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Project

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Project Number 60619

Curragh Resources Ltd.
117 Industrial Road
Whitehorse, Yukon
Y1A 2T8

Attention: Mr. Leo Hwozdyk

Dear Leo:

RE: **Room and Pillar Review**

The following is a brief report of the visit by Chris Page and Jim Mathis to the room-and-pillar operations at Faro during the week of 19 June.

The following points are briefly discussed under separate headings:

- underground stability
- rock support
- blasting
- pillar design procedure
- initial room-and-pillar layouts and procedures

The overall impression is of reasonable stability within a generally competent rock mass. In the southern section, though, the excavations usually have a sulphide roof. Where hanging wall material is exposed, patterned roof support is necessary. There will be a considerable difference between the present conditions and final extraction when the room size will have doubled and when most of the roof will be in hanging wall material. The present standards of excavation control and support installation are not adequate. Serious considerations should be given to leaving ore in the roof to reduce support requirements.

Mining cost is a key element in the operation. Operating costs will be sensitive to the room size but as the room size increases the need for more intensive support will quickly override the savings in production costs. The operating costs will be extremely sensitive to room stability. It may be necessary to limit percentage extraction in favour of overall cost reduction. This factor is addressed in the section on layouts.

The mining operations are proceeding in a fashion typical of a tunnelling contract. Room-and-pillar operations require much more careful control. As you are aware, the operations in Elliot Lake depend on control of continuous roof beams, careful cutting of pillars, efficient scaling of the roof and a tight pattern of well installed bolts. This level of care is not evident.

Finally, the use of numerical analysis technique to improve the calculation of pillar stresses, rather than using "tributary area theory", could increase the percentage extraction from approximately 70% to 81%. The analysis could be carried out in two phases; the first to demonstrate the potential reduction in pillar stresses and the second to evaluate actual pillar stresses. The approximate cost of the first phase would be three thousand dollars.

Yours truly,

STEFFEN, ROBERTSON AND KIRSTEN (B.C.) INC.

Gerald Mathis

for C.H. Page, P. Eng.

CHP/007cs
Attach

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1. Underground Stability

The underground excavations are generally very stable and safe. The ore material is strong and competent and most of the development is in ore. There are very few exposures of the 'true' waste hanging wall. Where these exposures occur the roof is often incompetent. Mineralised graphitic quartzite is very similar to the ore, especially where is has been silicified.

A detailed review of the southern area was undertaken. Figure 1 shows a plan of the area with rock type and a measure of the roof stability. This indicates that 55% of the 'true' hanging wall requires fairly intensive conditioning. Of approximately 2500 feet of development in the southern area only 15% has a non-sulphide roof.

In the SRK report 60610, December 1988 it was estimated that some 35 to 40% of the roof would require intensive support (bolting, mesh, straps and/or shotcrete). This value is supported by the conditions seen in the southern area with reference to the limited amounts of hanging wall that have been exposed.

The decision has to be made, as discussed in the section on roof support, whether to leave a 'skin' of sulphide ore against the hanging wall. This skin would have to be at least 3 feet thick with 30 foot rooms. There is not a clean, physical break between high grade massive sulphide and the graphitic quartzite but when very thin wedges of sulphide are left in the roof they tend to fall. But there are many instances of a 'mixed' ore and waste roof, Figure 2. The concern with thin bands of ore is that they may support, temporarily, very weak material. There is an instance of extremely weak, highly graphitic quartzite which has caused a large roof fall, Figure 2. The slabs of ore left in the roof must be stable enough to span the room and hold patches of weak strata in place. Whether this will be possible given the frequency of faulting and folding remains to be seen.

There is no equipment on site which would enable inspection and repair of excavation roofs if the full orebody thickness is extracted. Therefore inaccessible roofs must be "oversupported" and should include mesh or shotcrete to catch the smaller fragments. Where mesh is used it should be pulled tight to the back by installing bolts in hollows. In this way the mesh will add to the support effort.

