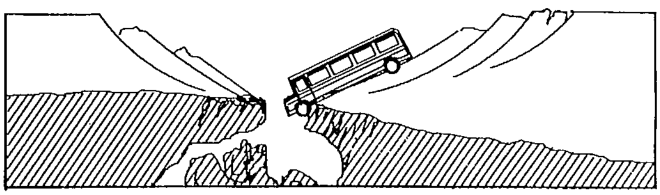
# *SURFACE CROWN PILLAR EVALUATION FOR ACTIVE AND ABANDONED METAL MINES*

PROCEEDINGS OF THE INTERNATIONAL CONFERENCE



# *ÉVALUATION DE PILIERS DE SURFACE POUR LES MINES DE MÉTAUX ACTIVES ET ABANDONNÉES*

COMPTE RENDU DE LA CONFÉRENCE INTERNATIONALE

*Senator Hotel*

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# FOREWORD

Over the last several years, surface crown pillar failures in active and abandoned mines have gained the attention of the mining industry and civic officials. The interest and research generated by these occurrences have led to the realization that the subject of surface crown pillars is diverse and complex, both in a geomechanical sense and a liability sense. The problems mine operators face do not go away with time and may even worsen, so that the subsequent encroachment by housing developments, schools, highways and railways may prove very risky.

In general, the public assumes that many civic structures built on or close to bedrock are free of risk. However, developers are generally unaware of the presence of underground workings and the need for remedial measures before construction begins.

Recent advancements in the understanding of surface crown pillars demonstrate that mine operators must carefully design these pillars to ensure stability. It is also apparent that municipal planners must be more aware of the location and structural integrity of old mine workings.

The conference objective is to provide a forum for the discussion of the short- and long-term behaviour of surface crown pillars and thereby advance the knowledge of how to deal with them safely.

William Mackasey

W.O. Mackasey
Conference Chairman

Marc Bétournay

M.C. Bétournay
Technical Chairman

# AVANT-PROPOS

Depuis quelques années, les effondrements de piliers de surface au-dessus de mines actives et abandonnées ont attiré l'attention de l'industrie minière et des autorités civiles. L'intérêt et la recherche générés par ces comportements ont démontré que le sujet des piliers de surface est varié et complexe, tant au niveau géomécanique qu'au niveau de la responsabilité. Les problèmes auxquels font face les opérateurs miniers ne se dissipent point et peuvent même s'aggraver, ce qui rend dangereux, par exemple, le rapprochement de nouveaux développements résidentiels, de routes, de chemins de fer, etc.

En général, le public croit que les structures civiles érigées sur ou près du socle rocheux ne comportent aucun risque. Les planificateurs des développements communautaires ignorent souvent la présence de chantiers souterrains et le besoin de travaux réparateurs.

Les récents progrès sur le sujet des piliers de surface indiquent que les opérateurs miniers doivent être prudents et doivent concevoir des piliers stables. Une meilleure connaissance des sites et des conditions des vieux chantiers miniers est requise de la part des planificateurs municipaux.

L'objectif de la conférence est de fournir un lieu de rencontre pour discuter des comportements à court et à long terme des piliers de surface, et pour améliorer les connaissances afin de les traiter de façon sécuritaire.

William Mackasey

W.O. Mackasey
Président de la conférence

Marc Bétournay

M.C. Bétournay
Président du programme technique

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KEYNOTE ADDRESS / ALLOCUTION CLÉ

*E. Hoek*

# A LIMIT EQUILIBRIUM ANALYSIS OF SURFACE CROWN PILLAR STABILITY

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## ABSTRACT

One of the failure mechanisms involved in a study of the stability of surface crown pillars is that involving downward sliding of the pillar under gravitational loading. This sliding occurs when the total shear strength of surfaces which define the boundaries of the pillar is overcome by the weight of the pillar. The total shear strength is dependent not only upon the properties of the rock mass but also upon the horizontal in situ stresses acting upon the pillar and upon the groundwater pressures within the pillar. A limit equilibrium analysis of this type of failure is pre-‧nted. The results of the analysis are displayed as a distribution of possible factors of safety and a probability of pillar failure, determined from the likely distribution of values for the input parameters. A BASIC listing of the program is given at the end of this paper.

## RÉSUMÉ

Un des mécanismes de rupture à considérer lors de l'étude de stabilité des piliers est le glissement descendant du pilier sous les charges de gravité. Le glissement se produit lorsque le poids du pilier dépasse le total des forces de cisaillement des surfaces définies par les contours du pilier. La force de cisaillement totale dépend non seulement des propriétés de la masse rocheuse mais aussi des contraintes horizontales in situ agissant sur le pilier et sur la pression de l'eau souterraine du pilier. Une analyse à l'état limite de ce type de rupture est décrite dans cet article. Les résultats de l'analyse sont présentés sous la forme d'une distribution des facteurs de sécurité possibles et par une probabilité de rupture du pilier. Les valeurs sont déterminées à partir de la distribution probable des données des paramètres d'entrée. Un listage du programme en BASIC est fourni à la fin de cet article.

## INTRODUCTION

An evaluation of the stability of surface crown pillars presents special problems for geotechnical engineers. The rock mass forming the pillar is usually weathered and it may also be covered with a layer of overburden soil. Consequently, the properties of this rock mass are difficult to quantify with any degree of certainty. Similarly, the in situ stresses acting on the pillar and the elevation of the groundwater table in the pillar may not be well defined.

A number of failure mechanisms have been postulated to explain surface crown pillar failures (Bétournay, 1987) and it is possible that there are other mechanisms which have not yet been investigated. It is also likely that more than one failure mechanism may operate in any given crown pillar and that the stability of that pillar is controlled by that mechanism or combination of mechanisms which give the lowest factor of safety for the unique set of dimensions and properties of that particular pillar.

In view of these uncertainties it is clear that any attempt to define a precise factor of safety for a surface crown pillar is unlikely to be successful. In order to provide a basis for practical decision making, an analysis should include some form of sensitivity study or Monte Carlo analysis which looks at the range of factors of safety associated with variations in material properties, in situ stresses and groundwater conditions (Priest and Brown, 1983). The limit equilibrium analysis presented in this paper deals with one mechanism of surface crown pillar failure and gives a mean factor of safety as well as a normal distribution of safety factors and a probability of failure.

## LIMIT EQUILIBRIUM ANALYSIS

Consider a rectangular horizontal crown pillar with plan dimensions $x$ and $y$ as shown in figure 1. The thickness of the pillar is $z$ and the water level is assumed to be at a level of $z_w$ above the base of the pillar. The pillar is acted upon by the horizontal effective stresses $\sigma'_x$ and $\sigma'_y$.

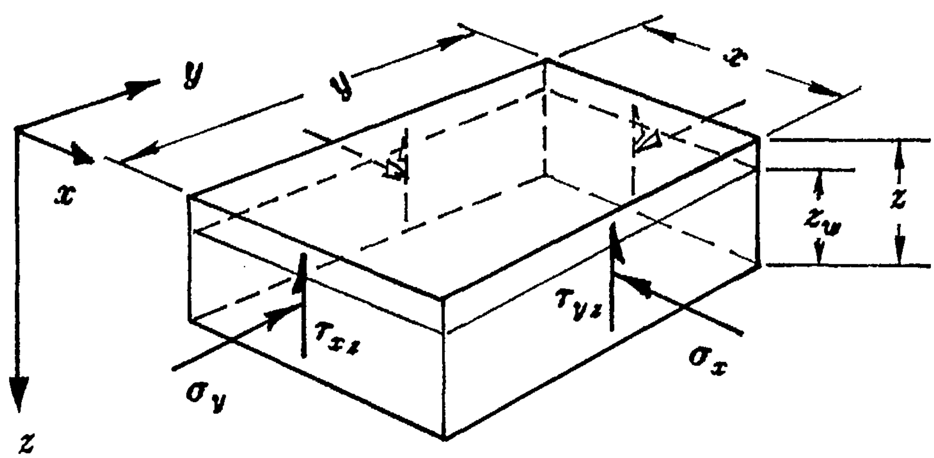


**Figure 1 :** Dimensions of pillar and directions of lateral stresses.

The factor of safety of the pillar against vertical downward sliding is given by the ratio of the sum of the shear forces acting on the four sides of the pillar to the weight of the pillar. Hence

$$F = \frac{2(\tau_{xz}.xz + \tau_{yz}.yz)}{\gamma_r.xyz} \tag{1}$$

giving

$$F = \frac{2}{\gamma_r}\left(\frac{\tau_{xz}}{y} + \frac{\tau_{yz}}{x}\right) \tag{2}$$

where $\gamma_r$ is the unit weight of rock and $\tau_{xz}$ and $\tau_{yz}$ are the shear strengths of the $xz$ and $yz$ faces respectively.

These shear strengths can be defined by the angle of friction $\phi'$ and the cohesion $c'$ of the Mohr-Coulomb failure criterion or they can be determined directly from the non-linear failure criteria proposed by Barton (1976) or by Hoek and Brown (1980, 1988). For the purposes of this discussion, the Hoek-Brown failure criterion will be used since this includes a means of estimating the shear strength directly from the rock mass quality determined in the field.

The shear strength $\tau_{xz}$ is given by :

$$\tau_{xz} = \left(\cot\phi'_{ixz} - \cos\phi'_{ixz}\right)\frac{m\sigma_c}{8} \tag{3}$$

$$\phi'_{ixz} = \arctan\frac{1}{\sqrt{4h_{xz}\cos^2\theta_{xz} - 1}} \tag{4}$$

$$\theta_{xz} = \frac{1}{3}\left(90 + \arctan\frac{1}{\sqrt{h_{xz}^3 - 1}}\right) \tag{5}$$

$$h_{xz} = 1 + \frac{16(m\sigma'_y + s\sigma_c)}{3m^2\sigma_c} \tag{6}$$

where $m$ and $s$ are the material constants of the Hoek- Brown failure criterion and $\sigma_c$ is the uniaxial compressive strength of the intact rock material. Note that the shear strength $\tau_{xz}$ depends upon the magnitude of the lateral stress $\sigma'_y$ while $\tau_{yz}$ depends upon $\sigma'_x$ as shown in figure 1.

The shear strength $\tau_{yz}$ is given by substitution of $yz$ values in place of the $xz$ subscripted parameters in equations 3, 4 and 5 and $\sigma'_x$ in place of $\sigma'_y$ in equation 6.

Assuming that the material forming the surface crown pillar has been disturbed by blasting, percolation of groundwater and movement of the rock mass surrounding the stope, the Hoek-Brown material constants $m$ and $s$ can be estimated from Bieniawski's rock mass rating ($RMR$) value by the following equations (Hoek and Brown, 1988) :

$$m = m_i.\exp\left[\frac{RMR - 100}{14}\right] \tag{7}$$

**Table 1 : Approximate values of constant $m_i$ for different rock types.**

| | |
|---|---|
| Carbonate rocks with well developed crystal cleavage<br>*dolomite, limestone and marble* | $m_i = 7$ |
| Lithified argillaceous rocks<br>*mudstone, siltstone, shale and slate (normal to cleavage)* | $m_i = 10$ |
| Arenaceous rocks with strong crystals and poorly developed crystal cleavage<br>*sandstone and quartzite* | $m_i = 15$ |
| Fine grained polymenerallic igneous crystalline rocks<br>*andesite, dolerite, diabase and rhyolite* | $m_i = 17$ |
| Coarse grained polymenerallic igneous & metamorphic crystalline rocks<br>*amphibolite, gabbro, gneiss, granite, norite, quartz- diorite* | $m_i = 25$ |

$$s = \exp\left[\frac{RMR - 100}{6}\right] \tag{8}$$

where $m_i$ is the value of the constant $m$ for the intact rock material given in table 1.

The lateral effective stresses $\sigma'_x$ and $\sigma'_y$ can be determined from the dimensions of the crown pillar and the water level in the pillar as defined in figure 1.

$$\sigma'_x = \frac{\gamma_r.z.K_x}{2} - \frac{\gamma_w.z_w^2}{2z} \tag{9}$$

$$\sigma'_y = \frac{\gamma_r.z.K_y}{2} - \frac{\gamma_w.z_w^2}{2z} \tag{10}$$

where $K_x$ and $K_y$ are the ratios of horizontal to vertical stress in the $x$ and $y$ directions respectively and $\gamma_w$ is the unit weight of water.

In order to determine the factor of safety of a surface crown pillar subjected to failure by shear of the four faces as defined in figure 1, the calculation follows the equations listed in reverse order. This calculation is included in the simple BASIC program listing given at the end of this paper.

## FAILURE PROBABILITY ANALYSIS

In view of the high level of uncertainty associated with many of the input parameters for the limit equilibrium analysis, as discussed in the introduction, a study of the likely distribution of factors of safety and of the probability of pillar failure is presented next.

For the purposes of this analysis it will be assumed that the pillar dimensions, defined by $x, y$ and $z$, and the unit weights of rock ($\gamma_r$) and water ($\gamma_w$), are known with a high enough degree of precision that they can be assigned unique values. The other six parameters are the water depth in the pillar ($z_w$), the rock mass rating ($RMR$), the uniaxial compressive strength of the intact rock material ($\sigma_c$), the value of the Hoek-Brown constant $m$ for the intact rock material ($m_i$), the horizontal to vertical stress ratio ($K_x$) in the $x$ direction and the horizontal to vertical stress ratio ($K_y$) in the $y$ direction. These six parameters are each defined by a mean value and a standard deviation and it is assumed that the values are normally distributed around the mean.

The mean $\overline{V}$ of a set of observations of a variable $V$ is given by

$$\overline{V} = \frac{\sum_{i=1}^{n} V_i}{n} \tag{11}$$

where $V_i$ is the $i$th observation and
$n$ is the number of observation

The sample variance $S^2$ is the square of the standard deviation $S$ and is given by

$$S^2 = \frac{\sum_{i=1}^{n} (\overline{V} - V_i)^2}{n-1} \tag{12}$$

The coordinates $(Y, V_i)$ of the curve defining the normal distribution of the variable $V$ are calculated from

$$Y = \frac{1}{S\sqrt{2\pi}} \, e^{-\frac{1}{2}\left(\frac{V_i - \overline{V}}{S}\right)^2} \tag{13}$$

One method which can be used to investigate the distribution of factors of safety is the Monte Carlo technique. In applying this method to the surface crown pillar problem, a random number generator would be used to determine a value $V_i$ for each of the six normally distributed variables and these values would then be used to compute a factor of safety. If this process is repeated say 1000 times, a histogram of the factor of safety distribution can be plotted.

The Monte Carlo technique is computationally intensive and, in some applications it can be replaced by Rosenbleuth's point estimate method (Rosenbleuth, 1972). This technique is based upon the fact that a quantity, calculated from an equation containing a number of normally distributed variables, will itself be normally distributed. Hence, for each variable, two point estimates are made at fixed values of one standard deviation on either side of the mean ($\overline{V} \pm S$) and the equation is solved for every possible combination of point estimates. This produces $2^m$ solutions, where $m$ is the number of normally distributed variables involved.

In the case of the surface crown pillar, the factor of safety equation contains six normally distributed variables and hence 64 values of the factor of safety are obtained from this process. The mean and standard deviation for the factor of safety are calculated by means of equations 11 and 12 and the normal distribution is plotted from equation 13. Figure 2 illustrates the screen display given by the program, listed at the end of this paper, in which the normal distribution of the factor of safety is calculated using Rosenbleuth's method. This distribution is plotted for a range of $\pm$ 3 standard deviations on either side of the mean factor of safety.

The probability of failure is given by the ratio of the area under the normal distribution curve for factor of safety values from $-\infty$ to 1 (shown shaded in figure 2) to the area under the curve from $-\infty$ to $+\infty$.

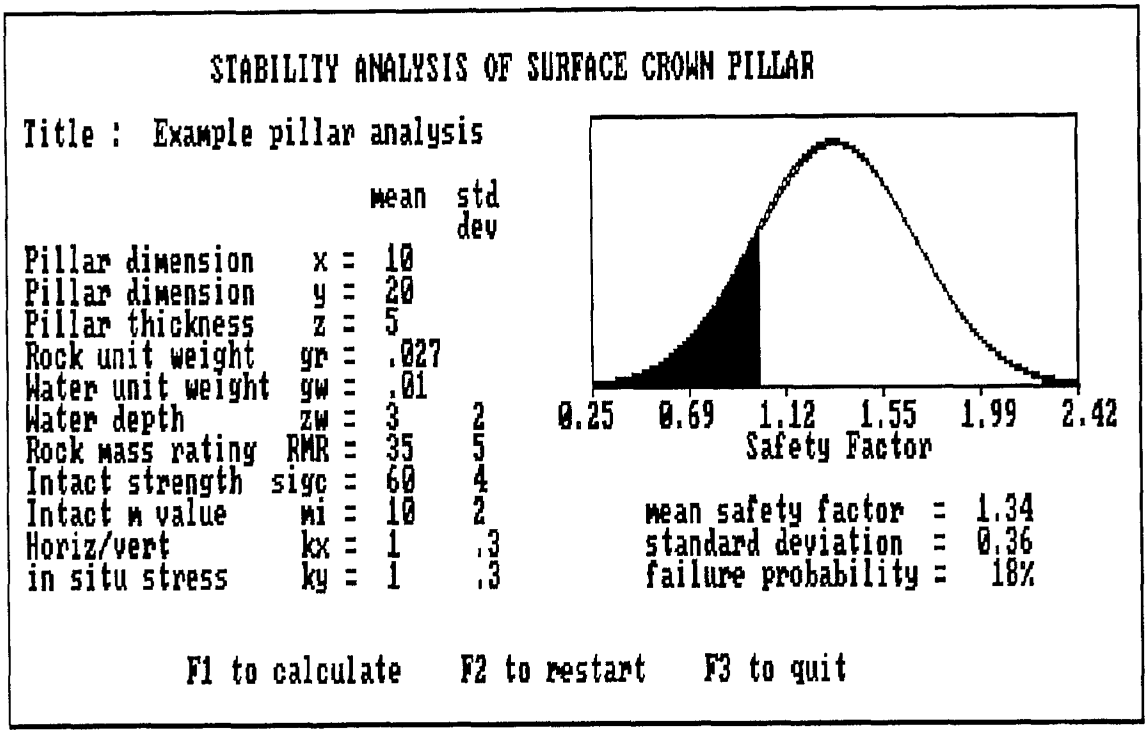


**Figure 2 :** Screen display of input data and calculated factor of safety distribution and probability of failure.

## SENSITIVITY STUDY OF FACTOR OF SAFETY

The program given at the end of the paper can be used to determine the safety factor distribution and probability of failure, as shown in figure 2, or it can be used to calculate a unique factor of safety by entering zero values for all the standard deviations. This method can be useful when it is desired to explore the sensitivity of the factor of safety to changes of each of the variables in turn.

Figure 3 illustrates the results of one such study for a case in which the following values were chosen to give a starting value of 1.00 for the factor of safety :

x = 7 m, y = 20 m, z = 5 m, gr = 0.027 MN/m$^3$, gw = 0.01 MN/m$^3$, zw = 5 m,
RMR = 40, sigc = 60 MPa, mi = 10, kx = 0.5, ky = 0.5

Figure 3 shows that the factor of safety is very sensitive to the value of the rock mass rating (RMR), particularly for values in excess of about 50. This trend is in agreement with practical observations which would suggest that this type of failure is unlikely in very high quality rock masses.

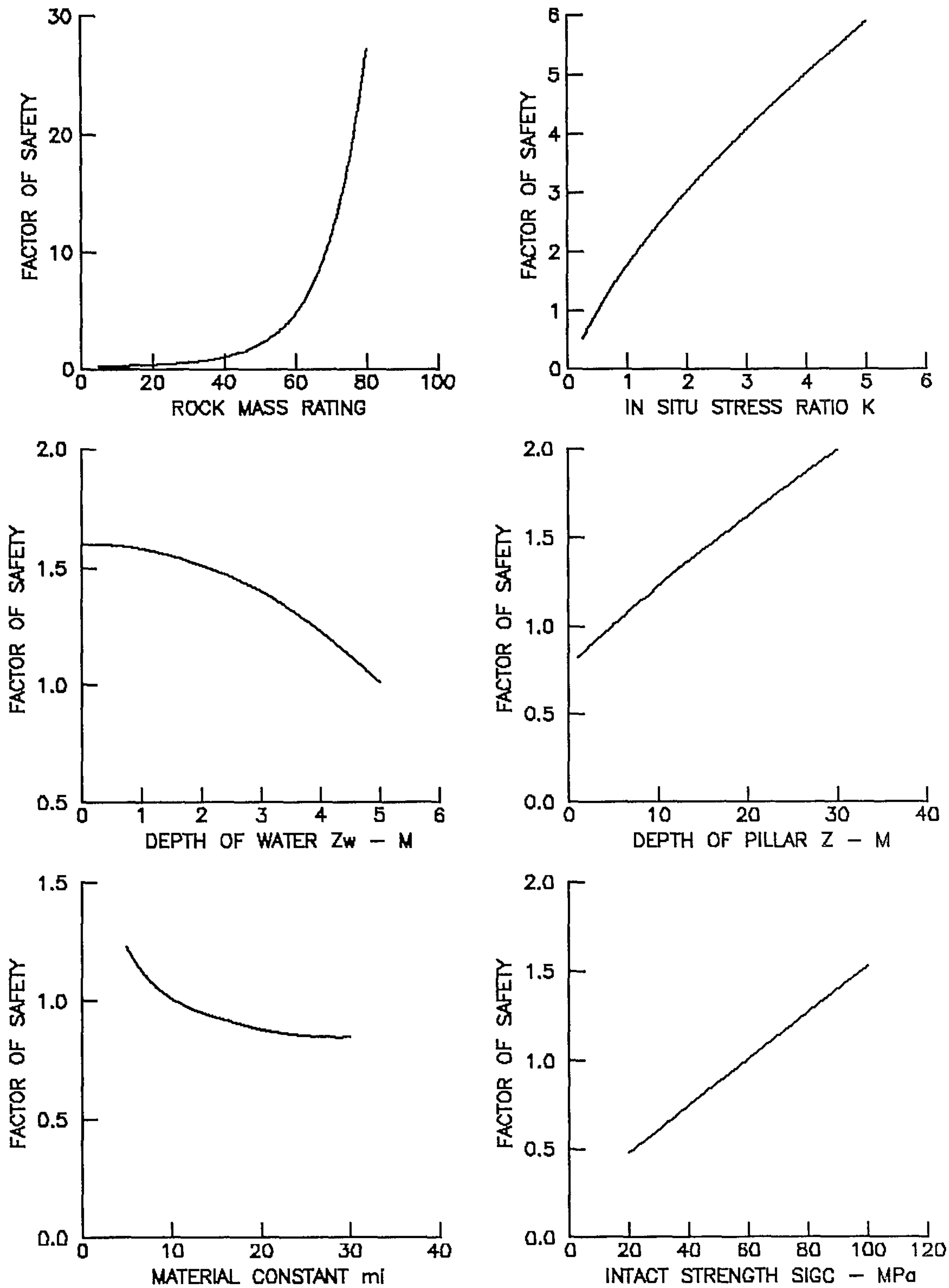


**Figure 3 :** Sensitivity of factor of safety to input parameter variations

The factor of safety is also very sensitive to changes in the value of the ratio of horizontal to vertical in situ stresses. Again, this trend is consistent with intuitive reasoning based upon practical experience. It also suggests that decreases in lateral stresses acting on the pillar, resulting from progressive failures of pillars or support in underlying stopes, may be responsible for some of the time-dependent surface pillar failures which occur in abandoned mines.

Variations in the other parameters involved in this analysis do not result in large changes in factor of safety. This suggests that failure induced by slow deterioration in rock strength or by fluctuations in groundwater level would only occur in pillars which have a factor of safety very close to 1.

## CONCLUSIONS

The analysis discussed in this paper deals with one type of surface crown pillar failure. No claim is made that this is the only or even the most important failure mechanism and the reader should be aware that many other types of failure are also possible. Consequently, in using this analysis to study actual mining problems, care should be taken that other possible failure mechanisms are also considered and that analyses of the type described by Bétournay are run in parallel with this analysis.

The BASIC program listing given at the end of the paper can be keyed in and used without restriction by any interested reader. The program will run under BASICA or GWBASIC or it can be compiled by one of the newer BASIC compilers. A very efficient executable file has been produced using Microsoft's Quickbasic 4.0 and a copy of this file can be obtained on disk by writing directly to the author.

The aim of this paper is to challenge the reader to think about one particular failure mechanism and to consider the consequences of wide variations in some of the input parameters. The presentation of results of the factor of safety calculation in the form of a distribution curve and a probability of failure is considered to be a realistic solution to this particular problem. The ease with which this calculation can be carried out will enable the reader to explore some of the practical implications of changes in pillar dimensions, in situ stress conditions and rock mass properties. The calculation can also be used to compare the benefits of investing in site investigations to improving the level of confidence in each of the normally distributed input parameters.

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Rosenbleuth, E. (1972), Point estimates for probability moments, *Proc. Nat. Acad. Sci., USA*, Vol. 10, 3812- 3814.

```
10 ' Limit equilibrium analysis of surface crown pillar stability
20 ' Evert Hoek, University of Toronto, July 1989.
30 '
40 ' Dimension variables and screen display
50 '
60 DIM V(20, 60), SF(70): KEY OFF
70 FOR I = 1 TO 3: KEY I, "": NEXT I: KEY 1, "c": KEY 2, "r": KEY 3, "q"
80 SCREEN 2: CLS : COL = 27: FLAG = 0: LINE (5, 5)-(635, 185), , B
90 LOCATE 3, 15: PRINT " STABILITY ANALYSIS OF SURFACE CROWN PILLAR"
100 LOCATE 5, 3: PRINT "Title : ": LOCATE 5, 11: LINE INPUT " ", T$
110 LOCATE 7, 27: PRINT "mean"
120 LOCATE 7, 33: PRINT "std": LOCATE 8, 33: PRINT "dev"
130  LOCATE 9, 3: PRINT "Pillar dimension    x = "
140 LOCATE 10, 3: PRINT "Pillar dimension    y = "
150 LOCATE 11, 3: PRINT "Pillar thickness    z = "
160 LOCATE 12, 3: PRINT "Rock unit weight   gr = "
170 LOCATE 13, 3: PRINT "Water unit weight  gw = "
180 LOCATE 14, 3: PRINT "Water depth        zw = "
190 LOCATE 15, 3: PRINT "Rock mass rating  RMR = "
200 LOCATE 16, 3: PRINT "Intact strength  sigc = "
210 LOCATE 17, 3: PRINT "Intact m value     mi = "
220 LOCATE 18, 3: PRINT "Horiz/vert         kx = "
230 LOCATE 19, 3: PRINT "in situ stress     ky = "
240 LOCATE 22, 14: PRINT "F1 to calculate"
250 LOCATE 22, 33: PRINT "F2 to restart"
260 LOCATE 22, 50: PRINT "F3 to quit"
270 '
280 ' Program for data input and cursor control
290 '
300 NO = 1: SUMSF = 0: SUMDIF = 0: A = 9: GOSUB 540
310 Q$ = INKEY$: IF LEN(Q$) = 0 THEN 310  'scan keyboard
320 IF ASC(Q$) = 13 THEN GOSUB 570: GOTO 310
330 IF LEN(Q$) = 2 THEN Q$ = RIGHT$(Q$, 1)
340 IF Q$ = "H" THEN GOSUB 520: GOTO 620                 'up
350 IF Q$ = "P" THEN GOSUB 520: GOTO 670                 'down
360 IF Q$ = "M" THEN GOSUB 520: GOTO 720                 'right
370 IF Q$ = "K" THEN GOSUB 520: GOTO 740                 'left
380 IF Q$ = "c" OR Q$ = "C" THEN 790                     'calculate
390 IF Q$ = "r" OR Q$ = "R" THEN 80                      'restart
400 IF Q$ = "q" OR Q$ = "Q" THEN CLS : END               'quit
410 IF Q$ = "0" THEN GOSUB 540: GOTO 530                 'enter zero
420 IF Q$ = "-" THEN GOSUB 540: GOTO 530                 'enter minus
430 IF Q$ = "." THEN GOSUB 540: GOTO 530                 'enter period
440 IF VAL(Q$) < 1 OR VAL(Q$) > 9 THEN 310               'enter number
450 GOSUB 540: LOCATE A, COL: PRINT VAL(Q$)
460 LOCATE A, COL + 2: INPUT "", IN$: V(A, COL) = VAL(Q$ + IN$)
470 LOCATE A, COL: PRINT "      "
480 LOCATE A, COL: GOSUB 600: GOSUB 580
490 IF COL = 33 AND A = 20 THEN 500 ELSE 510
500 COL = 27: A = 9: GOSUB 570: GOTO 310
510 GOSUB 540: GOTO 310
520 GOSUB 550: LOCATE A, COL: GOSUB 600: RETURN
530 LOCATE A, COL: PRINT Q$: GOTO 460
540 GOSUB 560: LINE (P, Q - 8)-(P + 25, Q - 2), 1, BF: RETURN
```

```
550 GOSUB 560: LINE (P, Q - 8)-(P + 25, Q - 2), 0, BF: RETURN
560 P = 8 * (COL - 1): Q = 8 * A: RETURN
570 LOCATE A, COL: GOSUB 540: RETURN
580 IF COL = 27 AND A = 19 THEN COL = 33: A = 13
590 A = A + 1: RETURN
600 IF V(A, COL) < .000001 THEN 610 ELSE PRINT V(A, COL): RETURN
610 PRINT " 0": RETURN
620 IF A = 9 THEN A = 9: GOTO 660                    'move cursor up
630 IF COL = 27 THEN GOTO 650
640 IF COL = 33 AND A < 15 THEN A = 15
650 A = A - 1
660 GOSUB 570: GOTO 310
670 IF A = 19 THEN A = 19: GOTO 710                  'move cursor down
680 IF COL = 27 THEN GOTO 700
690 IF A < 14 AND COL = 33 THEN COL = 27
700 A = A + 1
710 GOSUB 570: GOTO 310
720 IF A > 13 AND COL = 27 THEN COL = 33 ELSE COL = 27
730 GOSUB 570: GOTO 310                             'move cursor right
740 IF COL = 33 THEN COL = 27                       'move cursor left
750 GOSUB 570: GOTO 310
760 '
770 ' Factor of safety calculation
780 '
790 LINE (295, 32)-(632, 152), 0, BF: GOSUB 520: COL = 27: GOSUB 520
800 FOR J = 14 TO 19: IF V(J, 33) = 0 THEN V(J, 33) = 1E-09
810 NEXT J: X = V(9, 27): Y = V(10, 27): Z = V(11, 27)
820 GR = V(12, 27): GW = V(13, 27): ZW(1) = V(14, 27) - V(14, 33):
830 ZW(2) = V(14, 27) + V(14, 33): RMR(1) = V(15, 27) - V(15, 33)
840 RMR(2) = V(15, 27) + V(15, 33): SIGC(1) = V(16, 27) - V(16, 33)
850 SIGC(2) = V(16, 27) + V(16, 33): MI(1) = V(17, 27) - V(17, 33)
860 MI(2) = V(17, 27) + V(17, 33): KX(1) = V(18, 27) - V(18, 33)
870 KX(2) = V(18, 27) + V(18, 33): KY(1) = V(19, 27) - V(19, 33)
880 KY(2) = V(19, 27) + V(19, 33)
890 FOR A = 1 TO 2: FOR B = 1 TO 2: FOR C = 1 TO 2
900 FOR D = 1 TO 2: FOR E = 1 TO 2: FOR F = 1 TO 2
910 GOSUB 1040: SF(NO) = FOS: NO = NO + 1
920 NEXT F: NEXT E: NEXT D: NEXT C: NEXT B: NEXT A
930 FOR NO = 1 TO 64: SUMSF = SUMSF + SF(NO): NEXT NO
940 MEANSF = SUMSF / 64
950 FOR NO = 1 TO 64: SUMDIF = SUMDIF + (SF(NO) - MEANSF) ^ 2: NEXT NO
960 IF FLAG = 1 THEN FLAG = 0: GOTO 1580
970 SDFS = SQR(SUMDIF / 64)
980 IF SDFS > 0.0001 THEN GOSUB 1220: GOSUB 1320: GOTO 300
990 LOCATE 13, 45: PRINT "Safety Factor = ": LOCATE 13, 60
1000 PRINT USING "###.##"; MEANSF: GOTO 300
1010 '
1020 ' Subroutine for factor of safety calculation
1030 '
1040 M = MI(D) * EXP((RMR(B) - 100) / 14)
1050 S = EXP((RMR(B) - 100) / 6)
1060 SIGX = GR * KX(E) * Z / 2 - (GW * ZW(A) ^ 2) / (2 * Z)
1070 SIGY = GR * KY(F) * Z / 2 - (GW * ZW(A) ^ 2) / (2 * Z)
1080 HXZ = 1 + 16 * (M * SIGY + S * SIGC(C)) / (3 * M ^ 2 * SIGC(C))
```

```
1090 IF HXZ < 1 THEN 1120
1100 THETAXZ = .5235988# + 1 / 3 * ATN(1 / SQR(HXZ ^ 3 - 1))
1110 PHIXZ = ATN(1 / SQR(4 * HXZ * COS(THETAXZ) * COS(THETAXZ) - 1))
1120 HYZ = 1 + 16 * (M * SIGX + S * SIGC(C)) / (3 * M ^ 2 * SIGC(C))
1130 IF HYZ < 1 THEN FLAG = 1: RETURN
1140 THETAYZ = .5235988# + 1 / 3 * ATN(1 / SQR(HYZ ^ 3 - 1))
1150 PHIYZ = ATN(1 / SQR(4 * HYZ * COS(THETAYZ) * COS(THETAYZ) - 1))
1160 TAUXZ = (1 / TAN(PHIXZ) - COS(PHIXZ)) * M * SIGC(C) / 8
1170 TAUYZ = (1 / TAN(PHIYZ) - COS(PHIYZ)) * M * SIGC(C) / 8
1180 FOS = 2 * (TAUXZ / Y + TAUYZ / X) / GR: RETURN
1190 '
1200 ' Subroutine for printout of results
1210 '
1220 LOCATE 15, 53: PRINT "Safety Factor ": LOCATE 17, 46
1230 PRINT "mean safety factor  = ": LOCATE 17, 67:
1240 PRINT USING "###.##"; MEANSF: LOCATE 18, 46:
1250 PRINT "standard deviation  = ": LOCATE 18, 67
1260 PRINT USING "###.##"; SDFS: LOCATE 19, 46:
1270 PRINT "failure probability = ": GOSUB 1540
1280 LOCATE 19, 69: PRINT USING "###_%"; PROB * 100: RETURN
1290 '
1300 ' Subroutine to plot normal distribution
1310 '
1320 LINE (330, 32)-(600, 100), , B:
1330 XL = MEANSF - 3 * SDFS: XR = MEANSF + 3 * SDFS: XD = XR - XL
1340 XA = (600 - 330) / XD: YMAX = 0
1350 FOR XG = 330 TO 600: X = XL + (XG - 330) / 270 * XD
1360 GOSUB 1520: IF Y > YMAX THEN YMAX = Y
1370 NEXT XG: YB = 0: YT = 1.1 * YMAX: YD = YT - YB: YA = (100 - 32) / YD
1380 FOR XG = 330 TO 600: X = XL + (XG - 330) / 270 * XD
1390 GOSUB 1520: YG = 100 - (Y - YB) * YA
1400 IF X < 1 THEN 1410 ELSE 1420
1410 LINE (XG, YG)-(XG, 100): GOTO 1430
1420 LINE (XG, YG)-(XG + 1, YG + 1)
1430 NEXT XG
1440 FOR X = 330 TO 600 STEP 54
1450 LINE (X, 102)-(X, 100): NEXT X
1460 C = 38: FOR K = 0 TO 5: LOCATE 14, C
1470 S = XL + XD * K / 5: PRINT USING "###.##"; S;
1480 C = C + 7: NEXT K: COL = 27:
1490 Q$ = INKEY$: IF LEN(Q$) = 0 THEN 1490          'scan keyboard
1500 IF Q$ = "r" OR Q$ = "R" THEN 80                'restart
1510 RETURN
1520 Y = (1 / (2.506628 * SDFS)) * EXP(-.5 * ((X - MEANSF) / SDFS) ^ 2)
1530 RETURN
1540 PZ = (1 - MEANSF) / SDFS
1550 R = EXP(-PZ * PZ / 2) / 2.506628: T = 1 / (1 + .33267 * ABS(PZ))
1560 T = 1 - R * (.4361836 * T - .1201676 * T ^ 2 + .937298 * T ^ 3)
1570 IF PZ >= 0 THEN PROB = CSNG(T): RETURN ELSE PROB = 1 - T: RETURN
1580 LOCATE 7, 55: PRINT "WARNING": LOCATE 9, 48
1590 PRINT "Input data unacceptable,": LOCATE 10, 48
1600 PRINT "try reducing magnitudes ": LOCATE 11, 48
1610 PRINT "of standard deviations. ":  COL = 27: GOTO 300
```

STATE OF THE ART ADDRESS / ÉTAT DE LA QUESTION

*M.C. Bétournay*

# WHAT DO WE REALLY KNOW ABOUT SURFACE CROWN PILLARS ?

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## ABSTRACT

Surface crown pillars are complex mining structures important for safe and economic extraction activity near surface. The existing body of information reveals that since 1984 case studies rather than advance in knowledge have occurred with the intensification of activity on the subject. Few exhaustive and detailed references exist on surface crown pillars; none exist for abandoned mines. Failure mechanisms, stress conditions, in situ surveys, consideration of 3-D rock volumes involved in the stability of the pillar, and numerical modelling are recognized as major components of pillar design. Improvements in design could be possible by obtaining exact 3-D conditions and applying failure specific modelling (as opposed to generic, generalized modelling) for the large cross section of rock mass conditions found in surface crown pillars. Factor of safety use versus a probabilistic approach is discussed as is the interrelationship between design and response.

## RÉSUMÉ

Les piliers de surface sont des structures minières complexes, importantes à l'activité souterraine sécuritaire et économique se déroulant près de la surface. L'ensemble des connaissances démontre que depuis 1984 ce sont des études de cas plutôt que des avancements dans le sujet qui prédominent suite à l'intensification d'activité dans ce domaine. Peu de références exhaustives et détaillées sont disponibles; aucunes n'existent pour les mines abandonnées. Les mécanismes de rupture, les pressions de terrains, les sondages en place, la considération en 3-D des volumes de roc incorporés dans la stabilité de piliers et la modélisation numérique sont reconnus comme les éléments majeurs de la conception de piliers. Des améliorations dans nos conceptions sont possibles en obtenant des conditions exactes en 3-D et en appliquant une modélisation reliée à la rupture particulière (contrairement à la modélisation générique et généralisée) pour les nombreux types de conditions de massifs rocheux appartenants aux piliers de surface. L'utilisation du facteur de sécurité versus l'approche probabiliste est discuté de même que la relation entre la conception et le comportement.

INTRODUCTION

Since 1984, there has been an intensification of activity in the field of surface crown pillars. It has become evident that this is a complex subject which requires a broad knowledge of all geotechnical disciplines (structural geology, rock mechanics, soil mechanics, hydrology). These mining structures, situated above near-surface underground excavations figure 1, can range in material quality: from massive and competent for some to altered and weak for others.

Geotechnical and mining factors vary from minesite to minesite so that each pillar needs to be treated as a unique case. The challenge is to continuously evaluate their stability while optimizing their dimensions to maximize safety and extraction. In some cases, operators leave final dimensions for the life of the mine, in others the pillars are under continuing extraction; some pillars are recovered outright.

New design guidelines and dedicated applications of generic design methods are now in use for these structures.

This presentation, although it introduces the sum of knowledge from known sources of information, also examines more fundamental questions in relation to important stability elements and the interrelationship between design and material response. Some consideration is given to gaps in information and whether we can significantly improve our knowledge of complex situations and our ability to better our designs.

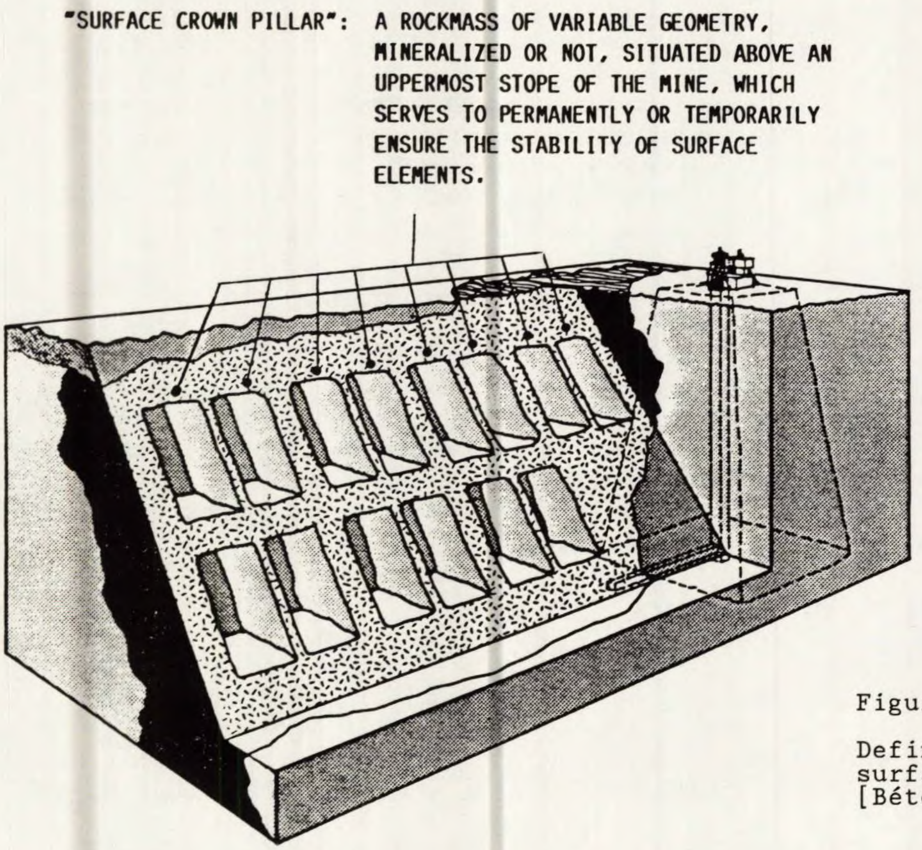



Figure 1.

Definition of a surface crown pillar [Bétournay 1986a]

## REVIEW OF PUBLISHED INFORMATION

Table 1 presents a breakdown in the content of surface crown pillar publications published since 1984.

Prior to 1984, no specific process, tried method, case study, terminology or other helpful information related to these structures was published for hard rock settings. In effect, the thrust of early efforts [CANMET Contract 1984; CANMET Contract 1985; Bétournay 1986a] was to gather all relevant information. These works served as foundation upon which future research and advancement of knowledge about the nature of these structures could be based.

The case studies examined indicated a wide range of rock mass competence and geological assemblage. However, there was no tendency to increase pillar thickness when going from competent to poorly competent ground [Bétournay et al 1988], the thickness to width ratio usually taking on a value less than 5.

The first, and so far the only exhaustive discussion of surface crown pillars, was formulated in 1986 [Bétournay 1986a]. Its objective was to amalgamate applicable information from all geotechnical fields into a body of scientific information capable of supplying the reader with the "big picture" as well as the reference for design including advantages/disadvantages and limitations of formulas, methods and mining strategies for the benefit of attaining the goal of stable pillars. A design process was established, figure 2, to enumerate and place in perspective the progression of design. An updated and enlarged version is planned as a handbook to serve as a reference to mine operators and mine regulators. An equivalent guidebook for the problems associated with abandoned mines does not exist.

The exercise was repeated subsequently [Centre de Recherche Minérale 1986] with a narrower descriptive scope.

### Major Components of Pillar Design

The idea of failure mechanisms as the focal point for the application of design and ground support methods was used sparsely at first [Steffen Robertson and Kirsten 1984; Bétournay 1986a; CANMET Contract 1986b; Biron and Labrie 1986] but has since been routinely taken into consideration. The failure mechanism concept is important. It places the onus on the field investigation, to be as complete and wide ranging as possible. It also challenges the designer to apply methods of dimensionning that can account for the failure mechanism(s). The proper application of one or several support types as well as a dedicated monitoring program could be achieved.

One study in particular is currently examining the failures of surface crown pillars of both active and abandoned mines [CANMET Contract 1988d]. Recognizing that the approach taken by most mines until recently relied heavily on "experience" and "rules of thumb" [Bétournay et al 1988] this study has undertaken to examine the factors that have controlled previous failures and those contributing to the stability of existing pillars. Basic geotechnical and mining data on a great number of minesites is

Table 1. Contents of surface crown pillar publications since 1984.

| Reference | Exhaustive Listing of Information | Exhaustive Discussion of Information | Design Guidelines | Other Basic Information | Case Studies (#) | In Situ Investigations | Material Testing | Failure Mechanism | Application of Design/Stability Methods | Mining Methods | Ground Support | New Investigation Methods | New Design/ Stability Methods | New Mining Method | Pillar Recovery | Economic Analysis |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CANMET Contract 1984 | X | | | | 6 | | | | | | | | | | | |
| Stefan Robertson, Kirsten 1984 | | | | | 18 | | | | | | | | | | | |
| CANMET Contract 1985 | | | | | 1 | x | x | x | x | x | x | | | | x | |
| Bétournay 1986a | x | x | x | | | | | x | | | | | | | | |
| Kelly et al. 1986 | | | | | 1 | | | | | x | | | | x | x | |
| CANMET Contract 1986a | | | | | 1 | x | x | | x | | x | x | | | | |
| Udd and Bétournay 1986 | | | | x | | | | | | | | | | | | |
| Biron and Labrie 1986 | | | | | 1 | x | x | | | | | | | | | |
| Closset 1986 | | | | | 1 | x | | | x | x | | | | | | |
| Bétournay and Thivierge 1986 | | | | | 1 | | x | x | x | | | | | | | |
| Bourbonnais, 1986 | | | | | | x | | | | | | | | | | |
| Lessard et Bienvenu, 1986 | | | | | | | | | | | | | | | | x |
| Bétournay 1986b | | | x | | | | | | | | | | | | | |
| Fortin and Gill | | | | | | | | | | | | | x | | | |
| Centre de Recherche Minérale 1986 | | | x | | | | | | | | | | | | | |
| CANMET Contract 1986b | | | | | 1 | | | | | | | | | | | |
| Bétournay 1987 | | | | | 5 | x | | x | x | | | | | | x | |
| CANMET Contract 1987a | | | | | 1 | | x | | x | x | | | | | x | |
| Bétournay et al 1987 | | | | | 1 | | | x | x | | x | | | | | |
| CANMET Contract 1987b | | | | | 2 | x | | | | | | x | | | | |
| CANMET Contract 1988a | | | | | 1 | | | | | | | x | | | | |
| Bétournay et Labrie 1988 | | | | | 1 | x | x | x | x | x | x | | | | | |
| CANMET Contract 1988b | | | | | 1 | | | x | x | | x | | x | x | x | |
| Bétournay 1988a | | x | x | | | | | x | | | | | | | | |
| Bétournay 1988b | | | | | 1 | x | | x | x | | x | | | | | |
| Bétournay et al 1988 | | x | | | 24 | | | | | | | | | | | |
| Yu et al 1988 | | | | | 1 | x | | x | x | | x | | | | | |
| Bétournay 1988c | | | | | 1 | | | x | x | | x | | | | | |
| CANMET Contract 1988c | | | | | 4 | x | x | x | x | | | | | | | |
| CANMET Contract 1988d | | | | | 46 | x | x | x | x | | | | | | | |
| Bétournay 1988d | | | | | 6 | x | | x | | | | x | | | | |
| Closset 1988 | | | | | | | | | x | | | | | | | |
| Bétournay 1988e | | | | x | | | | | | | | | | | | |

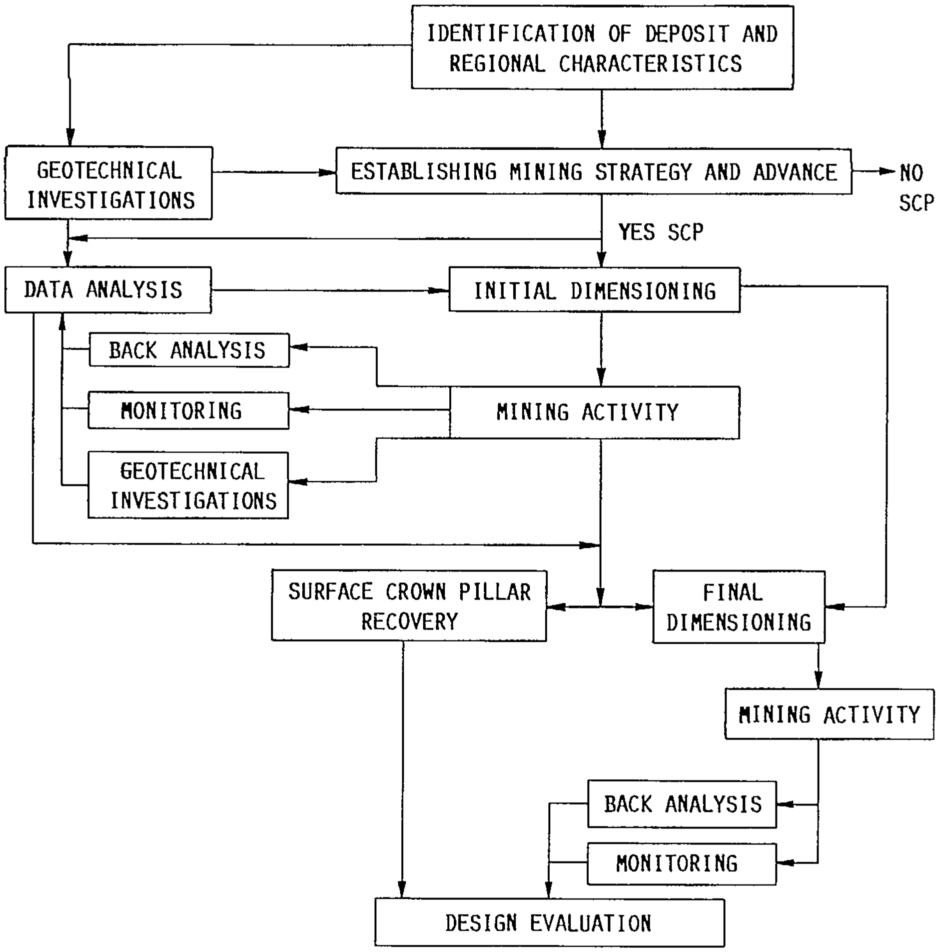



Figure 2. Surface crown pillar design process [Bétournay 1986a].

pieced together for individual mine situations. Back-analysis using a variety of methods, analytic solutions and numerical approaches will look at failure mechanisms, the effects of parameter variation, and level of stability/instability. The effectiveness of the approach for solving particular conditions is also outlined.

There has been to date no clear examination of in situ stress conditions near surface. Published results of stress measurements in Canada [Herget 1984] relate to depths of > 80 m. It is important to know what part stress effects play in helping surface crown pillar stability in order to carry out efficient designs and validate many of the postulated failure mechanisms. Speculation is that stresses near surface (0-20m) could vary greatly from established directional trends, figure 3. An extensive campaign [Bétournay 1988d] is presently underway to collect stress data near surface in undisturbed bedrock of various Canadian mining camps.

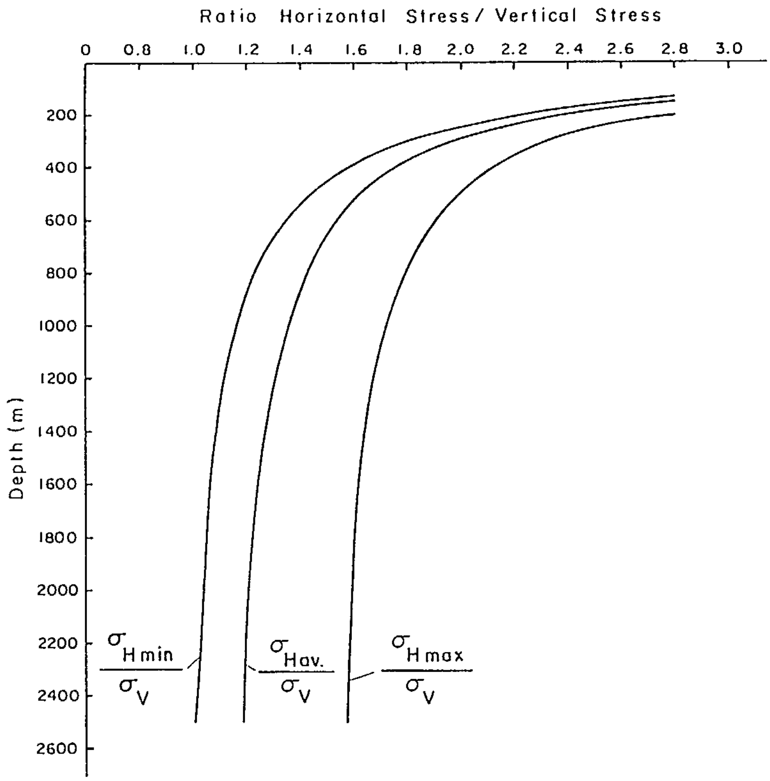
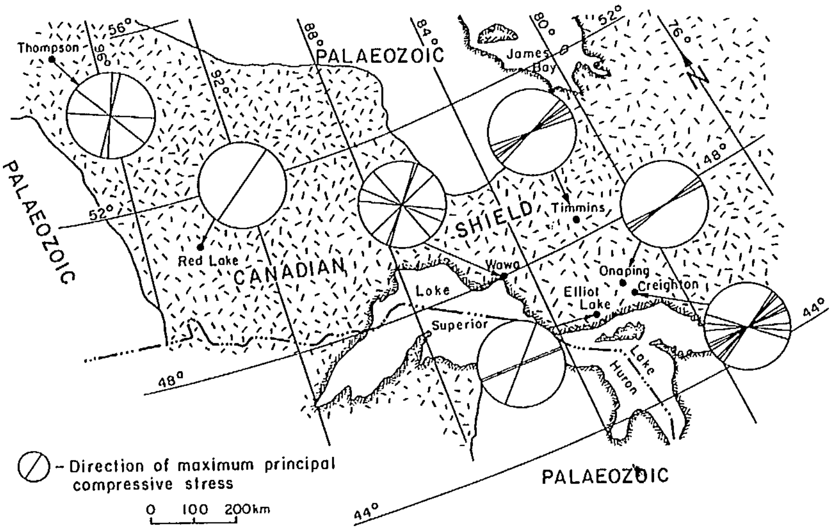



Figure 3.

Established levels and directional trends for ground stresses in the Canadian Shield [Herget 1984].




The requirement of stress data for numerical modelling of near surface openings [Steffen Robertson and Kirsten 1984; Biron and Labrie 1986; CANMET Contract 1986b; Bétournay et al 1987; Bétournay 1988c] has been filled so far by extrapolating from deeper measurement values and assuming zero stress at surface.

Other actual rock mass site investigations have by and large been limited to quality assessments and rock mass classifications [Steffen Robertson and Kirsten 1984; CANMET Contract 1986a; Bétournay et al 1987; Bétournay et Labrie 1988]. Classifications can only supply general outlines of site conditions and as such must not be used alone but in conjunction with other methods to formulate an overall design rationale. Thorough analysis of joint orientation, intersections, density and potential effects on stability is not practiced.

The dilatometer has become important in the estimate of general and localized brokenness of the rock mass. The data is used to specify the extent of rock mass competence indicated by the RQD of the drill core. For example, the upper bedrock of the Canadian Shield in contact with overburden has been proven to be a regolith of little competence [CANMET Contract 1988c; Monterval 1988]. Consideration of RQD data alone could not have confirmed this. Dilatometer tests can also be used to supply representative values of rock mass modulus of elasticity for numerical modelling.

Other in situ rock surveys such as hydraulic conductivity tests are not available. They could also indicate the extent of the rock mass integrity and qualify water effects on the stability of near surface openings.

Less is known about soil conditions. Although thicknesses and classification are commonly sought, actual field values for mechanical parameters remain unknown for the most part. Surveys of minesites now include in situ tests to measure density and shear strength of soils; samples are examined for water content and liquifaction potential.

So far available analytical formulas (elastic members, voussoir solutions, arch theory, etc.) and conventional numeric models (Finite Elements, Boundary Elements, etc.) [Bétournay 1986a] have been applied [Bétournay et al 1987; Bétournay 1988c; Bétournay et Labrie 1988; CANMET Contract 1987b; CANMET Contract 1988c; Steffen Robertson and Kirsten 1984; Yu et al 1988].

Numerical modelling has been recognized as the most powerful tool to solve the complex stability problems of these pillars, but it has been difficult adapting current models to field behaviour. Pillar behaviour rarely fits generic modelling assumptions: generalized failure, stress induced failure, homogeneous material types, etc. Localized, tensile/shear,non-linear and gravitational failures predominate. The closest fit has been in the application of block or distinct element codes for fissured but sound rock material. Dedicated models for surface crown pillars are beginning to be formulated [CANMET Contract 1986b; Fortin et Gill 1986].

## Research

By far, the bulk of the information pertains to single case studies rather than research in aspects related to surface crown pillars (Table 1). In most of these cases limited field work, analysis, and design was performed. It would appear also that some mines limit themselves to laboratory strength tests. In the case of surface crown pillars there is no substitute for testing in situ strength and behaviour testing for rock and soil. It represents the best hope for achieving representative stability assessments.

The few innovations that have taken place have been wide ranging: new investigation techniques to situate rock mass features in 3-D [CANMET Contract 1987a], new design methods to calculate pillar stability [Fortin et Gill 1986; CANMET Contract 1986b] and to recover pillars without removal of overburden [CANMET Contract 1988b], new approaches to recovery pillars after overburden removal [Steffen Robertson and Kirsten 1984; Kelly et al. 1986; CANMET Contract 1987b], economic considerations [Lessard et Bienvenue 1986].

There have been few innovations in ground support and monitoring instruments dedicated to surface crown pillar problems: single anchor extensometers for altered rock [CANMET Contract 1988b] and movement of weak hangingwall rocks using multiple inflatable anchor extensometers [Bétournay 1988d].

## CRITICAL ELEMENTS

From the number of elements that can affect the stability of surface crown pillars the designer must identify which are the most critical, and how they combine to affect the integrity of the pillar. A short discussion of general and particular elements follows.

It is necessary to describe with confidence the existing water, soil and rock properties surrounding planned/existing openings. This is important in identifying stable and unstable situations. New conditions exist every time an opening is created/expanded. Specific examination is thus warranted before excavation is begun. The usual question "How much of a pillar do we leave behind?" should be rephrased "What is the rock volume that is mobilized by the effects of the opening?". A narrow view of the situation would be to consider only the thickness of rock between the soil and top of the opening: The rock mass which can influence pillar stability stretches laterally beyond the crown of the opening and even into the abutments of the openings, figure 4. The regolith in contact with the soil must be discounted from contributing resistance to failure. Furthermore, if failure occurs, will the new opening become self supporting? Can the soil remain stable above rock failure and not enter the opening? Water saturated soils have been known to flow [Gouvernement du Québec 1981]; dense till-like soils have been known not to [CANMET Contract 1988d; Bétournay 1989].

The importance of understanding and cataloguing failure mechanisms was underlined earlier. In itself it is one of the most important critical elements that must not be overlooked in design. Yet, within this consideration there reside several other caveats.

The broad range of rock mass settings shows us that there are different failure modes and ranks of failure possibilities, figure 4, while stress may or may not help stabilize. In massive rock [Bétournay et al 1987] localised degradation and readjustment to tensile stresses are expected. In sound rock environments, the location and connectivity of discontinuities is the most critical element. If the rock mass is not effectively fissured it can be several times more resistant than a rock mass where blocks are commonplace. Research now underway [CANMET Contract 1987a] will provide the means to map major discontinuities (>1m) and anomalous zones in 3-D. This important leap forward in rock mechanics is aimed at eliminating guess work about conditions around openings, greatly helping in understanding possible

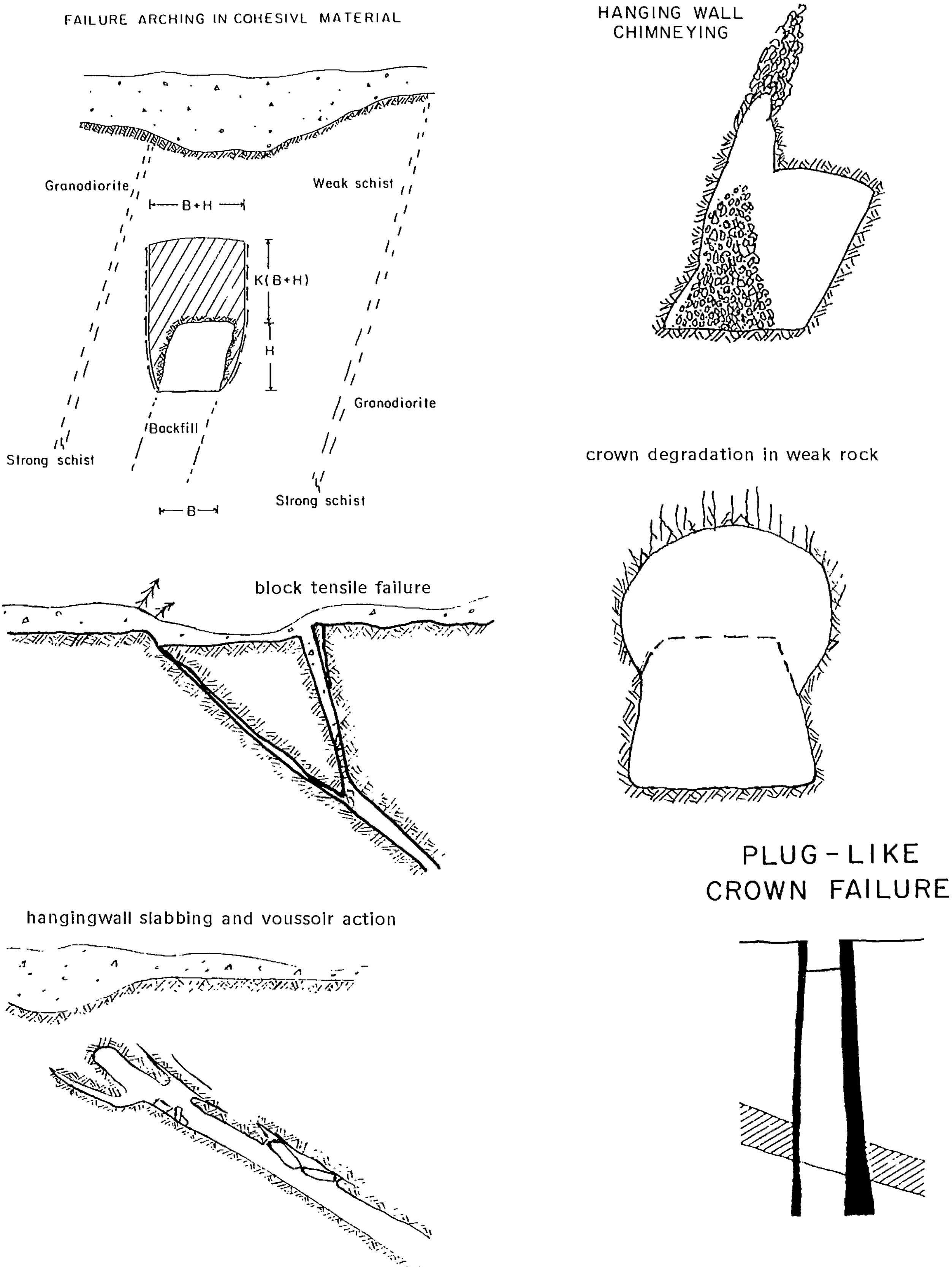



Figure 4 Examples of surface crown pillar failures and mobilized rock mass.

behaviours and making numerical models arrive at much more representative results. In altered rock, localised shear and tensile failures whether as chimneying, crown degradation or large scale movements are expected.

In all cases it is critical to know what values of stress exist and if the distribution around particular openings are sufficient to restrain gravity type failures, from sliding blocks to material degradation. Excellent indications of stress levels at failures have been obtained by back-analysis, [CANMET Contract 1988d] and confirmation of local Canadian shallow stresses [Bétournay 1988c] will provide working numbers. Further observations from back analysis studies confirm the expectations that the generic design methods widely thought to provide satisfactory estimates of stability, have narrow scopes of applicability and broad range of precision.

Proper care in sampling all materials, even altered rock, and in describing mechanical behaviour of all material types [CANMET Contract 1986a] is part of the overall goal of obtaining all the data and placing it in perspective with respect to openings, figure 5, to evaluate surrounding conditions. By using 2-D and 3-D projections, all of the geomechanical information can serve to separate the rock mass in zones of varying behaviour, figure 6.

## IMPROVING OUR DESIGNS

The current process by which we arrive at our estimation of stability is one which has gaps in information, uses generic analysis methods and usually measures results by a very narrow indicator: The single factor of safety.

Given such circumstances, how much closer to failure than anticipated is the material response? Historically, few failures have occurred while an active mine was creating surface crown pillars. One even discovers that past mining practices that have come perilously close to overburden contact have left openings which remain stable for long periods of time.

Can we then infer that we are currently erring on the conservative side and that the margin of allowable error is large? If this is true, designs incorporating exhaustive consideration of information and application of enlightened design methods should substantially reduce the risk of possible problems and perhaps permit higher extraction at satisfactory levels of safety.

Placing in perspective our grasp of these structures and our use of design methods, it is apparent that we are currently working with 3 generations of information, figure 7.

