Guidelines for use of the Scaled Span Method for Surface Crown Pillar Stability Assessment

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ABSTRACT: Reliably establishing the competence and adequacy of the rock cover that will remain in place above a near-surface excavation is key to preventing cave-ins and ensuring surface infrastructure stability above mine workings or civil tunnels. The Scaled Span approach, developed in the late 1980's, provides an effective means for empirically sizing a rock crown pillar over a near-surface excavation based on precedent experience. Despite the fact that this approach has been widely used worldwide for now well over twenty years, inadvertent collapses through to surface still continue to occur both of mine workings and civil tunnels. Disturbingly, a number of recent cave-ins have developed, again due largely to lack of adequate understanding of crown stability, commonly because of limited awareness of the lessons that can be learnt from the precedent experience embodied in the Scaled Span methodology. Citing examples from the now well over 500 cases with over 70 failure records of crown collapses, this current paper examines some of these failures as background to explaining the empirical Scaled Span methodology for aiding crown pillar design.

1 INTRODUCTION

Problems from collapses of near-surface workings have plagued mining operations for years, not just at remote mine sites but often also in urban settings, Figure 1. Civil tunnel collapses to surface are also not uncommon in urban areas. Reliably establishing the competence and adequacy of the rock cover that will remain in place above a near-surface excavation is key to preventing cave-ins and ensuring stability of any infrastructure above underground workings.

Assessing the risk for whether or not any near-surface mine opening or civil tunnel might collapse and break through to impact surface infrastructure is however challenging; while defining an appropriate minimum rock crown cover thickness that should be left above a mined opening is a particularly difficult design task. An attempt to address such problems in a mining context, more than 20 years ago led to the development of the Scaled Span empirical design guidelines for surface crown pillar rock thickness dimensioning discussed in this paper.

The initial guidelines, (Golder Associates, 1990, Carter, 1992, and Carter and Miller, 1995), which were mainly developed looking at steeply dipping orebody geometries, were targeted at helping mining engineers define potential collapse risk levels for new or abandoned mined openings, and also to help with establishing critical crown pillar dimensions for any proposed new mine designs.

Since its original development, based on a case record set of over 200 near-surface openings with 30 failure cases, (Golder, 1990), two minor, but quite significant improvements have been implemented to the methodology to aid practitioners in their use of the procedures for sizing surface crown pillars. These improvements to the original concept, which were published during 2002 and 2008 respectively, address (i) shallow dipping stopes and (ii) definition of pillar reliability for long term closure planning.

Figure 1. Surface Crown Pillar Collapse, at Waihi, in New Zealand, 2001 (from Tephra, June 2002, pp.29-33)

Unfortunately, despite the fact that the Scaled Span approach has been widely used worldwide for more than two decades for empirically establishing minimum rock crown thicknesses over near-surface openings, inadvertent collapses through to surface continue to still occur both of mine workings and of civil excavations. Exactly why collapses occur even today is enigmatic, and in some ways still driven by many of the same economic and logistical pressures that lead to mine collapses in the past. Almost, without exception, the numerous Turn-of-the-Century mine collapses and many others that occurred in the depression years, not infrequently, can also be tied to lack of knowledge about ground conditions, in particular, often lack of knowledge of crown competence.

Generally the reason that these more recent collapses have occurred can be traced to the same basic cause: – lack of knowledge regarding key factors pertinent to adequate design. Industry trends towards ever increasing sophistication in computer modelling tools and towards "designing out" problems on the computer, rather than spending money on getting more site specific geotechnical information on rock and soil conditions is perhaps also contributing to this apparent lack of improvement in collapse risk.

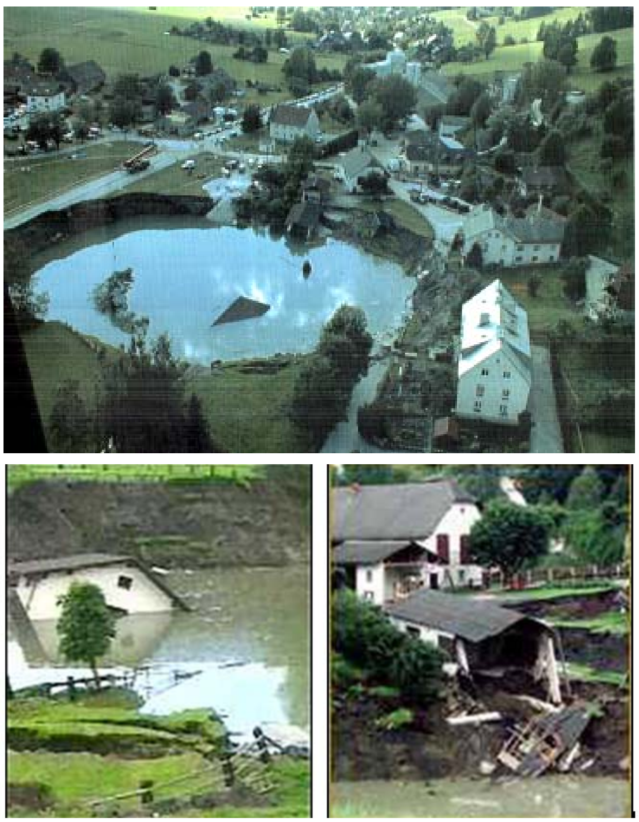

Figure 2. Adverse Impact on Surface Infrastructure from Crown Pillar Failure – Talc Mine, Lassing, Austria, 1998

Industry wide there are disturbing trends towards more and more "fast-tracked" or "value engineered" projects; with often a tendency to cut corners to achieve a better schedule on paper, by dispensing with as thorough a geotechnical site investigation as maybe should be undertaken. Another consequence of this trend is more reliance on computer modelling and less on observation. As a result "optimization" of opening dimensions and/or crown thicknesses is sometimes done completely out of context without benchmarking to actual field derived parameters or making subsequent checks during actual mining. In one of two recent cases, where crown pillars, that had been subjected to extremely detailed numerical modelling, failed as a result of the impact of deeper mining, not enough attention was being paid to early microseismic records suggesting disruption up into the crown, way before collapse occurred.

Needless to say, any near-surface mining or civil excavation will create stress changes within the crown pillar that will remain above the new opening. Whether the degree of change will be sufficient to create problems depends on crown thickness, on rock competence, on original insitu stress state and on a host of other factors. If these are not adequately understood, and excavations get planned and worse still executed much closer to the rock surface than is realized, then collapses and breakthroughs to surface, such as shown in Figures 1 and 2, are an inevitable consequence. Perhaps because of greater reliance on design sophistication nowadays, there is a greater lack of attention being given today than in the past to collecting reliable site-specific data. This perhaps is unknowingly increasing failure risk.

2 HISTORICAL BACKGROUND

The development of the Scaled Span concept in the late 1980's grew out of an industry need for a better method of assessing risk of crown pillar instability than was then currently available. The impetus for this change was that several significant failures had occurred in Canada that had attracted major public and media attention (Carter et al., 1988). In addition, it was soon recognized that these cases were not atypical worldwide, and indeed that numerous other collapses had occurred of crown pillars at other mines for many of the same reasons.

Research showed that many of the surface breakthroughs had occurred from heritage workings, but in some cases it was found that there were also recent collapses, in active mines, which had resulted from quite recent mining. Further evaluation found that many of the heritage collapses were related to ravelling and slow upward migration of caving, while most of the more recent collapses occurred due to mining too close to surface or due to poorer crown pillar rock conditions than expected.

A common theme for the more recent failures was lack of insight of the basic mechanisms controlling crown pillar stability. For almost all of the legacy workings, where subsequent surface breakthroughs had developed, and for many of the active mines where collapse problems had occurred in the period up to the 1980's when the research was initiated, it was established that "Rules of Thumb" were the principal, and in many cases, the only approach used for sizing of the crowns. While problematic, this might be excusable, given available rock mechanics understanding to that era. However, for a couple of the most recent crown pillar failure cases it is clear that there is also a worrying trend today to over-reliance on results from sophisticated numerical modelling. The critical missing element highlighted from these recent cases was adequacy of initial calibration with site specific geotechnical data. This, notwithstanding the need for vigilance during mining by looking for unexpected behaviour and then implementing investigation or monitoring measures to further refine the modelling to better reflect reality, are considered causative for misunderstanding the ground behaviour. Again it is lack of understanding that is the greatest problem.

These disturbing trends are obviously an issue that educators of young mining and civil engineers will need to address so that there will be improved knowledge in the design community in future years and so that these problems can be firmly set as a legacy of the past. Hopefully, some of the discussion in this paper will help shed some light on key issues of concern for ensuring reliable crown pillar designs.

Figure 3. Remediation underway for Surface Crown Pillar Stabilization after collapse of Provincial Highway 11B, Cobalt.

2.1 Public perception of risk

Historically, the impact that mining brings to most communities is seen as two-fold. Mining is seen to bring employment and growth opportunities, but also often once the mines close at the seeming expense of environmental and legacy stability problems,. As a consequence of decades of poor mining practices the public outcry for clean-ups and for dealing with legacy hazards has meant that Mines Acts nowadays include stringent regulations that demand assessment of the Stability of Underground Workings, with the objective of preventing the development of surface hazardous conditions due to ground subsidence into underground workings. Frequently, such legislation requires the Mines to restore their mining lease site to a state suitable for some final approved land use. As general standards, it is thus common nowadays to see the Mines Acts of various jurisdictions contain clauses such as “All surface and subsurface mine workings shall be assessed by qualified professional engineers to determine their stability. Any surface areas disturbed or likely to be disturbed by such workings in the long-term shall be stabilized. The study shall include an assessment of risk and of the consequence of crown pillar failure and be submitted for regulatory review and approval.” (ref. Yukon Energy Mines and Resources, 2005). Frequently these same Mines Acts call for all areas of concern to be “monitored for physical stability during all phases of closure until the Mine Site is closed out.”

In most Mines Acts, the regulations relating to surface impact and mine closure have largely been re-written over the last couple of decades to include requirements for specifically evaluating the stability of surface crown pillars as part of closure studies. The legal language regarding closure and financial guarantees has at the same time also become much more stringent, reflecting the increased awareness at Governmental level that public risk exposure can be significantly reduced by good crown pillar design.

2.2 Legacy Issues

One of the principal reasons that research into crown pillar stability, and particularly of old abandoned workings received significant funding in the late 1980’s, was the fact that a number of major collapses occurred in old mining municipalities in Ontario and Quebec that attracted widespread public attention. While crown pillars over old abandoned workings had been recognized by many mining municipalities for many decades as constituting a long term legacy issue, nothing much had been done to resolve or tackle the risk posed by these problems. The collapse of a section of Provincial Highway 11B through the old mining town of Cobalt attracted media attention and raised risk awareness to Government level.

Figure 3 shows the remnant section of the crown pillar remaining over the stope void as exposed once the overburden was removed down to bedrock. The remains of the asphalt pavement of the Highway can be observed in the upper part of the right picture where a grid of pegs is evident. These pegs mark out locations of airtrack drillholes used to define the problem crown geometry, (Carter et al, 1988).

The impact of the highway collapse on traffic flow and tourism and business in the area, and the realization by various levels of Government that mine workings of dubious stability existed under two public schools, under an old folks home as well as under several other municipal buildings spurred action towards resolving the highest risk problems, not just in Cobalt, but province-wide.

With the last active mine having closed in 1990, Cobalt was a shadow of its former past when more than 100 mines operated in the area. By 1987, when the Highway collapse occurred, the Town was just a small residential community with a population of 1480 people. Nevertheless, the legacy of its mining past, however remained very obvious, with glory holes dotting the landscape and safety fences, erected by past mining companies and/or by various Government agencies criss-crossing the community.

At the time of the collapse, studies were already underway to update the official zoning plans for the Town, specifically designating areas of potential mine hazards that needed to be put "off limits" for future building development, (ref. Mackasey, 1989, and Carter, Mackasey & Steed, 1995). Areas where possible future building would be inadmissible were identified, as also were "areas of caution" where buildings or other infrastructure was known or thought still existed over near-surface workings. While this study, (initiated in 1981, following a commission of enquiry into another mine collapse, this time in Quebec in the same year), addressed many concerns relating to potential for mine hazards to impact town planning, no major remediation or site investigation was implemented although some additional fencing was completed. It was, however, recognized, that there was a lack of data on absolute stability of the crown pillars of many of the known workings, and also that there was no ready means for assessing their stability.

2.3 *Precedent Case Histories*

The research initiative that began within a few months of the Cobalt highway collapse, but now with an wider Ontario focus, soon established that there was a history of collapses of old workings right across Canada, but mainly within the provinces of Ontario and Quebec where most of the mines were located, and that although collapses were not commonplace problems, they typically happened unexpectedly, and in some cases long after the mines had closed down.

In an attempt to try to quantify the extent of the potential hazard, and also gain some understanding of the mechanics of crown pillar instability so that future problems elsewhere in Cobalt and also across Canada (principally Ontario and Quebec) could be addressed, it was decided that initial research effort should be principally focused on old abandoned workings, but within about a year, now also with Federal funding, the initiative moved into looking into design methods that could be used more widely for evaluating surface crown stability for both active and abandoned mines.

Some of the earliest results of this research were published in the First International Conference on Surface Crown Pillars held in Timmins in 1989. This was a pivotal meeting, as one of the outcomes of the initial research to that point was that there was a dearth of methods available for reliable design, Hoek, 1989 gave the keynote conference address on crown plug failure mechanics, noting that even the now quite sophisticated available numerical analysis methods being employed at the time seemed inadequate, and generally incapable of appropriately defining onset of instability.

The evaluation of case histories to that time also showed that, except for a very few well documented failure cases, nothing much was known about the root causes for how, or why many of these failures had developed. The fact that some of the failures had occurred during initial mining and some had taken many decades to fail posed one of the most puzzling questions. Some light, however, had been shed on the different types of failure as a result of research underway at the time by CanMet to gather precedent case history data, (e.g. CanMet, 1984, 1985), but these very useful studies gave little comprehension of the actual behaviour of the surface crown pillars leading up to failure. Most of the focus to that time, despite the attention that was being addressed to the topic was on data gathering. The simple fact however emerged that it was difficult, and in some cases almost impossible to undertake rigorous back-analysis, because of paucity of good geotechnical information on the collapses, (Betournay, 1987).

While failures, by their very nature were treated as problems, and attracted much public attention and scrutiny, there was general reluctance to publicize any of the relevant facts related to the actual failure mechanics. In fact, mostly the technical publications available at the time were found to concentrate on successes, not failures. Research into Industry and Government records showed that even where a crown failure was known to have occurred; often precious little, if any, pertinent geotechnical data could be located. It seemed that the only exceptions to this situation was when expert witness testimony had been lodged in evidence in legal cases related to fatality enquiries (e.g. the Belmoral Investigation Report, Government of Quebec, 1981).

Lack of data on geotechnics, let alone design was found to be particularly commonplace where pillar failures had taken place on properties now defunct. Also, where pillars had been laid out on the basis of traditional "rule of thumb" methods, generally no documentation of the "experience" component could be found. The fact that "rules of thumb" were still widely being applied for crown sizing at that time in Canada, and worldwide, seems anachronistic, but the approach can still be found being employed even today.

This should not be criticized unnecessarily, as it may not actually be wholly detrimental to apply such methods, if they have been developed for a site specific situation, as such methods may in reality be entirely appropriate. Rockmass classifications and many design methods have, in fact, grown out of such "rules of thumb".

Often where traditional rules are still in use by the Mines today it is probably more from caution and uncertainty on behalf of the ground control staff that are using such rules that there is any more valid or appropriate procedure, than from any desire to short-circuit more rigorous design approaches.

One of the prime reasons for this reliance still on rules of thumb for crown pillar design, in the past, at least, stemmed from the perceived lack of sufficient realistic calibration between more esoteric numerical modelling and/or theoretical predictions and reality. This was in part attributed in the 1980's to lack of a sufficiently valid database with which to verify the then available analytical and/or numerical methods. The comprehensive crown pillar back-analysis report issued in 1990, based on a database of over thirty failure cases and more than 200 case records eventually provided such a basis and continues so to do through extra added case records providing a benchmark for calibration.

3 CROWN PILLAR FAILURE

3.1 Morphological Controls

Numerous similarities in morphology exist between various crown pillar collapses. Remarkably few of the documented cases show failure resulting directly from central crown cracking, as might be expected for a typical beam analogy. Most seem to occur as a result of dislocation or sliding on well-developed adversely oriented joints or shears within the rock mass, at either the hangingwall or footwall contact, or by ravelling or degradation processes. Further, only rarely do the failure geometries seem to have been dominated by single, weak structures, such as a fault or cross-cutting shear zone. Such weaknesses, (perhaps because they are more easily recognizable by mine operators, than more subtle problems), do not appear, in many of the case records as an important cause for unexpected failure as the more mundane features of the rock mass, which perhaps were ignored in the original crown pillar design.

Kinematics seem in many cases more important also than stress effects, with discontinuity controls taking a major part in dominating failure behaviour in most hard rock cases. For many narrow vein stope situations, failures seem to have preferentially occurred close to the intersection of adversely oriented cross-joints with the main ore-vein structure. In most of the ubiquitously weak schistose rocks, by contrast, failures commonly seem to have occurred by progressive crown destabilization through processes of tensile de-lamination at the hangingwall. In such cases, overall control of global stability may not be merely the kinematics of the weakness planes, but rather the influence of an adverse overall stress state within the abutment rock mass.

