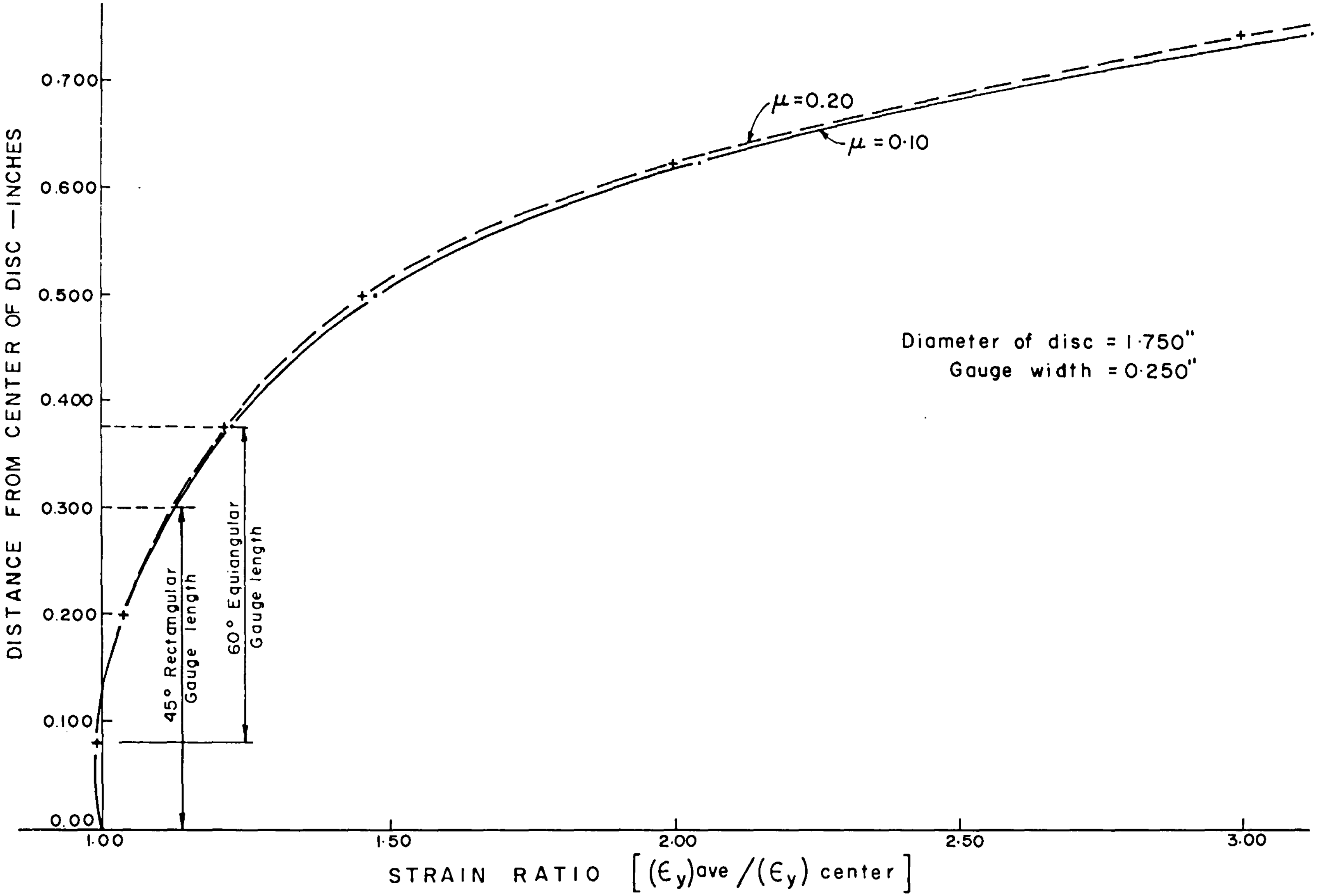


FIG. 5. A graph showing the average vertical strain that would be measured by a strain gauge of finite length compared to the theoretical value at the center.

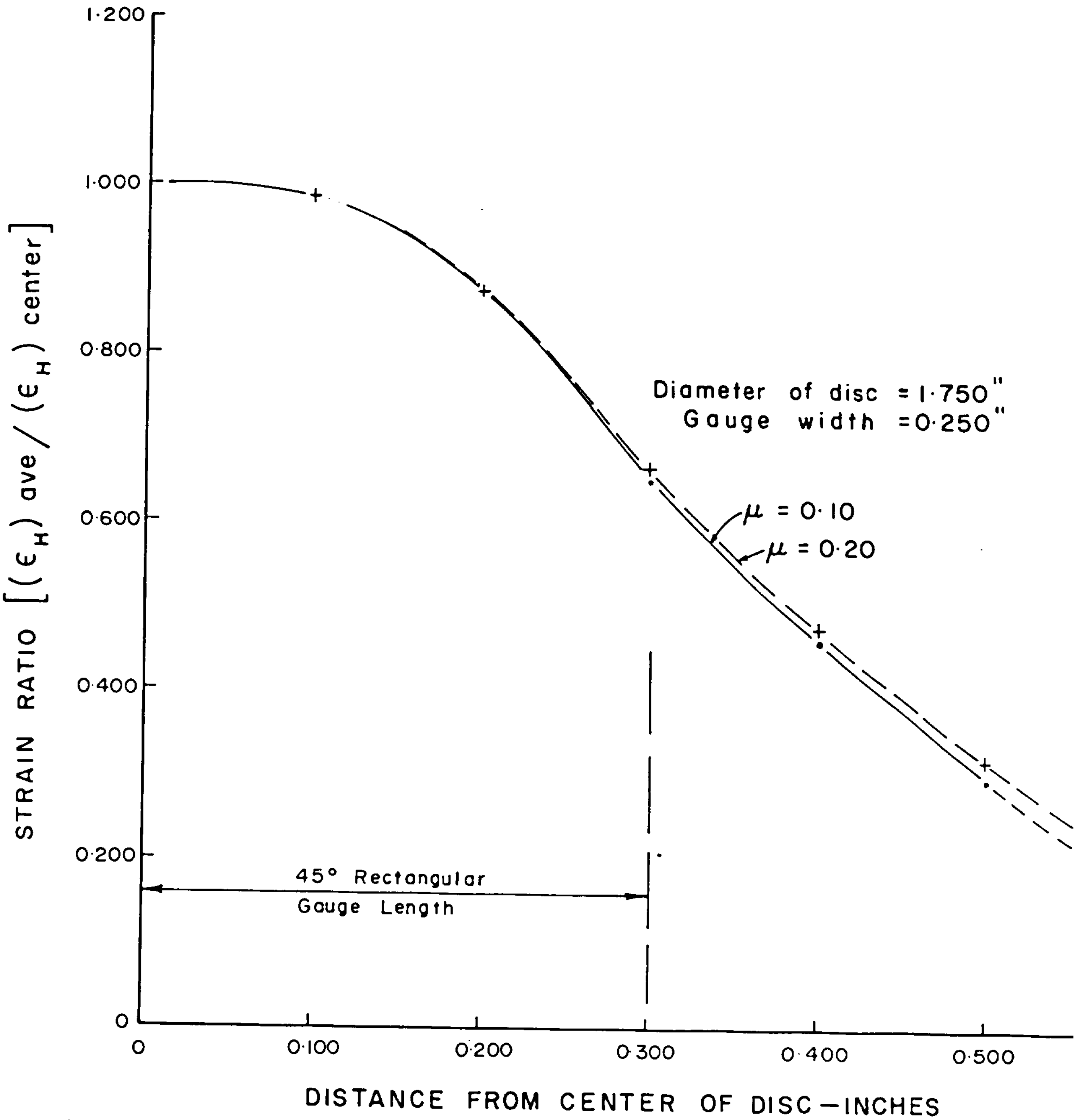


FIG. 6. A graph showing the average horizontal strain that would be measured by a strain gauge of finite length compared to the theoretical value at the center.

outwards, which is proportional to the effective modulus of deformation of the rock mass immediately around the borehole (similar to chamber tests that are conducted for the design of linings of pressure tunnels).

An equation can be written to calculate the modulus of rigidity from the measurements of the expansion of the hole. The expansion of the walls of the borehole is measured by the change in volume of the hydraulic fluid in the cell in the borehole. Actually part of the change in volume is accounted for by some compression of the fluid and expansion of the tubing under the increased pressure. Also, part of the pressure is required to overcome the resistance of the metal in the cell. Consequently, the apparatus must be calibrated, which is done by using holes in blocks of two different materials so the two different calibration constants can be determined.

Using this technique, an average value for the modulus of rigidity for the conglomerate of 4.8×10^6 p.s.i. was obtained (Zahary 1967). Using the previously determined value of Pois-

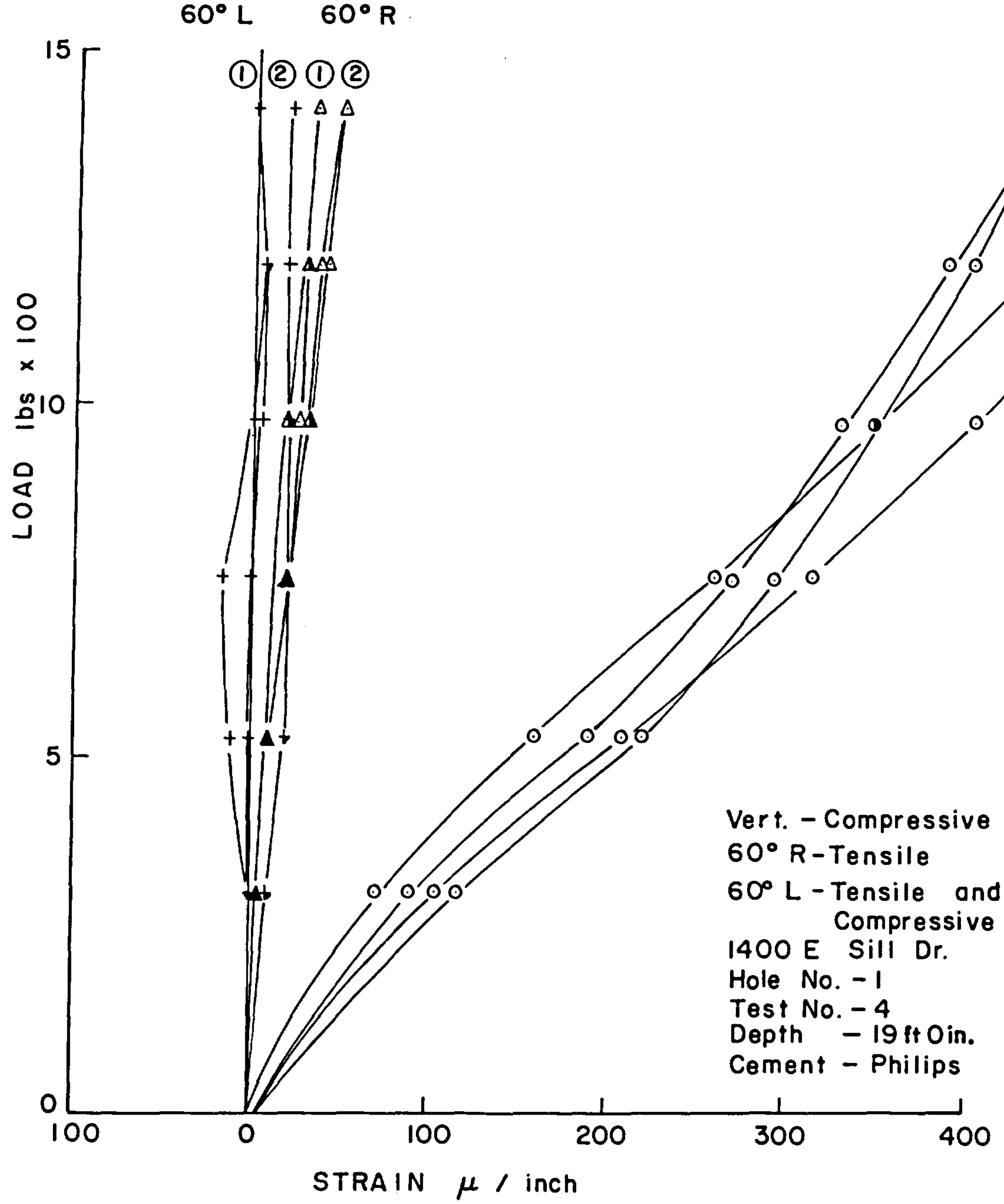


FIG. 7. Diametrical loading on cores for non-destructive quality control of strain-relief measurements for *in situ* stress determinations. Specifications were established for linearity, recovery, and creep.

son's ratio of 0.14, the corresponding modulus of deformation, E, for the conglomerate was calculated to be 11.0×10^6 p.s.i., which is consistent with the values obtained from the uniaxial compression testing, the Brazilian tests and the sonic velocity measurements.

After the trials, it was judged that the technique was not sufficiently versatile for most purposes. Because the outside diameter of the cell had to be very close to the diameter of the borehole, the unit was difficult to insert into some holes and sometimes difficult to retrieve. Furthermore, the depth to which the cell could be inserted was limited by the requirements for a calibration, that includes all of the pressure hose.

Seismic Velocity Measurements

Holes were drilled 10 ft (3 m) to 40 ft (12 m) apart in the pillars of the test stopes. Measurements were made between two holes with a hammer seismometer of the compression wave

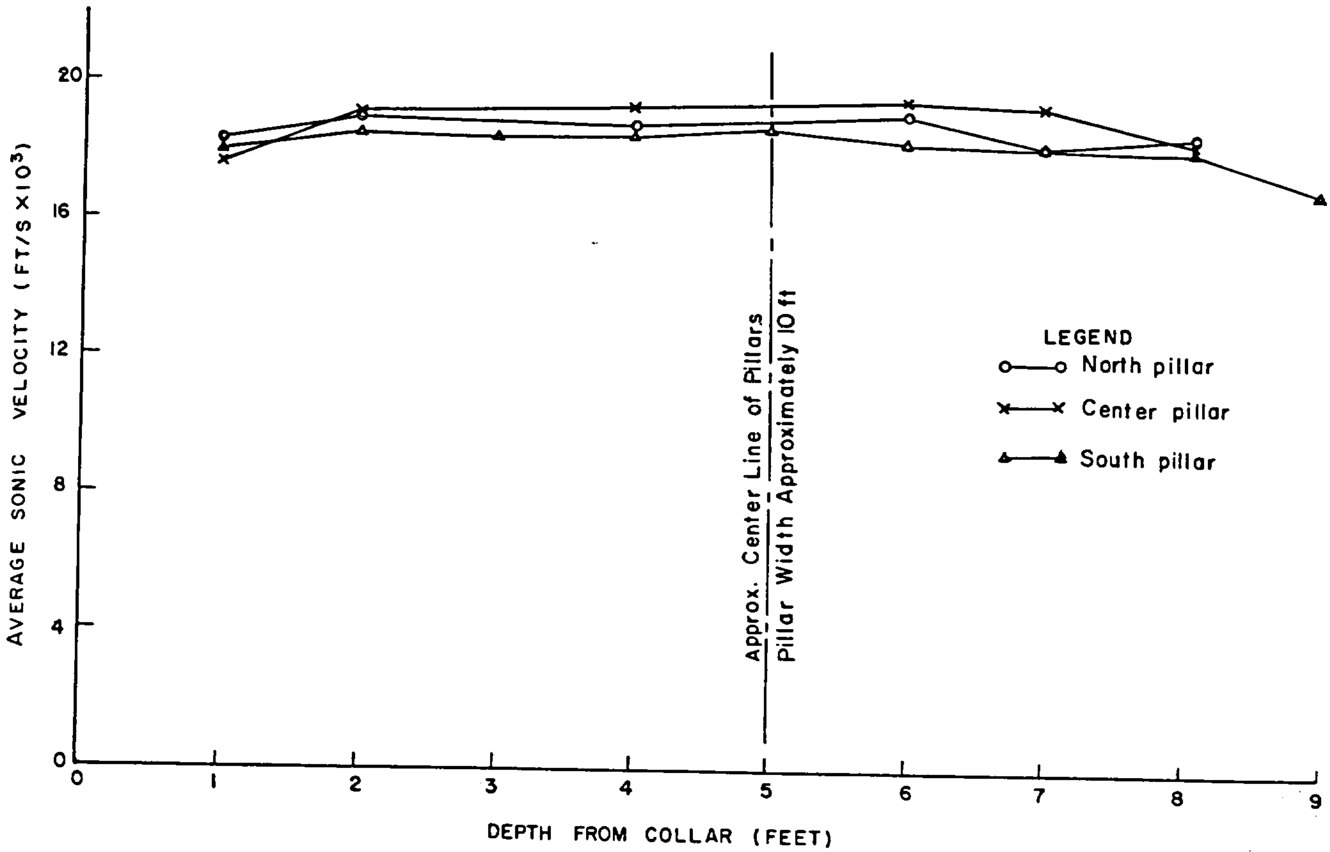


FIG. 8. Seismic velocity (C_p) measurements between holes drilled transversely in pillars, which were long and with a breadth of approximately 10 ft (3 m).

velocities along the lengths of the holes, transversely through the pillars (Larocque 1965, Yu 1966).