The first, the rudimentary experience, is limited to personal experience with little scientific information involved; limited effectiveness is achievable. The second consists of applying conventional tools (finite element modelling, general analytical methods, etc) which use detailed scientific information obtained on a bulk or generalised basis. Improved effectiveness is achievable and the use of several methods provides a range of possibilities. The third generation consists of using design methods dedicated to recognized conditions, obtained from sound knowledge of the location of elements surrounding the opening. An example of this is the

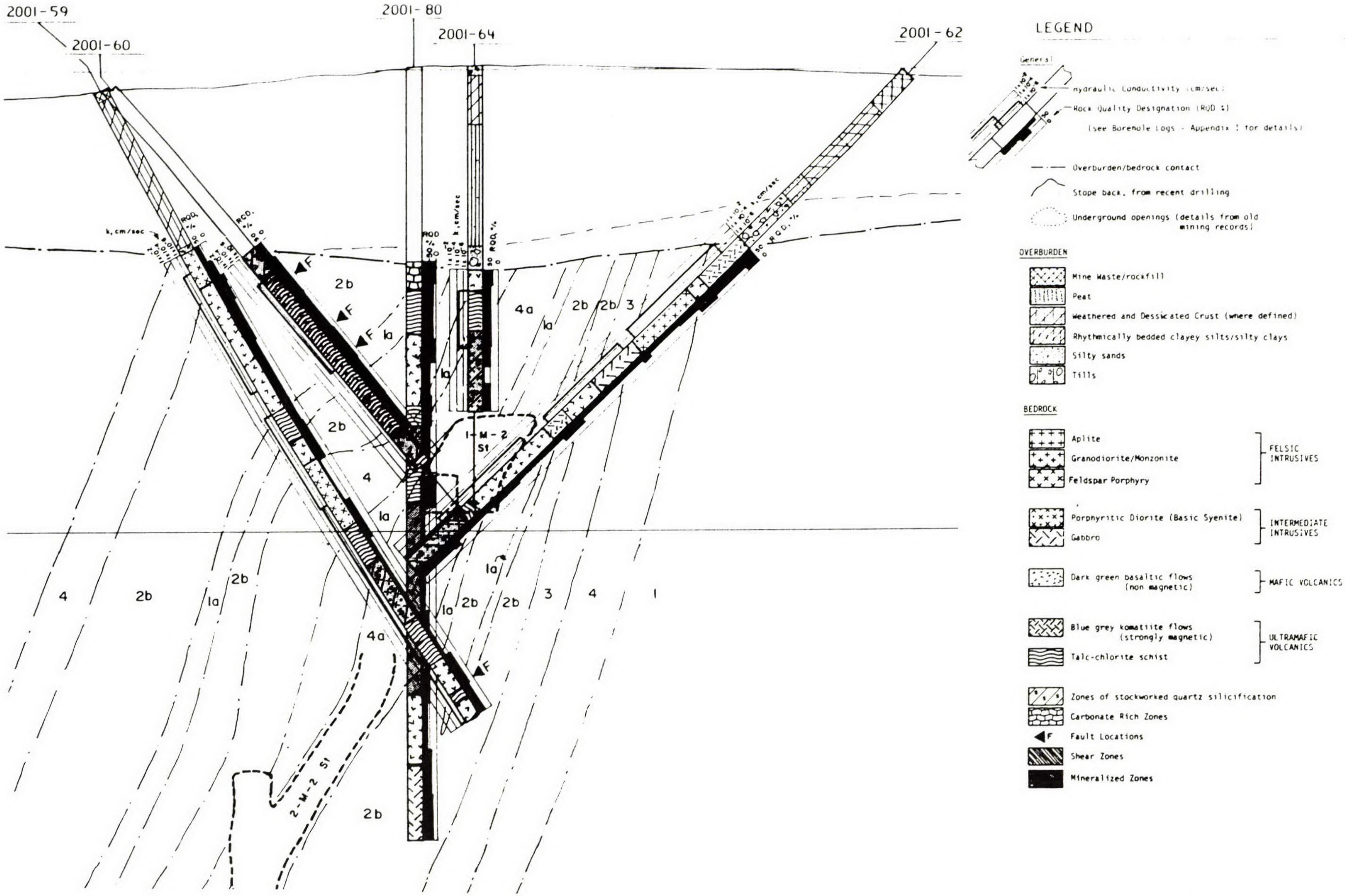
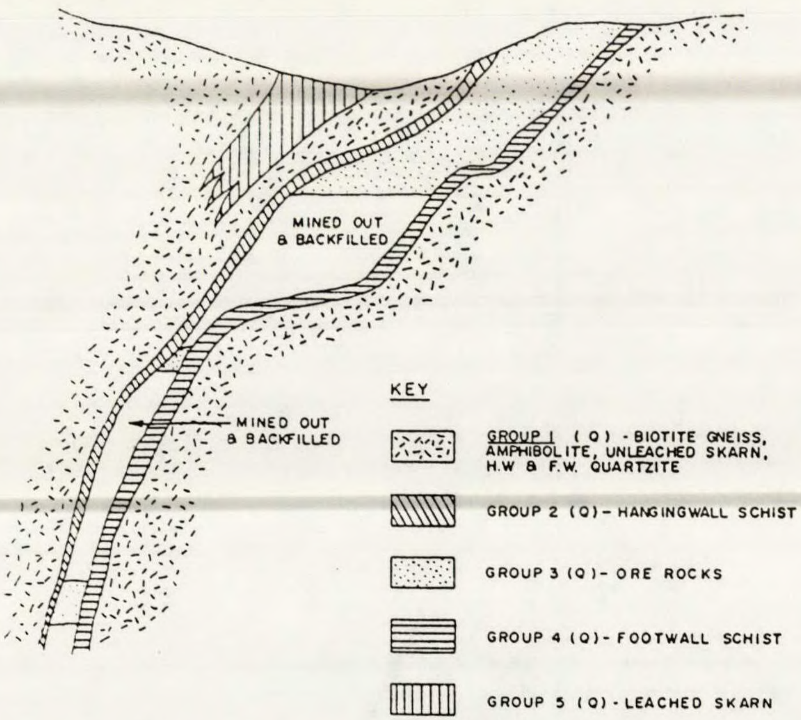



Figure 5. Borehole data placed in perspective to stopes (transverse section) [CANMET Contract 1988d]



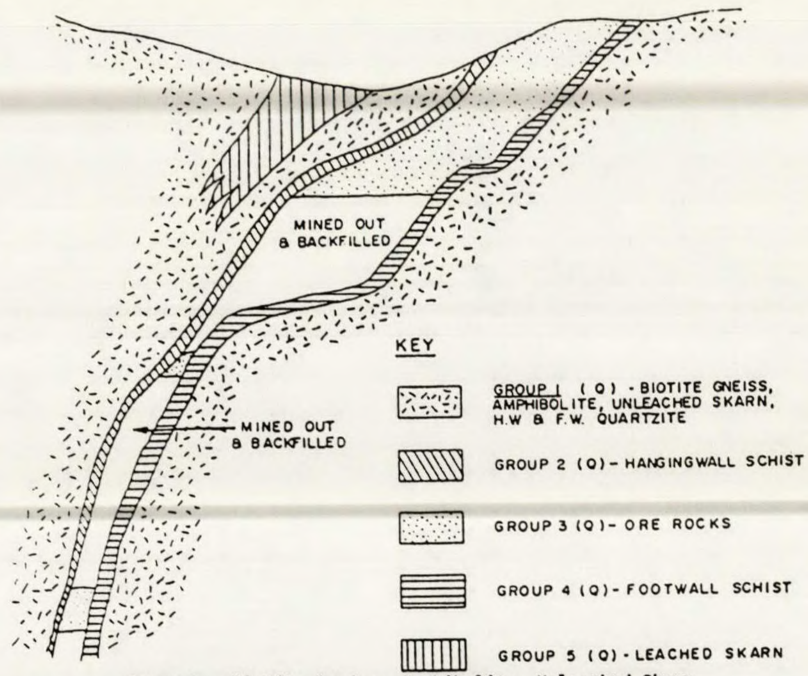

Class I (6 < Q < 11) - biotite gneiss, amphibolite, skarn (unleached, hangingwall quartzite and footwall quartzite).

Class II (3 < Q < 6) - Hangingwall schist (anisotropic properties).

Class III (3 < Q < 6) - Ore Rocks.

Class IV (Q = 1) - Footwall schist (anisotropic properties) Range Q = 0.1 to 3.

Class V (Q = 0.03) - Skarn (leached) Range Q = 0.001 to 0.1.




Group 1 - Biotite Gneiss, Amphibolite, Unleached Skarn, Hangingwall Quartzite and Footwall Quartzite with mean rock mass uniaxial compressive strength of 10 MPa (1450 psi); and deformation modulus of 25 GPa (3.6 x 10E6 psi).

Group 2 - Hangingwall Schist with mean rock mass uniaxial compressive strength of 3.5 MPa (500 psi) and deformation modulus of 20 GPA (2.9 x 10E6 psi).

Group 3 - Ore Rocks with mean rock mass uniaxial compressive strength of 3 MPa (435 psi) and deformation modulus of 10 GPa (1.4 x 10E6 psi).

Group 4 - Footwall Schists with mean rock mass uniaxial compressive strength of 1.5 MPa (220 psi) and deformation modulus of 5 GPa (0.7 x 10E6 psi).

Group 5 - Leached Skarn with mean rock mass uniaxial compressive strength of 0.5 MPa (72 psi) and deformation modulus of 1 GPa (0.15 x 10E6 psi).

Figure 6 Separation of a near-surface rock mass into zones of varying behaviour [Steffen Robertson and Kirsten 1984]

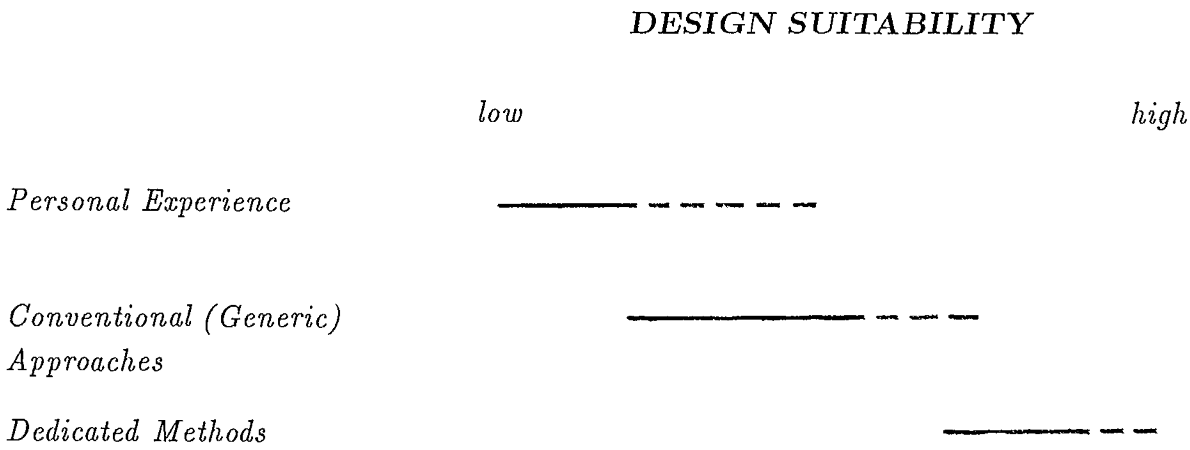



Figure 7. Three generations of surface crown pillar design methods and their range of suitability.

application of block codes to sound fissured rock. But for the most part such methods are absent, which therefore leaves us without ideal design tools for most of the existing types of pillar conditions.

One must keep in mind however that personal experience, when well founded, can contribute to the effectiveness of the second or third generation methods.

The effectiveness of the design is also affected by our basis for judging stability. Given the complexity of the problem, a factor of safety tends to simplify the situation. Even by using advanced tools such as modelling, the results are based on one failure criterion. The true nature of the rock volume involved in the stability/instabilities and progression of behaviour is not seen. One exception to this would be distinct modelling for blocky conditions. Furthermore, a factor of safety is based on well known conditions with predictable material behaviour. There is no limit to the "safe" value it can take on, i.e. >1.0. A factor of safety does not afford security in proportion to its value. There is always the possibility of failure or other problems, regardless of the value of the factor.

The probabilistic approach is better defined: a range of 0 to 1 is used, but it also requires all the necessary geomechanical information as input. However, it is the possibility to qualify each input (variability and certainty) and the combination of information to a qualified level of stability that makes this approach superior.

It goes without saying that in arriving at stability assessments several design methods should be used in the context of the existing problem.

By and large, the design of surface crown pillars has so far been done for short term stability (<20 years). Changing rock mass conditions can occur over time and are more obvious for weak rock material or rock masses tending to deform on structural features. Failures reaching the overburden in altered rock masses or in weak hanging-walls occur in days. In cases of thin pillars composed of sound rock, the opening can remain stable for decades even under adverse water conditions and imposed surface loads.

To be able to quantify the behaviour of underground openings with time would be of immense benefit not only to mine operators but also for regulatory agencies with respect to land use purposes.

Whereas conventional rib and post pillars have been designed on a yielding basis under a short term high extraction basis, similar approaches for surface crown pillars would present a very difficult challenge. For one, instantaneous failures rather than progressive failures could take place.

For another, the large number of elements controlling pillar behaviour may be too much for such a low factor of safety design. It is perhaps wiser to recover all of the pillar with bulk mining methods rather than risk worker safety and mine viability.

On the aspects of long term stand up time of surface crown pillars, it would be interesting to incorporate time steps in each of the advanced design methods for each type of ground condition. In this case, as in other cases where integrity of the underground operation is primordial, dedicated instrumentation have to be put in long term service.

## CONCLUSIONS

Surface crown pillars remain a complex subject. To arrive at knowledgeable stability assessments few dedicated tools or methods are available. Comparatively little research has been performed to provide these.

No design of surface crown pillars should be performed without a high degree of confidence and completeness about existing conditions being available.

Obtaining critical parameters in the three dimensions of the rock volume above and surrounding the upper portion of the underground opening is crucial to grasp the situation.

The adequacy of current rock mass classification schemes must be measured. The formulation of a scheme dedicated to near surface environments might be worth considering.

Consequences of failure have to be fully understood in regards to soil inflow. In weak or uncertain conditions recovery of pillars should then be made wherever possible with methods not involving upward underground progression. Backfilling and extraction from surface, whether with filled stopes or blasting rock down in the opening, are safer alternatives.

More dedicated, less generic design methods are needed to address possible failure mechanisms. Specialized numerical modelling methods must be developed to study shallow opening behaviour in weak, lowly stressed rock conditions, e.g. viable design methods for failures such as chimneying and plug types do not exist. Since back-analysis of failures shows stresses to be low in the surface crown pillars, modelling methods must use a workable failure criterion at such levels. Modelling must also consider all of the rock mass involved in the stability of the pillar.

As is the case for other rock mechanics/geotechnical problems, even with advanced approaches the subject of surface crown pillar is not expected to become an exact science.

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## INVESTIGATION OF IN SITU CONDITIONS

## SESSION I

## L'INVESTIGATION DE CONDITIONS EN PLACE

# GEOTECHNICAL ASSESSMENT OF THE NO. 5 SHAFT CROWN PILLAR: FALCONBRIDGE MINE

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## ABSTRACT

The No. 5 shaft crown pillar is a narrow veined steeply dipping, discontinuously jointed ore remnant of the Falconbridge Mine. It is bounded by a blockly well jointed norite to the north and a very competent greenstone to the south, all overlain by partially saturated clastic sediments. To-date, geotechnical investigations have involved ground penetrating radar, seismic refraction, drilling and sampling of soil and bedrock material, geomechanical analysis of overburden soils and bedrock material. This was followed by installation of dewatering production wells and associated hydrogeological studies, and examination of ground conditions during the early development of the overcut and undercut areas.

## RÉSUMÉ

Le pilier de surface du puits n$^{o}$ 5 de la mine Falconbridge est un résidu de roc minéralisé faisant partie d'une veine étroite à pendage élevé qui contient des continuités aléatoires. Il est entouré au nord d'une norite bien fissurrée en blocs et au sud de roc métavolcanique verdâtre très compétent. Le tout est recouvert par des sédiments clastiques partiellement saturés. Jusqu'à date, la campagne d'investigations géotechniques a inclu les sondages par radar, par séismique réfractaire, par forages et échantillonnage du mort-terrain et du socle, et par l'analyse des matériaux meubles et socle rocheux. Ceci fut suivi par l'installation de puits de dénoyage et études hydrogéologiques connexes et par l'examen des conditions associées à l'excavation d'ouvertures souterraines et en surface.

## INTRODUCTION

Falconbridge Limited is pursuing plans to extract ore from the #5 shaft crown pillar, a remnant of the Falconbridge orebody which extends over a total strike length of 1070 meters. Portions of the crown pillar are overlain by approximately 30 to 45 meters of alluvial till, up to 30 meters of which is saturated or water bearing (Figure 1).

In June 1988, a geotechnical assessment program was initiated to examine and evaluate soil and bedrock conditions while preliminary crown pillar ore development occurred simultaneously. Further analyses incorporating the data collected on past geotechnical work, recent geophysical programs, soil sampling and bedrock profiling, structural mapping, and geomechanical and hydrogeological studies were completed.

## GEOLOGY AND STRUCTURAL CHARACTERISTICS

The #5 shaft crown pillar is located within the Precambrian Shield, with Quaternary glacially derived sediments blanketing Precambrian basement rocks to various depths. The bedrock is undulating, dipping to the southeast with linear depressions aligned subparallel to the orebody trend created through glacial abrasion. Overlying the bedrock, 30 to 45 meters of clastic sediments are subdivided into 3 distinct horizons. Sandy gravel occupies the upper 3 - 9 meters, while a fine dense sand up to 30 meters makes up the central unit, and the bottom 3 - 9 meters contains gravelly sand with cobbles and coarse gravel.
The bedrock profile consists of a narrow east-west steeply dipping nickel sulphide vein, bounded by well jointed norites to the north, and more competent greenstones to the south. The hangingwall contact is schistose while a thin mine-wide fault is usually present at the footwall contact. The norite consists of 4 major joint sets, the greenstone 2 common sets, with the ore containing up to 5 discontinuous sets. At great depths and with high extraction ratios, footwall shear failure, hangingwall key block unravelling, and ore overcut floor and back deflections are well known.

## GEOPHYSICAL METHODS

In order to establish bedrock elevations where past surface drilling was sparse, various geophysical techniques had been previously used. With improvements in digital signal processing, geophysical methods were employed once again.

### Ground Penetrating Radar

This work was performed [Davis 1988] employing the PULSEEKKO III equipment utilizing 12 MHZ to 50 MHZ antennas. Vertical surveyed projections along the orebody strike baseline were located on surface and divided into smaller baseline segments (Figure 2). Within dry overburden the radar survey provided reasonable accuracy (10-20% error) but there was poor correlation where 15-30 meters of ground water was present within the overburden. An alternate seismic shear wave reflection survey was recommended but not pursued.

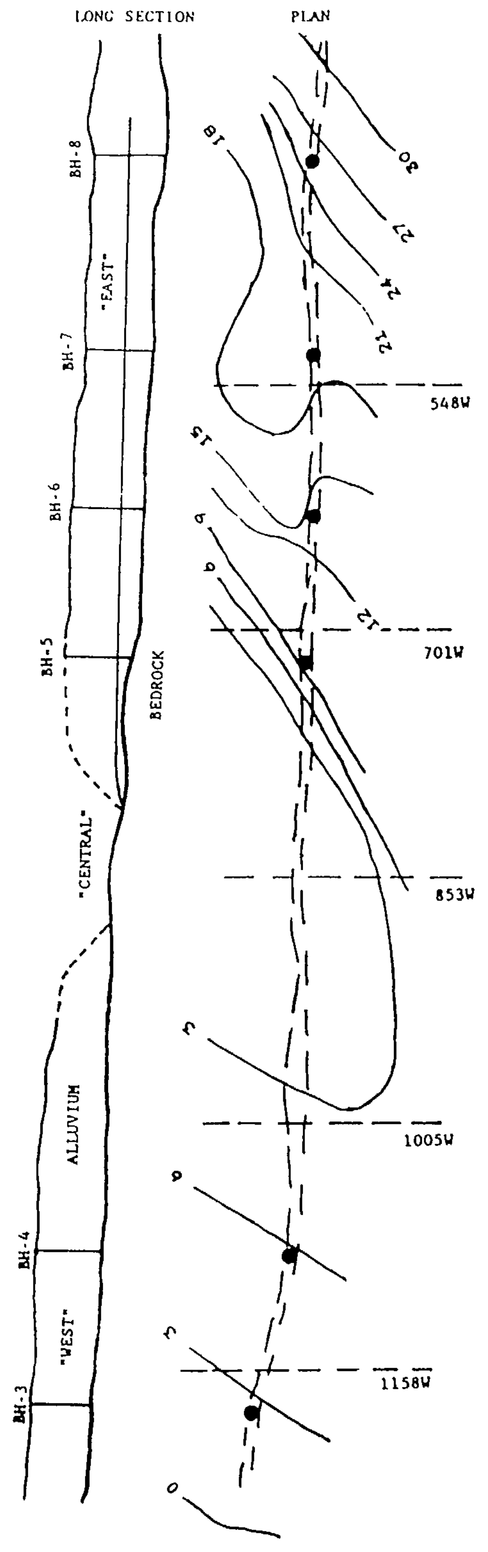


Figure 1 #5 Shaft Crown Pillar Long. Section and Plan with Groundwater Depth Contours (m)

## Seismic Refraction

A seismic refraction survey performed by E.R. Shepard in 1947-48 covered the entire crown pillar strike length, but the results of the survey were questionable when compared with past and current surface test hole data. The accuracy of the method appeared to be $\pm 6$ meters within a 30 meter depth. The survey did indicate irregularity of bedrock elevations in general.

A recent seismic refraction trial [Pehme 1988] revealed a strong seismic reflector 19-25 meters below surface where a sharp velocity increase from 400 m/sec to 2400 m/sec occurred. This compared with the 4000-5000 m/sec compressive wave velocity for the bedrock. The consensus reached was that spread lengths would have to be increased from 15 to 20 meters, and that soil-rock velocities would result in a survey that was not sufficiently detailed to pick up fault trenches. A considerably greater explosive energy was required than envisioned. This technique was abandoned and it was proposed that standard drilling techniques be implemented to determine the bedrock profile.

## BEDROCK PROFILING AND SOIL ANALYSIS

[Hunt et al. 1988] investigated subsoil conditions and ground water parameters at the surface crown pillar site. Drilling was subcontracted through Master Soils of Toronto, Ontario. A CME 55 drilling rig complete with mast and mobile track was used to probe the overburden using hollow stem augers to an average depth of 20 meters. This was usually followed by BW casing and wash boring to depths of 33 meters. A combination of bi-cone or tri-cone bit rotary duplex drilling together with BW or NW casing was used to complete the hole to bedrock. Within the BW casing 3.0 meters of BW core was recovered from the bedrock. A

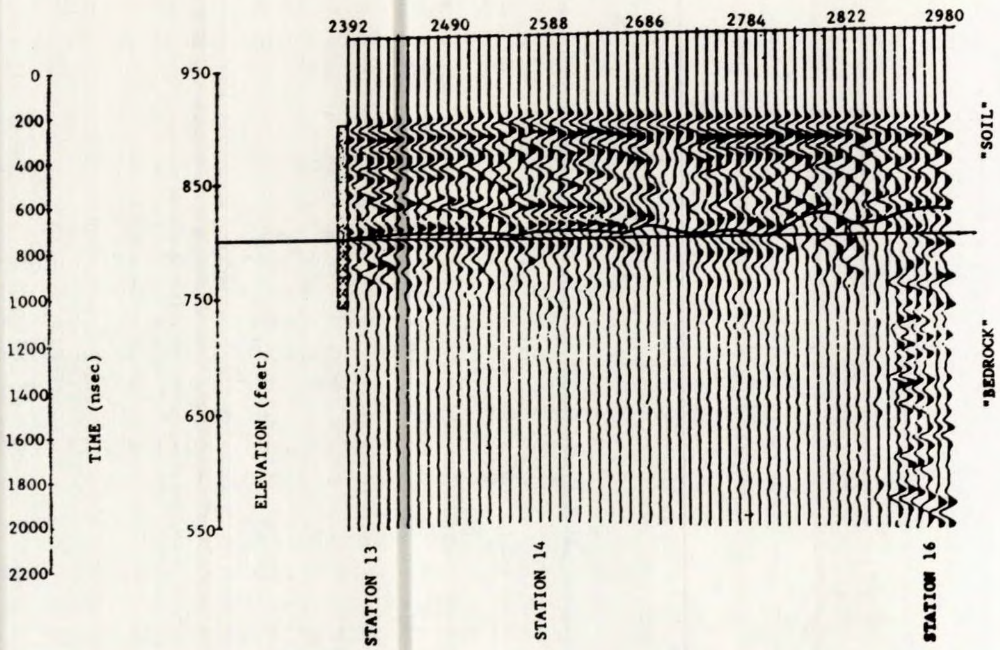


Figure 2 Radargram Along West Crown Profile Delineating Overburden and Bedrock

total of 18 boreholes, 8 with soil sampling, 7 rock probe holes without sampling, and 3 observation wells, were drilled.

Sampling occurred generally at 1.5 meter intervals using split spoons accessed through the hollow stem auger and later through the BW casing. At these same locations penetration resistance tests (blow counts) determining shear resistance were performed. As illustrated in Figure 3, it can be seen that for approximately 6.0 meters depth, sandy gravels were present, while from 6-30 meters the material was essentially sand, and the remaining alluvium again more gravel like in nature. Preliminary estimates using slug testing indicated a permeability of $1.8 \times 10^{-2}$ cm/sec. Soil sampled holes BH-5 to BH-8 were drilled through saturated overburden (Figure 4), with the depth of water varying from 12-26 meters at an elevation comparable to nearby swamps to the southeast. Natural moisture content and Atterberg limits were determined for all soil samples taken at 1.5 m intervals above and below the water table. In boreholes BH-1 to BH-8 simple stand pipe peizometers (12 mm diameter) were placed to record water level readings should a dewatering system be installed.

## GEOMECHANICAL ANALYSIS

Early discussions centered around the flowability of the overburden sand and the stability of the proposed 10 meter thick crown pillar, so computer simulations of both conditions were considered appropriate.

Simulation of the Overburden Sand Conditions

A 6.0 meter wide span geometry at the bedrock elevation was used to infer crown failure under fully saturated and dry sand case conditions. Analysis utilizing the Flac [Hanson 1988] distinct element code suggested that large inflows would occur under saturated conditions, while vertical piping would occur under dry conditions. This suggested that dewatering the overburden could provide increased stability.

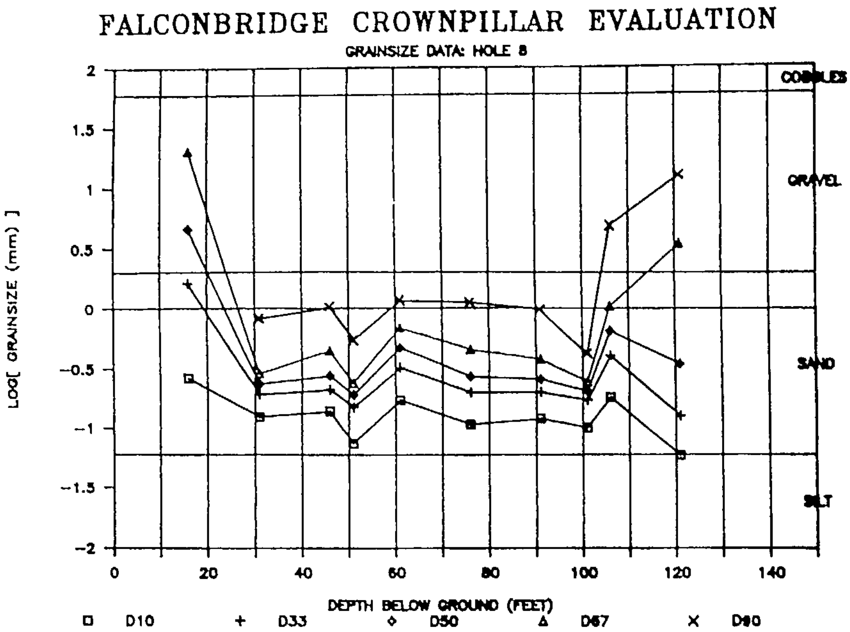


Figure 3 Typical Grain Size Distribution Along Borehole 8 Overburden Depth

Simulation of the Crown Pillar Conditions

Various numerical modelling techniques were used to evaluate the stability of the proposed 10 meter thick crown pillar. Sensitivities could be evaluated but calibration of these techniques was only possible through visual assessment of conditions in the existing overcut.

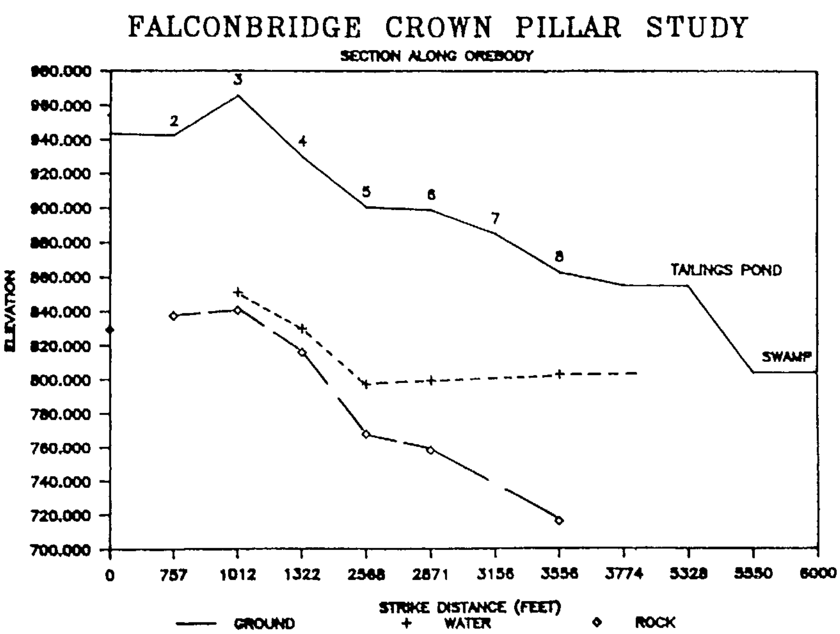


Figure 4 Borehole Locations, Surface, Water, and Bedrock Profiles Directly Above Orebody

Simulation of a Voissoir Beam

A horizontal beam of 6 meter length (ore width) and varying vertical thickness

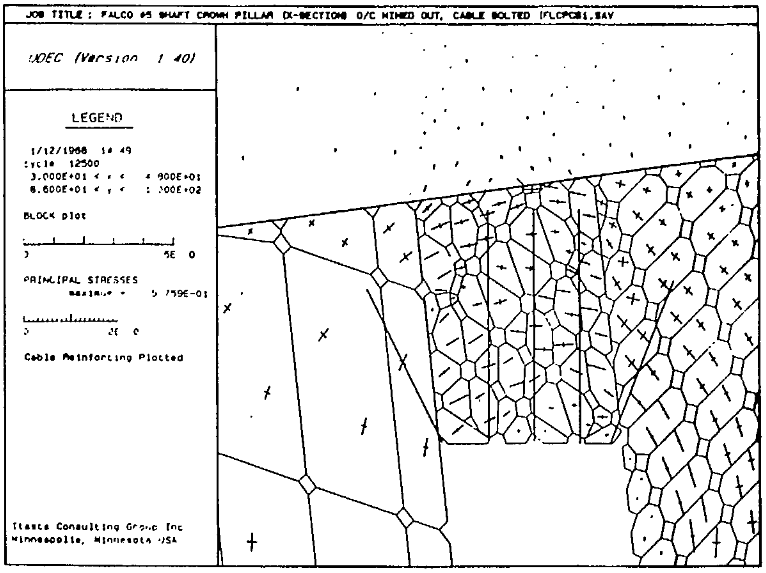


Figure 5 UDEC-Distinct Element Plot of Jointing, Stress Trajectories, and Cablebolt Layout

with and without vertical jointing was simulated. Shear sensitivity analysis [Hanson 1988] did show that changing the rock deformation modulus provided little influence, while reducing the wall contact friction values and increasing the beam thickness had greater influence. Buckling failure was indicated at a 2 meter thick beam and 1 meter joint spacing but stable conditions were indicated with a 10 meter thick beam.

Keyblock Simulation

A Goodman-Shi numerical program [Hanson 1988] complete with graphics hardcopy output was carried out to evaluate the potential of block fallout within the norite and greenstone wall rocks, and sulphide ore roof of the drill overcut. Structural mapping results from the nearby East Mine upper levels and the Falconbridge No. 1 Pit were sorted into joint families of both orientation and spacing. The Keyblock program generated a block shape, stereographic block projection, and stereographic stability analysis reflecting joint friction sensitivity. A worst case gravitational loading environment was examined.

Of the 10 block types identified in ore, 5 required less than 25 degree friction angles, 3 required 40 degree friction angles, while 2 required greater than 60 degree friction angles to stabilize. The last two block groupings suggested that where encountered these would require additional ground support. Within the hanging wall norite, 4 block types were identified as reasonably unstable, while the greenstone footwall containing only 2 dominant joint sets had no potential for block generation.

UDEC-Distinct Element Model

The Voissoir and Keyblock analyses were useful but limited in simulating existing rock mass characteristics. As a result, a reasonably extensive application of the Itasca UDEC distinct element (Version 1.4) program simulated the crown geometry [Clegg 1989].

A 6 meter cross-sectional span and 10 meter thick crown pillar was simulated. The unreinforced UDEC simulations indicated that most of the deformation above the overcut back occurred with excavation of the initial overcut rather than with the additional mining of the stope below. Additionally, maximum shear

Table 1 Water and Bedrock Elevation Data

| WELL NO. | LAND SURFACE ELEV | STK UP (ft) | MEAS PT ELEV | DEPTH TO WATER (ft MP) | DEPTH TO BDRCK (ft ls) | WATER ELEV | BEDROCK ELEV | SAT THICKN (ft) |
|---|---|---|---|---|---|---|---|---|
| OW-1 | | 5 | | | | | | |
| OW-2 | | 5 | | 48.9 | | | | |
| OW-3 | | 5 | | 51.2 | | | | |
| OW-4 | 850 | 1.6 | 851.6 | 53.3 | 153 | 798.3 | 697 | 101.3 |
| BH-1 | 943.3 | | 943.3 | DRY | 114 | DRY | 829.3 | |
| BH-2 | 942.3 | | 942.3 | DRY | 105 | DRY | 837.3 | |
| BH-3 | 935.8 | 5 | 940.8 | 115.6 | 135 | 825.2 | 800.8 | 24.4 |
| BH-4 | 915.8 | 5 | 920.8 | 105.0 | 119 | 815.8 | 796.8 | 19.0 |
| BH-5 | 900.5 | 5 | 905.5 | 106.9 | 153 | 798.6 | 747.5 | 31.1 |
| BH-6 | 892.5 | 2.8 | 895.3 | 98.3 | 141 | 797.0 | 751.5 | 45.5 |
| BH-7 | 875.2 | 4.5 | 879.7 | 82.5 | 141 | 797.2 | 734.2 | 63.0 |
| BH-8A | 858.8 | 4.7 | 863.5 | 63.2 | 147 | 800.3 | 711.8 | 88.5 |
| BH-8B | 858.8 | 4.7 | 863.5 | 63.6 | 147 | 799.9 | 711.8 | 88.1 |
| PW-1 | 849.5 | 2.4 | 851.9 | 52.5 | 151 | 799.4 | 698.5 | 100.9 |
| PW-2 | 878.1 | 1.4 | 879.5 | 80.5 | 145 | 799.0 | 733.1 | 65.9 |
| PW-3 | 895.3 | 1.4 | 896.7 | 99.8 | 149 | 796.9 | 746.3 | 50.6 |
| PW-4 | 865.2 | 1.6 | 866.8 | 67.0 | 125 | 799.8 | 740.2 | 59.6 |

stress and displacements occurred along the footwall and hangingwall contacts, with the highest shear displacements in ore isolated to 40 degree dipping joints. A tensile zone 2.4 meters high above the central overcut back was present but above this a stable arch existed.

A cablebolt design utilizing the AMIRA-Falconbridge Draft 2 Design manual [Fuller 1988] indicated a 5 bolt pattern per line. Cable lengths 6-8 meters were chosen as test drilling from the 200 undercut level had encountered bed separation up to a depth of 4 meters. This was the result of ground relaxation from a 10-13 meter wide stope directly below, idle for 30 years.

The same cablebolt pattern was then incorporated into the UDEC program and provided the following (Figure 5):

1) Net displacements in general were reduced by a factor of 10.
2) Nearly all shear displacements were eliminated along joint surfaces within the ore.
3) Examination of near horizontal major principal stresses revealed an increased stable arch thickness.
4) Shear displacements along the ore/waste contacts were reduced by a factor of 10 but not eliminated.
5) Cable restraining forces were highest at intersections where previously high shear displacements were observed with no reinforcement.
6) In all cases the cable restraining forces were well within the load carrying capacity of the cable.

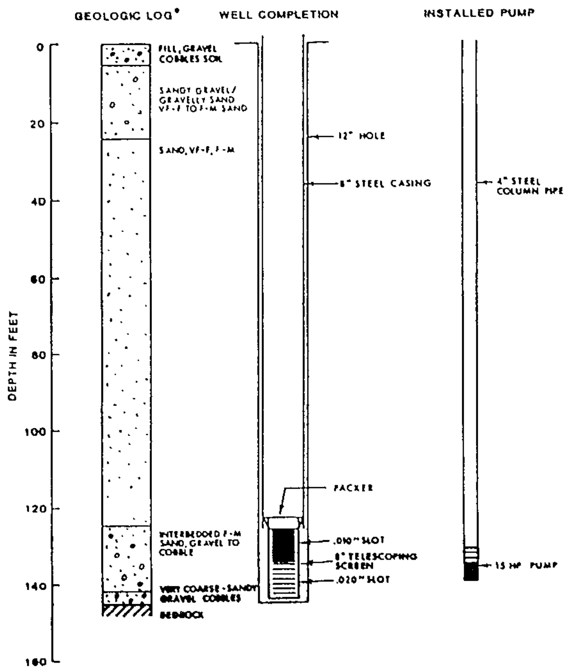


Figure 6 Typical Well Completion and Pump Installation Layout Production Well No. 2

## HYDROGEOLOGICAL EVALUATION

Preliminary analysis involving examination of the surface pillar site conditions, sieve analysis of soil samples, sources of water infiltration, and estimates of permeability indicated a prolific aquifer. The feasibility of a dewatering system specific to the eastern crown pillar section [Brown et al. 1988] was proposed. A review of ground water level data (Table I), at several locations, indicated a reasonably flat water table. Seasonal fluctuations were postulated and were later observed (spring 1989).

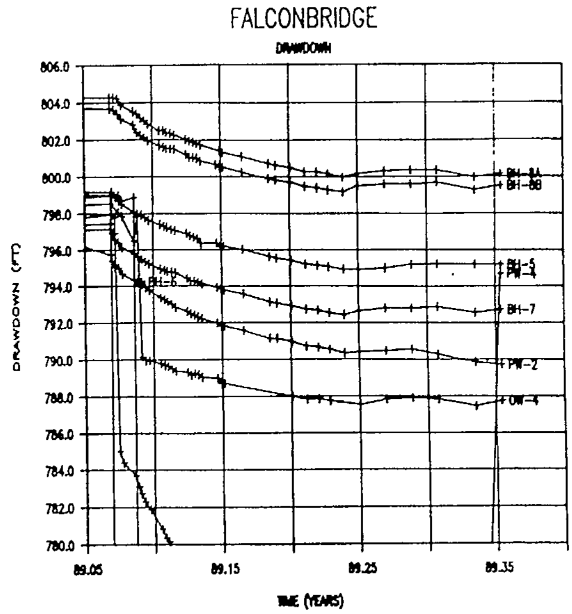


Figure 7 Hydrograph of Drawdown at Observation Well Locations

## Drilling and Pump Installation Program

An air-based ODEX drilling system was experimented with to drill an observation well (OW-4). This was followed by the more traditional mud-rotary duplex system augmented with a steel casing down to bedrock to prevent hole cave. A 6 meter to 12 meter screen was lowered to the hole bottom (Figure 6), and the casing pulled back to expose the screen. A jetting tool was used to develop the well and water subsequently air lifted, to remove the sand. Grundfos Model 225S pumps were selected based on dependability, production capacity, diameter, cost, and availability. The pumps were suspended inside the screen from 10 cm column pipes complete with water level sensors, lightning protection, and 13 mm PVC tubing for in-the-well water level measurement. Four production wells were installed with an average separation of 60 meters along the orebody strike with a combined flow rate of 3150 litres per minute.

## Hydraulic Testing and Analysis

Short and longer duration testing to determine transmissivity and hydraulic conductivity were completed for each production well. A Theis analysis computed on the well data after several days operation indicated a bulk transmissivity ranging from 995,176 lpd/m to 310,992 lpd/m. After 3 weeks of pumping it was apparent that the drawdown was about 0.3 m/week. However, after 13 weeks the drawdown had diminished to approximately 0.02 m/week (Figure 7). It became obvious that the system lacked sufficient capacity. To install adequate capacity was likely to be very expensive.

## Dewatering System Design and Operation

The 10 cm discharge lines from each of the wells discharged their water a total distance of 1062 metres into a polishing pond. Figure 8 shows the wellhead details. During the initial dewatering period water level readings were taken daily and then were reduced to a weekly basis. All electrical controls for the dewatering pumps were housed in a small pumphouse, and linked to a flashing warning light system should pump failure occur.

System Modifications

Dewatering by pumping alone does not appear to be cost effective. Alternatives such as grout curtains restricting water inflow or grout stabilization of the overburden are under consideration. However, indications are that the potential revenue of the ore is unlikely to make any of these options attractive. The final answer is likely to be a greater crown pillar thickness left behind and not recovered.

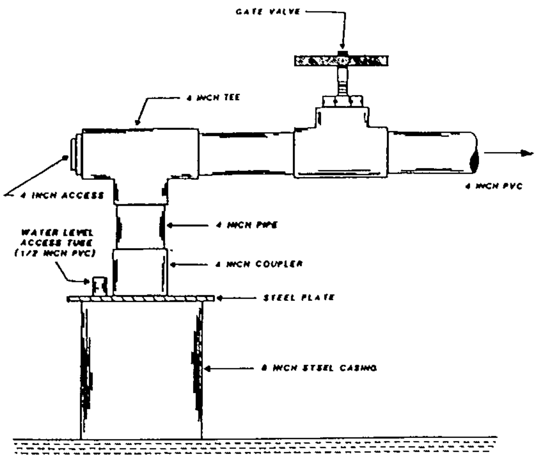


Figure 8 Standard Well-Head Configuration

CONCLUSIONS:

1) The dimensions and rock mass characteristics of the No. 5 shaft crown pillar cap indicate that it will be a stable structure with standard rock reinforcement techniques.
2) The addition of cablebolts together with standard rockbolt support will provide a sufficient factor of safety to mine the crown pillar whether the overburden is wet or dry.
3) Geomechanical models are useful in providing sensitivity analyses on the key rock mass parameters that influence crown pillar instability.
4) Geophysical techniques can provide valid data during the early exploratory stages.
5) With a dewatering technique, determining the bulk transmissivity and hydraulic conductivity is important, but because the drawdown was an everchanging quantity, scheduling of ore removal was difficult.
6) Early recognition of acquifer characteristics is important in order to choose the most effective means of hazard reduction.

FINAL NOTE:

This paper has presented evaluations and conclusions that were carried out from June 1988 until May 1989. A significant water problem in the overburden was found to exist only over the East Crown Pillar. On the basis of initial tests it was assumed that the water table could be lowered to bedrock and thereby eliminate the problem. It was not until May 1989 that results showed this was not economically feasible. As a result the mining plans for this area are being modified and the paper may reflect aspects of mining the East Crown that are no longer valid.

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# ROCK MECHANICS INVESTIGATION AND ASSESSMENT OF NEAR SURFACE CROWN PILLAR STABILITY AT FOUR ONTARIO AND QUEBEC MINES

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## ABSTRACT

The study of near surface crown pillars has recently gained much prominence in the mining community. This has been due to instances of catastropic pillar collapses and to situations brought about by old crown pillars posing hazards to surface structures. The objective of this paper is to present the rationale for investigation and analysis of data used to determine structural, deformation and strength parameters at four operating mines of the Timmins and Val d'Or mining camps. The data analysis techniques include the statistical evaluation of jointing frequency, stereographic evaluation of major jointing and field and laboratory evaluation of rock mass strength and deformational properties. Structural data and mechanical properties were used to produce a rock mass rating in the form of RMR and Q values. Various empirical methods have been used to assess the stability of near surface crown pillars. The applicability of such techniques is expanded upon.

## RÉSUMÉ

L'étude de piliers de surface a acquis récemment beaucoup d'importance dans le secteur minier. On attribue ceci aux effondrements catastrophiques de piliers et aux risques que posent les vieilles excavations sous les aménagements en surface. L'objectif de cet article est de présenter un cheminement pour l'investigation et l'analyse des données utilisées afin de déterminer les paramètres de la structure géologique, de déformation et de résistance, reliés à quatre mines des camps miniers de Val-d'Or et de Timmins. Les techniques d'analyse employées incluent le traitement statistique de la fréquence des joints, l'évaluation stéréographique de la fracturation majeure, et la quantification en laboratoire et en place de la résistance et de la déformation des massifs rocheux. Les données de la structure géologique et les propriétés mécaniques ont servi à calculer les cotes RMR et Q de qualité des massifs. Diverses méthodes empiriques ont été utilisées pour évaluer la stabilité des piliers de surface. L'applicabilité de telles techniques est également discutée.

## INTRODUCTION

The study of near surface crown pillars often conjures up images of the immediate superincumbent rock between the hangingwall and footwall of a mine opening. It is in part the objective of this paper to look at the consequences of broadening this view to include the overall near surface stability of mine openings. As such, stability considerations must be given to both the hangingwall and footwall zones, and hence extend the scope of the required field investigation.

The field investigation phase should be geared towards optimizing the information obtained from individual boreholes, taking into account expected section geometries and orientations along with consideration to possible surface obstructions. Such field investigations can be costly and therefore the most should be made of the recovered core in terms of logging, lab and field testing and the detailed analysis of the determined properties.

The summarized properties should then be placed in perspective around the opening and used in modelling the behaviour of the in-situ rock mass. A sensitivity analysis of stability over the statistical range of the data accumulated should be performed.

## FIELD INVESTIGATION AND DATA COLLECTION

The principle objective of the field investigation phase was to collect relevant strength and deformational properties of weak rocks from the hangingwall, footwall and ore zone rock types. In our study this included four Quebec and Ontario mining projects comprising Belmoral, Chimo, Bousquet and Pamour.

Boreholes were all pre-planned [Piciacchia, 1988]. Figure 1 shows a typical surface plan and section, Pamour mine, indicating the surface coordinates and inclinations of the three planned holes. The objectives for each hole drilled was to recover core from a minimum of two zones (e.g., hangingwall, ore zone). In addition, the boreholes were spaced along strike to provide a more representative and unbiased assessment of material properties. All borehole inclinations and locations selected followed this rationale and the determined locations where surveyed and staked, in the field.

Drilling at the Pamour and Belmoral sites was conducted with a Bombardier mounted Longyear 34 diamond drill. The other two sites, Chimo and Bousquet, were cored using a skid mounted Inspiration drill. All rock coring was conducted with NQ impregnated face ejection diamond drill bits and triple tube core barrels, as recommended [Betournay, 1988a, 1988b]. This was done to minimize core loss and disturbance. In addition, the coring was conducted with a polymer mud-water mixture of a sufficiently thick consistency to reduce bit vibration to a minimum. Even with these precautions, successful core recovery was still operator and machine dependent.

For instance it was found that the smaller Longyear 34 machine had considerably more problems drilling at depth than did the Inspiration drill. Consequently, the core recoveries were notably lower for the former machine.

This problem was, in part, corrected in the field by switching to shorter, 5 feet core runs which helped the situation by reducing the residence time of the core in the barrel.

The core was initially logged in the field with the rock type, joint orientation with respect to borehole axis, joint roughness and infilling, NGI alteration, along with the joint density, RQD and recovery being noted. In addition, the NGI and CSIR rock quality indices were later calculated and incorporated into the core log. A typical core log is presented in Figure 3.

In addition to logging the core as indicated on Figure 2, in-situ dilatometer tests were conducted in the field in order to determine the bulk elastic modulus. In-situ dilatometer testing was attempted in all holes with limited success. This was primarily due to weak rock caving into the hole on retrieval of the drill rods. Dilatometer results typically yield modulus values which were an order of magnitude lower than laboratory tests. Table 1 compares the following values with the surface crown pillar zones tested.

The recovered core was tested in controlled laboratory conditions, [Gorski, 1989] This data was also summarized on the core log, Figure 2. The laboratory determined data consisted of Brazilian tests, single stage uniaxials and triaxial tests. Primary analysis of this data resulted in the determination of peak tensile and compressive strengths along with the elastic modulus and Poissons ratio. Detailed analysis of the data combined the results of the Brazilian, uniaxial and various triaxial tests to determine the **m** and **s** values for the various rock types.

## DETAILED ANALYSIS OF DISCONTINUITIES

In a second phase, the core from each borehole was re-analized statistically. Firstly, the joint spacings for each borehole were used to develop a frequency distribution. These distributions were subsequently used to determine Means and Standard deviations along with modal values of joint spacing for each borehole, Table 2. This data can subsequently be used as input to numerical models such as some discrete block codes.

In addition to this type of analysis the joint orientations have been plotted on Schmidt equal area lower hemisphere projection stereonet. An example is presented in Figure 3 with orientations relative to top of core, as such no north arrow was provided. The poles have been statistically contoured using a gaussian function in order to provide a best estimate of joint orientation and to help identify individual families of joints. These have been summarized in Table 2.

Such detailed analysis of material properties data enables mechanical assessment of the rock mass and the verification and rationalization of input properties to subsequent modelling. In addition, the statistical analysis provides a logical range over which sensitivity analysis can be performed on prediction of stability.

## ROCK MASS ASSESSMENT

The dilatometer results are low and reflect the weak rock masses that were tested. The laboratory modulus of elasticity value is 20 to 50 times the field value. These low field values indicate the critical nature of weak schist rocks in which they were obtained. The lab and field translated **m** and **s** values available for these rock types are also very low. The field modulus translated from rock mass quality is also higher than field measured values, 2.3 to 9.3 times.

On the whole, the RQD value, NGI and CSIR rating is proportional to the field modulus of elasticity values.

## STABILITY ASSESSMENT

Several analytical, empirical and numerical models are available for assessing ground stability. Each of these general groups provides some pros and cons with respect to their usage in assessing the stability of near surface crown pillars. For the purpose of this paper we shall limit ourselves to a discussion of empirical models only.

The empirical models discussed here are all linked to a particular rock mass classification system. As such these empirical models are prone to the same assumptions and limitations as the classification system they use. Thus, it is not possible to discuss these empirical models without first examining the rock mass classification systems which form an integral part of the design process.

Several rock mass classification systems should be examined, these range from the simple [Deere classification, 6] to the more complex [NGI, 7] and [CSIR, 8] systems. It is noted that these systems were initially intended to provide a representation of rock mass properties. Each system helps describe the rock mass to varying degrees, however, none of the systems adequately incorporated the interaction between mining geometry, orientation and rock mass characteristics.

The models examined, included, the Merritt system, NGI tunneling system and the CSIR standup time. It is further noted here that each of these design concepts were geared towards design of tunnels, as such no direct account is taken of the dimensions and stability of the superincumbent rock above the opening (crown pillar). Therefore, at best, only an indirect measure of crown pillar stability can be made with these models.

It becomes evident from applying, Table 3 and Figure 4, these empirical models that they provide little assistance in crown pillar design. For instance the CSIR standup time yields totally unreliable and unreasonable results such as standup times for less than a month when field verifications indicate the pillar has been maintained in place for over 40 years. This is principally due to the fact that the definition of active span in this method is based on [Lauffer, 1958] interpretation which does not account for depth and dip length of openings.

In addition to these tunneling empirical models a modified version of the [Mathews, 11] method was used to assess stability. This design system is based on a modified NGI-Q rating and has been developed to be of assistance in stope dimensioning at depth. The system has been modified by applying lower levels of initial stress, to account for the shallow depths involved. An elasto-plastic finite element program was employed to assess induced stresses. This method uses a selection of geotechnical factors which are combined to provide a stability number. This number is then plotted against a shape factor to empirically assess the stability of surfaces which bound open stopes. The stability number used accounts for rock mass quality, the state of stress and the orientation of exposed surfaces, while the shape factor accounts for the shape and size of the opening. This method's biggest limitation is its inability to handle multiple openings. Typical stability plots are provided in Figure 5.

## SUMMARY

The results of a surface crown pillar rock mechanics field campaign have been presented. Full use of diamond drilled boreholes in weak rock hangingwall, ore zone and footwall have provided excellent indications of potential rock mass behaviour.

The rock core has yielded laboratory modulus of elasticity **m** and **s** parameters, RQD values, NGI and RMR ratings and empirical field scale modulus of elasticity. Orientation and density of joint families are presented.

In-situ dilatometer tests were performed on the boreholes to provide rock mass moduli of deformation.

The data collected indicates that modulus of elasticity values are directly related to RQD, NGI and RMR levels. Also, the field **E** is much smaller than the laboratory **E** and smaller than the empirically calculated **E** value.

The weak rock masses measured in-situ in this campaign have a very low modulus of elasticity.

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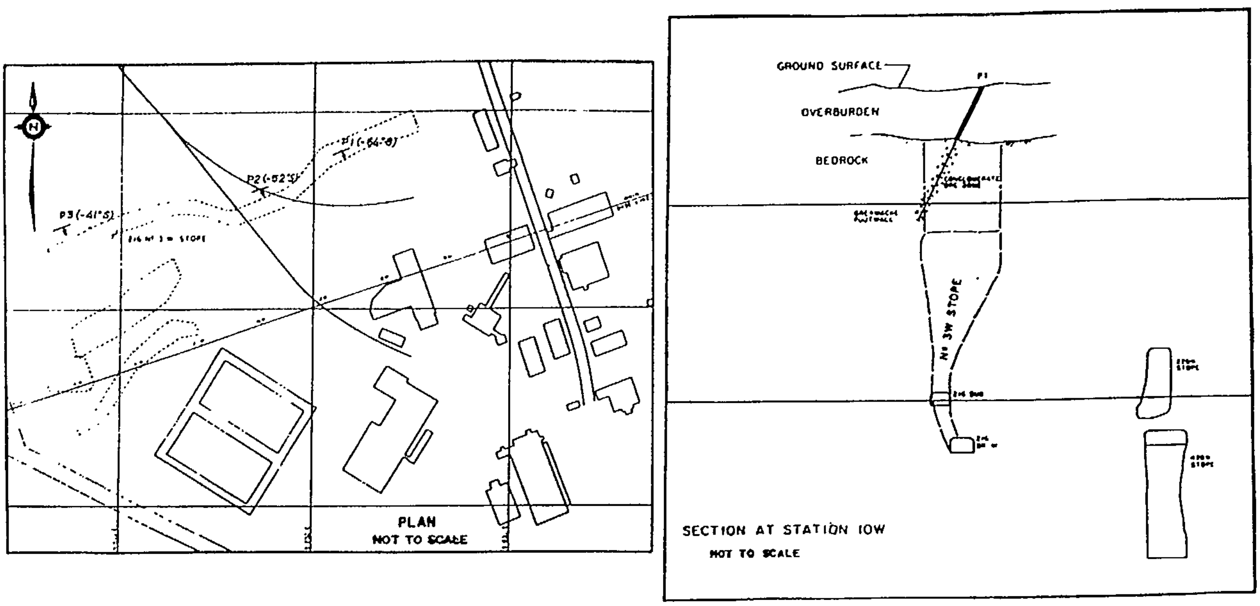


**Figure 1.** Typical Plan & Section - Pamour Mine

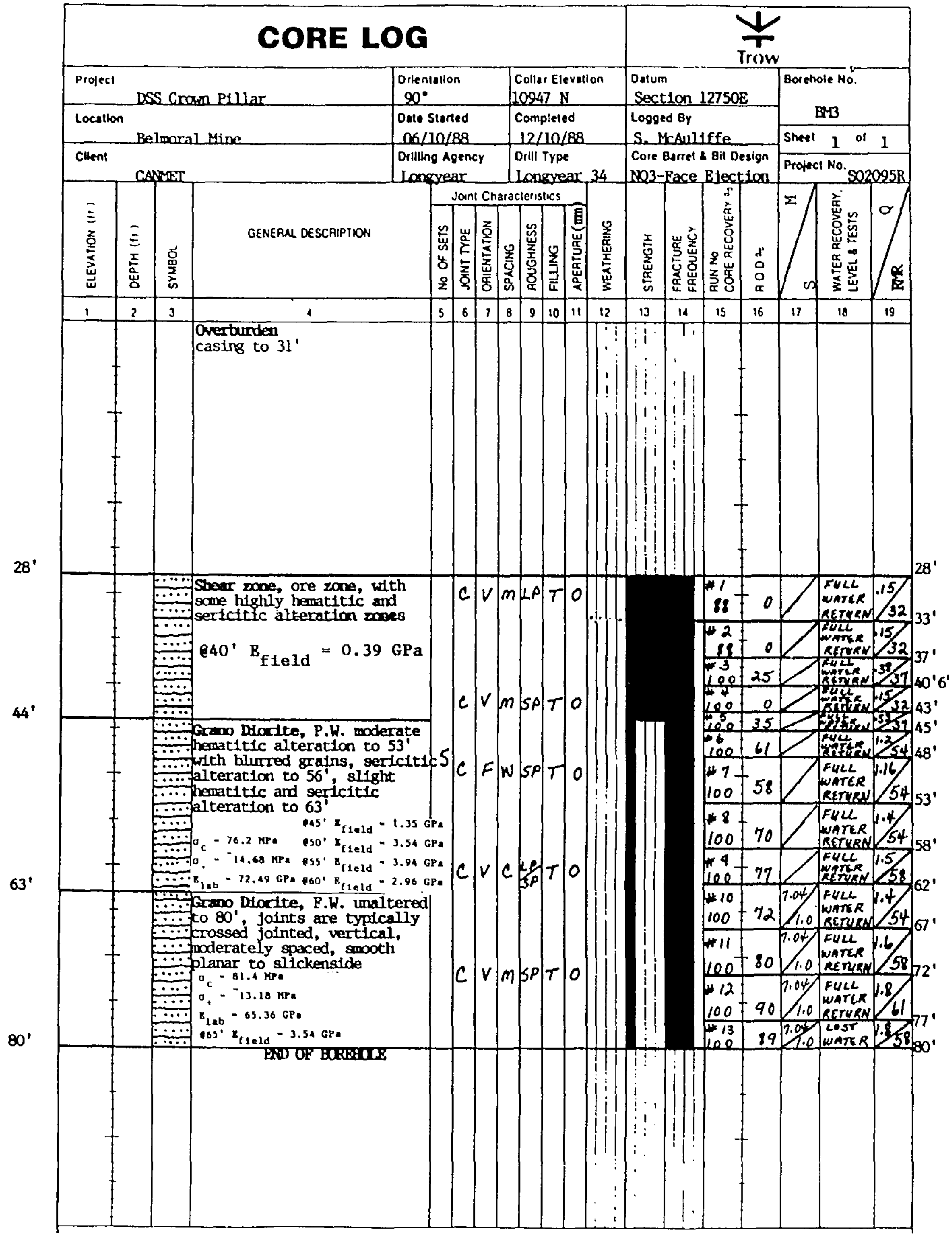


**Figure 2.** Typical borehole log - Belmoral, Borehole 3 after Piciacchia (5)

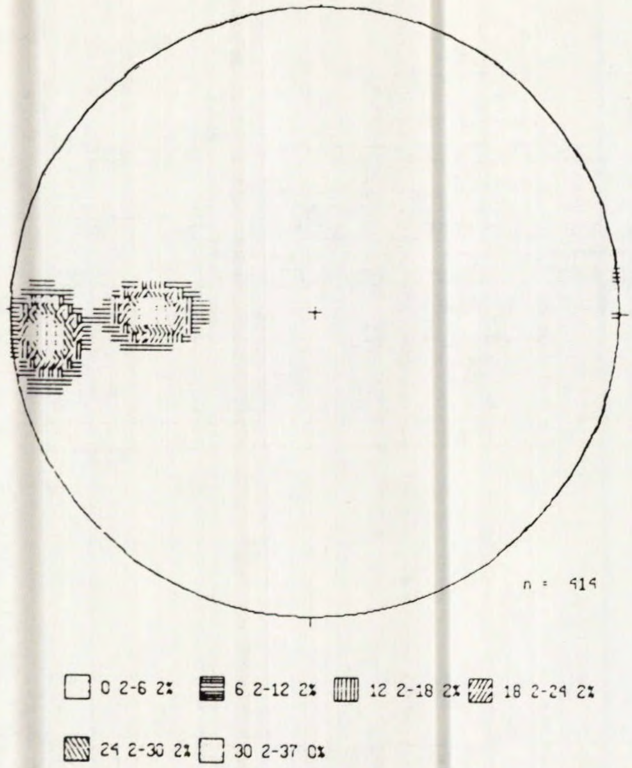


**Figure 3.** Contour of Stereographic Projection Poles - Chimo

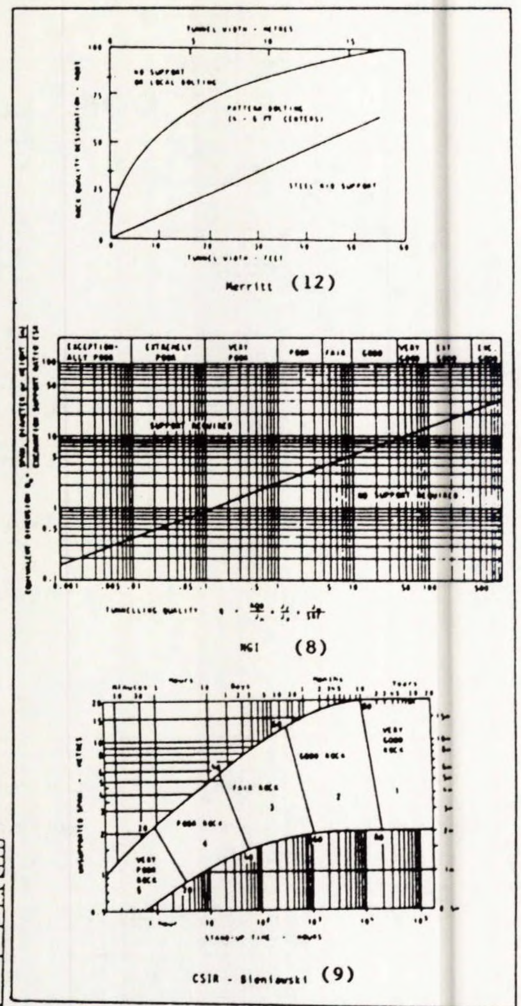


**Figure 4.** Empirical Design Methods

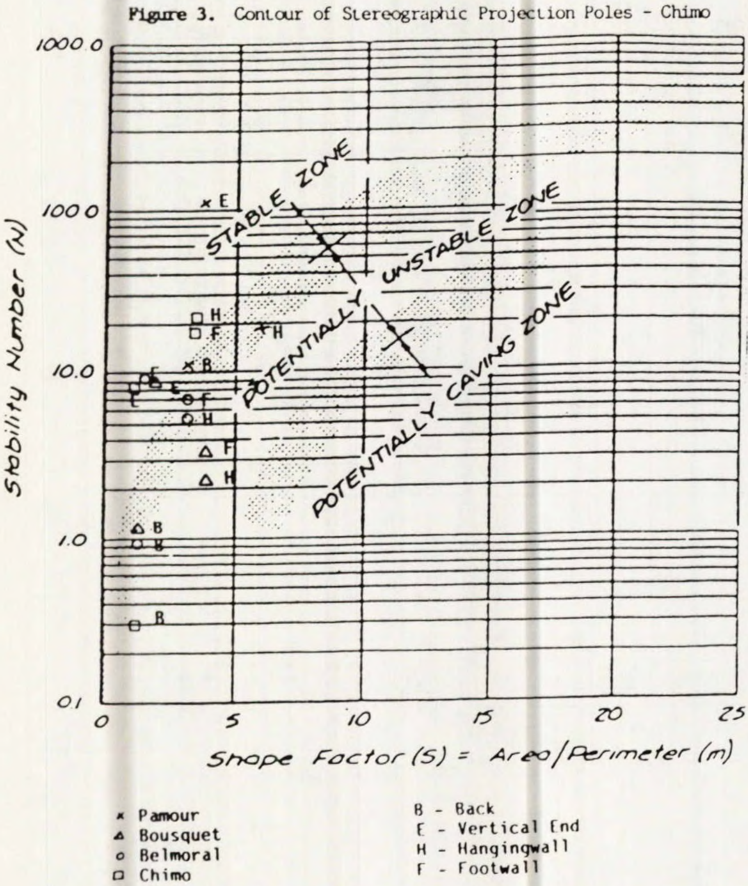


Figure 5.
Graph of Stability Number (N)
Versus Shape Factor (S)

(after Mathews et Al. 1981)

TABLE 1

Summary of Typical Lab, Field & Empirically Determined Rock Mass Properties

| Location | Depth (m) | Elab (Gpa) | Efield (Gpa) | Eemp | RQD | NGI (Q) | CSIR (RMR) | M | S |
|---|---|---|---|---|---|---|---|---|---|
| Pamour | 5.28 | 59.66 | -- | 14.96 | 53 | 3.1 | 57 | 4.8 | 1 |
| | 11.43 | 68.31 | -- | 70.8 | 100 | 26.7 | 84 | 4.8 | 1 |
| | 17.3 | 142.9 | -- | 63.1 | 92 | 18.4 | 82 | 4.8 | 1 |
| Bousquet | 16.5 | 63.4 | 1.90 | 5.96 | 64.5 | 3.2 | 41 | 2.75 | 1 |
| | 25.15 | 55.0 | 2.5 | 5.96 | 69.8 | 3.5 | 41 | 2.75 | 1 |
| Belmoral | 13.84 | 68.9 | 1.35 | 12.6 | 58 | 1.16 | 54 | -- | -- |
| | 15.34 | 69.5 | 3.54 | 12.6 | 58 | 1.46 | 54 | -- | -- |
| | 17.86 | 75.50 | 3.95 | 12.6 | 70 | 1.4 | 54 | -- | -- |
| | 18.02 | 76.03 | 2.96 | 15.8 | 77 | 1.5 | 58 | -- | -- |
| Chimo | 26.52 | 96.86 | -- | 35.5 | 97 | 7.2 | 72 | 7.41 | 1 |
| | 31.29 | 87.28 | -- | 17.78 | 45 | 3.3 | 60 | 7.41 | 1 |
| | 36.14 | 87.70 | -- | 29.85 | 75 | 5.6 | 69 | 7.41 | 1 |
| | 43.44 | 107.5 | -- | 23.71 | 73 | 5.4 | 65 | 2.84 | 1 |

TABLE 2

Summary of Structural Data

| Mine Site | Dip | AZ-Dip Direction | Borehole # | Mean (ins.) | Standard Deviation (ins.) | Modal Value (ins.) |
|---|---|---|---|---|---|---|
| **Pamour** | | | **Pamour** | | | |
| Major joint Set #1 | 43.78 | 357.26 | P1 | 3.77 | 3.70 | 2.5 |
| Minor joint Set #1 | 61.64 | 127.67 | P2 | 6.9 | 5.3 | 4 |
| | | | P3 | 3.80 | 3.50 | 3 |
| **Bousquet** | | | **Bousquet** | | | |
| Major joint Set #1 | 68.7 | 274.3 | B1 | 3.72 | 3.72 | 1 |
| Minor joint Set #1 | 26.8 | 123.5 | B2 | 4.95 | 5.17 | 3 |
| | | | B3 | 4.39 | 4.56 | 1 |
| **Belmoral** | | | **Belmoral** | | | |
| Major joint Set #1 | 51.9 | 272.9 | BM1 | 10.18 | 10.95 | 3.25 |
| Major joint Set #2 | 31.72 | 88.5 | BM2 | 6.49 | 5.64 | 2.5 |
| Minor joint Set #1 | 61.3 | 343.3 | BM3 | 9.59 | 9.39 | 3.85 |
| Minor joint Set #2 | 41.5 | 19 | | | | |
| Minor joint Set #3 | 38.37 | 185 | | | | |
| **Chimo** | | | **Chimo** | | | |
| Major joint Set #1 | 39.4 | 90.1 | C1 | 5.98 | 5.57 | 3.5 |
| Major joint Set #2 | 73.8 | 85.2 | C2 | 5.75 | 5.63 | 3.5 |
| | | | C3 | 5.76 | 4.82 | 2 |

TABLE 3

Summary of Simple Empirical Methods

| Mine | Stope Width (m) | Stope Height (m) | O/B Depth (m) | Merritt System | NGI | CSIR Standup Time |
|---|---|---|---|---|---|---|
| Pamour | 9.2 | 38.13 | 6.1 | pattern bolting on all walls | no support required | 2½ months |
| Bousquet | 5 | 40 | 15 | pattern bolting HW/FW | no support required | 1 day |
| Belmoral | 3.05 | 45.8 | 6.1 | pattern bolting on HW | no support required | 10 days |
| Chimo | 4 | 40 | 30 | no support required | no support required | 2½ months |

# SEISMIC IMAGING IN THE EVALUATION OF SURFACE CROWN PILLAR ROCK MASSES

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## ABSTRACT

Traditional approaches to the characterization of rock masses are often based on the assumption of a simple relationship between points of measurement (boreholes, surface exposures). Geophysical methods, however, allow the properties of rock away from the borehole/surface access to be investigated. Among different geophysical techniques, the seismic method is the most applicable in the delineation of fractured/poor quality rock within a surface crown pillar. Seismic tomography images the velocity/attenuation of a rock mass by transmission of seismic waves. The images obtained show how the seismic properties of the rock change within the rock mass and could be interpreted in terms of rock quality, in order to delineate areas of poor rock quality/major discontinuities. This paper describes the results of a crosshole seismic tomographic experiment carried out in a surface crown pillar mine.

