Energy, Mines and Resources Canada
Énergie, Mines et Ressources Canada

CANMET
Canada Centre for Mineral and Energy Technology
Centre canadien de la technologie des minéraux et de l'énergie

A DESIGN PHILOSOPHY FOR SURFACE CROWN PILLARS OF HARD ROCK MINES

M.C. Bétournay

Rock Mechanics Laboratory

December 1986

Revised version of the presentation at the 88th Annual General Meeting of the CIM, Montreal, Quebec, May 11-15, 1986, published in the CIM Bulletin, July 1987.

MINING RESEARCH LABORATORIES
DIVISION REPORT MRL 86-139 (J)
Revision of Divisional Report M&ET/MRL 86-62 (OP)

A DESIGN PHILOSOPHY FOR SURFACE CROWN PILLARS OF HARD ROCK MINES

by
Marc C. Bétournay*

ABSTRACT

Surface crown pillars, acting to safely protect underground workers and operations from surface elements, are recognized herein as distinct from conventional deeper underground pillars. Moreover, because of the variation in mining and geological conditions that exist from mine site to mine site, each case is viewed as unique. A review of numerous case studies and existing literature indicates that there is little general information or systematic problem solving approach associated with them.

A design process, not unlike that of other engineering disciplines, is presented to correct these deficiencies. As part of its purpose, it will inform operators of the key elements required for design and will address the problems of these appendages. The design, while taking into consideration possible surface crown pillar settings and inherent characteristics, is founded on a step-by-step procedure starting with site investigations through to monitoring or pillar recovery. The process is flexible, in that it incorporates decision making and changes in mining strategy. The purpose of each step is explained and each step is extensively described in terms of the required equipment and methods for field work, analysis and support measures. Analytical formulae and studies from several fields directly applicable to surface crown pillars are also given. Recommendations for future research studies are presented.

---

* Project Leader, Rock Mass Characterization; Mining Research Laboratories, CANMET, Energy, Mines and Resources Canada, Ottawa.

keywords
surface crown pillars, Canadian mines, design philosophy, design process, geotechnical investigations, mining strategy, data analysis, design methods, mining activity, monitoring, back analysis, pillar recovery, design evaluation, future surface crown pillar research.

UNE PHILOSOPHIE DE CONCEPTION POUR LES PILIERS DE SURFACE EN ROCHE DURE

M.C. Bétournay *

Laboratoires de Recherches Minières
Centre canadien de la technologie des minéraux et de l'énergie
(CANMET)
Ottawa, Canada

RÉSUMÉ

Les piliers de surface, existant pour protéger les travailleurs et les opérations souterrains des conditions de surface, sont reconnus ici comme étant distinct des piliers conventionels plus profonds. En plus, à cause de variations de conditions minières et géologiques de site en site, chaque cas est considéré unique. Une revue de plusieurs cas d'études et des publications existantes indique qu'il y a peu d'informations générales ou de méthodes pour solutionner les problèmes en existence.

Un cheminement de conception, semblable à ceux utilisés dans d'autres domaines d'ingénierie, est présenté pour combler ces déficiences. Une partie de sa raison d'être est d'informer les opérateurs des éléments clés requis pour la conception et d'adresser les problèmes de ces structures. La conception, en considérant tous les genres de piliers de surface et leurs caractéristiques, est fondée sur un cheminement d'étapes à étapes débutant avec les investigations du site jusqu'au suivi ou du recouvrement de pilier. Le processus est flexible parce qu'il incorpore les décisions et le changement de stratégie minière. Le but de chaque étape est expliqué et chacune est décrite en terme d'équipements et des méthodes requis pour le travail de terrains, les analyses et le soutènement. Des formules analytiques et des résultats d'études de plusieurs sujets applicables aux piliers de surface sont également présentés. Des recommendations pour des études potentielles sont énumérées.

---

*Chercheur en Sciences Physiques, Laboratoires de Recherches Minières, CANMET, Énergie, Mines et Ressources Canada.

mots clés
piliers de surface, mine canadiennes, cheminement de conception, étapes de conception, recherche

1.0 INTRODUCTION

For many hard rock mines with near surface deposits (0-100m), the surface crown pillar forms a first line of protection for the immediate underground workings, Fig. 1. In fact, this function points to using "protection pillar" as a better descriptive term.

Standardizing the same analytical formulation, techniques, for all surface cases is inappropriate given the wide variation in structural geology, deposit and host rock characteristics and mining methods from site to site. In fact, surface crown pillar design follows the maxim of geotechnical engineering: "every case is a unique case". Beyond applying rock engineering, the possible need for geotechnical input originates from the physical nature of the entire problem. Far from being well controlled conditions, the rock material is fractured and weathered to various degrees and often supports a load consisting of several soil types, perhaps a body of water such as a lake or a river, or even surface installations. It is evident that designing these surface structures, where gravity predominates, using deep level pillar formulation is also inappropriate.

CANMET is preparing a handbook on surface crown pillars to provide the Canadian mining industry with guidelines for safe and economic design. Being more than a reference volume, it will permit the operator to relate to existing circumstances and needs for surface crown pillar dimensioning, support requirements as well as mining developments. It will supply the operator with the basic tools–recognition of key elements for design, step-by-step procedure for site investigation, data analysis, analytical and modelling techniques, support measures, monitoring, etc.– including the advantages, disadvantages and limitations, so that he can best choose to appropriately address his needs.

This article introduces a systematic, integrated design for surface crown pillars, which requires step-by-step procedures found in engineering disciplines.

2.0 SURFACE CROWN PILLAR CHARACTERISTICS

2.1 Role

The definition of a surface crown pillar (Fig. 1) gives us a hint of its role, it is to:

i) protect the workers from surface elements

- bodies of water
- soil

- precipitation

and must be itself safe enough not to injure

ii) prevent pillar elements from interfering with mining activity

- water inflow
- surface subsidence
- soil removal
- rock removal due to sloughing

iii) keep extraction economical

2.2 Uniqueness

The procedure of dimensioning materials for engineering specifications requires a thorough and proper approach. For surface crown pillars this means formulating and shaping for optimum dimensions. With experience, this design becomes an art but never an exact science; the constituent materials and their properties make it so. Compared to design of steel or concrete, the designer of surface crown pillars must deal with unsuitable materials that cannot be substituted, and often faces dimensions and loads that cannot be altered.

Two recent surveys (1)(2) have focused on the settings of these pillars. 21 of the 24 mines examined are situated in the Canadian Shield, as are most of this country's hard rock mines. Several common features from hard rock settings are evident: steep dipping, narrow deposits, overlain by considerable overburden, Table 1. Alteration of the rock is often pronounced and hangingwall/footwall conditions have little competence. Two or three joint families and faults occur at each locality. Critical combinations of factors such as a lake or well-indurated sediments overlying incompetent rock/altered material must also be recognized. Such a scenario requires careful design to prevent major, disastrous failures.

The uniqueness of each mine site stems from the variations that exist between them. Even if there are resemblances in ore and geological characteristics between various deposits, chances are at least one of these factors will differ:

- structural geology
- mining method/dimensions

- bedrock conditions
- overlying soil
- water (overlying, circulating)

Also unique, and one of the most important factors in designing surface crown pillars, is the usual low values of lateral stresses occurring near surface which, under a broken mass condition, cannot be counted on to contribute to its stability. This implies the action of only gravity loads and structure on this type of pillar. Those separating open pits from underground workings can be subjected to more important lateral stresses. The lack of practical information pertaining specifically to this subject and dedicated provincial mining laws pertaining to them also make surface crown pillars singular.

3.0 DESIGN PROCESS

The design process suggested in this article retains the concept of following a step-by-step procedure for creating a safe structure. It was kept simple to remain flexible for all mining operations, but the content of each step was meant to be pertinent and wide ranging. Figure 2 shows the process flowchart. It incorporates decision making and changes in mining strategy. The objective of the design is to safely and economically dimension surface crown pillars.

Decision making occurs several times during the life of a mining project. The first design decision is whether to have a surface crown pillar or not and, if so, whether it will be mined leaving temporary or permanent dimensions and if it will be removed. This has a great impact on the state of expectations about the stability of the uppermost mine reaches. In the most demanding mining strategy, a mine can excavate the uppermost stopes to safe recommended final dimensions at the beginning of the life of the mine. This requires that all data and analysis be completed, permanent support methods emplaced, and an active monitoring program established. Here the burden is on the mine to have sufficient data to have a good grasp of the situation for such a design. Often, this necessitates work which would last well beyond an early mine stage. As well, the dimensions and support cannot be underdesigned. In the case of such long term stability requirements, fill should be emplaced immediately after final dimensions are reached.

Here are the recommended design steps.

1. Identification of Deposit and Regional Rock Characteristics
2. Geotechnical Investigation

3. Establishing Mining Strategy and Advance
4. Data Analysis
5. Design Methods and Initial/Final Dimensioning
6. Mining Activity
7. Monitoring
8. Back Analysis
9. Recovery
10. Design Evaluation

3.1 Identification of Deposit and Regional Rock Characteristics

In the pre-mining evaluation of a mineralized site, it is essential to obtain sufficient characteristics about the rock mass to base any pre-mining layout and excavation method on factual information. Failure to do so will often lead to unforeseen expenditures and ground control problems in the early mining stages. But it is evident that there is always the possibility that some unforeseen characteristics are discovered while excavating underground. It is thus highly recommended to tie in all data collecting at the deposit site exploration or earliest possible stage to obtain some preliminary evaluation of a potential surface crown pillar in the early feasibility or mine planning stages. Beyond this purpose, and that of providing a basis for analytical and model solutions, early gathering of data types covered in this and the next step will give the designer a "feel" for local conditions. The following characteristics are intrinsic to major mining activity and to the design of the pillar:

deposit characteristics

- shape of orebody
- deposit emplacement method
- ore concentration and distribution
- dimensional extent and attitude of orebody
- footwall and hangingwall conditions
- joint survey/location mapping

host rock characteristics

- rock types and formation history
- folding and faulting trends
- joint survey/location mapping

Each of the above characteristics will have a bearing on one or more of the following design considerations: pillar or not, pillar thickness, stope size, ground control.

General structural trends can be observed on air-photos and other indications may be obtained from other mines sharing the same orebody or structural unit. Surface mapping, where possible, should be done in as much detail as possible to outline rock contact relationships, dimensions, etc. and to pick up the nature and extent of altered or broken zones. Covering the same area as the geological mapping will point to the exact location of major joints and the variation of jointing between geological units or rock mass regions. The relationship between established sets (similar orientation groups) will establish the brokenness and interlocking nature of the rock mass, seen in detail in the next section.

Diamond drilling core provides underground rock information and holes permit future uses of instrumentation and borehole camera. Unfortunately, the core is too often drilled in small sizes, e.g. AX for reserve assessment, and subsequently split or discarded. Large size core can be used successfully for geomechanical testing or obtaining joint characteristics. The core should also be kept intact for future reference and use. It is also commonplace for the drillers to concentrate on core length rather than on recovery. In effect, this gives very low or no attention to geomechanical parameters. The results are usually disastrous: bad core shape, lower R.Q.D. values, lost joint infilling and roughness properties, undistinguishable joint orientation, loss of weathering material, etc. Double or triple barrel coring will preserve the core in an undisturbed state. Much information can be obtained from such core samples which cannot be obtained from single barrel coring.

3.2 Geotechnical Investigations

This step is aimed at obtaining numerical information on the geomechanical characteristics of the rock mass and the factors affecting it. The geotechnical characteristics to obtain are indicated in Table 2.

The most important reason to carry out field tests is to obtain large scale values and behaviour that cannot otherwise result from laboratory tests. Such is the case for strength, deformation and permeability of rock and soil. The intent here is to present only the methods and equipment better suited for surface crown pillars. So far, there has been no reliable method or equipment to determine the stress distribution within a surface crown pillar and the variables affecting it: structure, weathering, water, tectonic stresses, etc.

An estimate of the brokenness of the mass can be achieved by mass elasticity modulus measurements, permeability measurements and discontinuity surveys.

Determination of the deformability of rock in-situ is possible using different methods:

- Plate bearing
- Borehole pressure

More often, the results will reflect only the conditions neighboring the test area.

The plate bearing test consists of applying normal load to a large flat rock surface (wall, roof or floor of underground or surface workings). Results of plate load tests have great application in engineering design where the stability of the designed structure is greatly influenced by the deformation characteristics of the underlying or surrounding rock mass (3). The method can account for material anisotropy, structural compressibility and pressure distribution variations. By applying successively higher bearing pressures an elastic modulus and a deformation modulus (elastic and plastic strains) can be obtained.

Small borehole tests find more application on surface crown pillars because the tests can be carried out from surface without the need for outcrops. The principle is based on applying pressure to walls of a borehole and measuring the radial response of the wall. The variations of this test type are:

- dilatometers which measure with uniform radial pressure
- borehole jacks, which apply force on small plates along a limited portion of the circumference
- borehole penetrometers which force a small indenter into the walls

Determination of deformability using borehole devices has certain advantages. They allow deformability measurement at points far removed from the excavation not affected by induced stresses or site preparations. The method is simple, requires less site preparation and permits numerous tests at low cost in a short period. Each method yields a wide data scatter with very high standard deviations because of the presence of one or more types of discontinuities, inherent rock anisotropy and amount of joint infilling (4). These would affect the loading conditions, stress distributions, deformations, or any other parameters that are applied or measured to determine the in-situ deformability of the rock mass (5). Cyclic loading increases the material elasticity by closing discontinuities and consolidating soft material.

Hydraulic conductivity of soils, rock masses and large discontinuities cannot be obtained from small specimens. Ideally, measurements should be made in situ. In saturated circumstances, below a water table, a most commonly used method is the Thiem method, calculating permeability from constant rate pumping tests. Kirkham's method of rapid pumping from an equilibrium level permits less expensive surveys. In unsaturated conditions water must be pumped into the ground from which the amount accepted by the soil and permeability are calculated. Open ended (constant head over the entire hole length) and packer tests (constant head over a portion of the hole) are two such tests.

The variety of packer systems presently available and the conventional packer types used in them (pneumatic, mechanical) often yield variable results. Most errors are linked to the unknown value of the pressure in the test cavities and to the leaks of water encompassed in poor sealing and hole matching by packers. Some new equipment development (6) has had success in resolving these problems. The conductivity will be influenced by the different joint systems, fissure types and changes in aperture. Packer tests can also be employed for determining the conductivity of individual joints. The results are misleading if the pumping exceeds the internal stress of the surrounding rock. It is preferable to carry out the test at various pressures and plot the flow rate against pressure or gradient. This will supplement the permeability values by qualitatively indicating the deformability of the rock. This curve also serves as indicator of the maximum pressure to be applied in further tests and the deformability of the joints (7).