Carter, 1989 outlined some of the initial findings coming out of the back-analysis studies being undertaken at the time of the numerous failure cases in the Golder-CanMet database (Golder. 1990). Figure 4, which was presented at the time showed four of the ultimately five categories of crown pillar configurations that were consistently recognized from the database, reflecting similar patterns of geology and geometry, namely:

- **tabular, narrow vein orebody situations**, with usually, but not always weaker sheared ore zone rocks bounded by more competent hard wall rock conditions (**Category A**);
- **complex, blocky rock mass conditions**, often with structurally controlled weaknesses cross-cutting both the large scale ore body geometry and the wall rocks (**Category B**);
- **foliated rock environments,** almost always with well-developed anisotropic "weak" structure, generally parallel to the strike and dip of the ore (**Category C**);
- **disseminated ore zone situations,** usually with similar geometrical characteristics to Category B or C, but often set within an overall competent, but frequently slabby rock mass, which exhibits at least one sub-planar discontinuity set parallel to the ore zone dissemination, (**Category D**); and,
- **faulted or structurally dominated situations** where a weak or adversely oriented structure (other than the ore zone), usually either cross-cuts or parallels the stoping, (**Category E**).

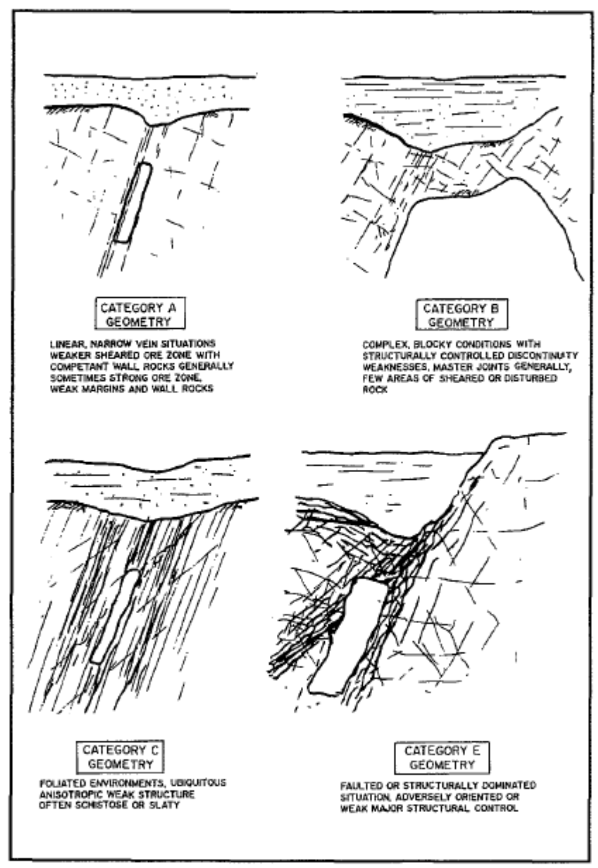

Figure 4. Four of the major sub-classifications of surface crown pillar geometries, (modified from Carter, 1989).

It will be noted that in Figure 4, Category D is missing. This is because; at the time of the 1989 paper only four of the five categories had been recognized. An extra “disseminated” category was added into the 1990 final Golder Report as numerous disseminated ore crowns were found which exhibited somewhat different failure behaviours than more common with the Category B and C cases.

3.2 Failure Mechanisms

While the general geometric categories encapsulated in Figure 4 were found useful for characterizing the various types of crown in the database, it was quite quickly realized that they did not always necessarily correlate with actual failure mechanisms, and in fact often the failure cases exhibited different processes even for crown pillars within the same morphology.

Examination of the failure cases included in the Golder-CanMet database (Golder, 1990) suggested five principal failure mechanisms, each of which has been shown schematically in Figure 5. As is evident, each is quite distinctive, and equally obvious, each is not amenable to the same type of analysis. It is of importance to note that classic beam theory methods have applicability in very few cases and certainly not for failures driven by ravelling or degradation.

Equally important is the observation that using just one type of numerical modelling approach may not be appropriate either. Discrete fracture models will function satisfactorily, but model block size replication has to be accurate, or the results will be totally misleading. It is therefore critical if numerical modelling is to be employed that the model be able to precisely mimic actual failure behaviour expected for that type of rockmass. Figures 4 and 5 and the bullet list below provide some guidelines on selection of appropriate analysis approaches:

- **Plug failure** – Limit equilibrium analytical plug models have most applicability here, empirical methods including the Scaled Span approach, plus also numerical modelling can be applied, but only if the models allow accurate replication of controlling plug structure;
- **Chimneying** – Several chimney/cave limit equilibrium analysis methods have applicability, the Scaled Span approach can be applied; also some forms of Voussoir limit equilibrium analysis and some numerical modelling approaches can also be workable;
- **Caving** – Mathews-Potvin / Laubscher stability graph assessments plus Scaled Span checks are all valid tools, plus it is feasible to use several discrete element forms of numerical modelling code, but only when such codes can accurately mimic the caving process;
- **Unravelling** – Numerical codes should only be used if they can incorporate or create discrete blocks and structure that simulates unravelling, which it must be appreciated is generally often different in character to caving. Some numerical codes which allow block cracking and others which permit Voronoi block tessellation and subsequent breakage along the micro-blocks have application for examining unravelling and disintegration processes, but most codes are not appropriate. Some empirical methods including Scaled Span assessments can be applied if they have adequate correlation cases on which they have been constructed; and
- **Delamination** – Some forms of limit equilibrium beam and plate analysis have applicability, also Voussoir solutions and numerical modelling with appropriate fabric replication, plus yet again the various appropriate empirical methods including Scaled Span assessments.

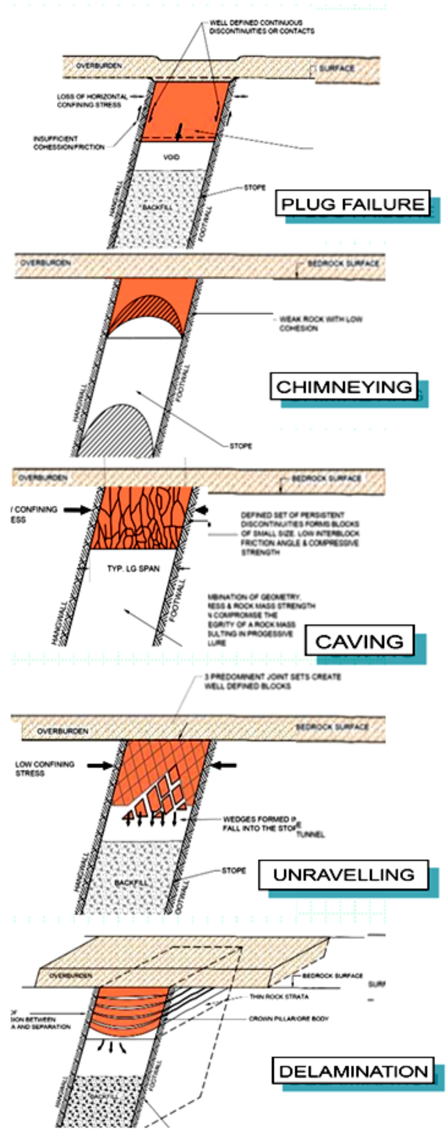

Figure 5. Principal Surface Crown Pillar Failure Mechanisms

4 CROWN STABILITY ASSESSMENT

For stability assessment three basic approaches can be taken for design of new crown pillar layouts or for evaluating the stability of old surface pillars, namely;

(i) *empirical methods* - using either Rules of Thumb, or more quantitatively, based on descriptive rock mass classifications,

(ii) *structural analysis and cavability assessments*

.. and/or ...

(iii) *numerical modelling procedures*.

Although each of these, seemingly might have equivalent applicability, they do not, and deciding the most appropriate approach can thus be tricky. What is clear is that whatever method is chosen it must be robust enough to be capable of handling the correct failure mechanism, selected from the various diagrams in Figure 5. In addition, irrespective of whichever approach and mechanism is chosen as appropriate, some key information is always needed on geometry and on rock quality and competence so that one can place oneself within the framework of the diagrams within Figures 4 and 5. One must also be aware of information limitations and that data may not always be available, or even adequate enough for undertaking sophisticated modelling, so often the best analysis approach is actually the most simplistic that replicates reality.

4.1 Data Adequacy

As commented above. obviously for any crown assessment to have validity, good reliable input data is needed. Some information is clearly required on bedrock geometry, data is needed on rock quality and also with respect to weathering effects or any other process that could degrade rock competence. The same basic suite of data needs collection in all cases in order to adequately evaluate the stability of any crown pillar situation (Table 1).

4.2 Empirical Methods

Design for most of the legacy crown pillars over near-surface workings that were mined at the turn-of-the-Century had been arbitrary at best, purely based on precedent practice, and random at worst based simply on … "leaving just one more round to surface". Traditionally, with years of mining in a given area, some cognizance of the effects of stress state and of weathering/degradation reduction of rockmass competence had been acquired through experience and thus was intrinsically incorporated into the classic Rules of Thumb for mining beneath crown pillars in <u>that</u> mining camp.

In many cases these Rules of Thumb were simple and could be applied in different mining situations where similar rock was encountered. However some rules were site specific, and applicable only locally. Attempts were therefore made to understand overall mechanics that might be controlling failures. It was recognized by the end of 1989 that more than 70% of the crowns had never been formally "designed" but had been simply laid out on the basis of traditional "Rules-of-Thumb". While most survived well, a few failures had occurred, obviously where a rule simply proved inappropriate.

Table 1. Basic data requirements for crown pillar design

Surface Conditions	- Topography - Presence or absence of water body
Overburden Characteristics	- Thickness, Material Properties - Stratigraphy - Groundwater regime - Bedrock/overburden interface topography
Rock Mass Conditions	- General geological regime - Ore zone dip - Rock types and classification characteristics - Hangingwall - Footwall - Ore zone in crown pillar - Structural controls Jointing, faulting, cleavage, etc. - Geometry of crown pillar and upper openings, width, thickness, stope spans, filling if present, support methods if present - Other factors - available data on stresses - complicating geometry – e.g. multiple ore zones, etc.

Through discussions with old miners and review of classical mining texts, such as Peele (1918, 1927, and 1941), attempts were made to try to improve the existing rules by undertaking detailed checks of available data to establish rock mass characteristics and pre-failure geometry for as many failed and non-failed surface crown pillars as possible.

Early checks found little in the way of published principles, but discussion with old miners and with Ministry of Mines' personnel familiar with Turn-of-the-Century operations, suggested a few benchmark guidelines – good rock 1:1 thickness to span; poor rock 3:1 or more. With this in mind the database was queried for rockmass quality and then sorted for thickness to span ratio. Figure 6 shows the resulting graph which is essentially a plot of the traditional "thickness to span ratio" rule-of-thumb, but tied to the Q rockmass quality scale, rather than to a single descriptive value – good or poor. A power regression dividing the failure cases (shown as black circles) from the stable cases (shown as open circles), has been drawn as follows:

$$T/S = 1.55\ Q^{-0.62} \qquad (1)$$

where T is crown thickness, S is span. and Q is the NGI rockmass quality index.

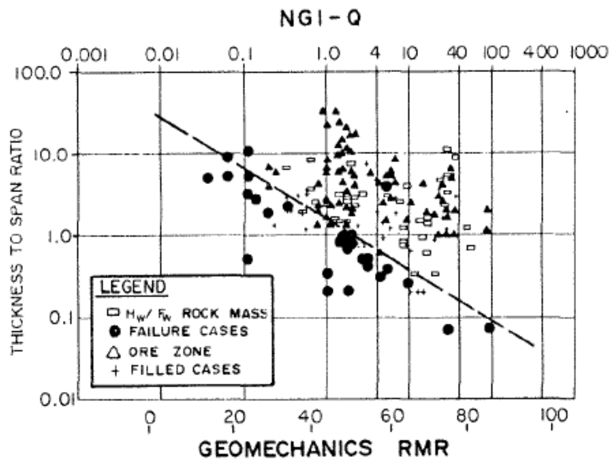

Figure 6. Thickness to Span plot of Crown Pillar Case Record Database with respect to Rock Quality, Q

Initially, it was considered that this simple update to the traditional thickness to span rule of thumb, but now tied to a rigorous rockmass classification would provide a basic, reliable guideline relationship that would be suitable for checking crowns in geological settings similar to those for which there were assessments held in the database. It was, however, quite quickly realized that since the relationship was not scale independent, its use without calibration could easily lead to significant errors. Accordingly, efforts were made to develop a better relationship that would more accurately describe crown pillar geometry with respect to rockmass quality, that could be properly scaled to crown geometry. This marked the starting point for the Scaled Span concept development as discussed in detail in the next chapter of this paper.

4.3 Structural Analysis and Cavability Assessments

Various structural beam-type analysis methods have long been utilized to examine surface crown pillar stability, typically making some simplifying assumptions of equivalent beam thickness and looking at the most critical failure mode of the beam in shear or tension, either cantilevered or supported at both ends. A thorough review of all sorts of beams, slabs, plates, arches and Voussoir and delamination modes was undertaken as part of the back-analysis studies completed in the late 1980's, (Golder, 1990). Some beam analogy geometries were in fact examined as part of the remediation design for dealing with marginally stable areas alongside the highway zone that had collapsed in Cobalt in 1987 (Figure 3). Figure 7 shows some results for a cantilevered and simply supported beam analogy for the competent crowns in this situation bounding weak silver-calcite veins at various locations above the mined stope. The Case 2, cantilevered situation in these analyses always proved the most critical design situation.

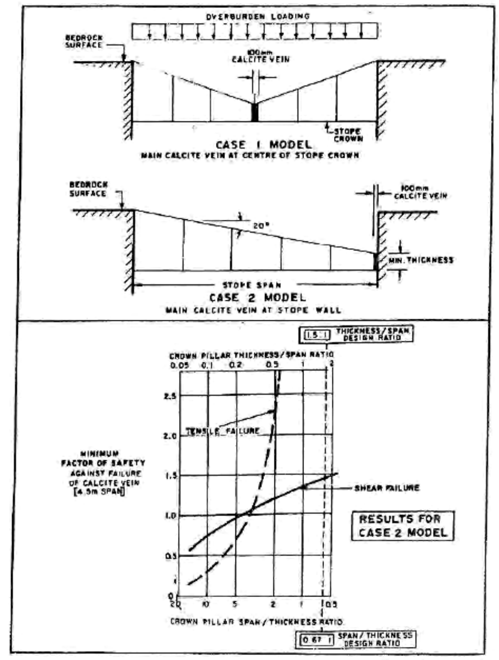

Figure 7. Results from parametric analysis of simple crown pillar geometry for two possible silver vein locations seen in Cobalt stopes (from Carter et al. 1988)

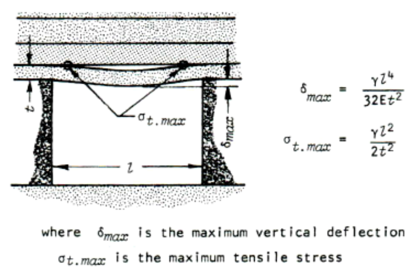

Figure 8. Beam analogy for thinly bedded strata (from Hoek and Brown, 1980, p.235)

Figure 8 shows another beam analogy, this one studied for classic de-lamination of thin strata in a crown.

In addition to these beam type analogies, wedge kinematics and plug type failures, such as sketched in the uppermost diagram in Figure 5 have been looked at by various authors, with the solution for the plug case, as analyzed by Hoek, 1989 now available in the program code CPillar© by Rocscience.

As with all of these approaches to simplify the problem to something tractable, either sensitivity or parametric analyses or probabilistic approaches are recommended as never are all the parameters known, not even the geometry. In the case of the plug failure CPillar code, data for the rockmass and the crown are entered as means and standard deviations to estimated ranges for the field conditions, and analyzed probabilistically.

The other side of the coin – the shape of stable and unstable opening geometries has also been the subject of extensive evaluation over the years, with various analytical and empirical approaches considered of value for different rock conditions. Numerous rules of thumb also exist for estimating the influence heights of cave-ins. Bell et al., (1988), in reviewing causes of ground movements, suggested that ratios of 1.5 to 3 times the span of a mine opening, was commonplace for cave heights in many rock masses. Piggot and Eynon,(1978) in their landmark caving study (ref. Figure 9) suggest maximum crowning heights of up to 10 times seam thickness in bedded strata above coal mine workings; a value supported by Whittaker and Breeds, (1977) and by Garrard and Taylor, (1988). Other workers, e.g., Madden and Hardman, (1992), looking at caving in South African coal mines above gate roadway intersections suggested relationships between seam thickness and the height of caving over an intersection, consistent with the ratio of 2½-3 times drift span width, again suggesting initial caving heights typically occur in this range before onset of void ravelling.