## RÉSUMÉ

Les approches traditionnelles à la caractérisation des massifs rocheux sont souvent basées sur l'hypothèse qu'une simple relation existe entre des points de mesure (forages, affleurements de surface, etc.). Les méthodes géophysiques, quant à elles, permettent d'évaluer les propriétés de la roche loin d'un forage/d'une surface accessible. Parmi les différentes méthodes géophysiques, la méthode sismique est la plus appropriée pour la localisation de zones de roche fracturée/de faible qualité à l'intérieur d'un pilier de surface. La tomographie sismique consiste à obtenir une image de la répartition de vitesse/d'atténuation des ondes sismiques transmises dans un massif rocheux. Les images ainsi obtenues montrent comment les propriétés sismiques varient à l'intérieur du massif et peuvent être interprétées en terme de la qualité de la roche afin de localiser des zones de faible qualité/des discontinuités majeures. Ce papier décrit les résultats d'une expérience in situ de tomographie sismique entre puits effectuée dans une mine de piliers de surface.

## 1. INTRODUCTION

Surface crown pillars are near-surface mining structures which separate and protect personnel and mining operations from surface elements. The stability of these structures depends on their dimensions, as well as the integrity and quality of the rock. As many surface crown pillars contain economic ore concentrations, an optimum design has economic (and safety) implications. A design process for surface crown pillars in hard rock was suggested by Bétournay (1986). As part of this design process, it was highlighted that anomalous ground and major discontinuities within surface crown pillars are the most important factors affecting their stability.

Traditional approaches to the characterization of rock masses rely on interpretation between boreholes and/or surface exposure. Several geotechnical rock mass classification systems exist, notably the Bieniawski and Barton (NGI) systems for classifying the changes in the integrity of rock masses for exposed rock (Barton 1974, Bieniawski, 1973). These interpretations are often based on assuming a simple relationship between the points of known information. Geophysical techniques, such as seismic methods, on the other hand, provide a method of mapping rock property changes away from the borehole or surface opening, while still utilizing the known borehole information for interpretational control. To date geophysical investigations in surface crown pillars have been restricted to the use of very simple surface refraction techniques to determine depth to bedrock and bedrock velocity.

The Canada Centre for Mineral and Energy Technology (CANMET) has in the past few years initiated a major research effort to gain more information for the safe and economic design of surface crown pillars. As part of this initiative, CANMET sponsored a research project undertaken by the Engineering Seismology Laboratory of Queen's University to develop necessary techniques for evaluation of surface crown pillar rock masses using seismic tomographic imaging techniques. This paper briefly describes seismic tomographic imaging in relation to surface crown pillar characterization, the outline of an in-situ experiment undertaken as part of the above project and preliminary results obtained to date.

## 2. SEISMIC TOMOGRAPHIC IMAGING

Tomography may be defined as the reconstruction of the internal structure of an object from a set of its projections, a projection being a series of line integrals of some physical parameter through the object at a given orientation (McMechan 1983). In order to determine the rock variation of seismic wave velocity throughout a two dimensional plane of rock, access must be gained to at least two sides of the plane in question. This may, for example, be between boreholes, mine walls, levels or around pillars. Seismic energy sources and receivers are placed in precisely known locations surrounding the plane to be imaged. The travel time for the seismic energy between every possible source-receiver pair is accurately measured. Travel time for a given raypath is inversely proportional to average velocity of the rock along the raypath, the constant of proportionality being the raypath length. The objective of tomographic processing is to assign velocity values to many small discrete areas, often called cells or pixels, of the area being imaged.

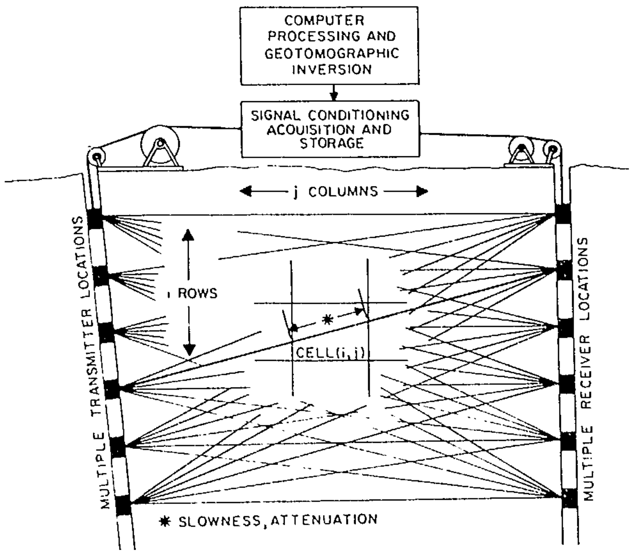


Figure 1 - Schematic diagram of a crosshole seismic tomographic experiment.

Travel time data for many crossing raypaths are required for an accurate solution (Figure 1).

A fundamental complication of seismic tomography, not encountered in x-ray tomography, is that raypaths are not straight where velocity is not constant. A general strategy for the determination of velocity values within each cell when the actual raypaths are not known is to initially assume straight raypaths and assign velocity values accordingly. The image obtained is a first approximation to the actual velocity field. Iterative techniques are then used to proceed from the straight ray solution to a best fitting refracted raypath solution. The result is a map of discrete velocity values throughout the plane being imaged, consistent with both measured travel times and the physical laws describing seismic wave propagation.

Geotomography is applicable to imaging both compressional and shear wave velocity, depending on which travel times are measured. The imaging of attenuation parameters is analogous to slowness imaging, with energy level rather than travel time being measured. Several authors have demonstrated the effectiveness of geotomographic imaging in the geotechnical field notably Mason (1981), Wong et al (1983), Cosma (1983), Peterson et al (1985). More recently Kormendi et al (1986) and Young et al (1988) have shown the applicability of the technique for sequential imaging of mining induced stresses and fracturing. The possibility of delineating areas of poor rock quality/fractured rock within a surface crown pillar using seismic tomographic imaging opens a new area of application of this technique which could be of valuable interest in the process of design of these structures.

## 3. DESCRIPTION OF THE SITE

The Pierre Beauchemin mine (formerly Eldrich) is situated near Evain, Rouyn-Noranda in Québec. The choice of this site for an in-situ tomography experiment was due to several factors. The presence of the Eldrich fault and its associated structural elements combined with at least four joint families creates concern about the rock mass quality and support requirements. A significant overburden is present (layers of silt and till) and a local body of water is continually replenished by a stream. Moreover, the mine has received a relatively significant amount of attention in terms of geotechnical evaluation. Since this site seems to be a very suitable one for the evaluation of tomographic techniques in the characterization of mine surface crown pillars, it was selected for the in-situ experiments.

## 3.1 The Geology

The Pierre Beauchemin site is situated near the west border of the Flavrian batholith, a regional intrusive of 17 x 18 km in extent. The principal rock-types at the mine are gabbro, tonalite, and various other hybrids. As described by Bray (1987), the Eldrich gabbro, which is called 'diorite' at the mine, is greenish-grey, fine and chloritized. Carbonate stringers within the gabbro exist at shear zones and as alterations at contacts following the schistosity. The tonalite is pink, silicified, and characterized by the existence of amphibole 'needles'. In general, this lithological unit is very heterogeneous in composition and texture. Its contact with the gabbro is smooth, extensive and easily detachable in the mining environment (Bétournay, 1987).

## 3.2 Fracture Field

Marcoux and Grenier (1986) have mapped different discontinuity families from cores, as well as underground exposures. Cores from three surface holes were logged and different families were determined after all the joint planes were plotted on a stereonet (Table 1). Exposures from the first level were mapped according to the procedure proposed by the Norwegian Geotechnical Institute (NGI). Four families of discontinuities were distinguished (Table 2).

TABLE 1 - Discontinuity families - borehole study, Marcoux & Grenier, (1988)

| Borehole | Discontinuity Families (strike/dip) |
|---|---|
| 88-335 | 291/82, 254/48, 180/4, 8/38 |
| 88-336 | 293/78, 259/30, 308/57 |
| 88-337 | 271/84, 252/39, 290/44 |

TABLE 2 - Discontinuity families mapped from exposures at level #1 (after Marcoux & Grenier, 1988)

| Orientation (strike/dip) | Type |
|---|---|
| 20/54 | fault |
| 298/90 | joint family |
| 215/48 | " |
| 345/87 | " |

In an earlier study, Bétournay (1987) presents the results of an underground inspection of the deeper drifts and stopes of the west lenses from levels 2 through 6 and level 6 around zone 5. Four joint families were distinguished in these areas ranked according to their predominance:

1) NNE-SSW, subvertical;
2) NW-SE, subvertical;
3) NE-SW, $50^{\circ}$ NW;
4) subhorizontal

These joint families are all extensive, usually greater than 2 m, planar, smooth, and sporadic in occurrence. Their spacing is variable (usually greater than 30 cm) and when they rarely intersect, blocks 0.5 - 1 $m^3$ are formed. Poorly developed schistosity oriented parallel to the fault creates associated discontinuities less than 60 cm long spaced 30 - 50 cm. The most common situation encountered is occurrences of only one joint family accompanying the pervasive fault (or its associated pseudo schistosity).

### 3.3 The Overburden-Bedrock Contact

A surface seismic refraction survey was performed at the mine in 1987 (Le François, 1988). This study was used to map the overburden/bedrock interface, to determine the stratigraphy of the overburden, and to infer weak zones in the bedrock as indicated by reduced seismic velocities. The results confirmed the irregular overburden/bedrock surface as well as the presence of water-saturated till and clay layers. Corroborative results were obtained during the drilling of seven shallow boreholes in the area. Bedrock samples observed indicated a general rock quality varying from good to excellent, however, locally the quality was mediocre. The hydrological observations of drainage within the overburden and upper part of the bedrock indicated rather high permeability.

## 4. DESCRIPTION OF THE EXPERIMENT

### 4.1 Borehole Layout

Multiple borehole access to the near-surface rock mass providing several tomography image planes has been determined to be best suited for the purpose of the experiment. Drilling the holes in a grid pattern would have only provided information in two mutually perpendicular directions. However, having boreholes distributed in a circular pattern allows for investigation of seismic wave velocity and attenuation anisotropy. This pattern was therefore chosen (see Figure 2). Redundancy was removed by drilling a semi-circular pattern over $180^{\circ}$ and replacing holes on one half of the circle by one drilled in the circle centre. Thus, the same anisotropy information ($360^{\circ}$) is acquired but drilling costs are reduced. The plunge of the boreholes was set to $65^{\circ}$. This was a trade-off between limitations due to the plunge of a fault zone and the presence of the first level of the mine (Figure 2). Note that hole R1 acts as a reference hole for tomographic surveys from C1 to Q1 and C1 to Q2. Likewise, hole R2 serves a similar purpose for surveys from C1 to Q3 and C1 to Q4.

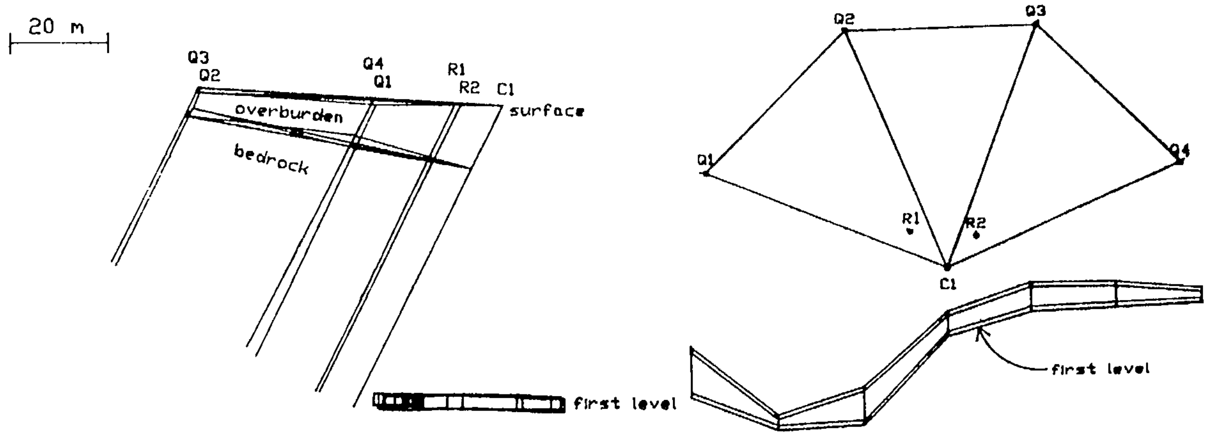


Figure 2 - Borehole layout, a) section looking Mine North; b) view looking down the boreholes.

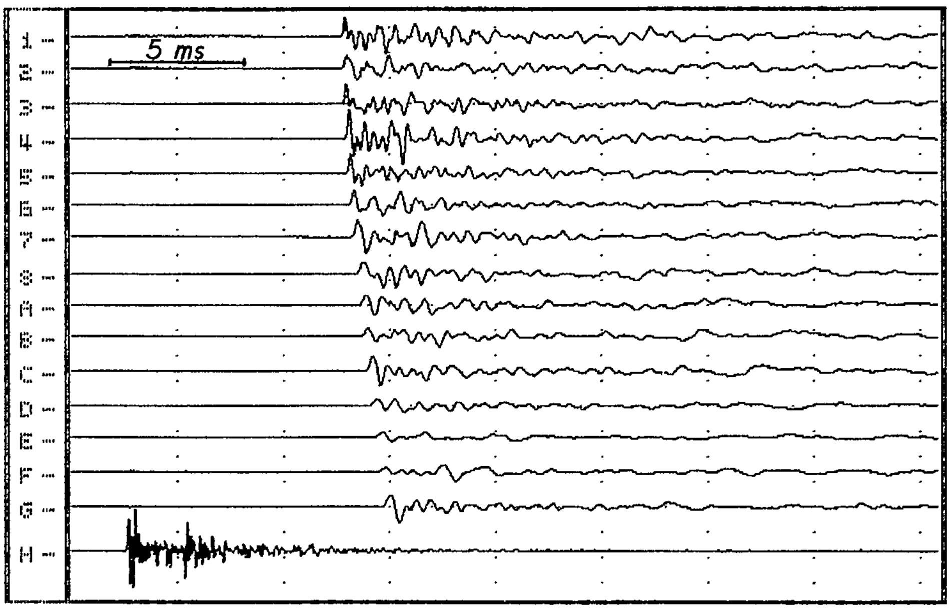


Figure 3 - Example of signals recorded after a blasting cap was shot in borehole C1. Channels 1 to G show signals from the string of hydrophones lowered in borehole Q2 and channel H shows signal from the reference hydrophone in borehole R1.

### 4.2 The Data Acquisition System

The Queen's University pc-based data acquisition system has been described elsewhere (Young et al, 1989; Talebi and Young, 1988); just a brief description is given here. The concept used in the design of this system was based on utilizing existing mass-produced technology rather than expensive specialized equipment. The sensors used in this survey were Wilcoxon's model 505 self-amplified hydrophones with a sensitivity of -180dB re 1V/μPa and a frequency band of 1Hz to 15KHz at +/- 3dB. They could operate down to a depth of 500 m. The signals from the hydrophones pass through a differential amplifier and anti-aliasing filter board where they are band-pass filtered in the frequency domain 500-10,000 Hz to avoid aliasing. Each channel has a gain switch and allows amplification of 0, 10, 20, 30, 40 or 50 dB to be operated. The high frequency roll-off rate is 72 dB per octave. An IBM Model 30 (personal system 2) computer was used to record the signals. The acquisition package is composed of a 16 channel, 12-bit, 1 MHz A/D board, an external interface unit and associated software. This package allows up to 16 channels of seismic data to be digitized at aggregate sampling rates up to 1 MHz. The sampling rate was set to 50 KHz for each channel; the data was then backed up on 60 Megabytes cartridge tapes using a tape streamer.

### 4.3 The Experiment

During a tomographic in-situ experiment carried out in November 1988 two planes were imaged: C1-Q2 and C1-Q3 (see Figure 2). Seismic blasting caps type MK2 were used as the source down the shot hole C1. Blasting caps were shot at 1 m intervals and seismic waves generated were monitored using a string of 15 hydrophones (2 m spacing) lowered in the receiver hole Q2 (Q3). A reference hydrophone was lowered down R1 (R2). The depth of this sensor was set depending on the depth of each shot and the position of the string of hydrophones in the receiver hole. The signal from the reference hydrophone was used to detect any variability in the source onset time and will be used in the next phase of processing to analyze attenuation properties of the rock mass. Each image plane consisted of two scans. The position of the receiver cable was changed between the two scans so that the whole length of the receiver hole in the bedrock could be covered. Figure 3 shows an example of the recorded signals.

## 5. DISCUSSION OF RESULTS

The first arrival times of signals of P waves of the recorded signals were picked for all the collected data. The travel time data was first inverted using a back-projection algorithm. The crude image obtained in this way is a good first approximation of the velocity field for the next stage of processing. A Simultaneous Iterative Reconstruction Technique (SIRT) algorithm was then used to obtain the final solution. Figure 4 shows the result obtained for the plane C1-Q2.

The tomographic image obtained (Figure 4) shows P-wave velocities mainly in the range 5.5 km/s to 6.2 km/s. These are reasonable values since the image is formed by those parts of the boreholes C1 and Q2 in the bedrock (from the bottom of the casing to the bottom of each hole). Although the results presented in this paper are of a very preliminary nature, they show some

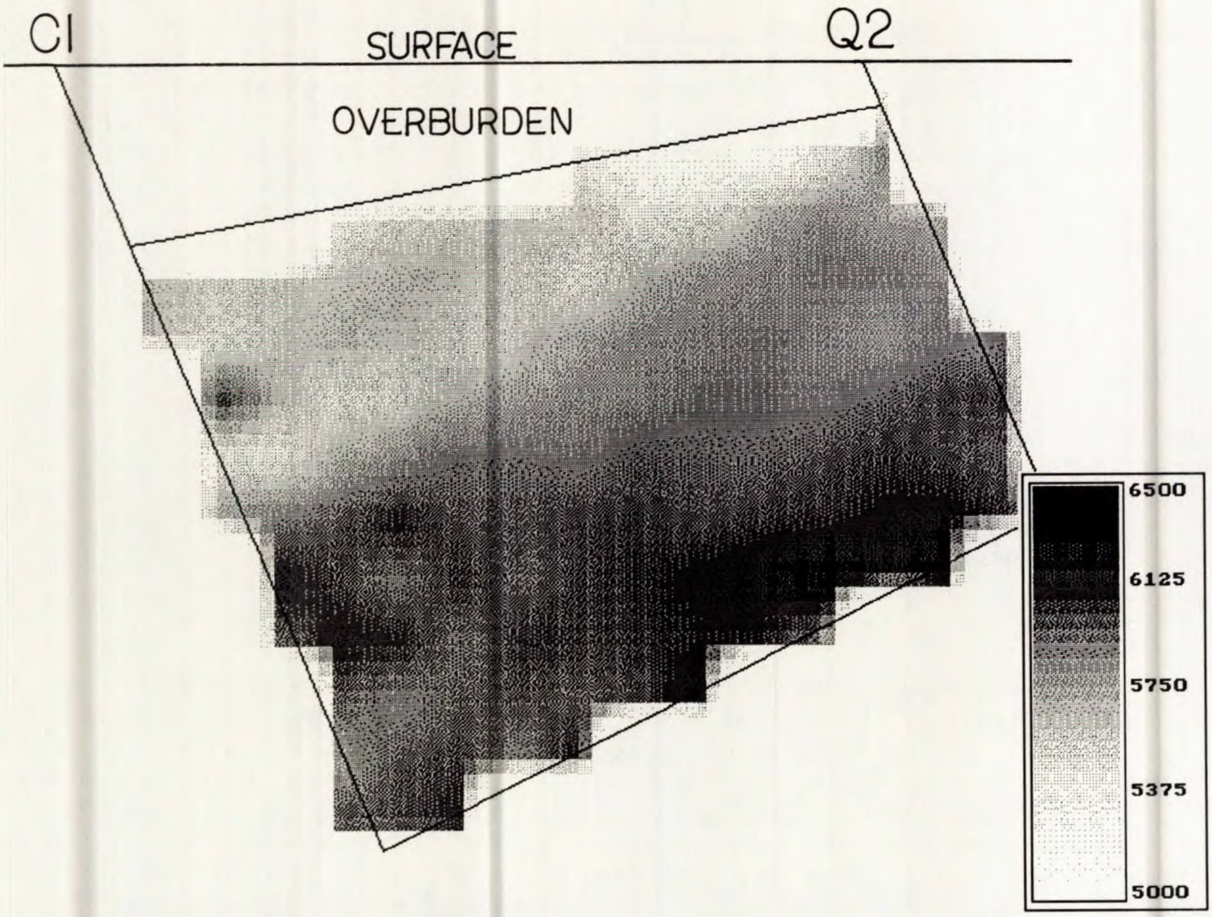


Figure 4 - Seismic tomographic image of P wave velocity obtained for the plane C1-Q2. The velocities are in m/s (see the text for more details).

interesting features. A very general trend observed is that lower velocities are located at the top part of the image plane followed by an increase in velocity with depth. This low velocity area corresponds to a zone of poor rock quality/fractured rock inside the image plane. The presence of this zone is a clear indication of the gradual action of the weathering process at the top part of the bedrock. This trend is, otherwise, compatible with the RQD values obtained during the drilling period of these holes. Indeed, the average RQD value for the 10 m length corresponding to the top part of the image was 75 and 88 respectively for C1 and Q2 while this average was 81 and 91 for the bottom part of these holes. The RQD data is, therefore, in agreement with the seismic results concerning the deterioration of the rock quality below the overburden-bedrock interface. A more detailed interpretation of velocity changes in relation to the natural joint/fault planes in the rock mass could be envisaged at a more advanced stage of the processing when several image planes could be examined.

## 6. CONCLUSION

Seismic tomographic imaging provides a non-destructive method of remote assessment of how the quality of rock changes between boreholes/surface exposures. The possibility of mapping areas of poor rock quality/fractured rock in a surface crown pillar has a large potential as a tool in the process of optimum design for these structures. The technique was used to image P-wave velocity between two inclined boreholes drilled from surface in a hard rock mine. Preliminary results obtained at this stage show a low velocity area below the contact overburden-bedrock while higher velocities are observed deeper. These results are compatible with RQD data from the same holes and seem to indicate the deterioration of the rock quality below the overburden-bedrock contact due to the weathering process.

## ACKNOWLEDGEMENTS

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# GEOPHYSICAL DELINEATION OF A SURFACE CROWN PILLAR AT THE HOYLE POND MINE

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## ABSTRACT

The Hoyle Pond deposit is partially covered by the Kidd Creek Metallurgical Site's tailings disposal area, which limited access for surface diamond drilling. Data from limited diamond drilling indicated that up to 50 meters of overburden overlay a variable thickness of regolith. Mining is predominantly conducted by cut and fill methods, and a 25 meter crown pillar has been left to protect the underground workings. In 1984 a seismic refraction survey was conducted within the water filled tailings ponds to determine the depth to bedrock. To verify the thickness of the crown pillar, a ground penetrating radar survey was carried out in 1987 to delineate the contact between competent bedrock and the regolith, and additionally to examine the contact between the regolith and the overburden. A series of blast vibration monitoring tests was conducted to evaluate the effects of underground blasting on the stability of the tailings dikes, as there was environmental concern should the dikes fail. This paper describes individual monitoring procedures for the above mentioned tests, together with the obtained results.

## RÉSUMÉ

Le gisement Hoyle Pond est en parti recouvert par le parc à résidus du Site métallurgique Kidd Creek limitant l'accès pour le forage au diamant. Des informations obtenues à partir d'un nombre limité de forages au diamant ont révélé jusqu'à 50 m de mort-terrain recouvrant une épaisseur variable de régolite. L'exploitation se fait principalement par la coupe et remblai; un pilier de surface de 25 m a été laissé en place pour protéger les ouvertures souterraines. En 1984, un sondage séismique réfractaire au sein du parc à résidus saturé d'eau a déterminé la profondeur du socle rocheux. En 1987, l'épaisseur du pilier de surface, et la localisation des contacts régolite-mort-terrain et régolite-roc sain ont été obtenus par sondages de radar pénétrant. Une série de tests de suivi de vibration fut entamée pour évaluer les effets de sautages souterrains sur la stabilité de la digue de retention de résidus, également une question environnementale. Cette étude décrit chaque procédure de suivi mentionnée de même que leurs résultats.

## INTRODUCTION

The Hoyle Pond mine is located approximately 20 km east of the city center of Timmins, Ontario, and it is 3 km west of Falconbridge Ltd's Kidd Creek Metallurgical facilities.

The gold bearing ore occurs as narrow quartz-carbonate veins filling structured zones within a pillowed basaltic host. The ore veins generally strike east-north-east within the basaltic unit and dip steeply to the south. Schistosity in the volcanic host adjacent to the veins dips moderately to the north. The main "16" vein is up to 3 meters wide, boudinaged in places, with a strike length of over 700 meters. Secondary veins striking sub-parallel to the main vein have variable dips and widths. Three near vertical diabase dykes up to 30 meters wide cut across the orebody. The vein systems are locally overlain by up to 50 meters of glacial till and varved clays, and a major portion of the orebody lies beneath the tailings disposal area of the Kidd Creek Metallurgical complex. The location of the ore zones (Figure 1) with respect to the tailings made a complete investigation of the upper orebody by surface diamond drilling difficult.

Production currently runs at an average of 400 tpd, based on a 5 day week. The ore is hauled to surface up a 17% decline by a 20 tonne electric truck. The ore zones (Figure 2) are mined through a combination of primary lateral development, mechanized narrow vein cut and fill stoping with ramp access, captive mechanized cut and fill stoping with raise access, and conventional narrow vein cut and fill stopes mucked by slusher. The ore and waste are handled by a fleet of scooptrams in mechanized areas, ranging in size from a Wagner ST5 for main level haulage, down to 1/2 cubic yard scoops in captive stopes. Stope widths range from 1.5 m to 6 m, depending on the width of the vein. The uncemented backfill is either underground development waste, or open pit waste from the nearby Owl Creek mine brought down from surface. Fill lifts are "capped" with a 15 cm layer of 15 MPa concrete to provide a mucking floor, and to prevent fine ore from being lost in the fill. [Walter, 1989].

It was realized early on that the presence of the overlying soils and tailings would require that a stable crown pillar be designed and maintained once full production mining commenced to protect the integrity of the underground workings. It was also recognized that due to the existing surface conditions, limited information on the bedrock topography and the extent and condition of the regolith zone could be obtained by surface drilling. Water inflows from the overlying tailings disposal area were similarly difficult to predict. It was known from surface drilling that the undulating bedrock surface tended to be topographically low and extremely weathered near the main ore zone due to preferrential glacial erosion and the collection of groundwater. The effects of production mining on the stability of the main tailings dikes also caused concern, due to the weak nature of the varved clays which underlay the dams.

As a result of these concerns, a number of studies were conducted in the past 5 years to assess the quality of the rock in the vicinity of the crown pillar, to gather more information to further rationalize the crown pillar design, and to determine the allowable proximity of production blasting to surface structures.

## GEOMECHANICAL STUDIES

A geomechanical analysis of the orebody was conducted very early in the design of the mine. The five materials studied included mafic volcanics, ore (quartz veins), diabase, regolith contact zone, and the regolith.

Physical Properties

Values of the uniaxial compressive strength, Young's modulus, Poisson's ratio, indirect tensile strength, and specific gravities were obtained from samples of AX and NQ diamond drill core, and the results are shown in Table 1.

Table 1: Material Properties; Hoyle Pond Mine

| Property | Mafic Volcanics | Quartz Veins | Diabase | Regolith contact | Regolith[1] |
|---|---|---|---|---|---|
| Mean Uniaxial Comp. Strenth (MPa) | 75 (26) | 138 (38) | 159 (68) | 20 (5) | 2.5 (0.7) |
| Mean Young's mod (MPa x $10^3$) | 79 (15) | 60.7(31.7) | 68.2 (23.4) | 48.3 (19.7) | --- |
| Mean Poisson's ratio | 0.21 (0.03) | 0.12(0.055) | 0.22 (0.017) | 0.17 (0.028) | --- |
| Mean Brazilian Tensile Str [2] (MPa) | 8.8 (3.3) | 13.4 [3] | 17.1 (2.5) | 6.2 (3.9) | --- |
| Specific Gravity | 2.81 | 2.61 | 3.03 | 2.64 | 2.39 |
| R. Q. D. | 85 - 98 | 70 - 80 [4] | 90 - 100 | --- | 40 (Max.) |

1 Regolith too friable to allow all tests/measurements to be made.
2 Estimated direct tensile strength can be taken as 50% of Brazilian test values.
3 Only one tensile test conducted on AX core.
4 Estimated. Core split prior to measurements.
Note: Bracketed figures indicate variance.

Rock Quality Designation (RQD)

Based on samples of AX and NQ diamond drill core, the RQD was measured for four of the five major rock materials defined within the mine. These results are also shown in Table 1. The rock mass fabric was also mapped, and the joint sets were counted. An estimate of the RQD can be obtained by counting the number of joints per cubic meter of rock, then applying the formula,

$$RQD = 115 - 3.3\,Jv, \qquad (1)$$

where Jv is the number of joints per cubic meter. [Hoek and Brown, 1980].

This method of analysis yielded an RQD of 80% for the mafic volcanics, which compared favourably with the results obtained from analysis of the core, and probably provided a more representative value for the rock mass as a whole. When the schistosity near the ore veins was included, the RQD value fell to approximately 62%. [Quesnel, 1985].

Rock Mass Classification.

The five materials in question were classified using both the CSIR and the NGI classification systems. Groundwater flows were measured in the underground exploration headings both before and after grouting. High groundwater inflow has been primarily restricted to the eastern section of the mine where the depth of overburden is greater, and hence depth of the regolith zone has increased. It was assumed for classification purposes that the post-grouting flows would be more typical of the conditions in the production areas as the practice of grouting would be continued in high inflow areas. Ungrouted areas in the ore zones had inflows of 6.5

to 7.5 L/min, whereas grouted areas had inflows of only 0.4 to 2.5 L/min.

Seven joint sets were defined in the underground mapping. The joint orientations, spacing, presence or absence of infilling, and condition are listed in Table 2. Stereonet plotting of the joint sets revealed that wedge type failures were possible from the back or walls of the development.

Table 2; Joint Sets, Hoyle Pond Mine

| Set | Strike | Dip | Separation in mm | Spacing in mm | Infilling | Comments |
|---|---|---|---|---|---|---|
| J1 | 335 | 30°E | <1 | 200 | No | Tight, clean, smooth, minor undulations |
| J2 | 335 | 85°W | <1 | 150 | No | As above, conjugate to J1 |
| J3 | 065 | 15°S | 1 | 200 | No | Tight, smooth |
| J4 | 325 | 67°E | 1 | 500 | No | Very smooth |
| J5 | 070 | 35°N | 2 | 550 | Yes | Qtz-carbonate infilling |
| J6 | 300 | 75°W | 1to10 | 400 | Yes | Qtz-carbonate infilling |
| J7 | 010 | 80°E | 10 | 300 | Yes | Qtz-carbonate infilling |

The classification systems yielded the following results as shown in Table 3. The NGI system was particularly usefull, as it allowed an estimate of the maximum unsupported span to be made. Portions of the 5 meter wide decline were driven without support in the mafic volcanics, and this development has been standing open without deterioration for a period of almost 6 years, confirming the results indicated by the rock mass classifications. It should be noted however that the classification systems are only a guideline to the degree of support required, and that the potential wedge failures delineated by the joint set mapping would still require local support.

Table 3: Rock Mass Classifications; Hoyle Pond Mine

| Material | CSIR Classification Rock Mass Class | NGI Classification "Q" | Max. Allowable Unsupported Span (meters) |
|---|---|---|---|
| Mafic Volc., Ore, and Diabase | Class 11 | Avg. of 9.4 | 6.4 |
| Regolith contact zone | Class 111 | 6.1 | 5.4 |
| Regolith | Class V | 0.03 | 0.6 |

Crown Pillar

Based on information gathered at a Crown Pillar seminar in 1984, the rock mass classifications performed at the site, the mining method (including the practice of filling each lift, restricting the amount of open ground within the mine) and the general response of the ground during exploration development, a 25 meter thick crown pillar in competent bedrock was initially chosen. The most practical control to date to maintain the thickness of the crown pillar is provided by diamond drilling upwards from stope development to delineate the regolith/bedrock contact in any mining zones approaching the crown pillar.

SEISMIC REFRACTION SURVEY

A seismic refraction survey [Hardy Assoc., 1984] was conducted to determine the depth to bedrock over the planned mining zones underlying the tailings ponds. In addition an estimate was to be made of the thickness of the regolith. The locations of the five seismic lines are shown in Figure 1. Two seismic spreads were used on each line. Each spread consisted of twelve hydrophones placed within the tailings ponds, spaced at 16.8 m (55 ft) intervals. Blasts were detonated at each end of both spreads. The shots consisted of approximately 250 grams of explosive charge detonated by an electric cap.

The seismic data on the first spread of Line 1 were recorded with an OYO McSeis 1500 seismic unit. The rest of the data were recorded with a Nimbus ES 1200 seismic recorder. Five time-distance plots were produced using seismic arrival times from both spreads on each line. The data was interpreted using a three layer model with dipping interfaces. A typical plot with available diamond drill hole information is shown in Figure 3. The dips which were calculated were less than $5^{o}$, except for Line 4 where the dip was $12^{o}$. Standard formulae were used for the calculations.

The first wave arrivals which travelled through the upper layer were very distinct, having a long wave train with very high frequencies. The arrivals from the other two layers had much lower frequencies and generally lower amplitudes. On some traces no arrival from the lower layers were observed, even for large shot offsets. The upper layer had a mean seismic velocity value of 1.5 km/sec (4,920 ft/sec). The thickness of this layer ranged from 6.5 m to 25 m. The values for velocity and thickness were consistent with the soft clay layer observed in the geotechnical drill holes. The seismic velocity in the second layer averaged 2.35 km/sec (7,700 ft/sec), increasing with depth. The thickness of this layer ranged from 22 m to 43 m. This layer was interpreted to be the till layer observed at the bottom of the geotechnical holes. The seismic velocity in the third layer was 3.5 to 4.0 km/sec (11,400 to 13,300 ft/sec). No significant increase in velocity with depth was observed in this layer. This layer could be indicative of the regolith observed in the geotechnical logs.

Comments on the Seismic Survey

1. The near surface layer of tailings and pre-existing organic material absorbed much of the seismic energy so that first arrivals were not always clear-cut.
2. The second layer was a transition layer whose seismic velocity increased with depth. Consequently, the interpreted depth to bedrock could vary by 10 %.
3. The depths to the third layer (bedrock) found by the seismic method did not correspond to the top of the regolith as indicated by diamond drilling. Rather, these appear to be the depths within the regolith at which there is only local weathering within predominantly unweathered rock.
4. The cause of the unusually high bedrock velocity (5.5 km/sec) on Line 3 cannot be identified, but it was probably due to propagation through competent bedrock.

## GROUND PENETRATING RADAR SURVEY

A pulseEKKO III radar survey [A Cubed Inc., 1987] was conducted in the mine from underground to determine if the technique was capable of detecting and mapping the lower regolith contact, and also the interface between the regolith and the overburden.

The surveys were carried out along the back of three separate drifts as shown in Figure 2. There was pipe, cable, and steel ventilation ducting in one drift. Water was dripping heavily from the back in another drift. These features and conditions were typical of an operating mine.

Ground penetrating radar (GPR) is similar in principle to seismic reflection and sonar techniques. The radar unit transmits a short burst of high frequency electromagnetic energy into the rock from the antenna which is mounted against the drift back. The reflected signals are detected and amplified at the receiver, digitized, and stored on a digital magnetic cassette tape for post survey processing.

The propagation of the radar signal depends on the electrical properties of the ground, which are in turn primarily controlled by the water content of the ground. Electromagnetic energy travels in the rock and part of the signal is reflected whenever it encounters a change in electrical properties, such as a change of rock type or a fracture in the rock. Radar reflections are also produced from interfaces such as the water table, or stratigraphic units within the overburden and bedrock.

The radar data was displayed as a section, with antenna position on the horizontal axis, and the two-way signal travel time to reflectors on the vertical axis. If the radar signal propagation velocity in the ground is known, then the vertical axis can be converted to a depth or range scale in the ground.

Antenna selection controls the pulseEKKO operating frequency, which in turn defines the signal penetration range and resolution in the ground. In general, the lower the frequency (10 to 100 MHz) the greater the sounding range of the radar, and the higher the frequency (200 to 1000 MHz) the greater the capability for the radar to detect thinner features such as fractures in the rock. The 50 MHz antennas were used for all of the surveys carried out in the mine.

A radar signal can propagate up to 50 m in rock with high electrical resistivity, like granite, and thin fracture zones of about 0.1 m can be resolved. The radar signal range and resolution decrease as the electrical resistivity of the ground decreases. [Holloway, et . al., 1986].

In the interpretation of GPR data, the radar signal velocity in the surveyed ground should be known. This velocity can be determined using a common mid point velocity (CMP) sounding, in the analogy of seismic wave velocity measurements, by varying the antenna spacing about a fixed location and measuring the change of two-way travel time to the reflectors. The radar signal velocity was determined to be approximately 0.12 m/ns in the rock at all locations, and it may be about 0.06 m/ns in the overburden.

Test in the 2-115 Drift

The objective of the survey was to delineate the contact between the crown pillar and the regolith. The survey covered 50 m along the back of the drift. This area was very wet with the height of the back ranging from 2 to 3 m. The spacing between the radar traces was 0.75 m.

The radar data obtained in this drift (Figure 4), show dipping reflectors 20 m in the back at the 20 m station along the profile, and 15 m in the back at the 35 m station along the profile. These reflectors occur along the survey line in contact with the regolith. The reflectors may be fractures or saturated zones in the regolith. It was noted that the reflections were strongest in the wettest areas. With high gain, the radar revealed a dipping reflector at a range of 35 m between stations 25 and 30, which may be the interface between the regolith and the overburden.

Test in the 2-416 Drift

The drift was dry with a back height of about 4 m, and it was clear of pipes and cables. Generally conditions were good for pulseEKKO sounding. The objective of the survey was to determine if the system would detect the bedrock to regolith contact located at a range of 30 m above the back.

The radar data show reflectors to a range of only 18 m above the back. It appears that the radar did not detect variations caused by the regolith. The reflections could be from severe fractures, or variations in the rock to such an extent that most of the energy was reflected at the frequency used.

Test in the 1-14 Drift

Originally the objective was to profile through a diabase dyke to see the response of the radar. Due to obstructions such as ventilation ducting, pipes and blasting cables, the survey was relocated to a place where a common mid-point velocity (CMP) sounding in addition to a profile survey could be carried out to obtain the radar signal velocity in the rock.

The radar data as shown in Figure 5 indicate reflectors to a range of 40 m above the back. The pronounced reflection at about 32 m was most likely from the bedrock/regolith interface. The CMP data in Figure 6 show that there were a number of spurious reflections within a range of 32 m.

Comments on the GPR Survey

1. For an operating frequency of 50 MHz, the radar detected reflectors to a range of 40 m in the rock. Reflections did occur from the bedrock/regolith interface at two sites. The interpreted results seemed to be in agreement with the drill data. The system, however, did not show any reflections beyond 20 m at a dry test site, possibly due to severe fractures in the rock, yet revealed reflectors at a range of 35 m in wet ground.
2. The system can be used practically in the mine environment. Some modifications to the antenna support unit would make it more effective.

## BLAST VIBRATION STUDIES

The majority of the underground workings of the Hoyle Pond mine are located beneath a tailings pond. Gravel dikes retain the tailings and provide a perimeter access road. There was great concern about the stability of the dikes, which were founded on weak clays, when they were subjected to the vibration effects of underground production blasts.

A blast monitoring program [Yu, 1984] was set up to measure the vibration level in connection with a liquefaction study. A tri-axial seismometer and accelerometers were used to measure the particle accelerations and velocities for drift round blasts having an average of 5.5 kg of explosive in each hole.

By taking the vector sum of three orthogonal components, the apparent peak particle acceleration and velocity were found to be 0.09 g and 0.076 cm/sec respectively. By using the incremental approach, the vibration level for a given charge weight at a certain distance can be inferred from the following equations:

$$A_b = A_a\,(W_b/W_a)^{0.33}\,(R_a/R_b)^2\,(C_b/C_a)^2 \tag{2}$$

$$V_b = V_a\,(W_b/W_a)^{0.6}\,(R_a/R_b)^{1.8}\,(C_b/C_a) \tag{3}$$

where A = peak particle acceleration
V = peak particle velocity
W = charge weight per delay
R = distance from blasting source to the detector
C = p-wave velocity in the medium

Based on the measured values of particle velocity and acceleration, it was believed that the effect of blasting vibration from the drift round blasts on the stability of dikes was minimal. The dikes have remained intact during the course of underground production blasting over the past 5 years.

## CONCLUSIONS

The 25 meter deep crown pillar originally designed for the Hoyle Pond Mine provided a margin of safety in areas where unknown changes in the overburden/regolith topography could result in failure of the crown pillar from beneath the tailings disposal area, and it also ensured that unreasonable water inflows would not be encountered in the underground development.

In general the correlation between the seismic survey and the overburden drilling results was not consistent, therefore the seismic results were used for reference only.

The G. P. R. survey has shown the potential application of the system for delineating crown pillars up to 40 meters thick from underground, using an operating frequency of 50 MHz. Refinements to the data processing system as well as to interpretation techniques are required to make more practical use of the system.

Blasting effects on surface facilities, including liquifaction of the overlying clay stratum resulting in failure of the tailings dams, was proven not to be a major concern. Vibration monitoring was considered to be the only way to obtain the necessary data, particularly in regions of complex stratigraphy.

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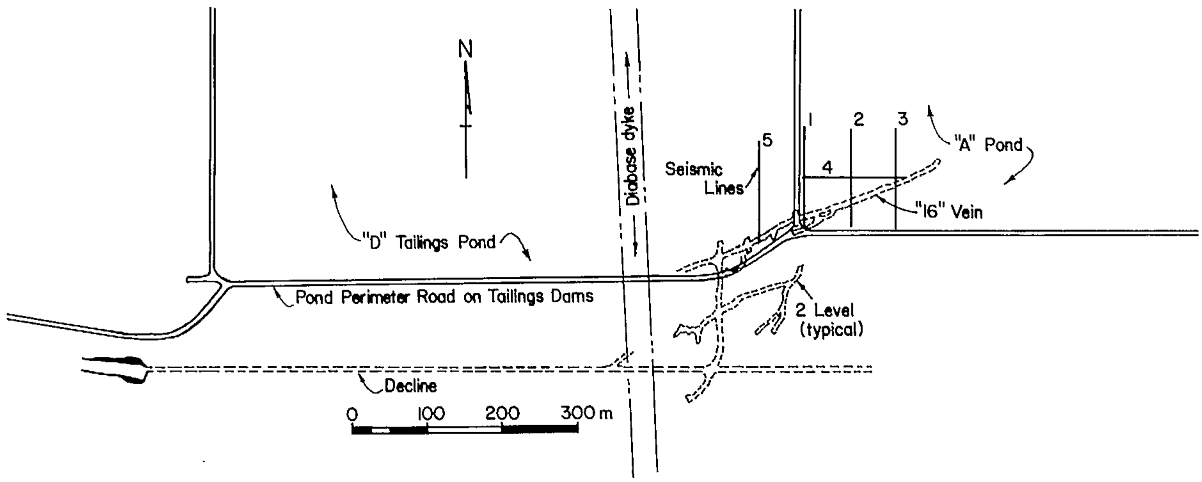



Figure 1. Plan of Hoyle Pond Mine (Decline and a portion of 2 Level), Tailings Ponds, and Seismic Lines

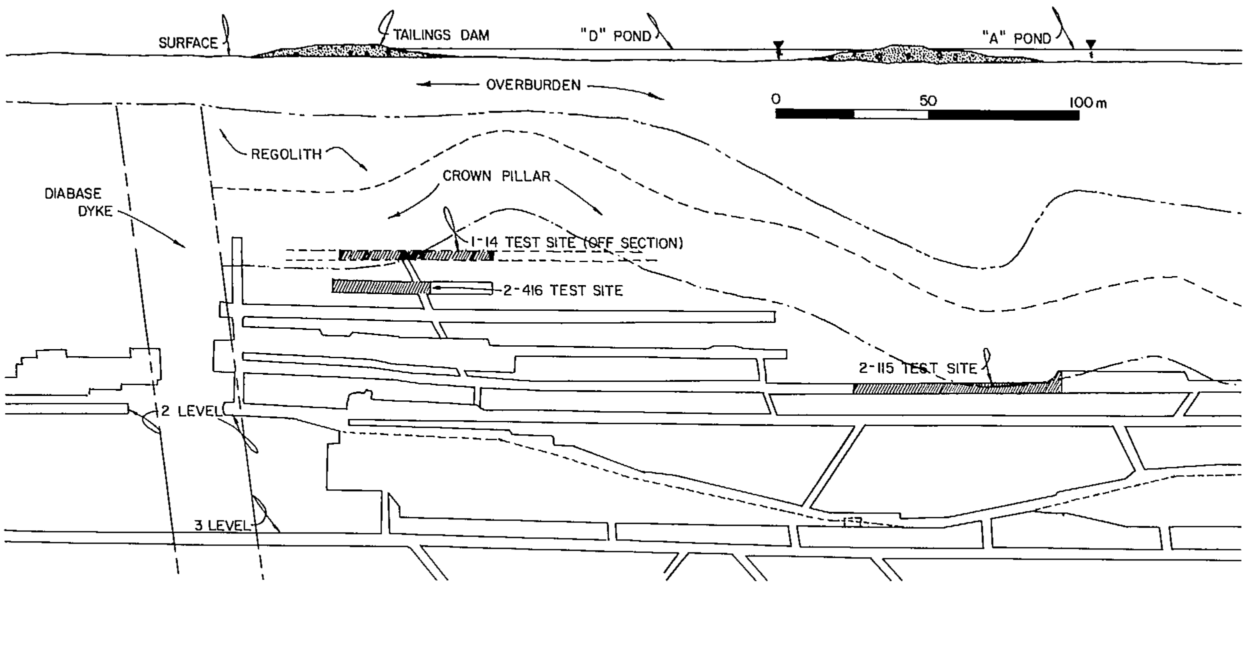
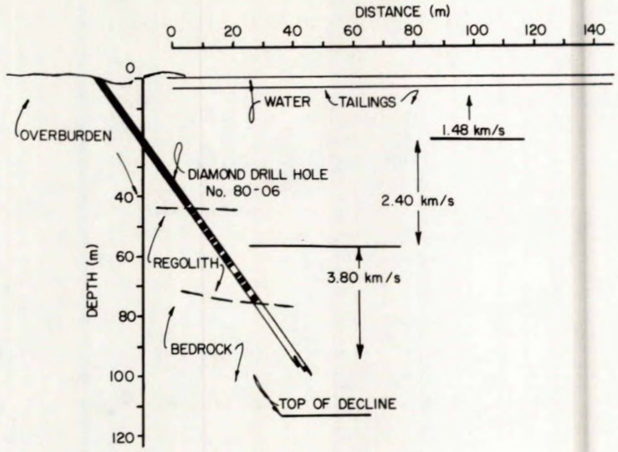



Figure 2. Section looking Northwest in the plane of "16" vein, showing the tailings dams, overburden, regolith, crown pillar, mine workings, and pulseEKKO III test locations

Figure 3.

Typical Seismic Survey results vs. Diamond Drilling



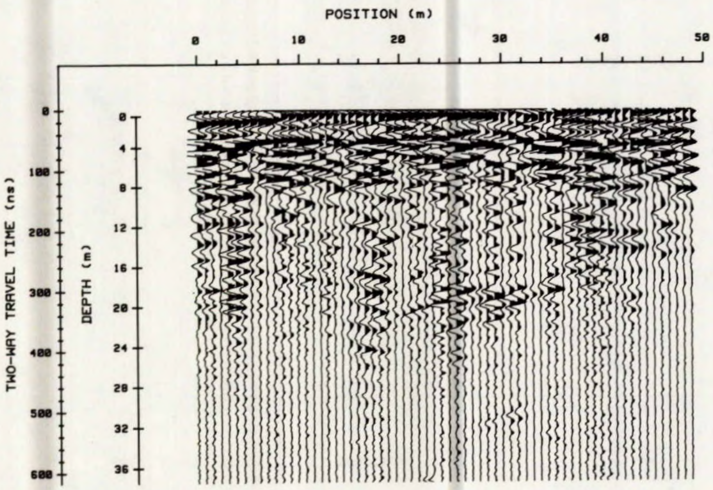




Figure 4. pulseEKKO III data; 2-115 Test Site

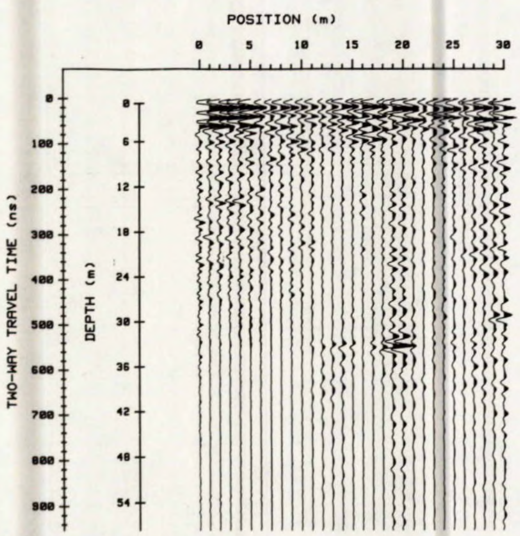



Figure 5. pulseEKKO III data; 1-14 Test Site

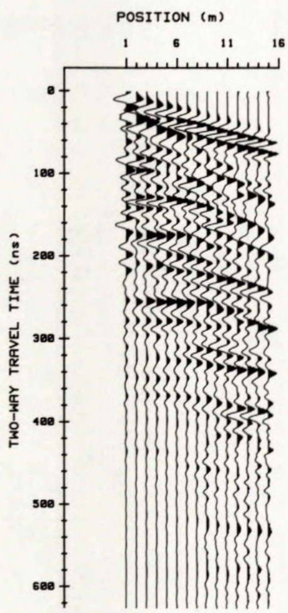



Figure 6. pulseEKKO III data; CMP survey, 1-14 Test Site

# DESIGN METHODS AND CONSEQUENCES

## SESSION II

# MÉTHODES DE CONCEPTION ET LEURS CONSÉQUENCES

# MECHANISMS OF CAVING AND CROWN PILLAR STABILITY

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## ABSTRACT

The authors review how rock, groundwater, stress and temperature conditions vary with depth, and how these changes affect the type of failure, and the mechanisms of caving in particular. A satisfactory predictive model can be obtained most easily by combining observations in active open stope mines, in abandoned mines and natural caves, and in stopes in longwall and block caving operations.

Among alternatives for predicting the initiation and progress of caving, empirical methods based on "cavability" are the most promising. Classifications are in their infancy, and reliable predictions will require an improved and coordinated approach.

The VP-Expert development tool provides an excellent framework for rock mass classification, making it simpler and less subjective. The authors have developed an expert system that classifies ground according RQD, RMR, RSR and Q methods and predicts various parameters of stability and support. As input, new methods are available for measuring near-vertical jointing patterns from air and satellite photographs, and for quantifying parameters such as joint orientation, RQD, spacing and roughness. These techniques of photoanalysis are outlined in the paper.

## RÉSUMÉ

Les changements en profondeur des conditions du roc, des eaux souterraines, des contraintes et de température sont examinés pour établir l'effet sur le type de rupture et le mécanisme du foudroyage. Un modèle de prédiction est obtenu en combinant des observations faites dans les ouvertures souterraines minières, et les cavernes. Les méthodes empiriques reliées au foudroyage sont les plus prometteuses au niveau de la prédiction du début et de la progression du foudroyage. Les prédictions par classifications (encore jeunes) nécessiteraient une approche améliorée et coordonnée. La structure du système expert VP-Expert permet une classification du massif rocheux plus simple et moins subjective. Il classifie les terrains selon les méthodes RQD, RMR, RSR et Q et prédit divers paramètres de stabilité et de soutènement. Des nouvelles méthodes d'analyses de photos aériennes et photo satellites fournissent des renseignements sur les systèmes de joints sub-verticaux et permettent de quantifier des paramètres de joints : espacement, RQD, rugosité. Ces techniques sont décrites dans cette étude.

## HOW MINING CONDITIONS VARY WITH DEPTH

Surface crown pillar instability is governed by conditions quite different from those that the miner meets later in the life of the mine. As mining goes deeper, changes occur firstly in the rock, secondly in the groundwater, and thirdly in the ground stresses and temperatures.

Rock quality improves with depth. Weathering lessens, the joints become more widely spaced, tighter and less weathered, and the rock becomes less decomposed, less porous, and hence stronger.

The soil/rock interface can be the focus of problems. Often the rock surface is scarred by deep channels of glacial erosion, backfilled with boulders and water. Channels are common above fault-emplaced orebodies. The top of rock above a stope may be shallower than expected, and rupture may lead to substantial inflows of water and soil.

If the crown pillar remains stable, two opposing trends control the influx of water into the stope. On the one hand, groundwater pressures tend to increase in proportion to depth below the water table. On the other, permeability decreases because the joints are further apart and tighter. The net result is that inflows are usually greatest into near-surface stopes. Deep mines are often dry in spite of the much greater water pressures that exist.

The vertical component of stress increases as a linear function of the weight of overburden. Much less predictable are the horizontal components, which at shallow depths can be ten or more times the vertical. These high stresses encourage arching and stability of the crown pillar.

Geothermal gradient results in a ground temperature increase of about 1°C per 40 m of depth. Mines even at the modest depth of 1 km operate in rock at a temperature of at least 45°C, which for softer rocks such as potash leads to a more ductile behaviour.

Finally, a depth factor little related to geomechanics. In early stages of mining at shallower levels, mine management aims to maximize extraction and return on invested capital. Pressures on production ease as the mine goes deeper, while experience and confidence are gained in rock conditions and mining methods. This means that from the crown pillar stability point of view, the mines with the greatest experience in rock mechanics are often those least in need. As a corollary, the greatest risks are often taken when the consequences of instability are the most severe.

## ROCK FAILURE MECHANISMS

Underground rock behaviour can be broadly classified into one of the five following categories:

Stable conditions in which the rock has reached a state of equilibrium. Rock displacements continue at decreasing rates and without damage to supporting systems.

Gravitational failures in which the crown and sometimes also the walls are progressively collapsing by ravelling or caving, i.e. the loosening and fall of blocks. The blocks become detached by sliding along their bounding joints without much distortion or rupture of the rock.

Squeezing ground conditions, in which the crown, sidewalls and sometimes the invert of the excavation converge slowly and continuously by mechanisms of stress-induced viscoplastic flow. There is substantial distortion of intact rock material, often accompanied by creep along joints within the zone of overstressed rock mass.

Swelling ground conditions in which the rocks exposed near the excavation walls expand by physico-chemical mechanisms associated with the adsorption of water by clay minerals or anhydrite. Note that swelling is the result of mineralogical changes, whereas squeezing is caused by overstressing.

Bursting ground conditions, in which the rock fails explosively by propagation of fractures through intact rock. Stored energy is released suddenly and violently. Included as a subcategory are gas outbursts, which occur as the violent ejection of gasses and rock into an underground opening.

Evidently, surface crown pillar failures fall into the second, gravitational category in which collapse is governed by blocky ravelling and sliding mechanisms with little or no rupture of intact rock material.

## CAVING MECHANISMS

### Stability and Caving

Surface crown pillar collapses in active mines number in the tens only. The scarcity of events, although fortunate from most points of view, leaves a scarcity of data on the mechanisms and hinders the development of preventive measures. Empirical design methods require many observations to be reliable, and empirical design tools are still the most practical and useful in most day-to-day mining work.

The problem of too few data can be overcome by recognising that surface crown pillar collapse is just one category of a fourfold and more general class of rock failure mechanism, which includes the following well-documented cases:

Surface crown pillar failures in active mines (e.g. Hatheway, 1968; Nicholson, 1985);

Subsidence and collapse above abandoned mine workings, particularly near-surface room and pillar mines (e.g. Hill, 1981; O'Riordan and Henkel, 1984; Aughenbaugh and Elifrits, 1986);

Collapse of natural shallow karstic caves in limestone, to form sinkholes (Jennings et al., 1965);

Caving of mine stopes in longwall and block caving operations.

Chimney caving, piping, sinkhole or crownhole formation, are different words for much the same phenomenon. Stability is essentially the inverse of cavability, so the empirical methods used to predict cavability may be applied with little modification to the design of crown pillars. Caving mining methods and cavability are discussed further by Kendorski (1978), Ferguson (1979), and Brady and Brown (1985) and in a report for the United States Department of the Interior by the Dravo Corporation (1974). Two volumes edited by Stewart (1981) and Hustrulid (1982) also contain numerous papers on this topic.

### Types of Caving

Ravelling subsidence starts with the fretting of small pieces from the stope roof which if untreated (for example by shotcrete) proceeds to chimneying, then to a more generalized rock movement and dilation that migrates upward through the overlying material. Caving is naturally arrested either when it meets a more massive horizon, or when the stope becomes full of bulked broken rock. Otherwise it reaches surface, forming a subsidence crater or "sinkhole".

A second, more sudden, mechanism, called plug subsidence, is characterized by instability of a large block bounded by near-vertical major structural features with low shear strength. Sudden collapse of a plug appears to be less common than progressive ravelling, although ravelling may well undermine and destabilize a plug in its later stages.

### Variables that determine arching and caving

Stability and caving of the arch of rock blocks above a near-surface stope depend on factors such as the following:

Properties of the rock:

Size of rock block in relation to the dimensions of the stope. Ravelling is most likely with small blocks and a large stope;

Strength of the block boundaries, i.e. the shear strength of the joints, which depends on roughness and on the strength of joint walls or filling materials. Ravelling is most likely with smooth, clay-coated joints;

Orientation of jointing. Vertical or steeply dipping joints permit caving, but flat-lying joints are also needed (Mahtab and Dixon, 1976; Tobie and Julin, 1982).

Stress conditions:

Magnitude of virgin horizontal stress. High horizontal stresses encourage arching across openings of large span (Krustulovic, 1979), but reduced levels of stress are likely in weaker and more sheared rock masses;

Augmentation of horizontal stress in the stope crown, as a result of joint dilatation during movement. Local crown stresses may reach three times the initial virgin stress value, but this depends on the rock mass deformability, which in turn depends on joint spacing, roughness, and the compressibility of filling materials. A closely jointed, flexible rock mass is more likely to ravel and cave than a stiff one.

Groundwater conditions:

Water pressures, which usually are low around shallow excavations. However, the resultant downward force on the crown pillar, particularly if stratified, can be high. Cracking of upper beds as a result of subsidence can throw sudden water pressures on bedded roof strata sufficient to precipitate collapse;

Hydraulic gradients and hence seepage velocities in the crown pillar, which can lead to internal erosion particularly if the rock is closely jointed and contains infilled or coated joints. As ravelling progresses upwards, the hydraulic gradient increases and erosion can play an active part in the later stages of accelerated collapse.

Geometry of the stope:

The initiation of ravelling seems to depend on the narrowest dimension of the stope and not much on its length or height. Kendrick (1970) found that the height of stable arching at the Urad mine was one-half the width of the arch and that once a stable arch was formed, the maximum dimension could be extended considerably without causing the ore to cave;

The maximum height of chimneying depends on the volume and shape of stope and the bulking factor (Kendorski et al., 1979). When deep stopes ravel and cave they fill with bulked broken rock before the chimney breaks to surface. Chimney heights can be more than ten times the original stope height but are usually 3 to 5 times the initial height. Kendorski et al. suggest equations for the maximum height of collapse for different shapes of stope.

Note the importance of jointing, which governs not only stability directly, but also the deformation of the rock mass and hence the redistribution of stresses during mining. Jointing and rock stresses in turn control the permeability of the rock mass and the water pressures and inflows.

## PREDICTIONS OF CAVING

### Numerical studies

Design is the process of making decisions on locations, alignments, sizes, shapes and sequences of mined excavations, predicting their stability, and deciding on their stabilization and support systems.

Numerical models help to examine strategies at the start of a mining operation, and in this application have become much more realistic in the last two decades. Block theory (Goodman and Shi, 1985) is one of the more promising developments. When caving is required for mining, block theory can help in the choice of undercuts to maximize the chance of caving. Inversely, by identifying key blocks and ensuring that these remain in place, progressive collapse of the excavation is arrested.

### Designs based on "Cavability"

In contrast, empirical methods allow the engineer to proceed directly from a ground quality assessment to the design of rock excavations and support systems. They are a scientific extension of "rule of thumb" methods based on experience, and are best applied in day-to-day "tactical" decision-making. Of particular interest are "expert systems" which allow decision-making based on a combination of judgement, numerical modeling and empirical design.

Empirical predictions of caving include those of Karfakis (1986) for estimating the likelihood of chimney occurrence over the Hanna Coal Field, Wyoming. A log-normal relationship was found between frequency of occurrence and overburden depth. Similarly, Goel and Page (1982) used draw density and geometrical parameters to predict the probability of chimney cave occurrence. Matheson (1986) investigated 3,000 chimney features in the Colorado Springs and Boulder Coal Fields and developed relationships describing the rate of sinkhole development and the dependence of locations on mined height, overburden thickness, and type of mining. Panek and Melvin (1987) are developing quantitative methods to predict the critical span to induce caving.

To date, the caving mine rock mass and support estimation technique developed by Cummings et al. (1984) is the most comprehensive. They developed a rock mass classification called the "Modified Basic RMR" (MBR), specifically to estimate support requirements in drifts in block caving mines, taking into account the special geological conditions inherent in these mines. The method could be adapted to deal with the specific concerns of crown pillar stability.

Sheory (1984) used a rock mass classification to estimate the caving span in oblong workings in coal, and Ghose and Duta (1987) developed a rock mass classification model for caving longwall roof rocks based on fuzzy set methodology, with a decision rating for cavability.

