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Mitchell, R. J.; Olsen, R.; Smith, J. D.

[Rhinelander, Wisconsin]: Exxon Minerals Company, 1982

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EVALUATION OF SURFACE EFFECTS
CRANDON PROJECT, EXXON MINERALS CO., U.S.A.

by

R.J. Mitchell, R. Olsen, J.D. Smith

September 14, 1979
Rev. APRIL 28, 1982

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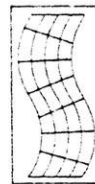
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APRIL 28, 1982

Mr. J. E. Grimes,
Chief Mining Engineer,
Exxon Minerals Company U.S.A.,
P.O. Box 813,
Rhineland, Wisconsin,
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54501.

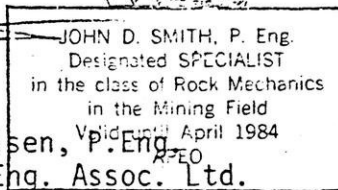
Dear Jim:

re: I.D. No. 21073 : Backfill Testing Program

With respect to section 4.9 of the above contract, herewith is submitted the revised and updated surface effects report. The report uses all technical information available to date, except for groundwater studies which are still in progress.

Respectfully submitted,


Richard S. Olsen, P. Eng.
J. D. Smith Eng. Assoc. Ltd.



RSO/bj

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EVALUATION OF SURFACE EFFECTS AT THE CRANDON PROJECT

Kingston, Ontario, Canada

APRIL 28, 1982

At a meeting in Kingston on October 26, 1978 with Ed May, Roger Rowe, Jim Grimes, Rusty Ford and Dave Mann, the subject of possible surface effect due to the mining operations was discussed. At that time, a request was made to produce an estimate of the potential problem areas, the data needed to predict and/or control the effects, and the cost of obtaining the data and doing the necessary engineering studies to predict both short and long term surface effects. This proposal was presented in a letter dated November 2, 1978. Subsequent to this date, technical data was received in Kingston and authorization to proceed was received in a letter from R. Ford dated July 23, 1979.

In addition to the rock and ore physical property data obtained from laboratory and point load testing done by this company and Exxon, and the extensive backfill testing done by this company, the input data provided by Exxon included the following reports and drawings:

- (1) Investigation of Feasibility of Dewatering and Other Alternatives For Open Pit Mine Option Near Crandon, Wisconsin. Dames & Moore - May 20, 1977.
- (2) Results of Geologic Geotechnical and Hydrological Investigations of a Portion of the Proposed Exploration Ramp. Dames & Moore - August 10, 1977.
- (3) Engineering Siting Report for Tailings Disposal Sites - Phase I.



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Dames & Moore - undated.

- (4) Results of Permeability Tests and Analyses of Water Samples from Deep Exploration Holes - Crandon Orebody. Dames & Moore - April 19, 1978.
- (5) Summary of Phase II Sub-Surface Investigations in the Crandon Project Area, Forest and Langlade Counties, Wisconsin. Dames & Moore - September 22, 1978.
- (6) Quaternary Glacial Isopach - December 12, 1978.
- (7) Subcrop Contour Map - December 12, 1978.
- (8) Longitudinal Sections showing mining and backfilling sequence by years - 1987-1991.

Further rock mechanics testing and backfill testing performed by this company included:

- (1) "Rock Mechanics Testing and Engineering of Large Diameter Core" - September, 1981 (hereafter referred to as Report A), and
- (2) "Testing of Conventional, Pyrite Concentrate and Pyrite Slimes Backfill Materials" - September, 1981 (hereafter referred to as Report B), both representing

major studies on rock and backfill properties. Data generated from these reports include inherent stress levels, rock strengths, pillar strengths, and all pertinent backfill properties. From Report B, the properties of 7 backfill materials were analyzed. Conventional backfill materials included samples 4, 5 and 6. The anticipated recovery of each (reference personal communication November 3, 1981, J.D. Smith, J.E. Grimes) is



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4,074, 2,919 and 1,756 metric tons per day, respectively. Because sample 4 constitutes nearly one half of the backfill material available, its properties were used in the assessment of surface effect. Subsequent analyses may be performed using the techniques presented in this report when exact backfill composition is verified.

For the purposes of this report, "surface effect" can be one or all of the following: changes to the groundwater table and rates of flow, as a result of the mining operation; elevation changes in the overburden as a result of dewatering these materials; and elevation changes in the overburden as a result of gross rock movement. This study assesses these possible types of surface effect with respect to postulated mechanisms and provides an order of magnitude measure of the expected movements and flows. Wherever possible, means to prevent adverse reactions are suggested. In addition, the important parameters are identified so that additional data can be obtained during development and early production periods, in order to confirm or modify the predictions in this report.

Surface effect is discussed for three phases of the mining operation. The primary mining stage is defined as when underground development and exploration openings are created and production mining begins. Gross rock deformation at this stage is prevented by rib, sill and crown pillars and backfill and only changes to groundwater table and flow are considered possible. The secondary mining phase is where the rib and sill pillars are removed and replaced with backfill, leaving the crown pillar and backfill to control surface effects. At this stage, any groundwater problems have been solved and are not expected to reappear.



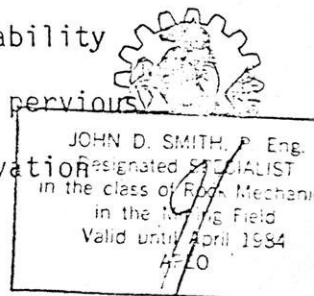
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The deformation of the H.W. and F.W. formations and crown pillar are controlled by the use of backfill and the proper mining sequence. Negligible surface effect can be achieved by proper geometry for the crown pillar and the design of this pillar is discussed in this section. The final mining stage is the recovery of the crown pillar. This is the stage that has the greatest potential for surface effect. This section of the report discusses the mining method and geometry developed to minimize such effects. At this stage of mining, mining experience and the accumulation of engineering data will suggest any necessary modifications. Long term effects after the crown pillar has been removed are also discussed in this section.

PRIMARY MINING STAGE

Alternate stope and pillar combinations will be removed during the primary mining stage, leaving the intermediate mining blocks (stopes and pillars) and backfill for ground control. Providing that caving does not develop in the crown pillar above the primary mining blocks, there will be no noticeable surface effects during the primary mining stage due to mine rock distortion, (i.e. only very small elastic deflections will occur in the rock). The design of a stable crown pillar is discussed in Section 3 of this report.

During primary mining, surface effects will then be restricted to effects due to drawdown of groundwater resulting from the sink created by underground openings at atmospheric pressure. The quantities of groundwater flow into the mine will depend largely on the permeability of the crown pillar which forms the impervious barrier between the pervious water-bearing overburden soils and the mine openings. Surface elevation



changes can occur as a result of dewatering of the overburden soils.

These potential surface effects are discussed below.

2.1 Drawdown of Groundwater

This subject is currently being studied in considerable detail by the company and their consultants. The results of the field work and subsequent analysis will be available at a later date and can be used in a more rigorous assessment of soil deformation. Data available from earlier studies suggests that any water drawdown will produce only very small surface effects.

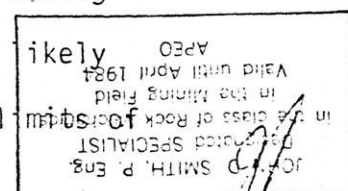
2.2 Subsidence Due to Overburden Consolidation

Excepting the relatively thin deposits of recent origin (lacustrine and marsh), the overburden soils are not considered to be compressible - they are coarse-grained and have been preconsolidated (densified) by glacial action. If the upper aquifer is preserved, there will be no subsidence due to compression in the overburden. In the event that the upper aquifer is drained, the soil compression would still be quite small.

From the soil identification and the data on Table 1 (underlined), it is estimated that the total surface subsidence could be less than 0.2 m. In view of this estimate and the intent to preserve the aquifer level, we would not consider further study of this problem necessary. If desired, however, consolidation testing of undisturbed samples of the overburden materials would quantify the drawdown-subsidence relation more accurately.

2.3 Summary

With an adequate and stable crown pillar, surface effects during primary mining will be negligible or non-existent. Grouting will be required in some locations, particularly towards the easterly limits of



the orebody, to effectively isolate the groundwater from the mine openings as mining progresses in the upper levels of the orebody. The extent of this grouting and the quantity of groundwater flow can only be accurately known during the early development. If crown pillar stability can be maintained during secondary mining, surface effects will, again, be minimal during these operations. The design of the crown pillar and crown pillar recovery are discussed in subsequent sections of this report.

SECONDARY MINING STAGE

In general, if gross rock distortions and failures develop, they do so at this stage of mining. They can be avoided by proper mining sequence and geometry followed by backfill. During the primary mining stage, the crown pillar is supported by rib pillars and backfill, but as secondary mining proceeds, these pillars are mined leaving the crown pillar as the only solid ore. Tight backfilling during stope and pillar mining supports the bottom of the crown pillar and thus prevents any surface effect - changes in elevation as well as in groundwater flows. It is only when pillars are removed and before fill can be placed that is considered to be the critical time. This section of the report discusses the design of the crown pillar in order to achieve these results. If the crown pillar is structurally stable, there can be no surface effect due to stope and rib pillar mining. In all cases, a conservative approach to pillar design height has been taken, with priority given to the factor of safety.

3.1 Crown Pillar Design

The geological assessment made by the Exxon geologists from examination of drill core is that the top 50 to 100 feet of bedrock is generally moderate

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to strongly supergene altered. The rock is strongly fractured with numerous shear and gouge zones. Most of these effects are thought to be strictly a product of surficial weathering and rapidly die out with depth. Recent analysis of weathering intensity was performed by Exxon geologists using data from all available drill holes. The degree of weathering was determined by such factors as oxidation, percentage of leached rock, development of clay and degree of fracturing. Four zones of different weathering intensity were defined, including strong to weakly weathered material. Point load data measured perpendicular to the rock foliation provides an estimate of the rock strength in the H.W. to F.W. direction and therefore can represent the strength of the crown pillar in the expected loading direction. Point load tests were done on a series of holes that intersected the orebody generally above the 140 m. elevation, although the plan of the strength distribution of the 140 m. level shows these data projected down to it. It was decided to correlate the point load strengths with the four degrees of weathering intensity determined by Exxon geologists. No correlation was found; the major reason being that equal degrees of weathering in rocks of different strengths produce reduced, yet dissimilar, strengths. For example, the ultimate strength of a moderately weathered hanging wall chloritic rock may exhibit a strength below 3,000 psi. Because of this difference, it is not possible to relate crown pillar strength to qualitative degrees of weathering. The basic strength criteria is the point load strengths perpendicular to the rock foliations.