The results showed that the average velocities for each set of two holes varied between 17 630 ft/s and 19 670 ft/s (5.4 km/s and 6.0 km/s) with a typical coefficient of variation at each location of only 3%. This variation invariably was accounted for by a slight increase in the velocity towards the center of the pillars as shown in Fig. 8. A similar increase in the modulus of rigidity had been obtained from the borehole cell measurements (Zahary 1967). Both measurements were consistent with the probability of the rock mass being looser adjacent to the faces of the pillars.

Using the previously determined value for Poisson's ratio of 0.14, the computed moduli of deformation in the conglomerate parallel to the bedding using the average velocities at each location varied from 11.9 to 14.4 × 10^6 p.s.i., which is consistent with the previously determined values on laboratory samples and from the borehole cells. However, from the tests on the effect of volume the values should have been considerably less. On the other hand, because the wavelength of the pulse is of the order of the length of the hammer-head, 6 in. (15.2 cm), the deformation moduli could be expected to be consistent with laboratory samples of a similar length, which would still suggest that the values should have been lower. Possibly the very low levels of stress in the compression wave also accounted for the difference between expected and measured values. In any case, these considerations suggest that a representative deformation modulus for pillars, or other large volumes of the rock mass, may not be obtained by measuring the seismic velocity.

Borehole Penetrometer

Another method for testing rock *in situ* was tried; this was the borehole penetrometer developed by the U.S. Bureau of Mines (Vanderzee 1966). The device consists of a probe that is placed down a borehole from which a piston can be extruded to apply a hemispherical indentor against one side of the borehole. The probe is connected by copper tubing to a screw-piston pump. The travel of the screw-piston gives a measure of the penetration of the indentor, and a Bourdon gauge measures the pressure in the hydraulic fluid.

The results obtained in the pillars of the test stopes gave pressure readings for the indentation of 0.100 in. (0.25 cm) of 1986 p.s.i. at a depth of 4.5 ft (1.37 m) from the surface. This variation is qualitatively consistent with the results obtained from measuring the seismic velocities and of the *in situ* deformation moduli. In addition, the coefficient of variation for the penetrometer readings decreased from 63% at the depth of 0.5 ft (0.152 m) to 19% at 4.5 ft (1.37 m), which supports the concept of the rock adjacent to the surface of the pillars being in a loosened and possibly semi-fractured condition.

The technique was found to have limitations. For example, different pressure readings were obtained in the same rock by using different hydraulic fluids. Furthermore, without elaborate de-airing of the hydraulic fluid the variation of pressure with indentation was not linear. Finally, no correlation was found between the pressure readings and either the uniaxial compressive strength of the rock or its modulus of deformation.

It was concluded that the technique could be useful for weak rocks, particularly in those strata that can not be cored. In these cases, some quantitative information could be obtained with a penetrometer from which both strength and deformation could be estimated.

Tectonic History

Regional Geology

The Elliot Lake area lies near the southern margin of the Canadian Shield. The regional framework has been documented by geologic mapping (Bielenstein and Eisbacher 1969, Collins 1925, Robertson 1968, 1969) and radiometric age determinations (VanSchmus, 1965, Fairbairn *et al.* 1969). The stratigraphy is shown in Fig. 9.

The Archean basement complex consists of greenstones and granitic rocks; radiometric ages of 2.5 b.y. have been obtained from the granitic intrusions. This complex is unconformably overlain by a succession of quartzose sandstones, argillites, and conglomerates up to 7000 ft (2135 m) thick (this is the Huronian Supergroup (Robertson *et al.* 1969) of Protero-

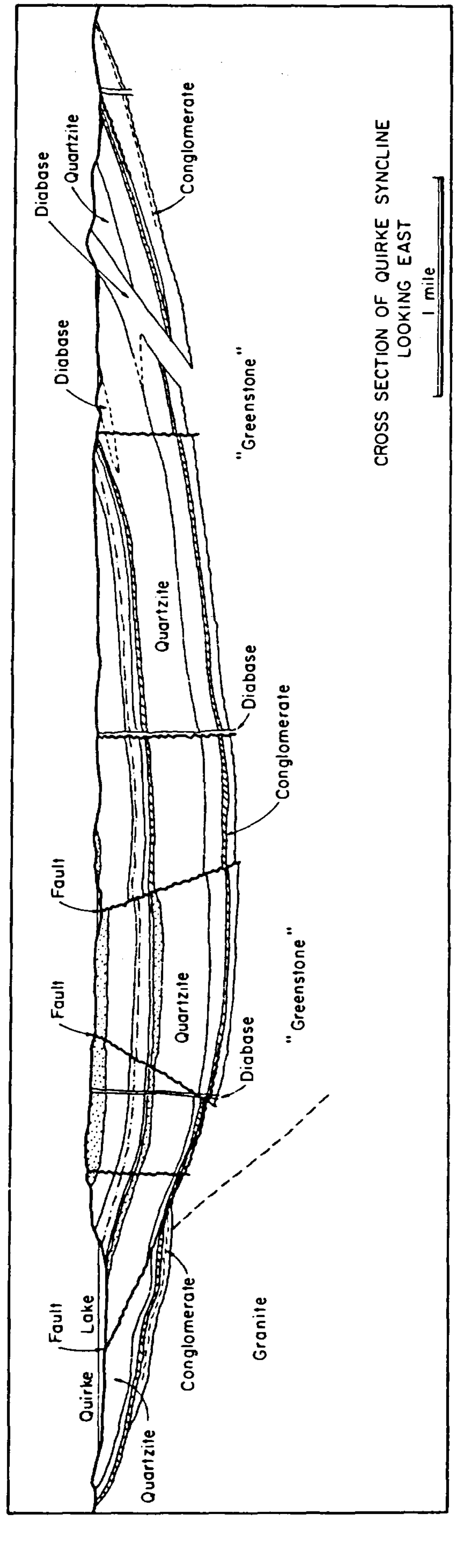


FIG. 9. Cross sections of the Quirke Syncline at Elliot Lake.

			Formation	Radiometric age
	Intrusives		Olivine diabase	1.2 b.y.
	Intrusives		Cutler granite	1.7 b.y.
	Intrusives		Diabase dikes	
	Intrusives		Nipissing diabase	2.1 b.y.
PROTEROZOIC	Huronian Super Group	Cobalt Group	Bar River - quartzite, conglomerate	
PROTEROZOIC	Huronian Super Group	Cobalt Group	Gordon Lake - cherty, quartzite, siltstone	
PROTEROZOIC	Huronian Super Group	Cobalt Group	Lorrain - quartzite, arkose, conglomerate	
PROTEROZOIC	Huronian Super Group	Cobalt Group	Gowganda - conglomerate, arkose	
PROTEROZOIC	Huronian Super Group	Quirke Lake Group	Serpent - quartzite	
PROTEROZOIC	Huronian Super Group	Quirke Lake Group	Espanola - siltstone, limestone, breccia	
PROTEROZOIC	Huronian Super Group	Quirke Lake Group	Bruce - conglomerate	
PROTEROZOIC	Huronian Super Group	Hough Lake Group	Mississagi - quartzite	
PROTEROZOIC	Huronian Super Group	Hough Lake Group	Pecors - siltstone, argillite	
PROTEROZOIC	Huronian Super Group	Hough Lake Group	Ramsay Lake - conglomerate	
PROTEROZOIC	Huronian Super Group	Elliot Lake Group	McKim - argillite, quartzite	
PROTEROZOIC	Huronian Super Group	Elliot Lake Group	Local volcanics	
PROTEROZOIC	Huronian Super Group	Elliot Lake Group	Matinenda - quartzite, arkose, U-bearing conglomerate	
ARCHEAN			Diabase	
ARCHEAN			Algoman granite	2.5 b.y.
ARCHEAN			Metavolcanics and metasediments	

NOTE: After Robertson et al. (1969).

zoic age). Basement and sedimentary cover were intruded and slightly deformed by diabase dikes, age 2.1 b.y. (VanSchmus 1965).

The main deformation in this area occurred after the intrusion of the Cutler granite, age 1.7 b.y. (Fairbairn *et al.* 1969, Eisbacher 1969). This deformation affected both basement and sedimentary cover and led to the development of the pair of broad folds now seen in the cover rocks. The reaction of the rock is characterized by both brittle fracture and discrete slip along pre-existing discontinui-

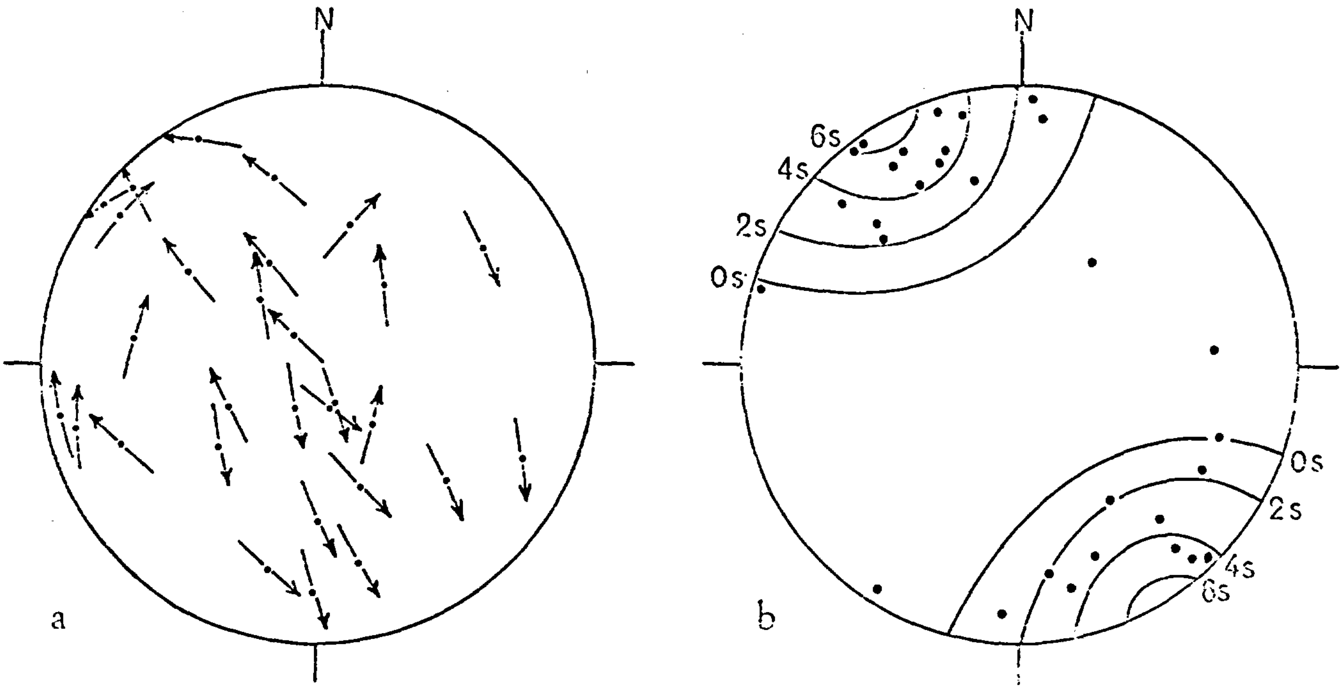


FIG. 10. Compression axes derived from slickensided fractures in granite.

ties, except for flow folding in argillite and limestone members (Bielenstein and Eisbacher 1969). Deformation must have occurred prior to 1.2 b.y., which is the age of intrusion for essentially undeformed olivine diabase dikes (Fairbairn *et al.* 1969). Broad regional arching along an easterly trending axis postdated the main deformation and may have continued into later times (*i.e.*, later than 0.4 b.y.) (Bielenstein and Eisbacher 1969).

Fabric Analysis

The following kinematic elements were used in the fabric analysis (Bielenstein and Eisbacher 1969): bedding plane slip in bedded quartzose units; contraction faults in bedded units; steeply dipping, shear fractures in bedded units; slaty cleavage in argillaceous units; flexural-flow folds in limestone and argillaceous units; matrix-boulder displacements in matrix-rich conglomerates; shear fractures in massive, granitic rocks; deformation of diabase dikes and sills; thrust faults, and strike-slip faults. The concepts for these detailed interpretations are related to shear and extension mechanics.

Each slickensided surface is a locus of shear stress along which there has been slip. The sense of displacement may be determined from offsets and the orientation of congruous accretion steps (Turner and Weiss 1963, Norris and Barron 1969). The axis of rotation for the displacement lies in the shear plane, perpendicular to the slickenside lineation. If the rock mass was mechanically isotropic prior to failure, the rotation axis coincides with the intermediate stress axis (σ_2). The principal compression axis (σ_1), compatible with this shear stress, must lie in the plane normal to the rotation axis but can only be located within one quadrant between the slickenside lineation and the pole to the shear surface.

Two examples show the application of these principles. First, the granitic rocks are considered to have been mechanically isotropic prior to brittle failure. Slickensided fractures from outcrops of this rock were used to derive compression axes for the deformation. The poles to slickensided fractures and respective slip linears (Hoeppener 1955) from a granite outcrop south of Elliot Lake are shown in Fig. 10a. Various assumed failure angles were tried (15°, 30°, 45°, 60°). In each case the derived compression axes were plotted on a Schmidt net and contoured (Kamb 1959). The best preferred orientations were obtained for estimated failure angles of 30° and 45°. Fig. 10b shows the compression axes (σ_1) derived from slip linears using an intermediate failure angle of 37.5°. The intermediate and minimum stress axes must lie in a plane perpendicular to σ_1;

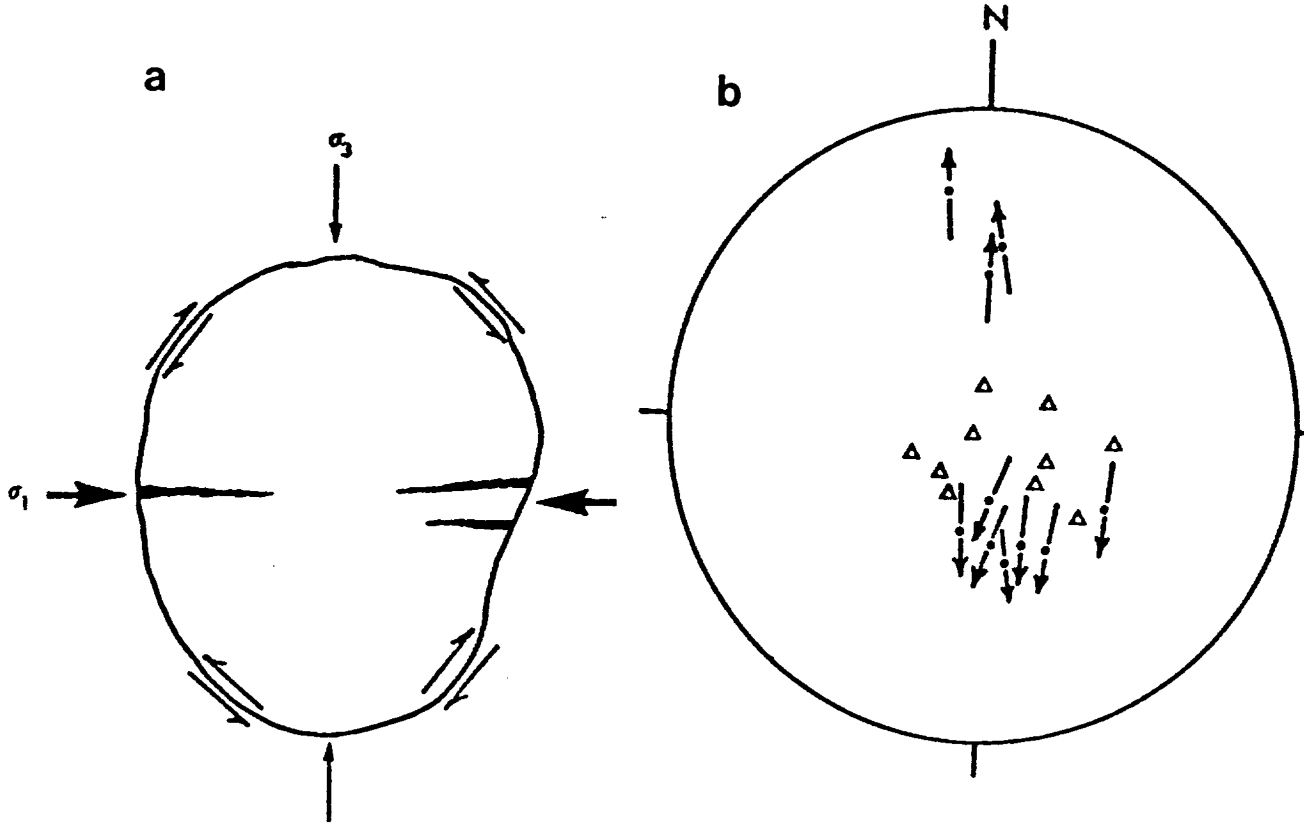


FIG. 11. Compression axes derived from boulder-matrix displacements. (a) Boulder section. (b) Poles to extension fractures (Δ) and slip linears ( ).

because their orientations differ greatly, it is possible that σ_2 and σ_3 had almost the same magnitude.