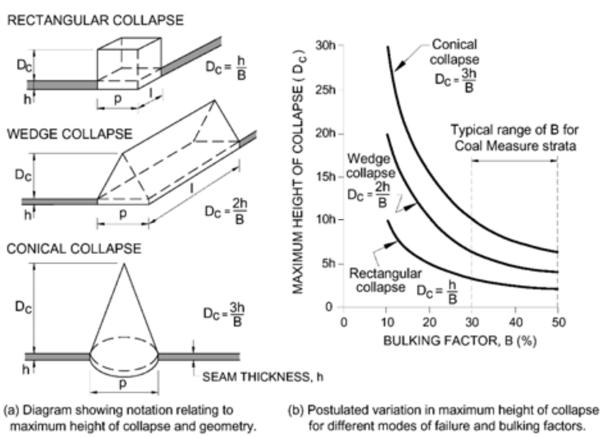

Figure 9. Empirical caving estimates for different geometry and "bulking" assumptions (from Pigot and Eynon, 1978)

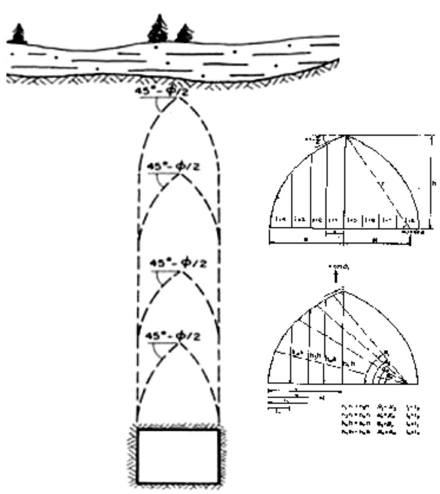

Figure 10. Chimney cave analysis methodology for weak rockmasses (after Bétournay, 2004, 2005)

Other approaches for predicting upwards caving for weak rocks derive generally from studies of particle flow behaviour in grain silos. Two approaches can have particular application for crown pillar considerations, depending on the rock conditions – the chimney caving approach of Bétournay, 2004, 2005 (Figure 10) and classic ellipsoid concepts per Janelid and Kvapil, 1966, Just and Free, 1971, and Kendorski, 1978 and others, (Figure 11).

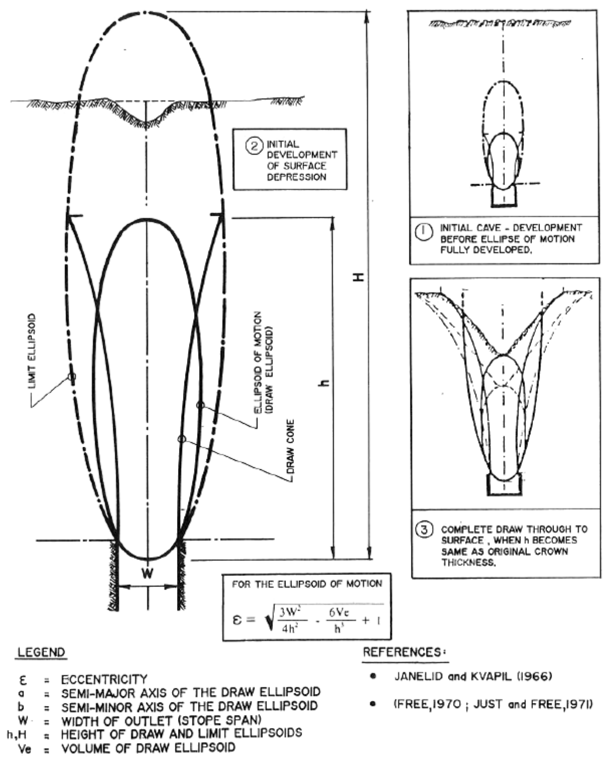

Figure 11. Concept of silo caving mechanics with the draw cone and ellipsoid of draw defining the rockzone in motion and the limit ellipse defining the influence extent of incipient rockmass damage (after Janelid and Kvapil, 1966)

Basically, the assumptions behind use of either of these models for assessing degree of potential upwards caveability, and thereby establishing a probable limit for caving height, (as a means to assess whether breakthrough to surface is feasible or not) devolves into checking balance between extent of available underground void space to extent of possible caved material. Establishing whether choking off occurs of an underground void depends almost entirely on the degree of probable "bulking" of the caving rockmass. For rock masses with high Q's/GSI's (Q>10, GSI>60 typically) bulking factors are in the 30-40% range, while for rocks in the low Q/GSI range, (Q<0.1, GSI<25) bulking factors can be lower than 20%, suggesting that upward caving for such rockmasses can extend way higher before a given stope chokes off. Analysis for different rockmass types and conditions is thus critical if accurate replication of behaviour is to be achieved.

The obverse of these same questions are of paramount importance to those designing block and panel cave mines, because "controlled caving" is much desired in these mining applications. As such, it is beneficial to look in a little more detail at the current design approaches being used in the block caving arena, as they help provide needed insight for assessing potential for crown pillar caving risk.

Firstly it should be appreciated that in the last twenty years, there has been a revolution in block cave mining, largely driven by the need to compete with large open pit operations. Traditionally most cave mining operations initially started with deliberate surface breakthroughs, termed glory holes, or as a result of unexpected crown pillar collapses. As a result, there is much synergy in the early design approaches, which again relied heavily on rules of thumb, which were learnt through hard experience from uncontrolled caving behaviour and inadvertent crown cave-ins. The silo models of cave mechanics (such as illustrated in Figure 11), which were developed in the 1960's, provided much needed understanding and improved insight to aid cave mine design.

Unfortunately, this analytical ellipsoidal algebra proved difficult at best, and misleading at worst to apply for accurately determining Height of Draw, drawpoint spacing and/or cave progression, let alone prediction of actual cave behaviour. In consequence, most modern block cave mine design over the period up to the last 8 to 10 years, resorted to and then remained heavily biased toward use of empirical caving rules, such as those published by Laubscher, 1977, 1994 and by Diering and Laubscher, 1987, rather than on the analytic approaches shown in Figures 10 and 11.

The use of empirical methods though was not without problems, and many criticisms have been levelled over the years against application accuracy of Laubscher's original design charts. The problems that occurred at North Parkes in 1999, when the cave "stalled", (Ross & van As, 2005) prompted a resurgence of research work into cave mechanics and in particular into trying to better understand more competent rockmasses. Several new or updated design approaches were formulated, some based on the existing empirical methods. In parallel major work was initiated on developing new numerical models that could truly cope with and replicate actual cave mechanics.

One of the empirical approaches that came out of this work and has applicability for crown pillar failure assessment, was developed by Trueman and Mawdesley (2003) as an extension to the Mathews method for stope design, (Mathews et al., 1981), They concentrated on re-examining and extending the original databases used for the previously published empirical caving charts, noting that the biggest variance in actual versus predicted caving prediction capability occurred with competent strong rock masses and partially with misinterpretation of the adjustments in the MRMR rating scheme for such rockmasses. The revised empirical Mathews chart for caving application, which was derived by Trueman and Mawdesley, solves a number of the perceived problems with the earlier empirical charts. However, as pointed out by Brown (2003), largely because of insufficient case record, its use may need to be restricted to low aspect ratio undercut geometry (with length to width ratios <3:1) due to inability to assess 3D confinement effects around high aspect ratio rectangular undercut footprints.

In parallel with these advances on the purely empirical front, some consideration was also given in the same time frame to see if it was feasible to define a semi-empirical caveability failure criterion.

Karzulovic and Flores (2003) found that by defining the rockmass as a Hoek-Brown material and examining the deviatoric stress state around a developing cave with respect to rockmass strength, crude checks were possible on whether or not caving would continue or not. They termed their deviatoric stress:strength ratio, the Caving Propagation Factor, $CPF = (\sigma_1 - \sigma_3) / (\sigma_{ci} (m_b.\sigma_3/\sigma_{ci} + s)^a)$. Application to cases such as North Parkes, which were used for validation, suggested this approach could be useful in the interim, until better numerical modelling methods could be developed.

Rather than being the poor cousin of the hardrock underground mining industry, the cave mines are now at the leading edge of research initiatives. In fact, as a result of significant work undertaken since the North Parkes disaster, not only have cave mine design approaches advanced appreciably, but all fronts of rock mechanics have benefitted as these new numerical methods now allow better replication of reality. One of the major developments that has come out of these cave mining needs and initiatives has been the merging of developments in discrete fracture network (DFN) and synthetic rockmass mechanics approaches to rockmass characterization, coming out of oil-field and high level nuclear waste research, with rock material caving mechanics and rockmass material particle flow codes, such as PFC and REBOP (Pierce, 2009, Chitombo, 2010,), thereby allowing progressive caving behaviour to be more holistically examined with synthetic rock mass models with constituent ubiquitous joint behaviour, Sainsbury et al, 2008, 2011.

4.4 Numerical modelling

All of these major advances coming out of cave mechanics have applicability for crown pillar design as essentially the problem is the same in reverse – for a cave mine – the need is for the rockmass to cave in a controlled manner; for a surface crown pillar – the need is for the rockmass to remain stable.

As will be evident from the preceding discussion numerical modelling advances have been significant since the initial publication of the crown pillar back-analysis report (Golder Associates, 1990), with the result that most of the now available numerical modelling tools are far more sophisticated than were available at that end of the 1980's. However many of the fundamental issues and problems remain the same, with most of the same analytic formulations still being used in rock mechanics evaluation today as they were 25 years back, but with much greater computing power and hugely improved graphics capability. While there is today less use of boundary element methods, and more use of finite element, finite difference and much more application of distinct and/or discrete element methods today, there still remain significant problems in getting appropriate and representative input parameters defined. Just like the caving situation, there also are still many difficulties evident in achieving accurate replication of actual failure processes observed with real surface crown pillar failures. This remains true, even when instrumentation data and patterns of microseismicity are available to guide back-analysis modelling. This continued problem of not being able to precisely replicate the complex, essentially gravity-controlled mechanisms which characterize cave mechanics and also crown pillar failure behaviour remains an issue and a caution for indiscriminate use of numerical models as a panacea for crown pillar rock mechanics design.

Several of the available current program codes, many of which allow post-failure behaviour to be modelled, have validity for being able to examine the types of geometry and failure mode portrayed in Figures 4 and 5 respectively, but the definition of parameters, and particularly, capturing the variability of real rockmasses still lags far behind advances made in the various modelling codes.

Major progress has been made in modelling rock breakage as part of various initiatives related to block caving design, but still the problem remains of input parameter definition for the reliable use of these codes. Advances have also been spectacular in modelling various progressive failure processes of caving, ravelling and tensile fragmentation of rock masses, and in developing discrete fracture network replications of known ground conditions, but from the perspective of analyzing and designing surface crown pillars, where there is often much greater variability in rockmass properties than occurs at greater depths in many other mining situations, current numerical modelling approaches still have some way to go. Making correct choices of input parameters, and ensuring that the codes can handle the most appropriate constitutive model are still major concerns to numerical methods becoming sufficiently reliable for use as a sole tool for surface crown pillar design.

Right now, reliance must still be placed on doing multiple analyses with different approaches, including numerical methods, with rigorous cross checks being carried out, not only between analysis techniques, but also back to empirical approaches to check against precedent case behaviour and hence expected rockmass response. Verification to known precedent experience has to be the watchword.

5 THE SCALED SPAN METHOD

While one may have high hopes that advances in computing power and in ease of applicability will continue; to the extent that creating DFN's will become the norm for initiating new rock engineering studies, and that any new mines will tackle crown pillar design with much improved sophistication; there will however always be a need for two things:

– (i) proper characterization of actual ground conditions, so that the models have validity, and

– (ii) a means for ensuring reliable verification against precedent behaviour for similar construction in similar rock conditions.

It was precisely for these latter reasons in the late 1980's that the Scaled Span empirical method was developed for evaluating the 30 failure cases in the crown pillar database, as by that time, it had been recognized than none of the then available analytical or numerical modelling approaches could adequately and uniquely replicate the actual failure behaviour documented from the historical records.

In fact, it was quite discouraging that no one tool or technique that was in current use at the time could achieve even half adequate replication, sufficient that it could be considered reliable enough to be recommended for ubiquitous design use, (Carter et al., 1990). This lack of reliable computer design tools is much less of a problem today as compared with the late 1980's, as so many of the numerical modelling tools have been so dramatically improved that replicating behaviour is much less difficult, but only if input data is reliable and the mechanism is correctly calibrated back to precedent experience.

In 1989, focus was therefore turned back to attempting to improve the old "Rules of Thumb" approaches as the best means for checking back to the precedent case records and using these for benchmarking the back-analyses of the old failures. This turned out to be a bit of a circular exercise as all the modelling that was at that time being undertaken was looking at different types of failure mechanism, with the intention being to use the precedent data for "calibration". It was because of this circular lack of a valid checking approach that eventually the Scaled Span concept came to be developed into a design tool in its own right.

5.1 *Scaled Span Concept*

When it was realized that all of the old empirical rules were scale dependent and thus problematic to apply for bigger or smaller crowns than they had been developed for, a method for treating the scale problem and for characterizing the three dimensional geometry of a typical crown pillar was sought. There was a desire also to try to maintain correspondence with the available rockmass classification methods so that the wealth of experience included in these procedures could be used to advantage for defining key rockmass characteristics pertinent to the stability state of a typical crown pillar. After some research into how the geometry of a typical pillar could be analyzed, the concept of dimensional scaling was explored, with the basic precept for the Scaled Span approach being that one should be able to scale the entire three dimensional geometry of the pillar down to some measurement, related solely to “span”.

It was therefore decided to follow the approach used in the Mathew’s Method for open stoping based on Laubscher’s scale for undercut sizing for block cave operations, and utilize a hydraulic radius scaling term to account for the third dimension.

For defining the other critical control on crown stability – rock quality, it was decided to use the readily available NGI Q-system quality scale as this seemed a reasonable benchmark. The decision was also made to keep the scaling term solely related to the geometry of the crown, following the analogy of treating the crown pillar as a simply supported beam, with some degree of two way spanning, that would depend on width and strike length. The initial concept at this stage was solely to create an improvement on the thickness to span chart in Figure 6 with an x-axis rockmass scale utilizing either *Q* or *RMR*, so that all the standard tables of descriptions for parameters could be maintained, yielding estimates of rockmass quality that would be widely understood.

The chart progressed quickly to something similar to the Q-support chart, (Barton, 1976) but was extended in a similar manner to the Mathews Method open stope design chart (Potvin et al, 1989) to consider the third dimension into the term plotted on the y-axis.

In the final 3D-scaling arrangement developed for the chart, it was decided in the end not to use a direct hydraulic radius scale, but rather to dimensionally scale all the key facets of a crown pillar, in a new term, - defined as the “Scaled Span”.

The intuitive principle on which this scaling was developed was that – as the size of an underground excavation increases, so does the degree of failure risk and the likelihood of collapse of that structure’s crown. Conversely, as rockmass quality into which the excavation is made increases, so the likelihood of failure of the crown decreases (because rock block size increases and intrinsic rock mass competence and strength improves, thereby helping to resist failure).

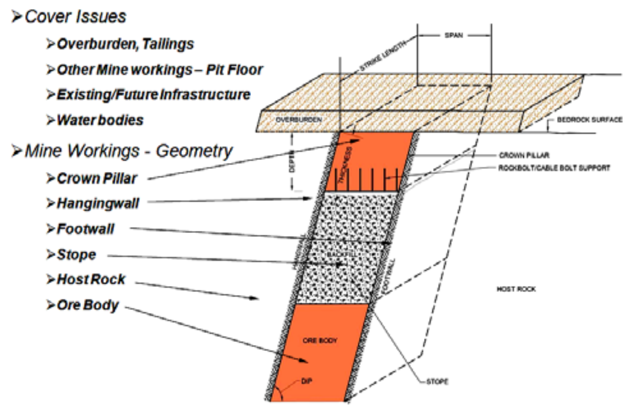

Figure 12. Crown Pillar Nomenclature

Given the typical stope geometry and geological and rockmass conditions comprising a typical crown pillar situation, such as shown in Figure 12, it was recognized that:

$$Crown\ Stability = f\left(\frac{T\sigma_h\theta}{SL\gamma u}\right) \qquad (2)$$

where *increased stability* for any rock mass quality would be reflected by an increase in:

T, the rock crown thickness

σ_h, the horizontal insitu stress

and/or…in θ, the dip of the foliation or of the underlying opening, and;

where *decreased stability* for any crown would result from increases in:

S, the crown span

L, the overall strike length of the opening

γ, the mass (specific gravity) of the crown

and/or…in u, the groundwater pressure.

Noting that this stability expression could be split into two, based on (a) mined opening geometry and (b) rockmass characteristics, led to the development of the basic deterministic assessment approach of comparing the dimensions of the crown pillar over the stope opening geometry, as characterized by the Scaled Crown Span (C_S), against a critical rockmass competence, Q_{crit} (at which failure might be expected) as defined from the boundary between the failure cases and the stable cases.

5.2 *Geometry Definition*

In developing the three dimensional geometry term for characterizing the overall crown pillar with respect to its actual span, S, four component terms were considered – a thickness to span ratio term, to maintain some connection with the original non-scaled “rules of thumb”, – an inclination factor term, reflecting the fabric of the rock mass and typically the stope dip angle, – a term to account for the three dimensional geometry of the crown (incorporating a hydraulic radius (span ratio) factor), and – a weight term to account for equivalent Terzaghi-style "rock loading" effects with respect to the size of the crown and due to any superincumbent loads – such as overburden.