From point load strength data, the weakest zone was classified as anything that tested between 0 - 5,000 psi.* On the 140 m. level plot, the average strength for this zone was 2945 psi in the H.W. - F.W. direction. The method of estimating the transfer of inherent stress to the crown pillar must be empirical since the rock quality (hence stiffness)

*

See Appendix "B" - "Creation of Rock-Strength Zones by point Load Strengths"



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varies considerably with depth and degree of weathering. It is possible that the upper rock strata is badly fractured because, due to intense weathering of the weaker rocks, the inherent stresses have already exceeded the rock strength and some crushing has already taken place. For purposes of design, it can be considered that load transfer cannot take place within this material and there will be no "concentration factors" as ore is removed. To be conservative, it will be assumed that the strength of all material that lies in Zone 1 of the strength classification is zero, and the minimum value of material lying in Zone 2 will be used, i.e. - 5,000 psi. Subsequent testing during early development will verify the operative rock strength.

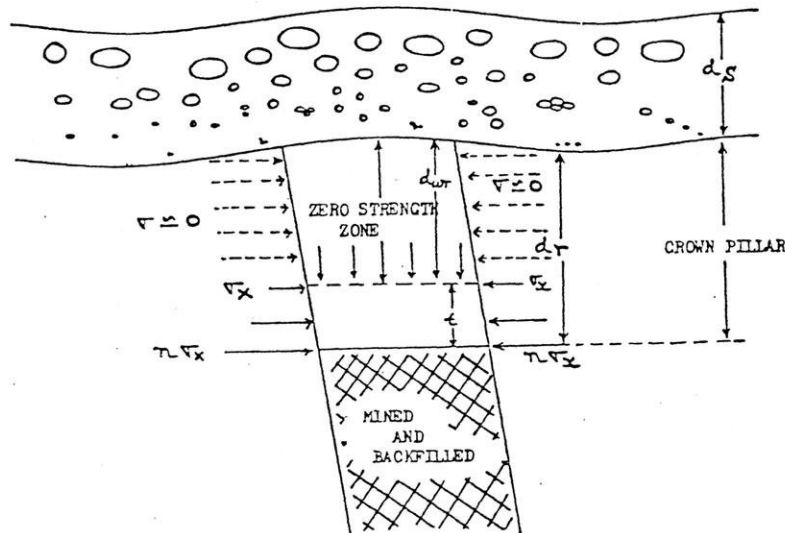
The crown pillar cannot fail locally during stope mining or pillar recovery or surface effect will result. This means that the roof beam (the bottom of the crown pillar) in the H.W. to F.W. direction must be stable and this provides a basis for design. (Details of the theory of this analysis are included in Appendix "B"). If one considers that the zero strength rock zone and the overburden soils provide the uniformly applied load on the beam and the horizontal inherent stress provides the prestress, the height of the rock beam in more competent ground can be calculated.

** See Appendix "B" - "Theory of Stope Span Design".



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The following diagram illustrates the simply supported, uniformly loaded beam column.*



* Refer to Appendix "A", Figure 2, for conceptual aid.

The limit of stability for this beam is when the stress at the extreme fibre at the centreline is zero and the rock is about to go into tension at this location. This is expressed by $n\sigma_x - \sigma_m = 0$. The inherent force field (σ_x) cannot be increased beyond the strength of the material by mining concentration factors. It was determined (refer to "Rock Mechanics Testing and Engineering of Large Diameter Core" September 1, 1981) that the average inherent stress perpendicular to the foliations is 2122 psi with a standard deviation of 718 psi. Taking the statistical minimum of $2122 - 718 = 1404$ psi, which is most conservative for design purposes, and assuming a concentration factor of 3,

$$n\sigma_x = (3) (1404) = 4212 \text{ psi.} = \sigma_m$$

But σ_m , at the extreme fibre of a simply supported beam centreline is equal to

$$\frac{Wl^2}{8} \times \frac{6}{t^2}$$

where W is the load per foot of beam, ℓ is the span of the beam, and t is the height or thickness of the supporting beam. The weight per foot of beam is $W = (d_s + d_r)\gamma_{avg}$. where d_s is the depth of soil cover, d_r is the depth of zero strength rock plus the depth of the beam, and γ_{avg} is the average bulk unit weight of the materials.

$$\frac{W\ell^2}{8} \times \frac{6}{t^2} = 4212 \text{ psi.}$$

$$\frac{\ell^2}{t^2} = \frac{4212 \times 144 \times 8}{W \times 6}$$

$$\ell = \sqrt{\frac{4212 \times 144 \times 4}{(d_s + d_r)\gamma_{avg} \times 3}} \times t$$

where ℓ and t are in ft and W is in lbs/ft of beam. This reduces to

$$\ell = \sqrt{\frac{4044}{(d_s + d_r)}} \times t$$

where $\gamma_{avg} = 200 \text{ lbs/cu.ft.}$

Between 94,100E and 94,700E, (the eastern portion of the crown pillar), the rocks and ore in the pillar run between 2 and 6 in strength, as estimated for the 140 m. level from point load strength data. This is a strength change of from 5 - 10,000 psi to over 40,000 psi in a lateral direction perpendicular to the contacts. If $n\sigma_x = 4212 \text{ psi}$ is used, it can be seen that a considerable factor of safety will result. At this location, the bottom of the zero strength zone is above the

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140 m level. As an example, if the $d_r = 150$ feet, the measured values of $d_s = 175$ feet, and the variation in crown pillar span ℓ of 50 to 80 metres can be used to calculate t .

$$\ell = \sqrt{\frac{4044}{325}} t = 3.53 t$$

For $\ell = 50$ m, $t = 47$ ft. (14.2 m)

and for $\ell = 80$ m, $t = 75$ ft. (22.7 m)

To summarize, t is the depth of competent rock below the zero strength zone required to support the overlying materials.

In the region of 93920E to 94100E, the material on the 140 m. level ranges from a strength classification of 1 to 5. Considering our design assumption that zone 1 has zero strength, the bottom of the crown pillar must be below the 140 m. level. For example, if $d_r = 350$ feet, $d_s = 170$ feet, the relationship becomes,

$$\ell = \sqrt{\frac{4044}{520}} t = 2.79 t$$

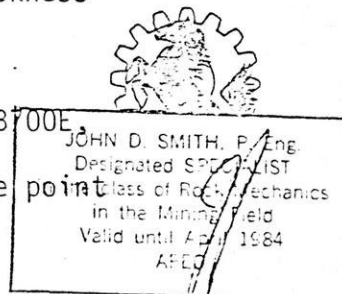
For a span of about 40 metres, t should be 47 feet or 14.3 m.

The next region to the west, located between 93780E to 93920E, the rocks and ore are estimated on the 140 m. level to range between zone 2 and 6 in strength. If $d_r = 200$ feet, and $d_s = 190$ feet,

$$\ell = \sqrt{\frac{4044}{390}} \times t = 3.22 t.$$

For a span of 50 m, the required intact crown pillar thickness would be 51 feet or 15.5 m. and as the span increases to 70 m, the thickness would increase to 71 feet or 21.7 m.

From the western extremity of the orebody to section 93700E, the material on the 140 m. level is predominantly of zone 1. The point



load data on the 350 m. level indicates that the rock and ore strength on this level have improved to classification 4 or better. If it is estimated that the bottom of the zero strength zone is near the 230 m. level, d_r would be about 560 feet, $d = 190$ feet, and

$$L = \sqrt{\frac{4044}{750}} \quad t = 2.32 \text{ t.}$$

For a span of 40 m., t must be equal to or greater than 57 feet (17.2 m) and for a span of 70 m., t would need to be 99 feet or 30.2 m.

The dimensions of the crown pillar, by this analysis, are dependent on the depth of overburden and rock, the depth of the zero strength zone, the H.W. to F.W. distance at a given location, and the magnitude of the lateral stress imposed by mining. The definition of the zero strength zone can be more accurately assessed with supplementary drilling and strength testing. The design basis presented in this section can be used again when further information is available to produce what is considered to be a very conservative vertical height of crown pillar. Such a conservative design virtually assures, along with the use of tight backfilling, that surface effect cannot take place during secondary mining. It should be kept in mind that much mining experience will have been gained before the more critical stages of secondary mining are begun and modifications to geometry and procedures can be implemented as the experience and engineering data become available.

CROWN PILLAR RECOVERY

The mining method and geometry outlined in this section should



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be treated as conceptual only. There has been no attempt to estimate mining costs or economics of this method but it is felt that it, or a similar method, can effectively control surface effect while maximizing ore recovery.

4.1 Mining Method and Geometry

The bottom of the crown pillar will be located in reasonably competent ground but the quality of the ore and abutment rocks will deteriorate with increasing elevation as the surficial weathering and associated decrease in rock strength intensifies. This, along with the need to control surface effect, suggests a mining method that does not allow men and equipment to work beneath a stope back and at the same time provides continuous support to the overburden soils.

Such a method is the sub-level V.C.R. cut and fill. This is a transverse mining method that has all development openings within the crown pillar itself, except for ore-pass cross-cuts. A slice, about 15 feet long in the strike direction, full width in the H.W. - F.W. direction, and possibly 130 feet high, would be mined by V.C.R. methods at each extremity of the orebody. Succeeding slices would be taken in order, minimizing stress concentrations and maintenance on permanent openings. The general arrangement is shown in Figure 3.1 and 3.2 in the Appendix. As each slice is blasted in 10 - 15 foot lifts by V.C.R. methods, enough ore is mucked from the lower level to take care of the swell. In this way, there is little or no void created and there is more or less continuous support on all sides by the broken ore. The V.C.R. method was chosen due to its excellent fragmentation, decreased vibration levels, and potential for no void creation. When blasting reaches the next

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about 30 - 35% of the ore in the slice will have been removed from below and as the broken ore is drawn down, waste rock is added from the upper level. The Radmark Stower could be used, in conjunction with scoop trams, to keep the back supported with waste rock. Beyond the solid face some distance, tailings fill can be introduced from the upper level through blast-holes in order to "top-up" the waste rock and provide the maximum support for the back. It should be noted the temporary and final backs are blasted in a series of "V" patterns, with the drifts at the apex of the "V". This ensures that the fill can be placed tight to the back and when completed, access is still possible in the drifts, for mucking purposes.