A comprehensive study of discontinuities is the most important estimate of surface crown pillar stability. A quantitative survey will:

i) Map the orientation and extent of major joints, faults, shear zones

ii) Obtain sufficient joint measurements to reflect general and locally different/critical conditions

iii) If local variations are many, require that the local study be expanded to survey systematic, non-systematic, incipient and blast-related features.

Barring suitable or available exposures of the pillar, underground and surface, trends of nearby areas can be qualitatively considered. The reliability of this information indicates that it be used in preliminary considerations only, not for dimensioning.

General surveys of joint orientation, number of families and joint density measures can use the line survey method, but to capture local anomalies and obtain precise values for persistence, spacing, etc. entire areas should be covered. In such a survey, joint termination based on exposure is also important, to provide supplementary information on how the mass is broken up. Qualifiers of joint termination are:

- throughgoing
- termination in rock
- termination against other discontinuities

Joint spacing for each family should be recorded and plotted as a histogram.

While each feature is being surveyed, the opening, infilling and roughness should also be quantified. For the latter, a scale such as the one used in the Barton mass classification (8) or joint failure criteria (9) is useful and can later be tied in to the dimensioning process. The roughness of the joint and any shear test performed on it will be more representative when larger samples are used. Scaling factors have to be applied when using small areas. Drill core measurements can supplement the field measuring campaign and include joint orientation, joint spacing, RQD, roughness, opening, infilling and weathering. Trying to correlate joint persistence and extrapolation of block size between boreholes requires a very closely spaced pattern; even so, these should be recorded as considerations, not facts.

Other means of obtaining certain geological formation properties include geophysical borehole logging using probes. Such methods include electric logs, radiation logs, sonic logs.

The lab test on rock material will yield properties reflective of smaller more intact specimens rather than actual field conditions. However, the results are used in various dimensioning and support requirements. The strength values at low confinement are sufficient to model field conditions. Beam type rather than Brazilian type tensile strength tests are recommended. Sufficient tests are required to provide a good cross section of values. In standard compression tests, ten tests for each confinement are usually adequate. In multiple failure state tests, where one sample yields peak strength and residual strength at all desired confinements, only ten samples suffice (10). Directional properties of geologic material (in the field as well as lab tests) such as foliation, bedding, etc. should be identified as well as their effect on strength properties.

Laboratory shear tests should be performed on as large and representative samples as possible (e.g. roughness, infilling) and under as varied normal loads as possible, representative of field conditions. The results obtained will support a failure criteria for jointed rock masses, using such approaches as the Barton-Bandis (9) and Ladanyi (11) models.

The soils overlying surface crown pillars require thorough investigation for the following reasons:

i) the load on the rock pillar must be known for dimensioning

ii) the water content and impermeabilization capability of soil units will supply an estimate of the flowing capability, should a pillar failure take place; groundwater seepage capability of units and/or overburden as a whole are also obtained

iii) the possibility of removing the overburden to reduce the load on the pillar and reduce the potential inflow of soil into the mine in the event of a failure

iv) the capability of a soil to carry some of the overburden load and prevent inflow, e.g. stiff, clay-rich till

v) often times subsidence of the rock will cause soil to follow, presenting problems to surface installations

Since the kinds of soils and stratigraphical relationships vary widely for Canadian hard rock settings (1)(2), there is always need to do a detailed survey to locate and quantify what the units are and the problems associated with them. Obtaining representative properties by lab tests can be most difficult. In-situ field tests are thus preferable and more representative; several methods are generally used: geophysical, excavating, boring and soil penetration techniques.

Geophysical methods will yield subsoil profiles when variations in strata types are enough to permit good separation of signals using seismic or resistivity surveys. These methods have depth limitations.

Physical sampling from excavations or borings followed by laboratory investigation is the most widely used technique to obtain soils information. Borings can be made with or without casing. Core samples of stiff material, such as certain tills or weathered rock, are obtainable by using dense cutting fluids and slow advance rotary drilling, as the samples are relatively undisturbed. Samples, both disturbed and undisturbed, are also obtained by driving or pushing a sampling spoon, an open-ended device, into cohesive or cohesionless soils. These samples can then be lab-tested for stress-strain properties (if undisturbed) and/or analysed for grain size and plasticity. These methods can be used to find variations in soil profiles. Without using coring techniques, there are difficulties in obtaining high quality undisturbed soil samples from great depths. The sampling operation, sample transportation and specimen preparation subject the soil to stresses which are quite different from those experienced in the ground. Such changes alter the structure and behaviour of the soil.

In-situ pressuremeter tests offer the advantage of producing representative field values for basic parameters while least disturbing to the sample, especially for the case of the self-boring model (12). At depth, the lateral stress, soil stress-strain behaviour, modulus of elasticity, soil shear strength and, in some cases, consolidation characteristics of soils can be obtained with this method. Other test types can obtain these parameters separately. Vane shear tests are inexpensive, easy to use, and yield reproducible results of peak and residual shear strength of cohesive soil. Penetrometer tests, widely used, consist of driving cone headed rods into the ground and recording resistance. Depending on several factors, the method yields relative values of sand density and unconfined compressive strength of clay. Visual determination of soil types and extent is obtainable from natural or man-made trenches, cuts, etc.

In-situ permeability testing will provide good information on the mass permeability of a soil under real conditions; lab tests tend to undervalue in-situ overall permeability, particularly for fissured surficial clay in which the impact of hydraulic conductivity of discontinuities on water infiltration is great. Hydraulic fracturing of the clay can occur if the water pressures used are too high, resulting in an overestimation of permeability. New techniques are available to eliminate these problems (13).

Since groundwater affects many elements of surface crown pillars its location needs to be established as accurately as possible. It is generally determined by measuring the water level in boreholes after suitable time lapse. If reliable data are necessary, a piezometer should be installed in a borehole and periodically inspected until the groundwater level stabilizes. However, there are some drawbacks to this method. In wet clay soils, the insertion of a piezometer or casing remolds the clay, closing fissures and consequently leading to an underestimation of permeability. Artesian pressures and perched water levels can create an interpretation problem. If the groundwater is under pressure, deeper borings tend to raise the water level.

3.3 Establishing Mining Strategy and Advance

The need for such a surface structure must be taken into consideration in long term mine planning. Accepting that a surface crown pillar is required creates the need to use all available information to shape a portion of a rock mass for maximum safety and extraction at the lowest possible cost. The solution must fit within the boundaries of the chosen mining strategy while respecting mining regulations.

Recognition that there may be a need for a surface crown pillar can occur at several points in the life of a mining project; ideally it should be when preliminary mine strategy is established, giving consideration to the following factors:

i) sequence of mining and advance to/away from the pillar

ii) time left unsupported after excavation

ii) nature of support for the remainder of the mine life/until final dimensioning

Support should be applied as early as possible to prevent gravitational mass movement which could degrade rock mass properties rapidly and cause problems. If the pillar is mined to permanent dimensions, then fill is the best method to ensure stability. With little overlying soil or water bodies, an appropriate bolting method can suffice when the mass is not critically broken. The support method should also be evaluated in the light of future mining activity to recover or reduce pillar thickness.

Extensive CANMET surveys have indicated that so far, most Canadian mines have dimensioned their pillars arbitrarily and conservatively, based on "personal" experience rather than using an engineering process and methods based on sufficient rock and geotechnical data. Upon recognition of further pillar ore extraction need or instability, detailed investigations were started. Few went beyond some preliminary data gathering and limited analysis; considering only some geomechanical characteristics was also common (14).

There exists certain restrictions associated with these rock structures that should be remembered. The emplacement of surface installations directly above a possible surface crown pillar is not recommended. Precluding the possibility of major failures, or subsidence and the problems it can pose for machinery with low displacement tolerances, a future need to recover the pillar would justify this decision. In many cases, keeping an open and inquisitive mind to potential geotechnical or mining problems attached to such a pillar is of crucial importance. Becoming familiar with the parameters necessary for design and means to acquire them early will reduce the impacts of unexpected problems and will be profitable in the long run.

3.4 Data Analysis

The purpose of this step is to place in proper perspective all of the geomechanical elements required for the safe dimensioning of surface crown pillars and stability of abutments. Beyond providing a clear picture of site conditions, the information will be assembled into characterizations required for design methods/dimensioning and support.

Specifically, the results of steps 3.2 and 3.3 should be used to map the structure in terms of areas having similar engineering properties and behaviour pattern. The method of portrayal to obtain a proper understanding of the physical nature of the problem can consist of plans, sections (longitudinal and transverse), 3-d views, graphic simulations and physical models. It is relatively easy to couple borehole geomechanical data (RQD, rock mass classification, strength, deformational properties, joint family distribution) with geology and mine sectors data, so that underground openings and mining methods can best take advantage of ground conditions.

Information on the following rock mass parameters is required for putting pillar characterization into perspective:

(a) discontinuity pattern and connectivity

block distribution

areas of weakness

(b) rock mass classification

(c) mass strength, elasticity

joint behaviour (Mohr-Coulomb, Barton-Bandis...)

(d) water infiltration

(a) Discontinuity pattern and connectivity

In a potentially low stress environment such as surface crown pillars, failure is expected to be controlled by structure rather than stress (one exception, the pillars separating open pits from underground workings where stress has to be taken under consideration). Faults, shear zones, schistocity, may occur to affect the stability. Joints are ubiquitous in rock, possibly extending tens of meters (master joints), meters (major joints), tenths of meters (minor joints), or smaller. The following outline will indicate the extent, orientation and characteristics of structural discontinuities and their effect on the stability of an existing mass.

Ordinarily, it is the arrangement of larger sized discontinuities that present problems. Their nature and their spatial disposition are described by characteristics.

i) spacing

ii) persistence

iii) number of sets

iv) orientation

The first three will determine block sizes within a rock mass and blocks of the mass, the last two their shapes.

It is worthwhile to examine three-dimensional geometric relationships of joints using stereonet contoured projections to plot faults, bedding, joints and shears with distinct markings in order to obtain a good evaluation of their influence. These discontinuities must be reflective of the 3-dimensional pattern. Since it is often difficult to obtain preliminary surface or underground joint measurements, rock core from various drill hole orientations will suffice until underground excavations can update the information. Structural failure conditions can be detected using the stereonet orientations of the major pole groupings. Various authors (15)(16) outline methods of stereonet analysis to detect potential failures. The former outlines methods which identify conditions leading to roof wedge sliding, gravity falls and sidewall wedge sliding, etc. The method is based on the key block principle, i.e. examining for the few blocks on the surface of an excavation that are critical to movements of any larger mass of rock.

Relative to the engineering structures under consideration, unstable conditions or excessive deformations tend to prevail when close spacing of unfavourable orientated weaknesses occur. Low interblock shear strength, provided by low joint roughness, soft joint infilling and opened joints, weaken the interlock and compound the orientation problems.

Another means of characterizing the joint data is to use the joint lengths (persistence) measured at different exposures, the values for each family plotted on log-log diagrams of lengths versus cumulative percent. There will be a linear relationship for each family, but normally a different slope. If observations on exposures of different orientation do not follow a straight line for the same family, the shape of the discontinuity is not circular or square (17).

Block size, qualified by means of average dimensions of typical blocks, is an indicator of mass behaviour. The combined properties of size and interblock shear strength can provide an overview of the extent to which the mass can deform without causing failure of the intact rock. When composed of large blocks, it tends to be less deformable and develops favourable arching and interlocking.

(b) Rock mass classifications

Several uses can be made of an empirical rock mass classification. They can be used by themselves in certain cases to plan the opening dimensions and support. Such a method will aid the design of all mining structures by crystallizing the interactive effects of several key elements and will produce a setting of various areas of distinct geomechanical properties, placing parameters into their exact perspective. It will sharpen computer modelling and analytical solutions when such studies are undertaken.

Various qualifiers are introduced to incorporate the effects of crucial parameters. Two of the popular systems were developed in hard rocks. One, the NGI system (8), yields a numerical index value, Q, based on number of joint sets, joint roughness, joint alteration, reduction factor for the presence of water on joints and reduction factor for stress level:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

The first quotient in the Q equation yields a rough estimate of block size, the second indicates frictional characteristics, and the third evaluates the effect pertaining to rock stress. Jr, Ja and Jn can be estimated from core logs or field surveys, Jw from field surveys. Tables and diagrams containing several different values paired with descriptions of each parameter are provided for selection of correct index factors and to choose from an outline of dimensions and support recommendations for the excavation.

The second system, the CSIR system (18), is based on:

1. Intact rock strength
2. RQD
3. Joint condition
4. Joint spacing
5. Groundwater condition
6. Orientation of joints around the opening

From a table of parameters and their ratings, individual ratings are obtained for 1-5 and then added; an adjustment based on discontinuity orientation is then made. Support predictions are also possible.

If rock core is unavailable to estimate RQD, Hoek and Brown (15) suggest using this approximation:

$$RQD = 115 - 3.3Jv$$

where $J_v = joints/meter^3$

(c) Mass strength, elasticity

The first step in understanding the mechanical behaviour of a rock mass is determining the nature of the discontinuities. Besides studying the geometrical relationships as previously outlined, the characteristics and behaviour of the blocks created by them are also pertinent. The mechanical behaviour of a jointed rock system is dependent on the following factors:

1.- The mechanical behaviour of the individual elements constituting the mass.

2.- The operating stress field.

3.- The configuration of the mass and its strength properties.

4.- The sliding characteristics of joints.

1.- Individual elements such as blocks can be considered as intact (unbroken) material. Lab tests on samples (altered or not) can thus provide the necessary information.

2.- As mentioned earlier, lateral stress conditions near surface may be too low to contribute greatly to stabilizing a pillar; gravity loads should be considered more important. The two exceptions are cases separating pits from underground workings and the cases where gravitationally induced stresses are present. Ordinarily, the value of horizontal stress produced by gravitational vertical stress is:

$$\sigma_h = \rho g z \frac{\nu}{1-\nu}$$

where σ_h= horizontal stress, ρ=rock density, g=gravity constant, z=depth, ν= Poisson's ratio.

This is for an elastic isotropic medium, not taking tectonic stresses into consideration (the horizontal stress is thus never greater than vertical stress in this case because ν is less than 0.5).