The final Scaled Span, C_S relationship was thus defined as follows:

$$C_S = S\left(\frac{\gamma}{T(1+S_R)(1-0.4\cos\theta)}\right)^{0.5} \tag{3}$$

where: S = crown pillar span (m);

γ = specific gravity (which is dimensionless but has the same numerical value as rock mass unit weight, tonnes/m^3);

T = thickness of crown pillar (m);

θ = orebody/foliation dip, and;

S_R = span ratio = S/L

(crown pillar span/crown pillar strike length)

In the same manner as the Q-chart, this scaling expression, C_S, is plotted on the y-axis against rock quality on the x-axis. Figure 13 shows the original chart published in 1990, with each of the case records identified as to whether the case was an open stope, or backfilled, and whether stable or not.

5.3 Rock mass quality definition

As shown on Figure 13, the Scaled Spans for all of the original case records have been plotted against rock quality, using both the RMR/Q classification axes, each positioned relative to each other on the basis of the well-known Bieniawski 1976 correlation expression; $RMR_{76} = 9 \cdot \ln(Q) + 44$, where RMR_{76} values were categorized according to the 1976 codings, thereby maintaining an equivalence with GSI, (Hoek et al., 1995, Marinos & Hoek, 2000).

In the original work, rather than merely defining the critical rockmass quality Q_{crit} for each given case and using this for comparing with the Scaled Span, a limiting span at which failure might be expected was defined based on a regression fit to the data.

As it was noted that the best fit line dividing the failed and stable case records was similar in shape and closely matched the empirical "unsupported span" curve originally proposed by Barton et al, 1974 for the tunnelling and natural cavern cases, a similar power curve regression fit was formulated to define the Critical Scaled Span (S_C or S_{Crit}), at which failure might be expected; viz:

$$S_C = 3.3 \text{ x } Q^{0.43} \text{ x } \sinh^{0.0016}(Q) \tag{4}$$

where the hyperbolic sinh term was introduced into the expression in an attempt to fit the marked trend of significant non-linearity to increased stability at high rockmass competence.

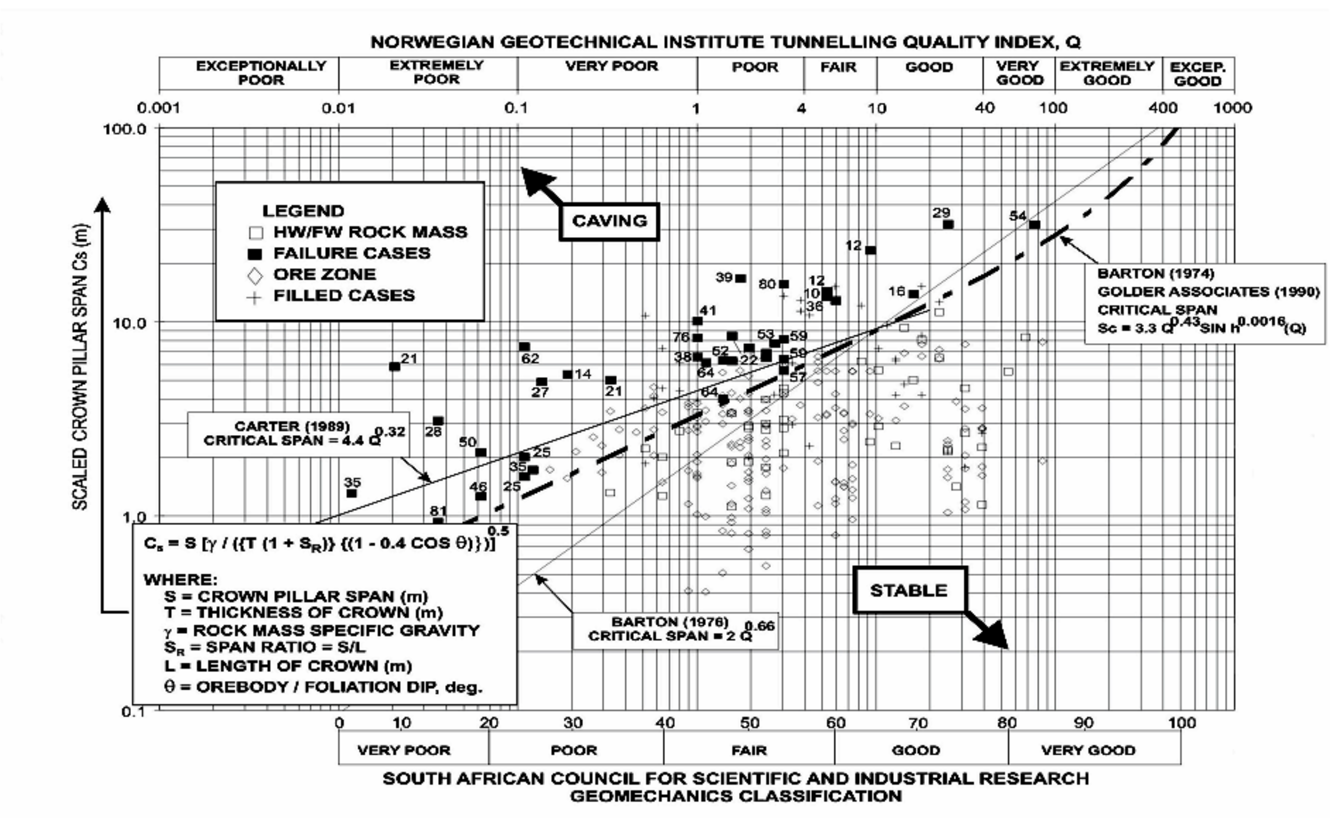

Figure 13. The Original Scaled Span Chart, showing stable and failed case records plotted as Scaled Spans versus Rock Quality

In Figure 13, each of the individual failure cases have been plotted as best as possible with respect to what could logically be defined as the "controlling rock quality" reflecting the condition thought most likely characterized the failure case behaviour. For cases where dislocation occurred on a footwall or hangingwall contact, the contact rock quality was taken as representative. For crowns which ravelled due to a weak ore zone rockmass, the ore quality was assumed as representative. Similarly for the non-failed cases, the geometry (Figure 4) and the most likely failure mechanism (Figure 5), were both assessed and then the most likely and representative (ie., controlling) Q value – for the ore zone, hangingwall or footwall then assigned in order to plot a point on the chart.

In determining the controlling rock quality, care had been specifically taken to examine the probable stress state and groundwater conditions prevailing in the crown zone, and Q was then reduced accordingly. In this regard, it must be appreciated that the crown pillar scaling expression for C_S characterizes solely the geometry of the crown. The influence of clamping stresses and/or groundwater are explicitly excluded. Rather it is expected that these will be considered in defining, and, if necessary, appropriately derating the defined rockmass quality Q and/or the RMR_{76} / GSI assessments.. Suggestions for assigning appropriate SRF and Jw (and other non-rock material property terms in RMR) are discussed subsequently, in §7.3.

6 COMPLICATIONS & REFINEMENTS

Application of the chart through the early 1990's, often as part of planning activities for mine closure studies, and mostly for making decisions on stability of old and/or abandoned mine workings, proved its usefulness as an effective tool for ranking various at-risk workings. However, merely defining a pillar as stable or probably of concern because it plotted on the non-safe side of the critical span line was found insufficiently quantitative for continuing use of the chart as a basis for multi-million dollar remediation decision-making. It soon became apparent that some formalized measure of acceptability and conversely of residual risk was needed. This lead to an attempt in 1994 to develop a risk ranking for crown pillars of various competence, so that prioritization of hazard remediation measures could be better formulated.

With increasing use of the chart to tackle actual mining cases, with real data, it was also found that there were certain stope geometries that were not adequately addressed by the then available scaling approach. Essentially the problem boiled down to the existing orebody dip and obliquity terms in the scaling equation not being able to properly define stability characteristics for shallow dipping stopes.

Two major improvements were therefore worked on throughout the late 1990's and through till 2008 – (i) improving the ability of the scaling expression to handle a wider range of stope geometries and crown configurations, and – (ii) extending the methodology so as to generate better estimates of potential failure risk to aid decision makers evaluating remediation options, mainly for mine closure planning purposes.

6.1 Revised Scaling Approach – Shallow Stopes

Detailed evaluation of the Golder-CanMet database following initial problems being highlighted with the applicability of the scaling expression for shallow dipping stopes, found that the problem lay in the fact that the original relationships had been mainly derived from steeply dipping case studies (i.e. with excavation/foliation dips, θ>40˚). This essentially meant that the original crown pillar database was not particularly representative of shallow dip stoping geometries. Further, experience showed that stability for the steep cases was largely controlled by ravelling of the ore zone or margins, or by shear on weakness planes at the hangingwall and footwall ore/reef zone contacts, whereas, for the shallower cases it was clearly more controlled by hangingwall breakthrough. As a result, in 2002 supplemental improvements were made to the definition expressions to better account for shallow dipping geological structure influences.

The 2002 paper, as a basis for extending the applicability of the method to shallower dipping situations, therefore reviewed available analysis methods commonly employed for design of flat and low dip workings and extended the original database by an additional 114 cases, primarily from shallow dipping situations, including twenty one additional failure cases. These cases included both hard rock situations and coal and soft rock examples.

Relevant analyses and data from near-surface shallow dipping hard rock mining locations along the gold reef mines in the Witwatersrand were also examined in order to check onset of tension crack opening (termed progressive hangingwall caving).

These analysis checks to real situations found that, provided that the controlling hangingwall rockmass characteristics, rather than the mined seam characteristics were utilized for analysis, the scaled span approach reasonably replicated observed crown stability on a single opening basis, such as illustrated in Figure 8. It was however established that the geometry definition for a crown over multiple rooms or wide-span non-flat situations, was not generally correct, as collapse in this sort of situation required breaking across the hangingwall strata in order for caving to progress. Accordingly, modifications were made to the geometry term to handle this situation.

The 2002 publication extending application of the Scaled Span concept to incorporate these shallow dipping stopes was therefore predicated on further study of failing and stable case records of shallow geometry (Carter et al., 2002). From this work it was found that with increasing obliquity, even with competent rock, crown arching would not develop and failure up into the crown would actually begin as shown in the left diagrams in Figure 14 either through hangingwall delamination and/or through voussoir buckling, rather than by direct sloughing of the orebody core or by propagation along the ore/host rock contact margins up into the crown.

In the revision to the basic relationships published in 2002, the controlling "span" at crown level was therefore redefined to take into account the equivalent "effective" span, S_{EFF} of the hangingwall projection up to the elevation of the stope crown, as shown in the right diagram in Figure 14, with the extent determined from the cave line, ξ_L with the failure geometry, that might develop post-collapse of the crown and hangingwall estimated by drawing break lines up from the base of the stope.

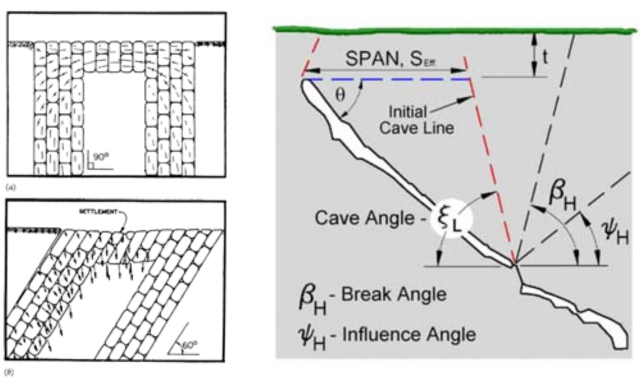

Figure 14. Concept of Effective Span for Shallow Dipping Stopes based on Zone of Influence and Cave and Break Angles

The logic for this definition was, in concept, that the mode of failure associated with a shallow dipping stope in blocky rock, while intrinsically different from caving in coal, would still tend on a gross scale to encompass a similar extent of ground deformation, as crudely might still be postulated based on inferred cave and break angles.

To apply this correction, an expression for the effective crown span (S_{EFF}) as defined as "the actual span at crown level, plus the horizontal projection of the hangingwall span-length (within the cave line extent)", was developed as follows:

$$S_{Eff.} = S + L_H\left(\cos\theta - \frac{\sin\theta}{\tan\xi_L}\right) \qquad (5)$$

where: S = actual crown span of the stope (m);
L_H = (hangingwall length of the stope (m);
ξ_L = cave angle, and;
θ = stope/ore body dip

Note that this expression has been modified from the original version published in Carter et al. (2002), as the original equation for S_{Eff} only considered the hangingwall projection of the stope and thus tended to underestimate true effective spans for stopes of wide extraction geometry, where the crown span itself was also of significant dimension.

While not critical to apply this correction for all moderately dipping cases steeper than about 45° this expression could be applied ubiquitously if adverse hangingwall behaviour were anticipated. Typically though, for angled stopes with relatively steep dips (i.e., greater than approximately 45˚) the competence of the stope crown itself or the strength of the ore/host rock contacts has generally been seen from the database records to control behaviour.

Control in the intermediate dip range (40°-50°), however appears from the case records to differ appreciably dependent on the competence of the rock mass comprising the ore zone and hangingwall (back). If the ore zone is weaker than the host rock mass, then stability seems to always be controlled by crown failure through ravelling of the ore or by shear on the ore contact margins, and as such in defining the controlling span, use should be made of the original uncorrected true stope span, S.

If, however, hangingwall rock mass quality is adverse, hangingwall failure might be more likely, and in this case the shallow mode effective span criteria should be adopted.

When the dip of the stope is very shallow but not actually flat (i.e., in approximately the 15˚ to 20˚ range), the hangingwall length will again be the main control on crown pillar stability, and the effective span calculated based on the back length needs to be used for the calculation of C_S.

Definition of an appropriate span for nearly flat dipping stopes (ie., with dips of less than about 15˚), such as would be found in many seam mining situations (gold and platinum reefs, bedded gypsum, coal etc), will depend on the extraction length between abutments or between pillars. For this range of dips, it is suggested that a span appropriate to the controlling room or panel longitudinal or transverse dimensions be selected. Obviously in the effective span expression, the seam thickness, S, would then be ignored in this calculation.

Application of the cave angle concept as an indicator of the extent and geometry of probable hangingwall disruption appears to remain valid up to a mined seam (stope) dip angle within ±5 degrees of the inferred friction angle for the roof rockmass. Beyond this point, breakthrough of the hangingwall is generally unlikely and sliding mechanisms within the mined reef/ore zone width, and shear on the ore hangingwall or footwall contacts tend to dominate.

As this is the dip range for which the original scaling relationship was initially developed, it was suggested in 2002 and experience confirms this, that the shallow stope angle refinement to the scaled span method should not be applied for stopes steeper than about 45°.

The fact that good matching to actual case behaviours as reported in the 2002 paper has over the past decade continued to be proven, supports ongoing use of this range for application of the modification for better characterizing the controlling span for shallow and intermediate dipping openings.

The equations for calculating the effective span needed for establishing if potential breakthrough to surface of shallow dipping, given in the 2002 paper, are summarized in the following paragraphs.

Appropriate break and cave angles for the hangingwall rock mass can be calculated, as follows:

$$\beta_{\mathrm{H}} = \tan^{-1}\left[\sqrt{2\pi}\,.\tan\{(45+\phi/2)\{1-0.32\sin(2\theta)\}\}\right] \quad (6)$$

…and $$\xi_{\mathrm{L}} \cong \theta + \tan^{-1}\left[\sqrt{2\pi}\,.\tan(45-\phi/2)\right] \quad (7)$$

Both equations require an estimate of the instantaneous friction angle, ϕ at an appropriate normal stress for the inferred break-line geometry. This can be readily achieved by means of the following relationships, given an assessment of the hangingwall rockmass quality (in terms of RMR/GSI or Q) and the Hoek-Brown failure criterion parameters, viz:

$$\phi = \mathrm{Arctan}\,\frac{1}{\sqrt{4h\,\mathrm{Cos}^2\alpha - 1}} \quad (8)$$

$$\text{where; } \mathrm{h} = 1 + \frac{16(\mathrm{m}\sigma_{\mathrm{n}} + \mathrm{s}\,\sigma_{\mathrm{c}})}{3\,\mathrm{m}^2\,\sigma_{\mathrm{c}}}$$

$$\text{and, } \alpha = 1/3\left[[90° + \mathrm{Arctan}\left(\frac{1}{\sqrt{\mathrm{h}^3 - 1}}\right)\right]$$

and where

- m and s can be defined from GSI, using the well known Hoek-Brown regression relationships, m_b/m_i = exp{(GSI – 100) / (28 – 14D)} and s = exp{(GSI – 100) / (9 – 3D)}… …where D is the disturbance factor as listed in Table 2.
- σ_c and m_i are respectively the uniaxial compressive strength and the Hoek-Brown intact material constant appropriate for the hangingwall rockmass, as determined from laboratory testing, or estimated from Table 3, and
- σ_n is an estimate of the prevailing normal stress that would be acting across the break-line (this can be initially calculated using the vertical depth to the bottom of the stope and a typical minimum break-line angle of 60° ~ equivalent to the Rankine wedge failure angle 45+ϕ/2), i.e.,,

$$\sigma_n = \frac{\sqrt{3}}{2}\gamma H \quad (9)$$

Thus, with the dip angle of the stope, θ, known, and the friction angle for the hangingwall rock mass defined from the above relationships, the break and cave angles can be derived, allowing calculation of S_{Eff} for the cave-line geometry shown in Figure 14.

For correct use of this shallow span approach, it is critical to always use the hangingwall rockmass Q/RMR values, not the ore zone characteristics.