### Good and Bad Empirical Design

Judgement and experience are major ingredients in rock engineering design. However, our ability to judge and forecast is always limited by the extent and quality of our experience. "Empirical design" solves the problem of our own limited experience by making available the accumulated experience of others. It requires three steps beyond simple judgement:

> Quantification of "ground quality" by a classification system, to provide a universal language whereby the experience gained globally working in ground of many different qualities can be related to future projects;
>
> Quantification of "ground performance" by a formalized system which defines such parameters as unsupported stand time and maximum span, rate of upward chimneying, angle of draw;
>
> Correlation of ground quality to performance by compilation and comparison of results from many projects over a full spectrum of ground conditions (case histories).

The quality of predictions obtained from a given empirical design depends directly on each of the above three steps. The ground classification must be appropriate and efficient, as must be the description of ground performance. The correlation of one with the other must be based on a broad range of observations.

At this very early stage in the development of empirical design procedures, the existing ground classifications are deficient in various ways. Nevertheless, the empirical approach is simple and useful, and improves as we gain a better understanding of ground characterization, and as new and better methods are developed for measuring the key classification parameters.

Ground classification depends on properties of the rock and of the joints. Devices such as the point load tester were developed to simplify measurement of intact rock characteristics. More recently, our group have been developing improved methods to simplify the measurement of rock jointing. These and their application to crown pillar stability assessment will now be outlined.

## MEASUREMENTS OF JOINTING

Techniques of photoanalysis have been developed that measure the geometry of rock jointing from digitized photographs or video tapes. The method is simpler, quicker, less expensive, and often safer than direct measurements using a tape and geological compass (Franklin, Maerz & Bennett, 1989).

An image enhancement program "WIEP",converts the image of the rock mass into a network of lines that represent the joints (Fig. 1a). Measurements are then be made of the geometry of this network, which can be extrapolated to infer three-dimensional structures.

Orientations of joints are measured and plotted as rosettes in two dimensions (Fig. 1b), using two or more non-parallel photographs to infer three dimensional orientations from a "rosette fusing" process. Joint spacings have been measured and automatically plotted in rosette form (Fig. 1c), using a rotating line-intercept procedure to determine spacings as a function of direction in the plane of the photograph.

We have used air photographs to map spacings and orientation of jointing in Precambrian rocks in Sudbury, and in sedimentary rocks in Prince Edward County. Regional trends in jointing and faulting can be identified. Similar data were compiled by Campbell et al. (1978) and McDonough (1977) who devised a competence map based on lineament analysis using aerial photography (without digital photoanalysis). The map was used to predict roof conditions prior to mining.

Photoanalysis of close-ups of jointed rock, taken in mine development shafts and drifts, allows quick and unbiased measurements of rock mass quality for use in classification and empirical design. Orientations and spacings can be measured, and also related properties such as RQD, Volumetric Joint Count, Block Size, etc. A technique based on digitizing the shadow cast by a straightedge onto the rock surface allows estimates of roughness and thus shear strength.

Development of the techniques has benefitted from field trials in Noranda's and Inco's underground mines. The work has been supported by CANMET, Noranda Mining Research Centre, Explosives Technologies International Inc., NSERC, Ontario Hydro, and the Ontario Ministry of Natural Resources.

## EXPERT SYSTEMS AND ROCK CLASSIFICATION

Expert systems show considerable promise in mining applications (Bandoadhyay and Venkatasubramanian, 1988). They can provide a self-contained system for classification and empirical design. The computer stores correlations that predict a wide variety of design and performance details. Explanations are given for each action, and automatic checks ensure that the input data are consistent. The user is prompted with supplementary reading. As he or she becomes more experienced, these reminders can be bypassed.

In our research at the University of Waterloo, we employ the VP-Expert shell and apply it to rock mass classification using the Norwegian Q and South African RSR Systems, also for measurement of simple RQD, and to obtain a rock quality rating using the "size-strength" approach (Franklin, 1986). The final product will be expanded to hold other systems currently in use (Laubscher, 1977; Cummings et al., 1984; Hoek & Brown, 1980).
Characteristics measured by photoanalysis from airphotos, surface and underground photography, provide the necessary input on jointing, as described above. Supplementary input is obtained by conventional testing, such as with a point load tester or Schmidt Hammer.

Predictions are being made of stability and cavability for purposes of empirical design. Estimates are obtained for numerical modeling, of bulk characteristics including rock mass deformability and shear strength, derived from the classifications using published relationships.

Rock mass classifications must be regarded as in a growth phase of their development. It is important to search for simplifications and improvements rather than to adhere rigidly to any one system. The next generation of classifications may well resemble much more the predictive models of the economist. With the benefit of computers and expert systems, the added complexity should not present a problem.

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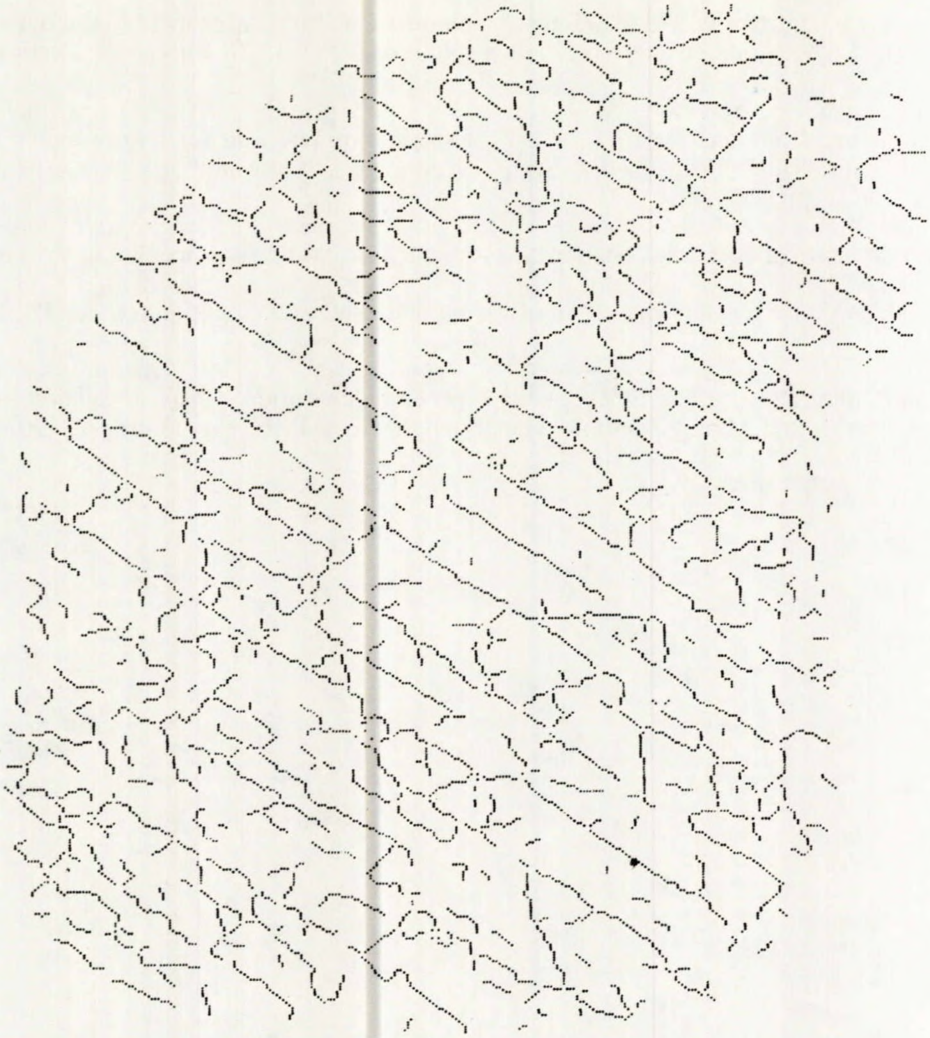
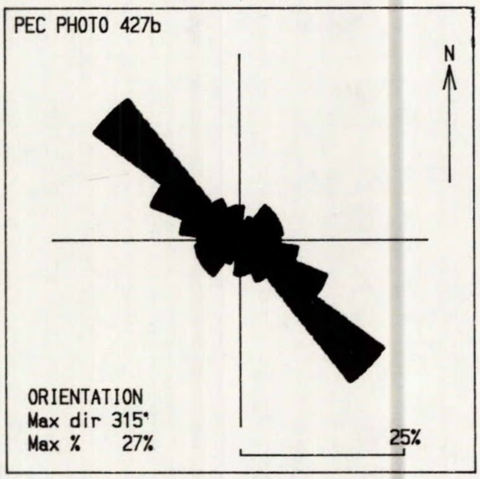
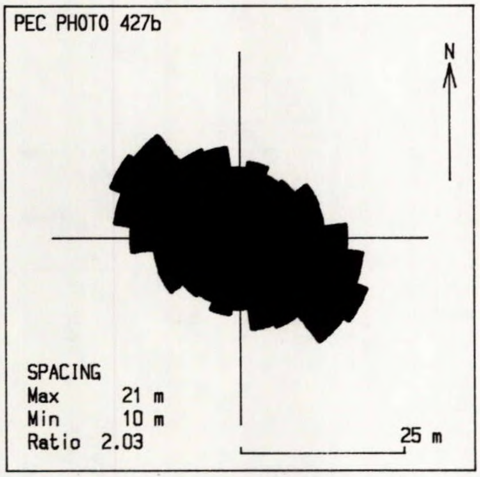


(a) Joint Network




(b) Orientation Rosette




(c) Spacing Rosette

Figure 1: Photoanalysis of Joint Network
(from air photograph, Prince Edward County, Ontario)

# APPLICATION D'UNE MÉTHODE GÉNÉRALISÉE D'ANALYSE À LA RUPTURE À L'ÉVALUATION DES PILIERS DE SURFACE

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## RÉSUMÉ

Les piliers de surface des mines de métaux exploitées dans le Bouclier canadien doivent souvent être considérés pour fins de design comme des masses dont le comportement est prédominé par des déplacements de blocs polyèdraux quasi-rigides, déplacements qui sont dépendants de la relation contrainte-déformation des discontinuités géologiques qui les délimitent. Les méthodes d'analyse alors utilisées pour évaluer s'ils sont sécuritaires doivent tenir compte de ce fait.

La communication présente une version bidimensionnelle d'une telle méthode. C'est essentiellement une généralisation des méthodes conventionnelles d'analyse à la rupture qui tire profit de la programmation linéaire pour contourner le problème d'indétermination mathématique qui caractérise toujours ces dernières. Elle consiste à séparer la portion contenant les blocs considérés comme potentiellement instables du reste du massif, à discrétiser cette portion en la découpant en éléments en suivant les discontinuités géologiques, à formuler les équations d'équilibre de chaque élément et les relations qui expriment le critère de rupture aux interfaces des éléments et, finalement, à résoudre le système en optimisant une fonction objectif. Cette dernière peut être, entre autres, un facteur de sécurité ou une force stabilisatrice . La formulation des équations menant à la solution du problème dans le cas d'éléments triangulaires est donnée à titre d'exemple; on y montre comment on y incorpore le poids propre du matériau, les pressions hydrauliques, les sollicitations externes telles les surcharges, les forces séismiques et les forces stabilisatrices. L'application de la méthode est illustrée à l'aide d'un cas très simple; les résultats obtenus sont brièvement discutés.

Diverses extensions et différentes variantes de cette méthode sont succinctement présentées afin de montrer tout le potentiel qu'elle possède.

## ABSTRACT

For design purposes, Canadian Shield metal mines surface crown pillars often have to be considered as discontinuous masses. Their behaviour is then dominated by the movement of quasi-rigid polyhedral blocks, the displacements of which depend on the stress-strain relationship of the geological discontinuities that creates them. Methods of analysis used to evaluate their stability should take this fact into account.

This paper presents a two-dimensionnal version of such a method. The latter is essentially a generalization of the conventionnal limit equilibrium methods. It takes advantage of linear programming to withdraw the mathematical indetermination that always characterizes them. The method requires that a portion of the rock mass, which includes the blocks that are considered to be potentially unstable, be isolated. This portion is then discretized into elements using the geological discontinuities. Equilibrium equations are formulated for each element and relations describing the strength along element interfaces are elaborated; this system is solved throught linear programming, optimizing an objective function which could be a safety factor or a stabilizing force . The elaboration of the various above equations is presented for the case in which the elements are triangles; it is shown how the weight of the material, the hydraulic pressures, the external loads, the seismic and stabilizing forces are incorporated into the equations. For illustration purposes, the method is applied to a very simple case; the results obtained are briefly discussed.

Some of the extensions of the method as well as some of its variants are briefly described in order to show its real potential.

## INTRODUCTION

Au Canada, l'exploitation d'un bon nombre de gisements métalliques en roche dure est pratiquée en souterrain et, souvent, jusque près de la surface du socle rocheux (Canmet, 1984; Canmet, 1985). Des couvertures de roche, appelées piliers de surface, doivent être laissées en place, temporairement ou en permanence, afin d'assurer la stabilité et l'intégrité des éléments de surface et souterrains.

De tels piliers, se situant près de la surface du massif rocheux, sont soumis à des régimes de contraintes habituellement de faible magnitude par rapport à la résistance de la roche (Herget, 1987). Ajoutant la présence de réseaux de discontinuités géologiques bien développés à cette profondeur, on suggère, dans bien des cas, un comportement pour ces portions de massif rocheux qui est prédominé par des déplacements de blocs polyèdraux quasi-rigides, déplacements qui sont dépendants de la relation contrainte-déformation des discontinuités géologiques qui les délimitent (John, 1962; Hoek et Brown, 1980). Les instabilités sont induites par la structure géologique.

Cette conception du comportement des massifs rocheux discontinus est à l'origine de plusieurs modèles d'analyse dont, entre autres, ceux de Belytschko et al. (1984), Stewart et Brown (1984 ), Hamajima et Kawai (1981), Burman (1974) et Cundall (1971). De telles méthodes d'analyse numérique sont certes des outils d'intérêt pour le dimensionnement et l'évaluation d'une certaine classe de piliers de surface et constituent des alternatives souhaitées aux méthodes existantes, souvent déficientes, employées en pratique (Bétournay, 1986).

La présente communication porte sur l'application, au problème des piliers de surface, d'une nouvelle méthode d'analyse qui a récemment été développée à l'École Polytechnique de Montréal (Papantonopoulos et Ladanyi, 1973; Papantonopoulos, 1979; Fortin et Gill, 1986; Fortin, 1989 ). Celle-ci est basée sur un modèle de comportement discontinu du massif rocheux, tel que discuté précédemment. Elle se veut être, à l'origine, une généralisation des méthodes conventionnelles d'analyse à la rupture, tirant profit de la programmation linéaire pour contourner le problème d'indétermination mathématique qui constitue généralement la principale objection soulevée à l'égard de ces dernières.

La formulation de cette méthode permet facilement d'incorporer au poids propre du matériau du pilier des pressions hydrauliques le long des discontinuités, des sollicitations externes telles des surcharges de terrains meubles et des forces séismiques, des forces stabilisatrices de soutènement et des champs de contraintes tectoniques. Elle donne à l'ingénieur la possibilité d'obtenir un cadre de solution à son problème même s'il dispose que d'un minimum d'information; en effet, cette méthode fournit une marge théorique de solution qui encadre nécessairement la solution unique recherchée, marge qui peut être restreinte avec l'ajout de nouvelles données.

## DESCRIPTION SOMMAIRE DE LA MÉTHODE

La méthode d'analyse qui fait l'objet de la présente communication découle de celle qui a été élaborée par Papantonopoulos (1979) pour procéder à des analyses de stabilité de talus rocheux. C'est d'abord la variante quasi-statique de cette dernière qui a été modifiée par Fortin (1989) pour être appliquée aux analyses de stabilité de piliers de surface, la raison étant que cette variante constitue, en tout temps, une borne inférieure. Cette approche quasi-statique ne considère que les équations d'équilibre ainsi que quelques hypothèses plausibles sur la cinématique du problème (les signes de contraintes de cisaillement). A ce stade-ci, les analyses ne se font qu'à deux dimensions; ce sont les sections droites des piliers de surface qui y sont soumises.

Dans sa forme la plus simple, la formulation d'un problème implique les étapes suivantes:

i) Séparer la portion contenant le bloc considéré comme potentiellement instable du reste du massif. Les limites de la portion isolée du reste du massif sont appelées frontières internes; ces dernières incluent la limite du socle rocheux sur lequel s'appuient les morts-terrains. Les limites de l'excavation qui créent le pilier de surface, sont appelées frontières externes.

ii) Discrétiser la portion en la découpant en un nombre fini d'éléments (triangles, quadrilatères ou polygones). Le découpage se fait en suivant les discontinuités géologiques qui ont fait préalablement l'objet d'un relevé de terrain.

iii) Formuler les équations d'équilibre de chaque élément en fonction des contraintes effectives agissant sur les côtés des éléments et en fonction de toute autre sollicitation externe.

iv) Formuler les relations qui expriment le critère de rupture aux interfaces des éléments (discontinuités géologiques).

v) Choisir une fonction des variables inconnues (appelée fonction objectif) dont on veut calculer la grandeur. Cette fonction peut être un indice de stabilité, comme par exemple, une marge de sécurité ou un facteur de sécurité, un critère de rupture sur une des interfaces d'éléments spécifiques, ou encore, une variable inconnue quelconque telle qu'une contrainte, une force stabilisatrice d'un soutèvement ou un coefficient séismique.

Le problème étant formulé, il s'agit ensuite de trouver un champ de contraintes licites qui maximise ou minimise la fonction objectif posée, étant donné un schéma d'instabilité potentielle. Par schéma d'instabilité potentielle, on veut signifier une succession de lignes (côtés d'éléments) découpant un bloc (groupe d'un ou de plusieurs éléments) dans la portion définie comme modèle, bloc susceptible d'être instable. Cette succession de lignes doit évidemment rencontrer les frontières externes du modèle et au moins un des côtés du bloc doit être constitué par les parois de l'excavation. Au stade actuel de développement de la méthode, la détermination du schéma d'instabilité potentielle le plus défavorable dans un cas donné ne peut se faire que par des essais successifs. L'expression "champ de contraintes licites" réfère ici à un champ de contraintes statiquement admissible (satisfait les équations d'équilibre et les conditions statiques aux limites du modèle) et plastiquement admissible (satisfait le critère de rupture).

## ÉLABORATION DES ÉQUATIONS

Bien que la méthode permette l'utilisation d'éléments quelconques, nous présentons, pour fins d'illustration, la formulation des équations pour le cas de triangles.

"A priori", il est possible de postuler n'importe quelle répartition de contraintes effectives (contraintes totales moins les pressions hydrauliques) le long des côtés des éléments. Nous allons accepter que les forces effectives appliquées$S_n$ et $T_n$ (forces totales moins les forces dues aux poussées hydrauliques; n liques n = a, b et c, les côtés du triangle; figure 1) donnent lieu à des contraintes réparties linéairement. Si $\sigma_{pn}$, $\tau_{pn}$, $\sigma_{qn}$ et $\tau_{qn}$ sont les contraintes normales et tangentielles appliquées aux points p et q qui définissent le côté n de l'élément (p et q = i, j et k, les sommets du triangle; figure 2), on a:

$$S_n = 0.5\ (\sigma_{pn} + \sigma_{qn})\ L_n \tag{1}$$

$$T_n = 0.5\ (\tau_{pn} + \tau_{qn})\ L_n \tag{2}$$

où $L_n$ est la longueur du côté.

La répartition des pressions hydrauliques est aussi considérée dans la présente communication comme étant linéaire; cette répartition est définie par deux pressions $u_{pn}$ et $u_{qn}$ appliquées aux points p et q du côté n du triangle (figure 2).

### Équilibre d'un élément

La figure 3 représente un élément triangulaire sollicité par $W_m$, le poids de l'élément, $KW_m$, une force séismique et $P_m$, une force concentrée appliquée sur le côté b du triangle dont l'orientation et la position sont définies par $\omega_m$ et $\in_m$ respectivement. On pourrait démontrer que le fait d'admettre qu'une seule force concentrée peut agir par élément et que cette dernière s'applique sur le côté b ne fait pas perdre de généralité. D'après Papantonopoulos (1979) les équations d'équilibre sont:

$$- X_{ij} (\sigma_{ia} + \sigma_{ja}) + Z_{ij} (\tau_{ia} + \tau_{ja}) - X_{jk} (\sigma_{jb} + \sigma_{kb}) + Z_{jk} (\tau_{jb} + \tau_{kb}) - X_{ki} (\sigma_{kc} + \sigma_{ic}) + Z_{ki} (\tau_{kc} + \tau_{ic}) = \gamma_m E + 2 \cos \omega_m P_m + R_1 \quad (3)$$

$$Z_{ij} (\sigma_{ia} + \sigma_{ja}) + X_{ij}(\tau_{ia} + \tau_{ja}) + Z_{jk} (\sigma_{jb} + \sigma_{kb}) + X_{jk} (\tau_{jb} + \tau_{kb}) + Z_{ki} (\sigma_{kc} + \sigma_{ic}) + X_{ki} (\tau_{kc} + \tau_{ic}) = K\gamma_m E + 2 \sin \omega_m P_m + R_2 \quad (4)$$

$$M_3 \sigma_{ia} - M_1 \sigma_{ja} + E (\tau_{ia} + \tau_{ja}) + M_1 \sigma_{jb} + M_2 \sigma_{kb} + E (\tau_{jb} + \tau_{kb}) + M_2 \sigma_{kc} + M_3 \sigma_{ic} + E (\tau_{kc} + \tau_{ic}) = 2\eta P_m + R_3 \quad (5)$$

Les coefficients $X_{ij}$, $Z_{ij}$, ........ E, $M_1$, $M_2$, $M_3$, $\eta$, $R_1$, $R_2$ et $R_3$ sont explicités dans l'annexe. Ils sont fonctions des coordonnées des noeuds i, j, k, de la position de la force $P_m$ et des pressions hydrauliques; $\gamma_m$ est le poids unitaire du matériau constituant l'élément triangulaire.

Équilibre aux interfaces

Si un des côtés de l'élément triangulaire constitue une frontière interne ou externe, les contraintes correspondantes doivent être égales aux charges appliquées. Si p et q sont deux points qui définissent un côté qui sépare deux triangles adjacents, m et m+1, l'équilibre exige que:

$$\sigma_{pn}{}^{m} = \sigma_{pn}{}^{m+1}, \ \sigma_{qn}{}^{m} = \sigma_{qn}{}^{m+1}, \ \tau_{pn}{}^{m} = \tau_{qn}{}^{m+1}, \ \tau_{qn}{}^{m} = \tau_{pn}{}^{m+1} \quad (6)$$

Bien que la méthode permette de considérer n'importe quel critère de rupture convexe dans l'espace de Mohr, la présente communication fait référence à un critère linéaire:

$$|T_n| \leq S_n \operatorname{tg} \phi_n + C_n L_n \quad (7)$$

où $\phi_n$ et $C_n$ sont respectivement l'angle de frottement interne et la cohésion apparente sur le côté n de l'élément.

L'équation (7) peut être réécrite en y combinant les équations (1) et (2) et en se départissant de la valeur absolue:

$$- 0.5 (\sigma_{pn} + \sigma_{qn}) \operatorname{tg} \phi_n - C_n + 0.5 (\tau_{pn} + \tau_{qn}) \leq 0 \quad (8)$$

$$- 0.5 (\sigma_{pn} + \sigma_{qn}) \operatorname{tg} \phi_n - C_n - 0.5 (\tau_{pn} + \tau_{qn}) \leq 0 \quad (9)$$

Nous admettons que la résistance en traction des discontinuités géologiques est nulle; on a donc:

$$\sigma_{in} \geq 0 \quad (10)$$

où i = p et q et n = a, b et c.

Équilibre global de la portion constituant le modèle

Le nombre d'inconnues qui caractérise un élément peut être réduit en tirant profit du fait que $(\tau_{pn} + \tau_{qn})$ apparaît dans les équations d'équilibre des éléments - (3), (4) et (5) - et dans les équations d'équilibre aux interfaces - (8) et (9) - et en posant:

$$\tau_n = 0.5\ (\tau_{pn} + \tau_{qn}) \tag{11}$$

Considérons que la portion constituant le modèle comporte $N_e$ éléments et posons que $N_c$ est le nombre de côtés d'éléments le long desquels le critère de rupture est défini, que $N_\sigma$ est le nombre de contraintes $\sigma_{in}$ inconnues et que $N_\tau$ est le nombre de contraintes $\tau_n$ inconnues. Pour que la portion soit globalement en équilibre, il faut satisfaire:

i) $N_e$ équations (3)
ii) $N_e$ équations (4)
iii) $N_e$ équations (5)
iv) $N_c$ inégalités (8)
v) $N_c$ inégalités (9)
vi) $N_\sigma$ inégalités (10)

Si N est le nombre total d'inconnues on a:

$$N_\sigma + N_\tau \leq N \leq N_\sigma + N_\tau + 2$$

et $N < 3\ N_e$

Les équations (3), (4) et (5) forment donc un système indéterminé offrant une infinité de solutions. Pour en obtenir une solution unique, il faut en même temps maximiser ou minimiser une fonction faisant intervenir les inconnues soit $\sigma_{in}$, $\tau_n$, $P_m$ et K.

L'examen des équations (3), (4) et (5) et des inégalités (8), (9) et (10) montre que ces dernières sont des fonctions linéaires des inconnues $\sigma_{in}$, $\tau_n$, $P_m$ et K; le système peut donc être résolu par programmation linéaire pour autant que l'on s'assure que toutes les inconnues sont positives (Dantzig, 1963). Puisque $\tau_n$ peut être négatif, on contourne la difficulté en posant:

$$\tau_n = \tau_n' - \tau_n''$$

où $\tau_n'$ et $\tau_n''$ sont deux variables positives.

## LA FONCTION À OPTIMISER

La méthode permet d'optimiser plusieurs fonctions différentes selon la façon dont le problème est posé. Les auteurs de la présente communication se sont intéressés, à ce jour, en relation avec les piliers de surface, à deux fonctions particulières. D'une part, dans le cas où l'expectative est la stabilité, la fonction optimisée est la marge de sécurité ou le facteur de sécurité. Dans le cas où l'expectative est d'autre part l'instabilité, la fonction optimisée est le calcul de la force stabilisatrice du bloc clef; une telle force pourrait être générée par des ancrages.

La marge de sécurité

Considérons un schéma d'instabilité potentielle composé de $\ell$ lignes (côtés de triangles). L'équation (7) donne:

$$\sum_{1}^{\ell} |T_n| \leq \sum_{1}^{\ell} (S_n \operatorname{tg} \phi_n + C_n L_n) \tag{12}$$

La marge de sécurité MS pour le schéma d'instabilité potentielle est alors définie à partir de l'équation (12) comme suit:

$$MS = \sum_{1}^{\ell} (S_n \operatorname{tg} \phi_n + C_n L_n) - \sum_{1}^{\ell} |T_n| \geq 0 \tag{13}$$

L'équation (13) est la fonction à optimiser. En tenant compte des équations (1), (2) et (11), cette fonction objectif devient:

$$MS = \sum_{1}^{\ell} \left[ \frac{1}{2} (\sigma_{pn} + \sigma_{qn}) \operatorname{tg} \phi_n + C_n - \tau_n \right] L_n \tag{14}$$

Dans la formulation de l'équation (14), les contraintes de cisaillement ont été considérées positives pour fins de simplification. Ceci ne fait perdre aucune généralité à la méthode étant donné que le signe de ces contraintes est toujours déterminé sur les lignes constituant le schéma de rupture potentielle.

L'optimisation de l'équation (14) donne les bornes de la marge de sécurité MS. En effet, la valeur unique recherchée de MS se situe entre les valeurs extrêmes obtenues en maximisant et en minimisant la fonction (14).

Le facteur de sécurité

Dans le même contexte que celui de la définition de la marge de sécurité, il est aussi possible d'utiliser comme fonction à optimiser le facteur de sécurité FS. Par définition, cette fonction s'écrit, partant de l'équation (12),

$$FS = \sum_{1}^{\ell} (S_n \operatorname{tg} \phi_n + C_n L_n) \Big/ \sum_{1}^{\ell} |T_n| \tag{15}$$

La valeur unique recherchée de FS se situe entre les valeurs extrêmes obtenues en maximisant et en minimisant la fonction (15).

La force de stabilisatrice

Il s'agit de calculer la force minimale nécessaire pour stabiliser un pilier potentiellement instable. La force $P_m$ montrée à la figure 3 est alors considérée comme variable inconnue et est transférée aux membres de gauche des équations (3), (4) et (5). La fonction objectif est alors cette variable.

## EXEMPLE D'APPLICATION

Afin d'illustrer l'application de la méthode à des problèmes concrets, nous présentons dans la suite un exemple d'analyse dans un cas très simple, tiré de Fortin et Gill (1986).

Le pilier de surface illustré en section transversale à la figure 4 est découpé entre deux épontes subverticales d'un gisement où les contacts avec le massif encaissant sont altérés, constituant ainsi des plans préférentiels de rupture. De plus, la roche du gisement est recoupée par deux familles de diaclases inclinées dont les directions sont grossièrement parallèles à celle du gisement.

Les épontes étant dans ce cas-ci d'une qualité exceptionnelle, les frontières internes latérales de la portion qui est isolée pour l'analyse coïncident avec les épontes.

Les charges dues au mort-terrain ont été considérées négligeables; il n'y a donc aucune charge sur la frontière interne supérieure. La distribution des pressions sur les frontières internes latérales est celle montrée à la figure 4. Une telle distribution s'obtient en procédant par exemple à une analyse de contrainte par méthode numérique en supposant que les contraintes préalables à l'exploitation du chantier suivent la règle de Brown et Hoek (1978) ou celle de Herget (1987) ou encore en invoquant sécuritairement, comme dans le présent exemple, une poussée propre aux terres au repos.

La figure 5 illustre la portion isolée du massif rocheux soumise à l'analyse et un découpage en éléments triangulaires; cette figure montre également la numérotation des lignes et des noeuds et le sens des contraintes du cisaillement. Le découpage a été fait en référant à deux schémas d'instabilité potentielle différents: le schéma A, qui implique une instabilité suivant les lignes 1 et 2, et le schéma B, qui implique une rupture suivant les lignes 3 et 6. La fonction objectif choisie est une force P au centre de la largeur de l'excavation; cette force représente l'action d'un soutènement et on désire la minimiser. Le sens des contraintes de cisaillement est établi par simple examen du problème.

Le tableau I présente les coordonnées des noeuds des éléments correspondant à la discrétisation de la figure 5 alors que le tableau II résume les propriétés résistantes des lignes des deux schémas d'instabilité potentielle.

La figure 6 illustre le champ de contraintes obtenu en tant que solution. La valeur de la force P minimisée est zéro. Ceci signifie que les conditions de rupture ne sont pas atteintes sur les différentes lignes des deux schémas de rupture; d'où présomption de stabilité sans soutènement. Le tableau III présente les facteurs et les marges de sécurité pour les deux schémas d'instabilité potentielle; puisqu'il y a présomption de stabilité, les valeurs des facteurs de sécurité sont supérieures à l'unité et les valeurs des marges de sécurité sont supérieures à zéro.

## VARIANTES ET EXTENSIONS

La notion de marge de sécurité partielle, définie comme étant la marge de sécurité sur un segment linéaire du schéma d'instabilité potentielle (plutôt que sur l'ensemble des lignes) permet de simuler dans les analyses, s'il y a lieu, la progressivité de la rupture. Il en est de même pour le facteur de sécurité.

La méthode élaborée par Papantonopoulos (1979) comporte aussi une variante cinématique d'analyse à la rupture qui présente également un certain intérêt en relation avec les piliers de surface. Fortin (1989) propose, à partir de cette variante et de pair avec la méthode présentée dans cette communication, une approche au calcul des déformations élasto-plastiques sur les interfaces des éléments et, par la suite, des déplacements rigides des polyèdres de roche; cette modélisation est utile, entre autres, pour la surveillance de l'ouvrage lors de sa réalisation.

## CONCLUSION

Certains piliers de surface possèdent des caractéristiques géologiques qui entraînent un comportement complexe de milieux discontinus et qui rendent les analyses difficiles. La méthode brièvement présentée nous apparaît être une alternative réaliste, relativement plus simple, plus adaptée ou mieux appropriée et plus avantageuse que les méthodes actuellement utilisées. Elle donne à l'ingénieur la possibilité d'obtenir un cadre de solution à son problème même lorsqu'il ne dispose que d'un minimum d'information.

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ANNEXE - Forme explicite des coefficients des équations d'équilibre (équations 3 à 5)

$X_{ij} = X_i - X_j$ . . . . . . . . . . A-1
$X_{jk} = X_j - X_k$ . . . . . . . . . . A-2
$X_{ki} = X_k - X_i$ . . . . . . . . . . A-3
$Z_{ij} = Z_i - Z_j$ . . . . . . . . . . A-4
$Z_{jk} = Z_j - Z_k$ . . . . . . . . . . A-5
$Z_{ki} = Z_k - Z_i$ . . . . . . . . . . A-6
$E = Z_i X_{jk} + Z_j X_{ki} + Z_k X_{ij}$ . . . . . . . . . . A-7
$M_1 = X_{ij} X_{jk} + Z_{ij} Z_{jk}$ . . . . . . . . . . A-8
$M_2 - X_{jk} X_{ki} + Z_{hj} Z_{ki}$ . . . . . . . . . . A-9
$M_3 = X_{ki} X_{ij} + Z_{ki} Z_{ij}$ . . . . . . . . . . A-10
$\eta = \cos \omega_m [X_{ik} (1-3 \varepsilon_m) - X_{ij}] - \sin \omega_n [Z_{jk} (1-3 \varepsilon_m) - Z_{ij}]$ . . . . A-11
$R_a = (u_{ia} + u_{ja})$ . . . . . . . . . . A-12
$R_b = (u_{jb} + u_{kb})$ . . . . . . . . . . A-13
$R_c = (u_{kc} + u_{ic})$ . . . . . . . . . . A-14
$R_1 = R_a X_{ij} + R_b X_{jk} + R_c X_{ki}$ . . . . . . . . . . A-15
$R_2 = R_a Z_{ij} - R_b Z_{jk} - R_c Z_{ki}$ . . . . . . . . . . A-16
$R_3 = - M_3 u_{ia} + M_1 u_{ja} - M_1 u_{jb} + M_2 u_{kb} - M_2 u_{kc} + M_3 u_{ic}$ . . . . . . A-17

Les équations précédentes sont tirées de Papantonopoulos et Ladanyi (13) sauf les équations A-12 à A-14 et A-17 qui ont été élaborées par les auteurs de la présente communication.

TABLEAU I - Coordonnées des noeuds des éléments du cas de la figure 5.

| Noeud | Z, in. | X, m |
|---|---|---|
| 1 | 24.4 | 24.4 |
| 2 | 12.2 | 24.4 |
| 3 | 12.2 | 18.3 |
| 4 | 12.2 | 12.2 |
| 5 | 24.4 | 12.2 |

TABLEAU II - Paramètres de Coulomb caractérisant la résistance le long des lignes des éléments de la figure 5.

| Ligne | Frottement interne, deg. | Cohésion apparente, kPa |
|---|---|---|
| 1 | 32 | 168 |
| 2 | 32 | 168 |
| 3 | 30 | 144 |
| 6 | 30 | 144 |

TABLEAU III - Facteurs et marges de sécurité

| Schéma | Lignes | Facteur de sécurité | Marge de sécurité, kPa |
|---|---|---|---|
| A | 1 et 2 | 1.92 | 197.2 |
| B | 3 et 6 | 1.37 | 113.6 |

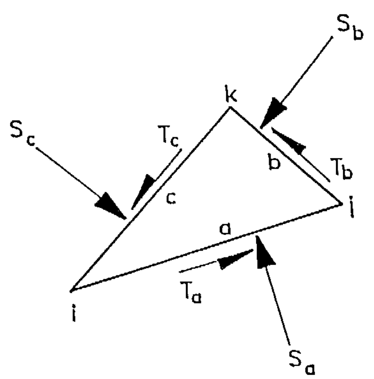


Figure 1 - Forces effectives agissant sur un élément triangulaire de sommets i, j, k et de côté a, b, c.

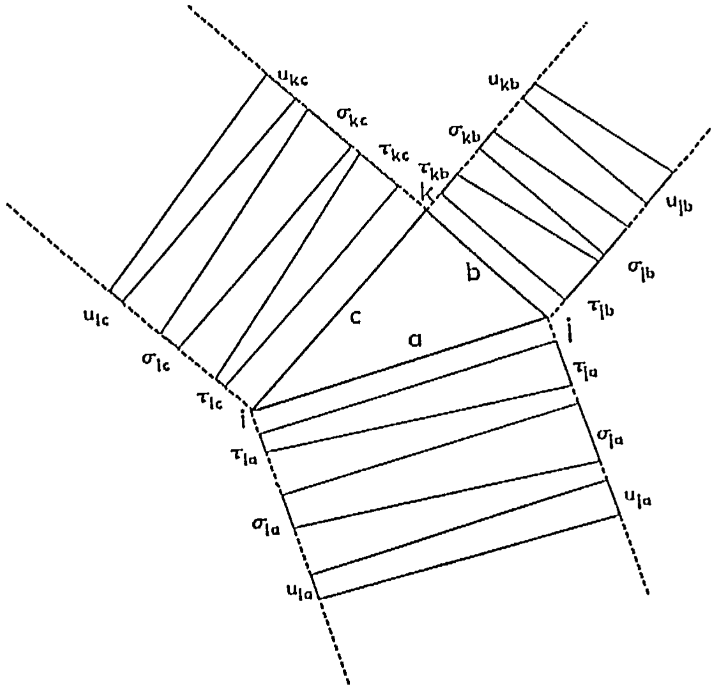


Figure 2 - Répartitions des contraintes effectives et des pressions hydrauliques acceptées.

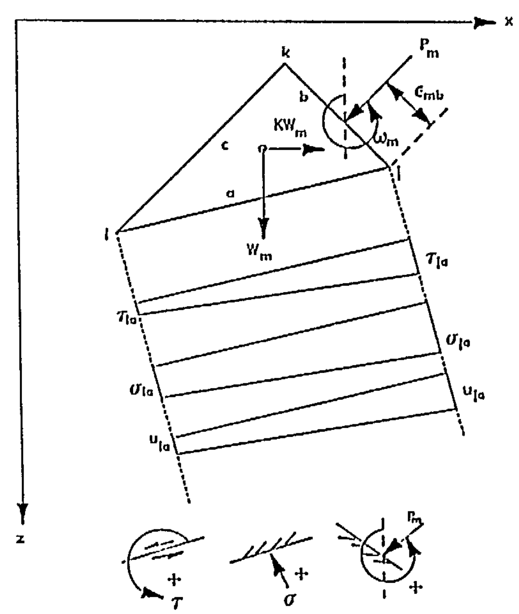


Figure 3 - Equilibre d'un élément triangulaire.

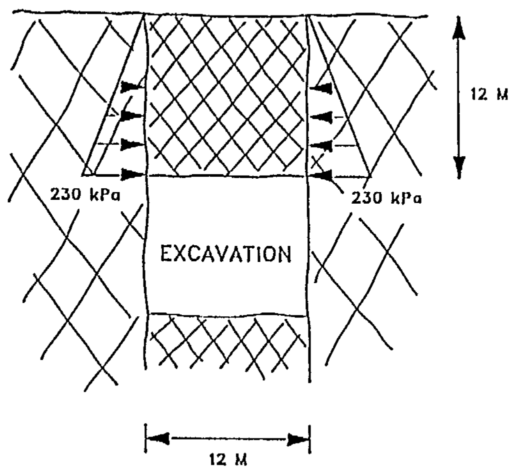


Figure 4 - Exemple d'analyse d'un pilier de surface; section transversale.

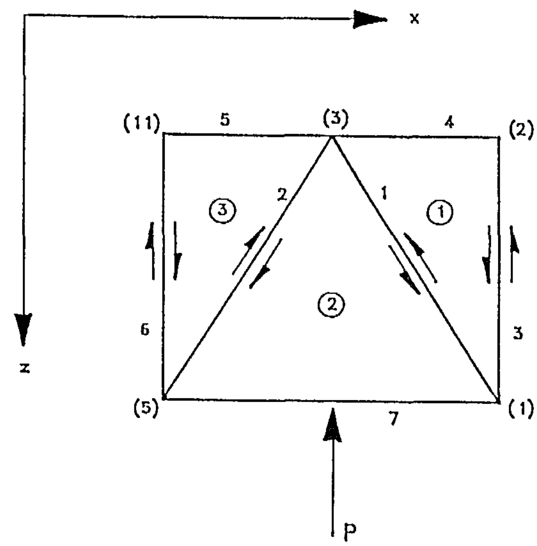


Figure 5 - Exemple d'analyse d'un pilier de surface; portion soumise à l'analyse et découpée en éléments triangulaires.

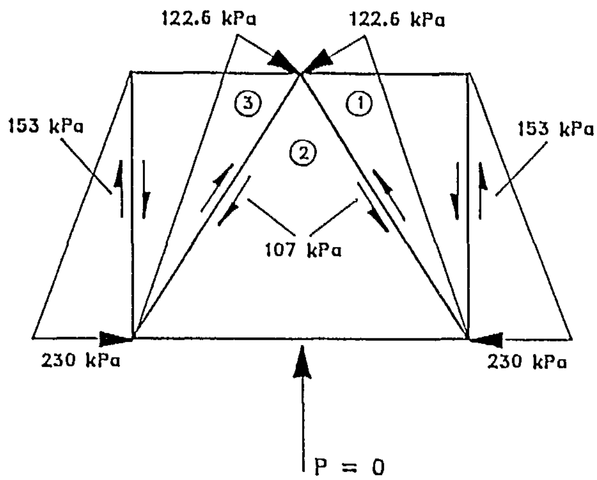


Figure 6 - Exemple d'analyse d'un pilier de surface; résultats de l'analyse lorsque la fonction à optimiser est la minimisation d'une force stabilisatrice.

# EVALUATION OF SURFACE CROWN PILLAR FAILURE BY COMPUTER MODELING WITH IN SITU MEASUREMENTS OF STRESSES, PROPERTIES, AND DEFORMATIONS

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## ABSTRACT

For realistic modeling of the crown pillar failure process, stress states (S), material properties (P), and ground deformation (D) should be measured accurately _in situ_. They should then be interrelated as a function of time and space using a finite element computer model (R) of the ground. A quantitative approach known as SPDR Technology has been achieved by developing the hardware necessary for measuring S, P, and D as well as the software for FE modeling. The SPDR Technology has been applied successfully to the geomechanical anlaysis of ground failure in non-metal mines. The technology should now be considered for quantitative analysis of surface crown pillar failure in metal mining.

## RÉSUMÉ

Pour modéliser d'une façon réaliste le mécanisme de rupture de piliers de surface, l'état des contraintes (S), les propriétés des matériaux (P), et les déformations du massif (D) doivent être mesurés en place avec précision. Ils doivent ensuite être corrélés en fonction du temps et d'espace en utilisant un modèle numérique par éléments finis (R) du massif. Une approche quantitative nommée SPDR Technology a été obtenue en développant l'outillage nécessaire pour mesurer S, P et D et en formulant le logiciel pour la modélisation par éléments finis. La technologie SPDR a été appliquée avec succès à l'analyse géomécanique de rupture en roche tendre. Cette technologie mérite maintenant considération pour l'analyse quantitative de rupture de piliers de surface en terrains de roche dure.

## INTRODUCTION

Crown pillar failure followed by surface subsidence is a basic geomechanics problem related to the time-dependent behavior of complex ground. Previous attempts at quantitative analysis of complex ground behavior have met with little or no success. Major improvement of the earlier methods of analysis requires basic technological advances in the following:

(S) *In situ* stress measurement in nonidealized prefractured ground
(P) *In situ* material property measurement in complex ground
(D) Accurate, long-term, continuous monitoring of ground deformation
(R) Realistic finite element computer program able to simulate failure and post-failure behavior of the ground.

An approach known as SPDR Technology has been developed to meet the above four basic requirements. This paper discusses how SPDR Technology can address the crown pillar failure problem in metal mining based on its successful application to non-metal mining over the past ten years.

## DOUBLE FRACTURE METHOD OF *IN SITU* STRESS MEASUREMENT (S)

An instrument system for measuring stress states in complex ground has been developed [Serata and Kikuchi 1986,1989]. Known as the Serata Stressmeter, the system is applied in two different methods of measurement: one for brittle-prefractured and the other for soft-ductile ground conditions. The "Double Fracture Method," applicable to brittle-fractured ground, is described in this section. The "SP Method," applicable to soft-ductile ground, is described in a separate paper [Serata 1989].

### Principle of Double Fracture Method

The probe component of the Serata Stressmeter is a plastic cylinder that is activated hydraulically to apply a uniform radial pressure to the borehole walls, creating multiple fractures around the borehole. The first fracture is created in a manner identical to that of hydrofracturing, as shown in Fig. 1. However, unlike the hydrofracturing method, the Stressmeter allows no pressurized liquid to penetrate into the newly fractured plane. Consequently, the width of the fracture remains microscopic and the length is limited.

With further increase of the loading pressure, a second fracture occurs perpendicular to the first. A set of two mutually perpendicular fracture planes enables us to directly calculate the maximum and minimum stresses $P_o$ and $Q_o$, as graphically illustrated in Fig. 1. The stress orientation is determined from the diametral deformation rosette obtained at the loading pressure $p^R$, which is greater than the free separation pressures $p_1^E$ and $p_2^E$, as shown in Fig. 2. Here, the directions of the minimum and maximum diametral expansions $D_1$ and $D_2$ match those of the maximum and minimum stresses $P_o$ and $Q_o$. The magnitudes of $P_o$ and $Q_o$ are determined by solving the simultaneous equations of Fig. 1.

The hardware of the Serata Stressmeter system is illustrated schematically in Fig. 3. The system is operated by a laptop computer which sends commands to and receives data from the downhole probe. Real-time data processing permits monitoring and analysis of the graphically displayed data, as shown in Fig. 1. The diagrams show an identifiable free separation pressure $p^E$ for each previously fractured plane. In addition, the Stressmeter is useful in obtaining material properties of the ground as mass media. A complete discussion of the Stressmeter is given in Serata and Ki- kuchi (1989).

## PENETROMETER METHOD OF IN SITU PROPERTY MEASUREMENT (P)

The Serata Propertymeter was developed to measure in situ material properties of complex ground, including those of weak zones. The probe is equipped with a set of eight pistons that are hydraulically activated to initiate eight independent penetration tests on the borehole wall surface (Fig. 3). The instrumentation system used by the Propertymter is identical to that used by the Stressmeter. By performing penetration tests within a borehole, the Propertymeter measures actual material properties (preserved in their natural state) and displays them graphically on a laptop computer, while the measurements continue. A typical field data display is shown in Fig. 4.

Laboratory studies [Lundberg 1974, Wang et al. 1976, Serata et al. 1983, Cook et al. 1984] have shown that a unique relationship exists between individual material properties and the piston penetration characteristics. This work further discloses that penetration tests applying varied contact geometries against the same material surface will produce different penetration behaviors, and that the common material properties can be determined by regression analysis of the behavioral simulation. The simplest example of this is the existence of a singular relationship between certain material properties determined by standard laboratory testing (such as failure strength and Young's modulus) and their corresponding values obtained from penetration tests.

The point contact properties obtained by the Propertymeter are utilized not only to analyze the variations of ground properties but also to synthesize media properties of the ground for FE analysis of underground designs. The penetration data are applied to determine material property coefficient values of Young's modulus, deformation modulus, failure strength, Poisson's ratio, viscoelastic coefficient, and viscoplastic constant.

## GROUND DEFORMATION MEASUREMENT (D)

A wide variety of ground deformation probes is readily available for crown pillar deformation measurement. Among them the inclinometer, extensometer, and surface subsidence meter are especially useful. A significant improvement is now being made in surface level measurement by combined use of a high resolution liquid levelmeter with a computerized noise filter, enabling monitoring of surface subsidence with a resolution of 10 to 100 microns. Using such a device, a surface network of stations can be installed over a sensitive area for early warning of potential problems as well as to calibrate the computer model of the crown pillar.

## REALISTIC COMPUTER PROGRAM FOR COMPLEX GROUND (R)

To be capable of representing time-dependent behavior of complex ground, a FE program must be capable of comprehending the fundamental behavioral characteristics of brittle fracture and creep failure as well as volume change and material strength deterioration. A constitutive equation of generalized earth materials (Fig. 5) has been developed for failure and post-failure behavior analysis of complex mine ground. The unique feature of this program is its ability to follow the natural deterioration process from the intact or prefractured rock media down to destroyed, sand-like material. The program utilizes FE meshes to model major discontinuity planes while the complex prefractured features are modeled by composite mass media.

As shown in Fig. 6, the REM program is utilized to model the crown pillar of a salt mine. The global stress distribution pattern of Fig. 7 shows that a small crown pillar of 20 m sill thickness is preferred over the conventional 80 m sill thickness between the two levels of mining. With the small sill thickness, the surface subsidence is predicted to be minimal, giving the required stability (as confirmed by subsequent measurements over the surface of the mine).

## APPLICATION TO METAL MINES

Crown pillar stability is directly related to the stress state in the underground, as shown in Fig. 8. The uppermost diagram of the figure illustrates the stable state of the crown pillar, in which all of the concentrated stresses remain around the individual openings. Stability is maintained by the individual stress envelopes. Such conditions may be disclosed by direct measurement of the lateral stresses in the overburden, as indicated by Curve A in Fig. 9. As more rooms are mined out, a secondary stress envelope forms, relieving stress immediately above the rooms and transferring the stress to the overburden formations, as indicated by Curve B in Fig. 9. The critical importance of the crown pillar develops with time as the overburden formations deteriorate in response to the increase in stress, and the secondary stress envelope expands upward, approaching the surface.

The final failure of the crown pillar begins when the stress envelope goes above the last stiff formation in the pillar and collapses. The primary stress envelope is renewed, with the full weight of the crown pillar now loading upon the vertical pillars at the mining elevation, as illustrated in the bottom diagram of Fig. 8. The corresponding stress distribution is indicated by Curve C of Fig. 9. The time-dependent variation of the stress distribution in the crown pillar can be used to disclose the current condition as well as the long-term stability of the crown pillar. Therefore, the Serata Stressmeter is considered most suitable for determining the stress distribution pattern in the crown pillar, and for assessing the present and future condition of the crown pillar.

Since the material properties (P) are determined simultaneously with the stress (S) measurement, the field data are used to construct the realistic finite element computer model of the entire ground. The model is then calibrated with the in situ ground deformation (D) measurement for long-term safety evaluation.

## CONCLUSIONS

The analytical SPDR Technology presented here has been developed through the study in non-metal mining. The concept of the stress envelope and its failure have been tested and effectively utilized in both hard and soft rock. Because of the basic similarity of the ground material behavior, regardless of the type and depth of mining, the proposed application of the technology to analyze the failure of metal mine crown pillars appears reasonable for immediate consideration.

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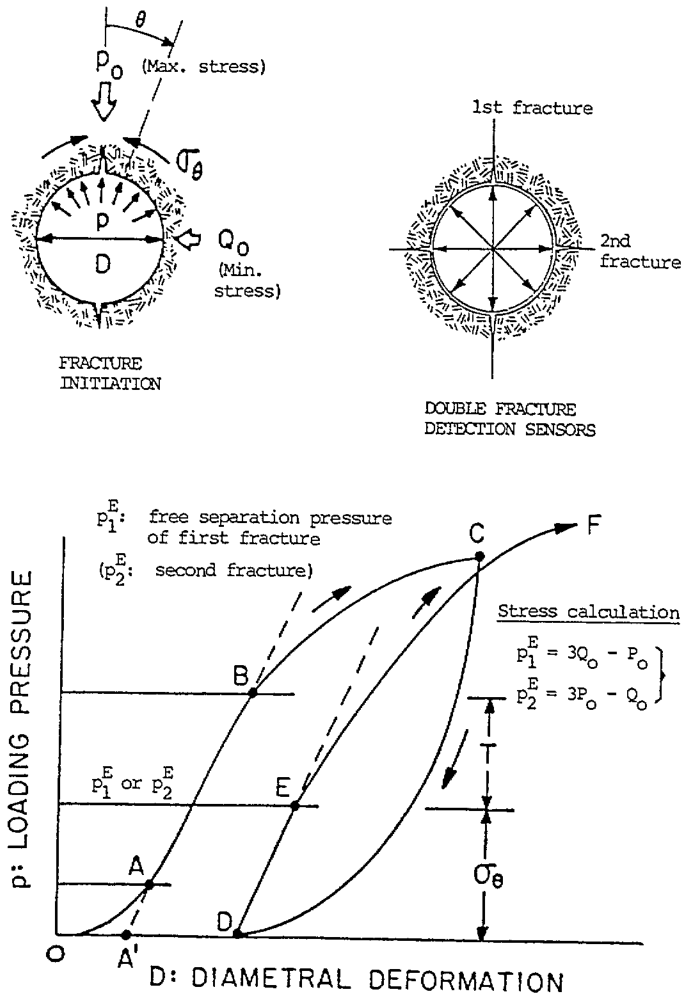



FIG. 1. Loading pressure vs. diametral deformation (p-D) curve under cyclic loading [B: fracture initiation point; E: free separation point]

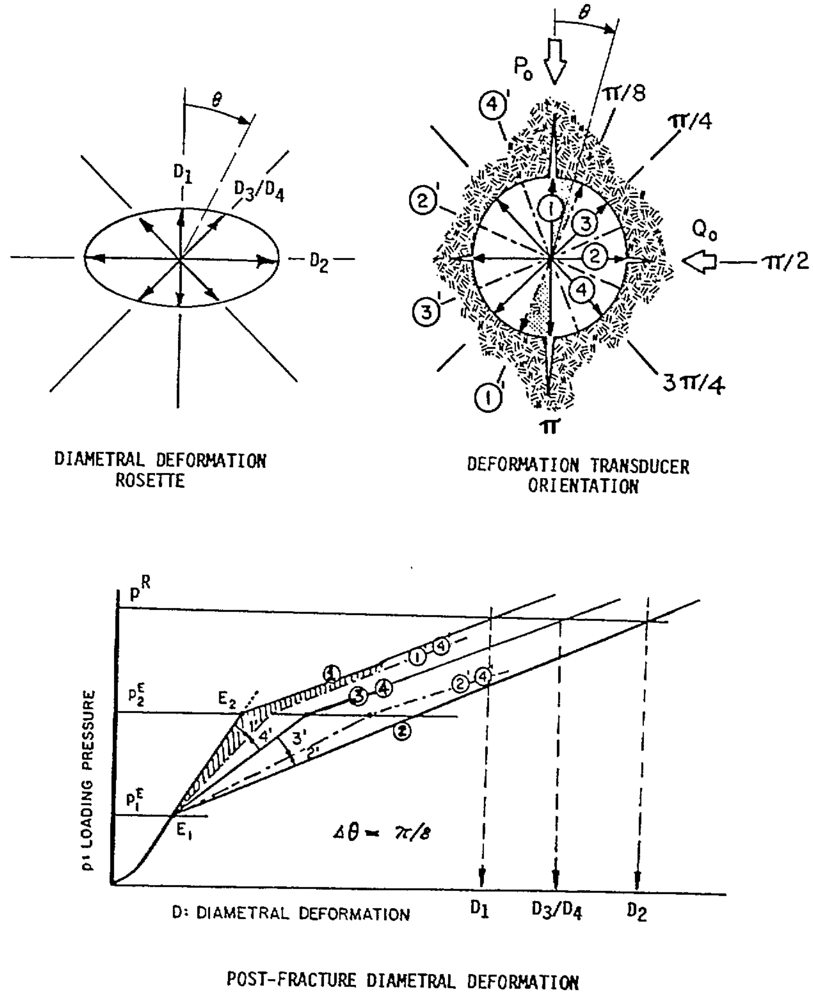



FIG. 2. Double fracture development and diametral deformation rosette in relation to p-D curves

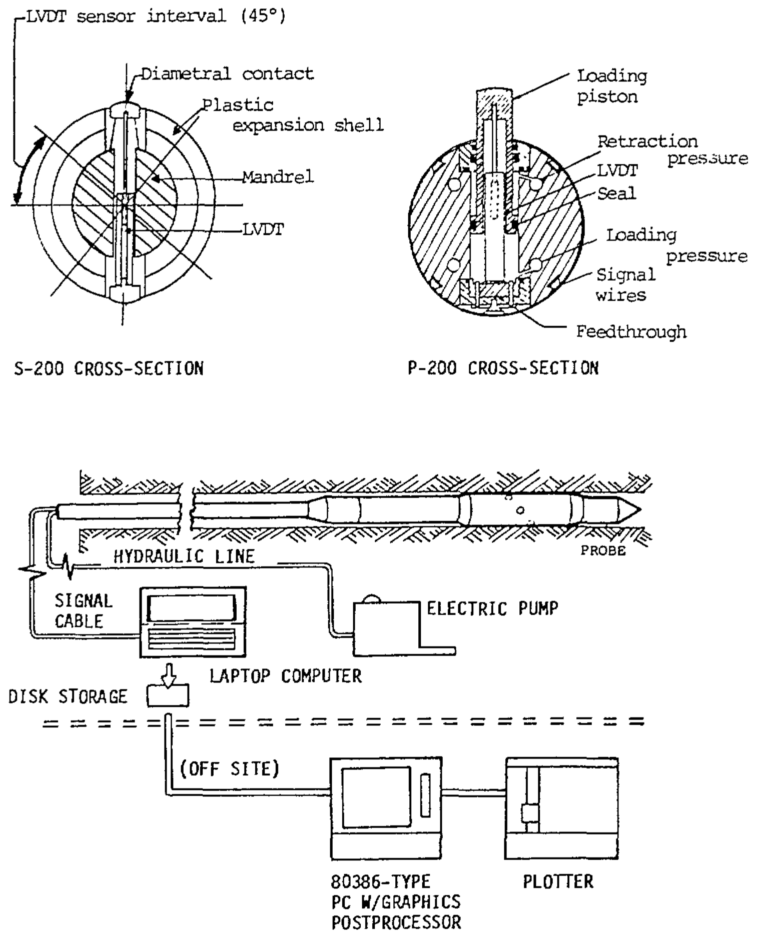



FIG. 3. Serata stress-property measurement system SP-200 showing cross-section of Stressmeter S-200, which measures stress state in brittle-fractured ground

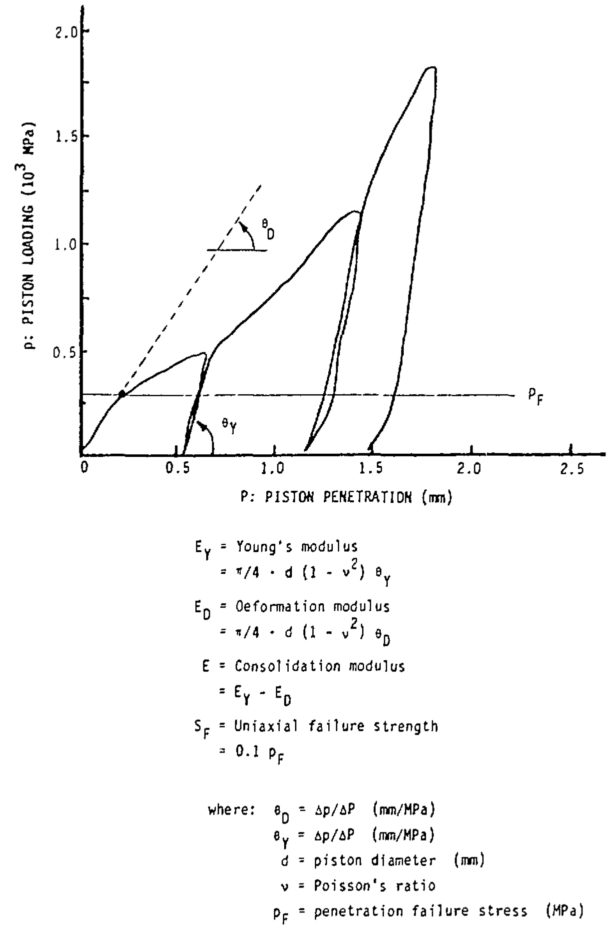
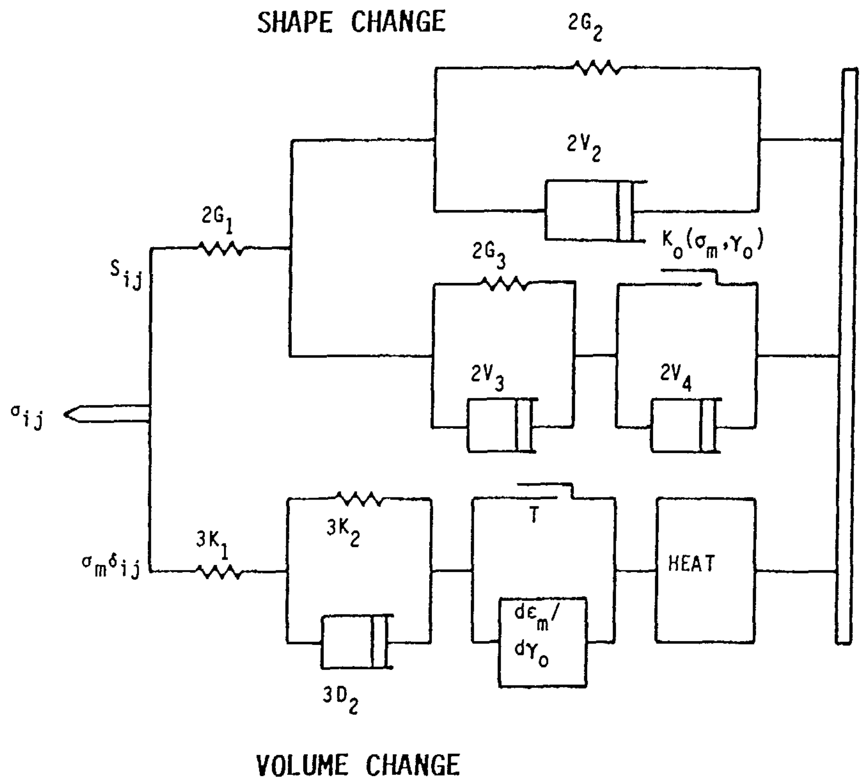



FIG. 4. Determination of elastic moduli and failure strength from p-P curve of P-100/200 measurement results




KEY

$\sigma_{ij}$: Stress tensor
$\varepsilon_{ij}$: Strain tensor
$\tau_o$ : Shear stress
$\gamma_o$ : Shear strain
$\sigma_m$ : Hydrostatic stress
$\varepsilon_m$ : Hydrostatic strain

MATERIAL PROPERTY PARAMETERS

$G_1$ : Shear modulus
$G_2$: Retarded shear modulus in viscoelastic state
$G_3$: Retarded shear modulus in viscoplastic state
$V_2$: Elastoviscosity in viscoelastic state
$V_3$: Elastoviscosity in viscoplastic state
$V_4$: Plastoviscosity in viscoplastic state
$K_o$: Octahedral shear strength
$K_1$: Bulk modulus
$K_2$: Retarded bulk modulus in viscoelastic state
$D_2$: Hydrostatic elastoviscosity in viscoelastic state
T : Tensile strength

FIG. 5. REM constitutive model of generalized earth materials

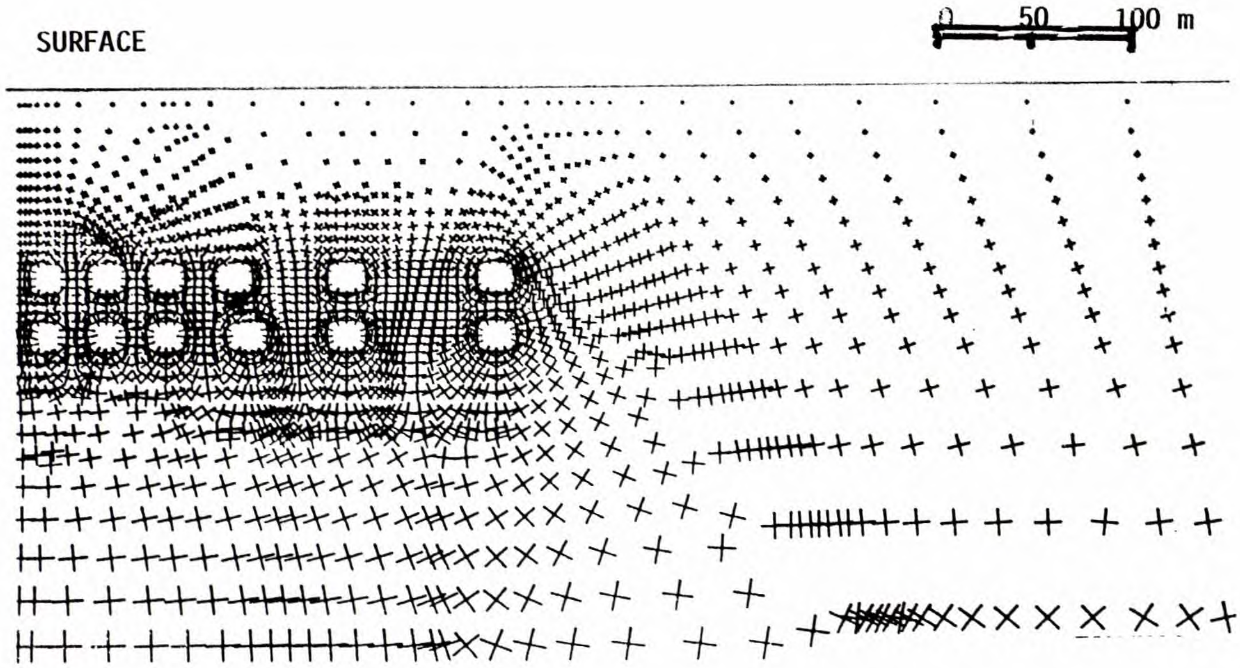



FIG. 6. Finite element mesh for analyzing crown pillar and sill pillar behaviors in shallow salt mine

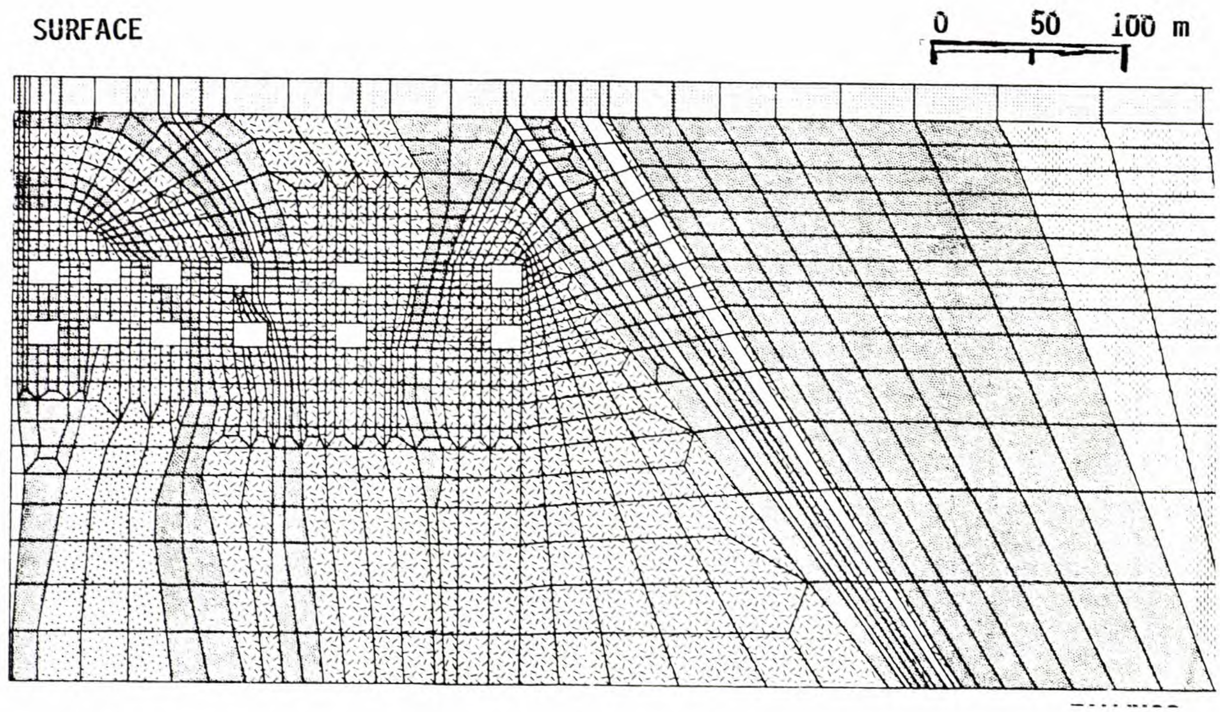



FIG. 7. Global distribution of principal stresses disclosing secondary stress envelope being developed in center section of mine, which coincides with initiation of surface subsidence

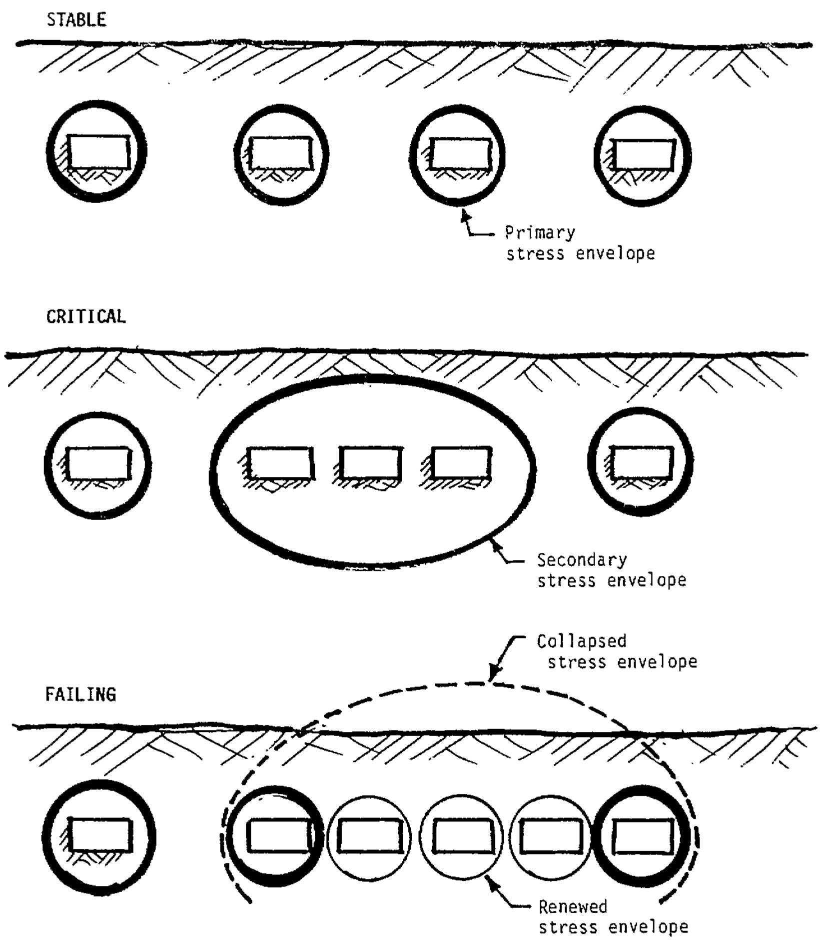



FIG. 8. Three stages of crown pillar failure in relation to rise and fall of secondary stress envelope

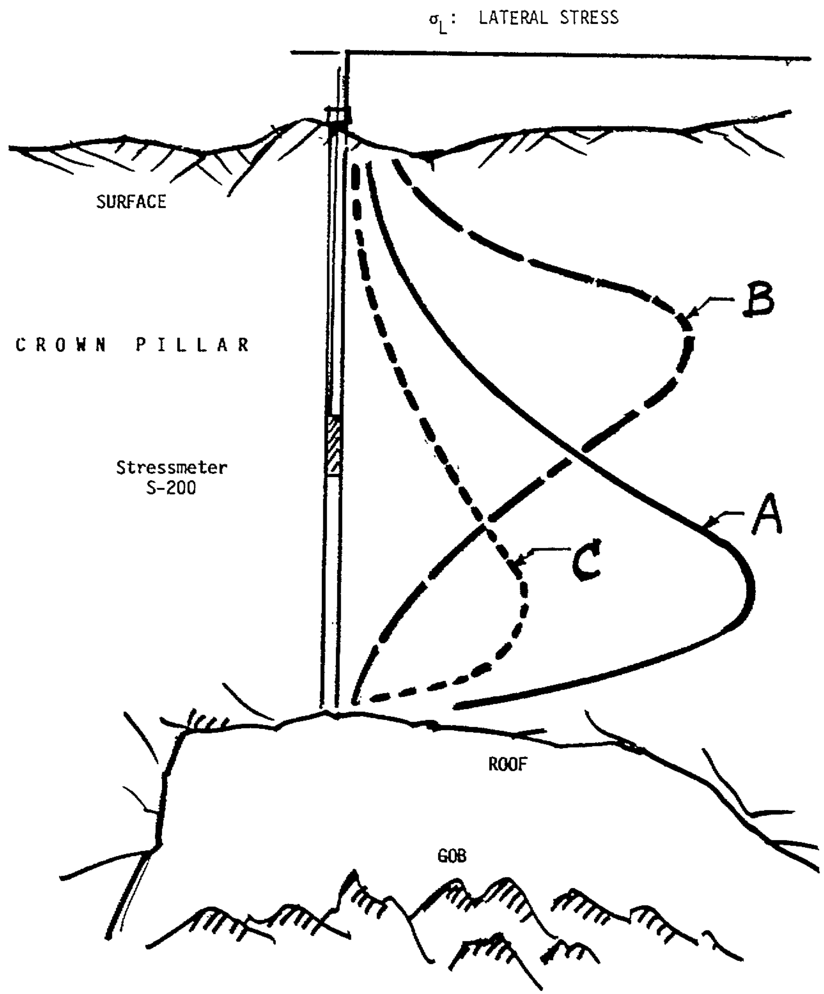



FIG. 9. Three distribution patterns of lateral stresses in crown pillar, disclosing various stages of failure of the pillar

# COMPLETE RECOVERY OF SURFACE CROWN PILLARS WITHOUT REMOVAL OF OVERBURDEN

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## ABSTRACT

The results of research sponsored by CANMET to investigate the nature, behaviour and complete removal of a weak soil-like rock mass constituting a highly enriched surface crown pillar at Les Mines Selbaie are presented in this paper. The geotechnical properties of the overburden soils were determined from pressuremeter and laboratory tests. A simple rock mass classification system was developed for the weak, soil-like rock of the surface crown pillar zone. Stope opening design charts were developed using both 2-D and 3-D numerical models. Surface subsidence prediction models were developed using analytical flow mechanics. The overburden soil bulking factor and negative pore pressures were found to be critical to the selection and application of control caving techniques for the complete recovery of the enriched surface crown pillar. Details of the field and analytical techniques used, and site-specific instrumentation developed, are given in the paper.