Pillar stability is very good, as expected in the competent sulphides. But the maximum exposure is about 15 feet and very little extraction has taken place. Pillar heights up to 50 feet may occur but there is no indication of continuous structures with weak infilling that could intersect pillars at unfavourable angles. It is evident that room-and-pillar layouts will be dictated by room stability.

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There is an important difference between the conditioning of excavation roofs and pillar walls. A roof should be well scaled before support. Pillar walls should not. Instead, the loose material should be well supported. There is a risk of scaling too much material from a pillar wall and significantly reducing its effective size. Where pillar walls become inaccessible, due to orebody thickness, they must be supported.

2. Rock Support

The predominant support method has been split-sets. Straps have been added in the poorer areas. The overall impression is that the holes for the split-sets are too large and there is insufficient closure on the split. It is reported, though, that the operators were having difficulty in installing the split-sets because the holes were too small. There are a considerable number of split-sets hanging out of holes, Figure 2. Many of these must have shaken loose during blasting. Some attempt at pull testing of randomly selected bolts should be carried out to assess the longer term stability of the installations.

Recently there has been a conversion to mechanically anchored bolts. Discussions have revealed that there may be problems in securely anchoring these bolts in the weaker strata. Our view is that the split-sets, if properly installed, are superior to the mechanical bolts. Both types are inferior to cement grouted bolts. These should be considered in the more critical areas or in areas where a thick sulphide skin or sulphide rich waste is left in the back. When straps are installed it is most important that the straps should be pulled in tight to the rock surface by installing the bolts in hollows. The installation of the strap should proceed consecutively, one hole to the next. There are many instances of straps bridging large hollows in the rock surface.

The majority of the ground conditions are good but, to date, there have been few exposures of the hanging wall, the spans are relatively small and pillar stresses are very low. Much of the development is within the orebody and there is very competent sulphide material in the roof.

In the few areas in the south zone where the hanging wall has been exposed straps are required in most areas with some areas being extremely incompetent. In the future when the final excavation exposes the hanging wall and much larger spans are mined then the requirement for correct, well installed support will become critical. The present level of scaling and quality of support will not be adequate for the planned exposures. It may be necessary to leave a skin of suphides in the roof to reduce the support requirements.

Most of the major problems with roof control have been with foliated material and the creation of unstable slabs. Part of the problem is that the development roof is not mined to a single plane. Overbreak and poor alignment cause an uneven roof, Figure 2. There were a number of instances

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where the roof has become loose after support installation. It is not known whether this is because of poor scaling or incompetent material. If it is a function of incompetent material then more intensive support methods, reference SRK report, 60610, December 1988, must be used.

The alternative is to maintain a minimum thickness of ore in the roof and this would appear to be the most economic option considering the blasting and support practices which are in use.

3. Blasting

It is evident that there is no auto-parallelism on the drill jumbos, or it is not in use, and that there are many instances of incorrect hole alignment. This problem is especially critical in the corner holes of the roof. These are consistently overbreaking due to large look-out angles. The roof often has a convex provile due to poor spotting of holes and differences in hole angle. These points are highlighted in the accompanying sketch, Figure 3. It does not appear that the hole pattern is formally marked-up at the face. The angle on the holes varies throughout the round. The result of poor layout is an uneven roof and walls, extra confinement for some of the holes and overbreak. The resulting surfaces will, therefore, be less stable.

A review of the standard drill layout indicates two problem areas. These are identified in Figure 4 and concern incorrect timing and overcharging of holes close to the roof.

The layout from CMD, "typical stope mining" has been reviewed. The layout appears effective but far fewer holes could be used. Most of the holes should have a good free face if the blast is broken into stages. A powder factor of 1 kg/m^3 should be adequate which is an average burden and spacing of 4.5 ft.