Model tests or simulation techniques should be conducted in order to establish the best spacing for drawpoint drifts, the angle of the blasting face, and the maximum vertical distance between drawpoint levels with the purpose of maximizing ore recovery and minimizing dilution. It is possible that additional drawpoint elevations would be required. At Avoca in Ireland, conventional rings using small diameter holes were drilled to produce a 75° inclined face. When a ring was blasted, the blast energy compressed the fill, making a stronger, interlocked backfill that didn't dilute the ore and at the same time creating enough void so that the desired fragmentation was achieved.

It is felt that the top drill and backfilling drifts can be placed about 30 feet below the overburden contact, thus allowing about 20 feet of rock cover. These will be in highly altered, weak rocks and ground support will be needed. These drifts could be required before production mining begins for grouting purposes. If so, their



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extended life suggests that grouted rock anchors, screening, and/or shortcreting might not provide long term support. The use of Armco Mineway in the drifts plus tight backfilling to fill the void between the mineway and the rock should provide maintenance free access at this elevation. Since this top elevation will only be a drilling, blasting and backfilling level for the crown pillar mining, the Armco sets never need to be removed. If ground conditions warrant, their use on mucking levels might be required, as well, in which case, they must be designed for easy removal, as each succeeding slice is taken.

There are several variations that can be used with the basic mining method. Ground conditions on the lower level should be better due to increased rock strengths and it is possible that the stand-up time for the wall rocks will be long enough for a slice to be completely mined out before backfill is needed. In this case, the previously placed waste rock must be stabilized with cemented tailings. This would delay the cycle time but less dilution from waste rock and stronger backfill would result. The Radmark Stower need not be placed underground. The blower, power pack and stower unit can be placed on surface where continuously feeding the unit is more easily arranged. The pipe and nozzle are all that is required underground, so that stowing and conventional backfilling can be carried on simultaneously.

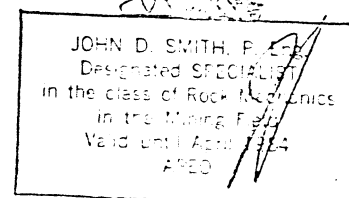
The mining sequence separates the different operating functions on the different levels. Only mucking is done on the bottom level of a block. Loading and blasting is done on the next level up until break-through occurs and then only backfilling is done until the broken ore has



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all been removed below. The third level up would be used only for drilling, and if more intermediate levels are required to reach the grouting elevation, development would be underway on the 4th level. This results in more efficient mining and a regular flow of broken ore to the mill. Since mining is done on retreat towards the shaft, two mining faces are in production at any time. Depending on the width of the orebody at a given location, anywhere from 6 to 14 drawpoints would be available for mucking. It is likely that the production rate of 14000 tons/day from underground would need revision downward during crown pillar mining. To illustrate, if the orebody is 165 feet in the H.W. to F.W. distance, a 15 feet thick slice, one foot high, produces 280 tons/vertical foot of hole in massive sulphide. This would require loading and blasting the equivalent of 25 feet of vertical height from each face per day. If the tonnage factor for the stringer sulphide is used (10.67 cu.ft./ton), the vertical height for each face would rise to 30 ft./day. To do this amount of work each day might not be possible.

By controlling the void space with this mining method, and leaving a permanent remnant crown pillar to prevent even minor vertical movement of the overburden, surface effect should be minimal. However, essentially the complete orebody has been removed and replaced with tailings and/or waste rock backfill. These materials are porous and are therefore much more readily compressed into a smaller volume. This characteristic presents the possibility of two potential long term mechanisms that can alter the ground surface and sub-surface ground-water flow. These are discussed in the next section.



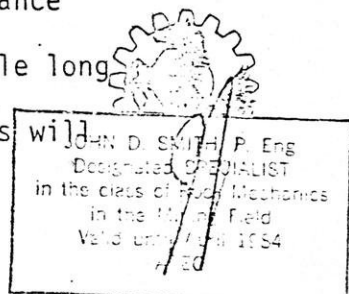
4.2 Hanging Wall Stability

Two possible cases of long term hanging wall slumping (after the removal of the crown pillar and backfilling) are envisaged. The first would involve a rotational slump restricted to shallow depths in the badly fractured upper rock strata identified towards the easterly end of the mine. The second would involve a wedge slump in the event that non-conformable planes of weakness daylight in the hanging wall. These cases are analyzed in the following sections.

4.2.1. Rotational Failure in the Hanging Wall

Figure 4 shows the geometry of a potential long term failure in the weak upper strata of the hanging wall. The driving moments causing the rock mass to rotate about the point C are given as $M_D = (W_r + W_o)D/2$ where D is the failure depth, $W_r = 0.175D^2 \gamma_r$ is the weight of the rotating rock sector and $W_o = Dd_o \gamma_o$ is the weight of overburden soil. The resisting moments are provided by shearing resistance in the rock ($F_r = \Sigma \tau r \Delta L$) and pressure developed in the backfill ($F_p = \Sigma \sigma p \Delta D$). The resisting moments about the centre of rotation, C, are then, $M_R = F_r D + F_p \frac{2D}{3}$. The forces, W_o , W_r , F_r and F_p are shown as free body forces on Figure 4 and the appropriate values for F_r and F_p are developed below:

As noted on Figure 4, the rotational mechanism of failure is limited to badly fractured rock and could not extend below about 70 metres in the hanging wall. The minimum strength that these badly fractured rocks could be assigned is a purely frictional resistance given by $\phi = 45^\circ$. To produce an estimate of the maximum possible long term subsidence due to rotational failure, a depth of 100 metres will



be considered and $\phi = 45^\circ$ ($\tan\phi = \text{unity}$) will be assumed. The value of F_r is obtained using the well known "method of slices", as outlined in detail on Figure 5. Since the length of the slip circle arc is $0.7D$ (see Figure 4) the value of F_r gives an "equivalent average shear strength" for the rock of

$$\tau_{\text{avg.}} = \frac{0.7D^2 \gamma_r}{0.7D} = D\gamma_r = 3500 \text{ kPa}$$

This value (3500 kPa) is approximately equal to 500 psi and is a very conservative value for the mass shearing strength of this upper rock strata.

The backfill pressure has two components. The pressure existing in an at-rest position (no rock rotation) is estimated as $\sigma_{p_0} = K_0 \sigma_v = K Z \gamma_f$, where K_0 is the coefficient of earth pressure at rest, Z is the depth in the fill and γ_f is the bulk unit weight of the backfill. This component of pressure has a triangular distribution, from zero at point C to $\sigma_{p_0} = K_0 D \gamma_f$ at the depth of the failure arc. The second component is developed if the rock rotates into the backfill. Again, the rotational displacement into the backfill will be zero at point C and a maximum at the depth of the failure arc. If the backfill is considered to have an elastic response, the resistance to rotation will result in a similar triangular pressure distribution which can be described by pressure at the depth of the failure arc in the form $\sigma_{pp} = K\delta$, where δ is the rotational displacement of the rock (hence the maximum surface subsidence) and K is called a sub-grade modulus and is the "spring constant" per unit area of backfill that is in contact with the rock. The method for obtaining a value for K is outlined on



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Figure 6 and can be related to test data when a value for δ is required.

The total resisting force due to backfill is then given as

$$F_p = \int^D \sigma_p = \int^D (\sigma_{p_0} + \sigma_{pp}) = (K_0 D \gamma_f + K\delta)^D / 2$$

With all forces evaluated, an equilibrium subsidence can be obtained by equating $M_D = M_R$, giving

$$D d_0 \gamma_0 + 0.175 D^2 \gamma_r \frac{D}{2} = 0.7 D^2 \gamma_r D + K_0 D^2 \gamma_f + K\delta D \frac{D}{3}$$

which, by dividing by $D^2 \neq 0$, and rearranging terms, yields the expression for δ as

$$\delta = \frac{1}{K} \left(\frac{3 d_0 \gamma_0}{2} - 0.146 D \gamma_r - K_0 D \gamma_f \right)$$

The value of δ then predicts the maximum surface elevation change occurring at a distance, D , from the backfilled mine (only positive numbers are real since the direction of rotation is assumed; reverse rotation (heave) would mobilize rock shear in the opposite direction and is not physically possible).

Results from recent backfill testing (refer to "Testing of conventional, Pyrite Concentrate and Pyrite Slimes Backfill Materials - Crandon Project" September, 1981) may be used in this procedure.

The major constituent of the backfill can be approximated by sample 4 (the 10 μ split conventional tailings) and it's properties will be used in the assessment of surface effect presented in this section.

The bulk unit weight of the backfill would be



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$$1.895 \text{ t/M}^3 \times 9.81 \times (1 + 0.25) = 23.2 \text{ kN/M}^3 \text{ assuming } p_d = 1.895 \text{ t/M}^3$$

and the retained moisture content at saturation equals 25%. The maximum potential value of δ is given with

$$\gamma_o = 20 \text{ kN/M}^3, d_o = 70 \text{ m}$$

$$\gamma_r = 30 \text{ kN/M}^3, D = 100 \text{ m}$$

$$\gamma_f = 23.2 \text{ kN/M}^3, K_o = 0.5$$

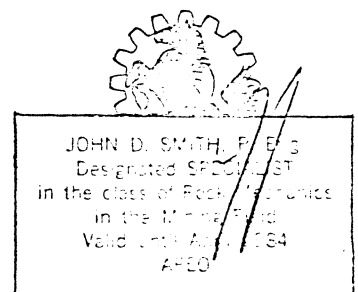
These values give $\delta = \left(\frac{502}{K}\right)$ metres.