Note however that these updated expressions not only include D, the disturbance factor, but also GSI has been substituted for RMR_{76}. These changes to the equations have been introduced to maintain consistency with the 2002 updating of the Hoek-Brown failure criterion to the following generalized expression:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\,(m_b\,.\sigma_3/\sigma_{ci} + s)^{a} \quad (9)$$

where a = 1/2 + 1/6 [exp{– GSI/15} – exp{–20/3}] as defined by Hoek et al, 2002.

For determining an appropriate friction angle for estimating the cave and break angles, however there is merit in making an overall change to use of GSI, rather than RMR_{76}, especially for very poor quality rockmasses, as definition of RMR, particularly at the low end of the competence scale can be problematic.

Table 2. Guidelines for the selection of the Disturbance (Blast Damage) Factor D (after Hoek et al. 2002)

Location	D-Factor	Disturbance Characteristics
Underground Excavations (confined conditions)	0	High Quality Perimeter Blasting (100% half barrel traces) or Mechanical Excavation with TBM or Roadheader
		NATM excavation in weak rock with mechanical excavation
	0.5	As above (but with invert heave issues)
	0.8	Poor Quality Blasting (<50% half barrel traces)
Open Cuts and Open Pits (de-stressed conditions)	0.7	Controlled Blasting (>80% half barrel traces)
	1	Poor Quality Blasting (<50% half barrel traces)
	0.7	Mechanical Excavation in Weak Rock with Face Shovel etc
	1	Typical Open Pit Production Blasting/Quarry Blasting

Table 3. Typical values for σ_{ci} and m_i for range of igneous, metamorphic and sedimentary rocks (ref. discussion in Carter & Marinos, 2014 regarding range variability for application with GSI for different parent rock type characteristics)

Typical σ_{ci} (MPa)	Metamorphic	Igneous			Sedimentary	m_i
		Intrusive		Extrusive (Volcanic)		
		Felsic	Mafic			
125-250		Coarse (Granite)				31-33
100-300	Granular Texture (Granulites, Quartz Gneiss)	Medium (Grano-diorite-Diorite)				28-30
85-350	Medium, amorphous (Amphibolite)		Coarse (Gabbro-member of ophiolites)	Mafic (Basalt) Intermediate (Andesite) Felsic (Rhyolite)	Coarse (Conglomerate-not clayey)	25-27
75-350	Fine, amorphous (Homfels, Quartzite)		Medium (Dolerite / Diabase – member of ophiolites)		Medium quartz cemented (Sandstone/ Sandstone members of flysch or molasse/greywacke)	17-20
50-200	Bended/ Gneissose (Biotitic Gneiss)		Fine (Serpentinite-member of ophiolites)		Medium carbonates (Limestone)	13-16
30-100	Folliated (Phyllite, Slate)				Fine, (clastics) (Siltstone/ Siltstone members of flysch or molasse/tuff)	10-12
20-60	Schistose (Schist)				Fine, Calc-rock (Chalk/marl)	7-9
10-50	Mylonitized (Sericitic Schist, Mylonite)				Ultrafine (Claystone, Mudstone / sheared Siltstone, Shale members of flysch)	4-6

As explained in Hoek et al., 2013 and in Carter and Marinos, 2014, GSI and RMR_{76} can be used quite interchangeably while in the normal range of blocky rockmasses, but, care needs to be taken towards the low end of the rock competence scale.

Similarly care must also be taken with Q definition in this low range, as also pointed out by Palmström, and Broch (2006). As correct definition of the controlling friction angle is important for getting good estimates of the cave and break angles, if rock strength has been seriously degraded, by pervasive alteration for example, so that σ_{ci} < 15MPa, there might be merit to consider application of the low end transition *m* and s values introduced by Carvalho et al, 2007 and further discussed in Carter et al, 2008, and in Carter and Marinos 2014, as follows, but possibly using the slightly modified equations for s* from Castro et al, 2013:

$$m_b^* = \left[m_b + (m_i - m_b) f_T\right] / \left(4a^* - 1\right) \tag{9}$$

$$s^* = s + [(1-s) f_T] / (4a^* - 1) \tag{10}$$

where a* varies from 0.5 when completely rock-like, and behaviour is Hoek-Brown through to 1.0 when behaviour is Mohr Coulomb. The expression for a* and for the transition function, f_T updated to better match ISRM strength grades for the R0 / R1 boundary, as plotted in Figure 15, are as follows:

$$a^* = a + (1-a) f_T \tag{11}$$

$$f_T(\sigma_{ci}) = \begin{cases} 1 \; , & \sigma_{ci} \le 1.0MPa \\ 1/\sigma_{ci}^2 \; , & \sigma_{ci} > 1.0MPa \end{cases} \tag{12}$$

where m_b, m_i, *s* and *a* are the normal range values of the regular Hoek-Brown constants, and the uniaxial strength σ_{ci} is expressed in MPa.

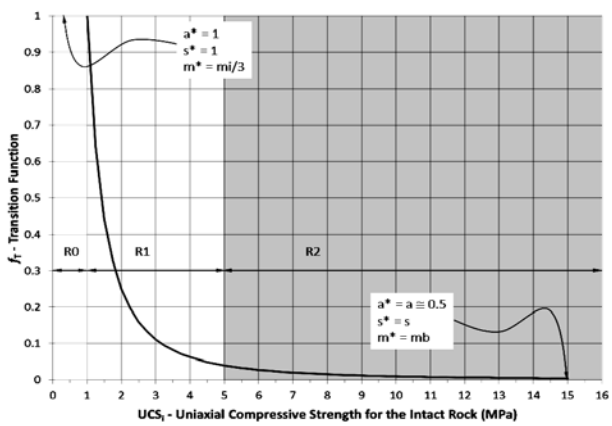

Figure 15. Range of Applicability for Transition Function to deal with Low Strength Rock Masses (after Castro et al. 2013)

Application of these relationships will generally not be necessary for most mining situations, as intact rock strengths are generally way higher than 15MPa. Nevertheless, care must be taken to ensure that an appropriate friction angle is used within the cave and break angle equations. It should be representative of the rockmass strength for a new break-line structure cross-cutting through lithology, such as shown in Figure 14, not the friction angle of specific weaknesses, unless they happen to sub-parallel the inferred breakline geometry.

6.2 Logistic Regression and the Improved Chart

The other significant improvement made to the Scaled Span methodology has been in improving the chart and rankings to make it easier to better address various levels of allowable risk from the perspective of different stakeholders, (Figure 16, Table 4).

In its most basic form the Scaled Span method can be applied deterministically to assess failure risk by simply comparing the Scaled Span (C_S) reflecting the mined geometry, to the Critical Span (S_C), reflecting the rockmass quality and calculating an approximate Factor of Safety, Fc ≈ S_C/C_S. If this is done and the Scaled Span, C_S exceeds the Critical Span, S_C, as defined by the assumed rock quality, unless the stope had been sufficiently supported, or fill approaches had been used in mining; for any opening with Fc less than 1.0 the likelihood of failure would be predicted to be high.

While this simplistic approach is straightforward, with increasing use of the Scaled Span approach being made for checking whether or not abandoned mine workings might or might not need remediation, this deterministic Factor of Safety approach wasn't found to provide sufficient discrimination for making expensive decisions, particularly at the high consequence end of the risk scale (where these needed to be made more on the basis of estimates of the likelihood of failure). As both regulators and mine operators wished to prioritize solutions based on the severity of perceived problem excavations using risk ranking matrices, so that the likelihood of failure and potential consequences could be better assessed, attempts were made in the mid-1990's to develop a better, probabilistic assessment approach for use with the scaling expressions.

Multiple probabilistic spreadsheet analyses using @Risk with Latin Hypercube sampling were therefore conducted on case records from the database where sufficient rock quality information was available to characterize rock mass quality variability.

Plotting the S_C/C_S ratios as a cumulative frequency distribution from this assessment suggested that the variability was approximately normally distributed, which then allowed a crude error function fit to be formulated between the assessed probability of failure (P_f) and the S_C/C_S quotients, (Carter, 2000); as follows:

$$P_f = 1 - \operatorname{erf}\left[\frac{2.9F_c - 1}{4}\right] \tag{13}$$

where: P_f = Probability of failure;
F_c ≈ an approximate Factor of Safety
= S_C/C_S, and;
erf( ) is the standard error function

Table 4 – Acceptable Risk Exposure Guidelines - Comparative Significance of Crown Pillar Failure
(from Carter et al., 2008, modified from Carter & Miller, 1995)

Class	Probability of Failure %	Minimum Factor of Safety	Maximum Scaled Span, Cs (= Sc)	ESR (Barton et al. 1974)	Design Guidelines for Pillar Acceptability/Serviceable Life of Crown Pillar				
					Expectancy	Years	Public Access	Regulatory position on closure	Operating Surveillance Required
A	50 – 100	<1	$11.31Q^{0.44}$	>5	Effectively zero	< 0.5	Forbidden	Totally unacceptable	Ineffective
B	20 – 50	1.0	$3.58Q^{0.44}$	3	Very, very short-term (temporary mining purposes only ; unacceptable risk of failure for temporary civil tunnel portals	1.0	Forcibly Prevented	Not acceptable	Continuous sophisticated monitoring
C	10 – 20	1.2	$2.74Q^{0.44}$	1.6	Very short-term (quasi-temporary stope crowns ; undesirable risk of failure for temporary civil works)	2 – 5	Actively prevented	High level of concern	Continuous monitoring with instruments
D	5 – 10	1.5	$2.33Q^{0.44}$	1.4	Short-term (semi-temporary crowns, e.g.under non-sensitive mine infrastructure)	5 – 10	Prevented	Moderate level of concern	Continuous simple monitoring
E	1.5 – 5	1.8	$1.84Q^{0.44}$	1.3	Medium-term (semi-permanent crowns, possibly under structures)	15–20	Discouraged	Low to moderate level of concern	Conscious superficial monitoring
F	0.5 – 1.5	2	$1.12Q^{0.44}$	1	Long-term (quasi-permanent crowns, civil portals, near-surface sewer tunnels)	50–100	Allowed	Of limited concern	Incidental superficial monitoring
G	<0.5	>>2	$0.69\ Q^{0.44}$	0.8	Very long-term (permanent crowns over civil tunnels)	>100	Free	Of no concern	None required

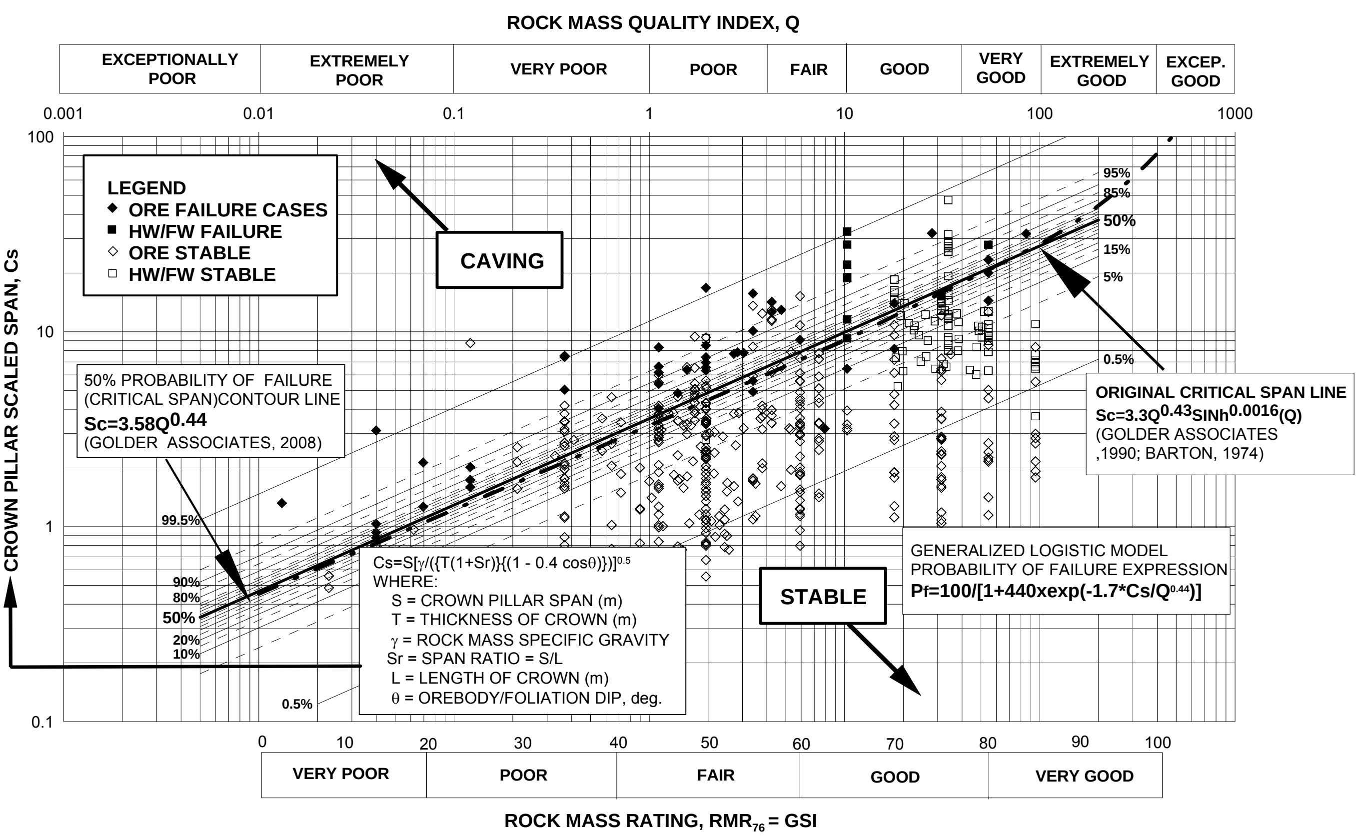

Figure 16. Updated Scaled Span Chart with Probability of Failure Contour Intervals

Although simple to apply as a method for rapidly estimating failure probability, use of this expression was found to consistently overestimate failure risk for very low probability situations. While not a problem from the viewpoint of helping prioritize crown pillar cases for defining which needed more detailed follow-up assessment, (the original purpose for developing the expression), the fact that the expression consistently over-estimated P_f values by 5-10% compared with more rigorous probabilistic methods, largely constrained widespread acceptance of this formula for more than ranking assessment purposes. This, in turn, also partially prompted development of the approach outlined below as it was clear from multiple projects where rigorous application of quantitative probabilistic methods had been employed in order to determine crown failure probability from Scaled Span data, that undertaking this degree of analysis for each and every crown was not only time consuming but also very difficult to objectively validate. To improve on this and to simplify and expedite stability assessment on a crown by crown basis, logistic regression analysis techniques were applied in early 2008 to develop the probability of failure contour lines shown on the updated Scaled Span chart in Figure 16, and itemized in Table 4, both from Carter et al, 2008.

The basic relationship for the Critical Span Line defining S_C, which is plotted through the centre of the chart as an update to the original division line was developed by simple regression fitting the selected cases on either side of the stable versus failed line from the original database. Plotting this update line constitutes a classic example of the use of classical linear regression for fitting continuous dependent variable data. This however is a simplification of the actual situation for the crown pillar database points as they have variability with respect to more than one variable and as a consequence ordinary linear regression cannot do full justice to assessing the probability of occurrence of an event, such as a crown failure, when there are discrete, ordinal, or non-continuous outcomes.

Over the past two decades, significant advances have been made in log-linear modelling for multivariate analyses of categorical data such as the crown database. As this was thought would hold promise for improving the applicability of the Scaled Span Method, analyses were undertaken using a special form of the general log-linear model, known as the logit model. As the "logit" itself, is the natural logarithm of the odds, or the log odds, for the crown pillar problem, using this model it became possible to express the log odds of failure (or stability) as a function of each of the key variables (rockmass quality, Q and the Scaled Span, Cs).

For the crown stability case the log odds expression (i.e. the relative probability that the outcome has, of falling into one of two categories) takes a form similar to an ordinary linear regression, as follows:

$$f(z) = 1 / (1+e^z), \qquad (14)$$

where $z = \alpha + \beta_1 ln(Q) + \beta_2 ln(C_S)$

- where z is the predicted log odds value,
- $f(z)$ is the predicted logit probability value, and α, β_1 and β_2 are the numerical coefficients assessed through the logit model.

The terms Q and Cs are respectively the rock quality and Scaled Span variables of interest.

This approach has allowed the statistical distribution of the case record Scaled Span data to be quantified to generate the suite of iso-probability lines shown across Figure 16. Each of these lines basically represents a Critical Span line for varying probabilities of failure (0.5% to 99.5% P_f).

This in turn allows definition also of the limiting 50% probability of failure ($F_c = 1$) line, analogous to the previous Critical Span expression, as follows:

$$S_C = 3.58Q^{0.44} \quad \text{(50\% Probability of Failure)} \qquad (15)$$

It is of note that in comparison to the original Critical Span equation, this updated Critical Span line for 50% probability of failure, (which incorporates the updated database records) follows very closely the original 1990 best fit line (Golder Associates, 1990; Carter 1992, Barton et al., 1974).

This mid-line, taken together with the other lines across the chart now allows very rapid assessment of possible risk for any known crown geometry and rockmass quality.

Such application of logistic regression analysis to rock mechanics related data sets is not new, the Mathew's Stability Method having been previously tackled in this manner by Mawdesley et al. 2004.