## RÉSUMÉ

Le compte rendu de la recherche subventionnée par CANMET pour examiner la nature, le comportement et la récupération complète d'un massif faible et altéré de pilier de surface est présenté ici. Les propriétés géotechniques des mort-terrains ont été évaluées, à partir d'essais de pressiomètres et d'essais en laboratoire. Un système simple de classification de massif rocheux fut développé pour le roc faible altéré de la zone du pilier du surface. Des courbes pour la conception des chantiers furent élaborées à partir de modèles numériques 2-D et 3-D. Des modèles d'affaissement de surface ont été mis au point à l'aide de la mécanique analytique d'écoulement. Le facteur de tassement des sols et les pressions interstitielles négatives se montrèrent critiques à la sélection et l'application de techniques de foudroyage contrôlées pour la récupération complète du pilier de surface enrichi. Des détails sur les méthodes de terrains et les méthodes analytiques utilisées sont présentés, de même que des détails sur l'instrumentation développée pour les conditions en place.

## INTRODUCTION

In 1985, research was sponsored by CANMET, under contract to Strata Engineering Corp., to develop and validate a weak rock mass model for support of surface crown pillars at Les Mines Selbaie, near Joutel, Québec. In 1988, a follow-up research contract was awarded to Les Mines Selbaie, with Strata as subconsultants, to investigate the economics and feasibility of complete recovery of the surface crown pillars without removal of the overburden. This paper presents the results of research since 1985.

Complete recovery of a surface crown pillar, without removal of overburden, requires the characterization and prediction of behaviour of both the rock mass and the overlying overburden soils. At Les Mines Selbaie, the overburden soils comprise a series of very dense till-like sands and silts with very little cohesion. The surface crown pillar in contact with the overburden consists of a weak soil-like, extensively altered rock mass for which conventional methods of analysis of surface crown pillar stability, such as those used in hard rock mining, are inapplicable. The ore values in the surface crown pillar zone (45 m to 60 m level approximately) are 8 to 9 per cent as compared with 2 to 3 per cent at the 200 m level. Hence, the complete recovery of this rich ore zone was considered extremely desirable, provided it could be accomplished economically and safely. Details of the work undertaken towards complete recovery of surface crown pillars at Les Mines Selbaie are provided.

## SITE AND GEOLOGY

Les Mines Selbaie is a 1650 tpd copper-zinc mine located in northwestern Québec, approximately 60 km northwest of Joutel and 40 km east of the Ontario/Québec boundary. The orebody is located within the Chibougamou-Matagami section of the Abitibi volcanic belt. The geology of this area has been documented [Sinclair and Gasparrani, 1979; Deptuck et al., 1982].

The bedrock consists of a series of tuffs and breccias, bedded chert-pyrite and volcanoclastic debris located near the southwestern margin of a plutonic complex known as the Brouillan Granite. The overburden within the area investigated is some 40 m thick and comprises glacial tills and glacio-fluvial sand and silt deposits which have been overridden by continental glaciation to form very dense sheets, with little or no cohesion, even though a cohesive clayey formation known as the Cochrane Till is found in some parts of the mine site.

The mine is situated in a tabular ore body dipping at 50°, and associated with a northeasterly striking fault consisting of quartz-sulphide vein systems up to 200 mm wide, occurring within several elongate lenses of heavily fractured rock, up to 30 m in width, in the hanging wall of the fault. The rock immediately surrounding the fault zone is characterized by pulverized sulphides and shattered quartz veins. The fault and open fractures have apparently provided an excellent channelway for descending meteoritic water. As a result, supergene alteration [Sinclair and Gasparrani, 1979] has enriched ore values as far down as the 200 m depth level.

The orebody is composed of three types of materials: rock-like; soil-like; and infill material. The rock-like materials comprise about 70 per cent of the orebody by volume and consist of dacite tuff, highly decomposed and kaolinized dacite tuff breccia containing lenses of moderately decomposed quartz and faintly decomposed chalcocite. The soil-like rock comprises about 30 per cent by volume of the orebody and consists of gravel sized angular pieces of rock in a clayey to sandy silt matrix. It includes stringers of decomposed sooty chalcocite and highly to completely decomposed kaolinized dacite tuff breccia. The infill materials generally comprise fine sands and silts, mostly found near the contact with the overburden.

The soil and rock materials were studied by drilling and sampling three boreholes in 1985-86 [C. Mirza Engineering Inc., 1986], dilatometer testing in two additional boreholes during 1988 [Monterval, 1988], piezometer installations, monitoring of groundwater levels, and laboratory testing. The field investigation incorporated new drilling/coring techniques which provided very high recovery of the undisturbed altered rock material [Bétournay et al., 1988].

From underground scanline surveys, laboratory testing, and field measurements, a Quick Field Classification system was developed for the in situ weak rock and soil-like rock materials, Table 1. Good correlation was found with certain index properties, as shown in Figure 1. Key properties of the materials in the surface crown pillar and the overburden are given in Table 2.

## DESIGN METHODOLOGY

Design methods available for the dimensioning of surface crown pillars are empirical, analytical and numerical. Most Canadian mines design underground openings on the basis of empirical and generic modelling methods. The selection of an appropriate numerical model depends on rock mass characterization. For the Selbaie site, the factors considered in the selection of an appropriate rock mass behaviour model were: scale, volumetric ratio, and failure mode.

The scale factor is defined as the ratio of the size of the crown pillar to the spacing of discontinuities in the rock mass (eg. block size).

The volumetric ratio is defined as the ratio of the volume of altered material to the total volume. Under different alteration conditions, the resulting size of the rock pieces and their packing may be different even though the volumetric ratio may be the same. Hence, the arrangement and packing of the discrete rock elements exerts a significant influence on the selection of the rock mass behaviour model. Based on earlier work [Arjang, 1984; Swan, 1985], the surface crown pillar zone at Selbaie is classified as a heavily jointed rock mass, and the volumetric ratio is estimated to be 0.3.

As to failure modes, the shape, size, distribution and packing of the rock pieces in the surface crown pillar could not be determined with any reliability. Therefore, the probable failure mode was studied using a base friction model, in which a tensile type failure due to gravitational forces was observed. The final shape of the opening was an arch, similar to a previous reported failure at the Mine. These findings favoured the selection of an elastoplastic rock mass behaviour numerical model. By examining the field data acquired [Arjang, 1984], it was decided to use a kaolinized soil model with an angle of internal friction, $\phi' = 33.5°$, and cohesion intercept, $c' = 0.046$ MPa. This model was calibrated against a 1983 cave-in in Stope 4, using a three-dimensional finite element program capable of handling Mohr-Coulomb type elements. The problem geometry used in the model is shown in Figure 2. The results of predicted versus actual failure modes, Figures 3 (a) and (b), show close agreement.

The cave-in above Stope 4 occurred some 6 to 8 months after the stope was mined. It extended from the 100 m level up to the 60 m level. An examination of the causes of the delayed failure lead to the conclusion that negative pore pressures, induced at time of blasting, were or might be largely responsible. This hypothesis was corroborated by using a simplified two-dimensional analysis of the drainage characteristics of the kaolin material [Strata, 1987].

A parametric study was undertaken, upon validation of the rock mass behaviour model, to develop guidelines for the dimensioning of the surface crown pillar at the Mine. The width of the mining panels or stopes to be used in safe design were determined, as shown in Figure 4.

## INSTRUMENTATION

In order to validate the model, it was necessary to develop instrumentation capable of measuring crown pillar deflections before, during and after blasting of a stope. At the Mine, the flat back blasthole method of mining has been used since 1983. In general, the stopes are set up at 22 m widths in the strike direction, with sub-levels every 20 m. A 4 m x 4 m slot raise is used for mass blasting. A typical layout of the flat back blasthole stope is shown in Figure 5.

Consideration of several readily available types of instruments, both mechanical and electronic, showed these would not be suitable due to potential breakage during blasting and due to reasons of inaccessibility after blasting. Therefore, a mechanical device was manufactured by the Mine, using a 50 mm diameter dial gauge as the primary movement monitoring device. Dial gauges are relatively inexpensive and, by trials carried out underground in the available or aided lighting, could be read from distances of up to 50 m with an ordinatry transit telescope [Strata, 1988].

The design of the instrument provides for measurement of deflections at the surface crown pillar relative to a fixed point some 8 to 10 m above the crown pillar, Figure 6. As the collar assembly moves in response to the surface crown pillar movement, the dial gauge reads the magnitude relative to the fixed grouted end of the central cable bolt. By measuring the ground movement at the surface immediately above the cable bolt, the absolute movement of the crown pillar can be obtained.

A prototype instrument was installed in Stope 5 at the Mine at the 55 m level and readings were commenced in early April, 1988. The slot raise was completed near the end of June, at which time the cumulative movement downwards was just over 1 mm. After blasting in mid-July, the cumulative deflection of the crown pillar was about 7 mm. The instrument was destroyed in mid-August when the stope was backfilled. Just prior to backfilling, the movement had reversed about 2 mm, resulting in a net downwards deflection of about 5 mm, (uncompensated for any ground movements above).

## COMPLETE RECOVERY

Complete recovery of the surface crown pillars at Les Mines Selbaie is currently underway through sub-level caving. It has been estimated [Les Mines Selbaie, personal communication, 1989] that the potential profit from complete recovery is \$1.7 million.

In the sub-level caving method of mining, the ore is fragmented using blast holes drilled upwards from drifts excavated into or beneath the orebody. The intent is to induce gravity flow of the fragmented material into the drift heading, from which the ore can then be extracted selectively, Figure 7. The success of the draw is determined by the flow characteristics of the ore and overburden. Dilution of the ore stream occurs due to entrainment of overburden materials; therefore, close control of draw is necessary to prevent excessive dilution.

Ore recovery is to be effected through 7.0 m or 3.5 m wide by 13.5 m high drifts from the footwall, with 5 m wide rib pillars left in previously placed backfill located immediately beneath the surface crown pillar. Ore dilution is to be limited to 50 per cent. Complete recovery of the ore in the remnant pillars is also to be attempted.

It has been shown, from a study of a number of cases, that surface subsidence is more or less symmetrical about the vertical axis of an underground opening, forming a trough-like depression whose shape can be related to the shape of an inverted normal probability curve [Peck, 1969]. There is also a direct relationship between the volume of lost ground, the size of the underground opening, and the amount of bulking, ß, of the material in the vicintiy of the excavation. The volume of lost ground may range from a few per cent of the excavation volume to 100 per cent when the ground is allowed to invade the opening, as in a block caving method of mining. The results of calculations of predicted surface subsidence, as a function of the overburden bulking factor, ß, are given in Table 3.

Janelid and Kvapil [1966], Yenge [1980, 1981] and Peters [1984] present models describing gravity flow of ore in sub-level caving. The drawing action of columns of coarse materials such as crushed rock is best described by "draw ellipsoid flow" models [Janelid and Kvapil, 1966]. With sub-level caving, the problem of recovery versus dilution is potentially very serious [Yenge, 1980], because the extraction of the volume occupied by the broken ore is complicated by the sequential overlap of the zones of flow of the ellipsoids of motion, and because the ore is surrounded on three sides and above by waste materials. With sub-level caving, the tonnage draw after blasting a slice is greater than that of the actual ore blasted. Excessive draw and dilution can be prevented by efficient draw control and slice design.

For a given ellipsoid of motion of volume $E_n$, and height $h_n$, there is a limit ellipsoid of volume $E_g$ and height $h_g$. The material between these two boundary ellipsoids will loosen and displace but will not reach the discharge point.

For practical purposes, $E_g \approx 15E_n$. The shapes of these elliposids are described by eccentricity, e, where

$$e = \frac{(a_n - b_n)^{1/2}}{a_n} \qquad (1)$$

and $a_n$ and $b_n$ are the major and minor semi-axes of the ellipsoid. In practice, e lies between 0.92 and 0.96. Calculations for the Selbaie mine [Strata, 1988] show that for 100 per cent draw, the height of the ellipsoid of motion, $2a_g$, is greater than the thickness of overburden; therefore, breakthrough of the sub-level caving operation to the surface is predicted which would result in substantial and intolerable surface subsidence, especially for multiple drift excavations. Therefore the draw must be limited to an amount which will prevent surface breakthrough.

Predictions of the ellipsoids and surface subsidence were made on the basis of a large strain 3-D finite element program . The model for the analyses consisted of 40 m of overburden, with openings 3.5 m and 7.0 m wide by 13.5 m high. The vertical displacements at various depths below ground surface, for a 7 m wide by 13.5 m high opening, are shown in Figure 8.

The 3.5 m wide surface crown pillars recovered so far have not lead to soil caving. In one stope examined in May 1989, a stable hemispherical dome formed across a span of almost 13 m within the boundary soil unit. The stability is attributed to the high in situ density of the silt, its low water content, and negative pore pressures.

## CONCLUSIONS

The research into the support, and if possible the complete recovery of, the surface crown pillars at Les Mines Selbaie shows that it is possible to have a stable pillar or to recover the enriched upper ore zone by control caving techniques. Both analytical and numerical models have been developed and verified, and prediction curves have been prepared for a number of possible extraction alternatives. The key factors to successful complete recovery of an altered pillar are: in situ density, negative pore pressures and arching capability of the boundary unit soils; the bulking factor of the overburden ; and control of ore dilution upon extraction. It is important that the mine remain fully dewatered and depressurized for soil arching to occur, and the control caving technique of surface crown pillar recovery to be safe and effective.

## ACKNOWLEDGEMENTS

The research at Les Mines Selbaie was sponsored by CANMET Contracts: 15SQ.23440-5-9017 and 035Q.23440-7-9195. The Authors are very grateful to the staff and management at Les Mines Selbaie and would like to acknowledge in particular the guidance and assistance provided throughout the research study by Messrs: R. Bryce, Luc Closset, Jean-Luc Chouinard and Denis Caron.

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Table 1. Quick Field Classification System

RANKING SCHEME AND AVERAGE VOLUME OF EACH ROCK TYPE

| RANK | CHARACTERIZATION | FIELD METHOD OF DETERMINING RELATIVE STRENGTH | AVERAGE VOLUME (%) |
|---|---|---|---|
| 1 | Very Strong | > 1 hammer blow to break | 9.7 |
| 2 | Strong | 1 hammer blow to break | 17.4 |
| 3 | Moderately Strong | 5 mm Indentations with pick | 12.4 |
| 4 | Moderately Weak | Cannot be cut by hand | 6.3 |
| 5 | Weak | Crumbles under firm blows with pick | 14.1 |
| 6 | Very Weak | Broken in hand with difficulty | 9.4 |
| 7 | Very Stiff | Indented by fingernail | 6.6 |
| 8 | Stiff | Cannot be moulded in fingers | 5.7 |
| 9 | Firm | Difficult to mould in fingers | 7.2 |
| 10 | Soft | Easily moulded in fingers | 10.4 |
| 11 | Very Soft | Can be squeezed between fingers | 0.8 |

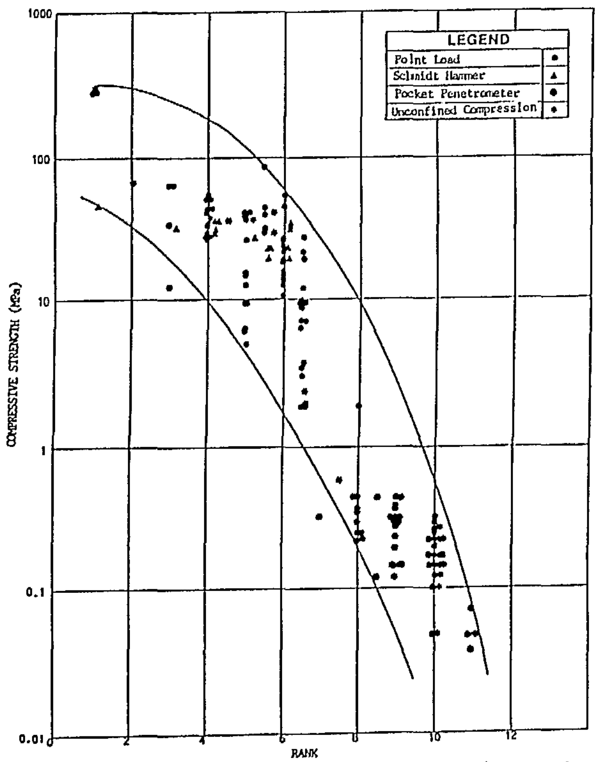


Figure 1. QFC Rank vs. Compressive Strength

Table 2. Range of Values - Key Properties

| Key Properties Measured | Overburden | Material Tested: Soil-Like Rock | Weak Rock | Strong Rock |
|---|---|---|---|---|
| Thickness (m) | 37.5 - 41.3 | - | - | - |
| Volume (%) | - | 30 | 40 | 30 |
| QFC Ranking | - | 7 - 11 | 3 - 6 | 1 - 2 |
| Moisture (%) | 6 - 12 | 5 - 25 | - | - |
| Liquid Limit (%) | Not Plastic | 19 - 25 | - | - |
| Plasticity Index (%) | NP | 1 - 7 | - | - |
| Unit Weight (kN/m³) | 22.6 - 23.1 | 18.6 - 25.9 | 25.6 - 33.0 | 27.0 |
| Slake Durability Index ($I_2$) -% | - | - | 68.9 - 98.3 | - |
| Friction Angle (°) | 34.7 - 35.2 | 40.6 - 40.9 | - | - |
| Cohesion (kPa) | 0 | 0 | - | - |
| Compressive Strength (MPa) | | | | |
| Point Load | - | 1.9 - 5.0 | 3.1 - 48.1 | 106 - 288 |
| Schmidt Hammer | - | - | 23.0 - 35.9 | 46.6 |
| Penetrometer | - | 0.04 - 7.43 | - | - |
| Unconfined | - | 0.10 - 0.59 | 2.0 - 35.9 | 68.5 - 308.0 |
| Tangent Modulus (GPa) | - | 0.03 - 0.20 | 12.4 - 24.1 | 28.3 |

Table 3. 3-D Analysis of Settlement as Function of Bulking Factor, B

| B | Max. Settlement $d_{max}$ (mm) | Settlement @ Office Corner (mm) |
|---|---|---|
| 1.0 | 1330 | 460 |
| 1.05 | 760 | 270 |
| 1.10 | 190 | 70 |

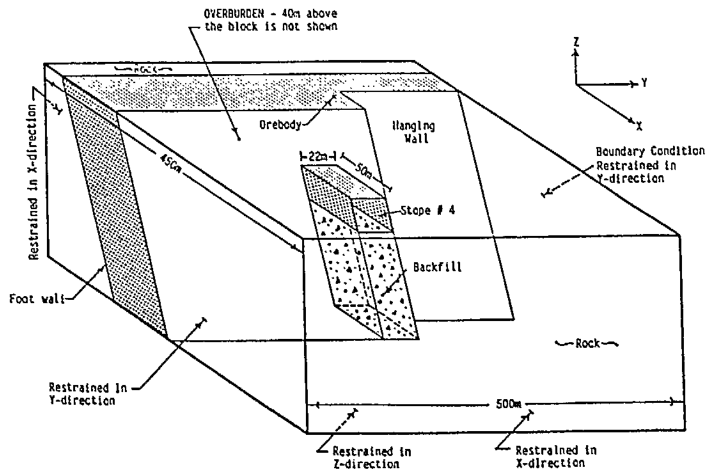


Figure 2. Model of 1983 Cave-in in Stope 4

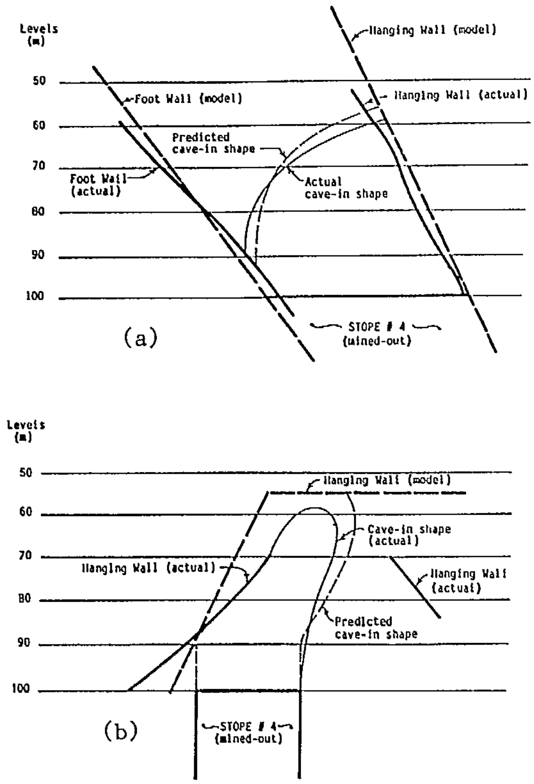


Figure 3. Predicted vs. Actual Failure
(a) along strike
(b) normal to strike

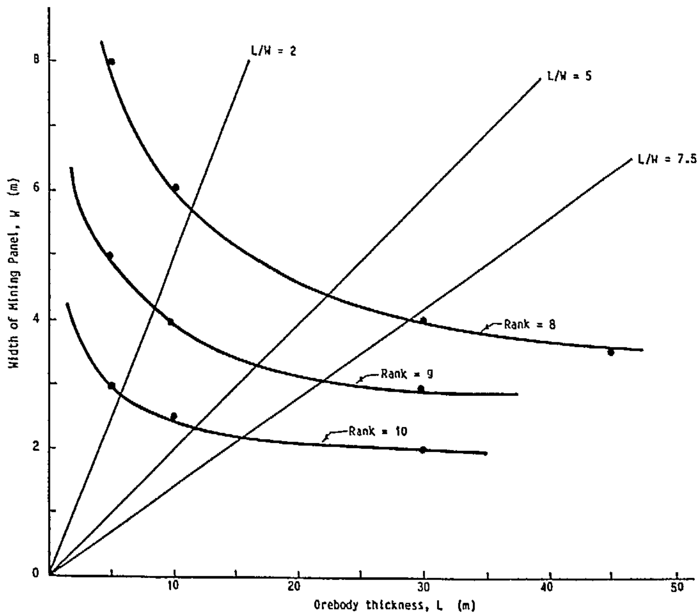


Figure 4. Design Guidelines

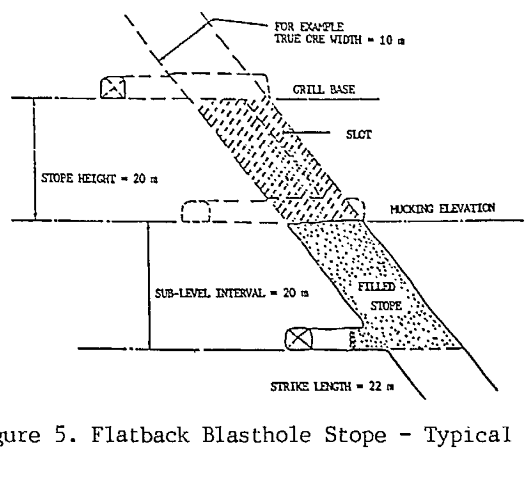


Figure 5. Flatback Blasthole Stope - Typical

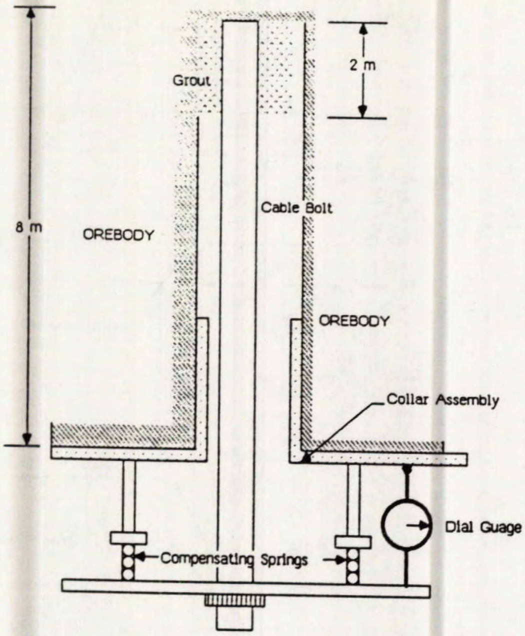


Fig.6 SCHEMATIC OF CROWN PILLAR EXTENSOMETER

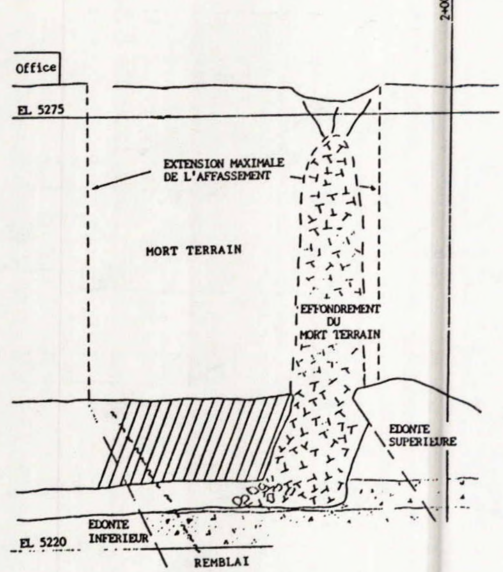


Figure 7. Sublevel Caving

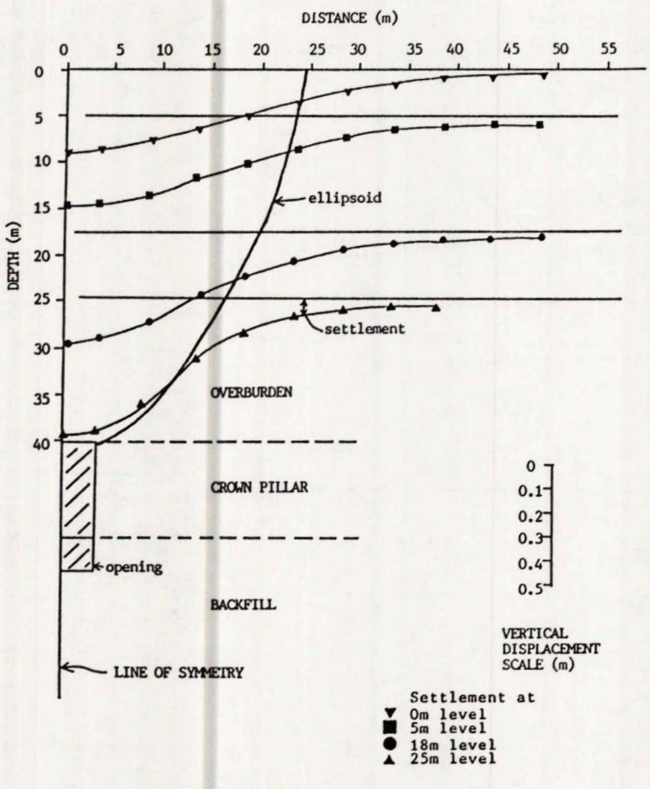


Figure 8. Subsidence Predictions for
7 m wide by 13.5 high opening

# MONITORING ROCK MASS DEFORMATION USING TIME DOMAIN REFLECTOMETRY

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## ABSTRACT

Time Domain Reflectometry (TDR) is an electrical pulse testing technique originally developed to locate breaks in power transmission cables. Recently, this technique has been adapted to monitor caving of rock induced by underground mining. Coaxial cables are grouted into a rock mass which is expected to cave and the progressive rock movement deforms the cable. Consequently, there is a progressive change in cable deformation which produces changes in TDR pulse reflection signatures, and these changes can be quantitatively related to the location and magnitude of rock mass movements.

## RÉSUMÉ

La méthode de réflectométrie en domaine de temps est une méthode d'essais par impulsions électroniques développée à son origine pour localiser des bris dans les lignes de transmission. Récemment, cette méthode fut adoptée pour la surveillance de l'effondrement de massifs causé par l'activité minière souterraine. Des câbles coaxiaux sont cimentés dans un massif rocheux où l'effondrement est attendu; le déplacement progressif du roc déforme le câble. Par conséquence, il y a un changement progressif dans la déformation du câble, ce qui produit des variations dans les tracés de réflectométrie dans le domaine du temps. Ces changements peuvent être reliés quantitativement au point et au degré de déplacement du roc.

## TDR OPERATING PRINCIPLE AND INSTALLATION

A TDR installation consists of a TDR cable tester and a cable grouted into the rock mass as shown in Figure 1. Ultrafast rise time voltage pulses are sent down along the cable from the tester. Cable defects such a crimps, short circuits or breaks cause reflected pulses which are detected by the tester. Pulse reflections from all changes in geometry along the cable are superimposed on the input pulse to form a reflected TDR signature. Consequently, the signature observed on a cable tester cathode ray tube (CRT) screen consists of many individual reflections associated with localized deformations along the cable. The characteristics of a TDR signature are determined not only by the magnitude of cable deformation but also by the type of cable defect (crimp, shear, break, abrasion, etc.).

Assuming a constant pulse propogation velocity, the distance to the cable defect is proportional to the elapsed time between initiation of the voltage step pulse and the arrival of the returned voltage. The cable tester measures this time in units of distance so it is possible to determine the location of a cable defect and the type of cable defect by inspection of the recorded TDR signature.

Features of a TDR coaxial cable installation are shown schematically in Figure 1. Prior to installation, the cable is crimped at 6 m (20 ft) intervals to optimize resolution and an appropriate connector (e.g., UHF) is attached to the top-of-the-hole end. The cable is connected to a TDR cable tester via a special 50-ohm connector cable and a reading is made to ascertain that all crimps will be recorded. The crimps must be deep enough to ensure a strong reflection but no so deep as to cause a short circuit in the cable. After being adequately crimped, the cable is attached to an anchor, lowered down hole, and bonded to the surrounding rock with an expansive cement grout that is tremied into the hole. Recommended installation procedures are presented by Dowding et al. (1986).

The capability of distinguishing cable crimps from cable breaks, shears, abrasions, etc. provides a convenient means for improving the accuracy with which cable defects can be located. Without reference crimps along a cable, accuracy is on the order of 2% of the distance from the tester to a cable defect so that a defect at a distance of 30 m (100 ft) can only be located to within 0.6 m (2 ft) of its actual position. If a cable is crimped at selected intervals, however, a set of reference reflections is recorded each time the cable is tested. The locations of cable defects which are created by rock mass movement can then be determined with respect to these references. If reference crimps are made in a cable every 6 m (20 ft), the accuracy with which defects can be located becomes 12 cm (5 in).

## DISTINGUISHING CABLE DEFORMATION

Significant recent developments in the interpretation of TDR signatures are presented in Dowding et al (1989) with respect to quantifying changes in TDR signature magnitude due to cable deformation as well as distinguishing cable shear from cable extension. Laboratory tests (Su, 1987) have shown that shearing of a cable produces a short voltage reflection spike, whose amplitude increases in direct proportion to the amount of deformation as shown in Figure 2b. On the other hand, extension causes only a subtle, trough-like voltage reflection that increases in length as the cable is extended as shown in Figure 2a.

Particularly significant is the difference in reflection signature for tensile failure versus shear failure. As shown in Figure 2, extension necking produces a small negative but extended voltage reflection trough just before the large positive open circuit reflection. On the other hand, shearing produces a distinctive negative voltage reflection spike before the large positive open-circuit reflection.

## APPLICATION TO LONGWALL MINING

TDR reflection traces shown in Figure 3b were obtained from a cable installed over Longall Panel 1 of the Old Ben No. 24 Coal Mine in Benton, Illinois (Wade and Conroy, 1980). The cable was installed just prior to undermining of the hole location so that the cable shortening and reflection amplitudes at the bottom of the cable are associated only with immediate, near-mine strata movement, fracturing and caving as the longwall face was advanced past the hole.

The installation consisted of a 183 m (600 ft) long 22.2 mm (0.875 in) coaxial cable which was grouted into a vertical hole through the strata shown in Figure 3a. The numerous initial TDR reflection spikes are associated with the calibration crimps made in the cable during installation. The magnitude of these spikes near the top of the hole did not change until the face was 37m (120 ft) past the drill hole location. At that point, fracturing and caving of strata had propagated up to within 30m (100 ft) of the surface.

The TDR traces in Figure 4 were obtained from a cable installed over Longwall Panel 2 of the Old Ben Mine (Conroy, 1983). The installation in this case was located approximately 244m (800 ft) from the longwall face, and thus, unlike the previous example, the cable was subjected to strata movements in front of the mining face. The distance between the cable hole location and the longwall face is given for each signature with the convention that distance is positive as the face approached the cable and negative after the face was past the cable.

Increasing local negative reflections in Figure 4, marked as (1) at a depth of 45 m (150 ft), indicate increased shearing perpendicular to the cable as the face approached. Shearing failure, marked as (2) at a depth of 105 m (350 ft), shows that shearing rather than extension severed the cable. As shown in Figure 2, the signature for shear failure terminates with a negative voltage reflection spike just before the large, positive, open-circuit reflection. On the other hand, tensile failure, marked as (3) at a depth of 36 m (120 ft), shows only a slight negative reflection.

## APPLICATION TO BLOCK CAVE MINING

Perhaps the first use of TDR to monitor rock mass deformation was accomplished by Panek and Tesch (1981) in which coaxial cables and other borehole instrumentation was installed in the San Manuel Mine, San Manuel, Arizona. For one of the instrumented panels, measurements were made in three boreholes drilled from two mining levels to intercept an approaching, extensive cave (Figure 5). Mining progressed mainly westward along this limb of the orebody and when data acquisition began late in January 1978, ore was being withdrawn from the areas designated as blocks 1, 2 and 3 with blocks 4 to 7 subsequently undercut and placed in production during the period of monitoring. At the time that data acquisition began, the end of cable TDR1 was about 63 m (205 ft) from the nearest caving boundary (block 3).

The installation consisted of a 100 m (300 ft) long, 12.5 mm (0.5 in) coaxial cable which was grouted into a hole oriented 20 degrees up from horizontal. Four recordings of significance are shown in Figure 6. Data acquisition was hampered by "noisy" recordings thought to be due to electrical interference; high-voltage trolley lines and an underground power distribution center were located in the vicinity. On several occasions special arrangement was made with the mine staff to shut off power for 1/4 hour shortly before the morning shift change, and the TDR measurements were taken during that period. The benefit was minimal; equally good results were achieved by waiting to record until, during a lull in the level of interference, the oscilloscope trace improved to the degree that the distance marker reflections could be identified.

The 10/29/78 record shows a crack developing about 1.5 m (5 ft) from the end of the cable. Because of the electrical interference, cracks at intermediate locations along the cable were difficult to detect, especially up to 57 m (187 ft). However, two severings of the cable, 78 m and 72 m (255 and 233 ft) are clearly exhibited on the records of 1/16/80 and 2/18/80, following the undercutting of block 7, which was completed 1/2/80.

## SUMMARY

Examples involving caving in both coal measure strata and hard rock have been presented which show that monitoring of TDR reflections along a crimped coaxial cable is an effective technique for locating and monitoring strata displacements along boreholes provided that the cable is adequately attached to the strata. Based on the results of recent research it has been possible to develop a quantitative relationship between cable deformation and changes in TDR signatures. It is now possible to monitor not only the location of strata displacements but also the type and magnitude of displacements.

For holes deeper than 30 m (100 ft), TDR cables are less expensive and easier to install than rod or wire type multiple-point extensometers. Data acquisition is simple and a permanent record can be obtained each time a reading is made. Since it is common for exploratory drillholes to be backfilled with cement grout, it would be a simple matter to install crimped coaxial cables in the holes for purposes of monitoring subsurface caving.

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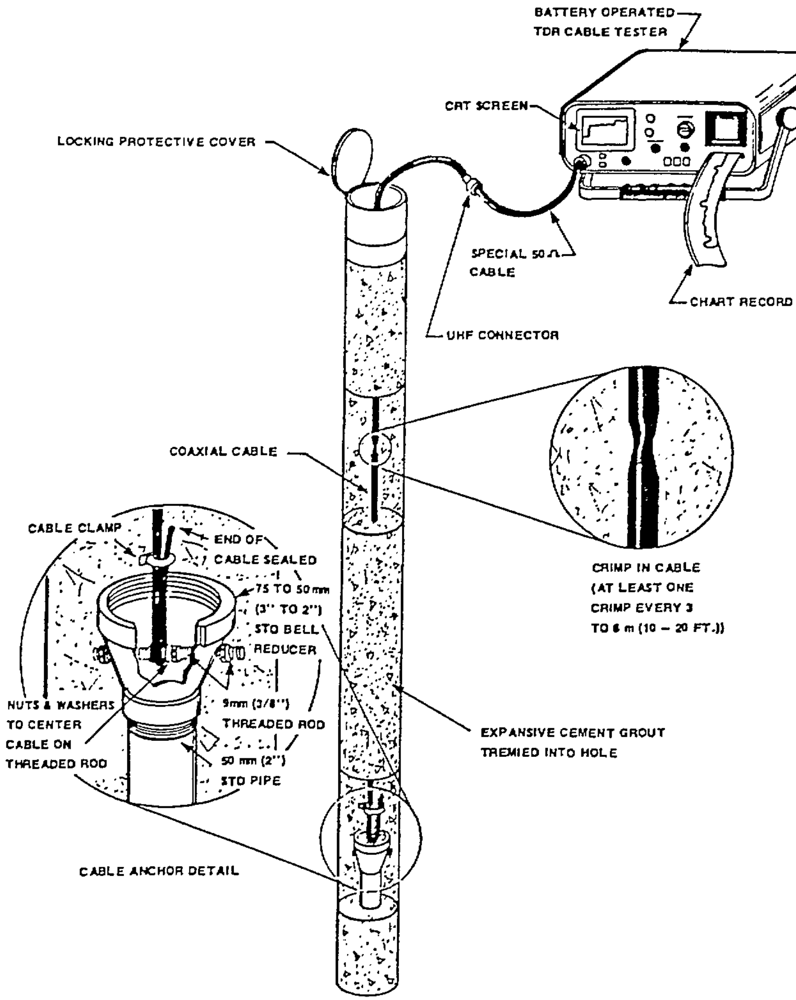


Figure 1 Schematic TDR Installation

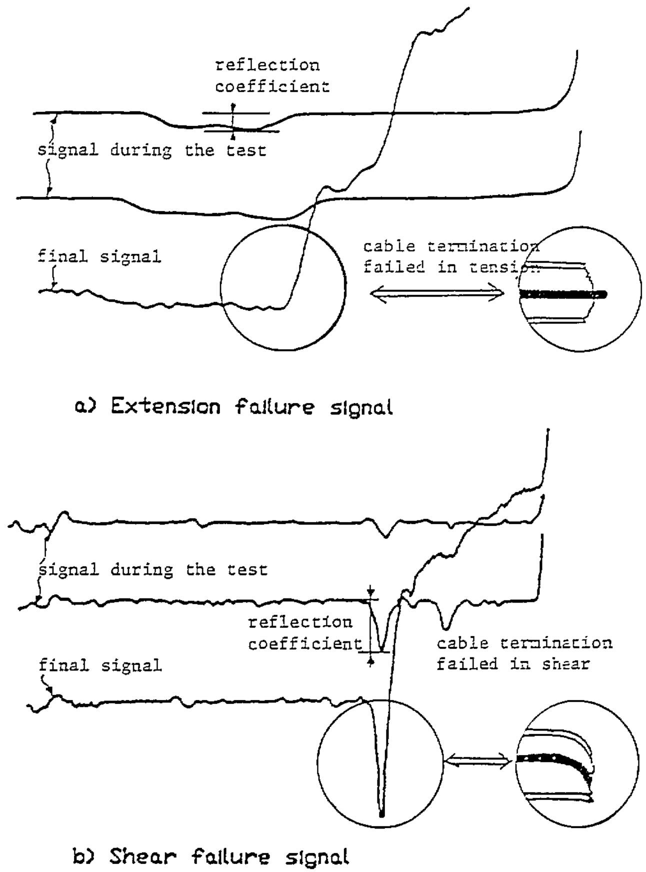


Figure 2 Comparison of Termination Conditions (Su, 1987)

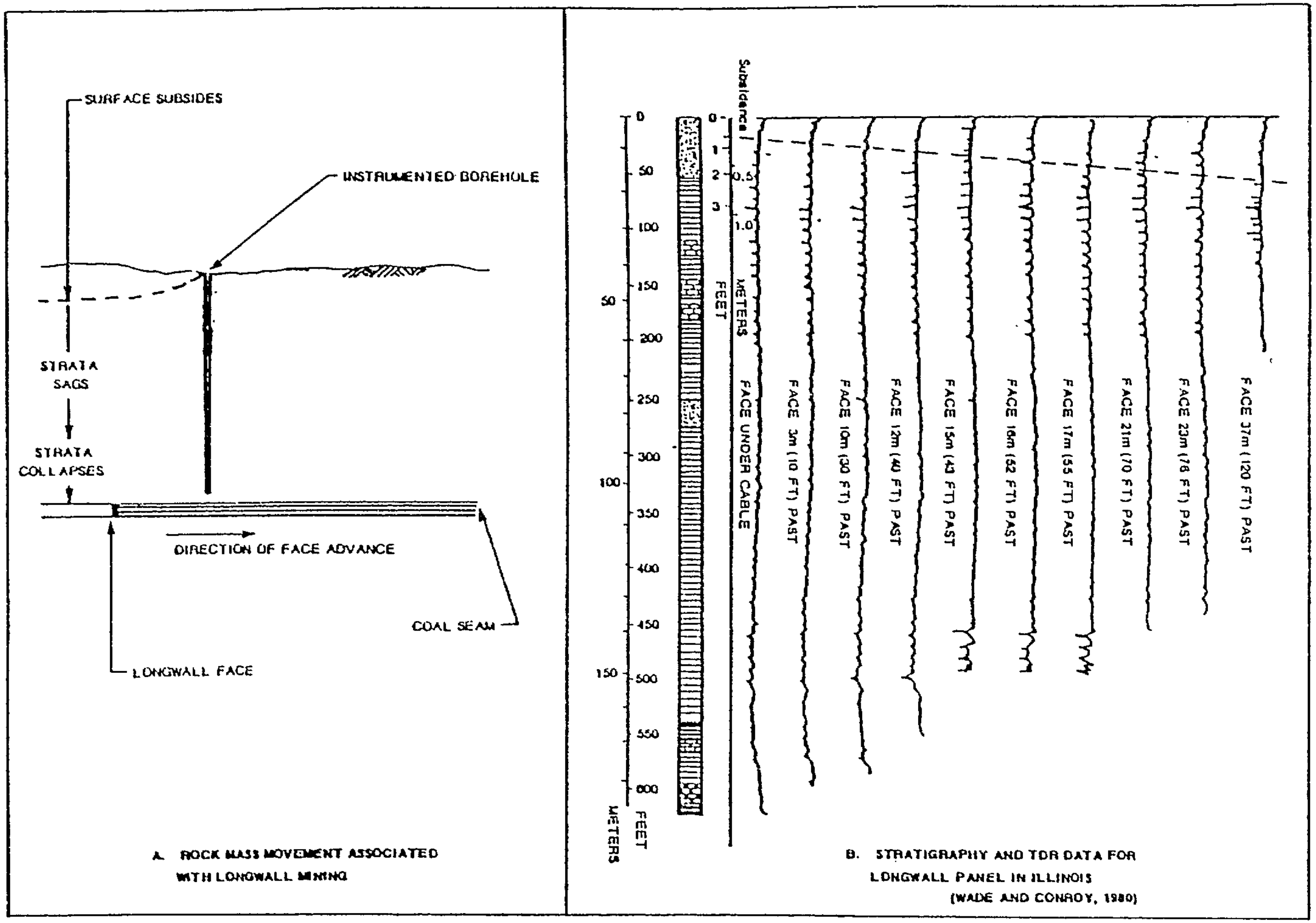


Figure 3 Strata Movement and TDR Signatures for a Longwall Mine

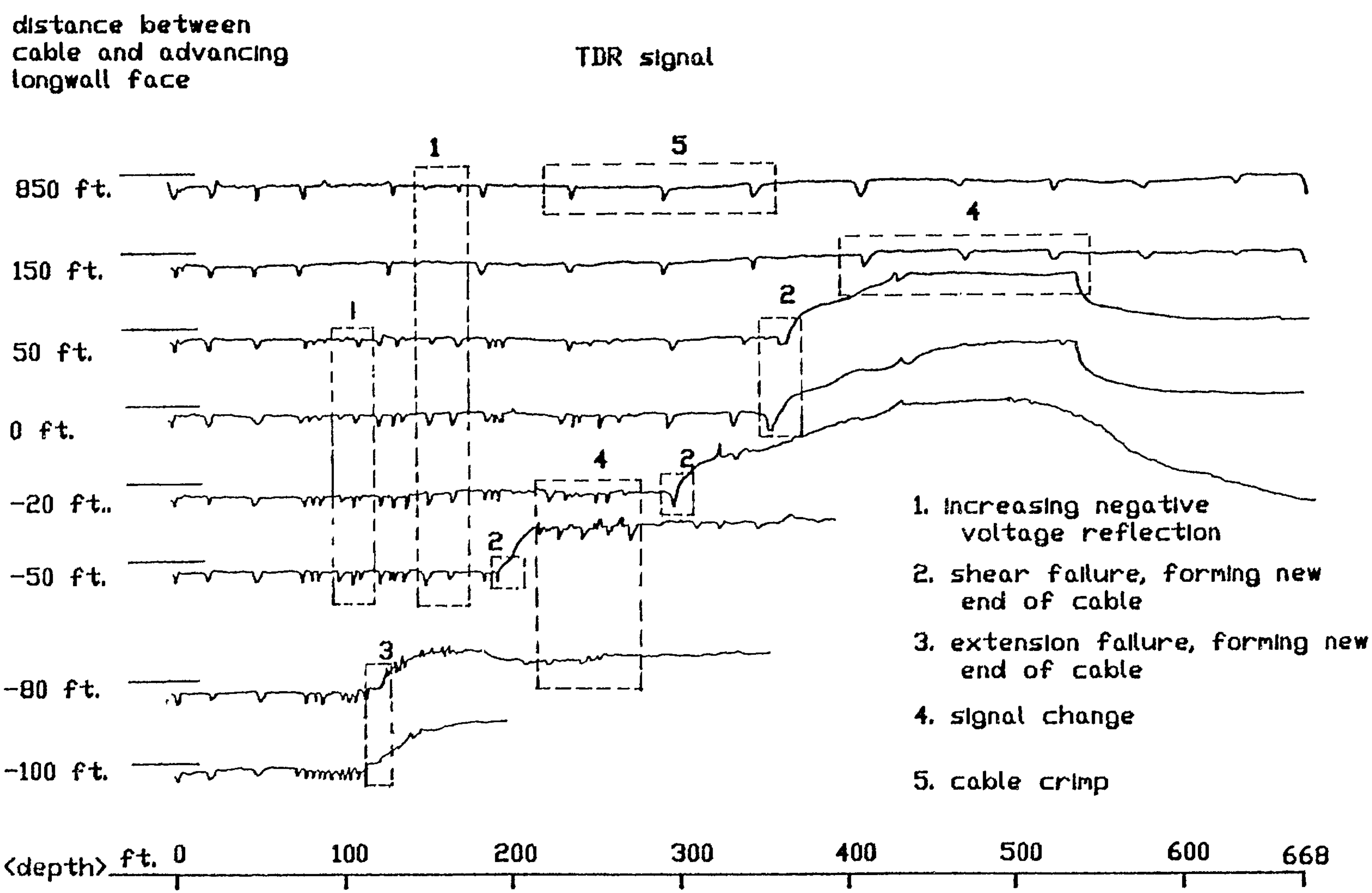


Figure 4 Old Ben Mine Panel 2 TDR Signatures (Conroy, 1983)

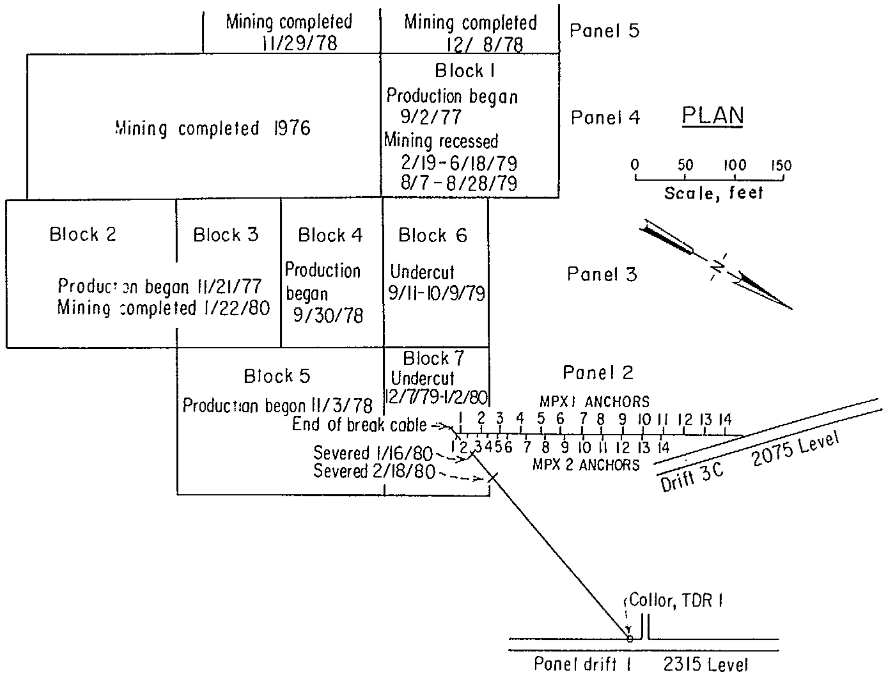


Figure 5 TDR Monitoring Location, San Manuel Mine (Panek and Tesch, 1981)

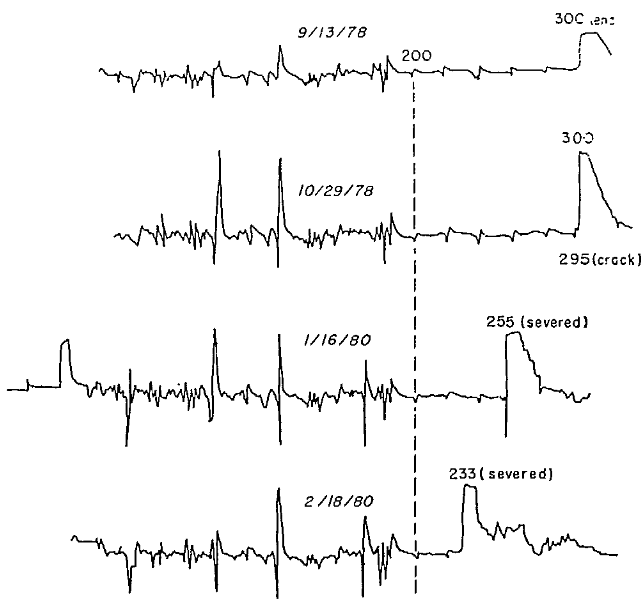


Figure 6 TDR Signature, Hole TDR1 (Panek and Tesch, 1981)

# THE FORGOTTEN PROBLEM

## SESSION III

# LE PROBLÈME OUBLIÉ

# CONCEPTS ON DEALING WITH ABANDONED MINE HAZARDS

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## ABSTRACT

A worldwide problem exists with abandoned mine hazards. The most difficult to deal with are the "hidden hazards" associated with surface crown pillars. Although some current legislation may deal effectively to prevent future hazards, a legacy of problems remains from the past.

The issue is best dealt with through a committee system involving officials from different jurisdictions. A successful approach is to collectively develop an inventory of abandoned mines, an emergency response plan, and a remedial work plan.

## RÉSUMÉ

Il existe des problèmes mondiaux associés aux risques de mines abandonnées. Les plus difficiles à traiter sont les "risques cachés" associés avec les piliers de surface. Bien que la législation actuelle peut prévenir des risques à l'avenir, une série de problèmes nous a été léguée.

La meilleure approche est de former un comité qui consiste de personnes ayant diverses qualifications professionnelles pour résoudre les problèmes de mines abandonnées. Un tel comité peut développer un inventaire de mines abandonnées, un plan de réponse d'urgence, et un plan de travail réparateur.

## INTRODUCTION

Over the last several years, a number of crown pillar failures have been reported in various parts of the world. Such collapses have generally been treated by the news media and public officials as isolated incidents that posed only a temporary threat to public safety. Recent studies however, have found that housing developments, schools, highways, railways, etc., have been built over areas of potential collapse. In general, the public assumes that civic structures built on or close to bedrock are free of risk. Indeed there are very few municipal requirements for checking the security of bedrock prior to construction.

Many municipalities are built within areas of high mineral potential and mining activity. Technical advances in mining exploration methods over the past forty years, coupled with a better understanding of local geological conditions have in many cases resulted in new discoveries within these municipalities. As a result, municipal officials must struggle to develop new land for community expansion. A dilemma is created when this expansion is directed towards abandoned mining lands.

Developers are generally unaware of the presence of underground workings and the need for remedial measures. Also, in many cases very little technical data is available to enable an evaluation of the risks associated with these abandoned minesites. Recent advancements in the science of rock mechanics demonstrate that a greater awareness of structural integrity of mine workings is required by municipal planners.

## TYPES OF ABANDONED MINE HAZARDS

Abandoned mine hazards can usually be grouped into four general categories: openings to surface, surface structures and machinery, toxic substances, and areas of potential collapse.

Openings to surface include shafts, raises open pits and trenches. These features are the ones that are most frequently reported to public officials. They are generally easy to recognize and are obvious hazards to public safety.

Surface structures and machinery are a less obvious hazard in the eyes of the public and are seldom reported. Tourists, hikers and children can commonly be found climbing on various old structures and equipment.

Toxic substances may include discarded reagents, some forms of mine tailings, and low level radioactive waste. Reagents are more easily recognized as hazards by adults that by children. The public is generally oblivious to hazards that may be associated with tailings and low level radioactive waste.

Areas of collapse are generally caused by weak surface crown pillars, buried storage tanks, and unstable tailings dams. The stability of surface crown pillars is probably the single greatest concern facing officials today when dealing with mine hazards.

## A LEGACY

At closure most mine surface expressions are securely fenced to prevent public access. With time the fences deteriorate or are removed for a variety of reasons. In addition to this, adventure seekers and history buffs are known to climb fences to inspect old mining machinery and buildings. One way or another people often gain access to abandoned minesites and become exposed to risk of personal injury.

While current legislation and municipal by-laws may deal, in some areas, effectively with today's mine closures, it is the legacy of past mine closures that presents the greatest problem to public safety.

In dealing with the legacy, one finds that problems have arisen because of: incomplete records of mine operations, weak surface crown pillars, no requirements for owners to inspect and maintain fences, insufficient zoning requirements to restrict land use, and so on.

Lack of public understanding and awareness of "hidden hazards" posed by surface crown pillars is one of the most common problems faced by officials who must deal with abandoned in hazards. Because of this, civic structures can sometimes be found on top of, or adjacent to abandoned mine workings.

Often the greatest part of the problem in dealing with this legacy is the lack of communication among mine operators, government officials, and the land users.

While mine development and operation is for the most part in the realm of the mine engineer, experience has shown that the issue of abandoned mine hazards is best dealt with by a multi-disciplined approach. The Province of Ontario has taken this route through the formation of an Interministry Abandoned Mine Hazards Committee. Senior officials representing the Ministries of Northern Development and Mines, Natural Resources, Labour, Environment, and Municipal Affairs serve on this committee. Through this group a coordinated approach can be directed toward abandoned mine hazard issues. Such matters as safety evaluation, future mining potential, mine engineering, rock mechanics, land use planning, environmental concerns, and municipal zoning are some of the topics that are dealt with. This committee oversees a coordinated approach of the development of an inventory of abandoned mines, emergency response plans, and remedial work programs.

## INVENTORY OF ABANDONED MINES

In Ontario, information on abandoned mines is collected by government field office staff and through contract work by consulting firms. To date, a number of northern mining municipalities in the province have been studied through contract work. Terms and conditions for such projects are drawn up by local committee consisting of officials of Ministries of Northern Development and Mines, Labour, Natural Resources, and municipal representatives.

A system has been developed whereby segments of the community have been chosen for detailed investigation. A report and maps are produced which describe mine workings in a manner that is useful to municipal planners. Maps are prepared at the same scale as used by the town for planning purposes. A successful outcome of this approach has been the development of the concept of "Areas Reserved for Caution Status". An "Area Reserved for Caution Status" has been defined as an area where, in the opinion of the consultant, the use of the land for purposes other than mining is potentially hazardous.

Figure 1 illustrates a typical inventory map prepared for a municipality. Several "Areas Reserved for Caution Status" are outlined with respect roads and buildings. Site 4 represents a suspected weak surface crown pillar. As this site was located at one of the most heavily used traffic intersections in the community, the town engineering department scheduled a geotechnical investigation of the surface crown pillar. This work determined that remedial action was required. The town was able to schedule an orderly work program that presented minimum disruption to traffic. Without this planning document, failure of the surface crown pillar could have occurred resulting in possible personal injury and property damage.