From test data on 15:1 T:C cemented tailings backfill sample 4, an average unconfined modulus of about 1.6×10^5 kPa was obtained (see Figure 7, for example). The approximate constrained modulus would be given as $E_c = 1.6 \times 10^5 (1+\nu) / [(1+\nu)(1-2\nu)]$ where ν = Poisson's ratio. For this material, ν would be about 0.35, giving $E_c = 2.6 \times 10^5$ kN/m². Constrained tests were carried out on cemented backfill sample 4, and results are shown on Figure 8. Since the compression is non-linear, a value for E_c should be obtained at an average value of the at rest pressure, $\sigma_{p_o} = D\gamma/2 = 1000$ kPa (see Figure 6). From Figure 8, it is seen that the average constrained compression index is .023/log cycle at pressures above 600 kPa. The constrained modulus is given, from this data, as

$$E_c = \Delta\sigma / E_1$$

$$= \frac{\sigma_{pp}}{0.023 (1-n_o)} \log \left(\frac{\sigma_{p_o} + \sigma_{pp}}{\sigma_{p_o}} \right)$$

where n_o = initial porosity = 0.46 from Figure 8.



The above equation reflects the non-linear behaviour of cemented tailings and, since σ_{pp} is not known, an iterative solution would be necessary for an exacting calculation. It is noted, however, that E_c increases as σ_{pp} increases, and thus the lowest limit of E_c can be obtained by using a small nominal value of $\sigma_{pp} = 500$ kPa (just sufficient to obtain the required decimal accuracy in the logarithmic calculation). For $\sigma_{p_o} = 1000$ kPa and $\sigma_{pp} = 500$ kPa, $E_c = 2.29 \times 10^5$ kPa. A value of $E_c = 2.29 \times 10^5$ is assumed for the calculations of subsidence (δ) in this report.

$$\text{Then from all the above, } \delta = \frac{502}{K} = \frac{502 d_f}{E_c} \quad (\text{see Figure 6}).$$

Thus, even for the largest value of $d_f = 70$ metres (maximum hanging wall to foot wall dimension), $\delta = 3.5 \times 10^4 / E_c$ or $\delta = 0.15$ metres.

In our opinion, this calculation represents the worst possible condition.

When weak fractured rocks are involved in rotational failures, there is always an associated mass dilation (swelling or bulking) in the rock.

A rough estimate of this effect can be obtained from dilatancy factors used in earth mechanics. A minimal volume dilation would be 20% of the rotational strain. Thus, volume increase would be $\Delta V = (0.2\delta/D)V = (0.03/D)(0.175D^2)$ for the above case. This would be reflected, from the geometry of the circular section, in a reduction in subsidence of

$$- \delta = 8\Delta V/D \text{ giving the overall subsidence as}$$

$$\delta = \frac{(3.5 \times 10^4}{E_c} - 0.04)m$$

Then, for the case in question, a more accurate estimate of the subsidence

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effect in the event of a long term rotational failure in the weak upper rock strata would be $\delta = 0.15 - 0.04 = 0.11$ metres (about 4.3 inches). No surface break would occur since this small subsidence would be moderated by the overburden. The ground distortion can be estimated as $\delta/D = 0.11/100 = 1/909$. Figure 9 indicates that this distortion would be allowable for all buildings, with essentially no noticeable effect.

4.2.2 Wedge Failure in the Hanging Wall

If failure were to extend below the upper zone of fractured rock, continuous planes of weakness such as faults, bedding planes or fracture planes would have to exist. While these are not expected to occur in the Crandon Mine rocks, it is of interest to estimate the effect of such weaknesses on long term subsidence. The maximum depth of mining, that is 600 metres, would be the most critical depth and Figure 10 shows the free body forces acting on the sliding rock mass. The forces can be summed along the plane of failure to obtain an equilibrium condition.

$$\text{Then } (W_r + W_o)\sin\alpha - (F_r + F_{pp} + F_{p_o} \cos\alpha) = 0$$

$$\text{where: } W_o = D(\cot\alpha + \cot 80^\circ) d_o \gamma_o$$

$$W_r = D(\cot\alpha + \cot 80^\circ) D \gamma_{r/2}$$

$$F_r = N \tan \phi$$

$$F_{pp} = K (\delta/\sin\alpha) D \text{ (considered constant, as shown).}$$

The normal force, N, is given as $(W_r + W_o) \cos\alpha + F_{p_o} \sin\alpha$ and

$F_{p_o} = K_o D^2 \gamma_f / 2$ as previously calculated for the rotational sliding



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mechanism. Substituting into the equilibrium equation above,

$$(W_r + W_o)\sin\alpha = (W_r + W_o)\cos\alpha \tan\phi + K_o \frac{D^2 \gamma_f}{2} (\sin\alpha \tan\phi + \cos\alpha) + \frac{\delta KD}{\sin\alpha}$$

$$\begin{aligned} \text{Then, } \delta &= \frac{(W_r + W_o)(\sin\alpha - \cos\alpha \tan\phi) - K_o D^2 \gamma_f (\sin\alpha \tan\phi + \cos\alpha)}{KD/\sin\alpha} \\ &= \frac{(W_r + W_o)}{KD/\sin^2\alpha} (1 - \tan\phi/\tan\alpha) - \frac{K_o D^2 \gamma_f}{KD/\sin^2\alpha} (\tan\phi + \cot\alpha) \end{aligned}$$

It is apparent from the above equation that no positive (downward) movement is possible unless $\alpha > \phi$. The minimum value of ϕ , even for slickensided weak planes, would be $\phi = 30^\circ$. Using this value, together with the following values (introduced earlier):

$$\gamma_o = 20 \text{ kN/m}^3, d_o = 70 \text{ m}$$

$$\gamma_r = 30 \text{ kN/m}^3, D = 600 \text{ m}$$

$$\gamma_f = 23.2 \text{ kN/m}^3, K_o = 0.5$$

$$K = E_c/d_f = 2.29 \times 10^5/70 \text{ cosec}\alpha \text{ (due to inclination)}$$

we obtain:

$$\delta = \sin^2\alpha [3.18 (\cot\alpha + 0.176)(1 - \frac{0.577}{\tan\alpha}) - 2.13(\cot\alpha + 0.577)]$$

For $\alpha = 45 + \frac{30}{2} = 60^\circ$, the calculated value of δ is negative, thus indicating that the resisting forces due to rock friction ($\phi = 30^\circ$) and the 'at rest' lateral backfill pressure are sufficient to prevent any movement of rock wedges on weak planes.

4.2.3 Summary

Calculations presented in the foregoing sections show that the potential for surface elevation change (subsidence) due to long term

failure of the hanging wall is negligible (even if weak planes or fractured upper rock strata exist in the hanging wall), provided that the mine openings have been backfilled with either cemented or uncemented fill.

ELEVATION CHANGES DUE TO CROWN PILLAR RECOVERY

The crown pillar mining method is designed to minimize the risk of caving during crown pillar recovery operations. In the long term, however, the remnant crown pillar cannot be expected to remain stable and back caving would be expected. The surface effect due to caving can be minimized by backfilling tight to the back during crown pillar mining operations. The void space between the settled backfill surface and the rock back should certainly be less than 20% of the thickness of the remnant crown pillar (i.e. less than 1 metre per 5 metres of rock remnant thickness) so that caving can be expected to be choked off before reaching the overburden soils. With the presence of overburden water, openings into the caved rock must be avoided to prevent possible piping and loss of ground into the rock voids. When the back caving is limited by tight filling, the overburden weight will be supported by arching in the overburden soils and backfill. Low permeability cemented tailings should be used for backfilling the crown pillar openings in areas where potentially high seepage could result from remnant caving. This would eliminate the potential for loss of ground and would reduce surface elevation changes. Where the maximum hanging wall to foot wall spans of about 70 metres exist in the upper levels of the orebody, elevation changes due to back caving would be



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expected to be observed at the ground surface. With tight backfilling, surface elevation changes can be reduced to tolerable amounts associated with the elastic deflection in the rock surface above the initial caving and the compression in the backfill due to imposed surcharge. The maximum surface subsidence can be estimated by assuming that 50% of the overburden loading (700 kPa) will be directly supported by the backfill which compresses to a depth of 70 metres (span width) before the surcharge stress is transferred to the wall rocks. Then,

$$\Delta H = H C_n (1-n_o) \log (\sigma_f/\sigma_o)$$

where ΔH = settlement (subsidence)

C_n = 0.023 and n_o = 0.46 from Figure 8

H = 70 metres

$$\sigma_o = (H/2)\gamma_f = 812 \text{ kPa}$$

$$\sigma_f = \sigma_o + (\text{surcharge}/2) = 812 + 406 = 1218 \text{ kPa.}$$

These assumptions, which are believed conservative, give ΔH = 0.15 metres. Thus, surface subsidence is not expected to exceed 0.15 metres (6 inches) and may be substantially less than this value.

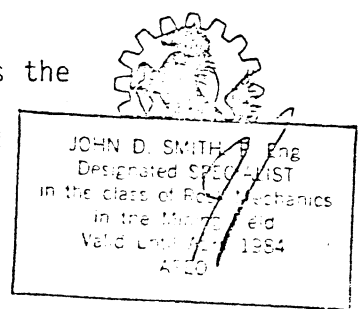
CONCLUSIONS AND RECOMMENDATIONS

1. During primary mining, surface effects will be restricted to the small deformations caused by drawdown of ground water due to the sink created by underground openings at atmospheric pressure.
2. The till cover cannot be relied upon to isolate the upper aquifer from mining activities.



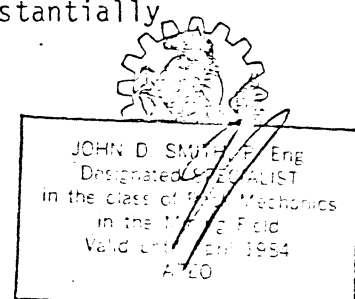
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3. From limited rock quality and permeability data available for the rocks near the overburden contact, grouting may be required to prevent upper aquifer drawdown. Any grouting done should be near the top of the crown pillar.
4. The overburden soils, except for the lacustrine and marsh, are not considered compressible. If the upper aquifer is drained, soil compression would still be quite small.
5. Ground water studies should be conducted during the early development program.
6. The design basis used for crown pillar height calculations is considered to be conservative and provides crown pillar heights measured downward from the altered zone. (The altered zone is considered to have zero strength).
7. The vertical height of the crown pillar is dependent on the depth of overburden and rock, the depth of the zero strength zone, the H.W. to F.W. distance, and the magnitude of the lateral stress in the crown pillar rock.
8. The exact extent of the zero strength zone can be better defined during early development or supplementary drilling and strength testing.
9. Much mining experience and engineering data will have been obtained before the critical stages of secondary mining and if needed would be used to modify geometry and procedures to control surface effect.
10. The conceptual mining method for the crown pillar is the transverse sub-level, V.C.R. cut and fill. Vertical



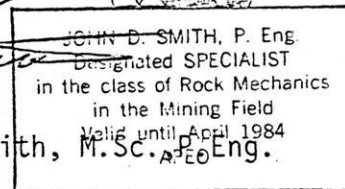
transverse slices are mined using V.C.R. from the extremities towards the shaft in such a way that ground support at all times is provided by broken ore, a combination of broken ore and waste rock, and waste rock.