While use of this new chart has indisputably made almost direct assessment of the probability of failure quite feasible for almost any given excavation opening geometry, crown pillar thickness and controlling rockmass quality, this ease also seems to have created a tendency to ignore variability in input parameters and report only the single value probability that is computed from the equations given in Table 4 or interpolated off the chart. Consideration still needs to be given to defining ranges, means and standard deviations for the controlling input parameters used to compute C_S and Q, so that not only is the probability of failure computed for the means, but some appreciation can also be gained for variation in risk due to differences in geometry or rock quality. It is clear that failure probability tolerance based on even small variance in Scaled Span (C_S) or Q becomes very low for poor to extremely poor mass quality rock masses.

For these poor quality rockmasses a slight increase in excavation span, or decrease in rock quality significantly increases failure likelihood for the crown pillar. On the other hand, for good to very good quality rockmasses a much higher tolerance for variation in excavation geometry is evident.

While the logistic regression relationships shown on Figure 16 and listed in Table 4, as derived from analysis of the more than 500 points now in the crown pillar database, help towards improving our means to better estimate the relative probability of failure (*Pf*) for any crown and opening geometry (as defined by the Scaled Span, (C_S) for any inferred rockmass quality (Q), this must not be done blindly.

Consideration needs to be given to the most likely failure mechanism (Figure 5) and also to the probably range of rockmass quality. Because of the fact that failure probability tolerance, based on the Scaled Span (C_S) variance becomes so very low for poor to extremely poor rock masses, it is recommended that computations for these types of rock masses always consider a range of probable rockmass qualities, rather than simply making deterministic assessments of crown stability using single value Q estimates.

Sensitivity deterministic evaluations can be carried out assuming a mean and a spread of say one standard deviation of probable rockmass quality, along the lines of the approach suggested in Carter and Miller, 1995, as shown in Figure 17.

This diagram shows the original chart, but in concept the revised chart can equally well be utilized, and then estimates made of probability of exceedances of any of the regression lines shown on the chart, either graphically as per the illustration on Figure 17, or algebraically using the individual regression equations listed in Table 4.

Alternatively, rockmass quality variability can be treated more rigorously using probabilistic analysis methods, ranging in complexity from two point estimation methods (such as used by Hoek, 1989 for examination of inter-dependency of rock quality variables on Factor of Safety calculations for crown evaluations), through to use of Monte-Carlo and/or Latin Hypercube random variable simulation models to capture variability in actual rockmass conditions.

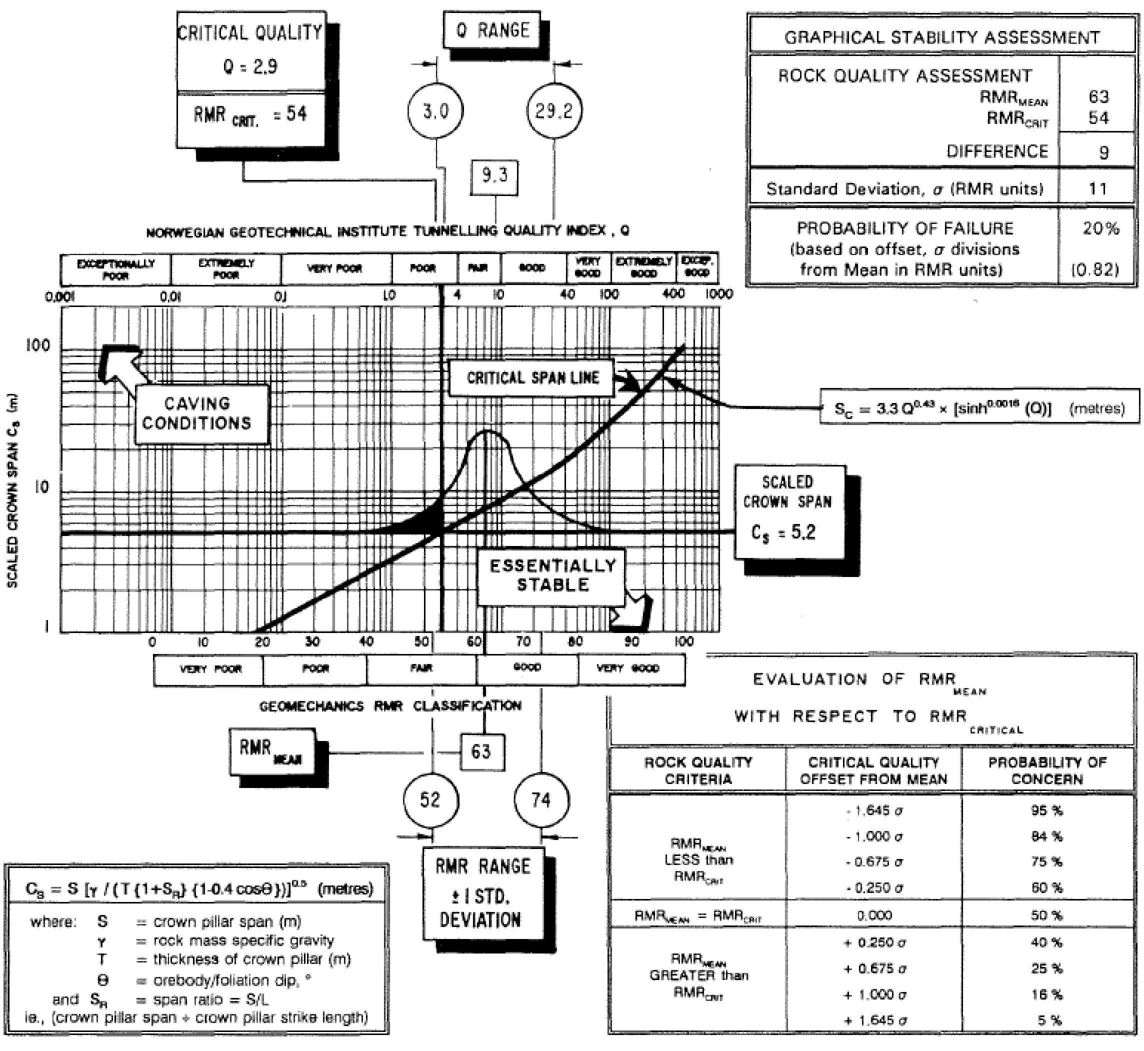

Figure 17. Example application from Carter and Miller, 1995 of method for considering variability in rockmass quality as a means to improve understanding of changes in probability of failure for a surface crown pillar of known geometry

Irrespective of the complexity or simplicity of the analysis approach chosen, it must be remembered that controlling rock quality can really only be properly defined once a viable failure mechanism has been postulated, (Figure 5). Estimation is then needed of the likely range of rockmass quality for that controlling rock mass segment, wherever it may be located – viz., the crown, the contact margins or the hangingwall or footwall. A representative Q histogram of rockmass quality should then be prepared so that appropriate characterization ranges can be considered for use of the Scaled Span chart. While direct assessment of the probability of failure can analytically be derived from the following expression:

$$P_f(\%) = 100 \,/\, \left(1 + 441 \cdot e^{\left(-6/F_C\right)}\right) \qquad (16)$$

…..where $F_c = S_C / C_S$, as previously defined, it is recommended that this equation also not be evaluated for a single value of rockmass quality, but rather calculations be made for the mean and also for a credible range of rockmass quality variation, either side of the mean.

As this might be more easily accomplished by direct input of Q values for the mean and for say one log standard deviation of Q either side of the mean (as per the estimation method shown on Figure 17), the exponent term ($-6/Fc$) in the above equation can be replaced with the expression …($-1.7C_S/Q^{0.44}$). This then allows direct calculation of P_f values either (i) deterministically directly from the Scaled Span chart or merely by substituting for C_S and Q or (ii) by use of more advanced probabilistic simulations to better describe rockmass variability.

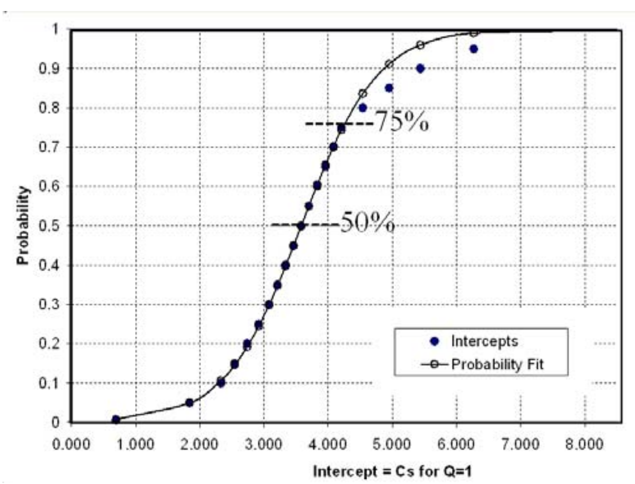

Figure 6. Probability of failure as a function of Scaled Span, C_S for $Q = 1$

As is evident from Figure 18, which plots the critical span intercept values for a Q of 1, derived with from this expression, this relationship generates a probability of failure curve fit that is near normally distributed for most of its range, with excellent matching between the actual logistic regression intercept points computed from the raw database records and the exponential curve fit up to a Pf of around 75%, with increasing divergence above. This mirrors the behaviour of the previously proposed expression derived from Latin Hypercube sampling evaluation (Carter, 2000), and reflects the increasingly non-gaussian distribution shape of the data spread as one moves to higher risk likelihoods.

As with the previous expression, it has been chosen to attempt to match the curve fit to the lower probability end of the distribution as this is the area which is, in general, of most concern and where this updated expression gives much improved results as compared with the earlier relationship. Comparison of the two expressions, in fact shows that the earlier relationship can overestimate failure probability by up to 10% in certain parts of the tail segment of the probability distribution, particularly for Pf <5%.

Thus, while the conservatism that is inherent in the original relationship has generally benefited initial screening assessments, its inbuilt pessimism at the low end of the probability scale has, in some circumstances, been seen as a hindrance to proper decision-making, where choices were needed to be made between expensive remediation measures in order to develop "walk-away" closure solutions.

Experience over the last five years since the introduction of this revised chart suggests that the chart allows much improved efficiency in assessing inherent risk associated with any new excavation under design, or any old excavation potentially requiring remediation measures.

7 GUIDELINES FOR SCALED SPAN EVALUATION

Designing for stability of near surface crown pillars over excavated openings requires an understanding of many factors including the excavation geometry, the characteristics of the rock mass, data on stress conditions, overburden loads, and ultimately an understanding of the relative degree of risk (factor of safety) associated with the planned near surface excavation, (Hutchinson, 2000). There are however two perspectives on design acceptability, and in fact a whole spectrum in between. At the one extreme, design for a crown may just be needed to be sufficient to maintain stability long enough to undertake underground excavation before open pitting down to the top of the previously designed workings. At the other extreme, many government regulators will be looking for essentially zero risk for public access over the top of a "closed out" crown pillar. This is realistically the situation demanded for civil tunnels and excavations where public access or buildings exist directly over near-surface underground excavations.

Collected experience from evaluation of the crown pillar database, as illustrated in Figure 19, clearly shows that there are two distinct time periods when a higher percentage of crown pillar failures seem to occur: – immediately with excavation and/or within a few years of service life and – much later, with this latter peak in the time dependency data being to some extent rock quality controlled, likely related to rockmass fabric degradation over time.

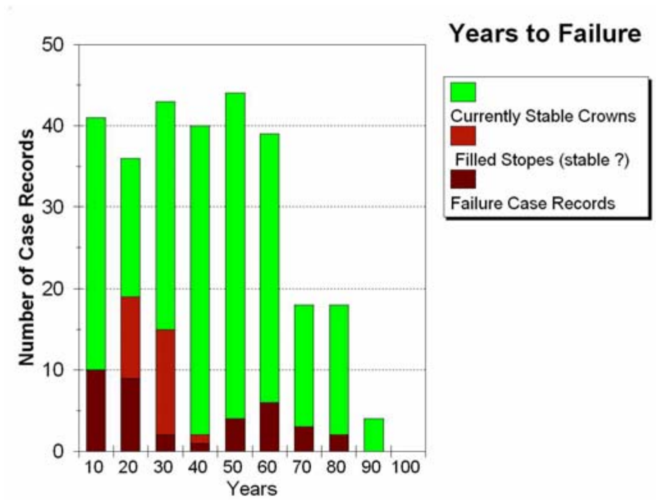

Figure 19. Time Dependency of Failure (from Carter & Miller, 1996, based on over 400 case records and nearly 50 failures in the Golder-CanMet Crown Pillar database)

Appropriate assessment of risk, to assess the likelihood of failure, must be cognizant of these observations, as deciding what level of risk is appropriate for a given situation governs decisions for both the design of new excavations, as well as for remediation of existing mine workings.

While the initial Scaled Span chart (Figure 12) was originally formulated as essentially a design chart for correctly sizing crown pillars over underground excavations, it has grown more to be used as a risk assessment tool than purely a design chart.

The updated chart (Figure 16) now includes iso-probability lines that define seven classes of stability state, based on their likely longevity / design ;life expectancy, based on extrapolated experience from stability assessments for geotechnically controlled risks as observed not just for crown pillars but also for other underground and surface excavations, as discussed by Cole (1987), Kirsten and Moss (1985), McCracken and Jones (1986), Priest and Brown (1983) and Pine (1992). While these stability classes can conveniently be plotted as shown in Figure 20 to readily define an appropriate stability state for a given crown geometry and rockmass quality, several facets of raw data input for such assessments require very careful consideration in order to ensure correct design decision making.

7.1 Estimating controlling rock quality

Much discussion in the literature on use of empirical design methods, such as Mathews, that include classification parameters such as Q' and that applicability of only Q' or RMR_{76}' is valid for use in the Hoek-Brown criteria has led to confusion with some practitioners with respect to use of Q in the Scaled Span method. Suffice it to say that the full Q with water and stress terms must be used so as to properly account for these important controls on crown pillar stability.

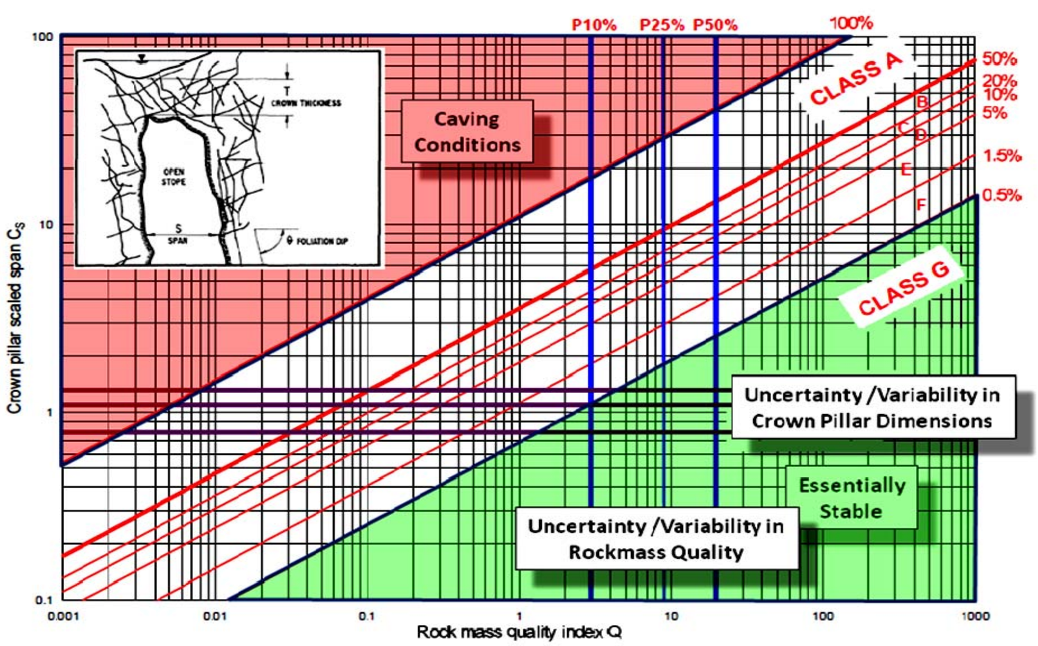

Figure 20. Example use of Scaled Span Chart with Probability of Failure Classes for assessing stability state for variable geometry and variable rock quality (as suggested by Richards, 2013, based on concepts outlined in Figure 17)

Irrespective of whatever the decided level of risk tolerance, considerable care is essential in correctly interpreting rock quality factors properly for the purpose of crown stability assessment. Precision in definition of the spread and variability of controlling rockmass quality is the most critical parameter for correct application of the chart.

As all the case records have been analyzed on the basis of controlling quality, so too should be the crown being considered for design. The aim should be to define the most likely controlling rockmass quality, which is generally not the mean for the overall rockmass, but often the Q for some part of the crown geometry or for some key lithological or structural component.

Proper understanding needs to be first gained of the presumed failure mechanism for the pillar under examination, (Figure 5) so that the proper Q value range for the key controlling rockmass element can be properly assessed. Then, the propensity, if any, for ravelling and/or degradation that could lead to variations in rockmass quality over time need evaluation, as long term stability is almost entirely dependent on rockmass competence state, given that the original pillar geometry was correctly sized for the mining conditions, and that subsequent mining does not bring about adverse stress changes.