A second example uses data from the conglomerates, consisting of well-rounded, granitic boulders, and cobbles in a fine-grained matrix. During the deformation slight displacements between boulder and matrix occurred producing a system of slickensides on the boulder surface. Some boulders also display extension cracks (Fig. 11a). Plotted on a Schmidt net (Fig. 11b) the slip linears tend to be parallel to a plane that contains the σ_1 and σ_3 axes. The poles to the extension fractures in the boulder define σ_3, therefore the compression axes can be derived.

Through detailed study of the fabric elements, it was possible to derive compression axes for the main deformation throughout this area (Bielenstein and Eisbacher 1969). These compression axes are oriented in a northerly direction and are plotted on Fig. 12.

Each fracture, which is filled, or along which there has been no shear, is a locus of dilation across which there has been extension. Poles to these elements are normally but not necessarily parallel to the minimum principal stress axes (σ_3).

Throughout the area, thin steeply dipping quartz veins cut across deformational structures such as slickensided bedding and contraction faults. The late quartz veins grade imperceptibly into joints that represent the youngest fabric element of the area. No offsets are found along or across these joints. They commonly occur in groups of closely spaced, very smooth surfaces. Individual joints rarely extend beyond 30 ft (9.15 m), whereas joint groups may extend over hundreds of feet (Fig. 13). These joints can be seen to cut slickensided bedding planes, cleavage, contraction faults, and flexural flow folds in many localities. Thus, their late origin in the tectonic development of the region is well documented. Both quartz veins and joints have a dominant easterly trend and are nearly vertical. Therefore extension in the north–south direction is the last event in the tectonic evolution of the area that can be documented geologically.

At the scale of the individual outcrop, cross-

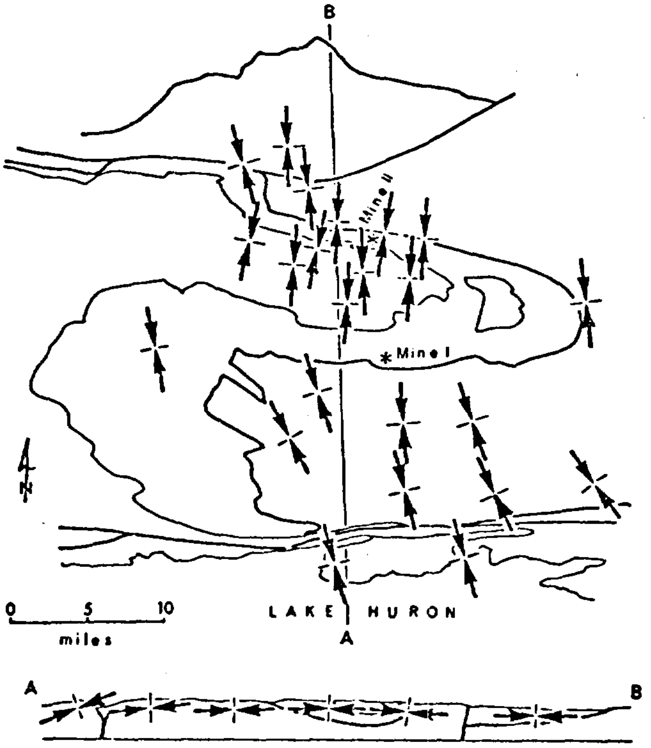


FIG. 12. Orientation of local compression axes for the major deformation.

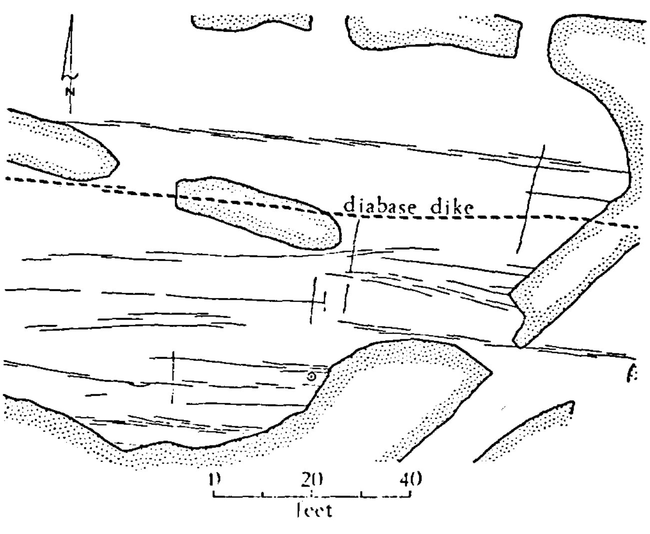


FIG. 13. Sketch map of postorogenic jointing at Mine I.

cutting slip surfaces indicate that deformation took place by brittle fracture. At the scale of the whole area, however, where overburden was at least 15 000 ft (4575 m), strain rates were extremely low, and slip surfaces formed a penetrative network of discontinuities; deformation can be considered analogous to steady-state creep (Bielenstein and Eisbacher 1969). Under these conditions the elastic components of the deformation could be "locked" into the competent units of the rock mass. During the process of regional arching and subsequent erosion, unequal stress relief reduced the northerly trending stress components below the level of easterly components.

Mine Geology

The structural features in Mine I were examined in some detail to determine what local deviations could occur from the regional pattern. This mine is situated in the southern limb of the Quirke Syncline (the axis plunges 5° W). The orebody is cut by three large and numerous small vertical easterly striking diabase dikes throughout the mine and by northwesterly striking diabase dikes in the eastern part. A few diabase sills are also present.

Of all the kinematically indicative fabric elements that were mapped on the regional scale, only the following were found in the mine: bedding plane slip, contraction faults, and steeply dipping, shear fractures. Contacts between diabase dikes and sediments are almost invariably shear surfaces. Contraction faults and bedding plane slip in the sediments continue through the diabase bodies. Surprisingly, no signicant displacement was observed along the sub-Proterozoic unconformity. The compression axes derived from the mine fabric analysis parallel those from the regional analysis. Some minor differences in structural elements were evident.

Most contraction faults dip to the north, slightly more steeply than the bedding. Striations and offsets indicate south–southwesterly displacements of the hangingwall of a few inches to a few feet. A few contraction faults dip to the south, striations indicate a relative displacement of hangingwall to the north–northeast.

Planar, extension joints are common. Their orientation is near vertical, striking in an easterly direction. The rose-diagram in Fig. 14 illustrates the frequency distribution of strikes for all joints measured in the mine. The southeasterly set that occurs in the regional pattern does not occur at the mine.

These joints are planar and continuous; some have been traced for over 500 ft (153 m). They tend to occur in groups up to 10 ft (3 m) wide. The spacing within each group may range from a fraction of an inch to 1 ft (.3 m). These joints rarely merge; they constitute a stack of individual discontinuities separated by material bridges. Swarms of smooth joints are commonly separated by 20 to 30 ft (6 to 9 m) of unjointed rock.

Many dikes are cut and offset by bedding-plane slip, contraction faults, and steeply dipping striated fractures. Internally, dikes are intensively fractured, particularly the smaller ones. Fractures with striations are common; they indicate very complex internal deformation, distinctly different from that of the sedimentary rocks. Calcite filling along the contact between the dikes and the sedimentary rocks and in fractures with the dikes parallel to the contact are common. Horizontal striations were found on the calcite-coated surfaces. The magnitude of slip along the dike contacts is thought to be small.

The objective of this study was to elucidate the possible effect of tectonic controls on *in situ* stresses. In the process of achieving this objective, some geologic parameters were found that directly influenced roof stability in the mine. Roof falls, although infrequent, are controlled by faults, dikes, and joints. An understanding of the structural geology, if available at the time of stoping, could have been used to assess the potential risk of particular stope layouts.

Stress Measurements

Measurement underground indicated high horizontal stresses (Coates and Grant 1966, Udd and Grant 1967, VanHeerden and Grant 1967). Additional stress determinations were obtained at two mines to relate the pre-mining stresses to the local and/or regional structure.

The direction of maximum strain recovery in vertical holes into the roofs of drifts changed systematically from north–south near the collar of the holes to east–west 10 to 20 ft (3 to 6 m) about the roofline. The strain recovery in the lower section of the roof may represent the zone of mining induced stresses; the direction of maximum strain recovery above this zone parallels the joint orientation (Bielenstein and Eisbacher 1969).

Mine II is located on the northern limb of the Quirke Syncline (Fig. 12). The bedding dips gently to the south. Two sets of late joints are present, each confined to a distinct part of the mine; northeasterly striking joints in the northeastern part, and easterly striking joints in the southeastern part. Both sets are nearly vertical.

Elastic strain recovery measurements were made in this mine after the tectonic analysis was completed to test whether the parallelism between maximum strain recovery and jointing, observed in Mine I, would apply to differently oriented joints. It was found that maximum strain recovery parallels the joint orientation in both cases (Eisbacher and Bielenstein 1971).

The transformation from strain to stress is based on the assumption that rock is an elastically isotropic medium. The static modulus of deformation, E, and Poisson's ratio, μ, for these rocks were determined by Brazilian tests of each core disc attached to a strain cell (Yu 1967). The mean values of the elastic constants for the two mines are almost the same (Mine I: $E = 11 \times 10^6$ p.s.i., $\mu = 0.16$; Mine II: $E = 11 \times 10^6$ p.s.i., $\mu = 0.14$).

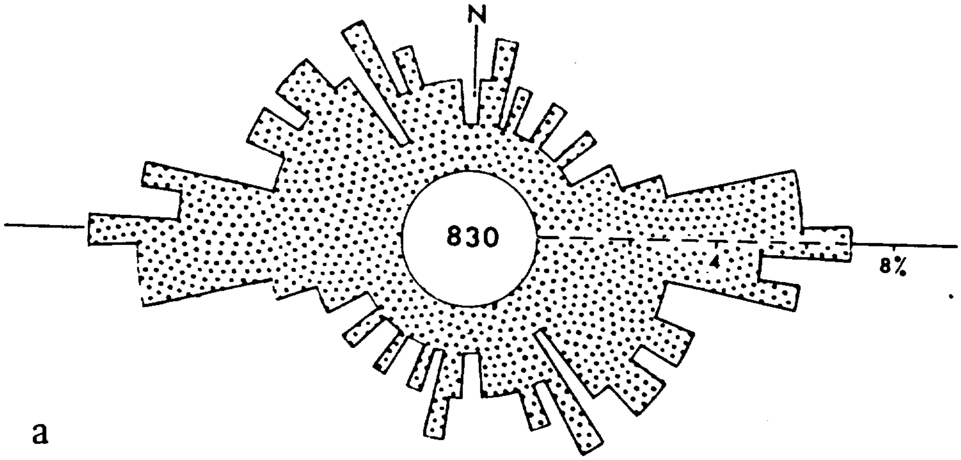
Mean stress levels are given in Table 3. The vertical pre-mining stresses, σ_3, is compatible with the gravitational stresses due to overburden in both mines. The maximum pre-mining stress, however, lies in the horizontal plane and is oriented parallel to the extension joints. Its magnitude is twice to three times that of the vertical component.

Current techniques permit the determination of *in situ* stresses at the small scale of the overcored specimen. It remains to be shown to what extent extrapolation of these stresses to a larger scale—the size of a mine—is valid. In the Elliot Lake area, postorogenic regional arching along an easterly trending axis is the last tectonic event that can be documented. Maximum pre-mining stresses, determined from elastic strain-recovery measurements, are horizontal and parallel to the postorogenic extension joints. Since arching could not have created the high horizontal stresses, the present stresses must contain a significant remanent component of the former tectonic stress field. Mechanically it would be possible to explain the extension fracturing by a more recent east–west compression; but no geologic data have been found to support this interpretation.

Underground Observations

Pillar Failures

It was decided to conduct an empirical study of mine pillars at Elliot Lake. Table 4 lists pertinent mining dimensions at the twelve mines in the area. Depth below surface ranges from zero to 3500 ft (0 to 1067 m). The dip



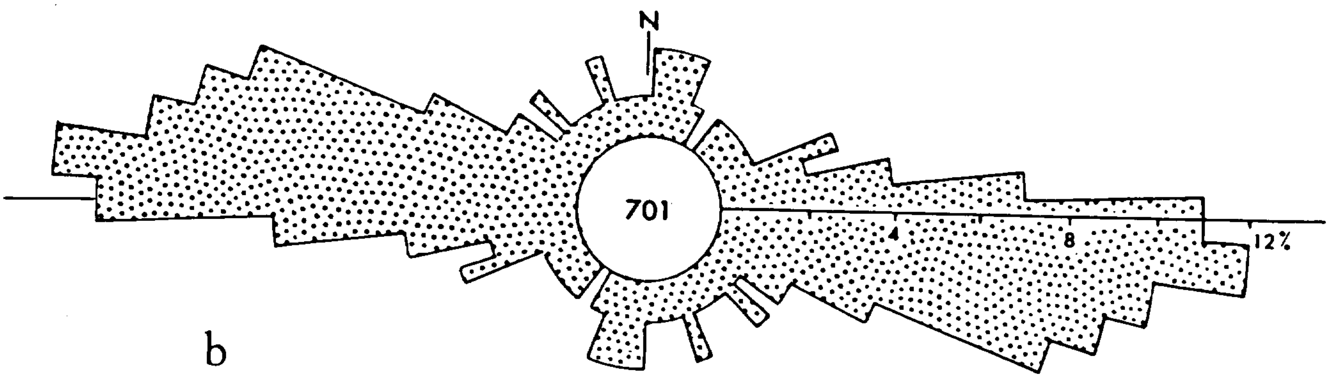


FIG. 14. Strike frequency diagrams of joints: (a) Elliot Lake region, (b) Mine I. The number of measurements is indicated in each diagram.