11. Model tests or simulation techniques are suggested to optimize drawpoint drift spacing, horizontally and vertically, and the angle of the blasted face.
12. After all mining is completed, two potential H.W. failure mechanisms could result in surface elevation changes. Taking the worst conditions possible for a rotational failure mechanism, the maximum surface subsidence is estimated to be about 0.11 meters (4.3 inches). This would not create any surface damage. Backfilling will prevent any long term slumping of the hanging wall, even if continuous weak planes exist in this wall.
13. The potential for noticeable surface elevation changes, due to either hanging wall slump or crown remnant caving, is negligible provided that the mined out openings have been backfilled with either cemented or uncemented backfill.
14. Low permeability cemented tailings should be used for backfilling at the bottom of the remnant crown pillar in areas where potentially high seepage could result from eventual crushing and caving of this remnant.
15. In total, the maximum ground surface elevation change is not expected to exceed about 0.15 metres and could be substantially less than this value.



The writers of this report look forward to discussing the design concepts and opinions expressed with the mine staff. The writers are confident that a safe and viable mining operation can be carried out at Crandon without causing any adverse or noticeable surface effect. The mining and engineering technology is available now and used in conjunction with the operational experience gained during mining, will produce the desired results.

Respectfully submitted



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APPENDIX "A"

Table 1,
Figures 1 to 10.

TABLE 9-1
Typical Properties of Compacted Materials

Group symbol	Soil type	Range of maximum dry unit weight, p.c.f.	Range of optimum moisture, percent	Typical value of compression		Typical strength characteristics				Typical coefficient of permeability ft./min.	Range of CBR values	Range of subgrade modulus k lb./cu. in.
				At 1.4 t.s.f. (20 p.s.i.) percent of original height	At 3.6 t.s.f. (50 p.s.i.)	Cohesion (as compacted) p.s.f.	Cohesion (saturated) p.s.f.	ϕ (Effective stress envelope) degrees	Tan ϕ			
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix.	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1			>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6			>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}		
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}		
CL	Inorganic clays of low to med. plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21								5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50 - 150
OH	Organic clays and silty clays ...	65 - 100	45 - 21								5 or less	25 - 100

Notes:

1. All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.

3. Compression, values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown. (-) indicates insufficient data available for an estimate.

TABLE 1 - TYPICAL PROPERTIES OF DENSE SOILS



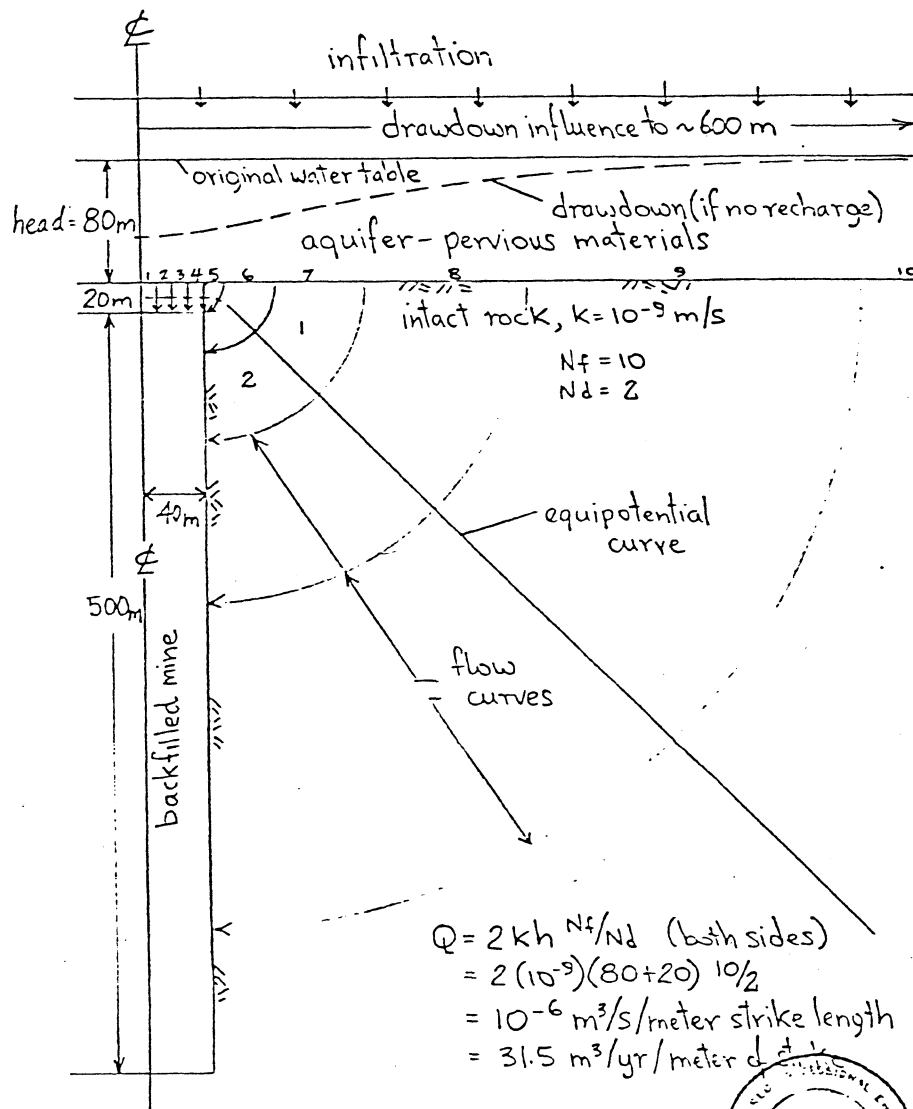
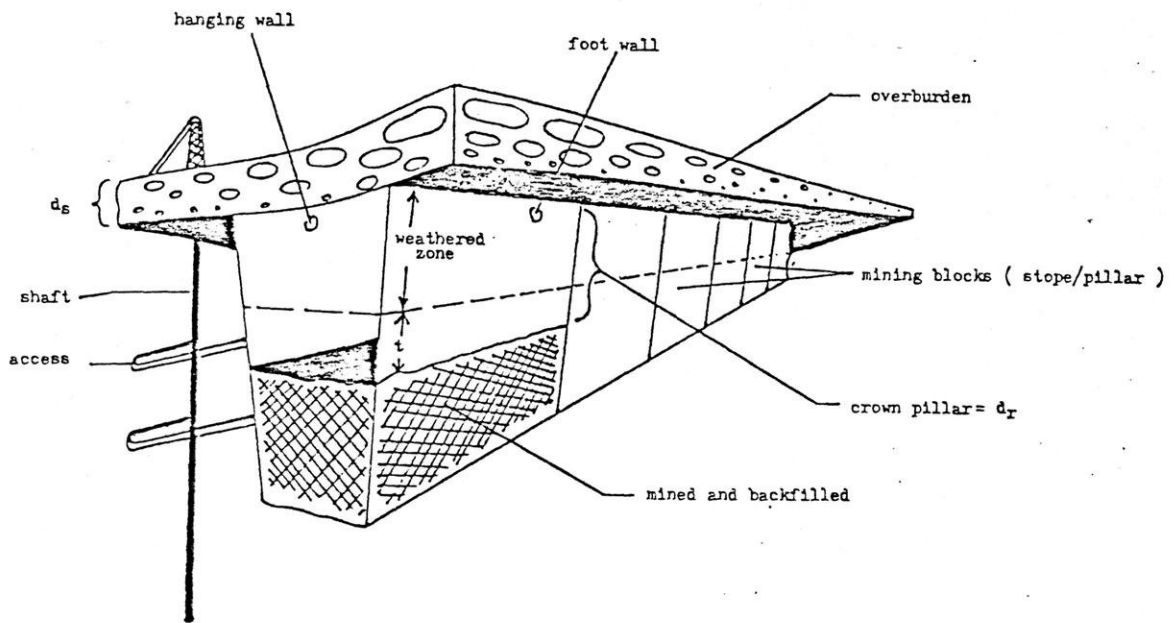


FIGURE 1 - AQUIFER BALANCE



$\nabla, \nabla_x, \nabla_y$ all act perpendicular to plane of hangingwall/footwall