An inventory of abandoned mines is most useful when it is readily accessible for use by government officials. Although it is essential to have full documentation of mine workings in the forms of mine plans and cross sections, it is also necessary to have carefully planned data management system. Such a system should include basic information on the location and hazard rating of abandoned minesites. Details on location of relevant mine documents, information on past remedial work, date and name of official who made the last inspection, and other such technical information is important in cases of emergency, and must be accessed rapidly. Laurentian University has recently completed a prototype computerised database system for the Ontario Government (Koczkodaj, 1989).

## EMERGENCY RESPONSE PLAN

Failure of surface crown pillars may be catastrophic. When such failures occur within communities a great deal of stress is placed on municipal officials, and the general public can be placed in a state of panic. The business community may suffer a loss in confidence which could cause a downturn in regional economic development.

The preparation of an emergency response plan can help prevent a mine cave-in from turning into a disaster. Approaches along the lines of those suggested by Mindszenthy et al (1988) can help prepare government officials for crisis management. With an up to date inventory of abandoned mines, officials are able to better anticipate where potential problems areas could occur within a municipality. Armed with this information a number of "what if" questions can be raised and possible solutions suggested and "Crisis Information Logs" can be prepared. These logs would contain lists of facilities that could be impacted in the event of a crisis, and the names and phone numbers of officials to be contacted.

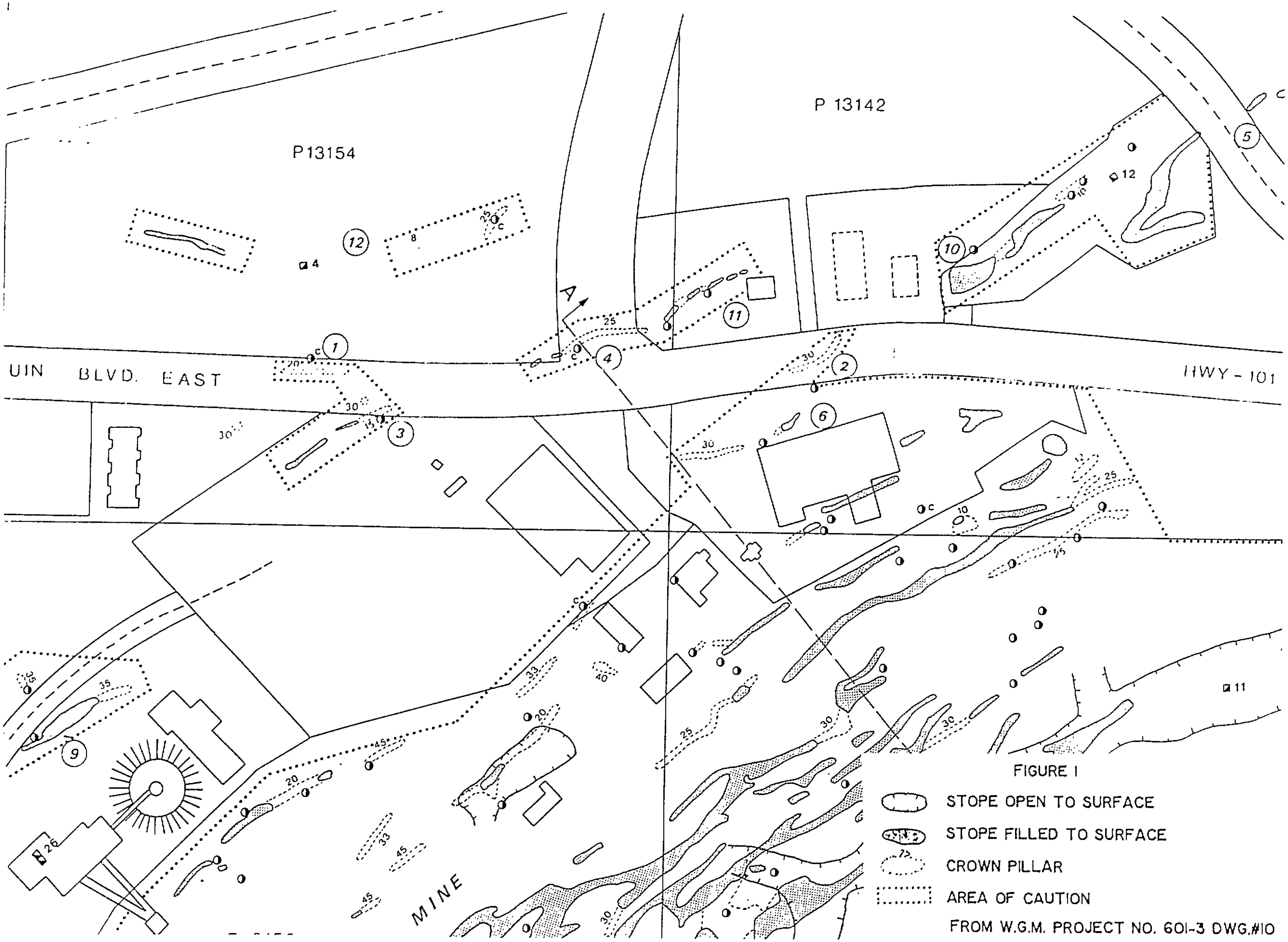
P13154
P 13142
UIN BLVD. EAST
HWY - 101
MINE
STOPE OPEN TO SURFACE
STOPE FILLED TO SURFACE
CROWN PILLAR
AREA OF CAUTION
FROM W.G.M. PROJECT NO. 601-3 DWG.#10

FIGURE I

## REMEDIAL WORK PROGRAM

Remedial work programs are developed to deal with matters of public safety, public health, impact on the environment, and aesthetic objectives. While priority must be placed on public health, safety and environmental concerns, care must be taken to not to unduly jeopardize future mineral exploration or mine development initiatives. With changes in economic conditions, some abandoned mines may have potential to go back into production. If, for example, a remedial program has allowed a mine shaft to be filled in, a permanent death blow could be dealt because the cost of regaining entry to the mine workings may far exceed that allowed for exploration.

A common misconception is to allow remedial work programs to be developed solely by mining specialists. While technically correct, plans could be drawn up, that may not necessarily meet the long term needs of the community. The most successful approach is the synergistic approach. In Ontario, remedial work plans are developed through local committees consisting of officials from various , provincial ministries along the municipal officials. Outside consultants sit as committee members for some projects. Without the multi-disciplined approach funds could be expended on costly work projects that are not necessary. For example, maintaining an access road that is no longer required by the community. A committee consisting, of only mining specialists may become so focused on the engineering merits of a project that other aspects may be overlooked such as the need to conduct a roads needs analysis for the community as part of the planning exercise.

The most effective remedial work projects are those which have been planned and carried out by a creative working group that can develop a balance between need for technical information and the ultimate budget cost. Commonly most technical information is gathered through inspection of mine document and talking to mine officials. In more complex projects geophysics and drilling is employed. Ground penetrating radar surveying methods have successfully been used in Ontario (Carter et al, 1988). Video cameras have been lowered into abandoned mine workings in a number of projects to gather visual information on ground conditions. For one project, video equipment and other technical devices were transported into flooded mine workings by means of a remotely controlled miniature submarine.

A common problem when dealing with abandoned mine hazards is the difficulty in establishing priorities. Generally those hazards closest to built up areas are dealt with first. Recreation tails and forest access roads are constantly being developed and in some cases traverse abandoned mine workings. It is very difficult to plan for such situations. To help establish priorities for their Abandoned Mine Program, the State of Wyoming developed a computer program to analyze known hazards and define a ranking for work to be done (Fowkes).

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# GEOMECHANICS METHODS EMPLOYED TO EVALUATE NEAR-SURFACE CROWN PILLAR STABILITY AT THE OLD MONETA MINE, TIMMINS, ONTARIO

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## ABSTRACT

Recent geomechanical investigations have been undertaken in Timmins, Ontario by Trow to evaluate the structural stability of several near surface crown pillars. This paper details the work undertaken at the abandoned Moneta Mine in downtown Timmins. The old narrow vein, near vertical shrinkage stope is located under the ONR railway tracks. Adjacent the railway tracks an extension of the same vein was mined to surface and subsequently filled. Subsidence at this location resulted in the loss of five parked buses in 1963. This paper outlines how Trow conducted the geomechanical investigation at the Moneta site. Stability analyses include empirical and numerical methods, and the evaluation of the subsidence potential in the filled stope. Trow's recommendations and instrumentation approach for this site are discussed, and recent improvements in both investigative techniques and instrumentation are presented.

## RÉSUMÉ

Récemment, des investigations géomécaniques ont été entreprises à Timmins (Ontario) par Trow pour évaluer la stabilité de plusieurs piliers de surface. Cet article décrit les détails du travail entrepris au site abandonné de la Mine Monéta, au centre-ville de Timmins. Un vieux chantier sub-vertical excavé dans une veine étroite se situe sous les rails du chemin de fer ONR. Avoisinant ce chemin de fer, une extension de la même veine fut exploitée jusqu'en surface et subséquemment remblayée. En 1963, un affaissement en surface à cet endroit a entraîné la perte de cinq autobus stationnés. Cet article présente les détails de l'investigation géomécanique que Trow a menée au site Monéta. On compte parmi les analyses de stabilité les méthodes empiriques et numériques et l'évaluation d'affaissement possible dans le cas de chantiers remblayés. L'approche de Trow au niveau d'instrumentation de même que leurs recommandations pour ce site sont discutés. Des améliorations récentes dans le domaine des techniques d'investigation et d'instrumentation sont présentées.

## INTRODUCTION

In 1987, geomechanical investigations were undertaken at several abandoned mine sites for the City of Timmins as part of an Ontario government program which in turn sponsored the Municipality. The rock mechanics evaluation, Hunt and Gore, 1987, was a follow-up to an earlier study that identified potentially dangerous or sensitive sites, Watts, Griffin and McQuat, 1986. One such site, referenced as site 7, was the old Moneta Mine, shown on Figure 1.

Mine records, dating to the early 1940's, indicated some of the stoping was narrow vein shrinkage, for which most of the stoping is unfilled. However, a wider extension of the gold bearing vein, labelled stope 1-2, had been mined out and fill; then mined to surface and subsequently backfilled to approximately the existing surface elevation.

A plan and longitudinal section representing the area of concern is shown in Figure 2. Stope 1-1 and 1-2 extend to depth. Stope "2" continues to 4-2 approximately 600 feet below surface. The mine is believed to be flooded generally below the first level.

Mine records were located at the Pamour offices, currently Giant Yellowknife, Timmins Division. Mine records for the Hollinger, McIntyre, Vipond and Moneta mines were generally found to be accurate, particularly with regards to crown pillar thickness and stope width.

Stope 1-2 is of particular interest to those involved in abandoned mine hazards and surface subsidence because this is the site where five parked buses were lost on November 12, 1963 when the fill suddenly and unexpectedly collapsed. The buses could not be retrieved and the stope was refilled. No loss of life occurred. The surface of the 1-2 site is privately owned, commercially zoned, and this provides additional complexity in matters of liability, rehabilitation and municipal affairs in general. The surface rights along the 1-1, 1-5, and 1-7 stopes are owned by the Ontario Northland Railway, and 1-7 borders again on privately owned, commercially zoned property.

The engineer must be aware of the acute nature of the risks associated with property loss and public safety involving abandoned mine hazards. This paper discusses some of these aspects in a later section.

## FIELD INVESTIGATION

Trow's investigative approach to abandoned mine hazards is shown on Figure 3, Hunt, Piciacchia, Gore, 1989. Figure 3 outlines a five phase procedure. The first phase, a desk study, had already been initiated by Watts as explained earlier, and the Corporation of the City of Timmins conducted the majority of the committee and public awareness work under the risk analysis phase.

The chief participation by Trow, in 1987, came through Phase 3 to Phase 5. Phase 3 work began when the stopes at the Moneta site were cross referenced to geodetic maps and surveyed in. Old survey stations are generally difficult to locate; however, this was not the case at the Moneta site wherein old recorded survey pins had been located. Arrangements were made with the property owners for access to the site and safety along the railway right of way.

Air track probes with a Joy drill were used along the railroad portion of the site to confirm crown pillar location and approximate geometry. This work was followed by careful diamond drilling with a Longyear 34 drill and NQ size equipment. Budget constraints restricted the number of borings undertaken, hence the objective was to attempt to confirm the reported final mine layout and determine the quality of the rock and backfill. The locations of air track and diamond drill borings are shown on Figure 2.

The filled stope 1-2 required a different approach, necessitating the use of routine geotechnical drilling with a CME 55. The sampled borehole revealed that stope 1-2 had been filled with a fine to medium sand containing some boulder sizes. The location of this borehole is also shown on Figure 2.

A typical core log and sampled borehole log is provided in Figure 4 and 5, respectively. The purpose of these borings is to gather sufficient information from which the engineer can base recommendations.

The crown pillar portion of the Moneta site provides a good example of a competent tuffaceous rock with a high RQD and corresponding good NGI classification with a Q value between 14 and 20. Uniaxial compressive strength tests undertaken on retrieved core indicated average rock strength of 15,500 psi.

The stope is steeply dipping and wall rock is comprised of competent basalts both HW/FW, although experience in the area indicates zones of vertical schistosity are present. Wall rock stability appeared competent from the limited near surface drill work.

The geotechnical borehole in stope 1-2 provided useful information and a dynamic cone test advanced near the borehole confirmed the sandfill was consistently compact to depths of 75 feet below surface. The water level in this stope was encountered at approximately 75 feet and may either be trapped water or indicate the general level of mine flooding.

A typical cross section of the 1-2 stope is provided in Figure 6, and it is believed that waste rockfill was placed in the stope above the first level which would prevent further drill access into the stope.

## ROCK MECHANIC EVALUATION

The emphasis during this investigation was the stability of the crown pillar. Air track and diamond drill work helped to confirm mine geometry which reveals the average crown pillar thickness to stope width exceeds slightly the ratio of 3 to 1. This ratio is a common empirical design ratio used even today. Others have reported lower crown pillar stability ratio, being stable, Betournay, 1986.

Trow considers the above empirical rule of thumb to be a crude but workable design guideline. We also used the empirical approach whereby we estimated crown pillar strength, based on a factored pillar width to height, and calculated pillar stress using tributary theory with assumptions related to the field stress ratio, Coates, 1981.

An estimate of pillar strength was given by:

$$Sp = DRMS \frac{W^{.5}}{H^{.66}} \qquad .....1$$

where Sp is given in MPa and pillar width (thickness) and height (span) H are in meters.

DRMS is the design rock mass strength, Stacey and Page, 1986, whereby:

$$DRMS = \sigma c \frac{RMR - 10 \text{ MPa}}{66} \qquad .....2$$

The rock mass rating RMR is calculated as $RMR = 9 \ln Q + 44$ .....3

Pillar stress was estimated from the formula:

$$\sigma p = 2.1 \frac{[tb + tp + ts]}{tp} \qquad .....4$$

in MPa where tp is the crown pillar thickness, tb is the depth of overburden and ts is the relevant stope height.

The ratio of $Sp/\sigma p = F$ was estimated for all crown pillars investigated in Timmins. It was found that the ratio varied from 1.6 to 23.2 for those crown pillars investigated. The Moneta crown pillar has an estimated crown pillar factor of safety, F, greater than 5.0.

Trow recommended immediate action for crown pillars with factors of safety less than 3.0 and monitoring of marginally safe pillars between 3.0 and 5.0. Crown pillars with F greater than 5.0 were generally considered safe, but in the Moneta case the crown pillar is being monitored in any event, due to its high risk assessment.

The empirical approach outlined above was compared to Q value versus unsupported span stability relationship given by Houghton and Stacey and found to be in good agreement. This relationship was presented by Stacey and Page, 1986., and is shown on Figure 7.

Two other design approaches can be mentioned briefly. Beam theory or slab design techniques have been applied to crown pillar, sill pillars and stope back analyses; however, we find these predictions of crown pillar stability conservative.

Recent success has been obtained however using a modification to the empirical method comparing a stability factor verses hydraulic radius. This modified method has only been applied where more extensive geological, geometry and rock mass data is known.

Numerical techniques were applied at the Moneta site. Two dimensional boundary element modelling was used to approximate stress and displacement predictions. The typical output from the modelling is shown in Figure 8. No stress failure was predicted along the crown pillar at 1-1 stope. Although, localized shear failure of the stope back and tensile failure along stope walls was predicted by boundary element methods for other sites.

Arguably, boundary element modelling has limitations in crown pillar evaluations. Finite element models at other crown pillar sites have been used by Trow with greater success and yield a higher level of confidence. These FEM models were also two dimensional. Our experience shows that magnitudes of stress and displacement are essentially the same for either BEM or FEM codes, however easy use of the Mohr Coulomb failure criteria with FEM codes such as SAP2D provide a better failure prediction for such work.

Discrete block modelling has been used to analyse the 1-1 crown pillar. Discrete block modelling offers several advantages for engineering interpretation during stability evaluation related to the mechanics of joint behaviour. Figure 9, depicts the output from the discrete block model for a single cross section of the Moneta 1-1 stope. Slippage along joints is predicted mostly along the stope walls rather from the crown area. This observation was not reported during or after mining but introduces new elements of uncertainty. Generally, numerical modelling confirms the empirical results that the Moneta crown pillar is relatively stable.

An evaluation of the subsidence potential at Moneta's 1-2 stope was based on soil mechanics rather than rock mechanics. A sampled borehole was advanced through the fill with grinding refusal at 86 feet below surface. The hole was dry to 31 feet and saturated below 72 feet. It was estimated that sufficient resistance is available within the fill to observe the phenomenon of arching as described by Terzaghi. In theory, the height of the collapsing arch should not encroach nearer than 25 feet from surface. It was noted that piping failure or erosion could however lead to collapse to surface.

Interestingly, recent discrete block modelling indicates a similar conclusion regarding the arch potential as shown on Figure 10. The use of discrete block modelling in soil mechanics problems appears promising.

## REMEDIAL ACTION

The field work and stability analysis at the Moneta site provided sufficient information to recommend that no remedial work, such as filling, concrete capping or earth reinforcement, be undertaken at the time. However, it was noted that conditions at the site were subject to change with possible surface construction, time gravity effects, and so on. Recommendations were made to install simple extensometer equipment and re-evaluate the geotechnical conditions in the early 1990's.

Extensometers were installed in BH 2 and BH 3 shown on Figure 2. A typical multi point extensometer installation in a crown pillar is shown in Figure 11. The extensometer in BH 2 is a six anchor extensometer to 40 feet below surface whilst the extensometer in BH 3 is a four point extensometer to 80 feet. Thus far, no significant change in movement has been recorded.

## MINE HAZARDS AND SUBSIDENCE RISK

The Moneta site investigation is typical of the problem encountered by communities left with the legacy of abandoned mines. Public safety, economics and liability issues must be addressed, and the engineers that face these problems must work with some form of risk analysis. Figure 12 categorizes how high, medium and low risk situations can be grouped according to four fundamental parameters. The Moneta site is considered to be a high risk yet stable site. Hence, the option to monitor and re-evaluate in time was chosen as the best option under these circumstances. Changes in property use are possible which may reduce the risk level and avoid property loss as occurred in 1963 when five buses were lost at the filled site location, Figure 13.

## CONCLUSIONS

The geomechanical investigation undertaken at the old Moneta site required that several study phases be completed. The comprehensive evaluation indicates the crown pillar and filled glory hole are stable at this time. Several groups are involved in abandoned mine problems but the rock mechanics engineer plays a vital role in interpretation of rock mass conditions, backfill quality and overall ground stability.

The Moneta site is a good example of how land use, current and future, is influenced by abandoned mine operations. The ONR railway is planning to relocate the tracks, and this will influence future remedial work over the crown pillar. No major structures will ever be planned for the privately owned property at the filled glory hole.

Rock mechanics analyses for this problem consisted of both empirical and numerical techniques, but the need to gather extensive in-situ geological data, mine geometry information and rock mass physical properties is obvious. Care must be taken during the investigation not to disturb the sensitive area, and at the same time stay within limitations presented by property ownership and surface structures. Geophysical techniques, such as ground radar penetration and gravimetric analysis are advancing, and may be employed at this site in future re-evaluations along with more definition drilling.

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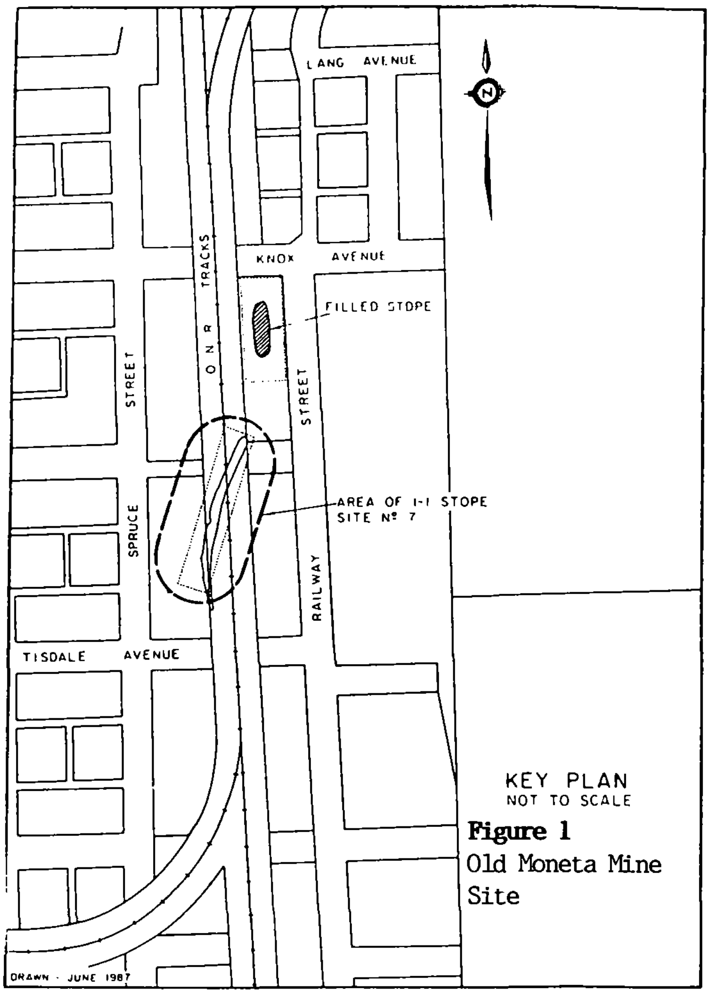



**Figure 1**
Old Moneta Mine Site

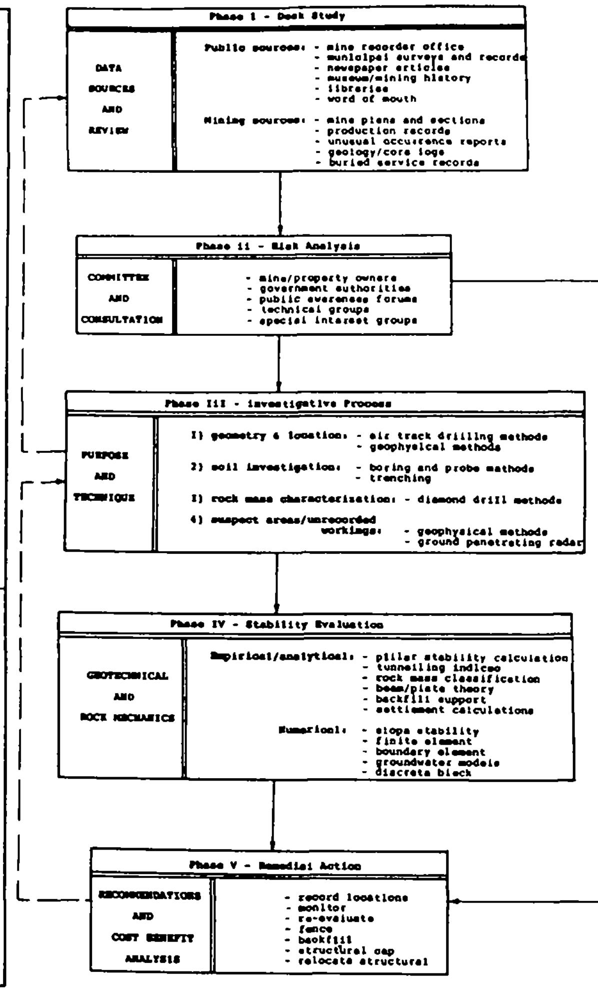



Figure 3 - Mine Hazards Investigation Approach

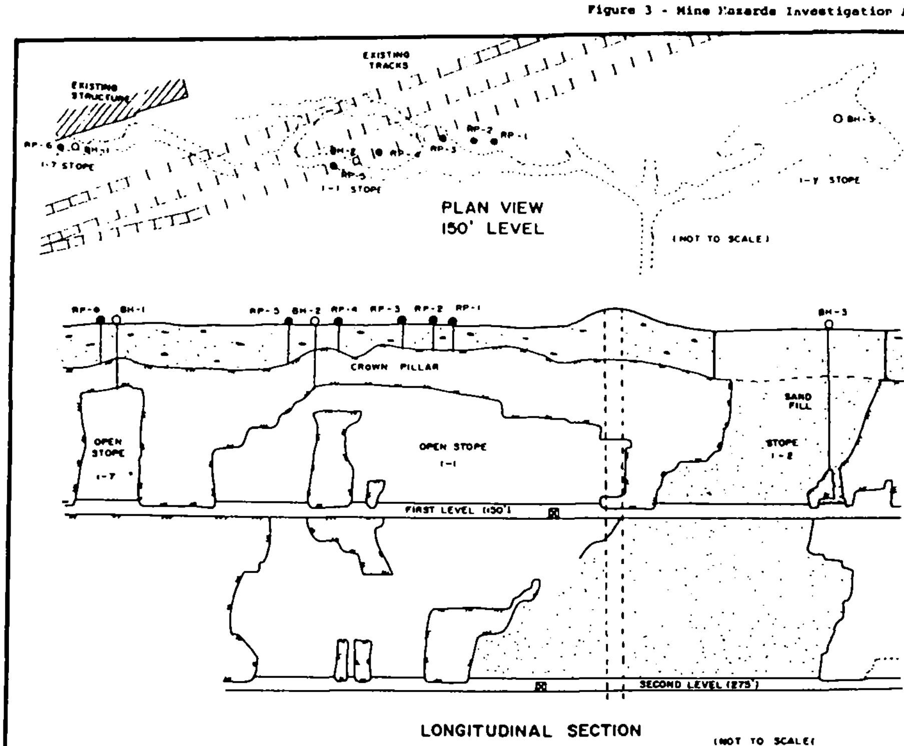



**Figure 2** - Plan and Section - Moneta Mine

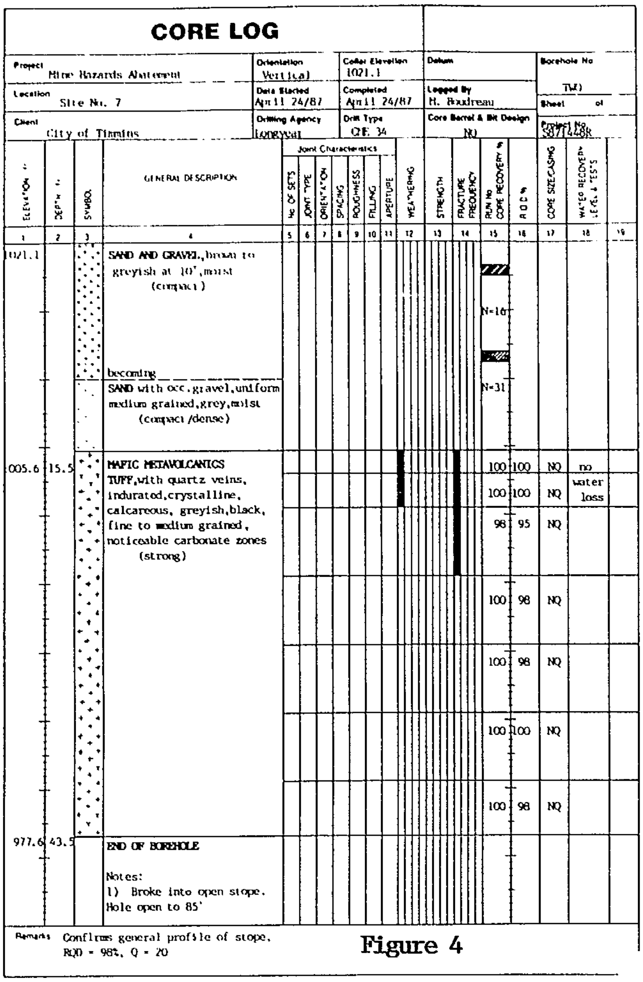
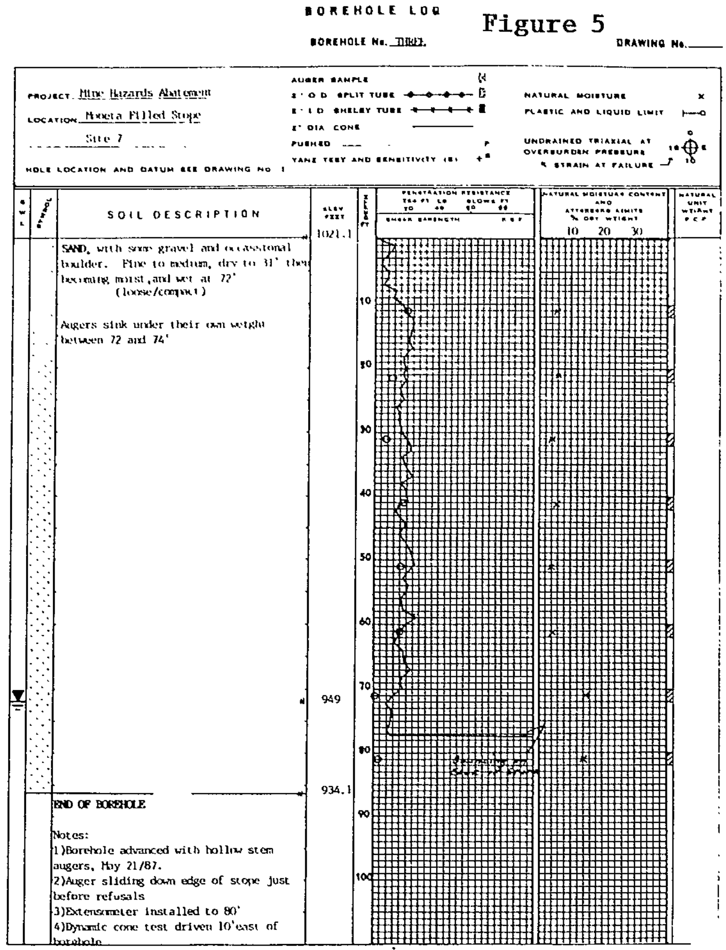



Figure 4

Figure 5



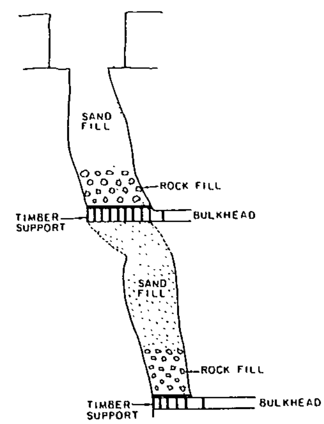




Figure 6

TYPICAL CROSS-SECTION THROUGH 1-2 STOPE, MONETA MINE

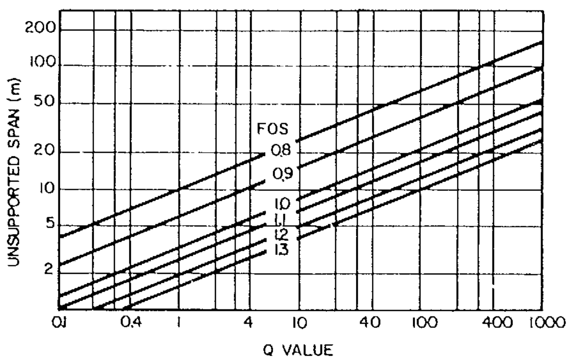



Figure 7

Rock Quality Versus Unsupported Span

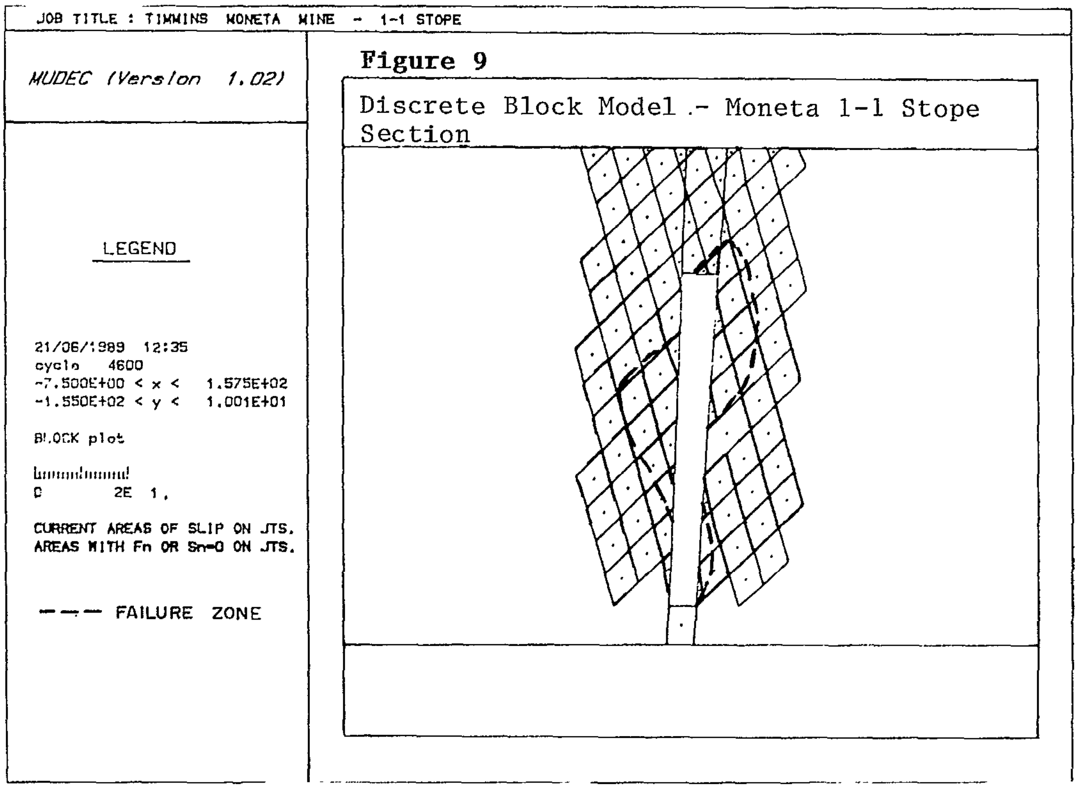



**Figure 9**

Discrete Block Model - Moneta 1-1 Stope Section

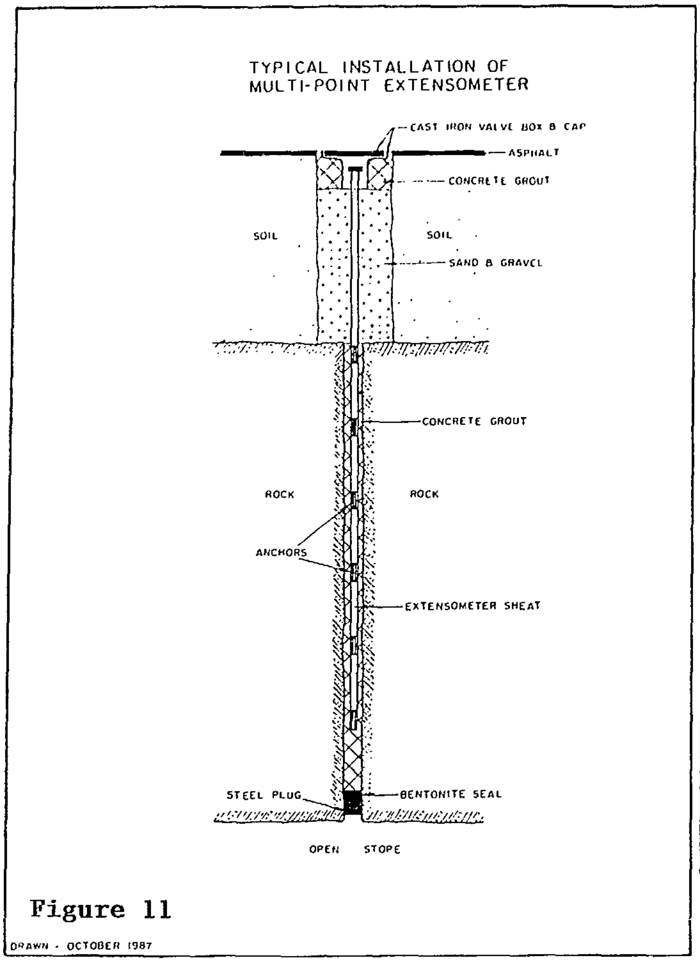



**Figure 11**

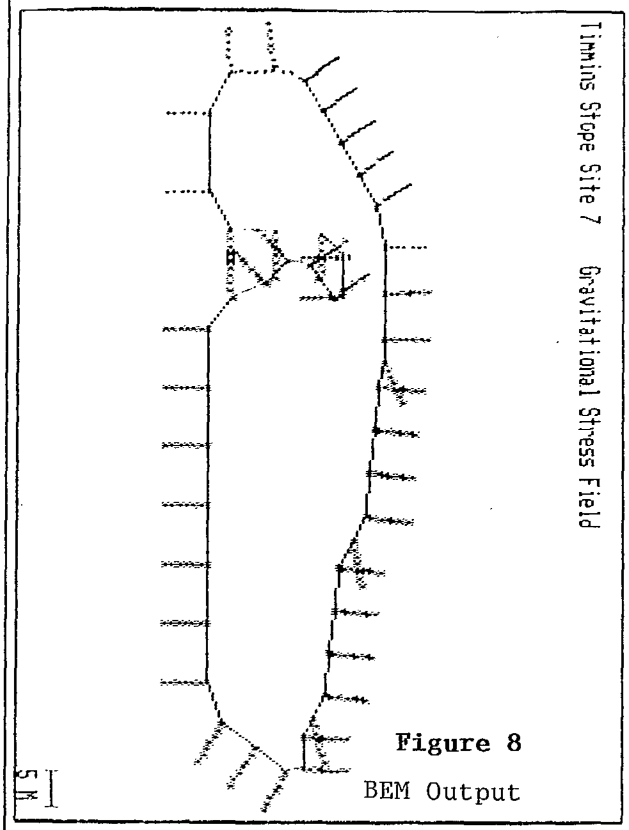
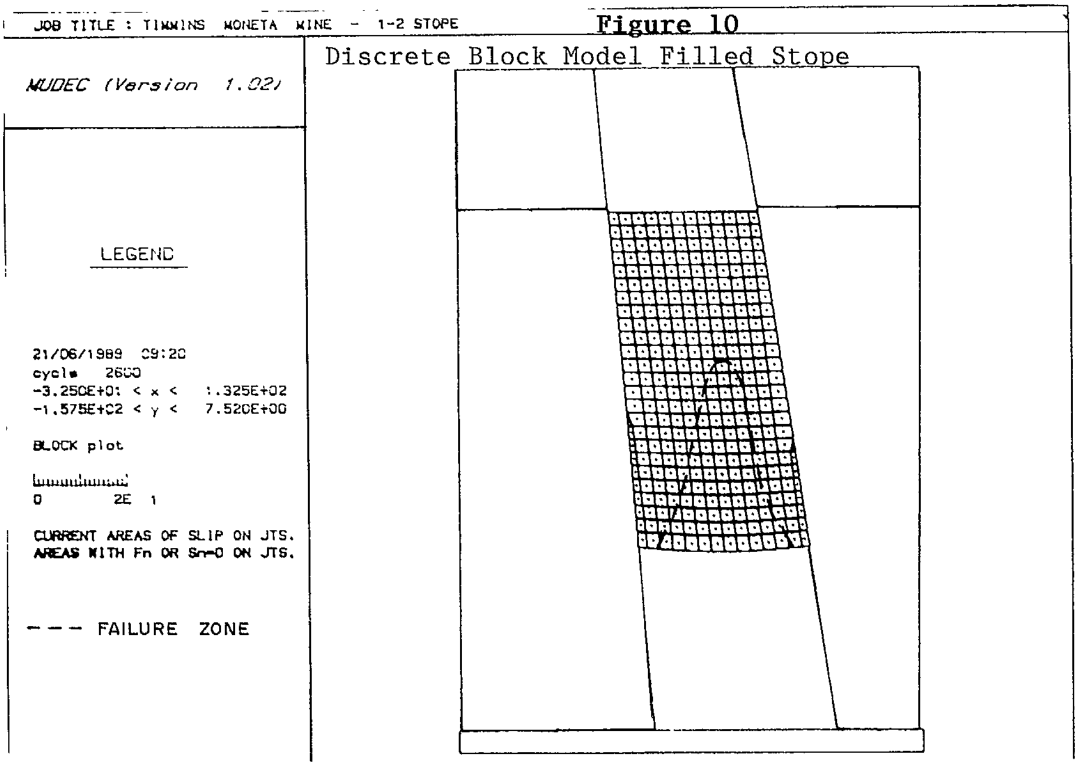



**Figure 8**

BEM Output

Figure 10

Discrete Block Model Filled Stope




| | Public Exposure | Acceptance of Failure | Property Disturbance | Financial Consequence |
|---|---|---|---|---|
| **High risk** | high frequency | not tolerable | interuption of flow and services | high cost |
| **Medium risk** | medium frequency | acceptable to some | temporary interruptions | moderate cost |
| **Low risk** | low frequency | acceptable to most | little or no disturbance | low cost |

**Figure 12** - Risk Assessment

**Figure 13**
Collapse of five buses November 12, 1963
Moneta 1-2 Stope

# INVESTIGATION AND TREATMENT OF CERTAIN ABANDONED MINE WORKINGS

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## ABSTRACT

Abandoned mine workings, especially those at shallow depths, may pose significant problems in urban redevelopment areas. In order to deal with such potential problems, the workings should be located, and their layout and condition determined. This can be accomplished by a combination of indirect and direct methods of exploration. In addition, the surface can be zoned according to the degree of potential hazard that the workings represent. This may result in buildings not being permitted in those areas of greatest hazard unless the ground is stablized beforehand. Stabilization measures may involve grouting the workings with relatively cheap bulk grouts, plugging stopes with concrete and backfilling to form stablized ground arches, or in-stope support by pillars. Case histories showing investigative techniques and the use of bulk grouts (rock paste) to fill old limestone workings are presented.

## RÉSUMÉ

Les sites de mines abandonnées, surtout les ouvertures de basses profondeurs, peuvent causer des problèmes sérieux lorsque situés en régions urbaines croissantes. Pour adresser de tels problèmes, les excavations devraient être localisées et leurs plans et conditions évalués. Ceci peut être accompli en combinant des méthodes d'exploration directe et indirecte. En plus, la surface peut être zonée en secteurs de danger potentiel que représente les excavations. Aux endroits les plus dangeureux, on pourrait interdire la construction d'édifices si les terrains ne sont pas stabilisés d'abord. Les mesures de stabilisation peuvent inclure l'injection de coulis en vrac dans des excavations, la fermeture des chantiers avec du béton et le remblayage pour former des arches stables, ou l'installation de piliers de support en chantier. Des histoires de cas sont présentés décrivant des techniques d'investigation et démontrant l'utilisation de coulis en vrac ("pâte de roc") pour remplir de vieilles excavations de calcaire.

## INTRODUCTION

The extraction of minerals of various types has been carried out for many centuries, especially in those countries which are now highly developed industrially. This means that significant numbers of old, abandoned mines exist, many of which are at shallow depth beneath the surface. Although methods of working different types of mineral deposits vary, mining activity can give rise to subsidence. Unfortunately, old mine workings generally are unrecorded and can give rise to subsidence effects at any time after they have been abandoned. Even when mine plans do exist, they often are inaccurate. These factors frequently have caused problems in relation to urban development and redevelopment.

Subsurface mining methods can be divided into three main groups. With partial extraction methods, subsidence can take several forms:
a) Collapse of the roof between pillars. This behaviour is most likely to occur in shallow workings and the subsidence profile may be very severe, with large differential subsidence, tilts and horizontal strains.
b) Collapse of pillars as a result of their strength being exceeded. Very large subsidence zones may result if several adjacent pillars collapse but the subsidence area can be restricted by the use of large barrier pillars. Otherwise, the subsidence profile tends to be irregular, with large differential subsidence, tilts and horizontal strains at the perimeter of the subsidence area.
c) Failure of pillar roof and/or floor. This is likely to lead to a similar type of subsidence behaviour as in b).

When mining inclined ore bodies, the steeper the dip the more localized the subsidence or potential subsidence is likely to be. Furthermore, in open or waterlogged workings, the pillar, roof and floor materials may deteriorate over a period of time, leading, ultimately, to failure and subsidence.

In practice, "total extraction" of tabular deposits never involves complete extraction. This type of mining, using longwalls, or at least tabular stopes with substantial spans, is practised for example, in the coal mines of Great Britain and in the gold and platinum mines of South Africa. Extraction at considerable depths 1000 m or more does not produce substantial subsidence of the surface. The subsidence profile is smooth and subsidence may be less than 0.3 m with very small surface tilt of the order of 0.05%.

At shallow depths less than about 250 m significant subsidence can result. The effects are dependent on the dip of the deposits. The surface profile resulting from subsidence over shallow "total extraction" mining is commonly very irregular - tension cracks have often been observed, and large differential subsidence can occur at the fault and dyke contacts (Stacey and Rauch 1980).

Many metalliferous deposits occur in disseminated ore bodies. Often the only way such ore bodies can be mined economically underground is on a large scale, in which very high volume production can reduce the unit cost. Three types of massive mining are open stoping, caving and fill mining. Open stopes may be very large, with spans of the order of 50 to 100 m separated by pillars but in competent rock masses they may remain as stable underground openings, although the longterm stability of the surface may be in doubt. Since the caving method entails the collapse of the ground above the ore body, it usually progresses through to the surface to form a large crownhole, for example, up to 1000 m diameter and 200 m deep, which often has several scarps around its perimeter. With fill mining, the openings are substantially backfilled so there is only a limited potential for collapse of the workings, although settlement and compaction of the fill may allow some ground movement to occur.

## INVESTIGATIONS IN SUBSIDENCE AREAS

The primary object of a site investigation is to assess the suitability of the site for the project concerned (Anon 1981). A thorough site investigation first of all involves a desk study and a reconnaissance survey, which are then followed by field exploration. In the case of sites located in areas underlain by shallow abandoned mine workings, the investigation of subsurface conditions is of particular importance.

The desk study includes a survey of appropriate maps, documents, records and literature. The presence on geological maps of mineral deposits which could have been mined suggests the possibility of past mining unless there is evidence to the contrary, and geological and topographic maps may show evidence of past workings such as old shafts, adits and spoil heaps. All the geological and topographic maps of the area in question, back to the first editions, should be examined. British Coal and the Abandoned Mines Record Office represent primary sources of information relating to past mining activity in the United Kingdom. Other sources include county record offices, public record offices, museums, libraries, private collections and the British Geological Survey. Similar organisations exist in other countries with long established mining industries. Where records of past mining are available, they must be treated with some caution as they are frequently inaccurate and incomplete. Nevertheless, such records provide useful information relating to the extent and method of mining.

The use of remote sensing imagery and aerial photography for the detection of surface features caused by subsidence is more or less restricted to rural areas. Colour photographs may be more useful than black and white ones in the detection of past workings and, if there are differences in thermal emission, then infra-red (false colour) photographs should show these differences. The detail obtained from aerial photographs should be represented on a plan at a scale of 1:2500 or larger. Scale is a critical factor in that the detection of the relatively small subsidence features (1.5 to 3 m across) provided by aerial photographs with scales around 1:10 000 and larger revealed by the reconnaissance survey.

The reconnaissance survey involves a walk-over visit of the site to allow familiarization. Subtle variations in the topography may be observed together with evidence of past land use. If sufficient information has been gathered at this stage, it may be possible to pass straight into a field investigation involving direct exploration of the ground by drilling. If this is not the case, then indirect subsurface exploration using geophysical techniques, may be undertaken. Considerable care should exercised at the planning stage of a geophysical survey for the location of subsurface voids because of the variable nature of the target. (McCann et al. 1987; Cripps et al. 1988). The selection of the most appropriate technique necessitates consideration of four parameters, namely, penetration, resolution, signal to noise ratio and contrast in physical properties. The size and depth of the workings and the character of any infill control the likelihood of the workings being detected as an anomaly. With the information obtained from the desk study and the reconnaissance survey, many of the available geophysical methods can be assessed at the selection stage, using a model study, and accepted, or rejected, without any requirement for field trials. Generally it is possible to detect a cavity whose depth of burial is less than twice its effective diameter. Otherwise, more sophisticated surface methods or drillhole methods have to be employed. However, since the presence of a void is likely to affect the physical properties and drainage pattern of the surrounding rock mass, this can give rise to a larger anomalous zone than that produced by the void alone.

The nature of the environment around a site affects the success of geophysical surveys. For instance, traffic vibrations adversely affect the results obtained from seismic surveys, as do power lines and electricity cables in the case of electromagnetic techniques. Of particular importance is that there should be sufficient physical property contrast between the void and the surrounding rock mass so that the anomaly can be detected. The application of different geophysical methods for the location of some mine workings have been summarized by Anon (1988).

Seismic refraction has not been used particularly often in searching for voids created by previous mining at shallow depth since such voids are often too small to be detected by this method because of attenuation of seismic waves in the rock mass, especially if it is dry.

After extensive field work, Maxwell (1985) concluded that, except for workings with a depth of cover less than 5 m, it was unlikely that resistivity profiling would detect the presence of dry pillar and stall workings. This conclusion was supported by Burton and Maton (1975) who indicated that cavities at depths greater than twice their average dimension usually are not recorded. However, terrain conductivity surveys frequently are successful for locating small, near-surface anomalous features.

In practice, most shallow mining voids cannot be detected by normal magnetic or gravity surveys. However, the fluxgate magnetic field gradiometer permits surveys of shallow depths to be carried out whereas the proton-magnetometer can more easily detect larger and deeper features and yields results which are more suitable for contouring.

Ground probing radar is capable of detecting small subsurface cavities directly. The method is based upon the transmission of pulsed electromagnetic waves, the travel time of the waves reflected from subsurface interfaces being recorded as they arrive at the surface so allowing the depth to an interface to be obtained. Leggo and Leach (1982) noted that useful data can be obtained only from sites where clayey topsoil is more or less absent. However, the technique appears to be reasonably successful in sandy soils and rocks in which the pore water is non-saline. Limestone, granite and halite can be penetrated for distances of tens of metres, and in dry conditions the penetration may reach 100 m.

Koerner et al. (1982) investigated a microwave method as a means of locating subsurface voids. Of the two microwave techniques, namely, the pulsed type and the continuous wave type, it would appear that the latter method is the more successful.

Most of the geophysical methods have a down-the-hole counterpart which can be used to log a hole. Crosshole techniques can be used when the depth of burial of the void is more than two or three times the diameter of the void. Ballard et al (1983) used a crosshole radar technique to detect cavities in karstic limestone. As expected they found that attenuation of the radar signal was related to specific geological anomalies, cavities occupied by clay showing a very sharp decrease in the strength of the signal. Anomalies varying upwards from 0.7 m in vertical extension were detectable up to distances of 33 m.

In interborehole acoustic scanning an electric sparker, designed for use in a liquid filled drillhole, produces a highly repetitive pulse. This signal is received by a hydrophone array in an adjacent drillhole, similarly occupied by liquid. Generally the source and receiver are at the same level in the two drillholes and are moved up and down together (McCann et al, 1975). Drillholes must be spaced closely enough to achieve the required resolution of detail. The method can be used to detect subsurface cavities, if the cavity is directly in line between two drilllholes and has at least one tenth of the drillhole separation as its smallest dimension. Air filled cavities are more readily detectable than those filled with water.

Crosshole seismic testing has been used, employing two or more drillholes, to detect near-vertical subsurface anomalies. Acoustic tomography techniques are now being developed to map voids between adjacent boreholes (McCann et al. 1986; Jackson and McCann 1988).

Old mine workings generally have been located by exploratory drilling, the locations of drillholes being influenced by data obtained from the desk study or from the data gathered by indirect methods. Drilling to prove the existence of old mine workings is frequently done by open holes, which allows relatively quick probe drilling (Bell 1986). The sequence should be established by taking cores in at least three drillholes. Although frequently successful in locating the presence of old mine workings, exploratory drilling may not establish their layout. However, if the drilling results are combined with a study of old mine plans, if they exist, it may be possible to obtain a better understanding of methods of working, sizes of cavities and directions of roadways and galleries.

Below surface workings may be examined by using drillhole cameras or closed circuit television, information being recorded photographically, or on videotape, and used to assess the geometry of voids and, possibly, the percentage extraction. However, their use in flooded old workings has not proved very satisfactory, although ultrasonic methods have been used under these circumstances. Occasionally smoke tests or dyes have been used to aid the exploration of subsurface cavities.

It is important to remember that, usually, no single investigation method will provide all the necessary information concerning the nature of old mine workings. Methods are much better used in combination, their selection depending upon the conditions at each individual site.

Occasionally, access may be gained to shallow abandoned mines, sometimes via specially constructed shafts or headings. When old workings are located, the number and depth of mined horizons should be recorded and the geometry and direction of the workings, assessed as accurately as possible. Of particular importance is the state of the old workings, careful note being taken of whether they are open, partially collapsed or collapsed. Whenever possible, the extraction ratio (the percentage of mineral mined) should be estimated.

## OLD MINE WORKINGS AND HAZARD ZONING

Where a site which is proposed for development is underlain by shallow old mine workings there are several possible courses of action. The first and most obvious method is to locate any proposed structures on sound ground away from the old workings or over workings proved to be stable. It is not generally sufficient to locate immediately outside the area undermined as the area of influence should be considered. The angle of draw which governs the area of influence tends to vary with the local geology.

After a detailed investigation at a site in Airdrie, Scotland, underlain by shallow abandoned mine workings, Price (1971) was able to establish safe and unsafe zones. In the safe zones the cover rock was regarded as thick enough to preclude subsidence hazards (about 10 m of rock or 15 m of till was regarded as sufficient to ensure that crownholes did not appear at the surface) and normal foundations could be used for the two-storey dwellings which were to be erected. On the boundaries between the safe and unsafe zone, the dwellings were constructed with reinforced foundations, or rafts, as an added precaution against unforeseen problems. Development was prohibited in the unsafe zones. In effect, Price (1971) produced a thematic mining information plan of the site to facilitate its development. Such maps are now being produced for larger areas. In Great Britain, thematic geology mapping has been undertaken for a number of areas which are underlain by some form of mine working (Culshaw et al. 1988). In relation to mining, three types of map may be produced:

a) those showing the location of known shafts, adits and other mine entrances,
b) those showing the known extent of worked and/or unworked mineral at various depths,
c) those showing the extent of fill and made ground, including infilled opencast workings and expanses of mine or open pit waste.

The scales of these maps vary, depending upon local requirements and the availability of information but, generally are 1:10 000, 1:25 000 or 1:50 000. Attempts also have been made to zone ground underlain by old mine workings in terms of its suitability for different types of foundation (Gostelow and Browne 1986).

However, it must be borne in mind that thematic maps which attempt to portray the degree of hazard represent generalized interpretations of the data available at the time of compilation. Therefore, they cannot be interpreted too literally and areas outlined as "undermined" should not automatically be subjected to planning blight (MacMillan and Browne 1987; Statham et al. 1987). Also, zoning based entirely upon depth of cover above workings cannot be relied on completely, since occasionally subsidences have occurred in zones labelled "safe".

If old mine workings are at very shallow depth, then it might be feasible, by means of bulk excavation, to found on the strata beneath. This is an economic solution, particularly at depths of up to 7 m or on sloping sites. Such excavations may be carried out rapidly if the overburden consists of soils or fragmented and weathered rocks.

## A BRIEF REVIEW OF THE TREATMENT OF OLD MINE WORKINGS

Where old mine workings are believed to pose an unacceptable hazard to development and it is impracticable to use adequate measures in design, or found below their level, then the ground itself can be treated. Such treatment involves filling the voids to prevent void migration and collapse of support. In exceptional cases where, for example, the mine workings are readily accessible, barriers can be constructed underground and the workings filled hydraulically with sand or pneumatically with some suitable material. Hydraulic stowing also may take place from the surface via drillholes of sufficient diameter. Pneumatic or gravity stowing often is considered where large subsurface voids have to be filled. If the fill material, once drained, is likely to suffer adversely from any inflow of groundwater, for instance, if it may become liquefied and flow from the treated area or if fines may be washed out so causing consolidation of the fill, then it may be necessary to prevent this by constructing barriers around the area to be treated. To prevent ravelling, it is not essential to fill voids with material of roughly equivalent strength to the host rocks and, generally, filling with crushed mine waste, fly ash or sand, via boreholes, provides a satisfactory solution. This method cannot guarantee that every void will be completely filled but pressure grouting need not give better results.

Grouting via boreholes from ground level has been used frequently as a method of treatment of old workings. The grouts used in these operations commonly consist of cement, fly ash and sand mixes, economy and bulk being their important features. If the workings are still more or less continuous, then there is a risk that grout will penetrate beyond the bounds of the zone requiring treatment. In such instances dams can be built by placing pea gravel down large diameter drillholes around the periphery of the site. When the gravel mound has been formed it is grouted (Scott 1957). If the old workings contain water, then a gap should be left in the dam through which the water can drain as the grout is emplaced. This minimizes the risk of trapped water preventing the voids being filled.

Gray and Meyers (1969) outlined the treatment used in the Pennsylvania coalfield. Gravel was added if a void was encountered during drilling. This was then grouted. The holes were simply grouted if no voids were encountered, the basic concept being to create a series of grouted columns, thereby strengthening the ground. The technique is possibly most successfully applied where roof subsidence has not occurred to any significant degree. The published costs indicate that there is litte economic difference between this process and consolidation grouting based on closely spaced drillholes. Foam grouts also have been used to fill old mine workings (Littlejohn 1979).

Further details of the treatment of old mine workings are given in the case histories.

## CASE HISTORY 1: ABANDONED LIMESTONE MINES IN THE WEST MIDLANDS, ENGLAND

From around 1780 to 1920 large quantities of limestone were extracted from mines in the West Midlands of England, giving rise to about 250 ha of derelict land at the present day. Unfortunately, the existence of these workings were not fully recorded. The potential subsidence risk has adversely affected attempts to redevelop the area.

The limestone which was mined is Silurian in age, the Barr, the Lower and Upper Wenlock and the Aymestry Limestones all being worked. Several extensive mines were developed at depths of up to 250 m below the surface. Pillar and stall workings occur where the dip of the limestone is less than 30° but where steeper dips occur, the mines take the form of long galleries which run parallel to the strike. Limestone mines were left unfilled once abandoned. Although some old shafts were filled with debris, most simply had a stage constructed near the top which then was covered with earth.

Since the existence of limestone mines was known, it was tacitly accepted that shallow workings (that is, at less than 60 m depth beneath the surface) could give rise to instability problems and so development did not take place in such areas. Where mines existed at greater depth they were not considered as likely to affect the surface hence no restrictions were placed on development. However, in 1978 partial collapse of Cow Pasture Mine at Wednesbury occurred and which is located at around 145 m depth. The resulting surface depression was approximately 200 m by 300 m and the maximum subsidence was 1.5 m. The initial rate of subsidence was slow but increased to a maximum of around 50 mm per day and then gradually tailed off over a period of several months.

This event caused an intensive investigation of these limestone mines to be put into operation, the object being to assess the risk of collapse and the need for remedial work. The desk study collected all information onto one data base and some 30 mines requiring investigation were identified.

Although some mines are dry and could be entered and inspected, most are flooded. Inspection of Castlefields Mine at Dudley revealed that the pillars were in good condition, showing no evidence of spalling or crushing due to overburden load. However, many roof collapses above the stalls have in the past led to the appearance of crownholes at the surface and major roof falls were observed within the mine. Many of the voids in the roof formed by falls were bounded by joint faces and were rectangular in outline. Forster (1988) showed that the falls were due to the deterioration of stiff clay in unevenly dilated master joints, roof falls occurring most frequently where joint dilation was greatest and clay infill was thickest. Microseismic monitoring of roof fall within Castlefields Mine indicated that rates of collapse were greater than expected (Miller et al. 1988).

An influence zone for each mine was identified within which damage could be caused if subsidence occurred. The boundary of such zones is taken as the limit of 0.2% horizontal ground strain, outside of which normal structures are unlikely to undergo more than negligible damage if affected by subsidence. Inside the zones new developments have been frozen until the conditions of the mines have been ascertained and any necessary remedial action undertaken.

The subsidence risk, either in terms of extensive trough subsidence as a result of pillar collapse, or the appearance of a crownhole at the surface due to void migration, was evaluated. Each mine was ranked in terms of its relative potential instability and land use within its aforementioned zone.

The principal objective of the investigations was to determine the extent and condition of the mine workings. To this end, an extensive drilling programme was carried out. Double barrel sampling tubes with inner plastic liners were used to obtain core. Core was photographed, logged and the rock quality designation (RQD) and fracture spacing index recorded. Drilling penetration rates, water flush returns and in situ permeability tests also were used to assess the degree of fracturing. The degree of fracturing is important in that it tends to increase as old workings are approached. In fact, the Ludlow Shales are intensely fractured for several metres above the top limestone in voids which had partially collapsed.

Geophysical logging, namely, gamma-ray, gamma-gamma, neutron, caliper, multi-channel sonic, resistivity and temperature, was carried out in drillholes. This was done to identify changes in lithology and to assess the degree of fracturing. In addition, a dipmeter was run in conjunction with a verticality log (which records the tilt and azimuth of a drillhole) so that the dip direction of the strata within the drillhole could be evaluated. Correlation between dipmeter dip and in situ dip proved to be very good.

An ultrasonic survey was carried out within each drillhole which entered a void exceeding 1 m in height at mine level. Such surveys were undertaken in flooded mines. Horizontal scans were made at 1 m intervals over the height of the old workings and then tilting scans at 15° intervals were made from vertical to horizontal. Thus, old workings within approximately 60 m of the drillhole were mapped and the shapes of voids measured.

Initially, the use of closed circuit television to explore old workings was of little value since illumination only allowed a clear view to a distance of some 2 to 3 m. However, the use of stronger lighting, image intensifiers and computer enhancement allowed the field of view to be increased to around 20 m.

It would appear that for given rock conditions and mine layout, as revealed at Cow Pasture Mine, there is a critical height of pillars in the Ludlow Shales overlying the limestone at which the overburden pressure causes it to be crushed. These pillars in the shales are an upward extension of the limestone pillars and have been formed by the collapse of roof rock in the stalls. In such cases a factor of safety can be regarded as the difference in height of a pillar between its present height and that at which failure by crushing will occur. Obviously, the rate at which roof fall occurs, so extending pillar height, is important since when one pillar fails its load is transferred onto those surrounding it, thus causing them to fail. There is no way of accurately predicting when and where such failures will occur, nor the form they will adopt.

In such circumstances the question is how to deal with the old workkings. One option is to do nothing other than maintain periodic checks on surface levels and underground events and then to take action if a subsidence event is forecast. This can be acceptable over agricultural land but is not satisfactory in areas of urban development. Here mines have to be treated, usually by filling, to prevent the possibility of future collapse.

Filling these large limestone mines (for example, some may occupy a volume of approximately 500 000 $m^3$) requires a cheap material which is locally available in large quantities and which can be placed at high rates. It also needs to be capable of spreading a long way, in the mine, before it stiffens, so that the number of injection holes can be limited. Fortunately, in the West Midlands there is an abundance of colliery spoil which can be used to make rock paste. Rock paste is waste rock debris, especially colliery spoil, mixed with water (Ward, 1988). However, it should contain enough fines (that is, silt and clay sized material) for it to flow under pressure through a pipeline and to spread through the mine workings as a plastic material at constant yield stress without segregation or drainage occurring during flow. Rock paste may have small percentages of pulverized fly ash and lime (less than 5 and 3% respectively) added to it. This is to ensure that it will achieve a design strength of 20 kPa by pozzolamic reaction between these two constituents.

## CASE HISTORY 2: ABANDONED GOLD MINES AROUND JOHANNESBURG

Gold deposits in the Johannesburg area occur in the conglomeratic reefs within the Witwatersrand quartzites. They have been extensively worked within the central area of the city where most of the mines now are abandoned. Three reefs have been mined and the separation between these varies from zero up to 15 m. Hence, the reefs have either been mined individually or, where two are very close, both have been worked together. Narrow tabular open stopes, supported by occasional pillars, have been left behind. Mining took the form of strike drives within the reef horizons, the first level occurring at around 30 to 40 m beneath the surface. Subsequent levels occur at approximately 30 m depth intervals. The records of early mining (that is, late nineteenth century) are poor and any available mine plans frequently are inaccurate. In particular, pillar robbing which commonly occurred towards the end of activity in a particular mine, is unrecorded.

Guidelines identifying the restrictions to be imposed on surface development were formulated in 1965 and basically prohibited development where old mine workings occur at a depth of less than 90 m, with a progressive relaxation to a depth of 240 m, after which no restrictions apply. The guidelines are flexible in that they also take account of the number and separation of the reefs worked, the extent of mining, the stoping widths, the type and amount of any artificial support and the dip of the reefs.

Four types of subsidence were recognized by Stacey (1986) and are shown in Figure 1. The type of subsidence which may occur influences the treatment measures which are undertaken. Much of the Johannesburg area, where the outcrops occur, is zoned for commercial or light industrial development, so that low rise, relatively low cost structures can be constructed. Any ground stabilization measures which are required must also be of relatively low cost and the aim of shallow stabilization methods is to develop a competent surface to promote arching across stopes and prevent collapses. The construction of flexible structures can then accommodate any subsequent minor surface deformation which occurs.