The difference in stress levels for an "orthotropic" mass (considerable elastic properties variation in two or more directions at right angles to one another) depends on the degree of anisotropy; if a medium shows different response in the horizontal (x-y) plane the resultant principal stresses along the direction of anisotropy become:

$$\sigma_x = \rho g z \frac{(\nu_{xz} + \nu_{yz}\nu_{xy})}{1 - \nu_{xy}\nu_{yz}}$$

$$\sigma_y = \rho g z \frac{(\nu_{yz} + \nu_{yz}\nu_{xz})}{1 - \nu_{xy}\nu_{yx}}$$

In the case of a simple, regularly jointed rock mass, a recent theoretical investigation has indicated that significantly higher horizontal than vertical stress can be caused by gravity in anisotropic rock masses (19). This anisotropy would originate from discontinuities such as joints. With one persistent horizontal joint set of constant spacing, S, the horizontal stress component induced at depth z becomes:

$$\sigma_h = \rho g z \frac{\nu}{1-\nu} + \frac{E}{S} V_m \frac{\nu}{1-\nu}$$

where V_m= maximum joint closure after normal loading

(assuming the same Poisson's ratio values in the horizontal rock unit as the one normal to it)

Fig. 3 shows, respectively, the effect of a range of joint spacings on horizontal to vertical and the horizontal stress with depth.

In a rock mass with a vertical, persistent joint set (parallel to the yz coordinate plane) two equations relate the induced horizontal stress components σ_x, σ_y to the intact rock and joint set properties:

$$\sigma_x = \rho g z \frac{\nu(1+\nu)V_m k_{ni}}{(1-\nu^2)V_m k_{ni} - \nu^2 \frac{E}{S} V_m}$$

$$\sigma_y = \rho g z \frac{\nu + \nu^2(1 + \frac{E}{k_{ni}S})}{1 - \nu^2(1 + \frac{E}{k_{ni}S})}$$

where k_{ni}= joint normal stiffness

Fig. 4 shows the influence of joint spacing on the respective horizontal stresses to vertical stress with depth as well as the effect of joint spacing on the two horizontal stresses.

Fig. 3-4 show that continuous joints can have a strong effect on the near surface gravity induced stress field. The clear effect of smaller joint spacing is to dramatically increase stress from surface to 100 m for these two joint settings. This depth range is representative of surface crown pillar settings. Since the rock mass is laterally restrained, this build up in stress can conceivably occur if lateral strains are not taken up by joint closure. The implications are that for decreasing joint stiffness and/or spacing, the blocks at the bottom of the pillar will be under confinement, retarding some types of gravity falls and slides.

3.- Rock mass behaviour intuitively should, and does, clearly vary from place to place, no more so than for mechanical behaviour where factors may vary within the dimensions of a surface crown pillar. Mechanical properties of rock masses are difficult to determine compared to intact rock and are better inferred from on-site characterization. Any approach to design under such conditions must take into account the nature of the mass anisotropy and its discontinuity properties. As such, the fundamental concepts of rock engineering in such a setting can be summarized as follows (7):

i) For most of the rock engineering problems, the engineering properties of a rock mass depend far more on the system of geological separations within the rock mass than on the strength of the rock material itself.

ii) The strength of a rock mass, together with its anisotropy, is governed by the interlocking bonds of the blocks forming the mass.

iii) The deformability of a rock mass and its anisotropy result predominantly from the displacements of the unit elements composing the structure of the rock mass.

From the geometrical arrangement obtained from discontinuity surveys, the mass configuration with respect to blocks will provide the basis for pillar mechanical behaviour beyond the immediate stope roof problems previously discussed with regard to stereonet analysis. It is unfortunate that the geomechanical literature and mining case studies are void of information on mass mechanical behaviour of similar settings to surface crown pillars, based on field behaviour. For the moment, the behaviour of such structures must be evaluated from small scale tests. Ideally, such tests should address potential failure modes such as block falls (local and progressive), local movements within the mass shearing at the abutments, prevention of mass movement, etc. The reader is encouraged to consult Lama and Vutukuri (7)(17) to obtain an overview of the effects of discontinuity density and orientation on the compressive, tensile and shear strength and deformation characteristics of lab size rock masses.

One approach is to assume that the mass has no tensile strength and that resistance to tensile forces depends on block interlocking with or without sufficient lateral stress. However, in the case of a massive rock, not effectively broken, some rock tensile resistance will be present.

Examining actual ground stability can be achieved by using actual physical model replicas, empirical models, numerical models, observations and limit equilibrium analysis.

The first three are covered in step five, the fourth in step seven. Limit equilibrium analysis may not be best suited for surface crown pillars. Global deformations may not be linear with load; often the deformation may be excessive without failure. The argument of one overall safety factor to account for all uncertainties is also important but may not fit a particular setting. More detailed considerations facing limit equilibrium analysis include the possible deformability and unequal stress distribution within the mass; also, progressive failure of the mass rather than sudden failure is more characteristic.

4.- The contact behaviour of sliding along joints has been the focus of much attention. Several factors are recognized as having an influence on joint behaviour:

i) Rock strength

ii) Loads

iii) Joint roughness

iv) Degree of surface alteration and infillings

v) Water

vi) Joint opening

Several models exist (11)(20)-(28) beside the Mohr-Coulomb criteria:

$$\tau = C + \sigma_n \tan\phi$$

which is a straight line relationship best suited to describe the shear failure of flat rock surfaces; since rock joint shear behaviour is usually a curved relationship, empirical power laws such as the Barton-Bandis model (11) have been used to fit such data:

$$\tau = \sigma_n \tan\phi[JRClog_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_r]$$

where σ_n= normal stress, τ= shear stress at failure, JRC=joint roughness, 0-20 smooth to rough, JCS=joint wall compressive strength, ϕ_r= residual angle of friction

(d) Water infiltration

Such action can create problems. It can wash away joint infilling thereby allowing severe water inflow, freer block movement and transport of fine sized soil particles from the overburden into the upper stopes. One can easily see that if water inflow from surface is possible, only very small block or structure movement will permit an increase of flow.

Secondly, if the jointing system is only slightly opened significant hydraulic pressures can develop within the lower reaches of the pillar.

A theoretical solution can be used to estimate the effective joint aperture required to allow this level of leakage (29).

3.5 Design Methods and Initial/Final Dimensioning

It is evident from the amount of information pertaining to surface crown pillars that dimensioning has to take several aspects into consideration. Here dimensioning is considered in terms of pillar depth, span and width required while keeping in mind the strength properties and failure modes outlined earlier. Two aspects of pillar dimensioning will be reviewed. Firstly, the known, applicable methods will be presented in a sequence that provides progressively more detailed and sensitive solutions. Both empirical and analytical methods are included. Secondly, the aspects of using a factor of safety versus a probabilistics approach will be discussed.

Dimensioning can be performed several times before final dimensions are reached; such is the case in shrinkage mining. Depending on mining strategy, a pillar can be mined initially to temporary or final dimensions and may include final recovery. Though it would be easy to assign arbitrary dimensions to a pillar in an initial step, it is better to complete a thorough integrated approach. Once minimum safe dimensions are calculated, initial dimensioning can take place and subsequent ground behaviour and ground control investigations used to arrive at final dimensions.

Several design methods exist, but in the unique case of surface crown pillars it is wise to consider all concepts, in other words to develop an integrated design. In this context a series of methods simple to complex can be employed. Each level of this analytic chain provides a broad view of design and expectation of behaviour. By themselves, numerical modelling or empirical rock mass classifications paired with local experience could be viewed as sufficient, but the complex nature of the settings that can be encountered (broken-blocky mass controlled by structure, inflow of water, weathering and joint infilling, low lateral stress, special failure modes) predicate the use of a well-rounded and complete approach based on consideration of the following:

(a) Elastic structural members

(b) Non-elastic structural behaviour

(c) Empirical approaches

(d) Numerical models

(a) elastic structural members

The simplest analysis of a surface crown pillar is to consider it an elastic rock beam. All elastic analysis will consider two fixed ends. Under a gravity setting, the distributed load consists of rock, soil (wet weight) and overlying body of water (where it exists):

$$w = \gamma_r h + \gamma_{s1} d_1 + \gamma s2 d_2 + \cdots + \gamma_{sn} d_n$$

where γ= rock density, h=surface crown pillar thickness, γ_{sn}= soil density for each unit, d_n= average thickness of each soil unit

To establish dimensions according to this theory, the rock mass is assumed to have tensile capabilities. The tensile stress occurring where the bending moment is maximum (top of beam at abutments) per unit width of the beam according to the elastic beam theory is:

$$\sigma_t = \frac{wL^2}{2h^2}$$

It is evident that a small decrease in beam thickness or increase in span will greatly increase the tensile stress imposed. The tensile value of the rock used to assess the structural stability of the real pillar should be obtained from large laboratory beam type tensile tests.

The minimum thickness required just to prevent tensile failure is (factor of safety of 1)

$$induced < lab$$

or:

$$\frac{(\gamma_r h + (\gamma_{s1} d_1 + \gamma_{s2} d_2 + \cdots \gamma_{sn} d_n))L^2}{2h^2} < lab$$

When the immediate roof is composed of more than one bed or lamination, upper, less rigid beds will be partly supported by the bottom bed. The load on the bottom bed is (30):

$$w_1 = \frac{E_1 h_1^3(\gamma_1 h_1 + \gamma_2 h_2 + \cdots + \gamma_n h_n)}{E_1 h_1^3 + E_2 h_2^3 + \cdots + E_n h_n^3}$$

where h= layer thickness, E= modulus of elasticity and 1 = bottom bed designation

For inclined beds:

$$w_{(inclined)} = w_{(horizontal)} \times \cos\theta$$

where $\theta=$ dip of strata

Plates (beam width greater than 1/2 beam span) do not have the transverse distortions of narrow beams due to plane stress. The circumstances are such as to produce triaxial stress (31). Information on elastic deformation and induced stresses under various loads, plate shapes and support conditions is available for use (32).

When the stope back is deliberately shaped like an arch, the modulus of curvature must be enough to take full advantage of the carrying capacity of the arch. This presents shaping problems and ground control problems in poor conditions. The literature treating arch behaviour under vertical loads takes into consideration elasticity and plasticity, depending on the condition of the material used (33). In this section elastic behaviour will be considered; in the next, plastic behaviour will be addressed. For circular fixed arches, the uniform critical load is (34):

$$w_{cr} = \gamma_c \frac{EI}{R^3}$$

where $\gamma_c=$ compressive force factor, L= chord span of the arch, E= modulus of elasticity, I= cross section moment of inertia.

Table 3 provides values for the factor under different rise to span ratios. The approach here is simplified; only a constant cross section-arch is considered to contribute support rather than all of the pillar.

A beam subjected to high lateral stresses can behave as a beam-column. These types of structures are slender, axially loaded and subject to bending. The possible instability of the loaded system, buckling, occurs when span to thickness is large, usually greater than 3. For a structure with fixed ends, the moments at the abutments and at the center are as follows (30):

$$M_{abut} = -\frac{wL^2}{12} \cdot \frac{3(tanv - v)}{v^2 tanv}$$

$$M_{center} = \frac{wL^2}{24} \cdot \frac{6(v - sinv)}{v^2 sinv}$$

where $v = L\sqrt{\frac{3p}{Ed^3}}$, $p=$ axial load per unit depth of beam

The negative sign of the first equation indicates a concave down bending. The stress values of the beam are then calculated by using the elastic beam theory.

(b) Non-elastic structural behaviour

The basis for this analytic approach is to consider the mass as composed of a number of discrete blocks separated by vertical joints which are incapable of supporting tensile stresses.

Pender (35) has carried out such an analysis to model a closely and pervasively jointed rock beam. Tension in this approach is resisted by prefailure joint dilatancy. Consideration of the stresses in individual blocks is neglected (the case for strong unweathered rock). Subjected to gravity loading only, a horizontal beam consisting of an even number n of discrete blocks of equal size is considered, Fig. 5. The length of each block is s, the thickness d, with weight w. It is assumed that there is some moment restraint and shear displacement is allowed. The shear force distribution is independent of support fixity (eg. simply supported, fixed).

At mid span the forces at the top and bottom of the beam are:

$$(F_{nt})_{midspan} = \frac{wn}{8}\left\{\xi\left(\frac{n+2}{n+1}\right)\left(\frac{k_n}{k_s}\right)+\frac{1}{2d}\right\}$$

$$(F_{nb})_{midspan} = \frac{wn}{8}\left\{\xi\left(\frac{n+2}{n+1}\right)\left(\frac{k_n}{k_s}\right)-\frac{1}{2d}\right\}$$

where ξ= slope of the normal vs shear displacement trace of joint shear tests, k(n), k(s) = normal and shear stiffness of each joint from shear tests (k(s) will vary with normal load imposed, k(n) increases with successive normal closure/opening cycle (9))

It must be stressed that the shear tests must be carried out using suitably representative sample sizes and normal loads.

A value for beam thickness should be chosen so that the subtraction in the bottom chord bracket remains greater than zero, that is compressive, to avoid tensile stress in a no tension mass.

Plastic behaviour in a well-interlocked mass of blocks can use masonry arches as design basis. Such a structure assumes that

- stone has no tensile strength
- stone has infinite compressive strength

- sliding failure cannot occur; friction is high enough or the stones are effectively locked

The collapse of an arch must be viewed as a geometrical problem rather than as a problem of strength of materials. If a uniform distributed load is imposed on a suitably dimensioned arch, it will be stable until the rock strength is surpassed. Heyman (36) uses the ideas of plastic theory developed for steel frames for application to circular masonry arches. The equilibrium condition will be satisfied by constructing a line of thrust in equilibrium with the loads acting on the arch (Fig. 6). The safe theorem of plastic theory states that to demonstrate arch stability (tensile stresses are avoided between the blocks), the thrust lines must be constructed in equilibrium with all the loads acting on the arch and lying wholly within the masonry. The assurance of any one line of the pair for a point load lies within the masonry gives complete assurance the arch will not collapse. The distributed load can be considered as a sum of several point loads around the circumference.

This limiting state indicates that an arch thickness to radius ratio, Fig. 6, is required. The geometrical problem is to determine the cross sections at which the thrust line intersects the interior portion of the arch. The required thickness to arch radius ratio for minimum stability is:

$$t/R = 1.06$$

The same author (40) has calculated t/R for arc angles less than 180°. By decreasing R (and therefore L) and maintaining the asame outer diameter, the thrust line will lie within the arch rather than the limiting state.