FIGURE 2 - 3-DIMENSIONAL (OBLIQUE) VIEW OF OREBODY

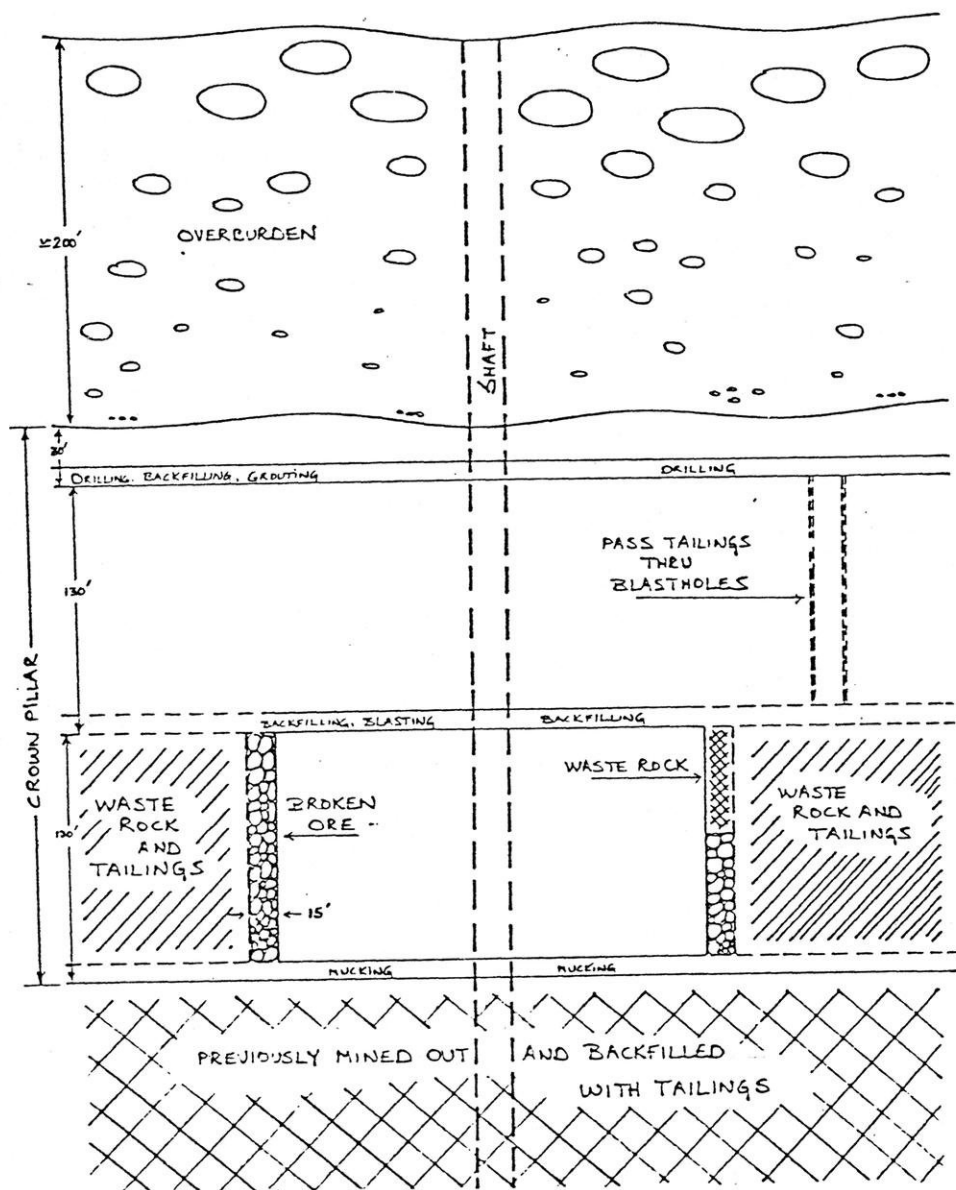


FIGURE 3.1 - GENERAL LONGITUDINAL SECTION OF CROWN PILLAR


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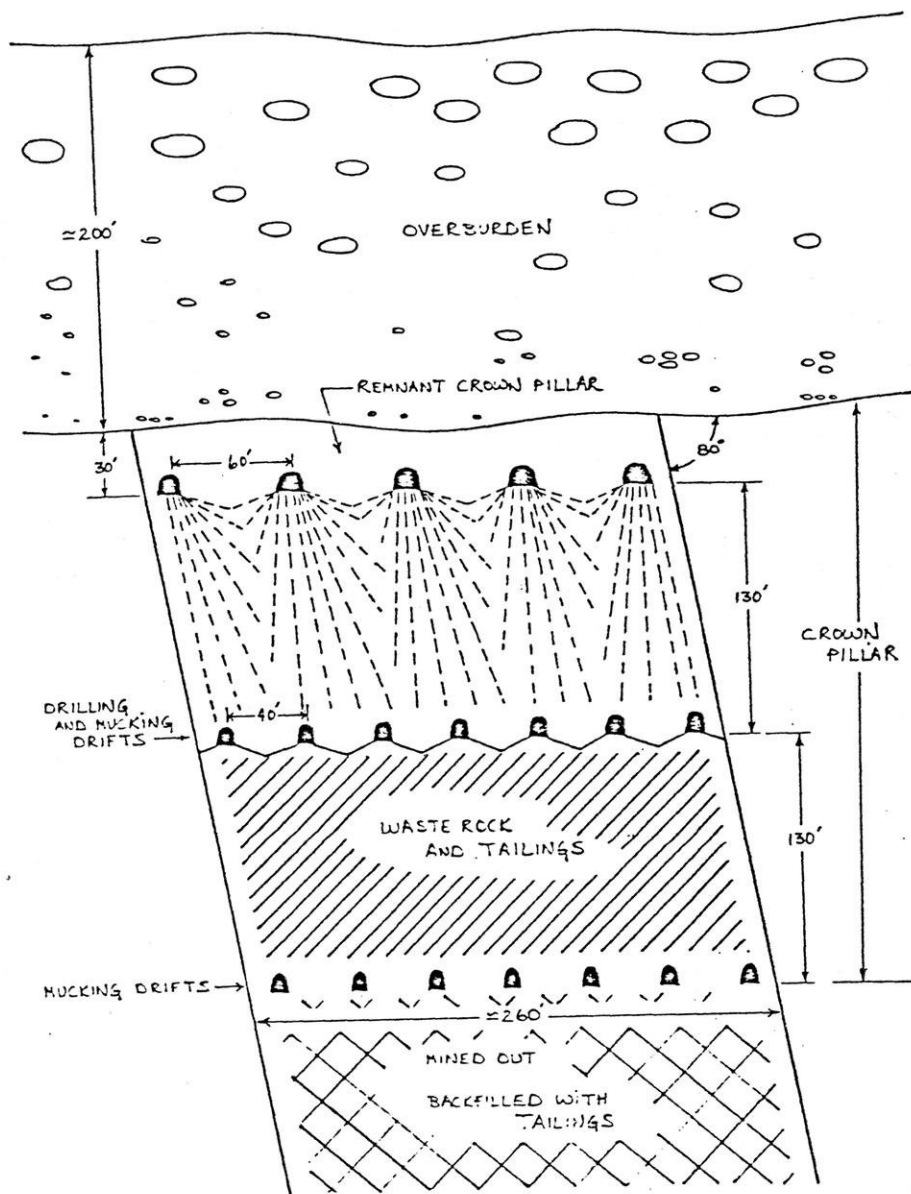