7.2 Assessing change in rock quality and failure risk with time

Assessing stability state under long-term conditions remains one of the key problems for characterizing crown pillars for mine closure. The historical database provides a time history of stable and unstable cases, some of which were known to have been stable, from a general perspective point of view for many years before they failed. As explained in detail by Carter and Miller, (1996), and as shown in Figure 19, the two clearly different maxima in the failure cases occur within a decade or so of original mining and then after 60 to 70 years. The reason for these two maxima is likely completely different.

The initial peak is largely thought to originate because of design problems due to the crown thickness being underestimated or the rock quality less competent than expected. The second peak is however likely related to rock competence, and degradability behaviour. This difference is of importance in assessing longevity, as although the updated Scaled Span chart and evaluation methodology can be used to rapidly assess failure risk, cognizance must always be given to the fact that longevity of a given excavation will differ dramatically dependent on the rock type and initial quality, and whether or not support has been designed for permanence, or not.

All of the case record data continues to support the 1996 observations that suggests that from the viewpoint of long-term stability, there appear to be two basic, quite different rock mass behavioural characteristics; i.e.,

– the essentially non-degradable, competent rock types (hard igneous and metamorphic types and well cemented sedimentary units) which exist, tend not to spall and hence seem to survive,

… and …

– the degradable, weathering susceptible, weak or highly fragmented rock types, that most commonly fail in due course of time, due to disaggregation and spalling.

These latter rockmasses are of most concern when applying the Scaled Span methodology, as they are notoriously difficult to properly characterize.

Of importance also is the fact that the exact timing of the second peak is not just a pure function of time (i.e. always problems happen after 60-70 years), but rather on original mined stope void space as well as crown pillar thickness, rock quality, propensity of the rockmass to degradation and also stress changes that may occur with ravelling.

It is recommended, therefore, that when examining the potentially degradable rockmasses using the updated probability chart that some parallel evaluation also be conducted of caving mechanics, specifically checking whether bulking will be a feasible restraint on cave ravelling or whether chimney caving might be problematic (Bétournay, 2004).

Figure 21, which was presented in the 1996 paper, provides the clue to the puzzle of better defining a long term stability state and for estimating an appropriate controlling rockmass quality that can be utilized for assessment of crown stability for long-term design. The diagonal line of data points across the centre of the diagram represents the immediate onset of instability for an unsupported span, in, for example, a tunnel or drift. This is the classic “stand-up” time as defined by Lauffer, 1958, Bieniawski, 1973, 1989 and others.

The y-axis on this chart is time; time from initiation of excavation to onset of collapse. As is evident all of the crown pillar points plot way high on the y-axis, reflecting the many years it takes for ravelling to occur if the crown pillar is thick, and the stope does not choke off. By contrast the data points across the centre of the graph show a clear trend of increasing time to failure based on rock quality. Accordingly, as stand-up time is controlled by excavation span, the y-axis on this chart can also be thought of in terms of block size, with the smallest block sizes close to the bottom axis and the largest block sizes near the top.s.

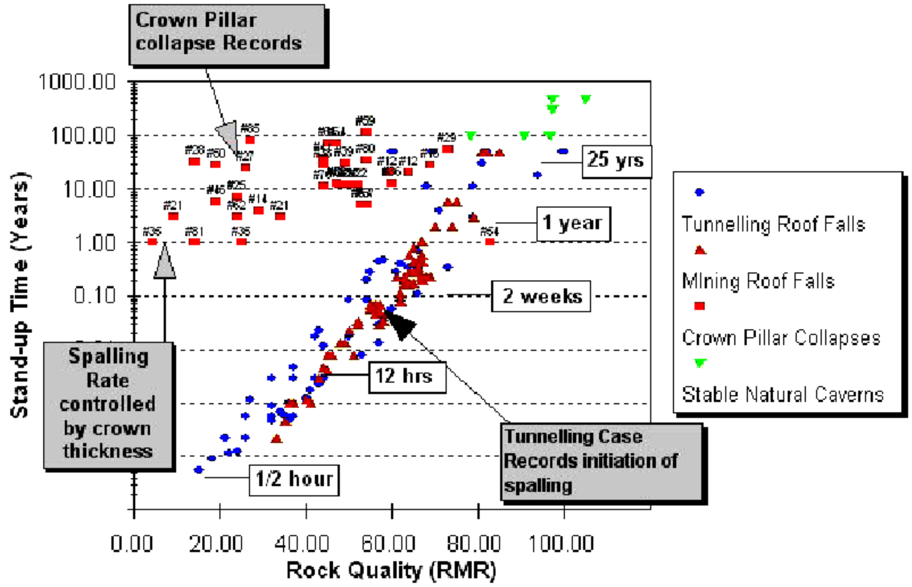

Figure 21. Rockmass Quality Controls on Time for Crown Pillar Collapse
as compared with Stand-up Time estimates from Tunnel Cases (from Carter and Miller, 1996)

Defining an appropriate rockmass quality Q value characterizing the controlling mechanism for a given crown pillar situation thus requires some judgement as with time there will inevitably be some degree of degradation for certain rock types, resulting in a change in effective rock quality over time.

For other situations, the enhancement provided by support will also degrade, which if an "improved-Q" has been defined (as per the method suggested in Carter et al, 1993) using the graph in Figure 22, this will also result in one moving left on the rock quality axis in the crown pillar Scaled Span chart, to a poorer and poorer effective rock quality. This in turn demands thicker and thicker crowns, as the only control on caving mechanisms is basically simply choking off the void space due to infill of the bulked rockmass into the remaining stope volume.

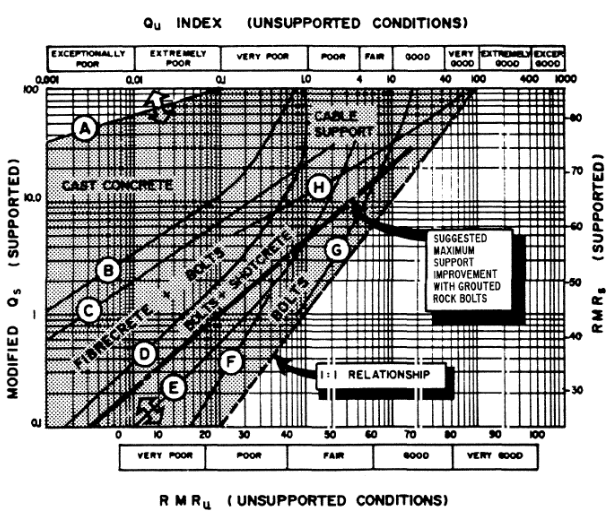

Figure 22. Equivalent effective rock quality "improvement" for different support types (from Carter et al, 1993)

Figure 23 illustrates the typical mechanism where caving of intact rock from the crown or the hangingwall falls as debris into the open void space of the mined working. As is obvious, if there is no or limited volume increase as rock blocks fall from the crown, then the void theoretically will eventually cave through to surface, ie., classic chimney caving.

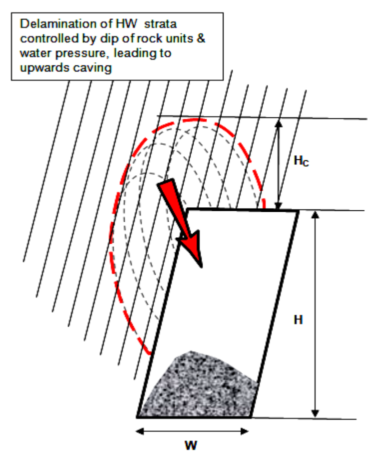

Figure 23. Typical crown and hangingwall cave progression in a steeply dipping rockmass

Almost all of the crown pillar collapses that have occurred years after excavation in poor rock qualities have developed because of progressive ongoing caving. In some cases, particularly for turn-of-the-century mines, no thought was given to the bonus provided by "free-muck", whereas in reality this "free-muck" was the product of upward caving of the stope crown or hangingwall. The fact that this caved muck was within the ore and thus made grade and moreover required no additional blasting or mining excavation cost outlay, was a bonus during the mining stage, but led to larger and larger open void space with no infill debris.

This inadvertently, in quite a number of the database cases, actually increased crown pillar collapse risk, as the additional void space created underground actually made matters worse, allowing progressive caving to continue unabated all the way through to surface, once the mine closed.

Assessment of the likelihood for a crown to cave through to surface within the life of the mine, or at some stage later thus clearly depends on the mass balance between available void space and infill volume of bulked broken rockmass. The bulking factor (sometimes called "swell" of the rockmass) as it caves is the critical parameter to determine in order to increase precision in these estimates.

Bulking depends on original rockmass quality and durability, on original block size and shape and also on propensity for degradation and breakdown, and last but by nowhere near least overall final bulking factors are a function of cave void height, as even large blocks falling from significant cave height will fragment more than those with less drop. Comminution and breakdown during compaction also mean that bulking factors decrease with age and depth into the debris. Much data is available for surface stockpiles on bulking of freshly quarried rock, less information is available on underground behaviour of different rockmasses. Many of the larger older cave mines, exploiting soft ores report bulking factors in the teens, as compared to some of the newer block caves and sandstone-rich rock cover zones above coal longwalls, which show bulking factors in the high 30's. Table 5 gives some ideas on the rock type dependency of different bulking behaviour.

Table 5. Bulking Factors for Different Rock Characteristics

Rock Characteristics	Typical Rock Types	Bulking Factor Range
Degradable, Weathering Susceptible, Readily broken down; Low LA Abrasions and Low Slake Durabilities	Many weak volcano-sedimentary rocks, shales, siltstones, mudstones, tuffs; Foliated, weak metamorphic rocks, schists, phyllites etc;	Low to Moderate 10% to 30%
Non-degradable, competent, durable, hard rock; High Los Angeles Abrasions and High Slake Durabilities	Most igneous rocks, eg., diorites, diabase etc, Competent metamorphic rocks, quartzites,, hornsfels	Moderate to High 30% to 50%

This to a large extent explains the difference n choking heights for different caves, and why with some rockmasses ravelling through to surface could be considered a real risk, whereas with other rockmasses the risk would be minimal. In consequence, if one is dealing with rockmasses with Q values way less than 1.0, then specific checks are recommended for defining an appropriate bulking factor for that rockmass, so that realistic estimates can be made of the ratio between probable mined stoped volume versus potential bulked caved volume. Establishing this ratio is key to defining to what height caving might progress before it chokes off – a factor of critical importance to establishing longevity for any surface crown pillar in weak rock.

7.3 Influence of Structure

The crown database and much of the preceding discussion has treated each crown pillar as basically comprising three different zones from the perspective of controlling rock conditions – the crown, the hangingwall and the footwall. In many cases, these zones are different lithologically and structurally. In some cases discrete foliation or shearing is a controlling factor. In all of these cases the rock quality of the weakest or most pervasive element should be chosen for defining the Q ranges for computation of the critical span S_C for use in the Scaled Span chart.

The difficulty that sometimes complicates crown assessment is deciding how much emphasis to put on defining the influence of faults and other major structures as these can completely dominate crown behaviour. Large weak faults that intersect a given crown must be considered the controlling weakness from the viewpoint of assigning appropriate Q values. Faults and other major structure must also be carefully considered with respect to evaluation of progression of ravelling and/or caving. Such features can totally over-ride natural break-back geometrical control, so must be carefully considered both in the design stage and for closure evaluation.

Depending on the angle of the fault structure with respect to the ore zone geometry and stoping zones, they may affect calculation of the effective crown span to be considered for C_S estimates. They also may exert considerable influence on cave mechanics and thus take control of potential extent of surface impact, as occurred at the Athens Mine in the 1930's (Obert and Duvall, 1967) and more recently at Ridgeway, Figure 24).

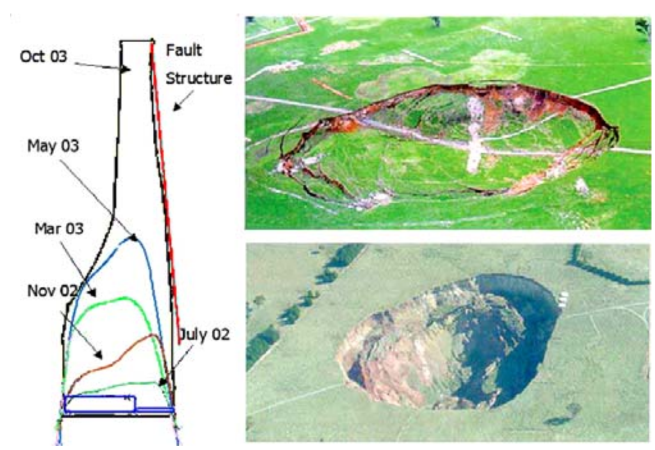

Figure 24. Influence on cave propagation of weak sub-vertical fault at the Ridgeway Mine, NSW (modified from Sainsbury, 2012, after Brunton, 2009)

In many such cases, adversely oriented faults not only can take control of back-break angle, they also often change cave rate and thus potential timing for surface breakthrough, by virtue of the mechanism change they create – from controlled caving due to ravelling and arch breakdown to more rapid chimney cave propagation along the fault. Well documented recent block cave mining cases, such as Ridgeway, where cave progression rates increased to three times normal cave rates seen from undercut initial development, support historical records from the Crown Pillar database that chimneying is a far more rapid process and therefore perhaps more problematic from a surface breakthrough perspective as it is harder to analyze (Bétournay, et al, 1994).

In the case of major structures, often there is a zone of more sheared rock that constitutes the controlling weakness. Similarly, sometimes within a metamorphic assemblage there may be weak schist zones. These types of feature need specific rockmass characterization in order to define appropriate Q/RMR values for use in the Scaled Span algebra. Use of the sub-parameter descriptors in the Q or RMR system (ie., Jr, Ja, Jcond) can aid definition of appropriate qualities for specific shear surfaces. The observational GSI chart, in combination with extrapolation of GSI from definition of the Jr/Ja quotient within the Q system, as per the following relationship from Hoek et al., (2013) can also assist markedly in reaching a satisfactory estimate of controlling rock quality for these difficult situations:

$$\mathrm{GSI} = \left(\frac{52.\mathrm{Jr}/\mathrm{Ja}}{1+\mathrm{Jr}/\mathrm{Ja}}\right) + \mathrm{RQD}/2 \tag{17}$$

7.4 Optimizing definition of water and stress terms

As indicated in Section 7.1, Q rather than Q' should be used with the Scaled Span method. However this raises another issue of concern to ensure that "correct" definition of rock quality is achieved for crown pillar stability evaluation. This requires rational definition of what stress state and what water term should be considered when using the Q system for rockmass characterization.

The crown pillar database records and hence the background to the Scaled Span design charts have been benchmarked to the complete Q, rather than to Q' specifically so that water and stress can be considered. Although Q and Q' both become identical numerically for many rockmasses, under typical water and stress conditions, i.e. J_W = 1, suggesting dry conditions, and SRF = 1, suggesting normal confinement. These may not be appropriate for many crowns where rivers and lakes exist above the crowns where Jw might have a lower value than unity, or for any thin crowns or crowns with any significant degree of weathering. In all these cases an SRF of 2.5 is suggested as per Barton's 1976 recommendations

For many cases, setting J_W and SRF to unity is quite reasonable for the stopes beneath the crown, however, in quite a number of situations these parameters can and should be set significantly differently reflecting imposed stress state or adverse groundwater conditions. An example case in point might be that one wishes to de-water an old mine, where the hangingwall rockmass contains bands of very low permeability schist. In such a situation drainage of the stope and of the footwall rockmass may be rapid and follow directly with the drawdown of the shaft or wherever pumping is taking place. Groundwater pressures in the hangingwall may however remain elevated, due to poor drainage through the schist bands. This in turn may lead to potentially significant differential pressures, between the drained footwall and stope zone and trapped water within the hangingwall. This, under worst case conditions could lead to hangingwall destabilization, such as shown in Figure 23. This sort of situation can be representatively modelled on the crown pillar graph using the J_W parameter to reflect the change in pressure gradient conditions. An effective lower Q will then be computed as J_W is matched to more and more adverse differential groundwater pressure.

Similarly, when de-stress influence occurs, due to for example, large-scale adjacent mining, such ad development of an open pit or large block cave or bulk stope excavation close to an underground crusher station. In such cases, the SRF factor for the crown pillar over the near surface working should also be altered to account for the change, leading again to an apparent decrease in Q, reflecting a lower stability state. The benefit of high horizontal stress clamping can also be replicated with a slightly increased SRF.

7.5 Accounting for overburden or lake bodies

In many situations, rock crown pillars exist beneath thick overburden cover. The question thus often arises, as to how one takes account of the impact that thick overburden, or high groundwater level imposes on crown stability. Utilizing the Scaled Span chart, two simple approximations have been found effective for considering these two situations. Instead of just using the natural specific gravity for the crown pillar rockmass in the geometry term for the y-axis on the Scaled Span chart, the influence of high overburden thickness or an overlying lake can be considered as a straightforward density increase in the geometry term; and if the stopes are open, so that a significant pressure gradient might also arise, as discussed in Section 7.4, an alteration can also be made in the J_W groundwater term within the rockmass quality definition.

Considering here just the effect of the additional dead weight of overburden, an equivalent increase in specific gravity can readily be computed to reflect this within the overall geometry term.

Although crude, these approximations allow some sensitivity assessment to be undertaken where these sorts of problems are of concern. However if the issue is significant, such as planning to undertake mining beneath tailings or beneath saturated overburden, this analysis approach using the Scaled Span chart should be thoroughly checked using numerical modelling techniques where the pore pressure controls within the overburden and within the upper part of the crown pillar can be more precisely examined.