TABLE 4. Mean Field Stresses

Mine	Depth (feet)	σ_1 (p.s.i.)	σ_2 (p.s.i.)	σ_3 (p.s.i.)
I*	800-1400	3000, E	2500, N	1500, vert.
II NE	1000	5300, NE	2900, SE	1600, vert.
		s† = 2100	s† = 1800	s† = 800
II SE	2300	5300, E	3200, N	2500, vert.
		s† = 800	s† = 500	s† = 600

*Udd, personal communication, 1969.
†s = standard deviation.

of the orebody varies considerably due to local rolls; the average is just over 20° for the northern limb of the syncline, just under 20° for the southern limb, and less than 10° at depths greater than 3000 ft (915 m). The mining height depends on the number of ore horizons being mined, the average for single reefs varies from 6 to 15 ft (1.8 to 4.6 m). The areal extent of the mines is quite large for those that have been in operation for a long time, and in most cases the minimum width of the mining zone is greater than the depth.

Dimensions of stopes and pillars have varied considerably. Present mining practice is fairly uniform with 65 ft (20 m) stope spans extending about 250 ft (176 m) on dip. Pillar breadths and heights are usually maintained at a 1:1 ratio. Extraction has ranged from 60 to 90%, the greatest extraction being obtained in the shallow mines.

The following is an example of the actual process of arriving at practical stope and pillar dimensions. The orebody at this mine is located between 2580 and 3000 ft (787 and 915 m) below the surface and dips from 11° to 17°. The original design was for 68% extraction with stopes 50 ft (15 m) broad. In the first stope deteriorating roof conditions were experienced when the span reached 30 ft (9 m). The layout was changed to 30 ft (9 m) stopes and 10 ft (3 m) pillars, representing 75% extraction. It was then found that the roof conditions in the first stope were exceptional, and the stope spans were increased to 40 ft (12 m), giving 82% extraction. As no ground control problems were encountered, the stopes were increased to their original 50 ft (15 m), retaining 10 ft (3 m) pillars which gave an extraction of 85% (Hedley and Grant 1972).

In this particular mine to reduce development, the stope spans were increased to 100 ft (30.5 m) leaving 20-ft (6-m) broad rib pillars and small pillars on strike at the mid-length of the stope, to realize 82% extraction. Pillar failures occurred in some of the older workings where their breadth was only 10 ft (3 m), some pillar recovery had occurred, and a fault passed through the area. Haulage drifts 30 ft (9 m) below the sill pillars deteriorated somewhat, this was alleviated by removing the sill pillars.

Also, near fault zones, the backs of some stopes had caved. Corrective action immediately followed. All faults in the mine were surveyed so that additional pillar support could be left where required. The stope layout was revised to give 75% extraction, with 70 ft (21.3 m) stopes, 20-ft (6 m) rib pillars, and a 15-ft (4.6 m) pillar down the center of the double stope. Also, the sill pillars directly above the haulage drifts were systemically removed at the end of the stoping cycle.

Pillar and roof failures continued to occur in the older sections of the mine. Consequently, a further revision was made to the stope layout. A stope 50 ft (15 m) wide was mined along strike directly above the haulage drift. With this completed, the remainder of the panel was mined in two 60 ft (18.3 m) stopes giving an extraction of 70%. Pillar breadths were standardized at 20 ft (6 m), and extra pillars along strike were left at the junction of the updip and downdip mining stope. This mine design appeared to give stable conditions.

In review it seems that 10 ft (3 m) pillars started to fail where the extraction was greater than 80%, especially in areas where two horizons were mined. Sixty-foot (sixteen-meter) stopes and 20-ft (6-m) pillars with an extraction of 70% appeared to give stable conditions.

General information was collected on both stable and unstable pillars throughout the area. The depth, dip, extraction, pillar breadth, and height are given in Table 5. There are 23 cases of stable pillars, 2 cases where partial failure occurred, and 3 cases of complete crushing.

Roof Failure

The information available on roof stability is limited. Stope span was found to be one of the most important factors. Thickness of the roof layer should be important; for instance, an 8-ft (2.4-m) thick bed spanning 65 ft (20 m) is likely to be less stable than a 30-ft (9-m) thick bed. Structural features such as faults and joints can greatly affect stability to the extent that the roof of a 20-ft (6-m) wide roadway can fail, whereas in another location an opening 300 ft (91 m) square has been mined with no roof collapse (Hedley and Grant 1972).

Safe mining spans of the stopes have been evolved over the years. Initial mining produced stope spans ranging from 20 to 110 ft (6 to 33.5 m), but, by the mid-1960's, most mines were using stopes 50 to 65 ft (15 to 20 m) broad. One mine has documented 18 cases of roof falls ranging in thickness from 2 to 17 ft (0.6 to 5 m). Of these 18 occurrences, 17 could be explained by thrust faults in the immediate roof, converging fault and joint plans, and prominent bedding planes in the roof. The one remaining fall 65 ft (20 m) by 65 ft (20 m), arches 12 ft (3.76 m) into the roof. Roof caving occurred in one mine when spans were 70 ft (21.3 m) or greater, especially where geological weakness planes existed. All the mines use rock bolts for supporting the roof.

Pillar Stresses

To supplement the empirical study of pillar stability and to test a new hypothesis for predicting pillar loads, about 200 individual strain relief measurements were taken in the pillars at one mine (Coates and Grant 1966, VanHeerden and Grant 1967, and Leeman 1964).

TABLE 5. Mining parameters in the Elliot Lake mines

Mine	A	B	C	D	E	F
Depth (ft)	0–300	20–1050	300–1200	50–1450	1000–1600	2200–2600
Dip range (°)	15–50	15–40	5–40	10–20	15–40	14–20
average (°)	20	20	12	17	28	17
Ore horizons	1	2	1	2 separately	1	2 separately
Mining height (ft)	7.5	5–12, average 9	14	7 upper 7–12 lower	7–9.5, average 8	5–12 upper 6–12 lower
Area mined Strike × Dip (ft)	4000 × 2500	7000 × 2000	3500 × 1500	6000 × 1400	1300 × 1400	3500 × 1400
Stope dimensions						
length (ft)	irregular	150–300	150–200	200–400	220–360	150–300
span (ft)		65	55–70	65	65	50
Rib pillars						
length (ft)		150–300	150–200	200–400	220–360	150–300
breadth (ft)	irregular	10	10–15	10	10	10
Sill pillar						
breadth (ft)		15		15	10	15
Extraction (%)	90	85	85	85	85	75

	G	H	I	J	K	L
Depth (ft)	2600–3000	600–3000	1700–3000	3000	3000–2500	3000–3200
Dip range (°)	11–17	0–60		5–15		
average (°)	12	18	12		8	17
Ore horizons	2 separately	2 together	1	2	2 separately	1
Mining height (ft)	7–12 upper 7 lower	5–32, average 15	14	5.5–8, average 6	10 upper 10 lower	
Area mined Strike × Dip (ft)	5000 × 3000	15 000 × 10 000	1700 × 2000	7000 × 2500	3000 × 2500	2000 × 500
Stope dimensions						
length (ft)	200–300	200–250		200–250	200	200
span (ft)	30–100, average 60	25–110, average 65	20–25	35	50	50
Rib pillar						
length (ft)	200–300	40 & 200–250	25–30	200–250	200	150–200
breadth (ft)	10–20	20–25	20	15	10	20
Sill pillar						
breadth (ft)		15–20		20–25		25
Extraction (%)	65–85	65	70	70	70	65

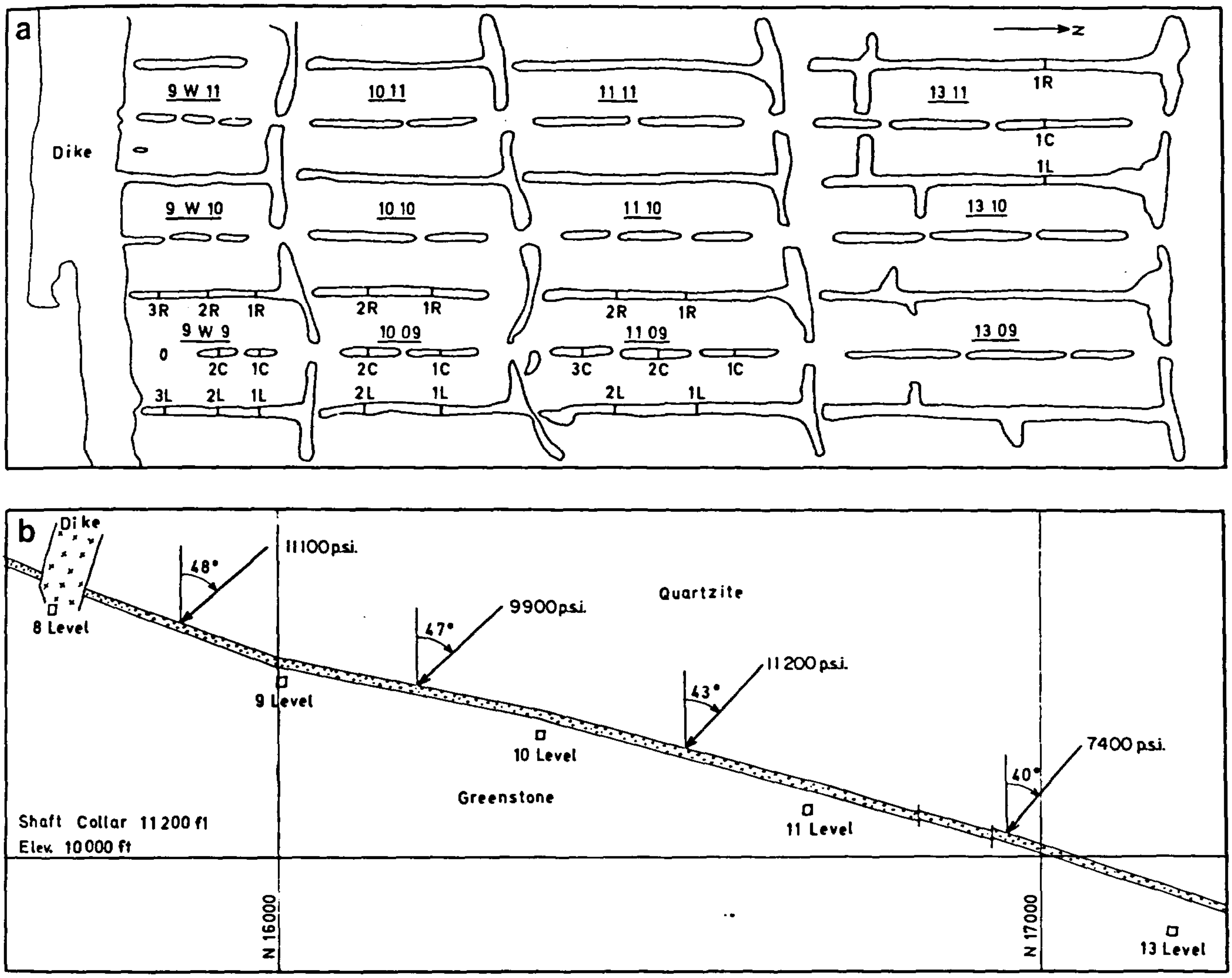


FIG. 15. A. Plan of stopes showing pillars in which measurements were taken. B. Magnitude and direction of major principal stress in pillars.

Two measuring techniques were used one after the other: The USBM deformation meter, where the instrument is installed in an EX hole and overcored with a 6-in. (15-cm) diameter bit (Obert *et al.* 1962); and the CSIR strain cell, where strain gauges are cemented to the flattened end of a BX borehole then overcored (Leeman 1964).

Figure 15a shows the layout of the pillars where the measurements were taken. Pillar breadths are between 8 and 15 ft (2.4 to 4.6 m), with a height of about 10 ft (3 m). Stope spans are 65 ft (20 m), which gives an extraction of about 85%. The orebody dips 18° towards the north, and the depths of the instrumented pillars are 910 to 1180 ft (277.5 to 360 m) below the shaft collar.

A comparison of stresses determined from the two measuring techniques was made by drilling two adjacent holes in the same pillar. Table 6, based on recalculated values using more appropriate concentration factors, shows for each borehole the average values of the major and intermediate principal stresses, and the pillar stress perpendicular to the dip (Coates and Yu 1970). There is a very close agreement in the magnitude of the major principal stress with both measuring techniques. The intermediate principal stress is consistently lower with the strain cell method.

In one location (11-1C) three intersecting horizontal holes were drilled and measurements taken with the strain cells. This configuration allowed the determination of the stresses in three dimensions at one point. The values of σ_1 and σ_2 from this determination are consistent with those obtained from a single borehole using a strain cell. However, they also show

Table 6. Survey of pillars

Depth (ft)	Dip (°)	Extraction (%)	Pillar Breadth (ft)	Pillar Height (ft)
Stable pillars				
500	17	85	10	10
700	17	85	10	10
800	26	65	20	18
850	20	85	10	10
950	11	85	10	10
1000	22	65	20	18
1050	15	85	10	10
1200	18	85	10	10
1300	20	65	20	20
1600	20	60	20	18
1600	20	65	18	18
1600	22	75	20	14
1700	22	65	40	20
1700	22	60	22	20
1700	12	75	20	14
1800	5	75	20	14
1900	23	65	19	18
2200	25	65	20	20
2400	11	65	20	8
2500	9	65	20	8
2700	13	65	20	8
2900	12	70	15	9
2900	12	75	20	9
Partially failed pillars				
1400	20	85	10	10
2400	18	80	10	9
Crushed pillars				
2800	12	80	10	9
2900	12	80	10	9
3400	5	80	15	10

that at the center of the pillar there is a transverse stress, σ_3, greater than zero, equal in this case to 2200 p.s.i.

The major principal stress in the pillars varies between 6000 and 15 200 p.s.i., and acts at an angle of about 45° from the vertical rather than perpendicular to the dip. This reflects the influence of the relatively high north–south horizontal stress. Figure 15b shows the average major principal stresses and directions on a vertical section through the stopes. There is a tendency for the stress direction to be steeper with increasing depth. This is to be expected since the vertical gravitational load increases with depth.

The pillar stresses (perpendicular to the walls) vary between 4800 and 13 700 p.s.i. From laboratory tests on small samples it is estimated that the strength of mine pillars 10 ft (3 m) high by 10 ft (3 m) wide is approximately 23 000 p.s.i. (Kostak and Bielenstein 1971). All the measured stresses are below these values, and although some pillars show some signs of high stress (*e.g.* by the core discing during drilling) there have been no pillar failures. There is some indication that the narrower pillars are more highly stressed than the broader pillars. This could mean that pillar stresses are influenced by the local extraction ratio (*i.e.* calculated for one set of 2 stopes) as well as by the overall extraction (*i.e.* for the whole mine).

A comparison can be made between the pillar stresses determined in the field and those calculated using the simple tributary area theory, *i.e.* just taking into account the overburden and horizontal stresses, dip of the orebody, and extraction ratio. The relation between pillar stress σ_p and extraction ratio, R, can be expressed by,

$$\sigma_p = S_0/(1-R)$$

where S_0 is the pre-mining, field stress acting perpendicular to the walls. Based on the field stress determinations, the north–south horizontal stress was assumed to be 3000 p.s.i. (Bielenstein and Eisbacher 1970).

For the pillars of the 9, 10, and 11 stopes, the agreement is good. The range in stresses could be explained if a variation of about ±5% in the average extraction ratio existed. However, the pillar stresses on the 13 stope level are only about half of the expected value; the explanation for this and the other discrepancies is largely connected with the effects of adjacent solid abutments and the shape of the mine boundaries as shown in the subsequent section. The presence of faults and other structural features can add further variations.

Nevertheless, by assuming the strength, Q_p, to be related to breadth, B, to the power of 0.5 and height, H, to the power of 0.75 an interesting graph can be produced as shown in Fig. 16. A basis seems to exist for developing design rules such as shown in Fig. 17.