4. Pillar Design Procedure

The empirical, pillar design formula which has been used for this investigation has been developed from work carried out by Hedley in Canada and Salamon in South Africa. Attached is a description of the relationship.

It is important to note that Hedley did most of his work on pillars that had the following shape:

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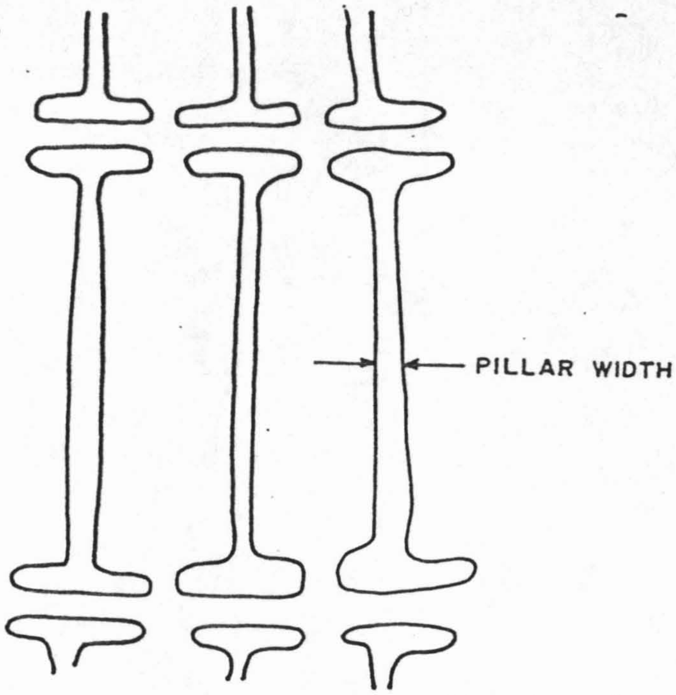

Plan view of room and pillar layout at Elliot Lake

The pillar design formula takes the following form:

$$\text{Pillar strength} = \text{rock strength} \times \frac{(\text{pillar width})^{0.5}}{(\text{pillar height})^{0.75}}$$

But in Hedley's relationship the pillar width only relates to the width of the high, thin rib pillars. The short, fat pillars around the access level are not included. Long rib pillars are more stable than small square pillars given the same extraction ratio. Therefore some adjustments must be made if a square pillar layout is envisaged. This can be taken into account by using an "effective" pillar width, (described in the attachment):

$$W_{eff} = \frac{4Ap}{R}, \quad \text{where } Ap = \text{pillar plan area}, \quad R = \text{pillar perimeter}$$

If the "effective" pillar width is taken into account then the rock mass strength of 173 MPa, in Hedley's formula, becomes 122 MPa.

The average rock mass strength that has been used for this present study is 54 MPa, which is similar to the difference in intact strength between the conglomerates for Elliot Lake and the sulphides at Faro; 270 MPa and 122 MPa respectively. The quality of the ore in both camps is similar.

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A design relationship is only as good as the data and experience that are used in the development of that relationship. The data base for Faro is small and the experience, at present, is zero. But experience will only be available when large areas have been extracted to the final percentage extraction. There is not time to accumulate this experience.

In the interim it is suggested that the following design relationship should be used by site personnel:

- Safety factor

$$\text{Safety factor} = \frac{\dfrac{8250\, W_{eff}^{0.5}}{H^{0.7}}}{\dfrac{1.1\, D \cos^2\alpha + (1.1\, D \times 2) \sin^2\alpha}{1-e}}$$

Where W_{eff} = effective pillar width = $\frac{4A_p}{R}$ (ft)

H	=	pillar height, ft.
D	=	depth below surface, ft.
α	=	dip angle
e	=	percentage extraction/100

The above equation assumes "tributary area theory", pillars carrying total overburden load. In practice, with a relatively small mining strike, the abutments will carry appreciable road and numerical analysis of the proposed mining system is recommended.