A second arch method to analyse a broken rock mass considers an arch made up of boulders or untightly matched blocks (31), Fig. 7. Failure is expected to occur in one of three modes:

- high loads opening the spaces between blocks permitting them to fall
- crushing of small areas leading to the possibility of freer movement
- blocks slipping out of the arch from low frictional resistance. Arch failure by slippage has been analysed by the same author. Based on an arch rise of d, the horizontal reaction at each abutment is:

$$R_h = \frac{wL^2}{8d}$$

When slip occurs, the vertical component Rv of the reaction is:

$$R_v = R_h \tan\phi \qquad or \qquad R_v = \frac{wl^2}{8d}\tan\phi$$

where ϕ= material angle of friction, w=weight per unit length, L=arch span

R_v is the shear strength that the abutment offers to resist the actual loading of wL/2. If the actual loading is greater than the abutment resistance, failure will occur.

In well-bedded rock, or instances of persistent jointing or planar fabric weakness, design curves for roofs and hangingwalls based on the voussoir beam and plate solution have been established by Beer and Meek (37). The voussoir theory assumes that the beam consists of a no tension material which carries its weight by arching, which models well the separation and tensile cracking of a well-bedded rock from excavation, Fig. 8. This theory can be used to predict the collapse of such a mechanical system, figures 9-11.

(c) Empirical methods

Here these are divided into two groups: those based on a substantial amount of data and those based on limited observations.

Barton et al (8) developed a fully integrated system to establish full support opening requirements in relation to opening dimensions. This system uses the classification system outlined earlier. Though not providing indications on an estimate of pillar thickness or width, stability of opening with chosen span is possible.

The second group includes local personal experience and consideration of other similar cases. Local experience, which is normally available after a mining project is well advanced, can provide an understanding of failure modes and opening size possibilities, but not to be used with complete reliance. This is case for the arbitrary approach of dimensioning the pillar to a specific thickness to width ratio to avoid a wider than thicker beam which by intuition and the elastic beam theory "bends more". This is an oversimplification which does not consider pillar span or rock quality. When the distribution of the thickness to width ratios for 132 pillars of 23 mines is reviewed (14) (based on mass quality determined from RQD, empirical rock mass classification, structural geology problems and hangingwall/footwall problems) it is evident that there is little difference between the ratio distribution under various types of terrain, Fig. 12.

(d) Numerical Models

The previously listed tools for engineering calculations are limited in their ability to fully address the scope of surface crown pillar design; they can relate to stresses and dimensions but lack the capability to describe in detail variations in stress and displacements throughout the pillar and adjacent rock zones. Numerical modelling can provide for this response while incorporating a sound geomechanical approach to the entire mass. Modelling can be divided into two approaches: A continuum approach, testing the mass as a continuum intersected by a number of discontinuities, or a discontinuum approach, regarding the mass as a group of independent blocks (38).

The differential type of continuum models, including boundary and finite element techniques, characterize the entire region of interest. Boundary element types feature discretization only along interior or exterior boundaries. The interface between different material types and discontinuities are treated as internal boundaries which must be similarly discretized. Boundary element procedures are most apt for modelling linear, homogeneous elastic systems, although certain forms of non-linearity may be treated. They provide economic means of two- and three-dimensional rock mass analysis. They are particularly suitable for use when conditions at the boundary are of most concern (38).

The finite element method is well suited to establish the possible fracture and weakness zones adjoining deep openings, by utilizing suitable failure criteria. Irregular geometries, non-uniform materials and non-uniform loadings can be addressed. Non-linear behaviour is also addressable.

Though this technique is useful in comparing the suitability of different conception scenarios, it does suffer from several drawbacks in the case of surface crown pillars. It will be difficult to simulate low stresses in the pillar, particularly localized gravity failures and water effects. Cases of high stress concentration have already applied finite element modelling successfully (39).

Finite element analysis requires input information. This includes a mesh, reflective of the size, shape and properties of the domain examined. The mechanical properties of each element of the mesh are included. This is usually the density, deformability of the elements along with the strength and stiffness properties of the between elements. The finite element analysis process will provide (16):

i) strains and displacement throughout the model

ii) the stresses and, with difficulty, these can be manipulated to find regions of potential danger

iii) parametric analysis once a model has been set up, to find a suitable shape for an excavation; it cannot provide much help in charting the best direction for the excavation

iv) results based on a specific mesh, with pre-defined directions and spacing of joints. Generic studies can be made only if several meshes are generated

v) numerous and occasionally complex computations (thus necessitating a computer using costly runs, not readily suited in the most complex situations to more widely used microcomputers)

Discontinuum models feature numerical procedures involving the equations of motion of blocks. In the case of surface crown pillars where independent block movements can be specifically recognized, this is an advantage.

Distinct element analysis is an example of a discontinuum model. Where discontinuities are pervasive in a rock mass in a low stress environment, the elasticity of individual blocks can be neglected. Rather grouping of blocks on the basis of the influence of discontinuity stiffness, as described by Cundall (40), is more appropriate. The response to applied load at these relatively large two-dimensional block systems are calculated in time steps taking into account block interactions. In this method, the solution process is based on a force-displacement law specifying the interaction between the blocks and a law of motion which determines displacements induced in blocks by out of balance forces (41). In some recent variants of the Cundall model (42), the rotation and displacements of each block and the resulting collisions with nearby blocks are treated in the model. The blocks can be treated as rigid or endowed with the ability to deform. The analysis can be performed with a microcomputer and displayed interactively. This method is restricted to two dimensions unless very large computers are used. As with finite element analysis, it is still necessary to compute using a pre-determined mesh, with precise locations of all joints.

In using design dimensioning stability equations, two methods for stability determination are available: the deterministic (factor of safety) method and probabilistic approach method.

The design considerations presented in this article deal with deterministic approaches, implying that a certain structure would behave exactly the same if all conditions were applied once or several times. One must remember that the factor of safety arrived at for a structure depends on the calculation hypotheses and the basic input information available. Using a higher level of a factor of safety does not necessarily imply greater security. There always remains the possibility of a failure. In this sense, the probabilistic approach, defined from 0 to 1 is more realistic. This does not mean a safer method, although a probabilistic approach requires more thorough investigations.

It could be argued that the main parameters associated with these pillars are not deterministic. If the examination of the frequency distribution of jointing, block distribution and other parameters yields statistically valid results, a probabilistic approach may be more appropriate. There will be a certain failure probability "P". But so far, there is no experience in using it for surface crown pillars.

3.6 Mining Activity

Admittedly, mining close to surface always has its share of problems: groundwater inflow, ground relaxaton, cave-ins and subsidence. The mining method chosen is included in the design to reflect the operator's ability to make a good prediction of the movements that will be caused by a particular excavation method and how to minimize their effect or keep that effect under certain values imposed by restriction of surface movements such as structure or flowable soil. In reality two major factors will almost always prescribe the method envisaged: ore concentration and orebody dimensions.

This design step includes two approaches: the mining method employed and its effects on the rock and soil surroundings and methods of stabilizing the pillar.

The CANMET survey of surface crown pillars (1)(2), Table 1, has shown that only 5 of 23 mines with surface crown pillars have used shrinkage, a bulk mining method. Under this method, several problems will occur if ground conditions are not stable. Beyond obtaining dilution from unstable hanging- and footwall, storing ore in the stope can lead to oxidation problems if the pillar permits water infiltration. Since the ore left in the stopes provides lateral support, if the amount left is too low, caves-ins and perhaps "chimneying" to surface can occur such as the Belmoral case.

Practical experience has shown that mining in highly fractured ground is possible, even under considerable clay overburden overlain by tailings sludge. The Lakeshore property of Lac Minerals mined in such ground using an undercut and fill method with hydraulic rigs for better control and efficiency. A monolithic concrete beam placed on the floor and pinned to the walls was used to form the back for the next lift. The remainder of the space was filled with cemented sand fill (43).

By placing fill as soon as possible after a stope has been mined (with proper local support during extraction) maximum benefit for the pillar and surrounding area is guaranteed. Mining activity disturbs the initial interlocking of joint surfaces. This decreases the capability of the rock mass to support blocks at the edge of an excavation. When lateral stress is larger than vertical stress, roof blocks can require less support.

When should the requirement for artificial support be decided upon, and how? It is suggested initially when the mining method, opening geometry and mass behaviour are estimated. Revisions can then be carried out during trial excavations or during early mining stages. The total cost of a support system will often take into consideration the critical nature of ground problems. This consideration will not be examined here.

What can be kept in mind is the end result versus costs. It is important to realize that special bolting programs can cost as much as fill which is simpler to employ and better in the case of surface crown pillar stability.

Three factors determine the type of support required in an underground opening apart from cost considerations (44):

1. Type of rock, and its structural features, immediately surrounding the opening
2. Method of stoping (roof control and permanence, controlled collapse)
3. Stress developed around the opening

The purpose of bolting is two-fold: general reinforcement and local roof/block control. General reinforcement in the case of low stressed masses is based on empirical rules rather than rock support interaction calculations.

Analysis of rock support interaction for these pillars vary from deeper excavation considerations. In-situ stresses in most cases are not equal because lateral stresses are absent to possibly very much higher in the case of the Amadei and Savage approach (19). The opening is not circular and spans are very long. The material properties are not linear-elastic. Failure is more likely to be from gravity or mass movement. Block interlocking, arching and bolting to produce arching are required for general stability. It must not be forgotten that the anchorage must be satisfactory and the design for suspension bolting is usually based on the assumption that total force can be carried by bolts which is at least equal to the dead weight of the rock hanging from them.

Bolting fulfills two functions. In simply layered conditions (without much cracking or jointing), bolting (if long enough with sufficient pre-tension) can bind together the layers to form a laminated beam of greater strength (31). Provided by Brady and Brown (41), the support in these laminated rocks is given as:

$$T = \gamma_r D s^2$$

where T= working load on the bolt, γ_r= rock density, D= height of unstable zone and s= rockbolt spacing in both the longitudinal and transverse directions

This method does not take into consideration shear or flexural strength of the strata above the abutments.

Bolting also aids development of the arch that tends to form over rock openings by maintaining tight interlocking. In experiments to substantiate the reinforced arch hypothesis (45), vertical bolts at various spacings were used on blocks formed from orthogonally oriented joint sets. In two cases, 90 and 45 degree orientation to the vertical, the beam created by the bolt tension was stable for bolt spacing to block width of less than 3. When bolt spacing exceeded bolt length, a stable beam could not be created.

Cording et al (46), based on support of underground power stations, found that the equivalent uniform support pressures in the roof and side walls could be calculated as

$$P_i = nB$$

$$P_i = mH$$

where Pi= support pressure required, B = excavation span, H = excavation height, n= 0.10 - 0.25 for the roof, m = 0.05 - 0.12 for the side walls

Another empirical scheme based on previous practice is the Barton et al (8) scheme. These authors proposed 38 categories of support based on the Q or tunnelling quality index, discussed previously in this text. Supplementary notes by Hoek and Brown (15) underlined further ground control considerations to be used with this method. A table and figure yield the prescribed pattern spacing and total measures to be undertaken, etc. The support required to restrain loose rock between the bolts (such as wire mesh) can be analysed using the block arching approach and the following equations:

$$P_v < 0.727\gamma_r s$$

where P_v= the pressure of the loose rock on the mesh, γ_r= the rock density, s = the bolt spacing

and

$$T = \frac{p_v s^2}{8h}$$

where T = tensile force per unit length of mesh, h = probable mesh sag

The stress in this support is obtained by dividing the tension by the cross-sectional area per unit length. Conversely, the required cross-section area can be determined by dividing the force by the permissible stress.

In the case of vertical cement-grouted rock anchors, the pull-out capacity in an orthogonally jointed rock mass (joints 0/90 degrees to bolt) has been empirically established from field and lab tests (47). The procedure is somewhat complex and the reader is left to examine it himself. The reader is also encouraged to read the Choquet bolting guidelines (48) which is an exhaustive work on bolting and the current uses in various ground conditions.

There are some general bolting rules stemming from observation and logic:

Efficient bolting will intercept more than one block and anchor in competent rock beyond a pervasive discontinuity. Bolts should always cross weakness planes at as great an angle as possible to maximize support capability. Beyond strengthening the roof, it will manage to prevent important degradation based on the aforementioned key block principle. For general, systematic support, the bolts should be installed in a regular pattern. Bolts give better reinforcement if they are installed before relaxation starts. The contemporary bolting practices, however, favour putting the support where the problem occurs rather than using a fixed pattern throughout an excavation. The minimum support system is adopted, followed by increased support in areas of greater or special need. Full advantage has to be taken of bolt steel strength; based on an anchorage equal to or greater than the steel strength, the load on the bolt can equal the steel's yield point or its ultimate capacity depending on the nature of the opening. This load should be at most the load produced by the weight of the tributary area of rock.

Using the yield stress value for the permissible stress in a steel mesh may seem conservative, but considering the loss of life and mining costs associated with support rupture and cave-in to surface, a safety factor greater than one (using ultimate strength) is advised. This would also counter the local concentration of load which produces greater than average pressures at some locations.

Other avenues of stabilization exist. The main sources of joint disturbance are blasting and impulsive excavation processes. It is clear that minimizing these, by using perimeter blasting and low impulsive mining methods, will benefit the surface crown pillar. The mining activity itself can be a source of stress concentration. The slower advancing methods such as shrinkage stoping will not cause rapidly imposed stress concentrations such as blasthole and longhole methods.

Dimensioning and excavating should take advantage of existing structural conditions. Geomechanically more stable openings can be made by using the presence of persistent joints oriented parallel to excavation boundaries. In most cases, an excavation periphery can be maintained in a state of compression. Weak rock, however, can bend, buckle or topple into the opening.

Certain precautionary steps can sometimes be taken to avoid ground stability problems. For example, a soil may easily liquefy and flow into the mine after pillar failure. In the case of a potentially hazourdous soil, it can perhaps be removed; if the soil cannot be removed economically, then the water can be removed from it and the pillar area. Injection grouting may help impermeabilize the overburden/bedrock contact to reduce water inflow; with this option, the disruptive effects of the injection pressure must also be considered.

3.7 Monitoring

Monitoring, or surveillance of behaviour, is an essential component of rock structure design. Beyond acting as a means of evaluating stability, monitoring can be used to supply quantitative data about the response of a structure (failures, overall movement, new conditions, etc.) which can be redirected into the design chain for better evaluation of dimensioning of this or the next pillar.

Monitoring can take on several forms which should all be applied to this situation. It must be stressed that monitoring should begin as soon as the rock mass has been disturbed. The methods, applicable to all aspects of ground control, can be divided into three categories:

(a) Visual evaluation

(b) Occasional monitoring instrumentation

(b) Continuous monitoring instrumentation

All monitoring results should be structured and properly logged so that interpretation, such as changes in time, mining activity, etc. can be made.