FIGURE 3.2 - GENERAL VERTICAL SECTION OF CROWN PILLAR

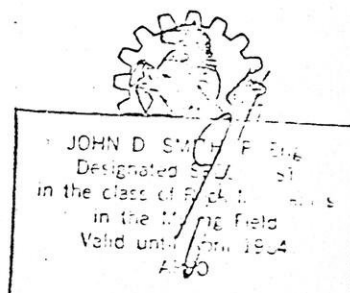




FIGURE 6 - CALCULATION OF PASSIVE RESISTANCE

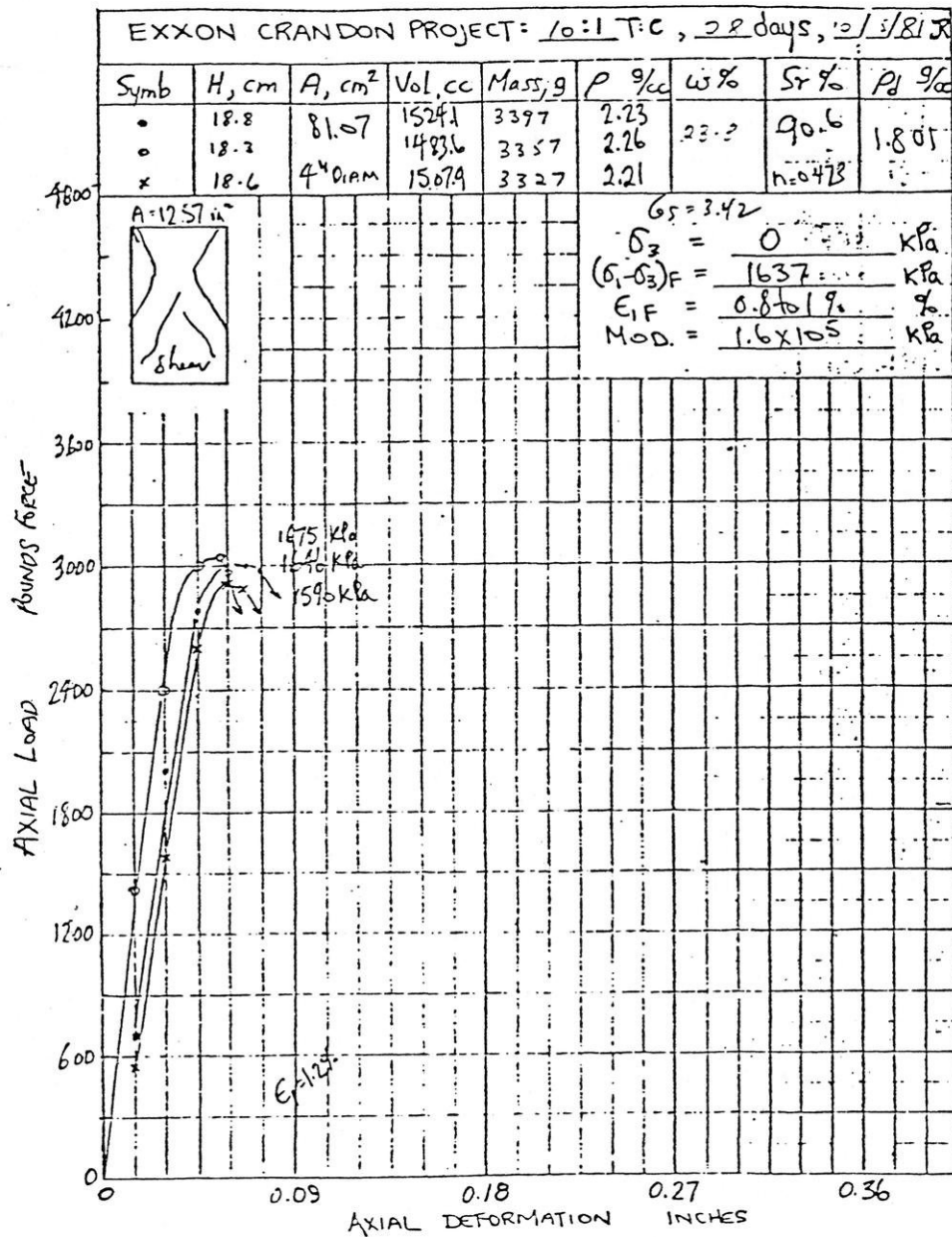


FIGURE 7 - (from report B) LOAD Vs DEFORMATION - SAMPLE 4



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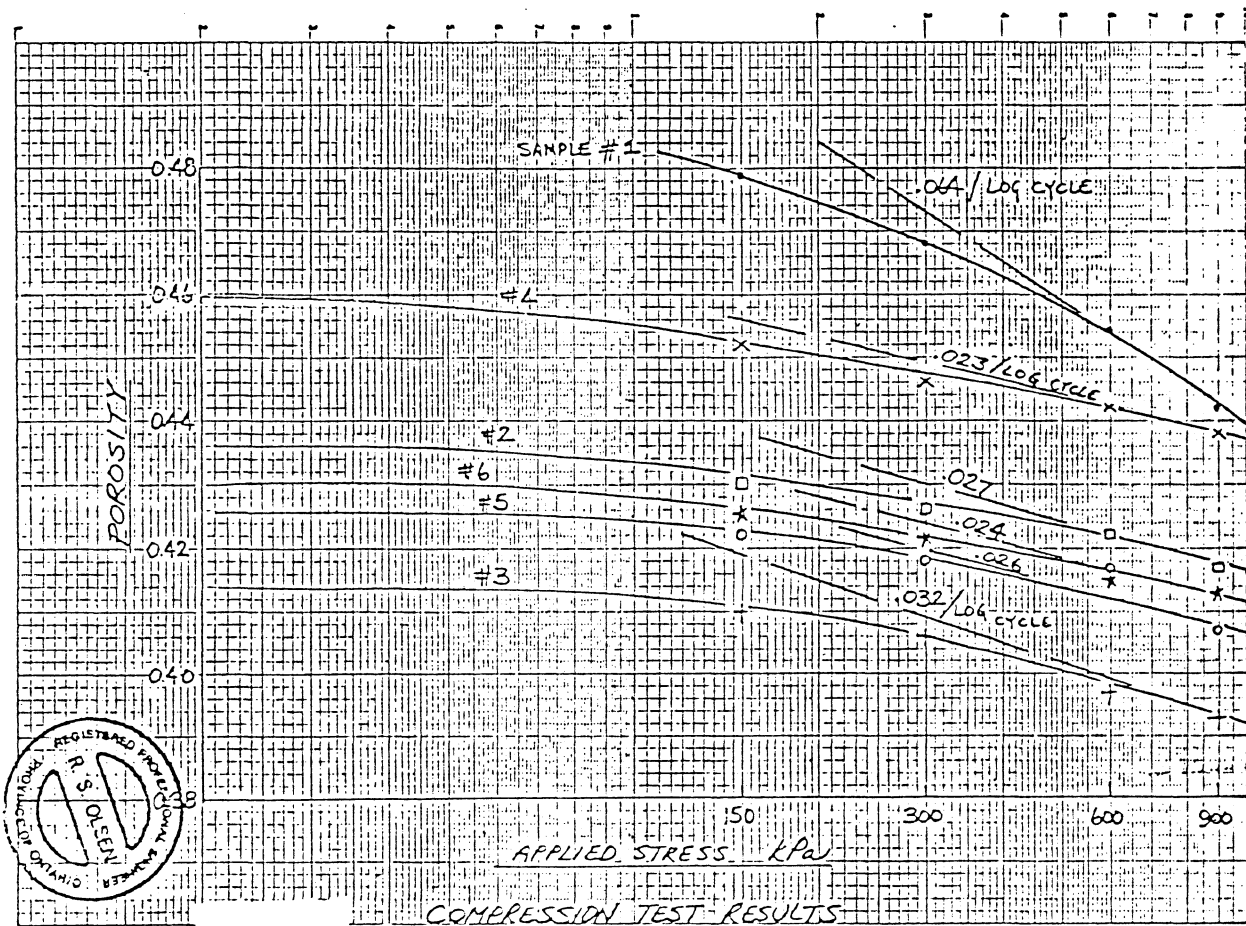
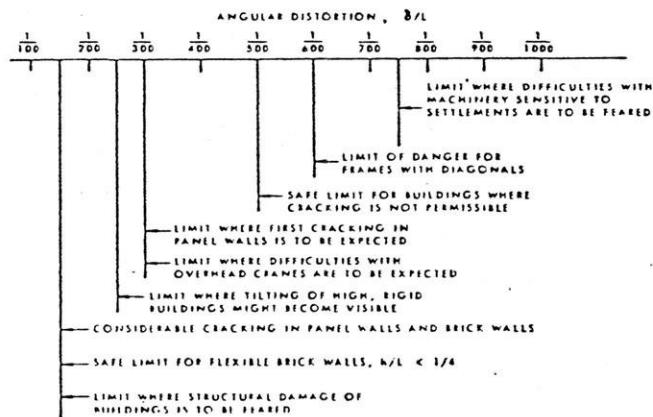


FIGURE 8 - (from report B) COMPRESSION TEST RESULTS

(11) Limitation on slope

Type of construction	Maximum slope*
High continuous brick walls	.005 to .001
Brick dwellings	.003
Brick cladding between columns	.001
Reinforced concrete building frame	.0025 to .004
Reinforced concrete curtain wall	.003
Continuous steel frame	.002
Simply supported steel frame	.005

Similarly values are given by BJERRUM (1963), in Fig 6.15.



(After Bjerrum)
DIFFERENTIAL SETTLEMENT AND ANGULAR DISTORTION RELATED
TO BUILDING PERFORMANCE

(from Canadian Foundation Manual)

* Maximum slope of deflected configuration from line of reference as-built configuration.



FIGURE 9 - DIFFERENTIAL SETTLEMENT AND ANGULAR DISTORTION
RELATED TO BUILDING PERFORMANCE



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APPENDIX "B"

Creation of Rock-Strength Zones

by Point Load Strengths.

Theory of Stope Span Design.

Creation of rock-strength zones by point load strengths

Point Load Data

Point load data was analysed from holes: 143, 160, 169, 170, 173, 174, 175, 176, 177, 178, 179, 180, 181, 182, 183, 184, 185, 188, 189, 190, 191, 193, 194, 197 and 198 to produce plans and sections of the Crandon project illustrating strength variations in the rock.

Borehole 178 is taken as an example to illustrate the presentation of the data. (FIGURE 1). The hole extends from surface to just above the 130 level. Point load data were taken at recorded footages along the hole. At each test point, a line perpendicular to the hole was drawn. The length of the line was determined by the magnitude of the point load strength (and the 1" = 20,000 psi scale of the drawing).

Strength contours were constructed parallel to the hole, defining areas 1 to 6. Each area represents a different strength interval.

Area	Strength Interval (psi)
1	0-5,000
2	5,000-10,000
3	10,000-15,000
4	15,000-25,000
5	25,000-40,000
6	over 40,000

In some places, the sample core was split prior to the strength test. These areas are represented on the drawings by "0".

NOTE THAT THE TERMS "AREA" and "ZONE" are used interchangeably



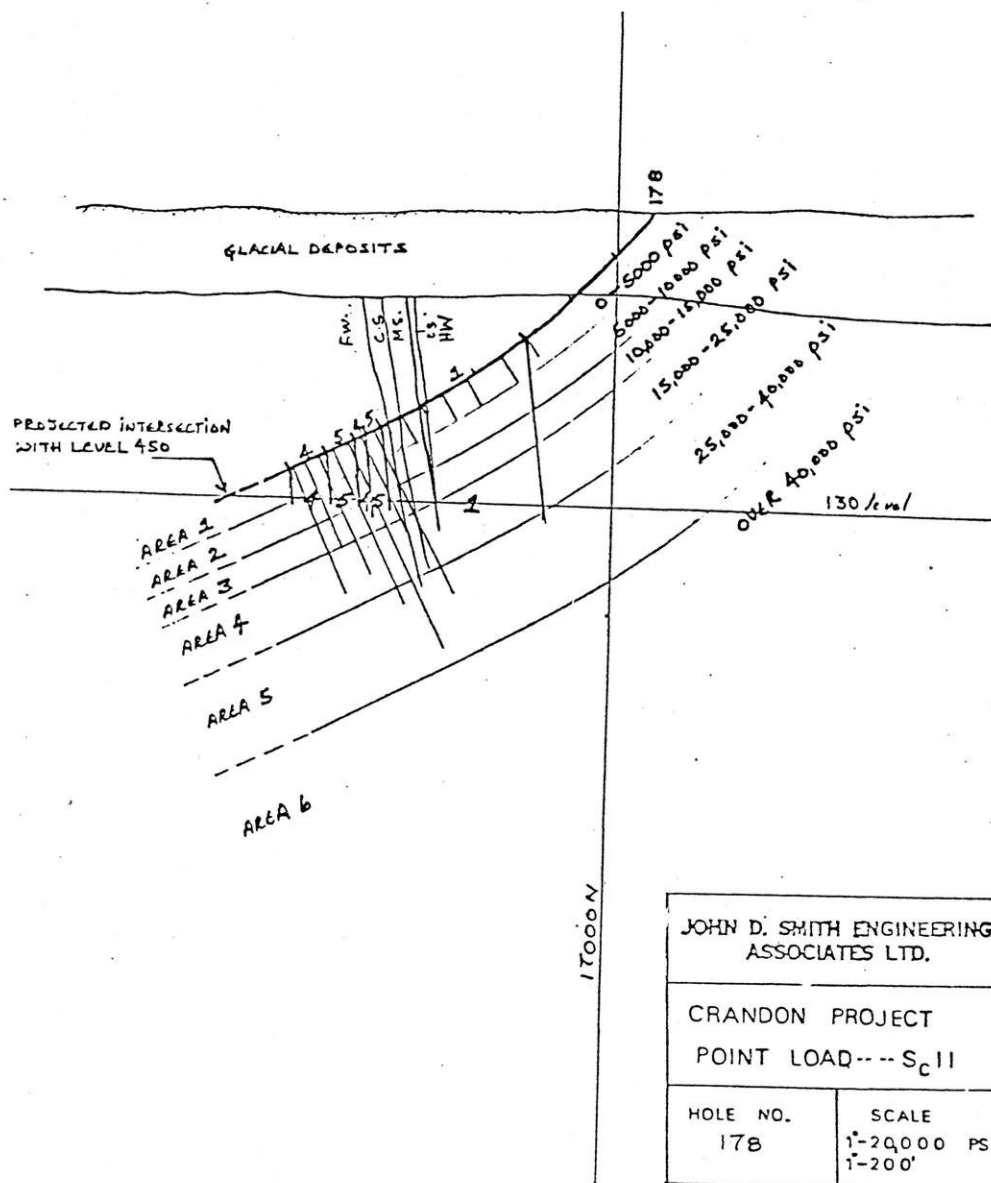


FIGURE 1



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THEORY OF STOPE SPAN DESIGN

Stope Spans

Roof beam span calculations can be done considering the structural element to be a beam-column. This procedure has been used by the writer for over 17 years in the design of safe roof spans. Although it is not perfect, due to the inherent characteristics of rock masses and the difficulty in measuring them, it has always provided a good base from which to make engineering decisions.