A factor of safety of 1.5 is suggested, to comply with recommendations from Hedley.

5. Initial Room-and-Pillar Layouts and Procedures

5.1 Rock mass classification

The exposed rock in the southern arm of the orebody was classified using Laubscher's Mining Rock Mass Rating system. This has been used at the site before, both before underground mining commenced, using drillcore, and on limited exposures during an SRK site visit in April 1990. The results are as follows:

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Average RMR

Rock Type	1988 core logging	Mapping April 1990	Mapping June 1990
White mica schist	31	46	38
Graphitic quartzite	39	52	49
Massive sulphide ore	47	56	66

The variability of the rock mass must be taken into consideration when comparing the above average values. The RMR ranges for June 1990 mapping are:

RMR

Rock Type	Mean	Maximum	Minimum
White mica schist	38	56	28
Graphitic quartzite	49	63	25
Massive sulphide ore	66	76	57

Given this range of values, the only material which appears of substantially higher quality than predicted from previous mapping and drill core classification is the massive-sulphide ore.

5.2 Pillar Design

A pillar design was done specifically for the south B area. This design was conducted using the following assumptions:

- Average pillar is 27 feet long by 35 feet tall
- Average drift width is 17 feet
- Maximum depth of cover is 590 feet

5.3 Rock mass strength

In order to calculate pillar strengths, the in-situ strength of the rock composing the pillar must be arrived at in some manner. In this design the pillars were assumed to be composed of (on average)

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90% massive sulphides and 10% hanging/foot wall waste. The average design strength is therefore (see appendix):

Design Rock Mass Strength (MPa)

Mean	Standard Deviation	95% Maximum	95% Minimum
54	7.5	69	39

5.4 In-Situ Stress

Pillar loads were calculated using two scenarios. The first, and the one which the design must be based on without numerical studies, is tributary area loading at maximum depth of cover. This, when adjusted for a 20° deposit dip and a 2:1 horizontal/vertical stress field, gives an in-situ load of 780 psi.

The second loading case, which was assumed to be feasible, was that which would occur if a stress arch formed over the mining area. When a stress arch forms, not all of the tributary area load is seen by the pillars. This occurs as most of the true load is transferred to the non-deforming intact rock mass acting as an abutment around the mining area. The width of, and the amount of load developed under, a stress arch is relatively unknown but can be approximated by numerical studies. For this case, a load of 390 psi was chosen as the under arch pillar loading stress. Details of this calculation may be found in the appendix. It must be remembered that this value is very approximate and must be confirmed by numerical methods.

5.5 Pillar layout dimensions

The following values were obtained from the pillar design equations in section 4 using the assumptions mentioned previously in this section. A safety factor of 1.5 was used for pillar design. Calculation details may be found in the appendix.

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Pillar Loading	Pillar Width (ft)	Room Span (ft)	Extraction Ratio (%)
Tributary Area	15	10	63
	20	16	66
	25	24	69
	30	30	70
	35	39	71
Stress Arch	15	35	81
	20	52	83

5.6 Maximum Roof Span

As pillars become proportionally stronger per unit increase in width, the maximum extraction would be achieved with one large pillar supporting the entire mine. As this is not possible due to roof failure some compromise must be found which minimizes pillar size and maximizes roof span and extraction ratio. This point is expected to be, in this case, the roof span at which intense support is required.

A support classification may be conducted using the hydraulic radius of the open room (Laubscher/Page). Given that the hanging wall materials have an average RMR of 46 (unadjusted) and an adjusted RMR of 32 (foliation dip and weakness) the following table was constructed:

Modified MRMR System

Room size (ft)	Support
13 x 44 (drift)	bolts at 1 m + strap
24 x 44	bolts at 1 m + strap
39 x 44	bolts at 1 m + mesh reinforced concrete
52 x 44	bolts at 1 m + mesh reinforced concrete + cable bolts as needed

Bolt lengths vary from 6 foot length at 13 foot span to 8 foot with 39 foot span

These indicate that the maximum average stable span width given an exposed hanging wall and reasonable support is likely between 25 - 35 feet. If a certain amount of failure can be allowed, such as with remote operated equipment, span widths may possibly be increased.