(a) Visual evaluation

Examining direct (block falls, water inflow, etc.) or indirect evidence (sheared bolts, drill hole closure, etc.) on a regular and mine-wide basis is the backbone of any monitoring program.

The key to this program is the knowledge of site personnel involved. Bieniawski (38) has outlined the minimum requirements for site personnel:

i) Good general knowledge of the tasks in which they are involved and realization of the importance of measurements/observations.

ii) Knowledge of the general features of the excavation being built.

iii) Knowledge of site administration.

A trainee should be introduced to the following items:

i) A thorough understanding of rock mechanics measurements.

ii) The function of the instruments to be installed and their practical handling

iii) Trial installation

iv) Registering of data, observations

v) Frequency of data recording

(b) Occasional monitoring instrumentation

Periodic monitoring instrumentation is used to measure seasonal variations in water inflow, stope closure and subsidence. In the case of the latter, surface surveying is used. For underground roof/floor closure measurements, convergence measuring tapes record the change in distance between two points. A reading is taken by connecting the read-out unit and the tape to the point of interest. However, these are relatively fragile instruments and dependent on operator technique.

(c) Continuous monitoring instrumentation

There are many instruments available to perform continuous measurements. Borehole extensometers measure the displacement inside the rock mass and provide information on the behaviour and extent of the zone of loosened and fractured rock around an opening. The reference is to a fixed point within a mass (unaffected by excavation) or relative displacements of selected points within the mass along the length of a borehole. Remote monitoring is possible. Extensometers are relatively expensive instruments and easily damaged if unprotected.

Another instrument used to measure stope closure is the convergence meter. It is less accurate than the extensometer but simpler to read and less expensive. In this system, two tubes, one inside the other, are used to produce a telescopic action from which relative displacements can be measured on a graduated scale.

Microseismic monitoring can be used to obtain an estimate of the stability of a rock mass. But this method is not recommended. In a broken altered mass a signal would be rapidly attenuated; the low energy of the signal emitted from mass movement would also preclude the method. These drawbacks are more accentuated for overburden. The method in general requires costly equipment and constant interpretation of data and attention.

3.8 Back Analysis

This is a design method that depends on failures and the conditions causing failure as indicators of design appropriateness/accuracy by reviewing the calculations, inferences, in the light of new data. It is often used in slope stability and pillar strength determinations.

Applications to surface crown pillars stress relaxation and block falls, indicating that there is not enough compressive stress at that area of the opening; on a larger scale can indicate the concentration/reduction in stress and strength and variations in parameters. But the analysis is limited by not knowing the actual level of stress/load that existed at the time of failure nor what part of the spectrum the strength of a particular pillar represents (31).

3.9 Recovery

Extraction of the surface crown pillar can be made at any time once the uppermost stopes are finished. Several methods of extraction have been used. This includes block caving methods, recovery by open pit over backfilled stopes (at the bottom of a pit or at surface) or from surface by blasting into open stopes. There is a need to apply geomechanical factors (rock mass characterization, mass strength/elasticity, stress domains, failure modes, monitoring, effects of the mining method, etc.) to the main recovery steps: pre-investigation, application of geomechanical and numerical models and behaviour during/after recovery. Removal of the pillar is not without side effects. The stresses that were carried by pillar will be redistributed to the footwall/hangingwall and nearby excavations, nearby underground openings, pit slopes and conversely the effect of underground excavation on these is also possible (49).

3.10 Design Evaluation

It is important to critically summarize and evaluate the completed design for application in another portion of the mine site or new project. The original data, assumptions, requirements and constraints, on the basis of a clear understanding of all interacting factors, will tell if any deficiencies, deliberate or not, should have been addressed or if major mining strategy decisions should have been made differently.

At this stage, reporting to other professionals the findings of this evaluation will be helpful to the Canadian mining industry.

4.0 FUTURE RESEARCH

The most efficient method of carrying out research on surface crown pillars is to address the fundamental field aspects that are yet not fully described in terms of behaviour, values, etc.

The exact locations of critically oriented/spaced discontinuities, their relationships as well as areas of weakness are essential to maximize the effectiveness of the dimensioning and stability approaches. So far no method exists to situate exactly in 3-D such features. New methods, tomography and seismic attenuation, offer promise in this respect. The methods are potentially applicable by using surface outcrops or boreholes.

A second aspect yet to be addressed is the instrumentation leading to the establishment of global behaviour of well-described cases.

The basis of developing strength within these pillars, considering arching, shear or other modes of interaction, depends on the condition of the pillar-abutment contact for load transfer. To measure the effectiveness of each of these approaches and the stability of this area, special monitoring could be established under intense mining activity and well known structural conditions. Large scale physical model behaviour could be used in that sense as well.

Water effects on such masses also need careful consideration. From the point of view of stability given water inflow, the pressure and removal of infilling aspects could be reviewed. The modelling of water effects in numerical models could also be studied. From the point of view of waterproofing, the aspects of grouting joints, with or without overburden, on the reduction of water inflow is another interesting topic.

One other subject that needs to be addressed is what kind of support can be used in badly broken/ weathered ground without interfering with mining and the development of improved mining methods to minimize excavating influence on surrounding rock.

REFERENCES

1. Surface Pillars, Contract Report # 26sq.23440-3-9005, CANMET, 1984.

2. Eighteen Surface Crown Pillar Case Studies, Contract Report # 26sq.23440-5-9014, CANMET, 1985.

3. Gyenge, M. and Coates, D.F. "Suggested method on field deformability determination using plate load test (surficial loading)"; Division Report MRP/MRL 74-17 (INT); CANMET, Energy Mines and Resources, Canada; 1974.

4. de la Cruz, R.V., Karfakis, M. and Kim, K. "Analysis of displacement and strain data for the determination of the in-situ deformability of rock masses"; AIME Annual Meeting; 1982.

5. de la Cruz, R.V. "Multiple measurement method of determining the in situ deformability of rock masses"; Proceedings Second International Conference on Stability in Underground Mining; pp. 235-250; 1984.

6. Grummich, J. and Lechasseur, P. "Développement et utilisation d'un obturateur triple avec capteur de pression pour déterminer les paramètres hydrauliques du rocher"; Proceedings 39th Canadian Geotechnical Conference; 1986.

7. Lama, R.D. and Vutukuri, V.S. "Handbook on mechanical properties of rock"; Trans Tech Publications; III; 405 pp.; 1978.

8. Barton, N., Lien, R. and Lunde, J. "Engineering classification of rock masses for the design of tunnel support"; Rock Mechanics; 6; pp. 189-236; 1974.

9. Barton, N. and Bakhtar, K. "Description of the Barton-Bandis joint model"; CANMET report for contract #03SU.23233-2-1478, 1983.

10. Bétournay, M.C. and Shimotani, T. "Triaxial properties of Indiana limestone"; Division Report MRP/MRL 83-99 (TR); CANMET, Energy, Mines and Resources, Canada; 1983.

11. Ladanyi, B. and Archambault, G. "Simulation of shear behaviour of a jointed rock mass"; Proceedings of the 11th U.S. Symposium on Rock Mechanics; pp. 105-125; 1970.

12. Benoit, J. and Clough, G.W. "Self boring pressuremeter tests in soft clay"; J Geotech Eng; 112 #1; pp. 60-78; 1986.

13. Lafleur, J., Giroux, F. and Huot, M. "Permeability testing of the weathered Champlain Clay crust"; Proceedings 39th Canadian Geotechnical Conference; 1986.

14. Bétournay, M. "Summary of 24 surface crown pillar case studies"; 89th Canadian Institute of Mining and Metallurgy Annual Meeting; presentation 44; 15 p.; 1987.

15. Hoek, E. and Brown, E.T. "Underground excavations in rock"; Institution of Mining and Metallurgy; 527 pp.; 1980.

16. Goodman, R.E. and Shi, G. "Block theory and its application to rock engineering"; Prentice-Hall; 338 p.; 1985.

17. Lama, R.D. and Vutukuri, V.S. "Handbook on mechanical properties of rocks"; IV; Trans Tech Publications; 515 pp.; 1978.

18. Bieniawski, Z.T. "Geomechanics classification of rock masses and its application in tunnelling"; Proceedings of the Third International Congress on Rock Mechanics; IIA; pp. 27-32; 1974.

19. Amadei, B. and Savage, W.Z. "Gravitational stresses in regularly jointed rock masses, a keynote lecture"; Proceedings of the International Symposium on Fundamentals of Rock Joints; pp. 463-473; 1985.

20. Ladanyi, B. and Archambault, G. "Evaluation de la résistance au cisaillement d'un massif rocheux fragmenté"; Proceedings of the 24th International Geological Congress; Section 13 D; pp. 249-260; 1972.

21. Newland, P.L. and Allely, B.H. "Volume changes in drained triaxial tests on granular materials"; Geotechnique; 7; pp. 17-34; 1957.

22. Cundall, P., Marti, J., Beresford, J., Last, P., Asagian, M. "Computer modelling of jointed rockmasses", Dames and Moores report for the U.S. Army Engineer Waterways Experiment Station, Vicksburg; 396 pp.; 1978. In Goodman, R.E. and Shi, G. "Block theory and its application to rock engineering"; Prentice Hall; 338 p.; 1985.

23. Heuze, F.E. "Dilatant effects of rock joints"; Proceedings of the 4th International Congress Society of Rock Mechanics; pp. 169-175; 1979.

24. Goodman, R.E. "The mechanical properties of joints"; Proceedings of the 3rd International Congress on Rock Mechanics; Volume 1, part A; pp. 127-140; 1974.

25. Walsh, J.B. and Grosenbaugh, M.A. "A new model for analysing the effect of frac-

tures on compressability"; J Geophy Res; volume 84, #B7; pp. 3532-3536; 1979.

26. Tsang, Y.W. and Witherspoon, P.A. "Hydromechanical behaviour of a deformed rock fracture subject to normal stress"; J Geophy Res; 86, #B10; pp. 9287-9298, 1981.

27. Swan, G. "Tribology and characterization of rock joints"; Proceedings of the 22nd U.S. Symposium on Rock Mechanics; pp. 402-407; 1981.

28. Garnett-Frizelle, C.J. "The shear strength of filled discontinuities in rock"; M.Sc. dissertation, University of Leeds, 1980. In Barton, N. and Bakhtar, K. "Description of the Barton-Bandis joint model"; CANMET report for contract #03SU.23233-2-1478; 1983.

29. Loeset, F. "Geological description of water leakages at the Kjela, Ulla-Fore and Nye Osa Hydropower projects"; Norwegian Geotechnical Institute, Report 540001-s; 1980.

30. S.M.E. Mining Engineering Handbook; American Institute of Mining Metallurgical and Petroleum Engineers Inc; 1; 1046 pp.; 1973.

31. Coates, D.F. "Rock mechanics principles"; Monograph 874; CANMET, Energy Mines and Resources, Canada; 1981.

32. Roark, R.J. and Young, W.C. "Formulas for stress and strain"; Mcgraw-Hill Kongashuka; 624 p.; 1975.

33. Bétournay, M.C. "Arch behaviour: failure mechanisms and observed behaviour"; unpublished paper; 1982.

34. Timoshenko, S.P. and Gere, J.M. "Theory of elastic stability"; McGraw-Hill; 1966.

35. Pender, M.J. "Prefailure joint dilatancy and the behaviour of a beam with vertical joint"; Rock Mechanics; 18 #4; pp. 253-266; 1985.

36. Heyman, J. "The safety of masonry arches"; Int J Mech Sci; 11; pp. 362-385; 1969.

37. Beer, G. and Meek, J.L. "Design curves for roofs and hangingwalls in bedded rock on 'voussoir' beam and plate solutions"; Inst Min Met Trans A; 91; pp. 18-22; 1982.

38. Bieniawski, Z.T. "Rock mechanics design in mining and tunnelling"; Balkema; 272 p.; 1984.

39. Hedley, D.G.F., Herget, G., Miles, P. and Yu, Y.S. "Case history of CANMET's rock mechanics at the Kidd Creek Mine"; CANMET, Energy, Mines and Resources, Canada; Division Report MRP/MRL 79-47(J); 1979.

40. Cundall, P. "A computer model for simulating progressive, large scale movements in blocky rock systems"; Proceedings International Symposium on Rock Fractures; 1971.

41. Brady, B.G.H. and Brown, E.T. "Rock mechanics for underground mining"; Allen and Irwin; 528 p.; 1985.

42. Taylor, L.M. "Blocks: a block motion code for geomechanics studies"; Sandia National Laboratories Publication Sand82-2373.

43. Canadian Air Comments, Atlas Copco Canada Inc.; 28 #4; 1985.

44. Thomas, L.J. "An introduction to mining"; Menthuen; 471 p.; 1978.

45. Coates, D.F. and Cochrane, T.S. "Development of design specifications for rock bolting from research in Canadian mines"; CANMET, Energy, Mines and Resources, Canada; 1970.

46. Cording, E.J., Hendron A.J. and Deere, D.U. "Rock engineering for underground caverns";Proceedings Symposium on Underground Rock Chambers; pp. 567-600; 1971.

47. Dados, A.T. "Design of anchors in horizontally jointed rock"; J Geotech Eng; 110 #11; pp. 1637-1647; 1984.

48. Choquet, P. "Guide d'utilisation du boulonage"; Centre de Recherches Minérales, Gouvernement du Québec; 268 p.;1987.

49. Bétournay, M.C. "Éléments géomécaniques de récupération de piliers de surface"; Compte Rendu du 3ième Colloque sur le Contrôle de Terrain, Association des Mines de Métaux du Québec; 38 p.; 1987.