Dynamic compaction has been used as a method of shallow stabilization where ground consisted of very loose, sandy gravel and fill, and occurred above mine workings occupied by loose rockfill and sand. It was hoped that dynamic compaction would improve the characteristics of the soil down to bedrock and close any cavities due to mine workings which occurred near the ground surface. In this way a ground arch was formed spanning across the reef outcrops. Up to 1 m of settlement of the surface was brought about near the reef outcrop as a result of dynamic compaction.

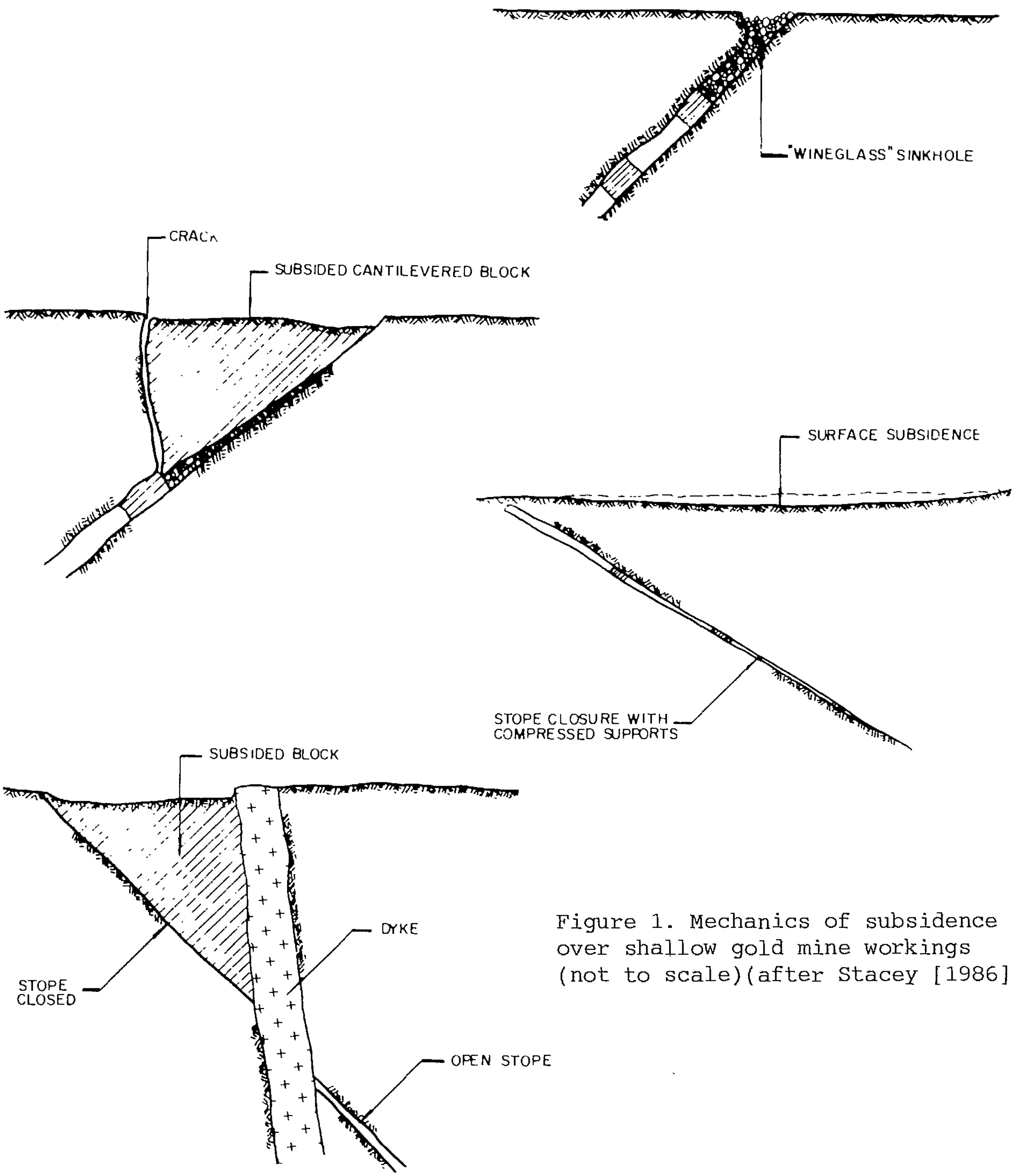


Figure 1. Mechanics of subsidence over shallow gold mine workings (not to scale)(after Stacey [1986])

Another, more common, form of shallow stabilization is to plug the stopes to provide the necessary local support and then to place backfill above the plug, the former being compacted to form the stabilizing ground arch. Exposure of the stopes during construction means that the plugs can be formed to suit the local variations in stope conditions. Excavation proceeds along the strike of the outcrop until the throat of the stope is exposed in which the plug, normally of concrete, is formed. The concrete plug seals the stope against the ingress of water. According to Stacey (1986) many sinkholes and subsidences which have occurred near stope outcrops have been initiated by inflow of water from the surface, causing gradual ravelling of backfill from the stopes.

A different approach is required in the case of major developments, as in these instances the risk of subsidence needs to be minimized. The greater cost of such developments allows more to be spent on remedial treatments. If such developments are to take place in restricted areas, which include reef outcrops, then the statutory authorities must be satisfied that the stabilization measures taken are acceptable. Deep stabilization of abandoned reef workings has to provide rigid support between footwall and hangingwall so that if collapse or instability at greater depth occurs, then the near-surface rigid zone forms an arch. In stope mining, support can comprise a combination of dip and strike pillars. Dip pillars tend to have greater ability than strike pillars to withstand shear deformation in a vertical plane normal to the reefs. Strike pillars, especially those at the bases of dip pillars, retain fill in the stopes so preventing slabbing from hangingwall and footwall. The pillars are constructed by sinking winzes in the stopes and backfilling them with mass concrete. The treatment is completed by constructing a concrete crown pillar across the stope.

## CONCLUSIONS

Most old abandoned mine workings are unrecorded and untreated and therefore can give rise to unforeseen subsidence problems in areas undergoing development. What is more, subsidence events can occur at any time after the closure of a mine.

A variety of mining methods have been used to work different minerals and they can have involved either partial or total extraction. Partial extraction methods are the most common and make use of some form of pillars to support the roof of the workings. Often these pillars were robbed as the mine came to the end of its productive life.

As many mine workings give rise to the presence of voids, they have to be investigated prior to any surface development. Such investigations involve a desk study and perhaps indirect as well as direct exploration methods. The principal indirect methods concerned are geophysical techniques. There is a wide range of these available but such surveys must be planned and interpreted with care. Indeed, many techniques may not yield worthwhile results because voids are at too great a depth in relation to their size, to produce an anomaly. However, most surface geophysical techniques have their down-the-hole equivalent and these have had some success in revealing the presence of voids. These methods can also indicate an increase in the degree of fracturing which can herald the occurrence of mine workings.

Direct exploration involves inspection of workings (which is not possible if they are flooded), drilling, and the use of ultrasonic methods and closed circuit drillhole television. Detection of voids beneath the surface by probe drilling can be unpredictable without prior indication of their presence. Angled drilling and some irregularity in the pattern of drillholes may help.

Two case histories have been provided. The former outlines the most extensive investigation of abandoned mine workings undertaken in the United Kingdom and was carried out with Government funding. Ordinarily such an investigation would prove very expensive but it does illustrate how much data can be obtained regarding large scale workings. The second casse history provides an indication of the stabilization measures which can be undertaken to treat old stopes in metalliferous mines.

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# GEOLOGIC STRUCTURAL STABILIZATION OF CAVING MINES USING GROUT COLUMNS

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## ABSTRACT

A stabilization design approach using grout columns in rubble to support consolidated overburden as a roof has been developed as means of mitigating progressive subsidence. The size and spacing of support columns within the rubblized zones beneath intact strata have been defined based on overburden loads, stress redistribution, and measured rock properties, A pilot grouting project has been carried out to establish grout mix designs and grouting techniques compatible with the rubblized strata such that competent grout columns can be developed from patterned vertical borings throughout a rubblized region.

The technique is being applied under the city of Rock Springs, Wyoming as part of an extensive multiphased stabilization program to mitigate the risk of future subsidence damage resulting from subsurface mining activities carried out between 1888 and 1926. In the southeast region of the city, a mine ranging in depth from 40 feet to 150 feet was extensively pillar robbed. The overburden is dipping sedimentary strata composed of both hard and soft consolidated lithology. Subsequent to mining activities, trough subsidence has occurred in places within the region with trough depressions defined by highly pillar robbed area and trough ridges defined predominantly by chain pillars associated with the mine entrees. A series of geologic investigations in the area has revealed a continuous high strength sandstone strata overlying rubblized roof zones in the mined out regions. Based on isopach mapping of borehole data, areas of potential future subsidence have been identified. Strength and competency of the hard sandstone strata and underlying strata have been defined throughout the region and are being used as criteria for the design of grout columns.

## RÉSUMÉ

Une méthode de stabilisation de chantiers supérieurs par colonnes de coulis emplacées dans du roc foudroyé fut mis au point contre l'affaissement de terrains. Leurs dimensions et espacements, ont été calculés en considérant les charges et propriétés du roc et la redistribution des contraintes. Un programme d'injection a trouvé les formules et les techniques compatibles au roc foudroyé afin d'obtenir des colonnes compétentes. Cette technique fait partie d'un programme de stabilisation pour prévenir le risque de subsidence découlant d'une activité minière souterraine au Wyoming de 1888 à 1926, incluant un écrémage considérable de piliers. Des affaissements en cuves et en vallons se sont produits. Des régions potentielles d'affaissement sont identifiées. La résistance et la compétence du roc surplombant les ouvertures ont été définies et sont inclues dans la conception des colonnes.

## APPLICATION TO CAVING MINES

The technique described in this paper is being used to stabilize a shallow dipping hard sandstone strata overlaying rubblized sedimentary deposits above extensive pillar robbed coal mines in Rock Springs, Wyoming. Its application is thought to be of particular interest for block caving mining activities in steeply dipping lithologies especially where very stable hanging wall conditions would allow the draw to continue until rubblization had propagated virtually to the surface. The envisioned application might be carried out as depicted in Figure 1. Continuous drawing of ore has allowed some amount of trough subsidence to have been observed at the ground surface. Preferably the stabilization technique would be applied prior to any subsidence having reached surface and prior to rubblization having compromised the harder strata selected to be the rock beam arresting further caving. The application of this technique would then begin by drilling either inclined or vertical holes from surface, through intact rock within the draw zone or high back rock and on through caved rubblized ore. Block caving would have to be suspended during this operation.

A sequential grouting program would be initiated to stabilize a selected intact roof sequence above the ore body being extracted. This may or may not be the immediate roof above caved ore. The stabilization as practiced in the Rock Springs subsidence control project does not require continuous grouting over the entire subsidence prone area. Rather it utilizes the natural structure of competent overburden founded on stabilized columns of rubblized material. In this application the rubble would be caved ore still resident within the draw zone. High strength grout would be injected to knit rubble into a composite column extending from the lower intact wall of the draw through caved ore (rubble) and tying into substantially intact high back rock and competent overburden above the ore body. Once the stabilizing columns are in place, additional ore could be drawn from the cavity. Rubblized ore would therefore potentially flow downward around the grout columns applying lateral live loads to the columns. The column design would have to take this type of loading into account should additional ore extraction be considered. It is anticipated that this additional live loading requirement would add substantially to the cost of grout columns due to the additional thickness required to withstand lateral loads.

## DESIGN ANALYSIS

The design concept is to provide a calculated uniform spacing of pillar replacement columns to support the rock beam. Columns would be composed of grout columns incorporating rubble and overlying intact rock into continuous columns to support the rock beam from the lower wall of the draw cavity. Column spacing, columns size, and grout strength are determined using structural analysis and laboratory measured properties of lower wall, column, and roof beam rock.

When grout columns are added to existing support from bulked rubble, the added columns increase this safety factor. When calculating column strength requirements, this existing support will be taken into account as a component of the factor of safety. If additional rubblized ore is to be extracted subsequent to column building, then rubble support should not be assumed in that calculation. When calculating allowable span for competent rock beam support of overburden, no intermediate support from rubble will be assumed. For grout column strength calculation purposes, a total safety factor of 2 has been used (FS = 2). For allowable competent rock beam span calculations a safety factor of 3 has been used (FS = 3), due to inherent potential for flaws in the beam structure.

Column support loads should be calculated by assuming columns are loaded by the weight of the competent rock beam and overburden above the beam. Due to lack of confining stress data, grout columns will be treated as unconfined columns when calculating load capacity. However, column buckling would be assumed to be controlled by support from adjacent rubble rock unless subsequent ore extraction is anticipated. In general, column loads would be calculated using the following formula:

Load/Column = $KS^2 T a$

where $K$ = stress concentration factor
$S$ = spacing between columns
$T$ = thickness of overburden above column including rock beam
$a$ = density of overburden

Column strength is calculated using the following formula:

$$\text{Column Strength} = \frac{3.14 D^2 fc}{4}$$

where $D$ = grout column diameter
$fc$ = grout strength

It should be noted that grout strength used for fc is valid as long as fc does not exceed the rock strength of the rubble material as an aggregate to the composite grout column structure.

Competent rock beam permissible span is calculated using the rock mechanics formula derived in Obert and Duvall:

$$\sigma max = \frac{a\,(S-D)^2}{2t}$$

where $\sigma max$ = allowable flexural stress
$S$ = spacing between columns
$D$ = grout column diameter
$a$ = density of span rock
$t$ = thickness of beam

In addition to these calculations the columns must be checked for floor and roof punching. In addition to calculating the grout column spacing and grout strength requirements, it is therefore necessary to measure the following parameters.

1. Flexural strength of competent rock beam
2. Thickness of overburden above beam
3. Thickness of beam
4. Compressive strength of mine floor or lower wall rock

It has also been necessary to verify that rubblization has not penetrated the competent rock selected to act as the structural beam.

## GROUTING

Based upon the data obtained from the drill logs similar to those shown on Figure 2, a grouting scenario can be developed. The biggest variable in data available from the drill logs is the porosity of the materials adjacent to the drill hole. The caliper log most readily indicates the existence of large voids, while the density log provides an indication of those zones of least density such as broken and rubblized rock, voids, and spaces in rubble, while the gamma ray log indicates lithology and stratigraphy. Drilling rates, etc., indicate hard zones, softer materials, loose rubble, or larger voids.

The existence of large voids, high porosity, low porosity and strength is the criteria upon which the grout mix is designed. Coarse grout, using a material which is basically aggregate sand, i.e., material passing the number 4 sieve, was designed for filling larger voids for the Rock Springs project. Intermediate grout, utilizing material passing a number 16 sieve, was designed for the rubblized zones, while a neat cement was designed for grouting the lower porosity zones. Similar mixes are envisioned for block caving applications.

In order to place the grout in the predesignated zone downhole, an inflatable double packer with a two foot separation between the upper and lower packer was initially recommended for grout injection. The two foot section of pipe between packers is perforated to allow grout to be ejected into the permeable zone. The grout injection system is designed such that the grout can be injected under low pressure but with a maximum pressure not to exceed pressure due to the overburden load. The quantity of grout injected in any zone is precalculated so as to produce a pancake shape or a segment of the grout column of a fixed height and diameter that will, when grouting is complete, produce a column which will support the overburden and surface loads, taking into account a predetermined factor of safety. Discuss use of accelerators to prevent down dip flow of grout, especially in steep dips.

A pilot grouting project was initiated in the fall of 1988 to evaluate the ability and procedures to construct the grout column with various grouts.

The initial type of grout selected for injection was the coarse or minus 4 aggregate grout. While pumping the course grout through the double packer system, unanticipated high pressures were encountered. Inspection of the grout ejection ports indicated build up of a filter cake; i.e., fluids were passing with the coarser aggregate acting as a filter and thereby plugging the system.

Neat cement was then pumped through the double packer system. Where very low permeability was encountered, high pressure resulted with little or no grout being injected. In highly porous or rubblized zones, a problem was encountered with "runaway" grout. In an attempt to minimize the injection of excessive grout into any one zone, a thixotropic additive was added. As an alternative to the thixotropic agent, a set accelerator was added to the grout mix. In general both the thixotropic agent or the addition of the accelerator resulted in better control of the grout.

Problems experienced while pumping the aggregate type grout resulted in a decision to pump the aggregate grout but to utilize only the upper packer. Removal of the lower packer eliminated a right angle turn at the ports between the packers, thereby eliminating filter cake buildup. Pumping aggregate type grout below the upper packer worked fairly well in most cases; however, in a single case a problem was encountered when downhole pressure began to increase and pumping was continued in an attempt to reach the objective quantity of grout. The end result was displacement of strata adjacent to the hole above the packer, which resulted in the packer becoming stuck in the hole. Increasing the hole diameter relative to the packer diameter helps to minimize the packer sticking problem.

In addition to pumping the minus 4 aggregate grout, a design using a minus 16 aggregate grout was pumped. Again, the intent was to determine grout take based upon the drill and electric logs and at the same time evaluate pumpability and formational reaction to grouts of varying viscosity and aggregates. It was found that the minus 16 aggregate grout could be pumped fairly easily compared to the minus 4 aggregate grout and the formation then took the grout quite well.

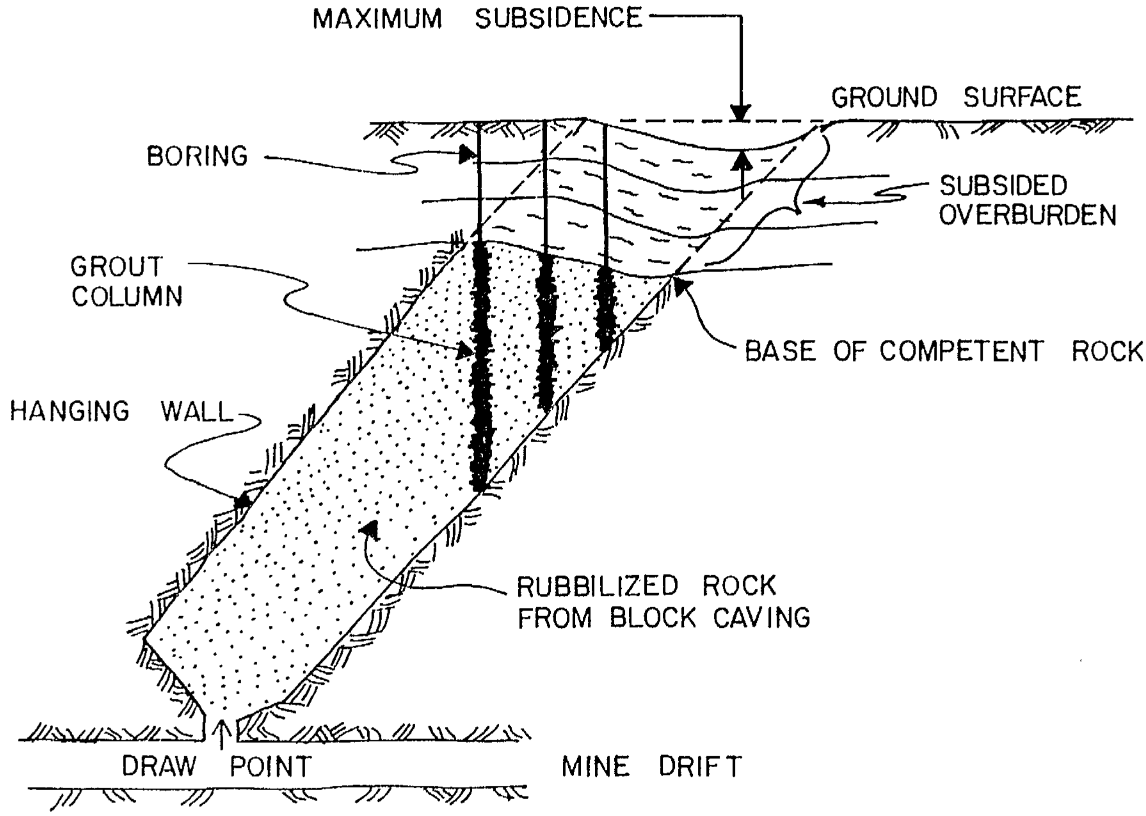
## CONCLUSION

The coring program has provided rock samples for testing and from which beam strength and column size spacing and strength can be determined. Logging of holes provided data for correlation of strata from hole to hole, as well as providing data on voids and rubblized areas within the strata.

The pilot grouting project in Rock Springs provided an opportunity to test various grout injection systems and grout mixes. A pilot project is recommended for new applications prior to initiating a full scale program. Basically, it has been determine that utilization of a single packer system works well when grouting at several intervals from the mine floor upward. Numerous grout mixes were tried depending on the downhole conditions encountered; however, it has been concluded that two basic grout types are adequate. One, a neat cement utilizing additives to preclude grout "runaway"; and two, a minus 16 aggregate grout that can be easily pumped and utilized for larger void filling. By larger it is intended to imply voids two feet in height or less, as other type grouts can be used for large voids. It is possible to conduct a total grouting operation utilizing only neat cement plus additives. This possibility could be a viable alternative depending on cost of materials and contractor equipment.

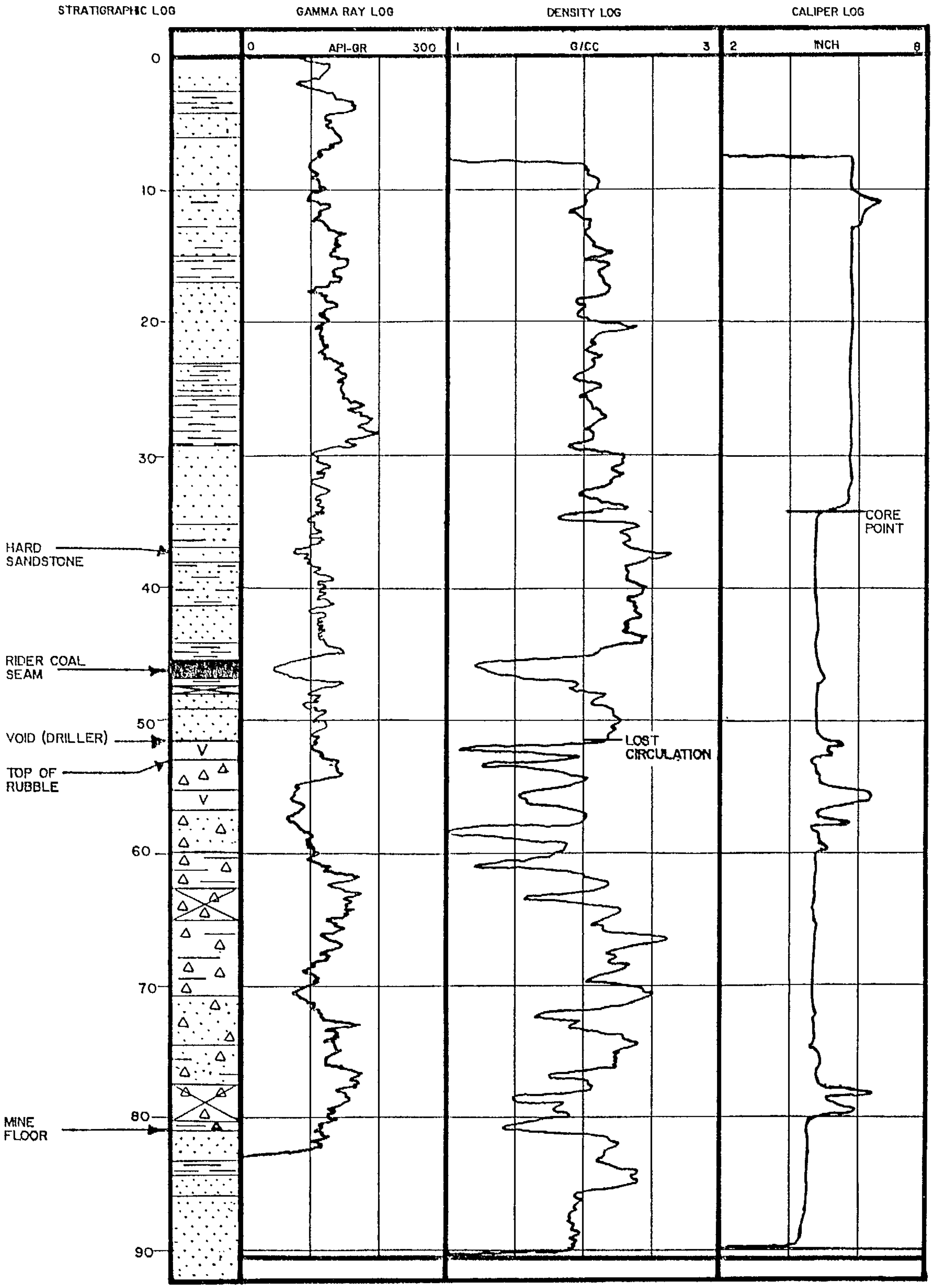
Confirmation coring around the perimeter of several holes which indicated the presence of a grout columns was carried out as part of the Rock Springs pilot project. Figure 3 shows an injection hole that was cored extensively. Examination of the data indicated a somewhat elongated column rather than circular. Nevertheless, a column was found to exist. Calculation of the size of a theoretical column is also shown on Figure 3. The theoretical column size is based upon the measured quantity of grout actually injected; however, a circular shape is assumed.

Based upon the calculated quantity of grout material required to construct grout columns spaced at 45-55 feet on centers, it is estimated that only 3 to 5 percent of the total mine voids require filling. As an alternative to column construction, where low strength, low cost grout can be utilized, it is estimated that costs would range from five to eight times greater.



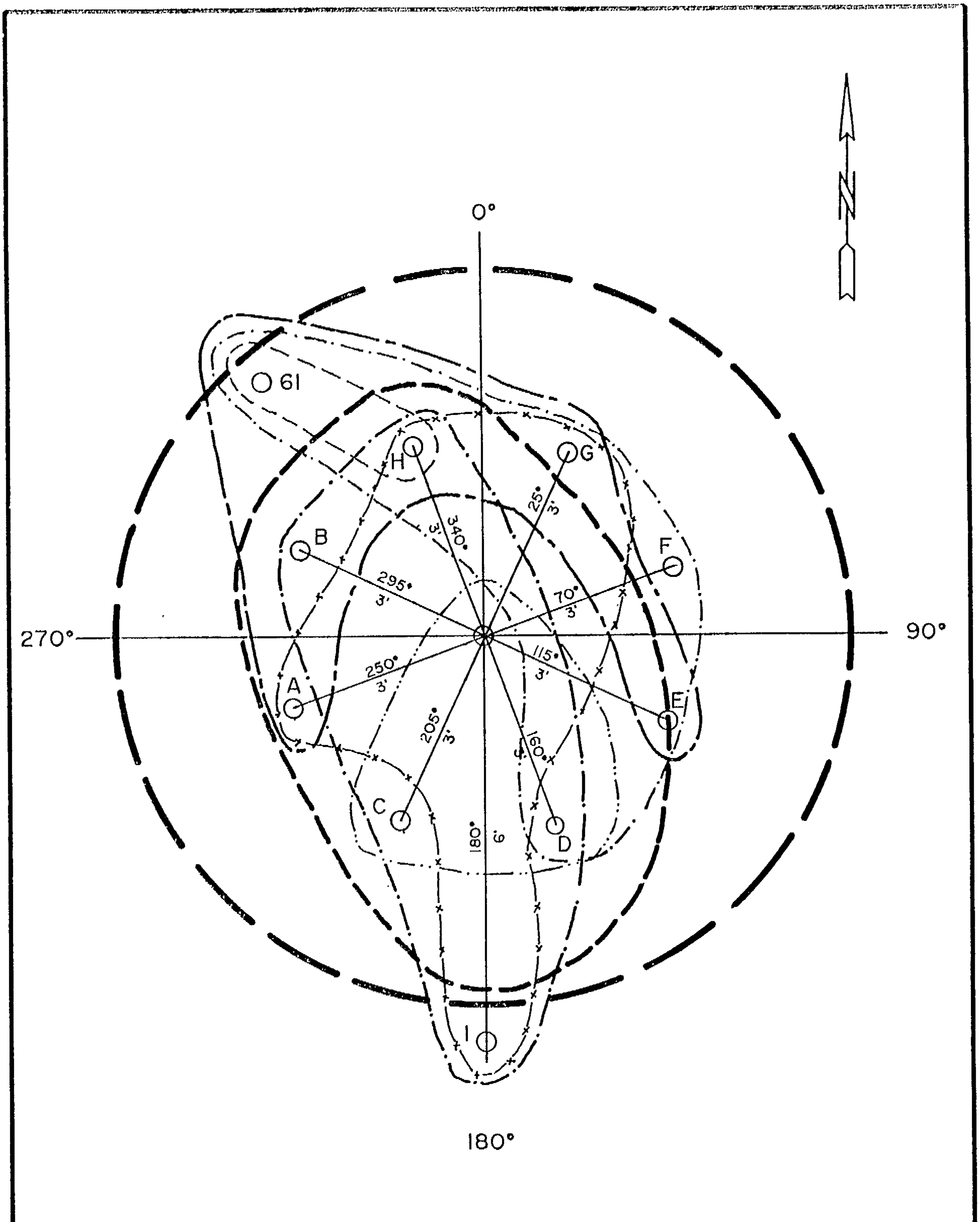

CONCEPTUALIZATION: BLOCK CAVING
SUBSIDENCE MITIGATION

FIGURE 1




TYPICAL DRILL HOLE LOGS

FIGURE 2




GROUT CONFIRMATION BORING

FIGURE 3

# Design Lessons from Evaluation of Old Crown Pillar Failures

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## ABSTRACT

Numerous small, and some major failures of crown pillars are known from the literature. Many of these failures occurred catastrophically, and some involved considerable loss of life. Evaluation of the causes of such failures has, in the past, concentrated more on the events leading up to the failure than in attempting to back-analyse the mechanism of failure from a rock mechanics viewpoint. The development of quantitative rock mass classification methods and the recent advancements made with numerical modelling procedures towards reliable replication of the behaviour of complex geometrical and geological conditions allows new insight to be gained of rock mass failure processes. This increased understanding provides the means for improving available crown pillar design approaches. In this paper a group of relatively well documented failures are examined, a generic classification for crown pillar geometry is proposed and several indicators of failure behaviour are described.

## RÉSUMÉ

Plusieurs ruptures de piliers de surface d'importance variable sont documentées dans la littérature. Plusieurs de ces ruptures ont été catastrophiques et quelques unes ont aussi causé des pertes de vie. Historiquement, l'évaluation de telles ruptures a surtout consisté à reconstituer la chaîne d'événements ayant conduit à la rupture du pilier plutôt qu'à faire une rétro-analyse du mécanisme de rupture du point de vue de la mécanique des roches. Le développement de méthodes quantitatives de classification des massifs rocheux et les développements récents des méthodes d'analyse numérique à l'aide d'ordinateur permettant de simuler le comportement d'ouverture souterraine de géométrie complexe dans un milieu géologique complexe permet un éclairage nouveau sur le comportement des piliers de surface. Cette nouvelle compréhension du problème permettra d'améliorer les méthodes de conception des piliers de surface. Dans cette présentation on examine un certain nombre de ruptures qui ont été bien documentées dans la littérature et on propose une classification générique des piliers de surface en fonction de leur géométrie. Finalement, on décrit plusieurs indices de comportement pré-rupture.

**BACKGROUND**

Our current understanding of the behaviour of surface crown pillars during failure is still markedly limited despite all of the attention being currently addressed to the topic (eg. Bétournay, 1987). This stems largely from the paucity of good geotechnical data on collapses. Failures, by their very nature, are treated as problems, and thus attract much public attention and scrutiny, (Figure 1). There is therefore a general reluctance to publicise failures. In fact, mostly technical publications concentrate on successes, not failures.

Even where a failure is known to have occurred, often precious little, if any, geotechnical data can be found. This situation is particularly common where pillar failures took place on properties now defunct. Also, where pillars have been laid out on the basis of traditional "rules of thumb" methods, generally no documentation of the "experience" component exists.

**Figure 1 : 1963 Subsidence of some old mine workings beneath a parking area in Timmins, Ontario.**

The fact that "rules of thumb" are still widely used even today seems anachronistic, but in reality it probably arises more from caution and uncertainty on behalf of the mines that there is any more valid or appropriate procedure, than from any desire to short-circuit more rigorous design approaches. One of the prime reasons for this stems from the perceived lack of sufficient calibration between more esoteric theoretical predictions and reality. This in part can be attributed to lack of a sufficiently valid data base with which to verify current analytical and numerical approaches.

Assessment of the absolute stability of the many widely varying geometrical configurations that occur in surface crown pillars is complex. Rock types vary, geological structural controls vary, groundwater conditions differ and in almost all cases, mining sequencing and extraction ratios are different. Considerable experience in applied rock engineering is essential in order to interpret these factors for the purpose of assessing the risk of collapse. Thus for effective design, such problems may need to be evaluated probabilistically.

Obviously, where surface crown pillars exist under Municipal infrastructure (i.e. Highways, Houses, Schools and such like) and the risk to the general public of failure to life and property is greater, factors of safety against collapse must be higher than might be tolerable in the active mine-site environment. This however makes the tacit assumption that all mining areas will never be developed - a situation known to be erroneous - as towns such as Timmins, Malartic, Kirkland Lake or Cobalt visibly demonstrate. Whatever the decided risk level, though, the same basic suite of data needs collection in order to evaluate the stability of any crown pillar situation (Table 1).

Significant improvements need to be made in our understanding of how and why surface crown pillar failures occur. Only then will it be possible to rationally modify current design approaches aimed at analysing crown pillar stability. Data on failure situations holds the key to solving the problem. Fortunately, for most of the tragic crown pillar collapses relatively extensive information is usually accessible either from the popular press or from the extensive testimonies associated with the Commissions of Enquiry which followed the disasters. However, even with this extensive published record, it is quite surprising how little rigorous geotechnically based documentation exists. Often, the only pertinent data of help towards back-analysing old cave-ins relates to the geometry of the failure. A typical case in point is the famous 1936 Moose River Mine failure (Figure 2, Stephans,1974)

## FAILURE SITUATIONS

Numerous similarities in morphology exist between various crown pillar collapses. In very few cases do the failures seem to result directly from central crown cracking. Most seem to have occurred from dislocation or sliding on well developed adversely oriented joints within the rock mass at either the hangingwall or footwall contact. Further, only rarely do the failure geometries seem to have been dominated by any major geological structural weakness such as a fault or cross-cutting shear zone. Such major weaknesses, perhaps because they are more easily recognizable by mine operators, are not apparently as important a cause for failure as the more mundane features of the rock mass which perhaps had been ignored in the original pillar design.

Kinematics, seem in many cases more important than stress effects, and

**Table 1 : Basic Data Requirements for Rational Crown Pillar Design Assessment**

| | |
|---|---|
| Surface Conditions | - Topography<br>- Presence or absence of water body |
| Overburden Characteristics | - Thickness, Material Properties and Stratigraphy<br>- Groundwater regime<br>- Bedrock/overburden interface topography |
| Rock Mass Conditions<br><br>*Full NGI(Q) and CSIR(RMR) classifications based on core data, lab tests, field assessment of discontinuity characteristics etc. | - General geological regime<br>- Ore zone dip<br>- Rock types and classification* characteristics - Hangingwall<br>- Footwall<br>- Ore zone in crown pillar<br>- Structural controls jointing, faulting, cleavage, etc.<br>- Geometry of crown pillar and upper openings, width, thickness, stope spans, filling if present, support methods if present<br>- Other factors<br>- available data on stresses<br>- complicating geometry - e.g. multiple ore zones, etc. |

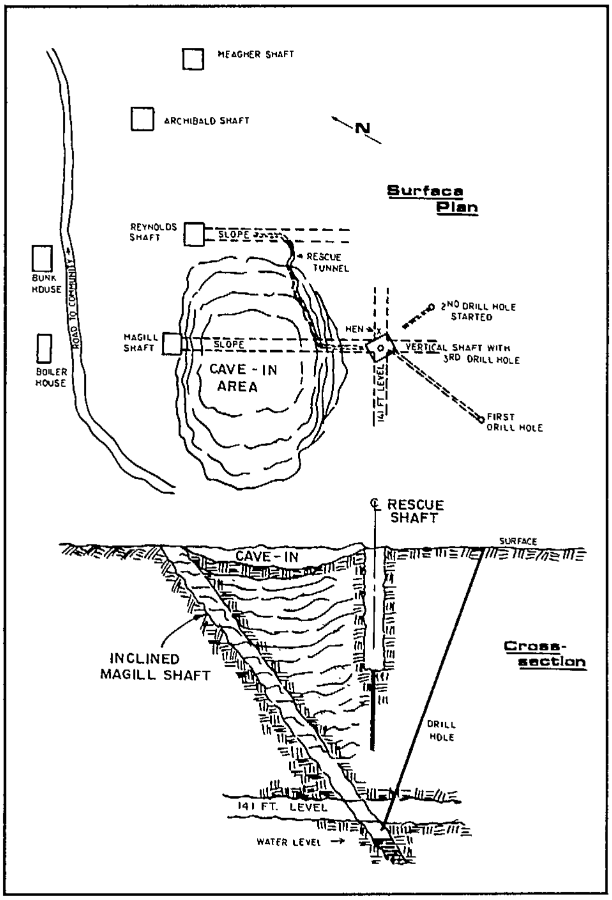


Figure 2 : Geometry of the 1936 collapse at the Moose River Mine in Nova Scotia

largely dominate failure behaviour in most hard rock cases. For both narrow vein stopes and in more blocky rock masses, failures seem to preferentially occur close to the intersection of adversely oriented cross-joints with the main ore-vein structure. In ubiquitously weak schistose rocks, by contrast, failures commonly seem to occur by progressive destabilization and tensile delamination at the hangingwall. In such cases, overall control of global stability may not be merely the kinematics of the weakness planes, but rather the influence of an adverse overall stress state within the abutment rock mass.

## APPRAISAL OF PILLAR STABILITY

Failures are the exception not the rule. In fact, it is both encouraging and noteworthy that so few surface crown pillar failures have occurred even where stoping has been advanced very close to surface. In some of the narrow vein Cobalt type situations, for example, 80 year old unsupported crown pillars of less than 3 ft. thickness still remain intact over 15 to 20 ft. wide open stopes. Gaining an understanding of what controls the stability of these types of situations requires comprehensive assessment of their geometry and geology.

### Classification of Surface Crown Pillars

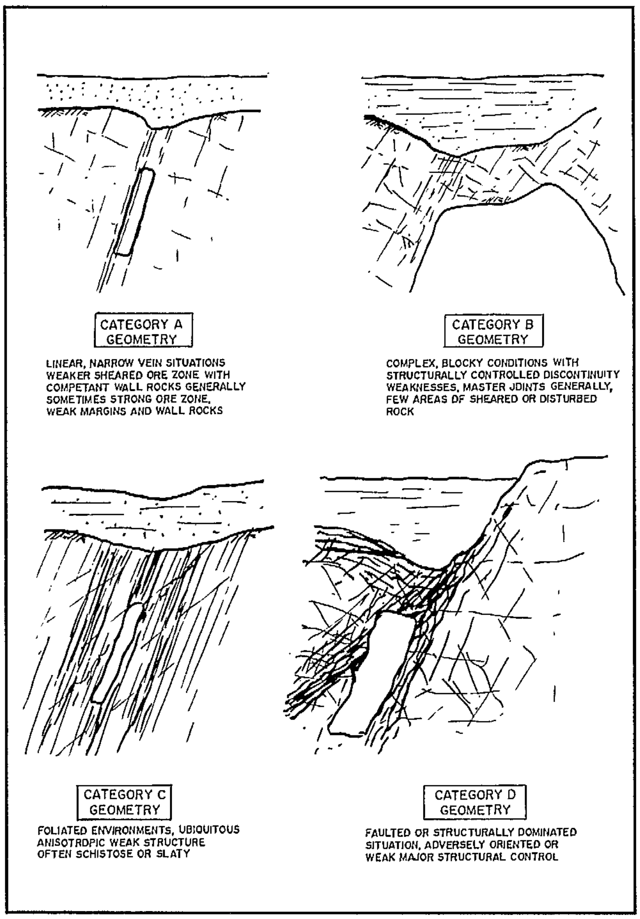


**Figure 3 : Major sub-classifications of surface crown pillar geometries.**

Appreciation of the differences and similarities between various crown pillars with thin or problematic geometries, has led to development of a broad scale four point geological and geometrical classification of pillars, (Figure 3), namely:

- o linear, narrow vein situations, usually, but not always comprising weaker sheared ore zone rocks bounded by competent hard wall rock conditions (Category A);
- o complex, blocky rock masses often with structurally controlled weaknesses cross-cutting both the large scale ore body geometry and the wall rocks (Category B);
- o foliated rock environments with strongly anisotropic weak structure generally parallel to the strike and dip of the ore body (Category C); and,
- o faulted or structurally dominated situations where a weak or adversely oriented major structure other than the ore zone, usually either cross-cuts or parallels the direction of stoping, (Category D).

**Crown Pillar Type A - Linear Ore Zones :** Many past cave-ins have occurred of the crowns over narrow vein mining stopes. Often, these have been associated with decay of wooden stull supports, however some collapses of poor quality ore zone material are known. Failure, in these cases, as well as destabilization of adjacent cantilevered sections of remnant crowns have generally been kinematically controlled, (Carter et al.,1988).

**Crown Pillar Type B - Blocky Rock Mass Conditions :** Failures in more blocky rock masses, although often larger in scale than those occurring over narrow vein stopes also seem to be kinematically controlled. The cave-in of the main crown rib pillar that separated two major areas of near-surface open stoping at the Beattie Mine provides one example, (Figure 4). Here failure of the pillar triggered major earthflows into the open workings from the deep varved clay sequences on either side, (Davidson,1948, Eden,1964).

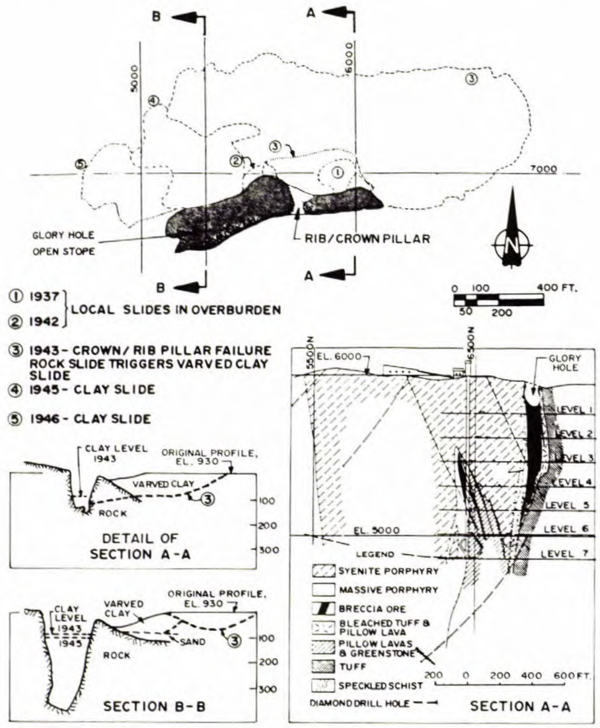


**Figure 4 : Geometry of 1943 crown rib pillar collapse and associated clay overburden slides at the Beattie Mine.**

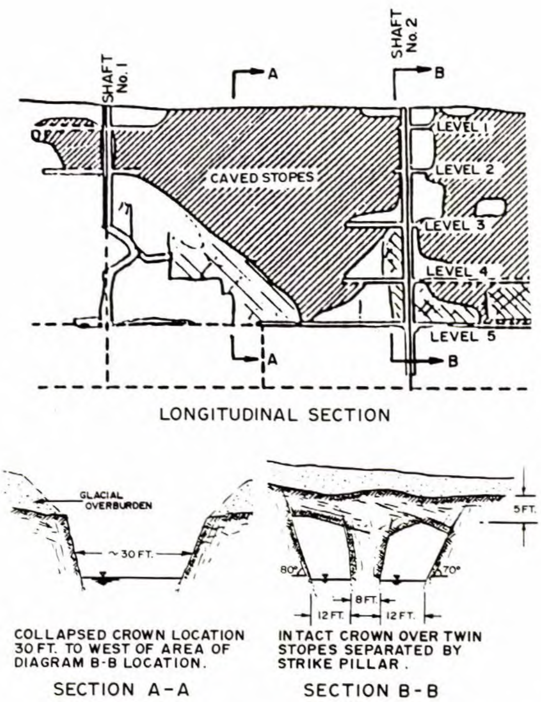


**Figure 5 : Geometry of the 1890's crown collapse and stope failure at the Huron Bay Copper Mine, Ontario.**

The crown pillar collapse at the Huron Copper Bay Mines in the 1890's (Knight, 1915) provides another instance where failure seems kinematically controlled. In this case a 20 ft. thick rock crown over a 30 ft. wide stope apparently caved by simple gravity sliding on slickensided sheet joints, (Figure 5).

**Crown Pillar Type C - Foliated Environments :** The catastrophic overburden flow slide that followed the 1980 crown pillar failure at the Ferdeber Mine in Quebec provides one of the best documented case records of a crown pillar cave-in initiated by destabilization of schistose rocks. The effects of the inrush of mud from the saturated soils, are, like the Mufulira disaster of 1970, (Spooner,1971) well documented in the Commission of Enquiry Reports (QBM,1980). However, data on the failure process in the rock mass are not as extensive.

**Crown Pillar Type D - Fault dominated Environments** : Several examples exist where failures have occurred associated with adversely oriented fault zones that cross-cut mining blocks. In such situations both the geometry of the failure, and the mechanism of failure appear to be largely controlled by the kinematics of the fault zone/ore zone contact intersection. Stress effects seem to play only a minor part in the process.

**Non-failed Situations** : For the majority of crown pillar situations even where actual geometries are known to be thin, no failure has developed. Determining the actual state of stability of these *stable* situations is not trivial. However, unless some measure of their stability can be acquired, possibly probabilistically, it will remain difficult to hypothesise why some remain intact when other similar looking geometries have collapsed. Sensitivity analyses, with use of probabilistic methods (as per Hoek, 1989) perhaps will provide some insight towards evaluation of stability controls.

## ANALYSIS OF CONTROLS ON PILLAR STABILITY

### Evaluation Procedures

Stability assessment for surface crown pillars demands evaluation of surface geometry problems as well as calculation of the effects of stresses and rock mass weaknesses. Currently, three essentially deterministic approaches can be used for optimizing new crown pillar layouts and for evaluating the stability or failure characteristics of old surface pillars:

(i) empirical methods - i.e. either "rule of thumb", or more quantitatively, based on descriptive rock mass classifications,
(ii) structural analysis and cavability assessments, and
(iii) numerical modelling procedures.

### Empirical Methods - Rules of Thumb

Historically, most empirical methods ultimately reduce to consideration of some guideline "width or thickness to span ratio". Many of these *rule of thumb* methods are difficult to theoretically justify, but are still extensively used. They should not be applied universally though, as no account is taken of rock mass condition. For instance, the stability of a one to one (1:1) ratio crown varies markedly with rock composition as well as overall stope geometry. For a classic narrow vein hard rock shrinkage stoping situation, where the walls are competent and often the ore zone itself is resilicified or cemented with carbonate, a one to one (1:1) ratio is likely in many circumstances to be conservative; whereas for a highly schistose metamorphic rock environment, a 1:1 ratio is totally inappropriate.

#### - Rock Mass Classification Approaches

The use of rock mass classification procedures takes the empiricism inherent in applying traditional "rules of thumb" the one necessary additional step to account for rock mass properties and geological conditions within the framework of precedent practise. Rock mass classifications using the NGI(Q) or CSIR(RMR) systems (Barton et al.,1974, Bieniawski,1973) have merit for assessing conditions within a given crown area. Again, they should not be applied indiscriminately, rather they should be used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the site geology. When applied intelligently, rock mass classifications can also be used to provide a tentative first estimate of stable unsupported spans using charts such as Figure 6.

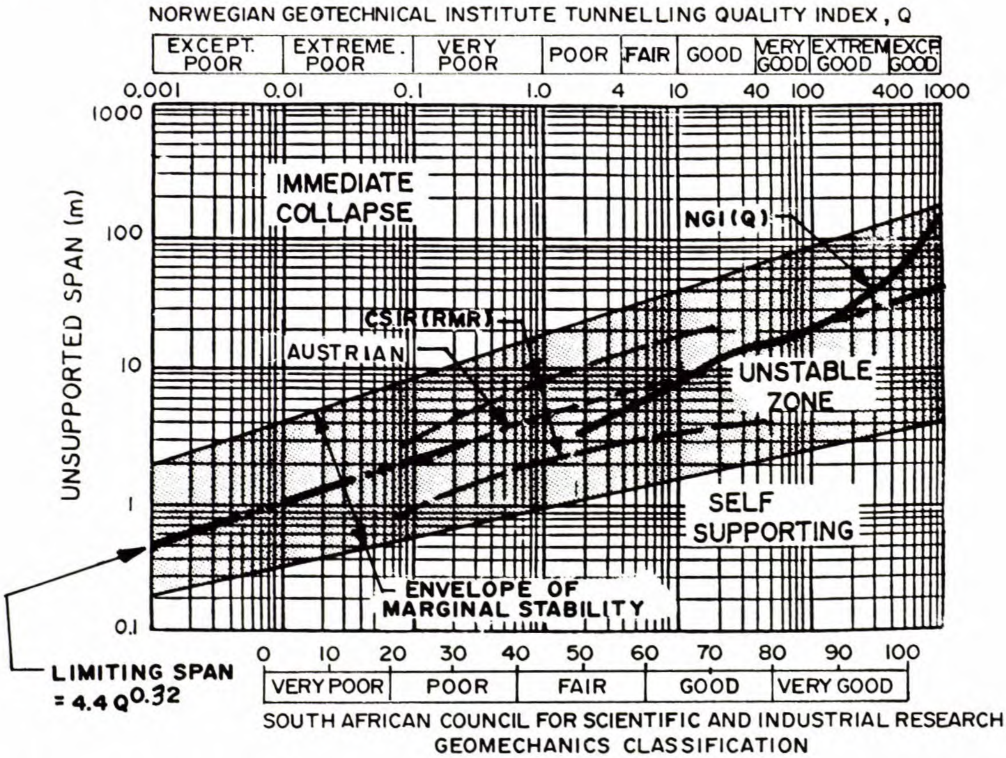


**Figure 6 : Suggested Limiting Span Relationship and Summary Unsupported Span Prediction Curves for NGI-Q and CSIR-RMR Rock Mass Classification Systems Developed from graphs published by Bieniawski, 1973 and Barton et al.1974**

It is important to realise, however, that not only the various curves plotted on Figure 6, but also the overall envelope and thus the suggested limiting span line are based primarily on tunnelling and civil engineering cavern experience. Considerable caution therefore needs to be exercised when defining spans for surface mining openings where the geometry and geology are different from those on which this chart is based. For this reason, Barton, in 1976 suggested the following limitations for use of such methods for design of unsupported spans:

$J_n < 9$, $J_r > 1.0$, $J_a < 1.0$, $J_w = 1.0$, SRF $< 2.5$
with the following conditional requirements:

1. If RQD < 40%, should have $J_n < 2$
2. If $J_n = 9$, should have $J_r > 1.5$ and RQD > 90%
3. If $J_r = 1$, should have $J_n < 4$
4. If SRF > 1, should have $J_r > 1.5$
5. If SPAN > 10 m, should have $J_n < 9$
6. If SPAN > 20 m, should have $J_n < 4$ and SRF < 1

Obviously, with so many provisos on use of graphs like Figure 6, such charts should never be used alone for sizing a surface crown pillar. Nevertheless, such empirical approaches hold promise for furthering our capability to incorporate the necessary lessons from empiricism into design procedures.

## Structural Analysis and Cavability Assessments

Techniques from civil engineering structural design or mining methods of caving analysis provide a more quantitative treatment of pillar stability than more empirical approaches, however, they require considerable simplifications of pillar geometries and material properties in order to be applied effectively. Many techniques are available from civil practise, the most useful approaches consider a surface crown pillar as a structural element in the form of a beam, a plate, a physical arch or a beam where arch action can develop under certain boundary conditions. Mining methods for block or chimney caving, which allow assessment of draw envelopes also provide insight into stability definition.

Using structural analysis methods, it is possible to take into account the geometry of the pillar as well as distributed surface or external loadings such as are induced by overlying overburden, by groundwater or by any influence from in situ stresses. To be valid, however, such methods not only require use of *realistic* input data representative of the rock material, but also must model actual failure mechanisms. Further, implicit in such analyses is the assumption that the crown beam is adequately supported (usually fixed) at its edges. Beam deflection then becomes a function purely of material properties and of external loading effects. Dislocation along weakness planes cannot directly be considered.

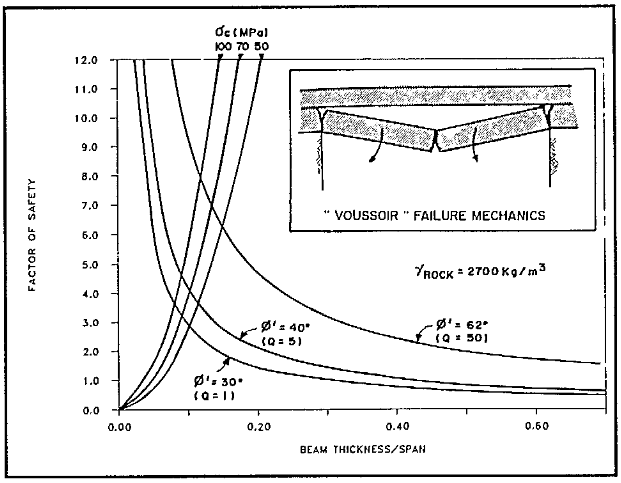


**Figure 7 : Failure behaviour for crown pillar "beams" of various thickness to span ratios and of varying rock quality**

Although numerous detailed differences exist between the various structural approaches available, they each have certain areas of applicability. For linear two dimensional ore-zone situations, beam theory or voussoir arch methods can have application, especially for sensitivity type assessments. For example, two sets of curves from voussoir analyses for a 10m wide crown for a range of rock quality, are shown on Figure 7. (NGI-Q values for this range of rock quality from 1 to 50 are shown alongside the shear failure curves with corresponding friction angles, $\phi'$, from the relationships given by Hoek and Brown, 1988. Overall rock mass compressive strengths, $\sigma_c$, are listed above the curves for tensile failure.) This plot is instructive in that it illustrates the classic behaviour exemplified by many actual failures. Namely, that if a roof beam has a thickness to span ratio in excess of about 0.25 (a common situation), it is likely to fail in shear at the abutments. If the thickness to span ratio is very low, i.e., less than 0.05 (a less common situation), then span stability is governed either by bending which would cause central crown or abutment cracking, or by compression failure at the voussoirs.

Alternate structural analysis procedures, such as beam buckling or plate methods are generally less realistic, although, buckling does constitute an important failure mode for hangingwall delamination in schistose rock masses. Other limit equilibrium methods, such as developed for hangingwall or chimney caving assessments (Brady and Brown, 1985) or semi-empirical methods of draw prediction for block or sub-level caving stoping, (e.g. Kendorski,1978), have application, particularly in furthering our current understanding of failure mechanisms, (see also Hoek, 1989). However, for back-analysing complex failure geometries, numerical procedures hold most promise.

## Numerical Procedures

Numerical analysis approaches generally allow much greater flexibility for analysing complex geometries than most limit equilibrium methods. Further, with current graphical output routines, modelling also allows ready appreciation of the global picture of stresses and displacements in the rock mass forming the crown pillar and abutments. This ability to rapidly display comprehensible graphics is one of the most useful features of modern numerical approaches. It allows not only rapid checking of the effects of varying input parameters but also provides hitherto unattainable insight into failure mechanisms.

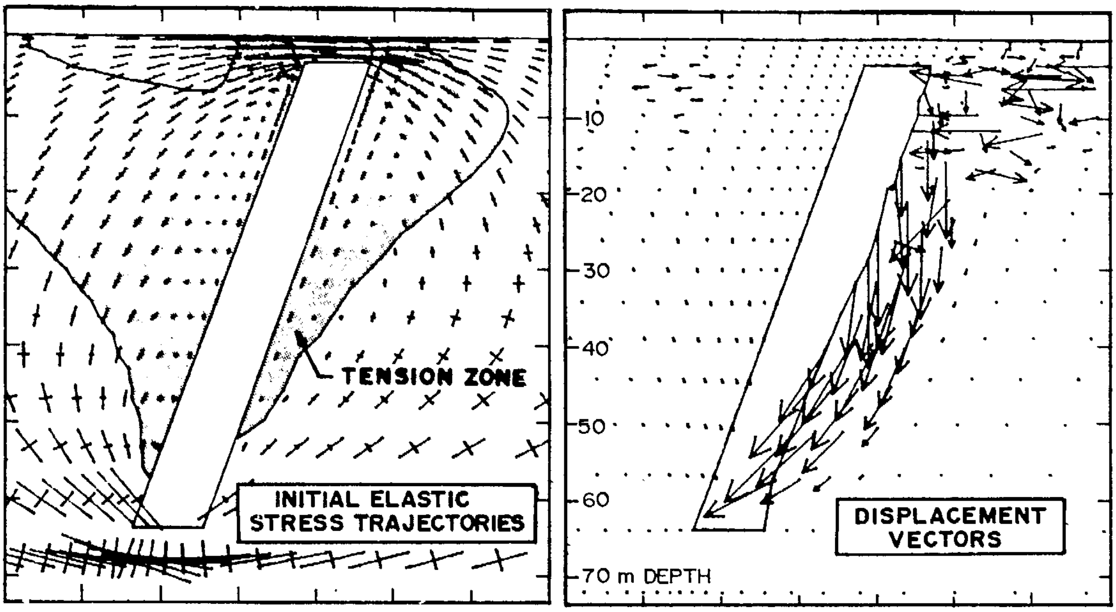


Figure 8 : Results of FLAC modelling of typical crown pillar and stope geometry showing initiation of classic footwall failure in a schistose rock mass.

Some caution, however, needs to be exercised when using some computer codes for modelling complex crown failure behaviour, where account has to be taken of both the effects of confining (clamping) stresses and gravitational kinematics. Not all codes are applicable, some in fact are completely inappropriate for tackling such surface problems. However, two particular 2-D codes - MUDEC and FLAC (Cundall,1971,1976) have been found particularly useful and effective for back-analysing old crown failures. Although the formulation of both codes is different they both seem to provide some measure of realistic assessment of the complex geometry and large strain problems inherent in crown pillar failure situations. More advanced programs will become available, particularly with formulation of workable 3-D codes and further development of hybrid codes, (such as discussed by Lorig and Brady, 1984). Nevertheless, even with these current 2-D codes, some considerable advances can be made towards understanding failure mechanisms. Figure 8 for example, shows two typical output plots from FLAC analyses for a weak schistose stope and crown situation. The rock mass for this case has been analysed assuming a Mohr-Coulomb failure model on ubiquitous parallel 60° schistosity planes. The initial elastic case principal stress trajectories are shown on the left hand plot. An extensive tensile zone is evident encompassing both the upper part of the footwall as well as a sizeable zone of the hangingwall. It is interesting to note that at least initially the crown pillar remains in compression. Failure appears to develop at the footwall, with gravitational effects creating large displacements within the stope footwall geometry. The pattern of movement corresponds to the typical behaviour seen with failure of open cuts, i.e., initial development of tensile cracking near the crest and outward bulging and failure of the toe, ultimately in this case leading to crown pillar collapse.

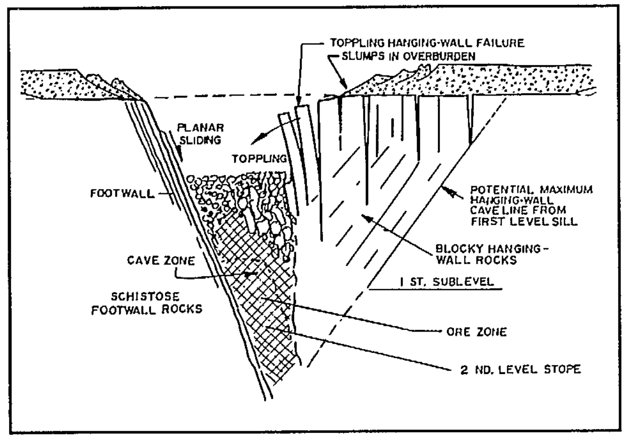


**Figure 9 : Characteristic footwall sliding and hangingwall toppling geometry of old stope crown collapse**

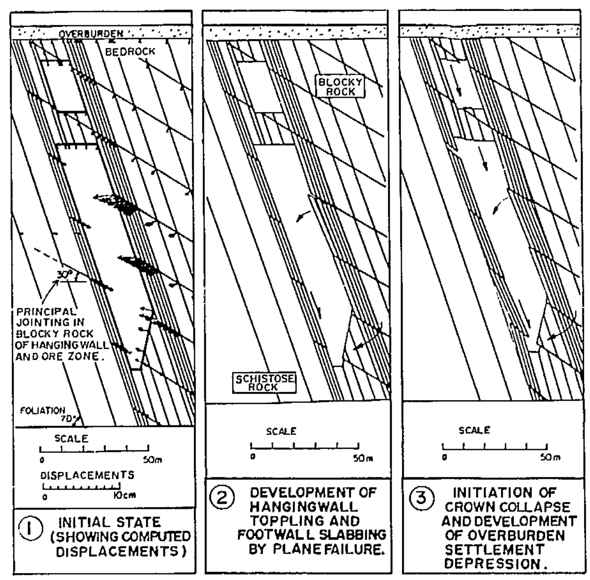


**Figure 10 : Reults of MUDEC simulations of failure initiation for open stoping situation depicted in Figure 9 above.**

Use of the distinct element method to examine discrete block motions (i.e., the MUDEC code) has considerable advantages over FLAC for modelling the actual progress of caving and thence collapse. Figure 9, for instance, shows conceptually, the characteristics of an actual failure, while Figure 10 shows MUDEC output plots at different stages throughout the process of failure initiation. For this example, initial distress appears to develop at the hangingwall of the larger lower stope, (see displacement plot - Stage 1, Figure 10). Slab delamination of the surface skin of the stope hangingwall continues to deepen (Stages 2 and 3) followed by dislocation and shear failure in the footwall associated with block rotation around the draw-point,(Stage 3). Throughout the early stages, the crown remains intact and relatively stable. In fact, unless it becomes significantly de-stressed, such that horizontal clamping due to insitu or induced confining stress is removed, shear failure at the crown/sidewall interface is restrained. Once shear failure begins however, progressive caving, such as actually occurred, is inevitable.

## CONCLUSIONS

Although some progress has been made by modelling and analytical methods toward increasing our current understanding of crown pillar failure mechanisms, far more data on old failures is needed in order to realistically develop these approaches sufficiently to formulate a rigorous design procedure. Development of semi-empirical rock mass classification based approaches, when coupled with probabilistic techniques for handling widely variable data, together hold some promise for sensibly incorporating field information into the design process.

ACKNOWLEDGEMENTS

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# SURFACE CROWN PILLARS - LIABILITY FOR SUBSIDENCE

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## ABSTRACT

Abandoned mine workings often are found beneath surface rights held by owners other than the former owners of the mineral rights. Land uses on those surface rights may include residential and commercial structures, and public highways.

The question arises of liability for harm to persons and property that may result from subsidence of the surface. The initial analysis includes a consideration of the nature of the risk involved and by whom knowledge of its existence is held. Knowledge by the former mine operator, or by municipal or provincial officials who may have plans or drawings of the abandoned workings, may create situations of liability if knowledge of a risk is not communicated to those who may suffer harm, and if harm subsequently occurs.

## RÉSUMÉ

Des chantiers de mines abandonnées sont souvent localisés sous des propriétés dont les droits de surface sont détenus par des propriétaires autres que les détenteurs originaux des droits minéraux. L'utilisation d'un tel territoire peut inclure l'emplacement de structures résidentielles et commerciales et de routes publiques.

La question de responsabilité se pose au niveau de la sécurité de la personne et de l'endommagement à la propriété, s'il y a affaissement de surface. L'analyse préliminaire porte sur la considération du genre de risque impliqué et la personne qui en connaît l'existence. Quand la situation est connue de l'ancien opérateur minier ou des agents municipaux ou provinciaux qui détiennent des plans ou esquisses des chantiers abandonnés, il y a possibilité de responsabilité si la connaissance d'un risque n'est pas communiquée à ceux qui pourraient subir un tort, s'il se produit.

## BACKGROUND

Under Ontario's Mining Act, the right to remove minerals may be conveyed by a lease or freehold patent of mineral rights only, leaving the surface rights able to be conveyed to third parties. Even where there has been a grant of both mineral and surface rights from the Crown to the mine operator, the surface rights may be conveyed by the operator to a third party.

In these circumstances communities may come into being, with residential and commercial areas developed, together with the normal water, sewer and highway infrastructure to support such development. Building permits issued by municipal officials will be at least subconsciously relied upon by the permit holder as stating that all things are in order for the construction of the house, store or other commercial structure to proceed.The situation from the municipality's viewpoint is that it may come into knowledge of abandoned underground workings (for example, in the course of carrying out planning, construction and installation of municipal services). If this occurs, and if there is danger of subsidence, clearly permits should not be issued for the construction of structures if harm to persons or property would result. Where knowledge of such risk is arrived at <u>after</u> development has already occured, how should the municipality proceed?

From the Province's viewpoint, the construction of highways, sewer or water facilities, or public buildings may give rise to similar knowledge. Also, for the last several decades mine operations have been monitored by and working drawings have been filed with the Province (Ministry of Labour) at the time of mine shutdown. If knowledge of potentially dangerous areas is gained, or of conditions dangerous to already-existing development, what is the proper course of action to be followed?

## THE LAW

Ontario's Building Code Act provides that the council of a municipality shall appoint a chief building official and such inspectors as are necessary for the enforcement of the Act within the municipality. In territory outside of municipal organization, Ontario is responsible for the enforcement of the Act.