Deformations

Roof to floor convergence measurements have been taken in a number of stopes to evaluate the compression of the pillars due to

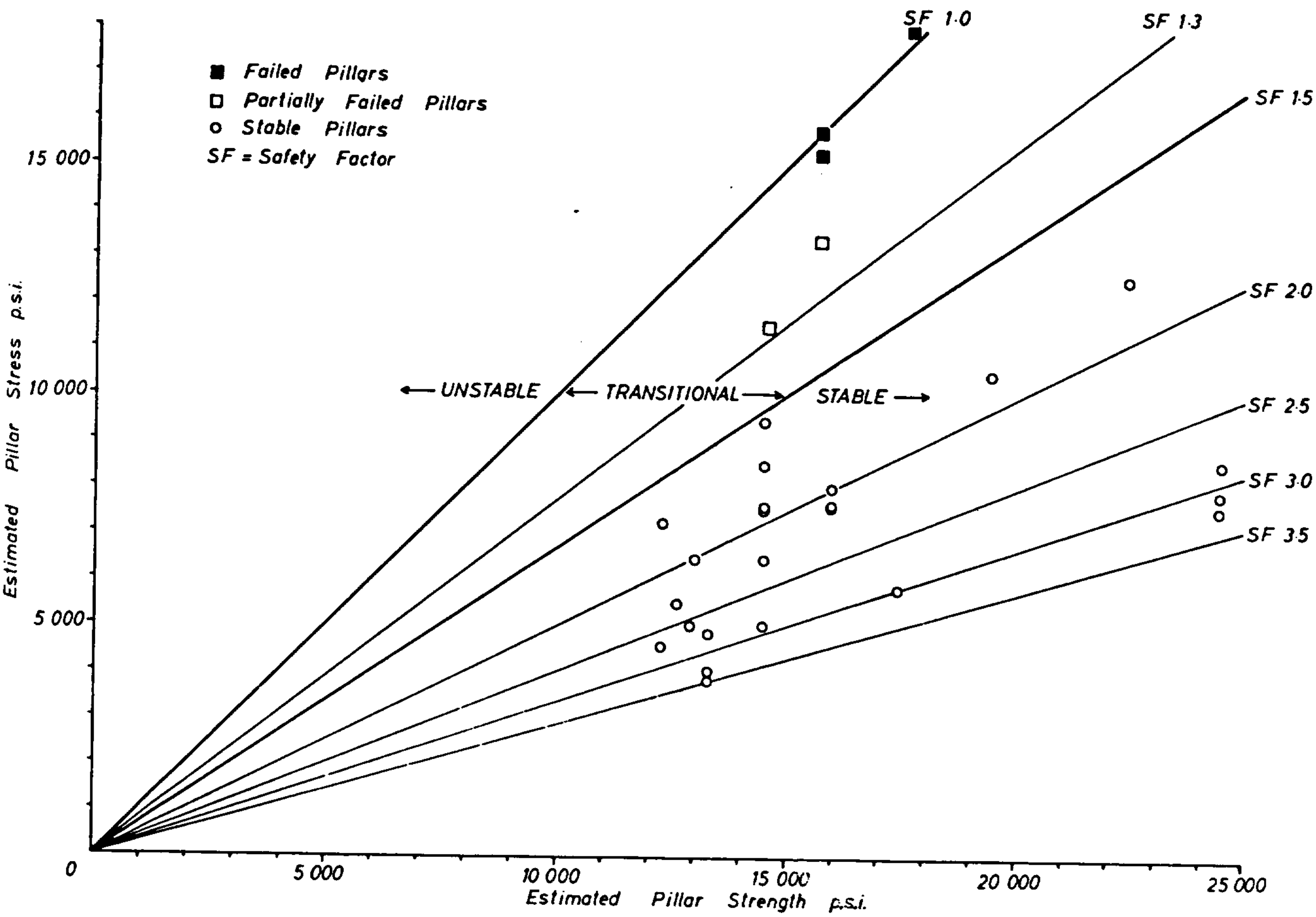


FIG. 16. Estimated pillar stresses and strengths.

mining, the distance through which the load is being transferred, and the sag of the roof.

Figure 18 shows the convergence profiles across one double stope while mining the stope to the right of Station 1. The full lines represent actual convergence, whereas the dashed lines indicate the probable convergence due to compression of the pillars. The difference between each set of two lines indicates sag of the roof which, after 500 days, amounted to 0.014 in. (0.035 cm) at both stope centers. Most convergence occurred at Station 1, which was nearest to active mining, with a progressive decrease with distance. It was observed that mining beyond 250 ft (76 m) did not produce any measurable convergence, and hence seems to represent the limit of load transfer under these conditions (Hedley *et al.* 1970).

In one part of the mine three pillars were systematically removed to form an opening 280 ft (85 m) square. The objectives of this experiment were to see if the roof would cave, and if adjacent pillars would deteriorate under the additional load. Measurements of convergence and roof deformation were taken as the pillars were removed. Figure 19 shows the stope and pillar layout and the blasting sequence. The types of instrumentation and location are shown on the vertical section of Fig. 20.

No significant roof deflection was measured in the roof of stope 14.28 during the first pillar removal (blasts 1 to 4) and after removing half of the second pillar (blasts 5 and 6). The movement at the convergence Stations 1 to 4 is shown in Fig. 21. Pillar blasts 1 to 3 produced appreciable convergence at Station 1 and less at Station 2. The 4th pillar blast resulted in no significant convergence, hence this portion of the pillar was probably relieved of stress prior to blasting. The readings were relatively stable when no mining occurred in the 200 days between the 1st and 2nd pillar removal. The 5th pillar blast produced significant movement at Station 1 as did the 6th pillar blast at Station 2. These stations then had to be removed.

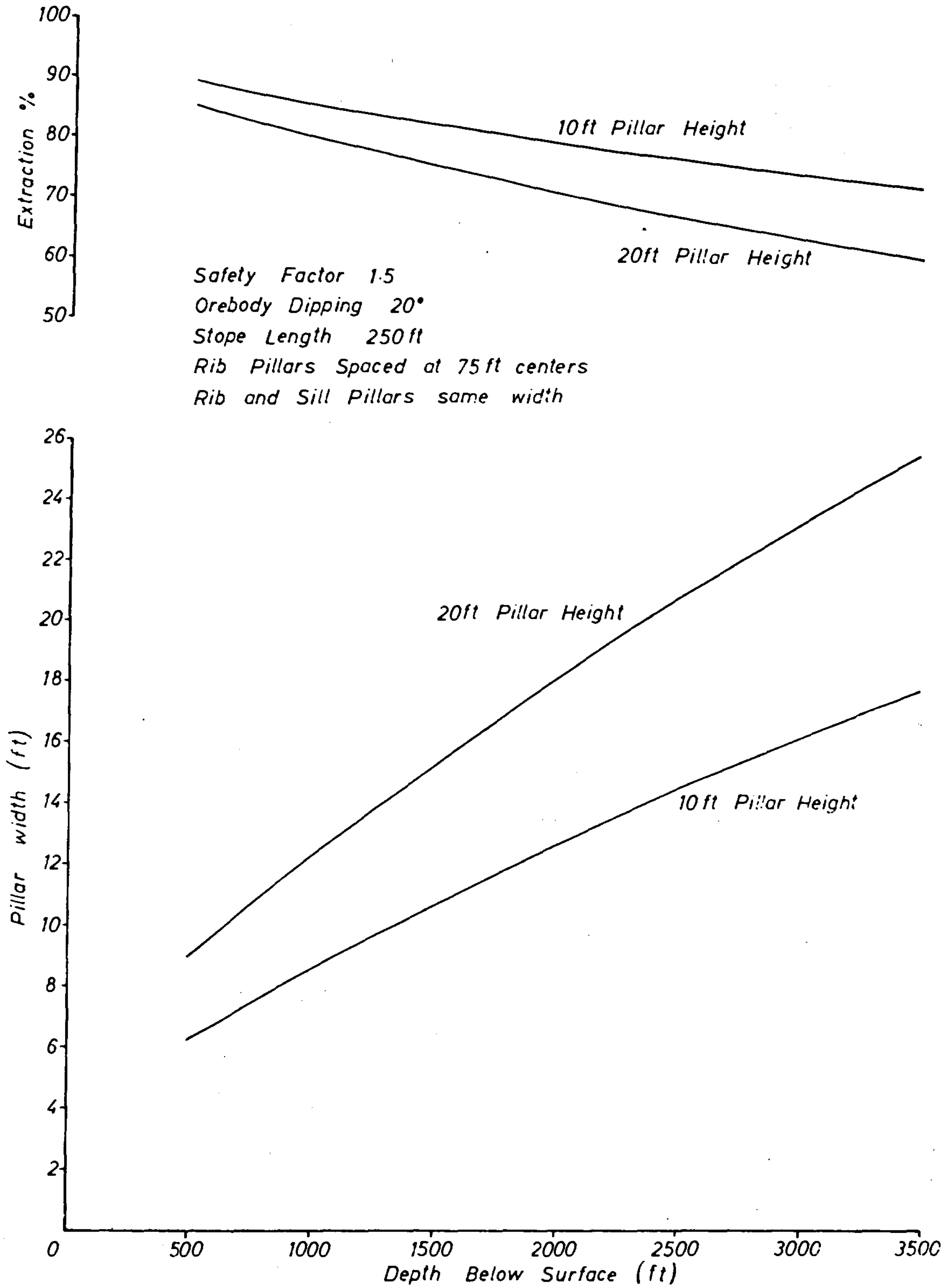


FIG. 17. Variation in pillar width and extraction with depth.

Convergence stations 3 and 4 were installed after the first pillar had been removed, prior to blasting the second and third pillars. Movement was recorded at Station 3 while no mining was taking place, which may have been due to local loose rock. Convergence was measured after pillar blasts 5, 6, and 7 but not after blast 8. Again, the last portion of the pillar seemed to be relieved of stress prior to blasting. Removal of the third pillar (blast 9), which was 200 ft (61 m) away, resulted in only minor convergence at these stations.

These results show that the stations downdip (Nos. 1 and 3) recorded greater movement. Hence, there may be a tendency for load to be transferred downdip rather than uniformly in

TABLE 7. Pillar stresses

Borehole no.	B (ft)	Z (ft)	σ_1 (p.s.i.)	σ_2 (p.s.i.)	θ (°)	σ_p (p.s.i.)
9-1R†	9.4	930	11 000	900	44	8550
9-2R*	8.3	920	11 000	4500	55	8200
9-2R†	8.3	920	11 600	2200	43	9340
9-3R*	7.7	910	14 200	4500	53	10 360
9-3R†	7.7	910	14 000	2300	47	10 560
9-1L*	14.5	930	10 500	4900	59	7700
9-2L*	8.7	920	9600	3600	49	7630
9-2L†	8.7	920	8000	2400	39	7000
9-3L*	10.7	910	15 600	11 000	54	13 700
9-3L†	10.7	910	15 200	6600	37	13 920
9-1C†	10.7	930	7500	2400	53	5480
9-10†	10.7	930	8200	1400	45	6430
9-2C†	8.8	920	8900	1600	45	7000
10-1R*	10.2	980	9500	3600	52	7270
10-2R*	15.2	960	8600	4400	37	8010
10-1L*	14.8	980	8500	3700	50	6840
10-2L*	14.2	960	8000	3200	50	6340
10-1C*	8.7	980	14 000	6100	42	12 260
10-2C*	9.7	960	10 500	4000	51	8150
11-2R*	13.6	1030	10 600	9600	80	10 460
11-1L*	13.6	1060	10 000	4700	37	9190
11-2L*	10.7	1030	10 800	4600	25	10 580
11-1C*	8.2	1070	13 300	4300	50	10 210
11-1C†	8.2	1070	10 400	2200	46	8280
11-1C††	8.2	1070	12 300	3240	43	10 730
11-2C*	11.2	1050	11 400	4000	46	9320
11-2C†	11.2	1050	10 900	1800	38	9290
11-3C*	10.3	1020	11 900	5600	45	10 230
11-3C†	10.3	1020	10 000	2300	41	8410
13-1R†	12.0	1180	7400	2000	45	6110
13-1L†	12.0	1180	6000	2100	34	5610
13-1C†	10.0	1180	8800	4100	40	7950

NOTES: B = breadth of pillar.
Z = depth below surface.
σ_1 = major principal stress in pillar.
σ_p = pillar stress perpendicular to dip.
θ = angle from vertical in downdip direction to σ_1.
*Deformation meter measurements.
†Strain cell measurements.
††Measurements from 3 intersecting holes.

all directions. Again there are indications that beyond 250 ft (76 m) load transfer is not significant.

It was estimated that prior to removing the pillars the stresses in the surrounding pillars were 3000 to 5000 p.s.i., depending on location. Using the convergence measurements to estimate the increase in stress, the final stress on the surrounding pillars varied between 7000 to 11 000 p.s.i. These values are below the laboratory determined pillar strength of 16 000 to 23 000 p.s.i. (Bielenstein and Eisbacher 1970).

At the completion of the experiment, neither the roof nor pillars had failed, although the pillars did show signs of taking weight.

Models

Abutment Stresses by Photoelasticity

Strain relief measurements were made around one mine for the purpose of determining the field stresses and the stress concentrations in the abutment zone. Before this work was done it was clear how far from the mining zone increased stresses would extend. This factor can be important for development in rock that is likely to burst.

A theoretical solution for the stresses in the abutment of a circular opening has existed for some time in the subject of elasticity. More recently comparable equations were developed for elliptical and oval openings in an elastic

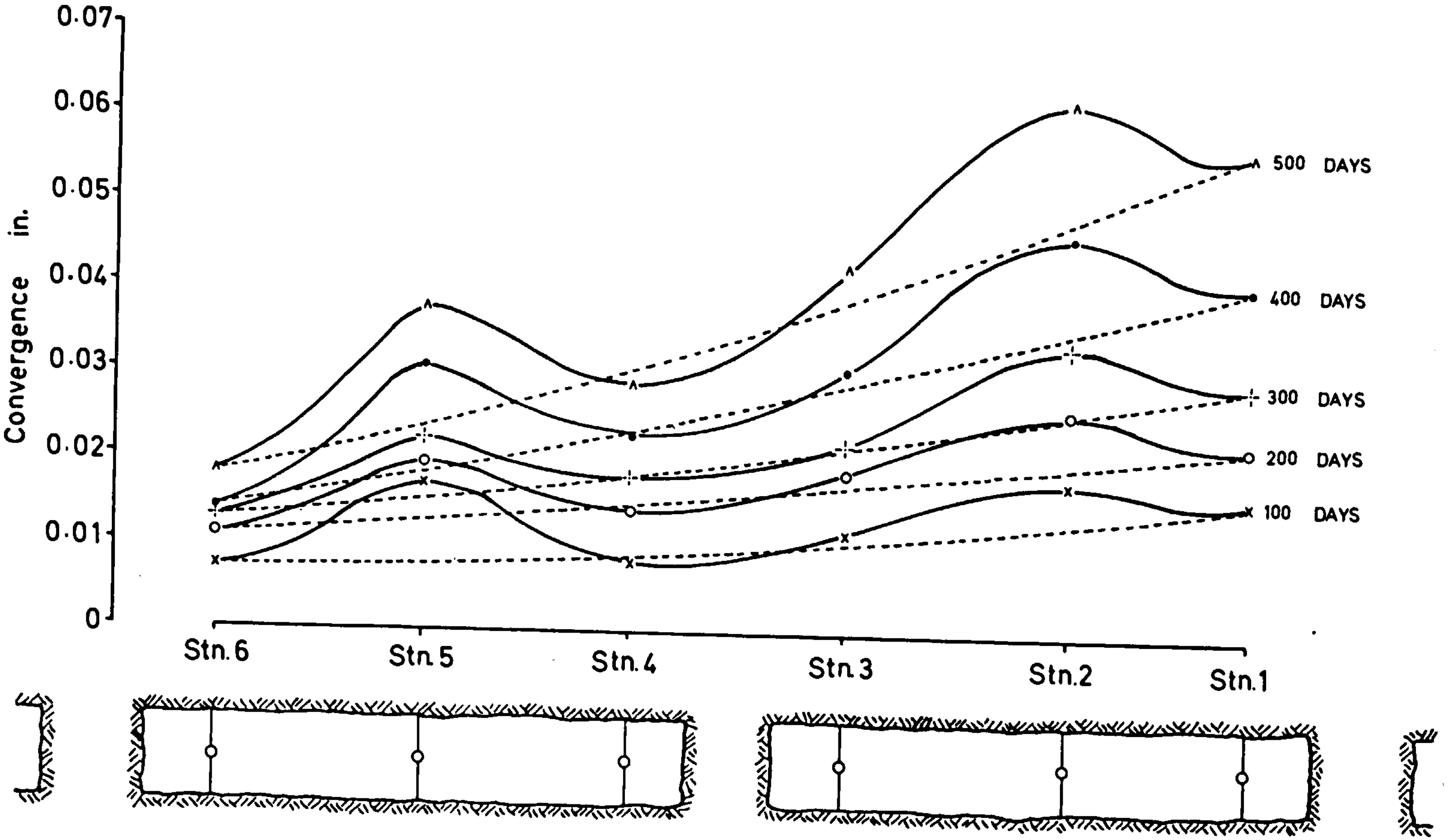


FIG. 18. Convergence profiles across Stope 13.11.