5.7 Recommendations

The recommended pillar design for test purposes in the South B area is:

- pillar width along strike = 30 feet
- pillar length down dip = 27 feet
- pillar height = 35 feet
- entry width = 17 feet
- roof span = 30 feet.

These dimensions must be modified to fit the constraints forced on the layout engineer by present entry location. An attempt has been made to balance the constraints, both manmade and geological, with the pillar layout shown in Figure 5.

It is advised that the pillars be bolted and the working back be reinforced with bolts (1m centres) with mesh/straps between bolts before benching is conducted. Longer bolts, preferably grouted rebar, may be required in areas of thick remnant sulphides. There is evidence from examination of excavated areas within the mine that the damage/overbreak zone is approximately equal to 2.5 feet at some point along the sidewall/back. This damage reduces the true pillar thickness with a proportionate reduction in pillar strength. Thus, given the present drilling inaccuracies, the actual drill line for the panel (trim ring) should be about 2.5 feet in the room side of the pillar.

A layout has also been prepared for the stress arch pillar loading scenario. In this case, pillars are 15 feet wide with a 35 foot roof span, adjusted to fit local conditions. This plan was drawn up only to indicate the potential change in extraction ratio for the two loading cases and is shown in Figure 6.

In both cases, an intact rib pillar was left between the South C ramp and the South B 100 entry. This was done to simplify ventilation and to protect the South C ramp in case of over extraction in the B panel. The same pillars where used along the South B ramp for both loading cases. These are conservative (over dimensioned) for the stress arch loading concept. The intent in this case was to protect the B ramp from roof failures originating in the B panel as well as to provide support in case of over extraction.

Staggering pillars during the layout would tend to reduce span related failure problems. This has been done, to a limited extent, in both proposed layouts. However, excavation constraints have forced pillar alignments in some areas. Future designs should attempt to incorporate staggered pillars.