FIGURE 1. DEFINITION OF A SURFACE CROWN PILLAR

"SURFACE CROWN PILLAR": A ROCKMASS OF VARIABLE GEOMETRY, MINERALIZED OR NOT, SITUATED ABOVE AN UPPERMOST STOPE OF THE MINE, WHICH SERVES TO PERMANENTLY OR TEMPORARILY ENSURE THE STABILITY OF SURFACE ELEMENTS. (1)

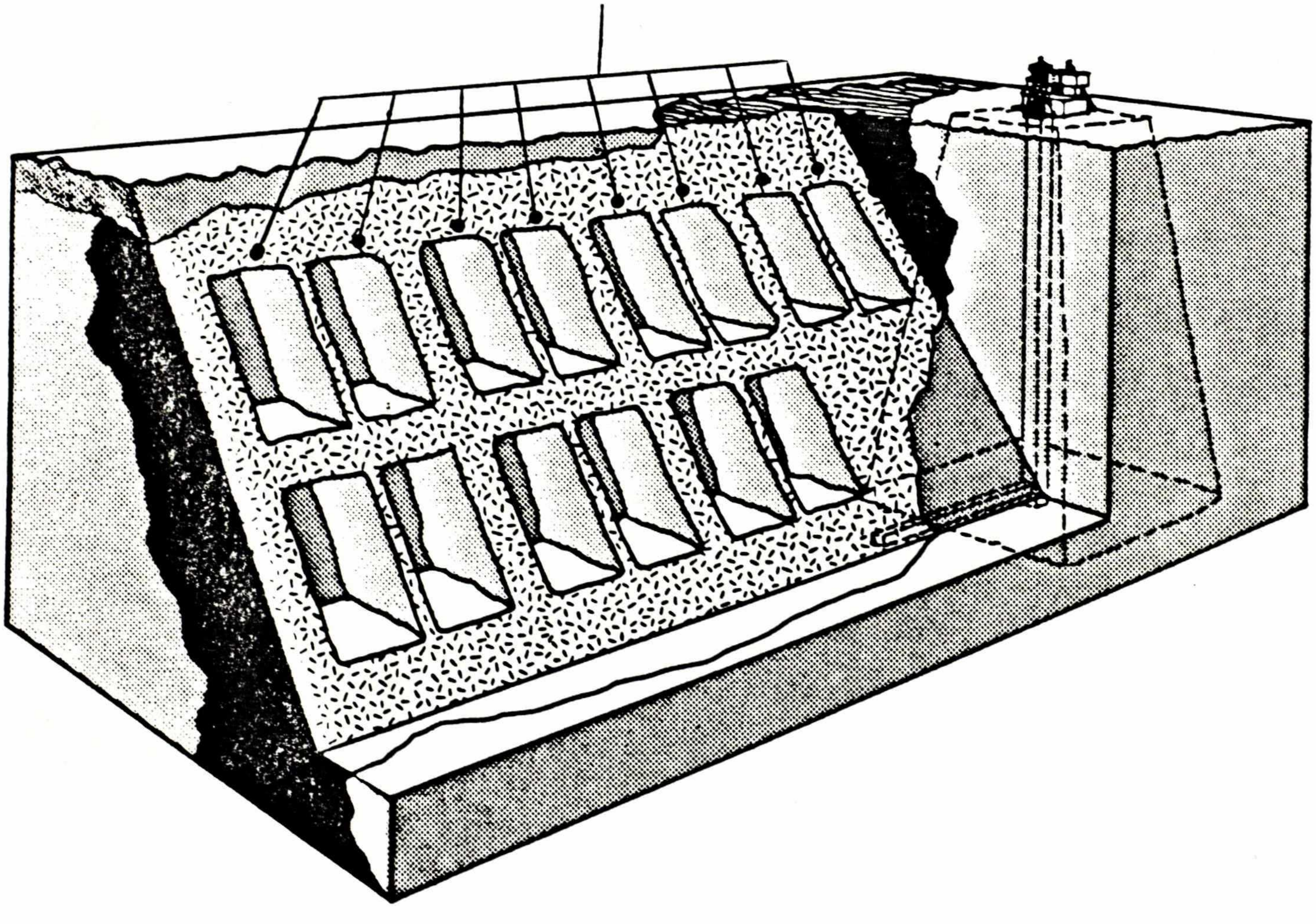
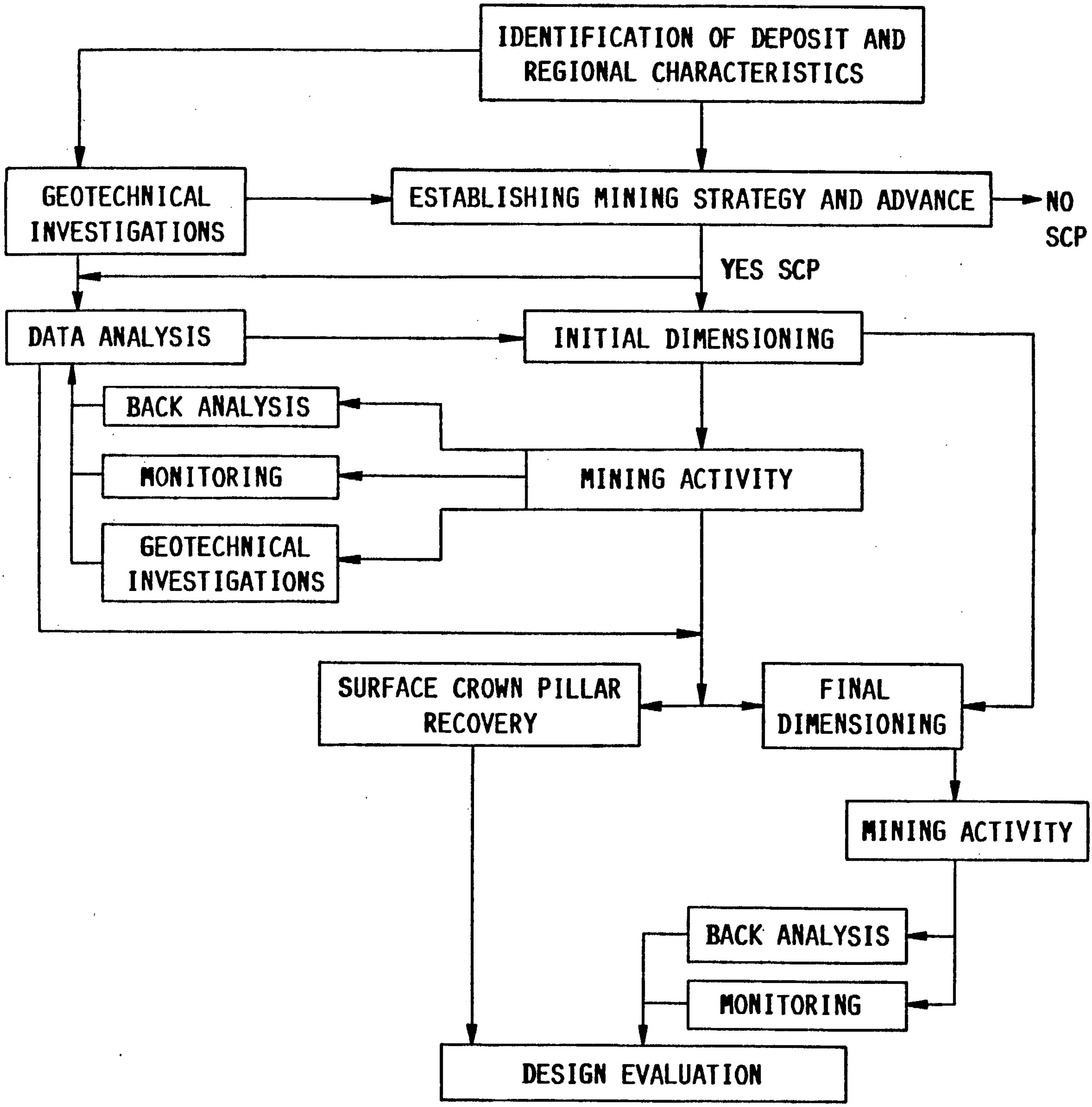


FIGURE 2

SIMPLIFIED FLOW CHART OF SURFACE CROWN PILLAR DESIGN



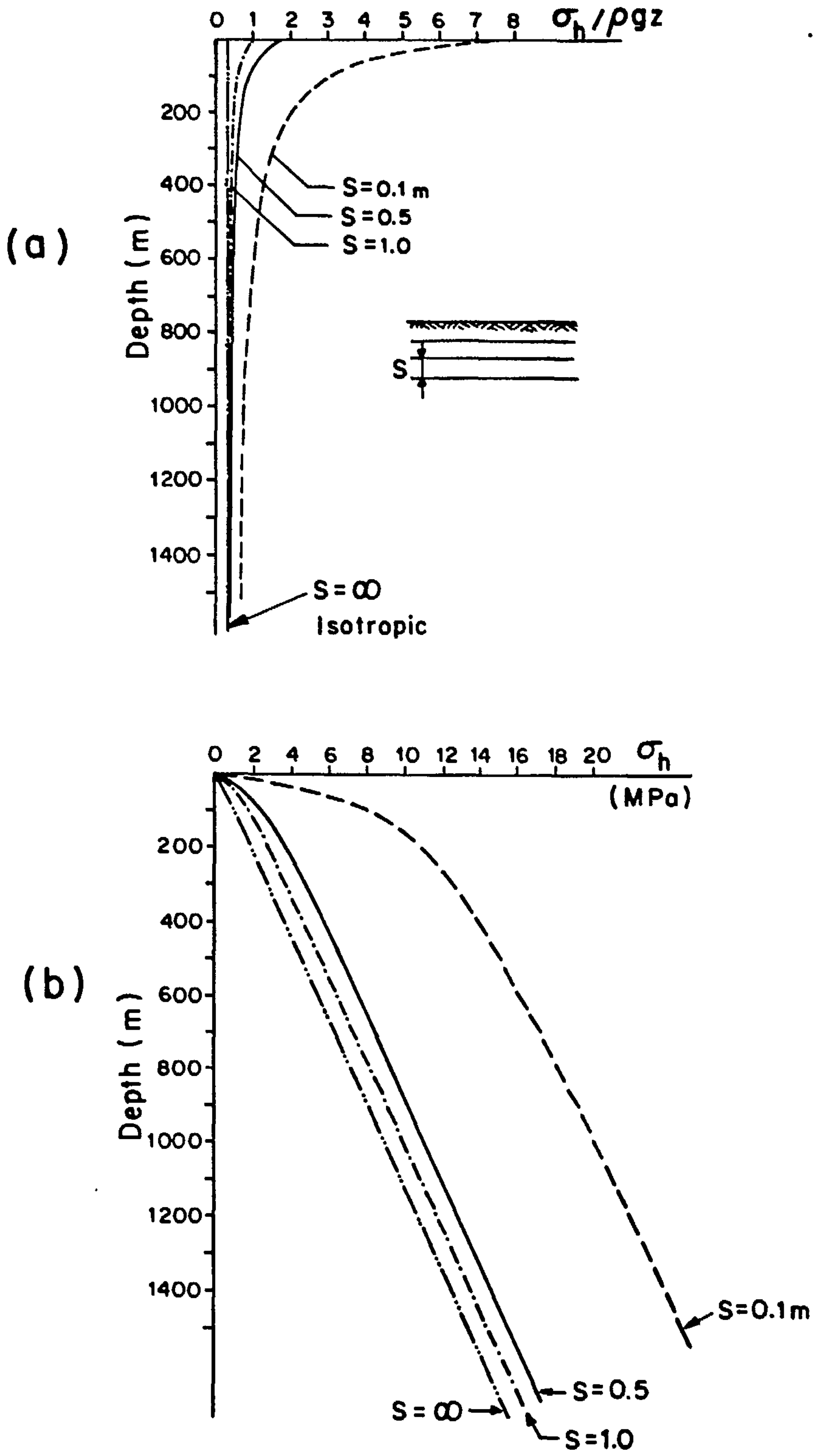


Figure 3. Rock mass with horizontal joints (19).
(a) influence of the joint spacing on the variation of the $\sigma_h/\rho gz$ ratio with depth
(b) influence of the joint spacing on the variation of σ_h with depth

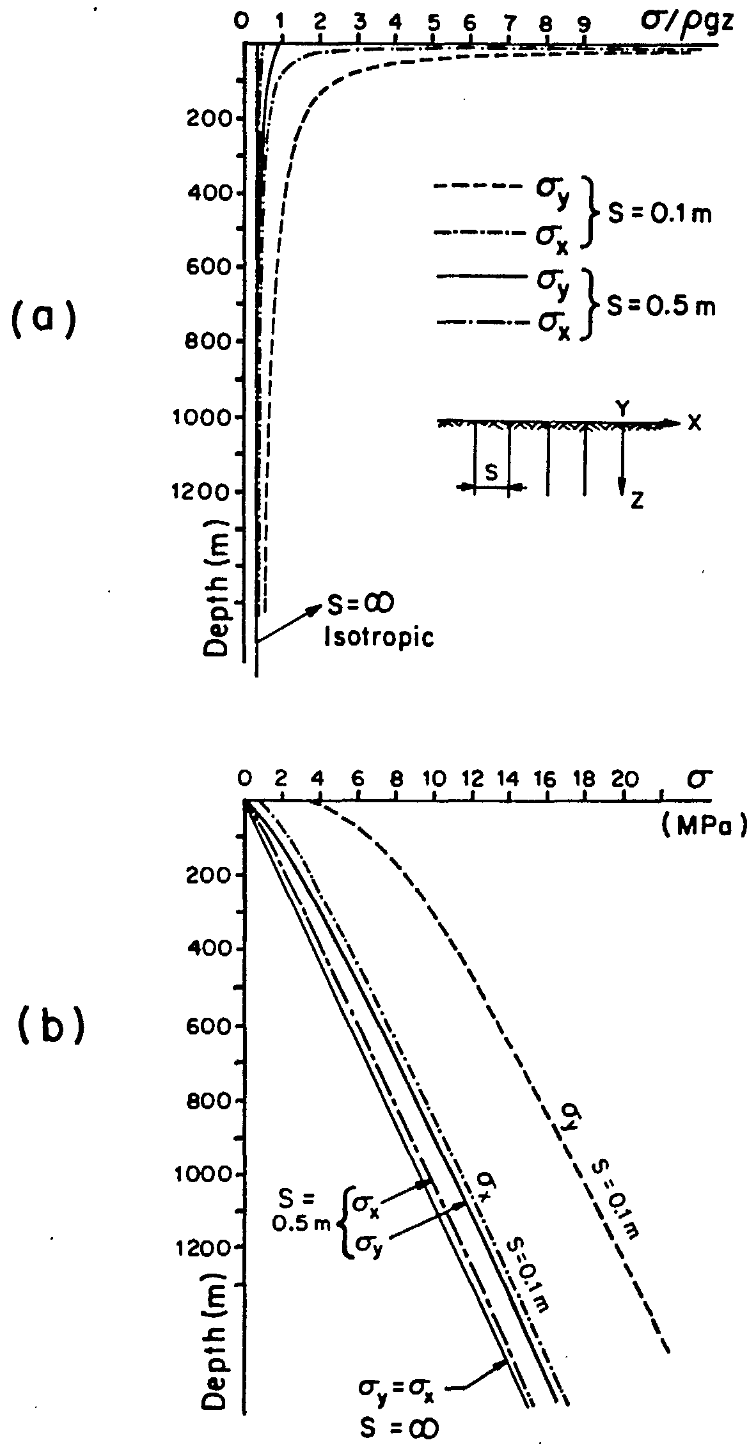


Figure 4. Rock mass with vertical joints (19),
(a) influence of the joint spacing on the variation of the ratios $\sigma_x/\rho gz$ and $\sigma_y/\rho gz$ with depth
(b) influence of the joint spacing on the variation of σ_x and σ_y with depth