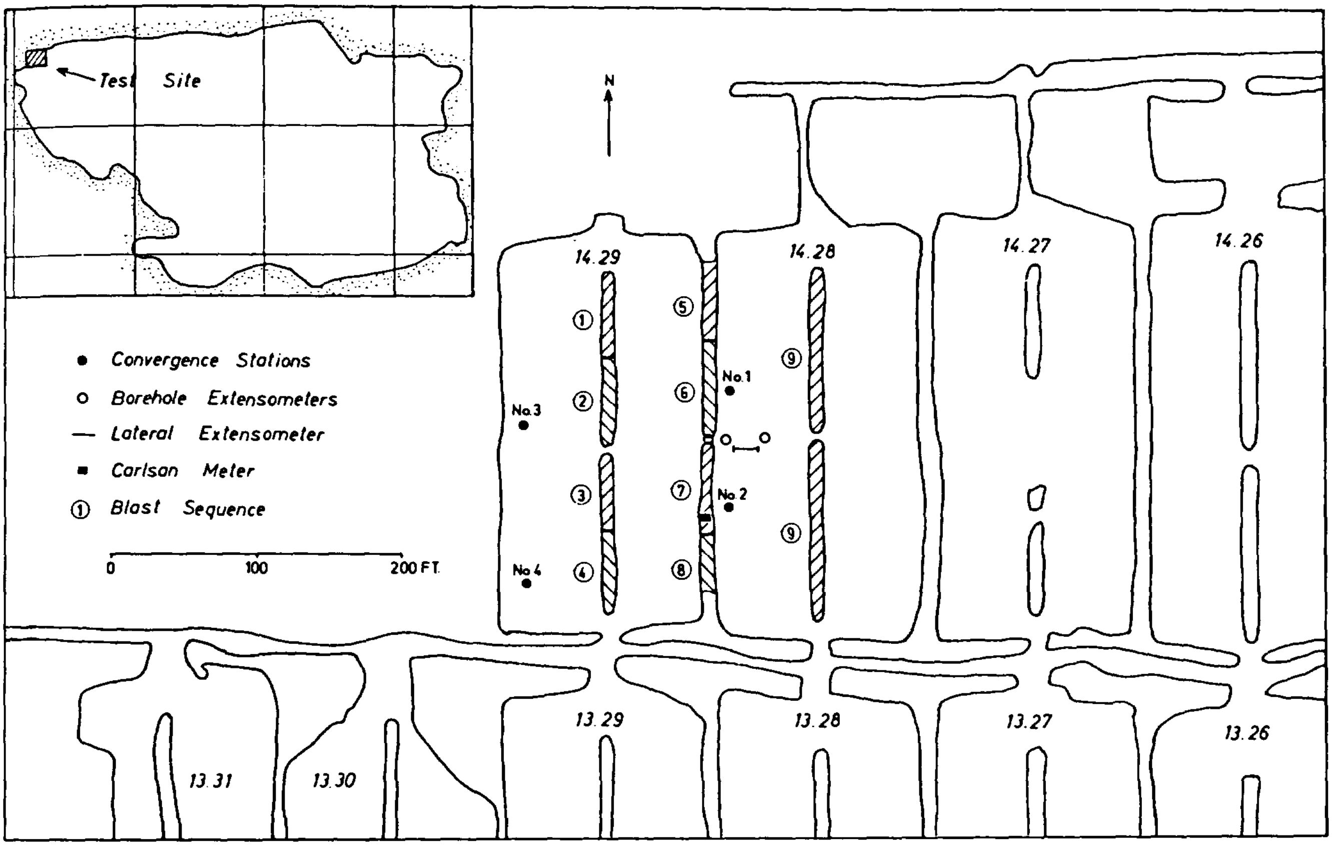


FIG. 19. Stope plan and instrument layout.

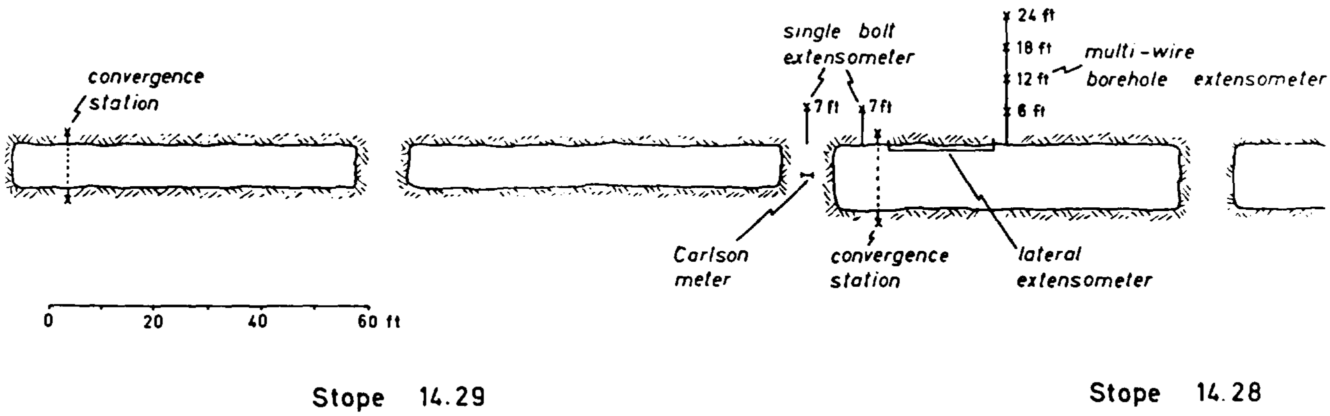


FIG. 20. Vertical section through instrumented area.

medium (see Fig. 22a). By comparison, the solution for ovals shows a reduced concentration in the major principal stress, σ_{ay}, owing to the increased radius of curvature at the mid-height of the opening together with a corresponding reduction in the minor principle stress, σ_{ax}, immediately behind the face.

For openings of equal height the distance into the abutment that would be subject to increased stresses varies with the span of the opening. For an axis ratio (span/height) of 1 the vertical stresses have not been significantly increased at a distance into the abutment equal to the height of the opening whereas for an axis ratio of 10 this distance is something like three times the height of the opening. On the other hand, this distance for an axis ratio of 10 is only approximately 1/3 of the span of the opening, which is less than for an axis ratio of 1.

Further study of the variation of abutment stresses showed that, whereas this critical distance was not proportional to the span, it did have a substantially linear variation. Further-

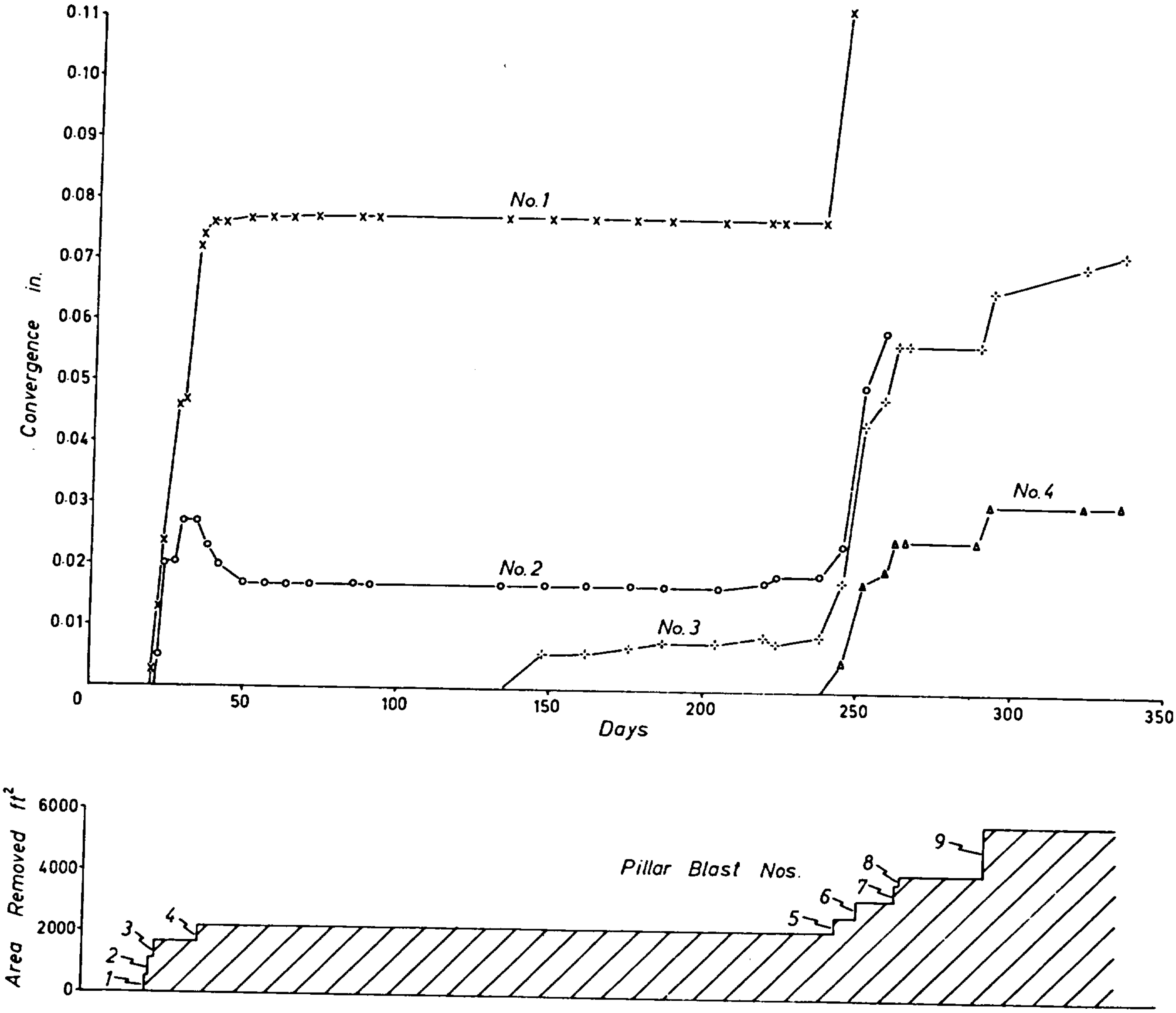


FIG. 21. Convergence resulting from pillar removal.

more, it was found that, although the theoretical stress concentration at the boundary would vary considerably depending on the shape of the opening (whether it was rectangular with fillets in the corner or whether it approached an elliptical shape), these stresses fell to a similar value at a short distance into the solid and thereafter varied substantially in the same common manner. From these observations, the following engineering equations were derived for determining an approximate value for the stress concentration at the face σ_A, and the various stresses into the solid, σ_{ay}, that would be applicable for relatively elastic rock (Coates 1970):

[1] $$\sigma_A/S_y = 0.5\ B_0/H + 1$$

[2] $$\sigma_{ay}/S_y = 0.09\ B_0/x + 1$$

where S_y is the field stress perpendicular to the long axis of the opening; B_0 is the breadth of the opening; H is the height of the opening; σ_{ay} is the stress at distances from 0.1 B_0 to 0.75 B_0 in from the face for openings with axes ratios up to at least 10 and for ratios of field stresses S_x/S_y in the range of at least 0.3 to 3; and x is the distance into the solid from the face. Considering the inelastic response of surface rock, even for relatively elastic ground, and actual irregularities in the geometry of openings, these approximate equations are considered to be sufficiently accurate for practical purposes.

Photoelastic models were used to obtain experimental data on abutment stresses for room and pillar geometry (Udd 1969). The results from the room-and-pillar models suggested that the abutment stresses around the mining

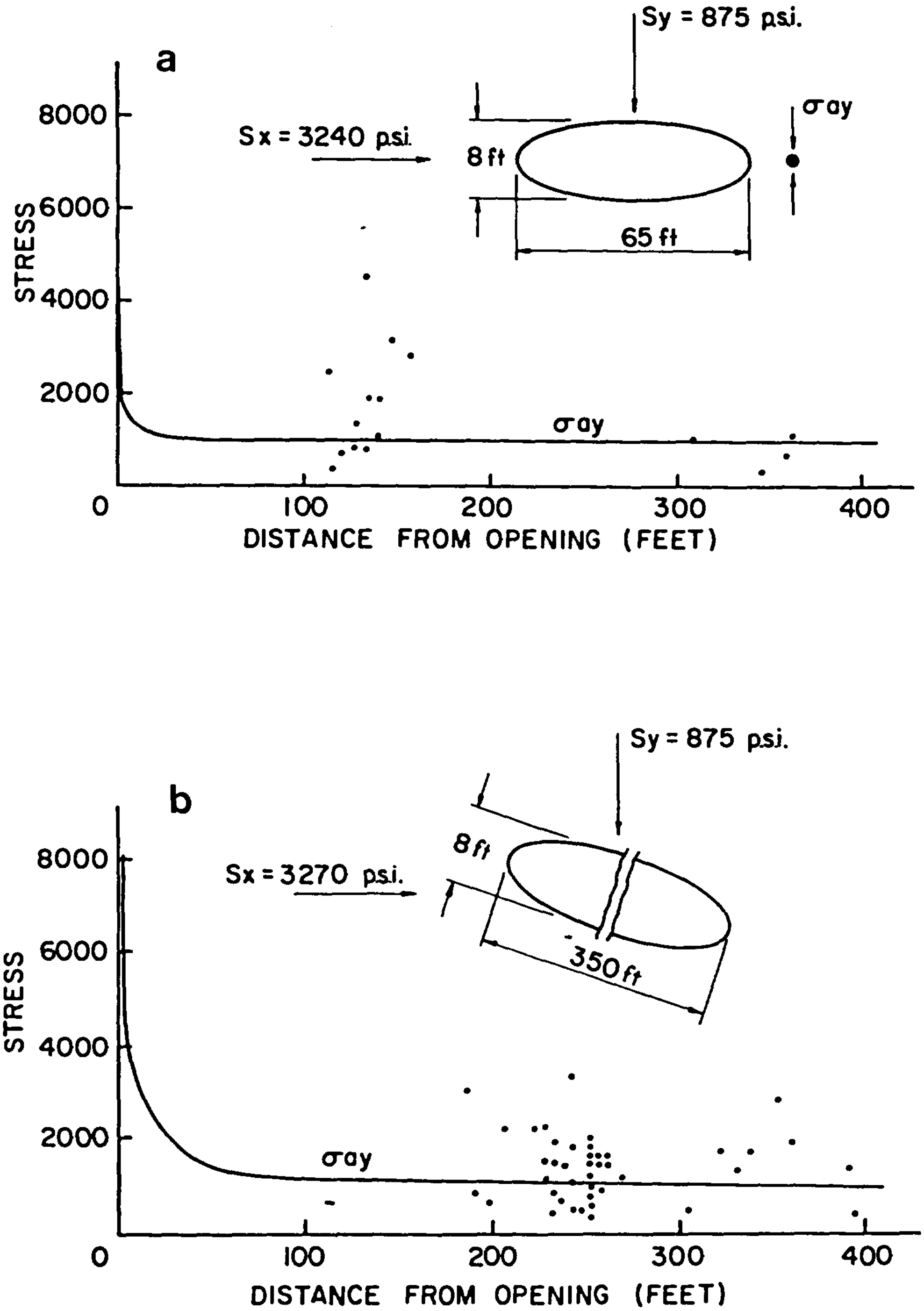


FIG. 22. a. Vertical stresses determined in the western abutment zone compared to the theoretical curve of stress variation around a single opening. b. Stresses in the northern abutment zone compared to the theoretical curve of stress variation.

zone were the same as those that would be produced by a single opening of the same dimensions as one stope; however, the data for the different models was not entirely consistent and later work showed that this suggestion was not valid (Coates and Yu 1970).