The Building Code Act provides that no building shall be constructed or demolished without a permit issued by the chief official. An official who issues a permit is not personally liable for any act or alleged neglect or default in the carrying out of his duty, provided that he has acted in good faith. (The Act goes on to suggest that if negligence in fact occurred, notwithstanding the good faith of the official, the municipality or Crown would be liable for the damages flowing from that negligence, but I am not aware of any court cases which have confirmed this result.) I would suggest, however, that once the responsible official comes into knowledge of areas of potential subsidence, it would be difficult to argue that no negligence occurred in the issue of a permit for the construction of a building in that area, and accordingly a claim for damages would arise. The specific facts of the case would determine whether the claim would be more successfully pursued against the official or the municipality (or Crown), or both.

The more difficult situation arises where knowledge is gained of danger of harm from subsidence after development is already in place. How is such knowledge to be communicated, what is the effect on affected persons' mental well-being, and who should bear the costs of re-locating the affected individuals or safeguarding their houses or buildings?

While there is no case law directly on this point dealing with municipal officials, it must be remembered that under the Building Code Act, the official responsible for administering the Act, or his inspectors, have the power to enter lands or premises to determine whether a building is unsafe. The official also has the power to order that remedial work be carried out to render the building safe, and to prohibit occupancy of an unsafe building. While case law is not clear on liability for civil damages of such an official for failing to carry out a statutory power, it would be difficult to avoid criticism for not disclosing knowledge of a situation which renders a building unsafe and which results in harm or injury. With respect to the next question, then, of who pays to safeguard the structure, there is no legal obligation on the municipality or Crown to bear these costs where the original construction was done by all parties in good faith, but the particular circumstances in a specific case may compel the council or Crown to try to find a way to make a financial contribution.

With respect to Provincial officials who gain knowledge of a potential risk to persons or property, there is relevant case law which suggests there may be liability in some situations. The case of Heighington v. The Queen, 60 O.R. (2d) 641, involved lands in Scarborough (in Metropolitan Toronto) which had been contaminated with radioactive waste during the Second World War. A Provincial official became aware of this contamination in 1945. Subsequently, under Ontario's Planning Act, the local official plan was approved by the Province; the lands were approved for subdivision for residential purposes, and this was carried out in the early 1970's.

The trial court found that there was negligence in that no steps were taken to ensure that the lands were properly cleared of the radioactive waste prior to approving the official plan. Even in 1945 the danger of radioactivity was well recognized, and the Province should have anticipated the effect this contamination would have had on residential property values and on the owners themselves.

This case was appealed to the Ontario Court of Appeal (decision August, 1989, unreported at the time of the preparation of this paper), which did not seriously modify the trial decision. This case then raises some questions about a duty of disclosure arising out of plans and drawings that have been filed with various Provincial officials during the last few decades, insofar as those documents may disclose areas prone to subsidence and on which structures should not be or should not have been erected.

Finally, with respect to surface rights sold by mining companies for the purpose of the buyer erecting a house or other building, the facts of each case would determine whether there is any liability on the vendor company. Generally speaking, the "caveat emptor" rule would apply, except in those cases where it could be shown that there was knowledge by the vendor at the time of sale that the surface rights were unfit for the purpose intended, and that the vendor concealed that knowledge. (Even if this could be shown, one wonders how many corporate vendors still exist, or if they do, if they have assets to satisfy a judgement.)

## WILLING ASSUMPTION OF RISK

In theory the owner of surface rights which are found to be in a dangerous area could make an informed decision to continue to occupy those lands at his own risk. Again, depending on the facts of a given case, the discretionary powers under the Building Code Act to make an order prohibiting continued occupation of an unsafe building might not be exercised, but one can only conjecture about what type of situation might lead a municipality or Crown to permit that assumption of risk.

This paper has not addressed the question of civil liability for damages suffered by trespassers or recreational users on abandoned mining lands.

# RISK STRATEGY FOR THE TREATMENT OF ABANDONED LIMESTONE MINES IN THE WEST MIDLANDS OF ENGLAND

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## ABSTRACT

In the West Midlands there has been a long history of ground disturbance above abandoned limestone mines. A particularly severe disturbance in 1978 prompted a two year study starting in 1981 to determine a risk strategy. The works undertaken have included preliminary and detailed investigations of mines. In addition to rotary drilling to obtain core samples, the investigations have used ultrasonic surveying of mine cavities and downhole geophysical logging of the overlying strata. The ultrasonic surveys have confirmed the plan layout of mines, and have measured the changes in the shapes of deteriorating mine cavities. The early risk strategy was based on statistical and observational data. Subsequently, the physical conditions of the rocks, and the dimensions of the mine pillars and cavities, have been introduced into the risk strategy. The case history of the investigation and treatment of the Castle Fields mine is described.

## RÉSUMÉ

Le Centre-Pays Ouest a une longue histoire de perturbations de terrains au-dessus de mines de calcaires abandonnées. En 1978, une perturbation particulièrement sévère engendra une étude de deux ans qui débuta en 1981 avec but de formuler une stratégie de risque. Le travail entrepris inclua des investigations préliminaires détaillées de mines. En plus du forage au diamant pour échantilloner, les investigations ont fait l'objet de sondages ultrasoniques d'ouvertures de mines et de sondages géophysiques en trou appliqués aux lits surplombants les ouvertures. Les sondages ultrasoniques ont confirmé le plan des mines et mesuré les changements de forme des ouvertures souterraines dégradantes. La première stratégie de risque fut basée sur des données statistiques et des observations. Subséquemment, les aspects physiques du roc de même que les dimensions des piliers et des ouvertures de mines furent incorporés à la stratégie de risque. L'histoire de cas de l'investigation et du traitement de la mine Castle Fields est décrit.

INTRODUCTION

The Black Country, the industrial heartland of England during the Industrial Revolution, lay in the West Midlands, north west of Birmingham, see Figure 1. Coal, ironstone and limestone were abundant in relatively shallow seams which were extensively mined. The limestone was used as flux for ironmaking, or burnt to make lime for building and agricultural use.

Limestone mines were abandoned as they stood, some with cavities exceeding 10m high, neither being stowed nor deliberately collapsed. The rocks within and above the mines have gradually deteriorated, and various collapses have occurred, from minor roof falls having little affect on stability, to multiple pillar failures causing extensive ground subsidence.

A study of the abandoned limestone workings in the West Midlands was commissioned in 1981 by the Department of the Environment and the Local Authorities of Dudley, Sandwell and Walsall together with the then West Midlands County Council (Ref. 1). The objects of the study were to discover and record information about the mines, to develop a risk strategy in order to assess future priorities and to recommend methods that could be used in the future to alleviate the problems that are associated with the deterioration of the mines. The results of the study have been summarised by Cole, Turner and O'Riordan (Ref 2).

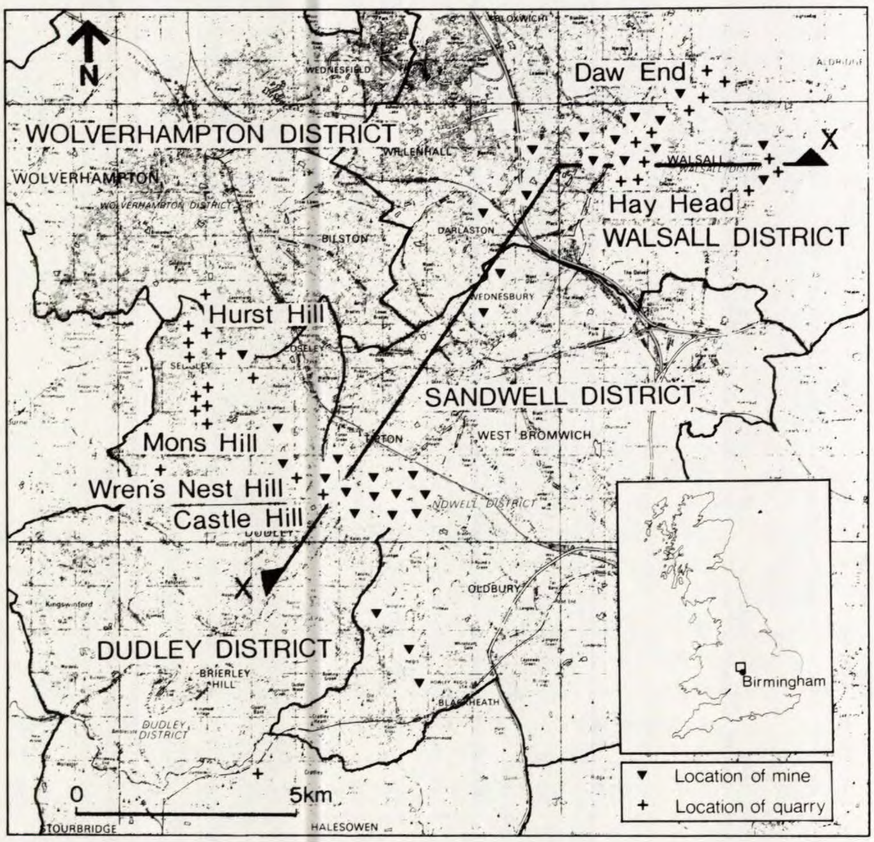



Figure 1 Plan of Black Country showing locations of limestone workings

Since 1983, the Department of the Environment has made derelict land grant aid available to the Local Authorities affected, to fund a programme of investigation and treatment of the abandoned limestone mines. Investigations into 25 mines have been made to date in Dudley, Walsall, Sandwell and Wolverhampton. The programme has proceeded in stages with preliminary investigations followed by more detailed investigations; these provide information on the extent and condition of the mine for the design of treatment measures.

Investigation methods adopted during the early years of the limestone project have been described by Braithwaite and Cole (Ref 3). The following sections summarise the local geology and the investigation methods used, in particular those that have been found to be of most value. The assessment of relative risk and importance is then discussed and the final section describes an infilling project.

## GEOLOGY

The strata containing the seams of limestone were laid down during the Silurian period some 400 million years ago. In general the limestone seams are found within grey mudstones which contain varying amounts of calcareous material. The Silurian strata within the West Midlands are shown in Figure 2 and are, in descending order with approximate thicknesses, as follows:

| | |
|---|---|
| Upper Ludlow Shales | up to 12m |
| Aymestry or Sedgley Limestone | 4.5 to 7.5m |
| Lower Ludlow Shales Shales Approx. | 150m |
| Upper Wenlock Limestone | 6 to 7m |
| Nodular beds | 30 to 36m |
| Lower Wenlock Limestone | 9 to 10m |
| Wenlock Shales Approx. | 150m |
| Barr or Woolhope Limestone | 10m |

The principal mined seams are the Upper and Lower Wenlock limestones, both mainly light grey fine grained limestones, with laminae of calcareous mudstone. The Upper bed was in general mined to a height of 4m and the Lower bed to 8m.

The Black Country lies in the southern part of the South Staffordshire Coalfield, and the Coal Measures lie unconformably on the Silurian Strata, the Devonian series of rocks being absent, see Figure 2. There are several productive coal seams, most notably the 9m thick Staffordshire Thick, and a number of ironstone seams. Many of the coal and ironstone seams were worked between 1750 and 1900, and mining ceased in the area over 20 years ago.

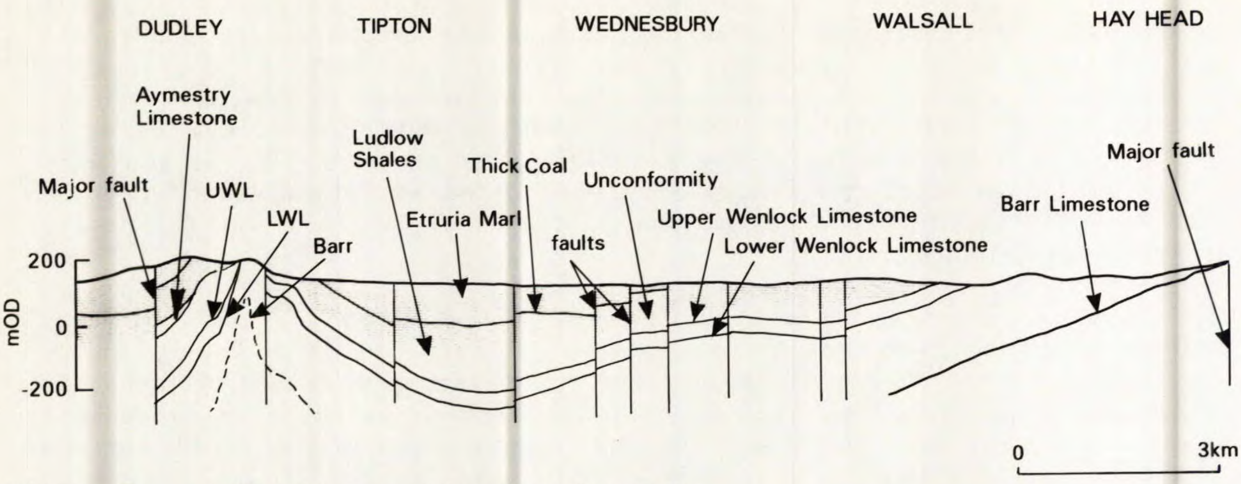



Figure 2 Geological section on Line X-X

NATURE OF WORKINGS

In both Walsall and Dudley outcrops of limestone are present and the earliest workings were quarries at the outcrops. As the quarries became exhausted the limestone was followed underground to form mines. Where the seams dipped steeply mining took the form of tunnels or "galleries" along the strike of the dipping seam, see Figure 3. Where galleries were made in the same seam at different levels they were often later interconnected by adits driven up dip to produce a pillar and room type working. Workings of this type can still be seen at the Seven Sisters Caves at Wrens Nest, Dudley, see Figure 18.

Where the dip of the limestone is much flatter, mining of the limestone was done by traditional pillar and room working, see Figure 4. The layout of a mine was largely determined by the dip, principal joint directions and faults of the limestone seam. Pillar spacing and the size of rooms varied between the different seams mined, typical dimensions being 5 to 10m square pillars with the spacing between pillars varying from 7 to 20m. Extraction ranged from 75% to 95%.

In all but a few cases the mines were below the natural water table and they flooded soon after they were abandoned.

Figure 3 Gallery working in Upper Wenlock Limestone

Figure 4 Pillar and room working in Lower Wenlock Limestone

## THE PROBLEM

Deterioration of the mines since they were abandoned has resulted in disturbances of the ground surface. Depending largely upon the depth of the mine, these are either crown holes or general subsidence. The majority of disturbances are of the crown hole type. Instances of general subsidence have resulted in widespread damage to surface structures.

### Crownholes

If the roof of a mine is less than 70m from ground surface then it is probable that collapse within the mine will progress to form a crown hole at the surface as shown in Figure 5. Crown holes are most common from workings in the Upper Wenlock limestone because of the weak nature of the overlying Lower Ludlow Shales.

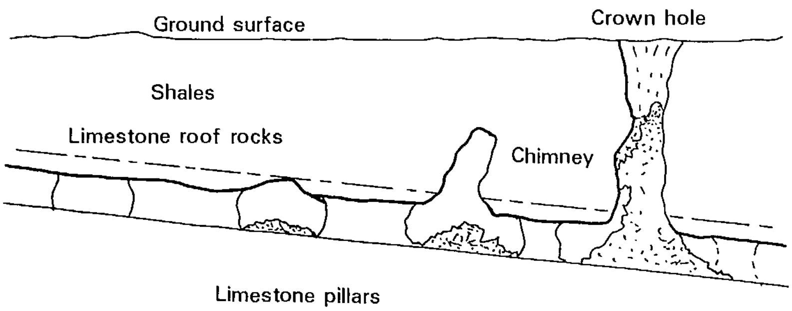



Figure 5 Roof failure leading to chimneys and eventual crownholes.

### General Subsidence

General subsidences are less frequent occurrences than crown holes. The cause is either collapse of pillars or crushing of the rocks immediately overlying the pillars. This has occurred at mines with a depth to the roof as little as 40m. The form of surface disturbance is similar to that produced by the effects of modern-day longwall coal mining, see Figure 6, the difference being that a localised "basin" of subsidence is produced, unlike the widespread and elongated "trough" of subsidence caused by coal mining.

A general subsidence occurred in 1978 above a mine some 140m deep at Wednesbury. It happened over a period of some six months, and resulted in a shallow "basin" of subsidence about 350m in diameter with the largest subsidence of 1.5m at the centre. An industrial estate of factory units was badly damaged and rendered unusable, but no structural collapse occurred. The factory units were subsequently repaired and restored to use.

Last year a similar subsidence occurred over another part of the same mine. Although not so extensive the largest subsidence was also about 1.5m, as simultaneous collapse of overlying abandoned coal and ironstone workings occurred.

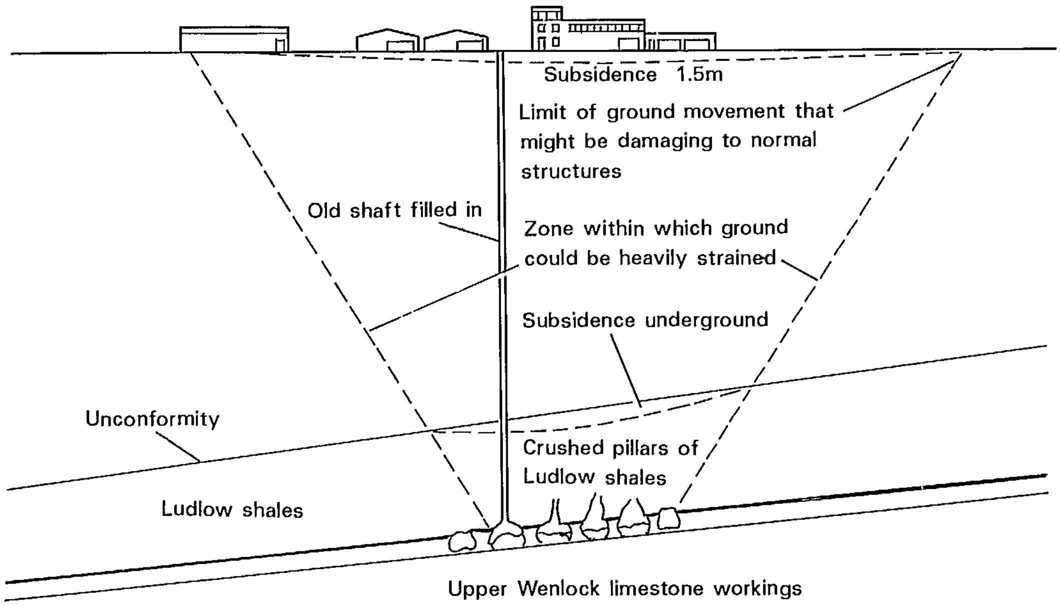



Figure 6 Pillar collapse or crushing leading to general subsidence; example of Wednesbury mine

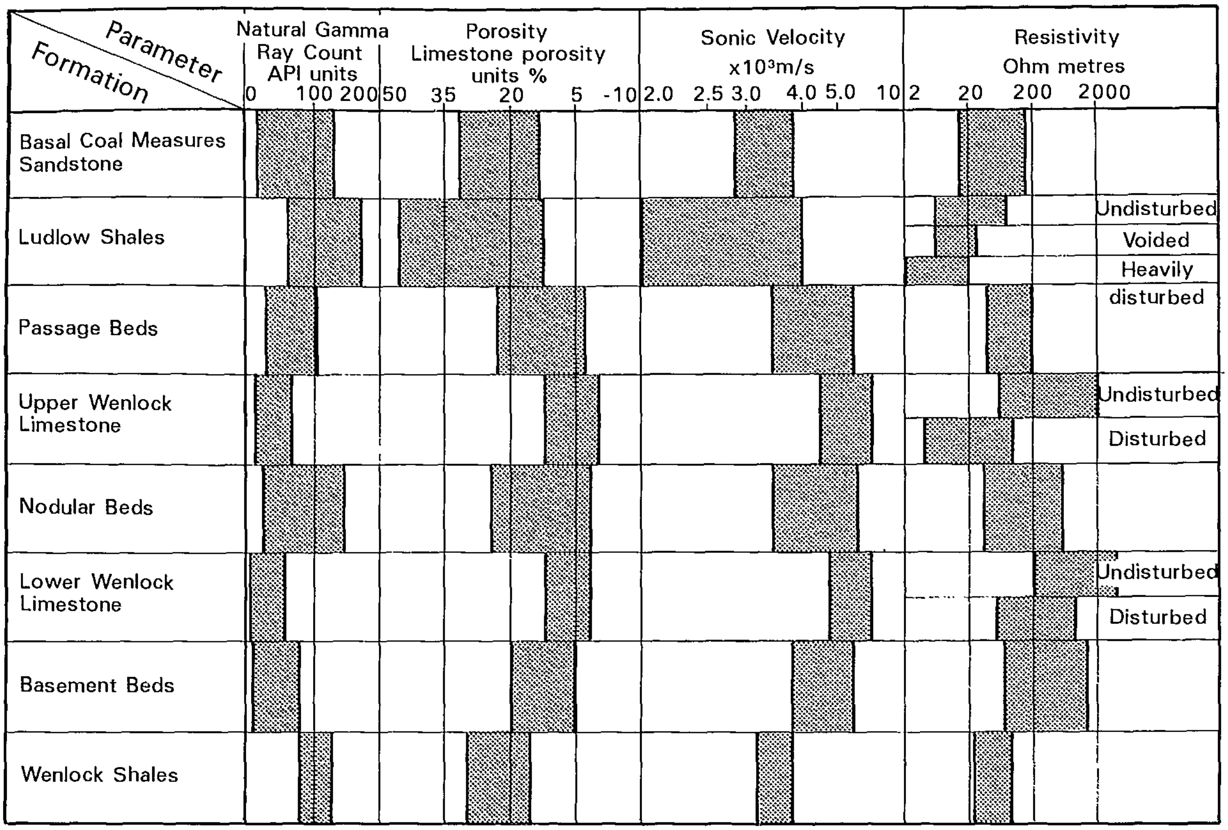
## INVESTIGATIONS

### Drillholes & probeholes

The principal method of investigation, to locate abandoned mines (some of which are poorly recorded) and discover the condition of the remaining rocks, is by drillholes and probeholes. Core samples are obtained from drillholes, whereas generally only the drilling cuttings are available for examination from probeholes, in which case the sections of hole made are referred to as "open-hole". The flushing fluid for both drillholes and probeholes is specified as "clean" (fines free) water, as the use of air reduces the chances of good core recovery and seriously affects the accuracy of ultrasonic surveying by locally changing the density of the water. The problems of the identification of abandoned limestone mines by investigation methods have been described by Jackson and Braithwaite (Ref. 4).

All core samples are logged in great detail, and high-quality photographs taken for the record. Drillholes and probeholes entering voids greater than about 1m in depth are permanently lined with plastic lining tube grouted into place using a tube-a-manchette process, after the temporary steel lining tube is withdrawn.

Preliminary investigations comprise mainly drillholes from which cores are obtained in the Silurian strata including the Upper and Lower Wenlock Limestones. The Coal Measures are generally penetrated by open hole drilling. In further stages of investigations greater use is made of probeholes drilled entirely by open hole techniques. Penetration rates, water flush losses and in-situ permeability from packer tests are measured to provide supplementary information on rock condition for qualitative comparison.




NOTE: Parameters based on data gathered up to August 1985 and are for uncased holes

Figure 7 Downhole geophysics in water-filled holes; correlations with strata

Geophysical Logging

Geophysical logging is carried out using 65mm diameter tools suspended by wireline in the boreholes. The logging is done before permanent casing is installed to open mine cavities, or before backfilling if no mine cavity has been encountered. The sondes used include gamma ray, density, caliper, compensated neutron, multichannel sonic, focussed electric (resistivity) and dipmeter. All of the logged measurements are dependent on lithology, and to varying degrees on rock mass condition. The response of the different sondes to the lithological units encountered in these investigations is summarised in Figure 7.

The output from the combination sonde, comprising gamma ray, density and caliper instruments, is used to compute the relative proportions of shale and sandstone in Coal Measures, or mudstone and limestone in Silurian strata. An example from one probehole is shown in Figure 8. The analysis has been run twice, once to compute the lithology log for the Coal Measures strata and then for the Silurian strata. The computed lithology logs have been found to show good agreement with descriptions of cores from both water and air filled holes.

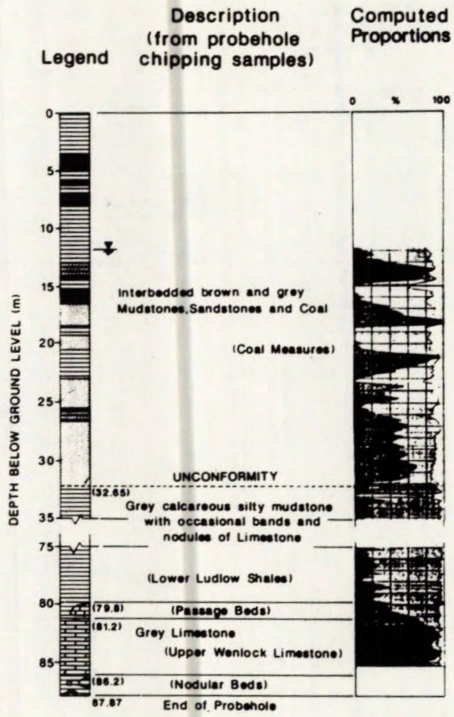



Figure 8 Computed lithology logs of Coal Measures and Silurian strata compared with strata descriptions

The degree of fracturing within the rock mass affects the response of many of the geophysical measurements made. Figure 9 shows tentative relationships between the degree of fracturing and porosity logged by the compensated neutron sonde, against the gamma ray results. The Ludlow Shales and the Nodular Beds are those which overlie the principal worked limestone seams; the amount of disturbance of the seams, measured by the degree of fracturing, see Table 1, is significant in considering the potential for collapse of the mine.

Table 1 - Fracture Index and Fracture State:

| Degree of Fracturing | Fracture Spacing mm | Fracture Index mm | Fracture State |
|---|---|---|---|
| Very widely spaced | > 2000 | < 0.5 | Scarcely |
| Widely spaced | 600-2000 | 0.5-1.7 | Slightly |
| Medium spaced | 200-600 | 1.7-5.0 | Moderately |
| Closely spaced | 60-200 | 5.0-17 | Highly |
| Very closely spaced | 20-60 | 17-50 | Very Highly |
| Extremely closely spaced | < 20 | > 50 | Intensely |

Geophysical correlations have allowed a higher proportion of probeholes to drillholes to be used in the investigations. The information from the probeholes has been greatly enhanced, and a saving on the amount of core sample taken has been achieved.

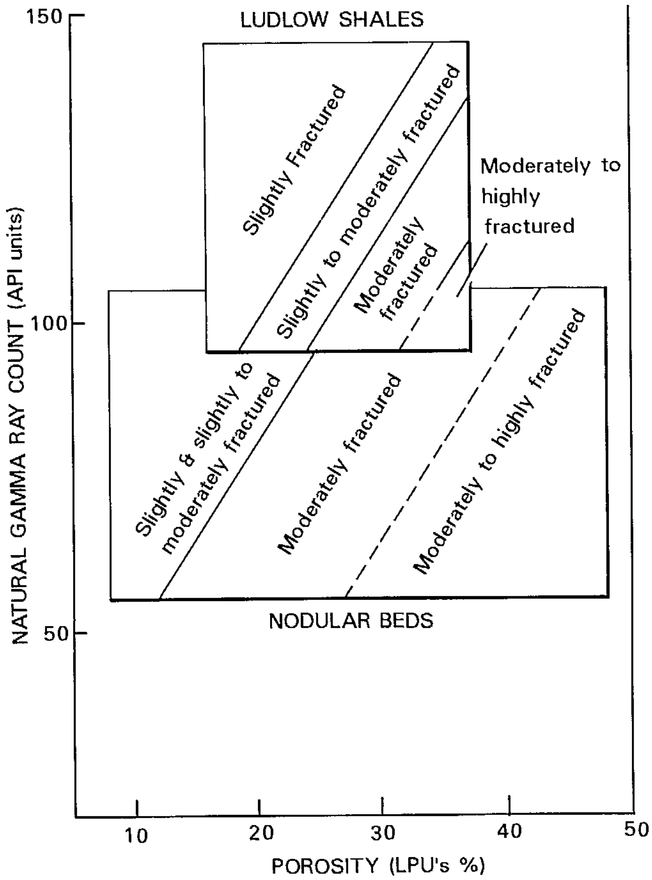



Figure 9 Tentative relationships between the degree of fracturing and porosity (logged by compensated neutron sonde), against gamma ray count

## Ultrasonic Surveying

The ultrasonic surveying tool can operate in air but is very restricted in range (not more than 5m). Underwater the tool can receive a clear "echo" of its ultrasonic pulse from a solid surface as far as 80m away from the tool.

The principle of the tool is illustrated in Figure 10. Two ultrasonic pulse emitters/receivers are attached to a tilting and rotating head. This enables them to be rotated (step-wise) through 360° and tilted (also step-wise) from vertically downwards through horizontal to nearly vertically upwards, giving nearly spherical "cover". At each step a pulse is emitted and the time for the "echo" to return is measured, thus giving the distance to the solid surface.

The two systems currently available require a minimum 90mm internal diameter cased hole. Both provide output which enables the positions of solid surfaces to be recorded and plotted on plans and vertical sections to an accuracy of 0.1m. The cost of ultrasonic surveys is a significant proportion of the cost of drilling and testing. However the value of being able to "see" the shapes of cavities, and detect changes in shapes between successive surveys has proved to be invaluable.

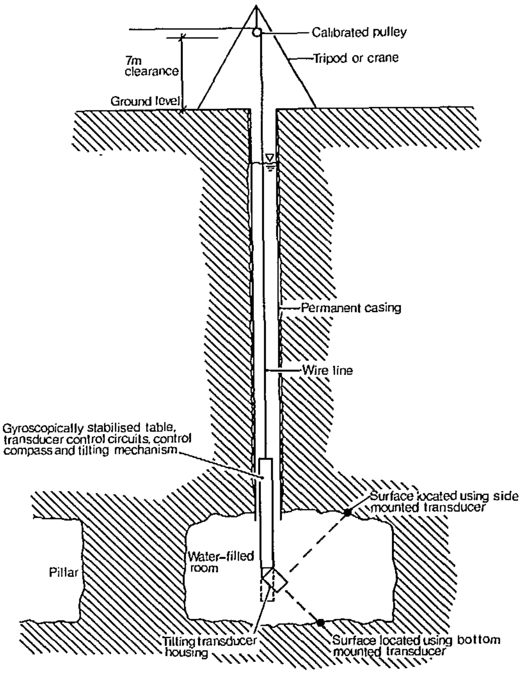



Figure 10 Downhole ultrasonic surveying tool with two pulse emitters/receivers mounted at right angles

A laser scanner for use in air filled cavities has been developed. The 89mm diameter tool is lowered down a 120mm minimum internal diameter cased hole. The recorded data and horizontal plans and vertical sections produced by laser surveys are similar to those derived from ultrasonic surveys.

To ensure that an accurate relationship in plan is achieved between the drillhole at surface and the hole at the level of the cavity roof, each hole greater than 50m deep which enters a cavity is checked for verticality by a multi-shot gyroscopic compass. By this means, accurate plans and sections can be drawn of inaccessible cavities and their locations can be tied into the National Grid.

Closed circuit television (CCTV)

Closed Circuit Television (CCTV) with video recording is still used to a limited extent. The main problem in water filled voids has been the restriction on the depth of view caused by back-scatter of light from suspended particles. Because of this the use of down-hole cameras in water filled cavities is limited to inspection of features within a few metres of the camera. The system can be of qualitative use in air-filled voids as viewing distances are greatly increased but no reliable measurements of size or distance can be made.

RISK STRATEGY

An object of the study of the abandoned limestone workings in the West Midlands, reported in 1983 (Ref 1), was to devise a risk-based strategy for future action. see Ref 5. It became clear as the study progressed that there was inadequate information on the quality of the rocks within and above each mine. In addition, although 100 collapse events spanning 150 years had been identified, in the main the exact nature of each collapse was unknown, only a few had caused damage, and no fatalities had been recorded.

An analytical approach to determine the physical factors controlling stability was therefore not feasible, and a combined observational and statistical approach was advised.

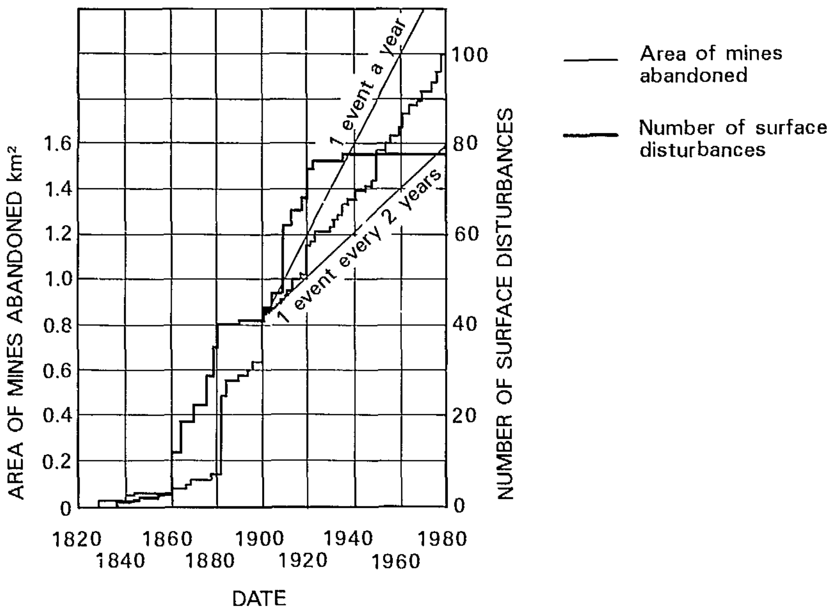



Figure 11 Number of surface disturbances since 1828 against time

## Frequency of disturbances

The cumulative area of mines abandoned, and the cumulative number of identified surface disturbances are shown in Figure 11. The disturbances were mainly crownholes, many identified from successive publications of the large scale Ordnance Survey maps. There appears to be a rough relationship between the frequency of disturbances (about 1 a year) and the area of mines abandoned throughout the period to 1935 when the last mine was abandoned. Between 1935 and 1980 the recorded disturbances continued at a frequency of about one every 1½ years, but in the last few years the frequency has reduced to one every two to three years.

## Time since abandonment

Figure 12 shows the number of surfaces disturbances in various time periods since the respective mines were abandoned. It indicates that, since the majority of mines were abandoned 70 to 130 years ago the frequency of disturbances should be about one every 2½ years ( 9 events in 20 years), which is in good agreement with recent records.

The distribution in Figure 12 also indicates that in future the frequency of disturbances will continue to fall, reaching about one every five years in 70 years time when the majority of mines have been abandoned longer than 140 years.

This situation may be thought to be contrary to the concept that, as the time since mining stopped grows longer, so the affected rocks deteriorate more rapidly. However, deterioration has been found to be most rapid (collapse within tens of years) where the strata are already severely disturbed, or are highly stressed as a result of the mining activities; our observations have shown that at such places within individual mines localised deterioration proceeds towards collapse while the remainder of the mine is commonly unaffected. Eventually, in "geological" time, it is probable that even the least disturbed and lowest stressed parts of mines will collapse.

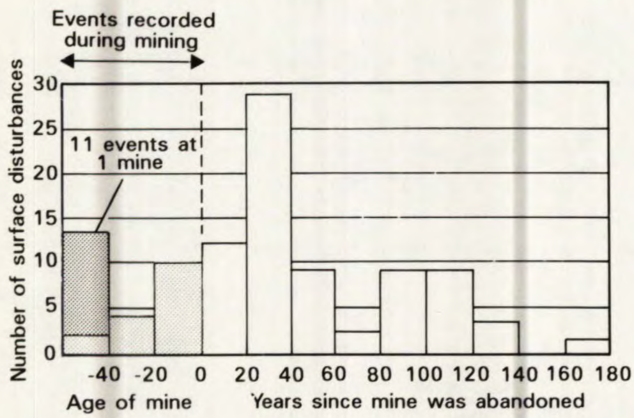



Figure 12 Number of surface disturbances since mines were abandoned

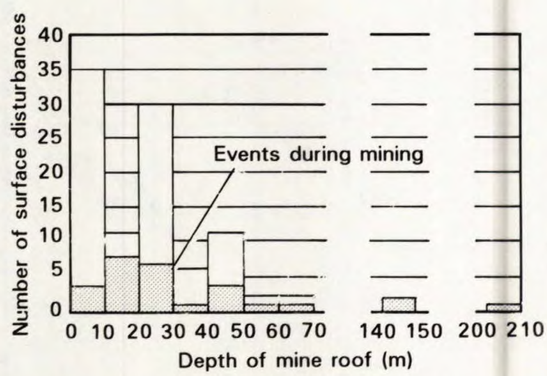



Figure 13 Number of surface disturbances related to depths of mines

Depths of mines

Over two thirds of the identified surface disturbances have occurred over mines shallower than 30m, see Figure 13, and these have all been crown holes. The greatest depth of mine giving rise to a crown hole has been 70m, but it is considered possible that a crownhole could result from collapse in a mine as much as 100m deep.

Four disturbances (one recent event at 140 to 150m depth not shown on Figure 13) have occurred where the mine is at a greater depth than 100m, and all have been of the nature of general subsidences.

Layouts of mines

Mine plans exist for many of the known mines, and in general where the mines have been surveyed by the ultrasonic tool the plans have been shown to be reasonably accurate. Where the mines are still "open" the dimensions of pillars and distances between walls are generally in good agreement, but the position of the mine in relation to surface features may be 10m or more in error.

Figure 14 A pillar believed to have been "robbed"

However, many mines have been found to be so fully collapsed that it has not been possible to take measurements by ultrasonic surveying, and reliance has had to be placed on drillhole and probehole information to locate very approximately the positions of the original mine cavities. A further difficulty in such mines has been in deducing what caused them to become so fully collapsed when the dimensions of the cavities and pillars on the mine plans suggest (from examples elsewhere) that collapse was generally unlikely.

The most ready explanation is that the mine pillars were severly "robbed" to the point of collapse, as a means of getting cheaply and easily removed limestone, when the miners withdrew from the mine; the results of such activity would not appear on the mine surveyor plans! A photograph of a pillar believed to have been "robbed" is shown in Figure 14; this was found when an abandoned mine 50m deep was opened up in the 1940s to become an ammunitions dump.

Strata

The Aymestry, Upper Wenlock and Barr limestones are all overlain by mudstones (the Ludlow and Wenlock Shales). These are moderately strong rocks in the intact state, but weather deeply on exposure, separating along the closely spaced bedding planes to leave lithorelicts in a clay matrix, see Figure 15. Generally the calcium carbonate content is low, although there are occasional massive "reefs" of limestone.

The Shales usually show a transgression to limestone at the top and bottom of each seam. The so-called "Passage Beds" above the Upper Wenlock Limestone were left in place by the miners as protection against roof collapse. As shown in Figure 5, when they eventually give way the overlying Lower Ludlow Shales are liable to quite rapidly deteriorate and fall away in thin layers to give rise to chimneys with near-vertical sides. Eventually such chimneys undermine the surface and a crown hole forms, often quite suddenly, see Figure 16.

Figure 15 Near surface exposure of weathered Ludlow Shales at steep angle of dip

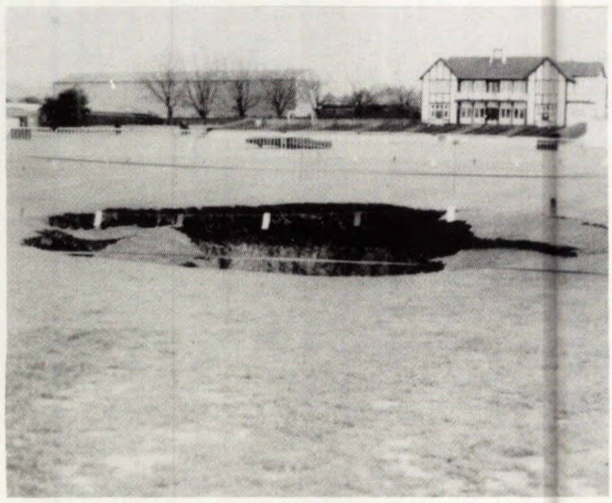


Figure 16 This crown hole appeared in the cricket pitch on the morning of Saturday 25 May 1985. There was no prior warning

The Nodular Beds some 30 to 36m thick, lying between the Upper and Lower Wenlock Limestones are strong and resistant to weathering, see Figure 17. Thus chimneys and crown holes are unlikely to develop unless, as has happened, pillar failures have occurred in Lower Wenlock Limestone mines, as a result of suspected pillar "robbing", removing support from an area of

Figure 17 Near surface exposure of Nodular Beds at a steep angle of dip

Figure 18 Shallow vaulted roof in Nodular Beds above Lower Wenlock Limestone mine dipping at about 45°

<u>Information available for a risk strategy</u>

Prior to the study (from 1981 to 1983) there had been no high quality investigations of the known mines, and no concerted attempts had been made to search for evidence of other mines which might not have been well recorded.

During the study a limited amount of drillhole investigation was made at three mines at Wednesbury, Daw End and Littleton Street in Walsall, so that specific experience could be gained in, and recommendations made concerning the investigation techniques described here. Details of the ground conditions at only three small parts of three out of 31 known and suspected mines were therefore available to be considered. Specific findings included;

i) the ability of the ultrasonic tool to accurately measure the shape and size of underwater cavities and thus enable the mine plans to be verified, or newly drawn if none existed.

ii) the ability of a combined drillhole/probehole/geophysics investigations to identify the degree of disturbance of the rocks in and above the mine cavities, leading to the possibility of correlation with the degree of collapse of the mine.

iii) the likelihood of a correlation between pillar size and strength and pillar failure, particularly in relation to crushing of the shales exposed above the mine pillars following collapses of the mine roof.

The above listed findings put the state of knowledge of the three mines concerned so far above that of the remaining mines, that it was clearly not possible to attempt a unified risk strategy, except at the level of the basic information common to all mines.

<u>Potential for Collapse</u>

The concept of Potential for Collapse was that it should indicate the relative likelihood, that one mine was more likely to give rise to surface disturbance than another mine.

The potential for collapse of each of the 31 mines (24 had plans, and the remainder were suspected to exist on the basis of at least two pieces of evidence) was therefore devised by factoring

a) the time since the mine was abandoned
b) the depth of the mine
c) the type of the strata above the mine

Individual mines were accorded High, Intermediate or Low Potential for Collapse depending upon their relative position in the range. In the presentation of the study (Ref 1) the Potential for Collapse was combined with an assessed importance of the surface features (social value) to give the Relative Risk of surface disturbance. The table had some merit in that it gave the mines, thought to have the highest relative risk, the highest rating. The actual risk at the highest category was assessed in the study as being at an annual rate of about 1 in 500 for structures only (comparable to the risk of a fire in a house). Adopting the risk nomenclature proposed by Cole (Ref 6) this would indicate "some risk" tending towards "risky".

The concept of potential for collapse has been maintained in reporting on the state of some 25 mines investigated since 1983. In several cases, as explained under "Layout of mines" above, part or all of mines has been found (unexpectedly in some instances) to be fully collapsed. In these cases all potential for collapse (surface disturbance) has been eliminated and the "Consideration Zone" area above the mine is recommended to be removed.

Consideration Zone

A "Consideration Zone" was a line drawn around the plan area of each mine to indicate the area within which consideration should be given to the likely affects of the presence of a mine. It was drawn at a distance from the mine edge at which calculations (using essentially the NCB method (Ref 7)) show the horizontal ground strains are 0.2% and thus the damage to domestic structures is likely not to be greater than "very slight".

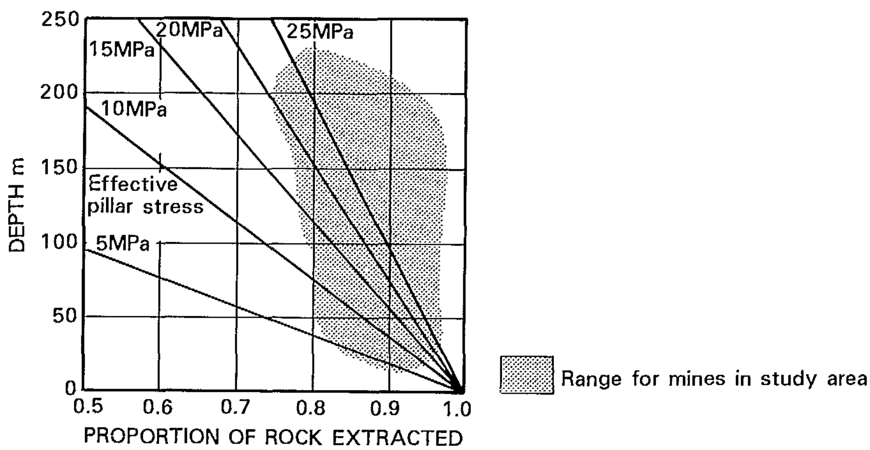
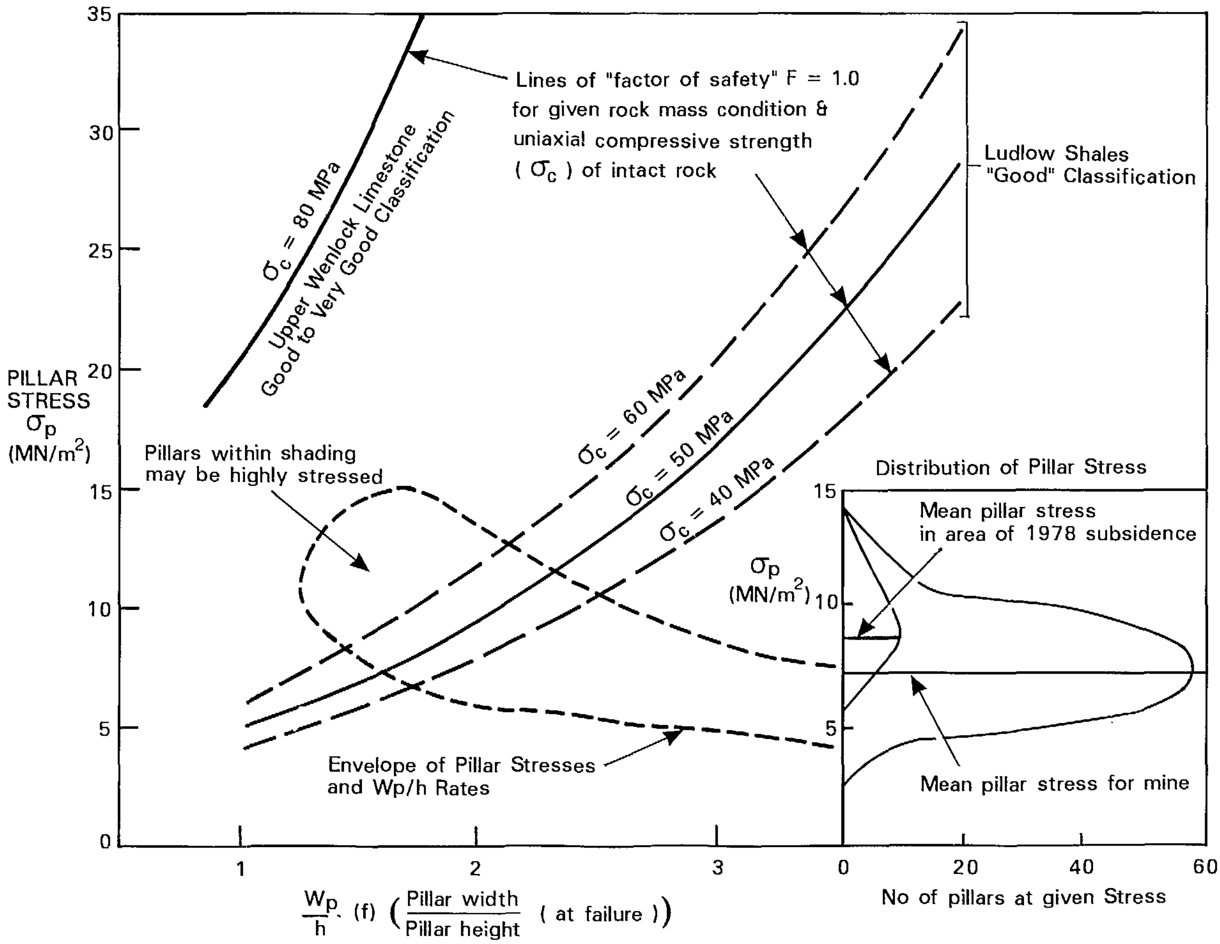



Figure 19 Extraction, mine depth and calculated pillar stresses

Pillar Failure

Some concept of the wide range of calculated pillar stresses is given in Figure 19. Extractions exceeding 95% were common in some of the shallower pillar and room mines. It has already been mentioned that the likelihood of a correlation between pillar size and strength, and pillar failure, was one of the findings of the 1983 study report. The correlation is explained in the paper by O'Riordon, Cole and Henkel Ref 8, the basis being the empirical failure criteria for rock masses developed by Hoek and Brown (Ref 9), in association with rock quality systems devised by Bieniawski and NGI (both detailed in Ref 9). Since writing the paper the manner of presentation of the figures showing the correlations between pillar sizes and strengths (pillar materials) and pillar failure has been changed, so that now calculated pillar stresses against pillar width upon height ratios are plotted. The plotted points are then referred to lines of F = 1.0 for various rock classifications "good", "fair" etc.) and rock strengths, see Figure 20 for an example.




1 Pillar stress calculated by tributary area theory

2 Wp measured from mine plan or obtained from ultrasonic survey; h obtained from drillholes

Rock mass classifications are based on CSIR method, Hoek & Brown (Ref 9)

Figure 20 Relationship between pillar dimensions (wp/h), at failure (F= 1.0), calculated pillar stress ($\sigma_p$) and rock condition ( $\sigma_c$ and rock mass quality).

For several mines quite a large proportion of the plotted points lie above the F = 1.0 line for the assessed mass quality of rock in the pillars. In these cases further assessment of the actual condition of the pillars is made. This takes into account the likelihood that the pillar may be larger than revealed by the ultrasonic survey, and part of the load may be supported by the nearby mine walls or larger pillars. Where the mine has suffered considerable amount of roof collapse the mine cavity is assumed to have "risen" into the overlying stratum and the properties of this stratum are then used in deriving the F = 1.0 line. If the overlying stratum is weaker and more heavily fractured than the limestone then failure of the "pillars" in the overlying stratum may occur before failure of the underlying limestone pillars.

Such failure (crushing) is believed to have occurred at several of the Upper Wenlock limestone mines deeper than 100m. Even allowing for the mines being flooded the overburden pressures are large, and the overlying stratum is the weaker Ludlow Shales. In particular it has been shown to have been the probable cause of the two general subsidences which have taken place in recent years above the mine at Wednesbury.

Vulnerability of mines deeper than 100m

The Potential for Collapse described above was derived on the basis of disturbances above mines deeper than 100m having low frequency of occurrence, see Figure 13. This may have been true in 1981-83, as most deep mines may have been either already largely collapsed, or were largely "open". It seems possible that each particular mine could reach a critical state of deterioration over much of its plan area at much the same time, and the mine would then become "vulnerable" to a succession of collapses each giving rise to a general subsidences, with periods of quasi-stability between.

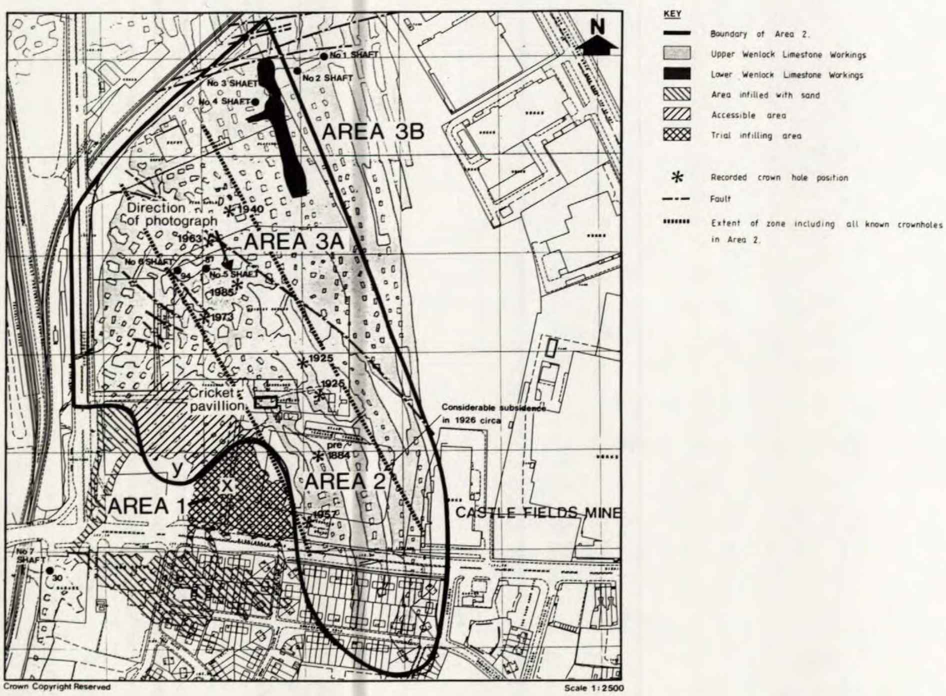



Figure 21 Plan of Castle Fields Upper Wenlock Limestone mine showing areas and locations of crown holes.

CASTLE FIELDS MINE, DUDLEY

Mine Layout

The Castle Fields Upper Wenlock Limestone Mine in Dudley, was worked between 1845 and 1910. The mine had a plan area of about 20 hectares, and an estimated volume of about one million cubic metres. A complete plan does not exist, and the mine plan shown in Figure 21 is a composite made from several old overlapping plans, which do not accurately match.

Plans of the earlier stages of working show that the mine was worked as rooms and pillars on the western side, where the depth to the floor is less than 50m and the dip is less than 15°. On the eastern side the dip increases to 45° to the east-north-east and the mine reached a depth of 145m. The mine on this side was first worked as galleries, along the strike of the seam, followed by secondary working of the rib walls between the galleries leaving rows of relatively thin pillars. In some areas the pillars may have been completely removed.

Mine plans also show a north west to south east trending fault crossing the southern area of the mine. Parallel to this fault there is a zone between dotted lines within which crown hole collapses have occurred, see Figures 21 and 16. South of this many of the pillars are large and of irregular shape, the directions of pillar faces being determined by the dominant joint sets.

The overlying ground surface had several different uses, and the site was therefore sub-divided into Areas 1, 2, 3A and 3B according to its importance.

Investigations

The shallowest area of the mine was some 20m deep and was above the natural water table. The majority of that area was accessible until it was filled as part of a trial of the use of "rock paste", see below. Figure 22 shows photographs taken in the mine at locations X and Y, of the joints in the roof rocks, roof collapse debris on the floor, and a chimney reaching to within 8m of ground level.

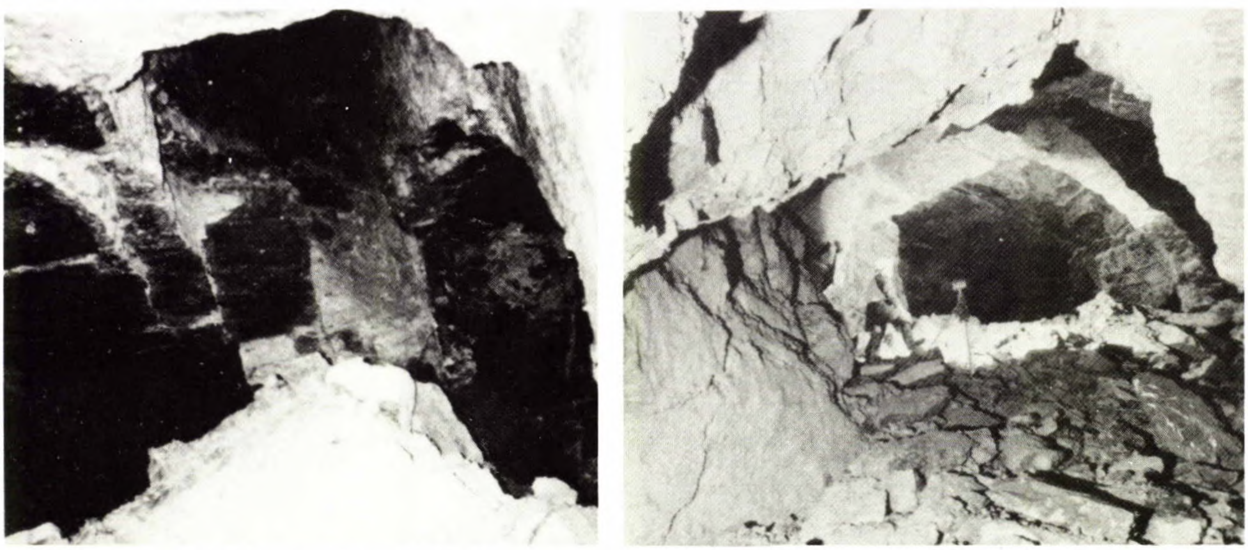


Figure 22 Photographs taken in Castle Fields Upper Wenlock Limestone mine. Left photograph showing an "early" chimney was taken at position X; right photograph showing chimney within 8m of ground surface was taken at position Y.

The major part of the Castle Fields Mine was inaccessible, because it lay below the water table and was therefore flooded. A drillhole and probehole investigation was made to discover the condition of the mines and that of the strata above.

Separate Stage 1 (Preliminary) and Stage 2 (detailed) investigations were made, the Stage 1 comprising 33 drillholes and the Stage 2 13 drillholes and 58 probeholes. The full range of ancillary investigations were made, by geophysics, and ultrasonic and CCTV surveying, during both stages.

Results of the investigations

The results of the investigations showed that many parts of Areas 2 and western side of Area 3A remained "open", with cavities up to 6m high. The mine layout was largely in accordance with the mine plans, with no evidence of pillar "robbing" in the open areas.

The eastern side of Area 3A and much of Area 3B had collapsed, as was indicated by the absence of pillars, or very small pillars, shown on the mine plan. Within the whole mine extraction ranged from below 75% to in excess of 95%.

In many of the drillholes the core recovered showed the Ludlow Shales for much of the depth from the surface to be moderately to highly fractured (joints at 60 to 600mm spacing) rather than slightly fractured (joints greater than 600mm spacing) for intact shale. Joints were commonly stained indicating percolation of water to the mine when it was being worked. In several drillholes which penetrated broken ground (the result of roof collapse into the mine cavity) there was a small void at the top of the broken ground.

Geophysical records were obtained from drillholes and probeholes in the intact and disturbed strata, but it was not practical to lower the sondes into holes through broken ground, for fear of losing them. Broken ground was therefore largely determined in probeholes by the increased rate of penetration combined with the total "loss" of flushing water into the broken strata.

The report on the investigation concluded that the mine was of high relative risk and infilling of the open cavities with rock paste was recommended. Where the cavities beneath an estate of terrace houses (several houses attached) were largely choked with collapsed roof material, it was recommended that the collapsed material should be solidified using a cement/pfa grout.

Rock paste

Many of the mines to be filled are extensive in plan, and the height of workings is typically about 3m to 8m; thus a cheap form of bulk infill is needed as cost is a major consideration. Rock paste, which primarily consists of colliery spoil and water mixed together to form a paste, was thought likely to be able to fulfil the requirements. Initial pumping trials were carried out by the Building Research Establishment during 1893

(Ref 10). These indicated that rock paste was capable of being pumped at upto $80m^3$ per hour through 220m of 260mm diameter pipeline. Trials also showed that rock paste was capable of flowing for long distances on surfaces with a small gradient.

The advantages of rock paste were thus seen to be the minimising of the number of surface injection holes, as compared with sand flushing, and the use of cheap waste materials. A dual benefit of restoring a spoil source site while stabilising an underground working also accrued.

The development of rock paste as a bulk infill material was carried out under the supervision of Ove Arup and Partners and culminated in 1985 with a full scale trial infilling of part of the Castle Fields mine beneath the Dudley Sports Centre Area 1 (Ref 11). Figure 23 shows the rock paste flowing into the mine during the trial infilling.

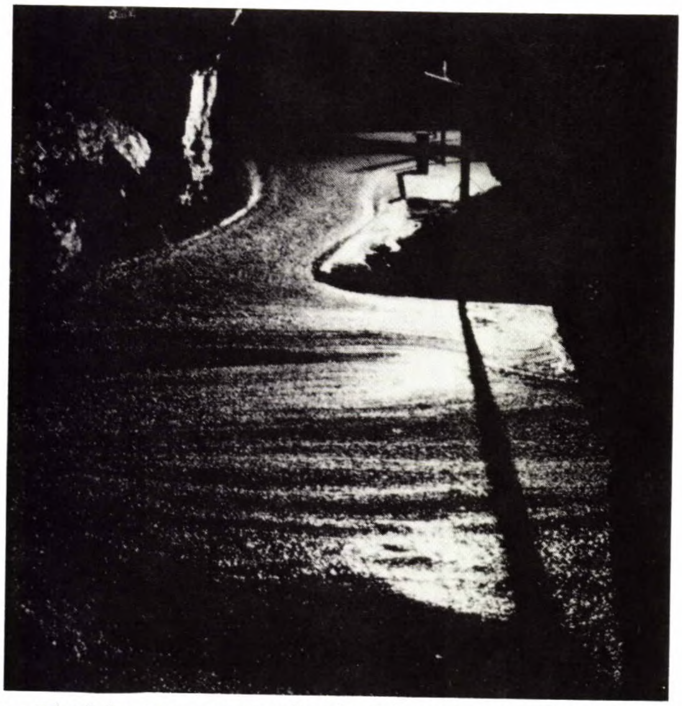


Figure 23 Castle Fields Upper Wenlock Limestone mine Area 1, showing rock paste flowing into mine

Infilling of Area 2

After the completion of the trial in Area 1 in October 1985, agreement was reached between the Department of the Environment and the Metropolitan Borough Council of Dudley for Ove Arup and Partners to proceed with the design for infilling of Area 2.

Area 2 lies to the east of the trial area, see Figure 21. This section of the mine dips eastwards and is at a depth of between 20m and 80m below ground surface. The average height of the workings is about 4m. The western half of the workings are submerged and the eastern half are dry. There was no safe access into the workings so all monitoring had to be carried out from the surface.

Since the completion of the trial in Area 1 further development of rock paste had been carried out Cole & Figg, Ref 12. The rock paste used for the Area 2 infilling contract comprises colliery shale, water and small percentage of pulverised fuel ash (pfa) and lime. The addition of pfa and lime ensures a gradual increase in strength with time and also the amount of consolidation is reduced as compared with the plain rock paste in the Area 1 trial, Ref 13.

Part of the specified works was to infill 60 000m$^3$ of open mine workings with rock paste. During the Contract, the works were extended to include topping up Area 1 above the consolidated rock paste.

The contractor used an electronically controlled Belmix continuous mixer. Covered conveyors delivered pfa from a storage site to the mixer and colliery spoil from the covered store to the mixer. Lime was stored in a silo and fed into a tank where water was added. The lime water slurry was kept agitated until fed into the mixer.

Two 65m$^3$/hour capacity Hydroseal pumps powered by 110 kW motors fed rock paste along th 200mm nominal diameter pipelines. The pipeline was of seamless steel pipes with two bolt, two piece pipe couplings of forged steel with rubber gaskets, similar to that used for the trial. Pressures up to 50 bars were expected.

The method of infilling was generally to inject the rock paste on the up-dip side of the mine, allowing the paste to flow down-dip to the lower extremities of the mine. The paste then rose and filled the higher parts of the mine, 'backing-up' towards the injection point and filling all hollows in the roof.

Test on rock paste

A routine for checking the making and placing of rock paste was developed, based on the tests carried out during the trial. The strength gain of the rock paste was measured at intervals to ensure that the design strength of 13kN/m$^2$ would be attained one year after completion of infilling.

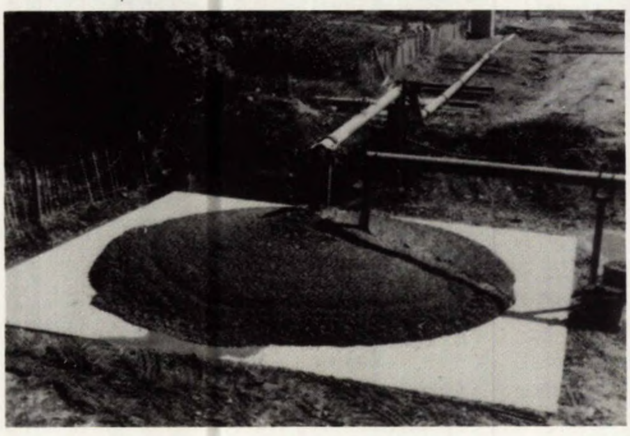


Figure 24 Rock paste heap test to determine paste flow characteristics

Heap tests were carried out, and recorded photographically, at the start of using a new colliery spoil source, see Figure 24. From these tests the flow qualities of the rock paste at various placement strengths could be measured.

Large samples of rock paste were placed in $5m^3$ skips and stored for at least 100 days, with the rock paste under water. The strength of rock paste was measured in the skips, using a plate penetrometer, at intervals in time. These results, together with similar results from tests on small samples stored in oil drums, provided a guide to the strength gain of the rock paste, see Figure 25.

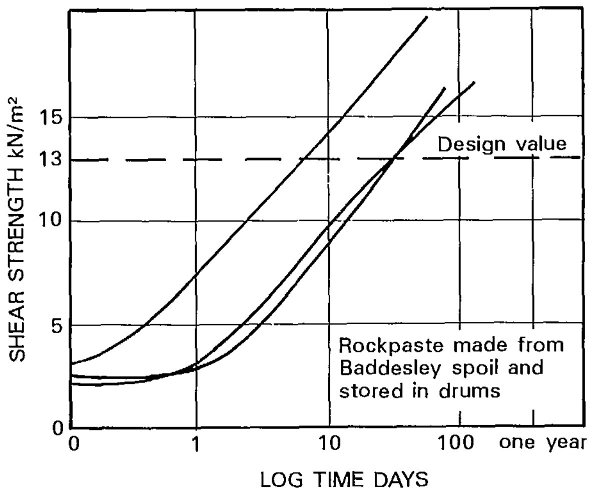



Figure 25 Gain in undrained shear strength with time of rock paste. Strength measured with plate penetrometer

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