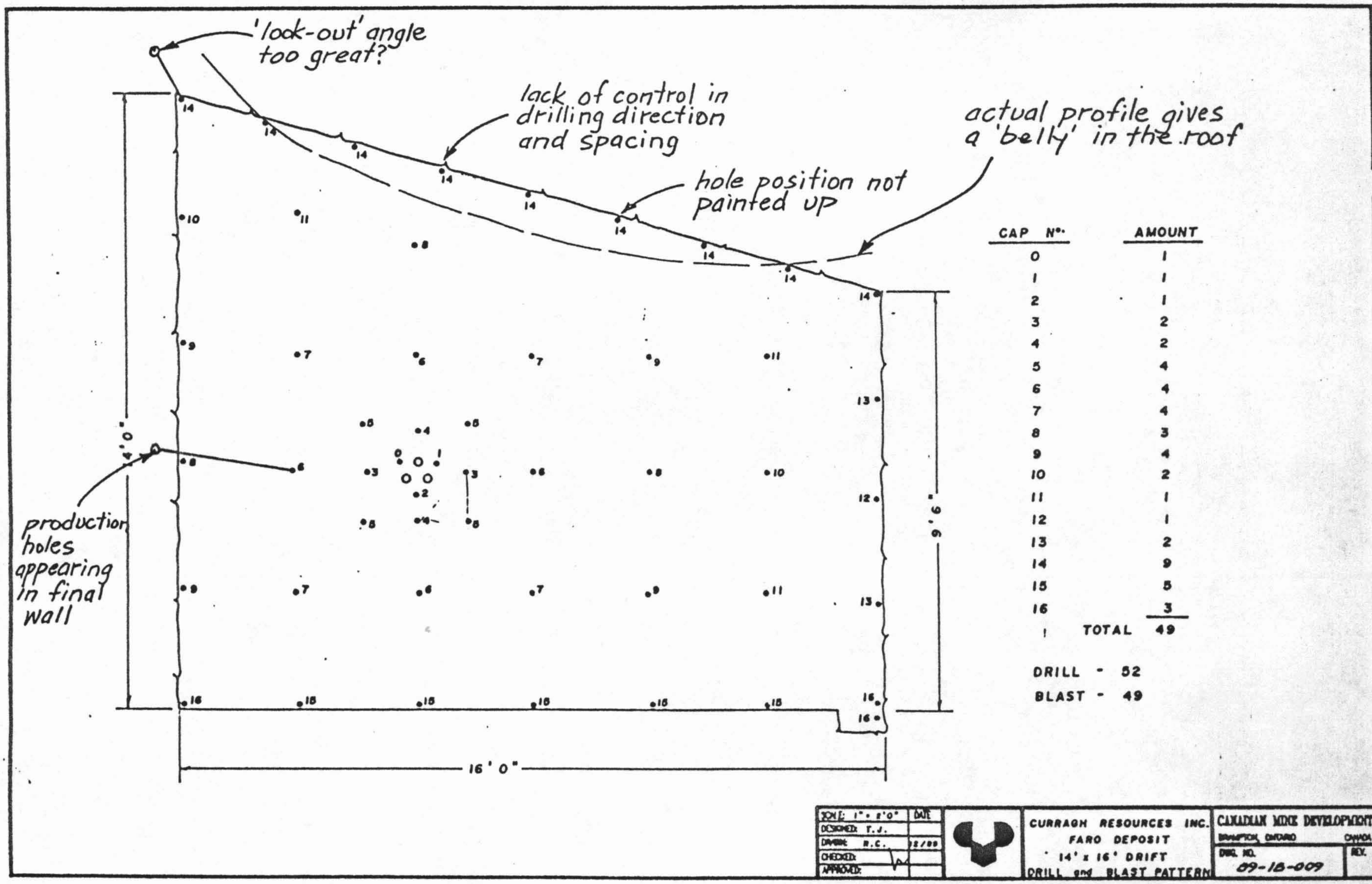
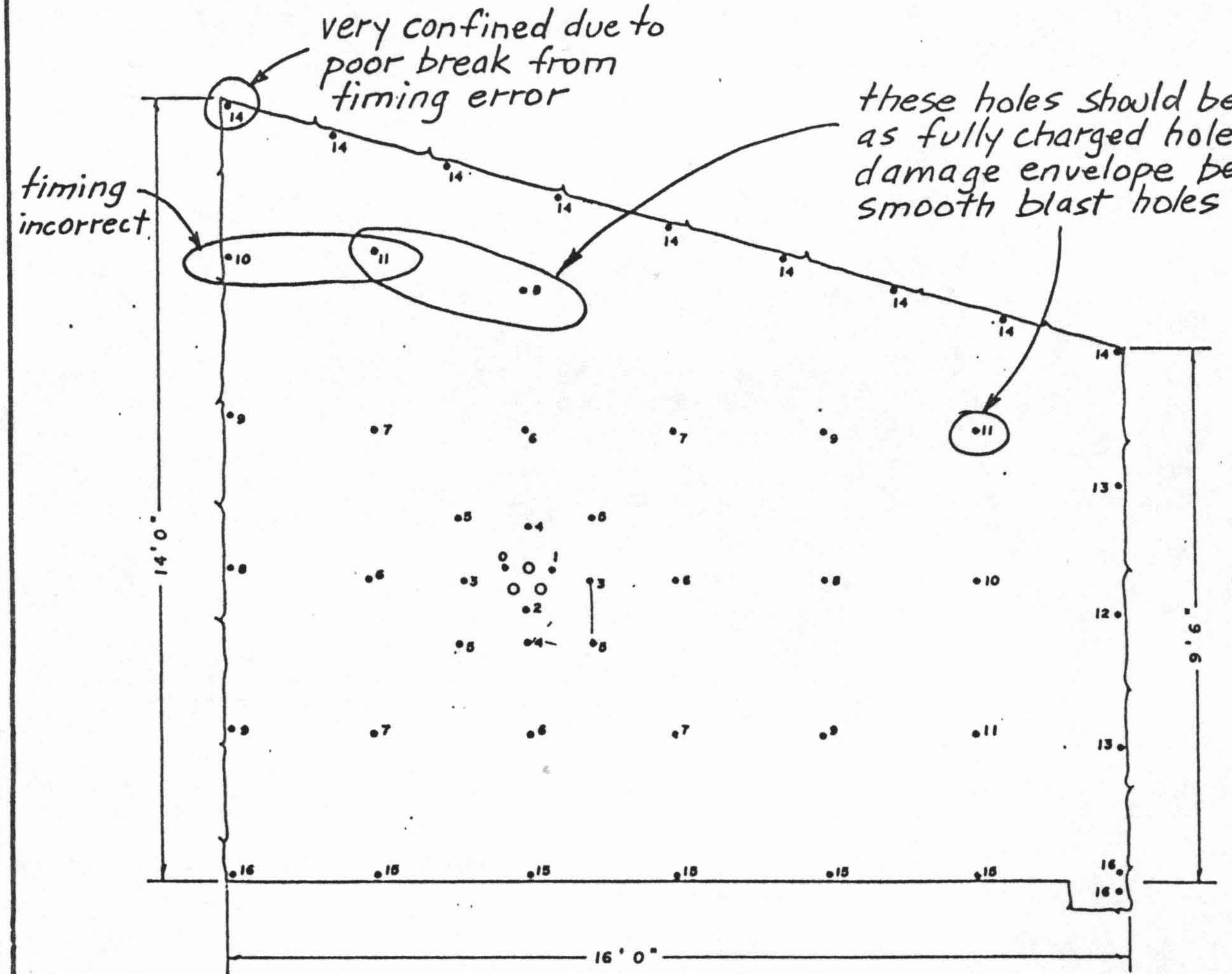