The photoelastic results were compared with the field measurements. Two hundred and eighty-one field measurements were made to determine the stresses in the abutment zones. Of these, many were rejected owing to the drift of the strain gauges, the failure of the cells to pass certain performance testing and the presence of rock fractures that were unacceptably close to the cells. Only 152 of the measurements were used for computations (Udd 1969).

Figure 22 shows the results of the field measurements in the abutment zones of one mine

(Udd 1969). As it turned out, most of the measurements were farther from the face of the outside stopes than would be desirable to obtain a measure of the stress gradient adjacent to the abutment face.

If it is assumed that the mining zone is approximately rectangular in shape with the dimensions of 2000 ft by 6000 ft (610 m by 1830 m), then the field stress normal to the orebody would pass around the mining zone in such a way that the central zone of the 6000 ft (1830 m) would be substantially in a condition of plane strain, *i.e.* a 2-dimensional case, whereas the ends would be a 3-dimensional case, *i.e.* the passage of stress would be around the sides and end.

Figure 22 shows the curves that would be obtained for the variation of the vertical stresses in the abutment zone using equations 1 and 2, which are applicable to a single opening. The data is not inconsistent with these curves, although finite element models later showed that, for a mining zone of several stopes, the actual stresses close to the abutment face would be somewhat greater than predicted by equations (1) and (2) (Coates and Yu 1970).

The Finite Element Method

Several novel mining methods had been considered. One would have had men only working at the faces of the stopes, which would have meant that possibly the backs could be left unsupported although the consequences would be of interest. Another would replace ore pillars with cemented fill pillars, and a third would involve mining a second reef. These changes would produce situations that had not been previously experienced. To provide some guidance to the operators in appraising such innovations, finite element models were used to compare the results from new geometrics with those for which the miners have experience.

Models with 20 ft (6 m) pillars and stopes 65 ft (20 m) in breadth and 10 ft (3 m) high were used. The average depth was 1000 ft (305 m), providing vertical field stresses equal to 1150 p.s.i. It was assumed that the horizontal stress for one case was 250 p.s.i. and for a second case was 4000 p.s.i.

A series of models were examined where 2, 3, 4, 5, and 6 stopes were excavated. These models show that the maximum elastic deflection (which is of the order of 0.1 in. (0.25 cm)) at the roofline only increased up to 25% with the excavation of adjacent stopes. They also show that the distance into the back that the ground moves down towards the stope is equal to two to three times the semi-breadth of the stope (or 1 to 1.5 times the breadth).

The model was examined with the pillars reduced to a breadth of 10 ft (3 m). Whereas the maximum deflections at the roofline increased by up to 20%, the curve of variation into the back was substantially the same as for the original case.

Another model was examined for stopes 20 ft (6 m) high. In this case, the maximum deflection did not increase and the variation into the back was substantially the same as for the previous models.

When the horizontal stress on the model was increased to 4000 p.s.i., the deflections were reduced by about 6%.

Finally, the effects on the deflection of the roofs of mining a second reef 100 ft (30.5 m) above the first were found to be insignificant, both when the stopes were aligned with the lower stopes and when the pillars were left over the lower stopes.

Figure 23 shows the variation of the deflection of the immediate roof from the centerline of the stope to the centerline of the pillar. Comparisons with previously determined analytical expressions for this variation are also shown (Coates 1966). Tensile stresses occurred in the roof of the central stope with a horizontal field stress of 250 p.s.i. About 80% of the roof would be subjected to some tension, but the extent of this zone into the back would be less than 0.5 of the semi-span, or 16 ft (5 m). It was found that if the actual horizontal stresses in the ground were more than 500 p.s.i., all tension would be eliminated from the backs; consequently, with the measured stresses in the area being well above this level, it seemed that there would be no requirement for long bolts in the central area of the stopes.

The effects were examined of recovering the rib pillars using cemented fill pillars with a modulus of deformation 1/20 of the rock). It was found, as shown in Figs. 24a and 24b, that the roof deflections were almost equal to those that would occur without any pillars. Furthermore, it was found that the stresses that would

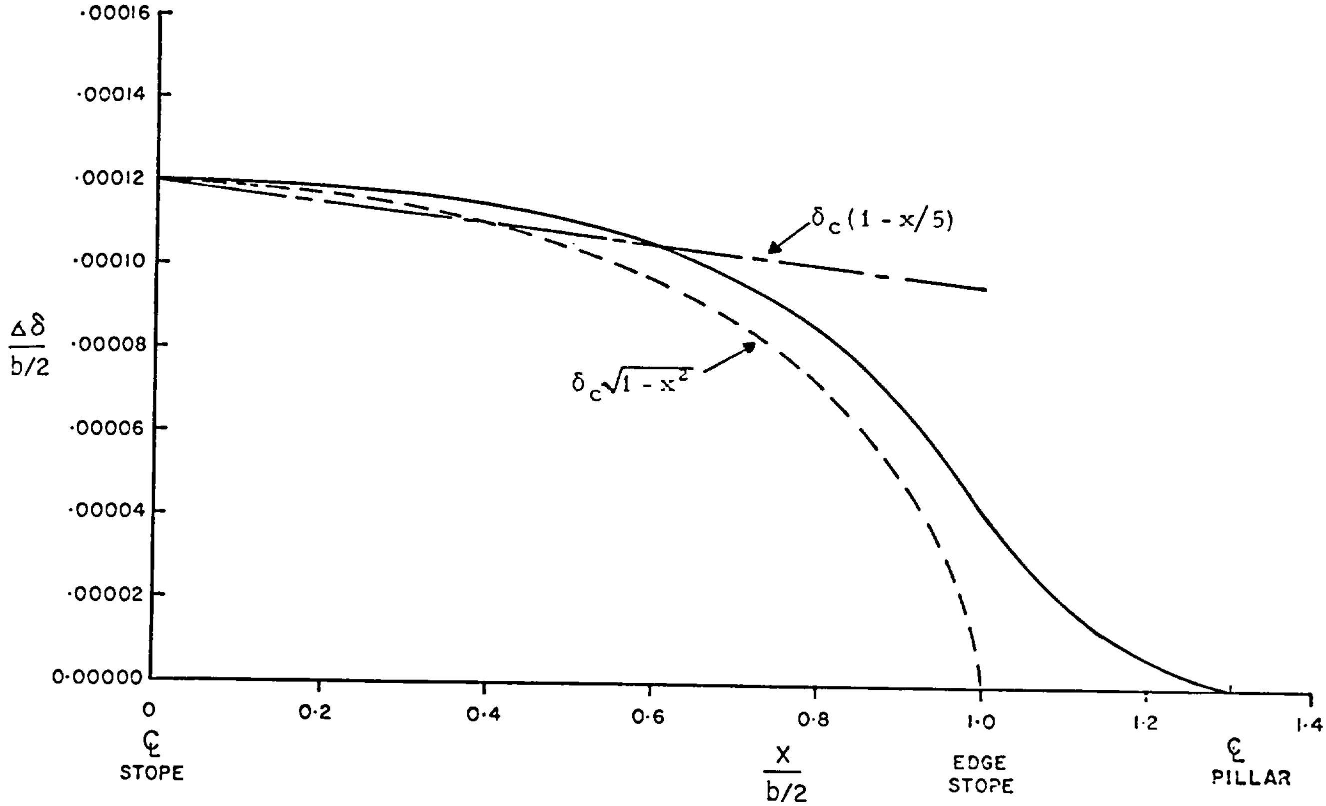


FIG. 23. Variation of deflection of the immediate roof between the centerline of a stope to the centerline of an adjacent pillar (all adjacent stopes excavated).

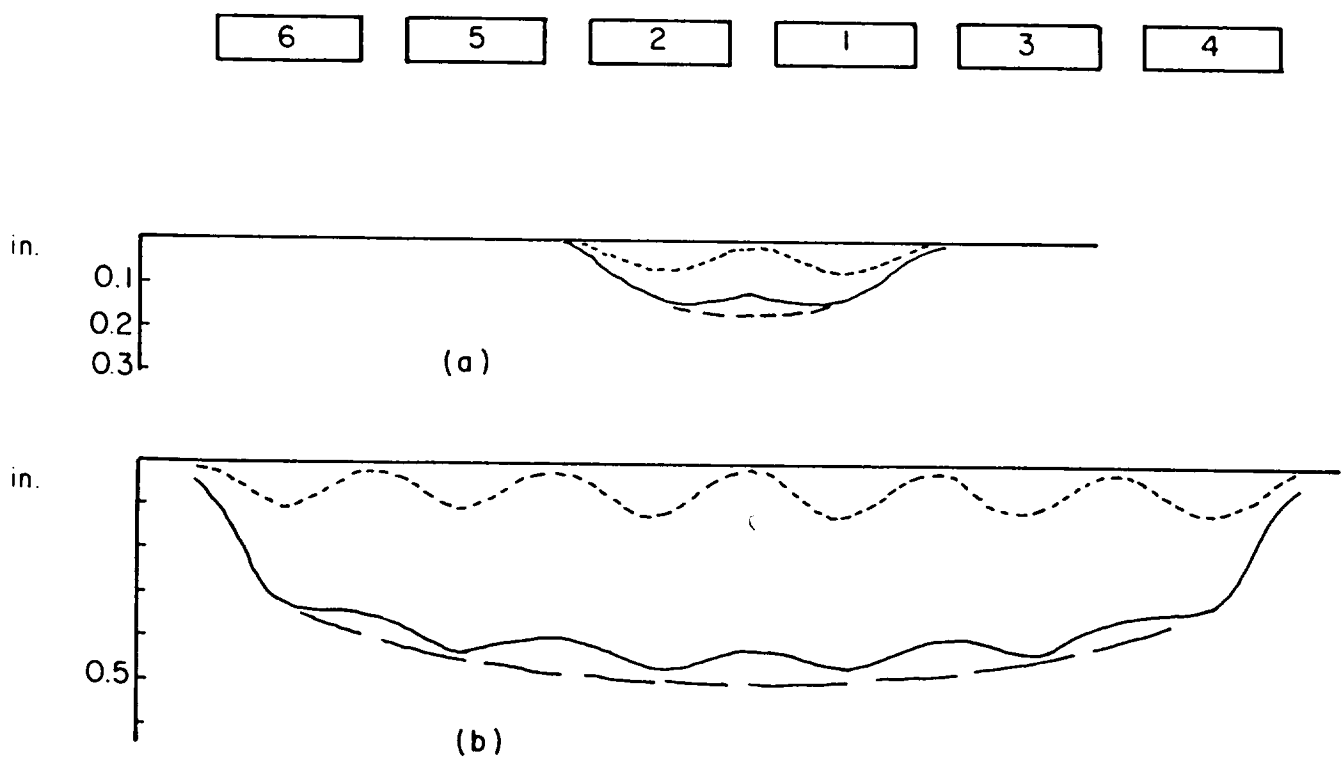


FIG. 24. Vertical deflections at the roof line.

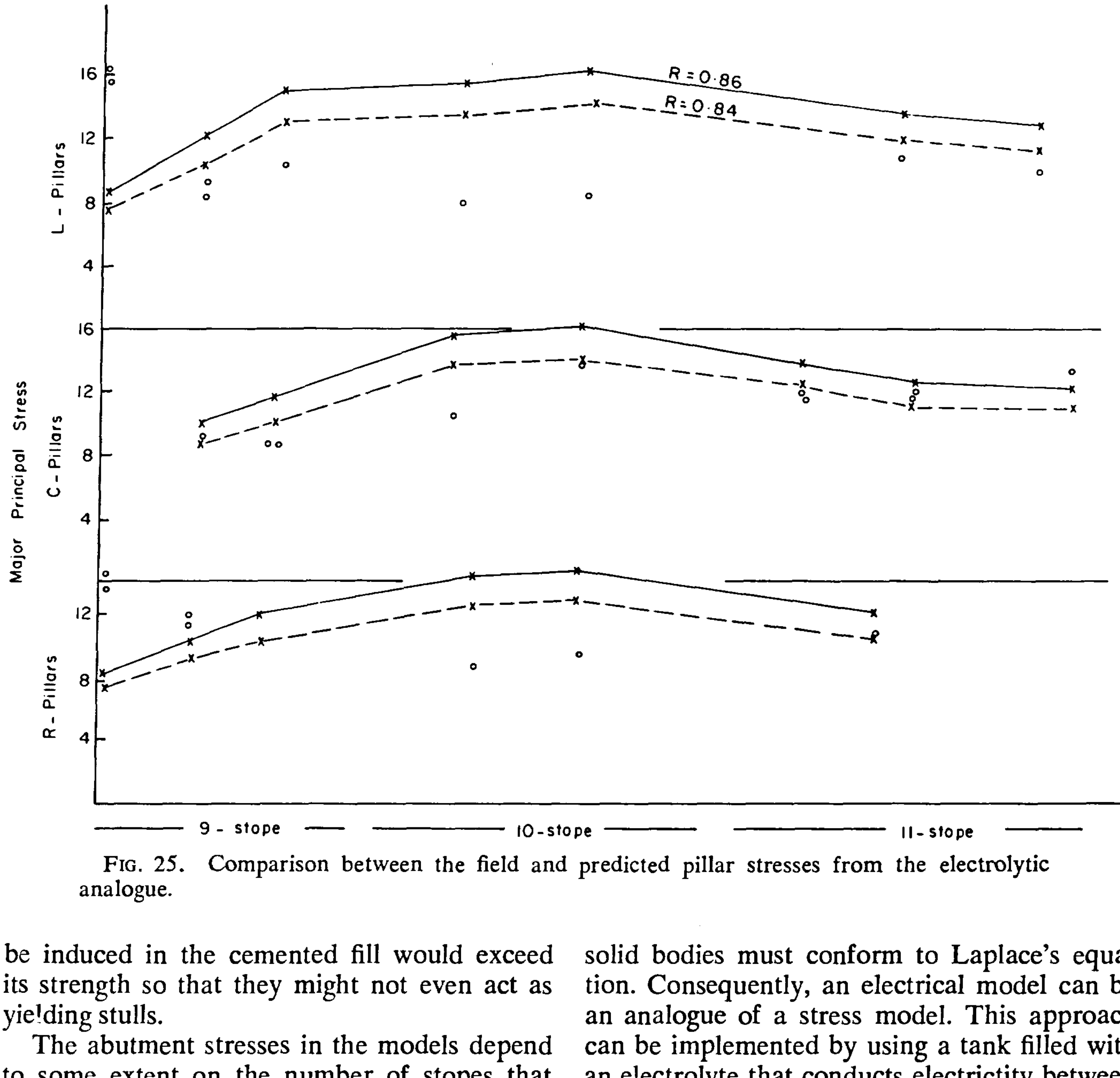


FIG. 25. Comparison between the field and predicted pillar stresses from the electrolytic analogue.

be induced in the cemented fill would exceed its strength so that they might not even act as yielding stulls.

The abutment stresses in the models depend to some extent on the number of stopes that have been mined. The distances to which the abutment stresses decrease to within 10% of the vertical field stress are typically 1.5, 2.6, to 3 times the semi-span of a stope for the cases of 1, 2, and 3 stopes being excavated. The effect on the abutment stresses of replacing the ore pillars with cemented fill was also examined. The maximum stress in the face increased from 4160 p.s.i. to 5640 p.s.i.

Electrolytic Analogue

It was mentioned above that the prediction of pillar and abutment stresses when plane strain conditions are not applicable requires a different approach. Steady state field theory shows that electrical currents and stress lines in solid bodies must conform to Laplace's equation. Consequently, an electrical model can be an analogue of a stress model. This approach can be implemented by using a tank filled with an electrolyte that conducts electrictity between a lower and upper electrode (Dhar 1970). The upper electrode contains the geometrical outline of a tabular but otherwise irregular orebody.

Figure 25 shows the results obtained from a model of Mine I compared with the field measurements. Figure 26 shows contours of the model values. In general, the predictions from the tank correspond to the field stresses very well. The maximum difference is about 30%, occurring where the pillars are located adjacent to two steeply dipping faults parallel to the line of pillars, which indicates that tectonic factors accounted for some of the differences (Dhar and Coates 1971).