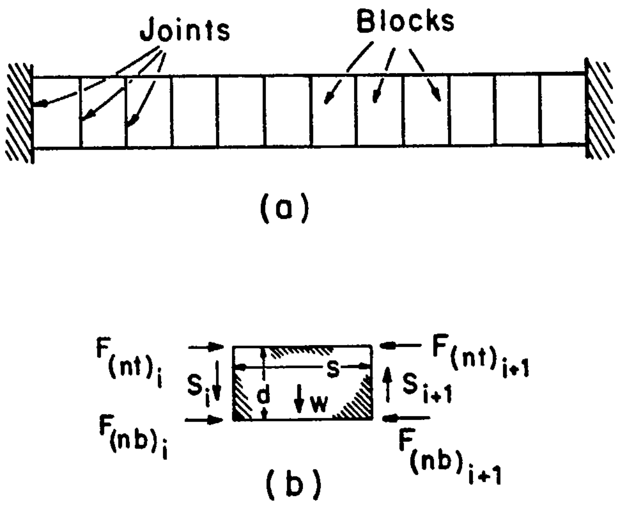


Figure 5. Jointed beam analysis (35).
(a) configuration
(b) forces on a single block

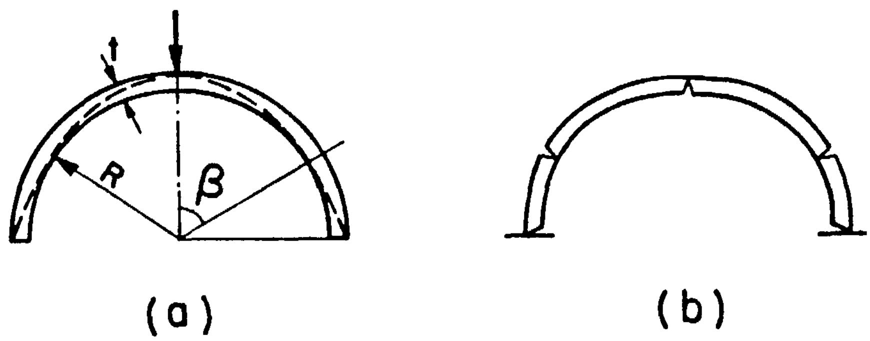


Figure 6. Plastic arch analysis (36).
(a) least thickness equilibrium state,
β = angle between centerline and hinge point
(b) arch failure mode at hinge points.

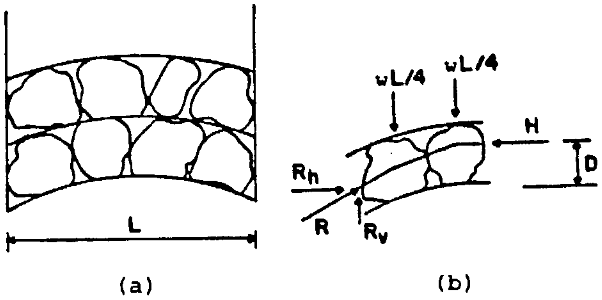


Figure 7. The analysis of a boulder arch (31).
(a) boulders wedged between abutments
(b) forces on one layer of boulders for half the arch.

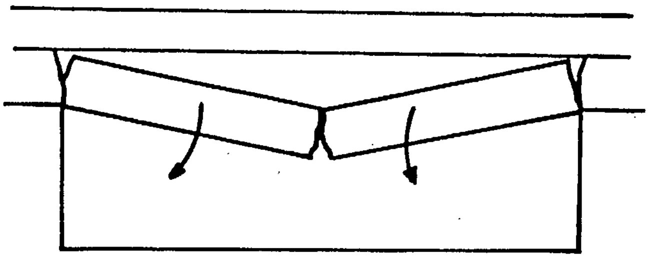


Figure 8. "Voussoir" failure in a laminated roof (37).

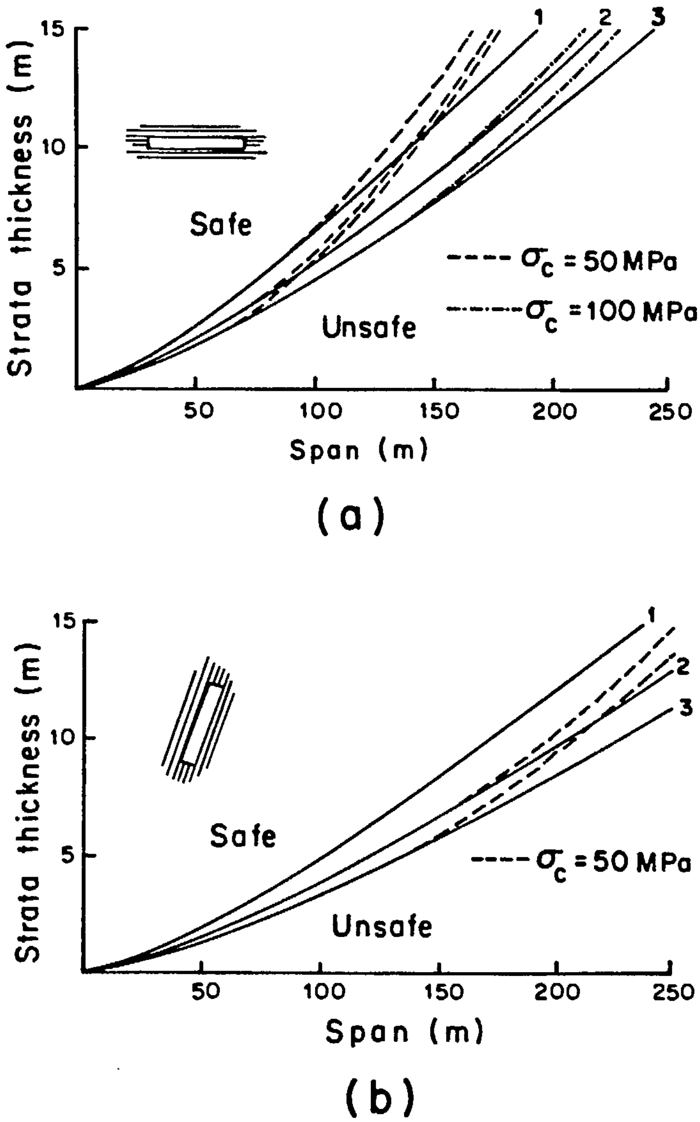


Figure 9. Design curves for roof beam subjected to "voussoir" action (37).
(a) 0°dip (b) 65° dip
curve 1: E= 8 GPa
curve 2: E= 16 GPa
curve 3: E= 24 GPa

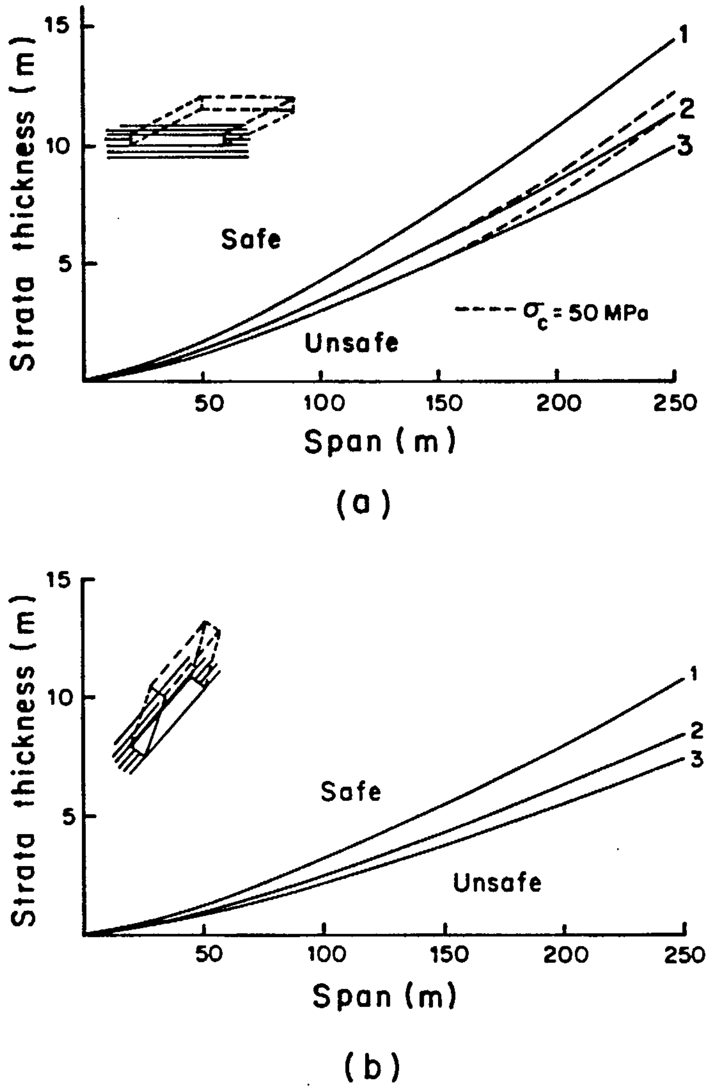


Figure 10. Design curves for square roof panel subjected to "voussoir" action (37).
(a) 0° dip (b) 65° dip
curve 1: E= 8 GPa
curve 2: E= 16 GPa
curve 3: E= 24 GPa

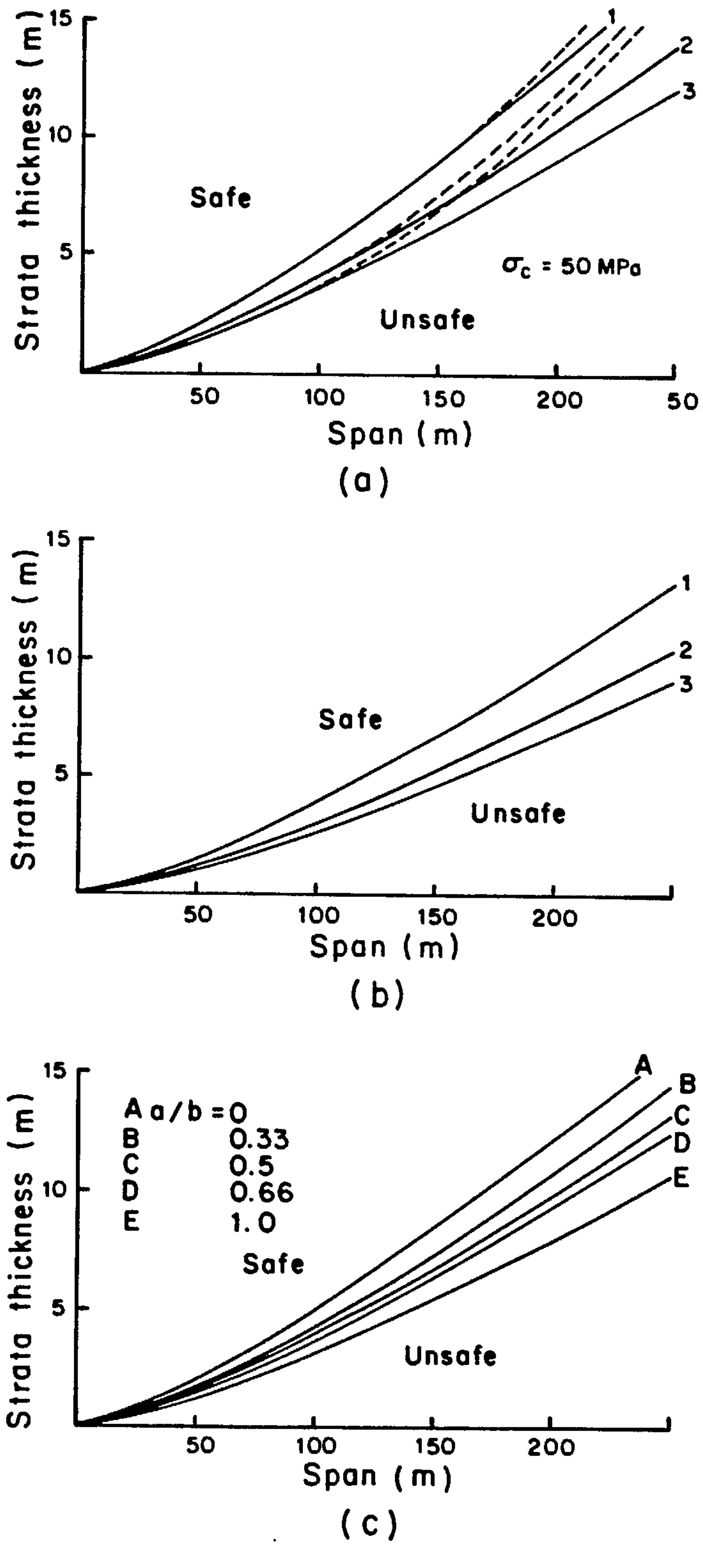


Figure 11. Design curves for rectangular roof panel subjected to "voussoir" action. E=8 GPa (37).
(a) 0° dip, width to span of 0.5
(b) 65° dip, width to span of 0.5
(c) 65° dip various widths to spans