'look-out' angle too great?
lack of control in drilling direction and spacing
actual profile gives a 'belly' in the roof
hole position not painted up
production holes appearing in final wall
4' 0"
9' 6"
16' 0"
CAP N°. | AMOUNT
0 | 1
1 | 1
2 | 1
3 | 2
4 | 2
5 | 4
6 | 4
7 | 4
8 | 3
9 | 4
10 | 2
11 | 1
12 | 1
13 | 2
14 | 9
15 | 5
16 | 3
TOTAL 49
DRILL - 52
BLAST - 49
SCALE: 1" = 2'0"
DESIGNED: T.J.
DRAWN: R.C.
12/89
CHECKED:
APPROVED:
CURRAGH RESOURCES INC.
FARO DEPOSIT
14' x 16' DRIFT
DRILL and BLAST PATTERN
CANADIAN MINE DEVELOPMENT
BRAMPTON, ONTARIO
CANADA
DWG. NO.
89-18-009
REV.

FIGURE 3



CAP No.	AMOUNT
0	1
1	1
2	1
3	2
4	2
5	4
6	4
7	4
8	3
9	4
10	2
11	1
12	1
13	2
14	9
15	5
16	3
TOTAL	49

DRILL - 52
BLAST - 49

SCALE: 1" = 2'0"	DATE	
DESIGNED: T.J.		
DRAWN: R.C.	12/89	
CHECKED:		
APPROVED:		

CURRAGH RESOURCES INC.
FARO DEPOSIT
14' x 16' DRIFT
DRILL and BLAST PATTERN

CANADIAN MINE DEVELOPMENT
BRAMPTON, ONTARIO CANADA
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FIGURE 4