Previously, a more comprehensive analysis

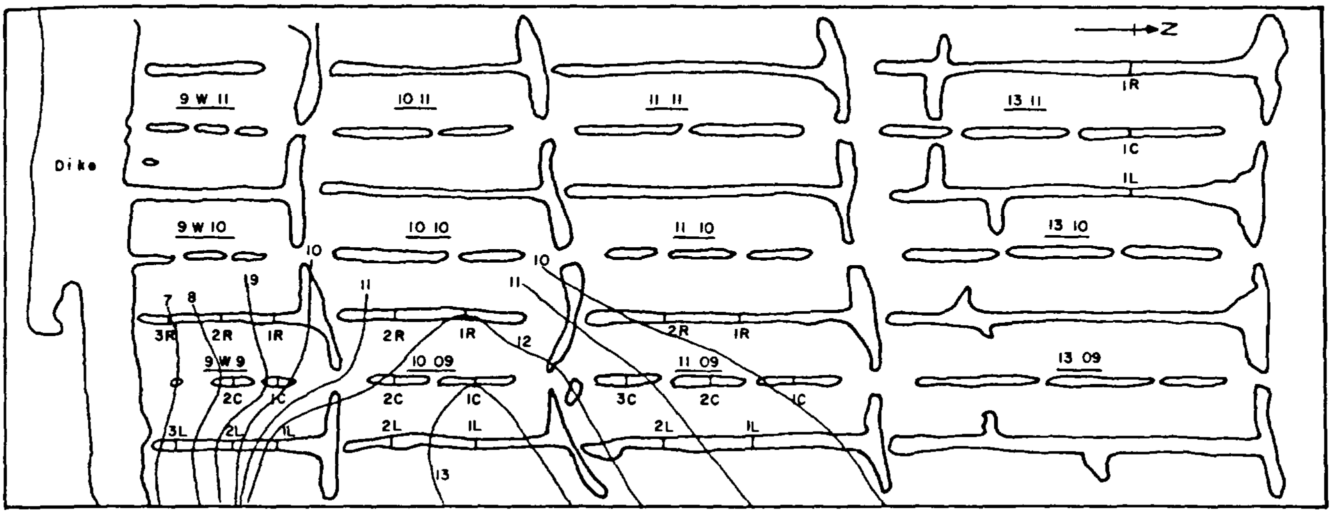


FIG. 26. Contours of predicted pillar stresses obtained from the electrical analogue in 1000's of p.s.i.

than the tributary area theory explained variations that would occur in pillar stresses across the span of a mining zone in plane strain conditions (Coates 1966). The electrolytic tank then showed how pillar stresses will vary throughout the mining zone as a result of the geometry of the boundaries (Dhar 1970). Superimposed on these variations, of course, are those resulting from the structural and tectonic conditions in the formation, which as yet cannot be accommodated in a model.

A second model was examined using the geometry of the mine but eliminating the large transverse dikes. This study clearly indicated that the pillar stresses would have been 50% to 100% greater without the important support that the dikes provided.

The variation of the abutment stresses with distance for models of the mine in 1965 and 1967 show that the amount of ground affected by the abutment stresses clearly is greater than that which would be associated with just one stope and is thus in agreement with the findings using the finite element models.

Conclusions

Testing

In general, progress has been made in the techniques of testing the rock substance and the rock mass; however, much remains to be done to be able to characterize adequately the mechanical properties of the rock mass.

The testing of uniaxial compressive strength and modulus of deformation for small samples of the rock substance, *i.e.* less than 6-in. (15.2-cm) diameter, has been satisfactorily resolved. A novel random sampling approach showed that the mean strength of such samples, many of which included fractures, is surprisingly little lower than that of solid samples; however, the dispersion of values in such a suite is much larger. The practical application of such data remains in the realm of research.

Size-effect operates in the quartzite so that below 1 in. (2.54 cm) the strength of the rock substance increases with size and then above 1 in. (2.54 cm) the expected decrease occurs. The relative decrease in stiffness of the testing machine with increase in size of sample may have contributed to the apparent decrease in strength with size. To predict the strength of the rock mass will require that the size of the structure be specified, *e.g.* it was found in 6-in. (15.2-cm) diameter holes that the strength of the rock in the walls corresponded to the strength of 5-in. (12.7-cm) diameter samples. Clearly the strength of the rock mass in a 10-ft (3-m) pillar would be different.

Brazilian tests were useful for quality control of stress, for relief requirements, and for determining the elastic properties of the rock substance.

The use of a borehole pressure cell is an attractive method for the determination of the modulus of deformation of the rock mass, but problems remain with the use of existing equipment in hard rock.

Seismic velocity was used successfully to detect relaxed zones in pillars, but the information is somewhat imprecise because it is not known whether the velocity along a straight line path has been reduced by the relaxation or the signal takes the route around the relaxed zone to produce the longer transit time.

The borehole penetrometer is useful for obtaining a quantitative measure of strength and/or modulus of deformation in soft seams, but trials in the quartzite indicated that this technique provided little practical information.

Tectonics

The objective of the geologic study, both regional and at the mine scale, was to elucidate the possible effect of tectonic controls on *in situ* stresses. In the process, the tectonic history of the region was resolved. The tectonic evolution provides an explanation for the existence of horizontal stresses greater than vertical stresses and for the major principal stress to be oriented parallel to the axis of the syncline. It was also shown that the major principal stress axis is essentially parallel to the strike of extension joint surfaces, even when the strike deviates from the predominant 090°-azimuth direction.

The geologic evidence indicates that the present stresses must contain a significant remanent component of the former tectonic stress field. The presence or absence of *in situ* stresses cannot be predicted on the basis of a geologic investigation. But, a detailed analysis of the tectonic evolution can be used to predict the most likely direction of the principal stress axes.

Underground Observations

After considerable experience with mining in geologic conditions that probably are more uniform than in most metal mines, the determination of stable spans of stopes and breadths of pillars can be done very well by judgment. However, for examining new layouts, possibly involving multi-reef mining or different mining methods, relatively simple theoretical analyses, particularly for the determination of stable pillar sizes, are rational and useful for extrapolation.

The stresses determined in pillars are found to be subject to considerable dispersion. A tectonic legacy of variable field stresses might explain such a dispersion; on the other hand, it is possible that the variations in geometry of the pillars, stopes and mining boundaries account for these variations. In any event, the complexity of the existence of such dispersion together with the inevitable variation of strengths suggested that at the present time the relatively simple tributary area theory for predicting pillar stresses in the central areas of mine is appropriate for design studies.

Because the horizontal stresses in the formation are greater than the vertical stress, the roofs of the stopes must be subjected to compressive stresses, which assists in maintaining stability over spans as much as 280 ft (85.4 m). At the same time, roof falls do occur, albeit with a low frequency that varies with the span of the stope and is strongly affected by the presence of faults.

Models

Model studies, theoretical developments and field measurements combined to provide new information on abutment stresses. Increased stress in the abutment zones extended into the solid a relatively limited distance, that is related substantially to the span of the adjacent stope.

The finite element method is flexible and useful in studying specific questions, particularly related to novel mining plans.

The electrolytic analogue, although less flexible than the finite element method, made it relatively easy to examine stress distributions in the tabular orebodies. Although the models were relatively idealized, they still were able to explain a significant source of some of the variance of pillar stresses—the irregular mining boundaries were shown to have a distinct effect.

Acknowledgments

The entire ground control group plus external associates contributed to this work. In particular, mention should be made of the valuable contributions of G. Eisbacher, F. Grant, B. Kostak, R. Parsons, J. Udd, Y. Yu, and G. Zahary.

In addition, we have been most grateful for the ready cooperation of Rio Algom Mines Ltd., Denison Mines Ltd., and of the other companies that have operated in the area.

AIRTH, M. W., and OLSEN, E. R. 1958. Algom–Nordic—development to production. Bull. Can. Inst. Mining Metall. **51**, no. 559 pp. 666–677.

BARRETT, R. E., BASSERMANN, R. R., BOURNE, B., FUTTERER, E., and HART, R. C. 1958. Mining methods and production costs of major Canadian uranium mines. Can. Min. J. **79**, no. 10, pp. 105–113.

BIELENSTEIN, H. U., and EISBACHER, G. H. 1969. Tectonic interpretation of elastic-strain-recovery measurements at Elliot Lake, Ontario. Mines Br., Res. Rep. R210.

——— 1970. *In-situ* stress determination and tectonic fabric at Elliot Lake, Ontario. Proceed. 6th Can. Symp., Rock Mech., Montreal, May, pp. 91–101.

COATES, D. F. 1966*a*. The effect of stress concentrations on the stability of tunnels. Proceed. 1st Congr. Int. Soc. Rock Mech., Theme 7, **11**, no. 14, pp. 299–305.

——— 1966*b*. Pillar loading, part III: field measurements. Mines Br., Res. Rep. R180, Information Canada.

——— 1970. Rock mechanics principles. Mines Br., Monogr. 874, Information Canada, 3rd ed.

COATES, D. F. and GRANT, F. 1966. Stress measurements at Elliot Lake. Bull. Can. Inst. Mining Metall. **59**, no. 649, pp. 603–613.

COATES, D. F., and PARSONS, R. C. 1966. Experimental criteria for classification of rock substances. Re. Int. J. Rock. Mech. Mining Sci. **3**, pp. 181–189.

COATES, D. F., and YU, Y. 1970*a*. A note on the stress concentations at the end of a cylindrical hole. Int. J. Rock Mech. Mining Sci. **7**, pp. 583–588.

——— 1970*b*. Deformations and stresses in the roofs of broad, multi-reef stopes. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.

COATES, D. F., UDD, J., and MORRISON, R. 1963. Some physical properties of rocks and their relationship to uniaxial compressive strength. Proceed. Rock Mech. Symp., McGill Univ. Mines Br., pp. 27–51.

COLLINS, W. H. 1925. North Shore of Lake Huron. Geol. Surv. Can., Mem. 183, 160 p.

DHAR, B. 1970. A three-dimensional study of pillar stresses in mines with irregular mining boundaries. Unpubl. Ph.D. Thesis, McGill Univ.

DHAR, B. and COATES, D. F. 1971. A three-dimensional study of pillar stresses in mines with irregular mining boundaries. Proceed. 6th Can. Symp. Rock Mech., Ecole Polytechnique, Mines Branch, pp. 131–148.

DHAR, B., GELDART, L., and UDD, J. 1970. Stresses in depth around elliptical and ovaloidal openings in an infinite elastic medium. Trans. Can. Inst. Mining Metall. **73**, pp. 94–101.

EISBACHER, G. H. 1969. Regional structural study in the Elliot Lake areas, Ontario, Geol. Surv. Can., Pap. 69–1A, pp. 158–161.

EISBACHER, G. H. and BIELENSTEIN, H. U. 1971. Elastic strain recovery in Proterozoic rocks near Elliot Lake, Ontario. J. Geophys. Res. **76**, no. 8, pp. 2012–2021.

FAIRBAIRN, H. W., HURLEY, P. M., CARD, K. D., and KNIGHT, C. J. 1969. Correlation of radiometric ages of Nipissing diabase and Huronian metasediments with Proterozoic orogenic events in Ontario. Can. J. Earth Sci. **6**, pp. 489–497.

HEDLEY, D. G. F. *et al.* 1970. Ground movement associated with pillar removal. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.

HEDLEY, D. G. F. and GRANT, F. 1972. Stope-and-pillar design for the Elliot Lake uranium mines. Bull. Can. Inst. Mining Metall. **65**, no. 723, pp. 37–44.

HERGET, G. 1970. Determination of elastic strain recovery in five inch diameter drill core subsequent to coring. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour., Ottawa, Can.

HOEPPENER, R. 1955. Tektonik in Schiefergebirge: Eine Einfuhrung. Geol. Rdsch., Bd. **44**, pp. 26–58.

KAMB, W. B. 1959. Ice petrofabric observation from Blue Glacier, Washington, in relation to theory and experiment. J. Geophys. Res. **64**, no. 11, pp. 1891–1909.

KOSTAK, B. and BIELENSTEIN, H. U. 1971. Strength distribution in hard rock. Int. J. Rock Mech. Mining Sci. **8**, pp. 501–521.

LAROCQUE, G. E. 1965. Sonic system for the determination of *in-situ* dynamic properties and for the outlining of fracture zones. Mines Br., Tech. Bull. 75, pp. 1–23.

LEEMAN, E. R. 1964. The measurement of stress in rock—part II: borehole rock stress measuring instruments. J.S.Afr. Inst. Min. Metall. **65**, no. 2, pp. 82–114.

MAMEN, C. 1956. Uranium mining methods. Can. Min. J. **77**, no. 6, pp. 89, 156, 157.

MCCUTCHEON, A. D. and FUTTERER, E. 1960. Mining operations at Consolidated Denison Mines Limited. Bull. Can. Inst. Mining Metall. **53**, no.h575, pp. 157–172.

MORRISON, R. G. K., CORLETT, A. V., and RICE, H. R. 1961. Report of the Special committee on mining practice. Part 1. Accidents and related representations (1959). Part 2. Accident review, ventilation, ground control, and related subjects, Ont. Dep. Mines, Bull. 155, Part 1, pp. 1–133, Part 2, pp. 1–120.

NORRIS, D. K. and BARRON, K. 1969. Structural analysis of features on natural and artificial faults. Proceed. 1st Conf. Res. Tectonics, Geol. Surv. Can., Pap. 68–52, pp. 136–167.

OBERT, L., MERRILL, R. H., and MORGAN, T. C. 1962. Borehole deformation gage for determining the stress in mine rocks. USBM RI 5978, pp. 1–11.

PANEK; L., HORNSEY, E., and LAPPI, R. L. 1964. Determination of the modulus of rigidity of rock by expanding a cylindrical pressure cell in a drill hole. 6th Symp. Rock Mech., Univ. Missouri, pp. 427–449.

PARSONS, R. C. 1968. Testing the size-effect on strength of the Elliot Lake quartzite. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.

PARSONS, R. C. and EVERELL, D. 1966. Determination of preferred planes of weakness in geological materials —part I. Unpub. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.

PARSONS, R. C. and SULLIVAN, J. D. 1965. Moduli determinations for the Elliot Lake stress relief overcoring work. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.

ROBERTSON, J. A. 1968. Geology of Township 149 and Township 150. Ont. Dep. Mines, Geol. Rep. 57, pp. 1–162.

——— 1969. Geology and uranium deposits of the Blind River area, Ontario. Bull. Can. Inst. Mining Metall. **62**, no. 686, June, pp. 619–634.

ROBERTSON, J. A., FRAREY, M. J., and CARD, K. D. 1969. The federal-provincial committee on Huronian stratigraphy (progress report). Can. J. Earth Sci. **6**, no. 2, pp. 335–336.

TERVO, R. and LYALL, K. D. 1968. Rock breakage research program report project 54—impact fracture studies. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.

TURNER, F. J. and WEISS, L. E. 1963. Structural analysis of metamorphic tectonites. McGraw Hill, Toronto, pp. 15–16.

UDD, J. E. 1969. Studies relating to abutment stresses for mines. Unpubl. Ph.D. Thesis, McGill Univ., 324 p.
UDD, J. E. and GRANT, F. 1967. Measurements of stresses in the abutment zones, Nordic Mines, Rio Algom Mines Ltd., Elliot Lake, Ontario. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.
VANHEERDEN, W. L. and GRANT, F. 1967. A comparison of two methods for measuring stress in rock. Int. J. Rock Mech. Mining Sci. 4, pp. 376–382.
VANSCHMUS, W. R. 1965. The geochronology of the Blind River – Bruce Mines area, Ontario. J. Geol. 73, no. 5, pp. 755–780.
VANDERZEE, T. J. 1966. Evaluation of the borehole penetrometer for estimating rock strength. Unpub. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.
YU, Y. S. 1966. Sonic measurements at Nordic Mine, Rio Algom Mines Limited, Elliot Lake, Ontario. Unpubl. Rep on file, Mines Br., Dep. Energy, Mines, Resour.
——— 1967. Diametrical compression tests for the Elliot Lake strain-relief overcoring work. Unpub. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.
ZAHARY, G. 1967. Modulus of rigidity from pressuremeter tests. Unpubl. Rep. on file, Mines Br., Dep. Energy, Mines, Resour.