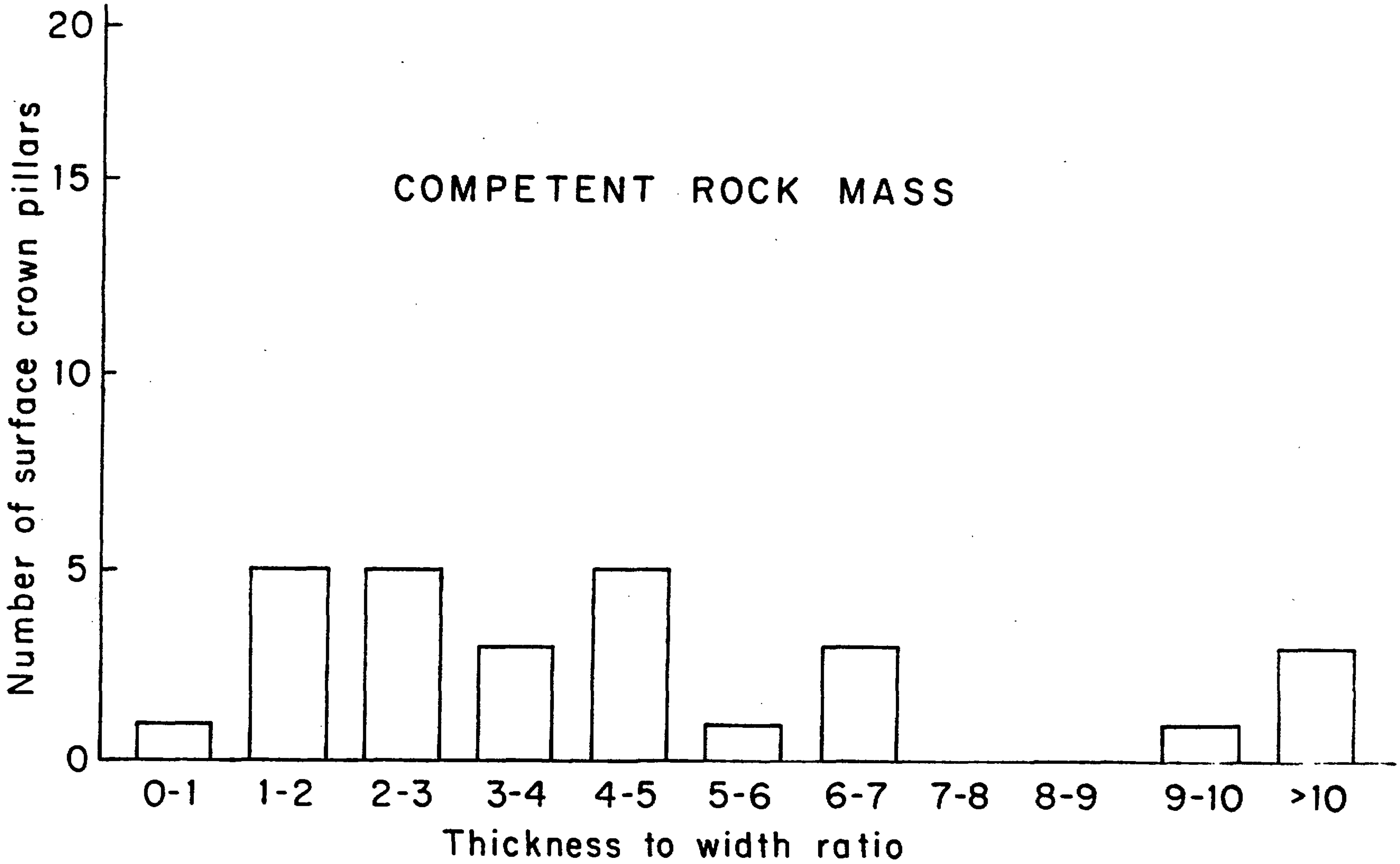


Figure 12 (a). Surface crown pillars: Thickness to width ratio, competent rock mass, 27 pillars (14).

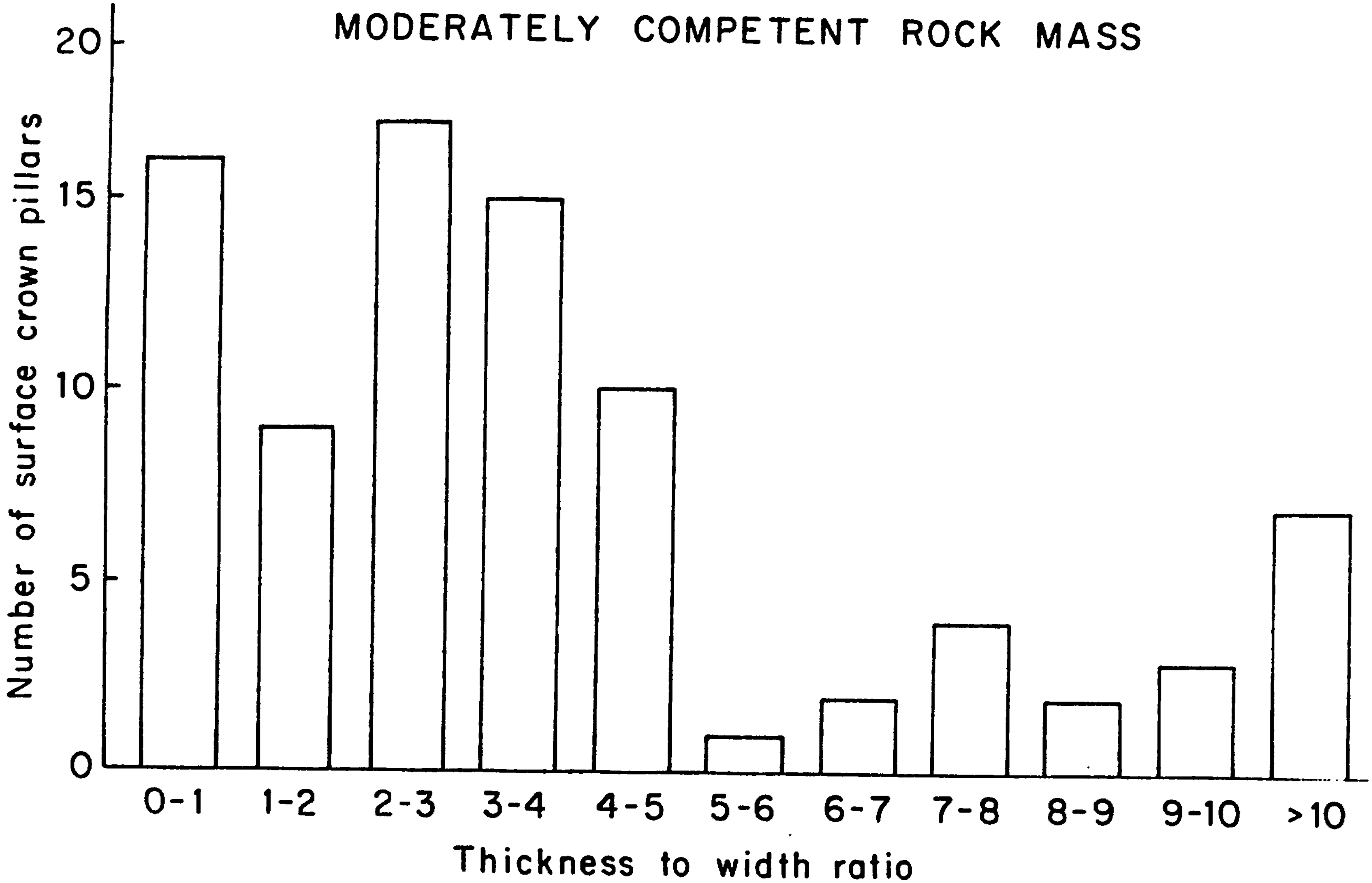


Figure 12(b). Surface crown pillars: Thickness to width ratio, moderately competent rock mass, 86 pillars (14).

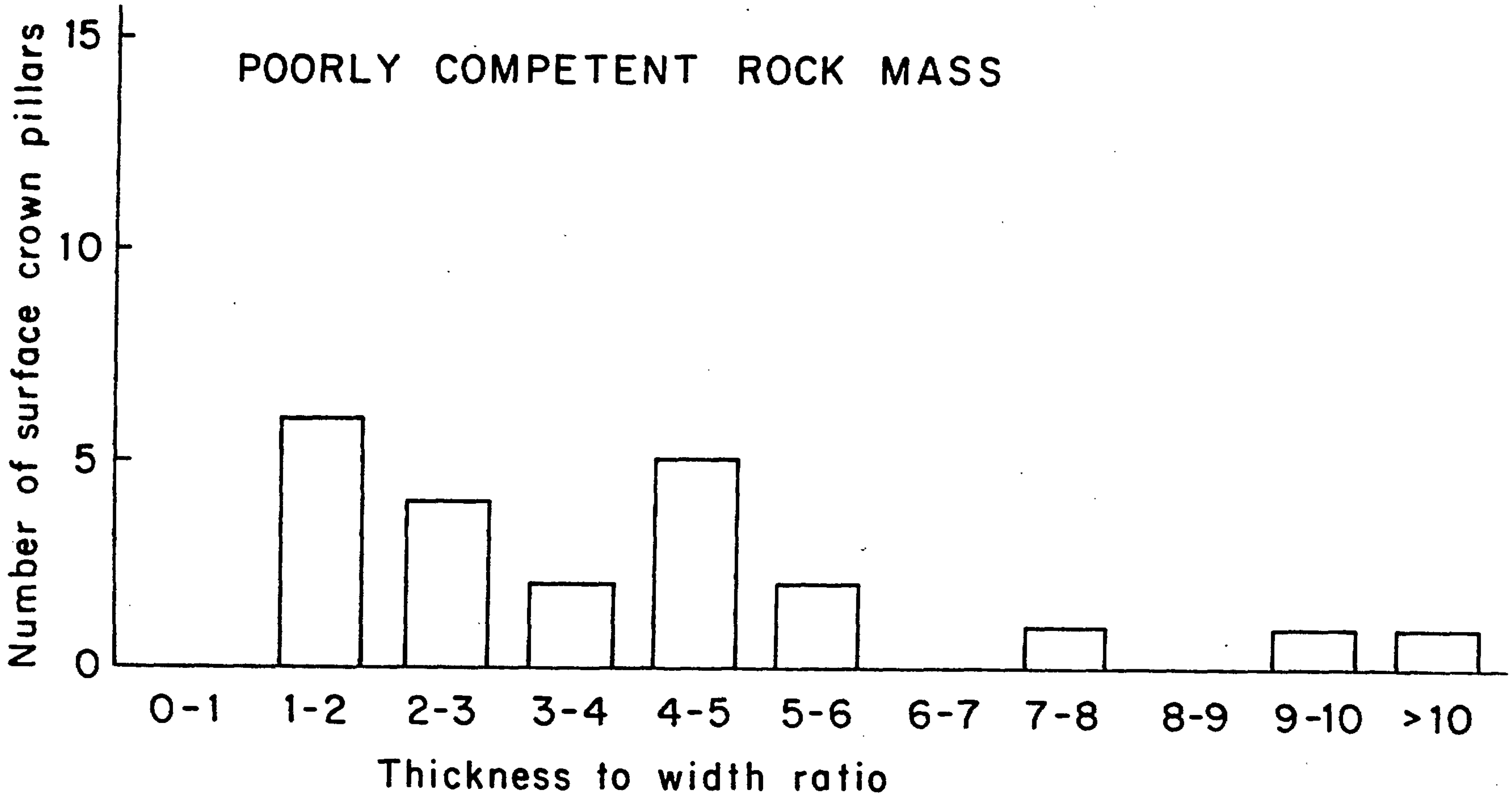


Figure 12 (c). Surface crown pillars: Thickness to width ratio, poorly competent rock mass, 19 pillars (14).

Table 1. Basic Characteristics of Surface Crown Pillars of Hard Rock Mines (1) (2)

MINE	1	2+	3+	4	5	6	7	8	9	10+	11	12	13	14	15	16	17	18+	19	20	21	22	23	
Items (3)																								
* BODY OF WATER (m)			3	(1)	-	-	-	7.6	-	N/A	N/A	-	20	-	-	-	-	-	11	-	-	-	13	
* OVERBURDEN (m)	(2)	(2)	27	36	4	15	5	20	17	16	20	15	3	5	30	9	1.5	(2)	5	-	45		19	
Substantial clay deposits			*	*				*	*		*	*		*								N/A	*	
* FORM OF THE DEPOSIT																								
- tabular	*				*												*				*		*	
- single vein			*	*				*	*	*	*		*		*			*						
- multiple veins		*				*	*							*					*	*		*		
- mass												*				*								
*Pronounced alterations	*			*	*			*						*			*		*		*		*	
*Walls of low competence	*	*	*		*			*					*	*	*	*			N/A	N/A	N/A	N/A	*	
*Walls of high competence						*	*			*									N/A	N/A	N/A	N/A		
DIP (degrees)	70°	70°	65°	45°	72°	80°	80°	90°	45°	70°	80°	85°	45°	85°	75°	75°	33°	70°	70°	60°	50°	75°	30°	
IMPORTANT FAULT(S)		*	*	*		*		*	N/A	*	*	*	*		*	*		*	N/A	N/A	*	*	N/A	
NUMBER OF WELL DEFINED JOINT FAMILIES	N/A	2	3	N/A	2	2	3	N/A	N/A	N/A	N/A	N/A	N/A	2	N/A	N/A	1	3	N/A	N/A		N/A	N/A	
* MAIN MINING METHOD																								
- stope and pillars																	*							
- shrinkage stoping					*	*	*		*													*		
- cut-and-fill			*					*					*						*		*		*	
- blasthole stoping	*	*		*						*		*		*	N/A	*		*		*				
* USE OF FILL		*	*					*	*		*	*	*	*	N/A	*		*	*		*		*	

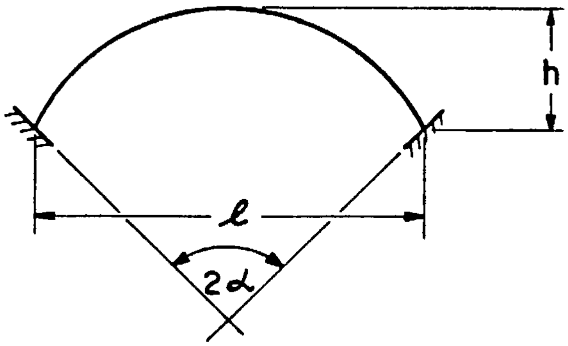
N/A not retrieved, or not available
(1) drained
(2) overburden removed
(3) average, applicable to surface crown pillars
(+) pillar(s) separating open pit from underground opening

Table 2 Geotechnical Characteristics Related to Surface Crown Pillars

ROCK	SOIL	HYDROLOGY
Field	**Field**	
Mass modulus of elasticity	Mass modulus of elasticity	Surface water
Permeability	Permeability	Circulating water
Joint orientation	Water content	
Joint properties (length, spacing opening infilling, roughness)	Density	
Faults (recent movements, net throw, brecciated zone extent and nature)	Shear strength	
Shear zones	Soil types, extent	
RQD (estimate using joint spacing)		
Degree of weathering		
Lab	**Lab**	
Uniaxial strength	Uniaxial undrained	
Triaxial strength (low confinement)	Triaxial undrained	
Tensile strength	Grain size distribution	
Modulus of elasticity	Plasticity	
Poisson's ratio		
Density		
Directional properties		
Degree of weathering		
Joint properties (roughness, infilling)		
Joint shear properties		
Core RQD		

h/l	γ_c
0.1	58.9
0.2	90.4
0.3	93.4
0.4	80.7
0.5	64.0

(a)



(b)

Table 3. Values for the compression factor γ_c (a) for uniformly compressed fixed ends elastic arch of constant cross section (b) (34).