Basis for Design

The following discussion has been taken from lecture notes prepared by the writer.



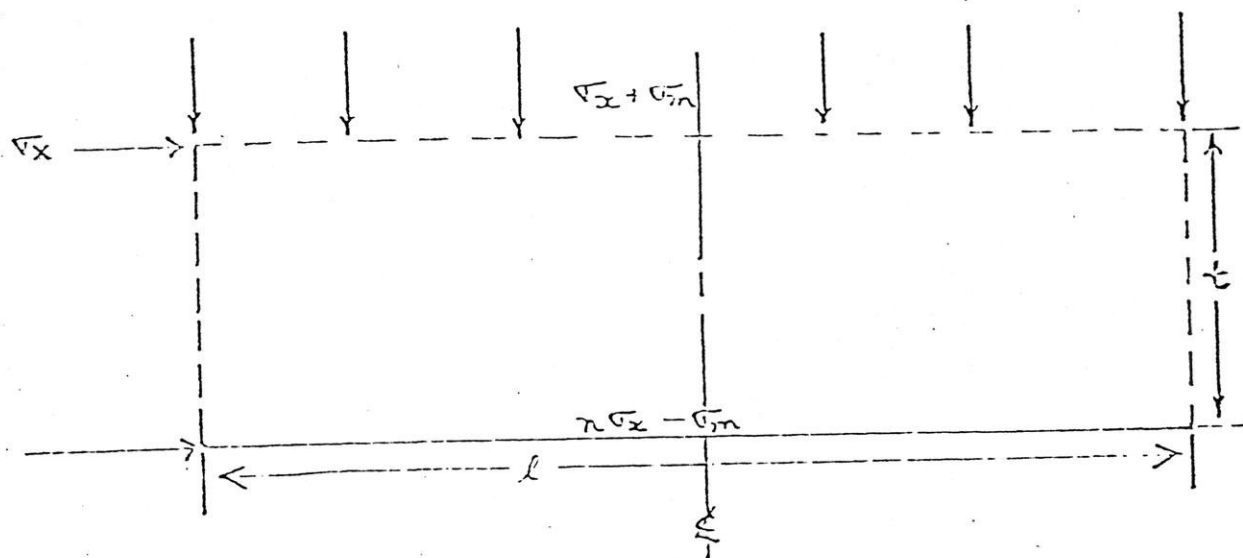
The inherent stress component in the strike direction provides the prestress for the beam column and it is considered that since the inherent stress in the H.W.-F.W. direction is concentrated in the pillars, the beam loading comes largely from its self-weight. The prestressed beam-column structural element shown in the diagram allows for compression or tension in the bottom chord depending on the size of the beam considered, and the ratio of vertical to horizontal loading. There are three possibilities for the condition of stress in the lower chord at the centre of the uniformly loaded, simply supported prestressed beam-column. If the lateral stress has a horizontal component in compression less than the tensile stress due to bending moment, the resultant extreme fibre stress at the centre will be tension. Somewhere along the extreme fibre, however, will be compression. If the horizontal stress is equal to the tensile stress, the bottom fibre at the centre will be neutral, but at a short distance on either side of the centreline there will be compression. If the horizontal stress is greater than the bending moment tensile stress, the bottom chord will be in compression everywhere along its length.

For design purposes, the worst condition would be to consider the structural element as a simply supported, uniformly loaded beam with added horizontal prestress. Since, underground, the rock in the back is not simply supported, except in extreme cases of failure, by using the elastic theory bending moment for a simply



supported beam, a safety factor is built in because the bending moment for a restrained-end beam is 1/3 times that for a simply supported one.

If one thinks of the roof as a slab of effective depth equal to the rock-bolt length or to the depth of some geological contact in the roof and accept the worst possible condition, that of simple beams one foot wide placed side by side and supported by the force field prestress, the result is the condition shown below:



σ_x = force field stress

n = multiplication factor due to geometry *

σ_m = unit fibre stress due to bending moment

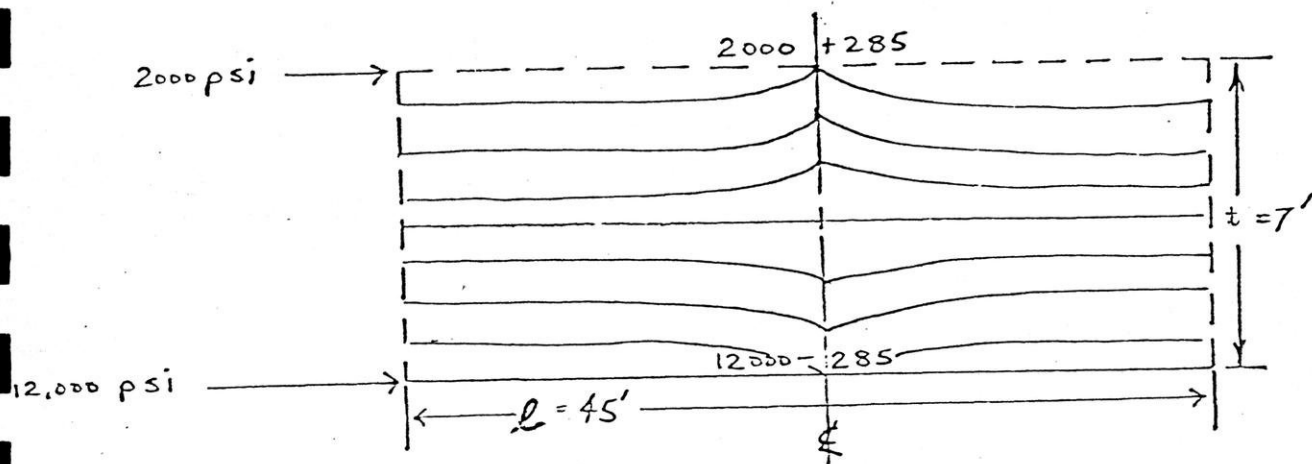
t = depth of beam - inches

l = length of beam - ft.

* multiplication factor and concentration factor are synonymous



From data obtained from an existing mine, the following diagram was developed:



The average rock density was 190 lb/cu. ft., and n was determined to be 6. For a simple beam, the unit fibre stress in the extreme fibres will be

$$\sigma_m = \pm M \times \frac{C}{I}$$

where $M = \frac{Wl^2}{8} \times 12$ lb. - in. and $\frac{I}{C} = \frac{bt^2}{6}$, $W = 1330$ lb/ft.

$$\text{Then, } \sigma_m = 1330 \times 45^2 \times \frac{12}{8} \times \frac{6}{12 \times 84^2}$$

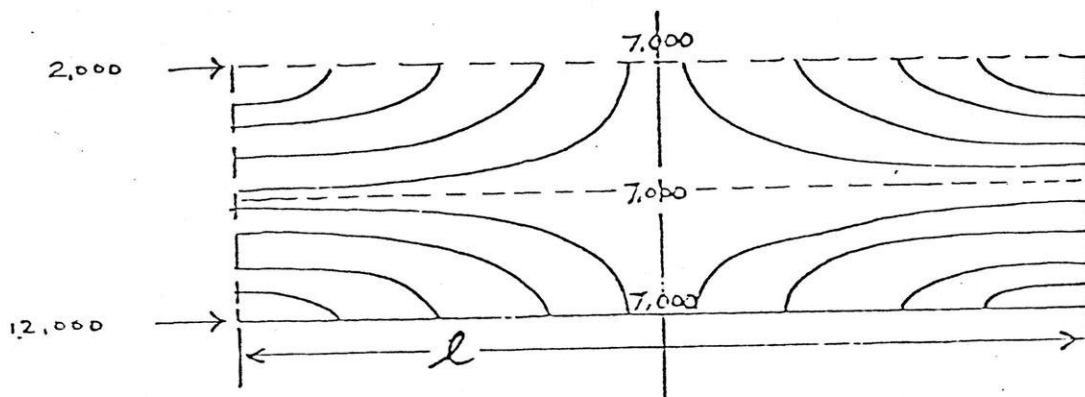
$$= \pm 285 \text{ p.s.i. (approx.)}$$

The isostress lines are almost horizontal with uniformly large stress differences within short distances vertically. The top fibre of beam would shorten by about 0.0077 inches and the bottom by about 0.64 inches. Such differences might be tolerated by rock if enough time is allowed. There would be a tendency to split along



horizontal shear planes which might be additive to the inherent shear strains already in the rock.

To improve the condition, σ_m must be increased such that at the centre of the beam the top and bottom extreme fibres shorten the same amount. The following diagram shows such a condition.



Top fibre stress at midpoint is $\sigma_x + \sigma_m$ and the bottom fibre stress at midpoint is $n\sigma_x - \sigma_m$ with $\sigma_x = 2000$ p.s.i. and $n = 6$. For equality, $2000 + \sigma_m = 6 \times 2000 - \sigma_m$

$$\sigma_m = 5000 \text{ p.s.i.},$$

$$\text{but } \sigma_m = \frac{Wl^2}{8} \times 12 \times \frac{6}{bt^2} \quad \text{from } \sigma = \frac{Mc}{I}$$

and from this, $l = \sqrt{\frac{\sigma_m \times 8 \times bt^2}{W \times 12 \times 6}}$ ft. where b and d are in inches,
 W is in lbs/ft. of beam and

$$l = \text{ft.}$$

$$l = 188 \text{ ft. approximately.}$$





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