8 RISK ASSESSMENT & DECISION CHART

8.1 Background

In many mining and civil engineering projects, qualitative, and in some cases quantitative risk assessments are becoming more commonly applied in order to attempt to rank and prioritize measures to deal with identified risks. The results of Scaled Span assessments are often used in these situations as part of decision matrices for hazard ranking

In discussions related to such risk assessments the question often arises as to which Stability Class included on Table 4 corresponds to the state when risks are synonymous with the public's appreciation of "being as low as reasonably possible". (ALARP). Questions also arise regarding definition of acceptability and Expected Service Life, as longevity, for closure, is a key issue.

8.2 Expected Service Life

As with all natural systems there is an expectancy that stability will deteriorate with time, and this is implicit in the guidelines included in Table 4. The recommendations on service/design life from the perspective of crown pillars, take into account the fact that real change from a longevity perspective will occur to stability state because of material changes in rock quality, stress and water pressure conditions, irrespective of any change in crown geometry due to frittering and fall-out from the crown and/or hangingwall slabbing effects if the underground openings beneath the crown have not been backfilled. Figure 25 attempts to put some time frame around the degree to which there is a direct relationship between expected crown pillar performance as listed in Table 4 and likelihood of failure, as extrapolated from the logistic regression Probability of Failure fits to the database of failed and stable cases.

In the chart in Figure 25, each of the Stability Classes shown on Table 4 have been plotted with respect to estimated "Design Life" based on potential ravelling rates suggested from Figure 21. The concept of a ravelling progressive breakthrough rate is illustrated also in Figure 25 by the two dotted lines towards the left side of the graph, showing time lines for ravelling and potential breakthrough for pillars of Stability Classes C and D, for which minimum longevity is estimated as being in the order of 5 and 10 years, minimum, respectively (ref. Table 4, column sevent). The dotted curves shown on Figure 25 are asymptotic to the upper axis, representing Probability of Failure P[F] = 1, ie., in terms of crown stability, breakthrough to surface.

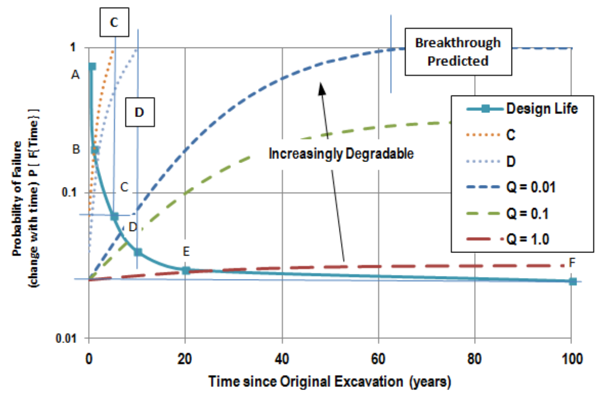

Figure 25: Conceptual relationship between Probability of Failure, Design Life Expectancy and Crown Pillar Longevity

Three curves have been plotted with dashed lines in the central part of the chart on Figure 25 to show potential change in P[F] with time for three different initial quality rockmasses, each with the same initial P[F] corresponding with Class F. Uncontrolled degradation has then been assumed, with the breakthrough time of 60 years set for the behaviour of the worst initial rock quality to match with the database records for the second peak of failure events (Figure 19). . For the best rock quality of the three plotted curves, the change in stability state is almost imperceptible, while for the intermediate rockmasses failure appears to extend to well past the 50-100 years service life expectancy.

Table 6. Observed residual subsidence duration over longwall mines (after Singh, 2003, from Sainsbury, 2012)

Reference	Country	Residual Subsidence Duration
Institution of Municipal Engineers (Anon., 1947)	UK	2 - 10 years
Orchard and Allen (1974)	UK	3 - 6 years (strong overburden)
Collins (1977)	UK	2 - 4.5 years
Grard (1969)	France	6 - 12 months
Brauner (1973)	Germany	1 - 2 years
Brauner (1973)	USSR	2 years (shallow mines) 4 - 5 years (deep mines)
Shadrin and Zomotin (1977)	USSR	0.2 - 2 years
Gray *et al.* (1977)	US	0.3 - 3 years
Hood *et al.* (1981)	US	1 year

These curves while conceptual do give an appreciation of potential magnitude of change in P[F] depending on rock quality. For the higher risk categories, Classes A and B, some verification of service expectancy can be gained from looking at breakthrough timing for full surface collapse following longwall extraction. Data on the time taken to reach full relaxation and cessation of residual subsidence movements above longwall coal panels, as summarized in Table 6 suggests that interaction is relatively short, typically varying between a few weeks and about 5 years, consistent with the suggested design life timings listed in column seven of Table 4. The geometry in all these cases can be considered super-critical (Figure 26), ie., surface impact is inevitable.

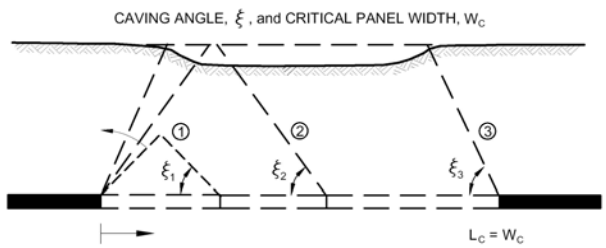

Figure 26: Definition Sketch for 1–Sub-Critical, 2–Critical and 3–Super-Critical Conditions for onset of Subsidence

8.3 ALARP risk

As discussed in Carter and Miller 1995 and in Carter et al, 2008 there is merit in using the Scaled Span approach as a ready means for defining likelihood of failure in the context of qualitative or quantitative risk assessments. Determining the acceptability of a given P[F] value though will depend on the perspective of the stakeholders involved in the decision making. The levels of risk that may be tolerable within an active mine site, for example within an open pit as one is mining down to the top of old workings, where necessary precautions can be taken, may be totally different from the perspective of a regulatory authority looking at a mine site closure report.

Pillar Failure Likelihood	Consequence Severity				
	Low	Minor	Moderate	Major	Critical
Almost Certain	High	Very High	Extreme	Extreme	Extreme
Likely	Moderate	High	Very High	Extreme	Extreme
Possible	Low	Moderate	High	Extreme	Extreme
Unlikely	Low	Low	Moderate	Very High	Extreme
Rare	Very Low	Low	Moderate	High	Very High

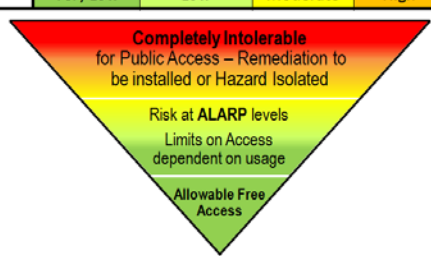

Figure 27: Typical Risk Matrix for Ranking Crown Pillar Stability for Closure Planning, with colour coded Decision Chart for selection of allowable post-closure usage

The type of hazard likelihood-consequence matrix chart shown in the top part of Figure 27 is in very common used for such risk evaluations, with appropriate subjective or quantitative evaluations of the cell contents in the matrix being used to aid the rankings of each key identified hazard. The triangle diagram in the lower part of the figure is less commonly applied, but shows where the ALARP concept of making sure that risks (in this case to the public, at closure) are as low as reasonably possible, fits in with moving from the definition of risk to assessment of mitigation measures.

9 GUIDELINES FOR REMEDIATION DECISIONS

In designing any new underground excavation, or designing remedial measures for an old excavation near surface, engineers and managers often are required to make decisions using charts such as included in Figure 27 to establish some acceptable level of risk associated with a particular situation.

The scaled span empirical approach not only gets utilized in these situations as a diagnostic measure for decision-making regarding closure options, but also gets used to help in deciding whether to remediate a given situation, or to fence it off and remove it from access by the public.

The acceptability of solutions can be quite different depending on the stakeholders. The public and regulators have a very different perspective as compared with folk within the mining industry who typically live on a daily basis with the sort of risk levels that crown pillars pose. In fact, even surface collapses are perceived differently as a consequence of familiarity, and expectation. This does not justify allowing collapse to occur, but it does provide some insight on measures that may be acceptable in different municipalities and jurisdictions.

In an area of still ongoing mining, even though many of the workings may now be defunct and the mines closed, fenced hazards provides an acceptable solution to the local population, as they have lived with these sorts of mining related problems for generations. An expensive full crown remediation with heavy capital civil construction costs, would thus not be seen as the optimum solution in many of these cases. However, if the zone of concern was located where a new highway or new school was planned, the approach would be different and the acceptability of the remediation solution different, although the risk of collapse would remain identical.

In Table 4 guideline equations for each iso-probability contour interval have been added for each category of exposure risk to help make use of the updated Scaled Span chart for assessing stability risk clearer for decision makers, moving forward on their specific projects.

For new excavations, such as a subway or water intake tunnel in an urban area, tolerance to risk is limited and the acceptable degree of risk must be very low. However, for remediation of say an area of existing 50 year old mine workings in a desolate region, still on mining property, the acceptable degree of risk against crown pillar failure could be higher. A higher acceptable risk tolerance allows for more cost effective remedial measures, such as fencing and signage to be utilized rather than adopting a more costly, arguably safer, alternative, such as backfilling or capping or plugging.

Figure 28 shows a matrix of different types of remediation solution amenable to different types of geotechnical characteristics of a given crown pillar and stope geometry.

Case	"Bridging" Solutions		Filling Schemes		
	Concrete Capping	Roller Compacted Concrete	Gravity Filling	Pneumatic Filling	Hydraulic Backfilling
Incompetent Sidewalls			●	●	●
Inaccessible Openings	●	●	●		●
Overlying Infrastructure				●	●
Flooded Openings	●	●	●		●

● Applicable Solutions

Figure 28: Matrix of feasible remediation measures for differing rock conditions (from Carter and Steed, 1990).

Two options can generally always be considered – remediation with complete public access or partial access, versus – remediation with isolation and no access or availability.

In the former case one would be envisaging stope backfill, concrete caps and/or similar civil construction measures as to make the crown safe for public use of the surface over the old stope zone. In the latter case the design approach would be to isolate the hazard far enough from public access that it would not present a risk for personnel or surface infrastructure.

9.1 Remediation with access

The risk levels and recommendations included in Columns 8 and 9 of Table 4 describing Public Access and the Attitude of Regulatory Bodies to Closure can be used directly to estimate the potential extent of work that may be required to bring a crown pillar of a certain stability up to adequate acceptability for public access.

Many tens of remediation projects have been undertaken using these crown pillar empirical guidelines as a framework for definition of measures for remediation worldwide. The approach has validity when used as a guideline but should not be considered more than a planning aid. As a minimum, detailed analysis and evaluation of potential risk levels for the crown and for the various remediation solutions should be undertaken bearing in mind longevity requirements, not just for the remediation measures, but also for the rockmass (as discussed in Section 7.2).

9.2 Remediation with isolation

One of the major issues for deciding to isolate a potential hazard from access is the extent of prevention of such access. In remote areas when fencing is put up around hazards, often these fences are broken down or otherwise damaged, to allow access to interested, inquisitive parties. In the Canadian North often ski-doo trails, rather than avoiding a hazard area, pass straight through, particularly if winter conditions have created high snowbanks so that the fences have been wholly or partially obscured. In many situations, curiosity of the public, particularly of small boys, leads to uncontrolled access to hazardous locations. The onus of responsibility for maintenance of fencing then becomes onerous if one considers that the hazard is real and sufficiently dangerous as to warrant isolation. In a number of older mining areas, legacy workings, which had broken through to surface or contained extremely thin crown pillars have been tempting to young rascals to go and explore them. Such bravado has resulted in some unfortunate fatalities. It is however not practical, nor economic to undertake full-scale civil remediation of all mine hazards as many thousands of open excavations and old workings exist in every country where mining has been undertaken for centuries. The maxim must thus be practical decision-making, based on minimizing exposure and reducing public risk, with the aim being to implement pragmatic cost-effective solutions, satisfactory to the three principal stakeholders:

- the Mining Company
- the general public and
- government regulators

9.3 Establishing Safe Set-Backs

Last but not least, one of the major issues with respect to closure and possible restrictions for public access, is how much land to cordon off to encompass the zone that could potentially be at risk for subsidence impact.

Public perception of underground mining is generally that it should involve no surface impact. For most hard rock mining situations this is true, as surface impact should be negligible to non-existent. It is only in the rare situations where a surface collapse occurs unexpectedly, or where significant disruption/subsidence settlement occurs on surface due to planned caving impact (e.g., from a block or panel mine operation, or from longwall coal extraction) that the public's awareness may be raised. While planned impacts can be a topic of concern to regulators trying to ensure that a mine closure is well managed; the bigger problem is for situations where unexpected surface caving could occur, such as clearly was the case for all of the crown pillar failure cases in the database.

Of concern here is attempting to characterize the unexpected, and the most appropriate means for rapidly assessing impact influence and risk likelihood.

Two methods have ready applicability for assisting in defining the extent and area of possible impact.

- subsidence strain estimation, and
- progressive hangingwall caving (PHC) analysis

Both approaches require knowledge of the geometry and geology of the mined zone and adjacent rock mass, so that potential for break-back post crown pillar failure and collapse can be evaluated.

9.4 Checks of extent of influence and likelihood for progressive hanging wall failure

Estimating the extent of potential break-back is not trivial particularly where no experience exists of previous collapse or caving in the particular geology. This again brings us back to looking carefully at the geological controls on potential failure, and then looking at the geometry of the mined excavations. Typically for a near-vertical stope in near-vertical geological structure, with competent hangingwall and footwall rockmasses, the potential for possible back-break and surface impact is usually relatively narrow, whereas for more shallowly dipping stopes in similarly inclined geological structures the potential surface impact width could be much wider. Ultimately, for flat or extremely shallow dipping excavations of significant plan area, (e.g., a longwall coal extraction panel) the impact from underground extraction will almost exactly mirror the footprint of the extraction panel with some break-back extension on the sides.

The two easiest approaches for assessing potential impact width, cover this spectrum - the approach used in soft rock mining of strain estimation as a means for predicting zone extent of surface impact provides arguably the best approach for evaluating the potential impact from shallow dipping workings. For steeper, and sub-vertical workings one of the best and simplest approaches for establishing possible setback is to use the progressive hangingwall caving analysis model (Hoek ,1974).

In the context of crown pillar design neither method should be considered as providing absolute magnitudes of subsidence or ground disturbance. Both should again only be used as diagnostic indicator methods for establishing potential back-break extent outside the excavation footprint. Considerably more information is needed to undertake evaluation of magnitudes of potential subsidence impact, depending on the strain created at surface by the underground extraction, which is a function not just of extraction size, but of depth, rock mass characteristics and extraction ratio.

10 CONCLUSIONS

Many of the most problematic legacy mine workings with remnant crown pillar were mined at the turn-of-the-Century. For such cases, crown pillar design had been arbitrary at best, purely based on precedent practice, and random at worst based simply on … "leaving just one more round to surface". Theoretical advancements to improve on this simple design approach prior to the 1980’s and the development of discrete fracture computer codes, such as UDEC and now various synthetic rockmass numerical equivalents, have so far met with little general acceptance for crown pillar design, because the complexities of the geometry and geology of the typical rock masses comprising such crowns are difficult to categorize and simplify for analytical calculation or modelling purposes.

Although this is changing to some degree today as modelling codes are increasingly becming more sophisticated and better able to tackle these problems, this certainly was not the case in the late 1980’s and this was one of the main rationalizations for the development of the Scaled Span method. Its original development was targeted to try to fill the gap between rules of thumb and use of analytic and computer modelling methods of analysis. As such it was based solely on calibration back to case record failure behaviour.

As the stability of any given crown pillar was recognized as being quite clearly three dimensional in nature, it was problematic that in most of the crown pillar case records little if any data on 3D geometry existed, let alone details of crown rockmass conditions. In most cases some information was available on general geometry, on orebody dip and mined thickness. Only sometimes was data available on ground conditions and rockmass competence.

Some approach to simplifying the problem was clearly necessary so that the salient observations from the failure cases could be synthesized and out of this perhaps a method for undertaking future analysis could be formulated. This led to the development of the scaling relationships.

Since the introduction of the original Scaled Span chart in 1989, further updates, including the addition of several hundred new database case records and modifications to the span definition to account for shallow dipping workings, have been completed; none necessitating any revision to the basic concepts.

Initially, the Scaled Span concept was put forward on the basis of assessment and back-analysis of over 200 case records of near-surface mine openings and crown pillars, including 30 documented failure cases; but this database now has been extended to over 500 cases with more than 70 analyzed failures. Ongoing development of the Method has generally been concentrated more on refining the analysis approach and on adding new crown pillar case records, to improve its applicability. The basic information required for conducting a Scaled Span analysis has remained essentially consistent, viz:

- defining Thicknesses of Rock Crown cover
- documenting Opening Spans and tunnel, cavern, drift or stope dimensions
- outlining data on Dip/Orientation of the principal structural fabric of the rock mass
- calculating and assigning Rockmass quality classification values, either as Rock Mass Rating (RMR_{76}, Bieniawski, 1976) or as *GSI* (Marinos and Hoek, 2000) or as *Q* (Barton et al. 1974, Barton, 1976), including assessment of water and stress terms, and
- tabulating and assigning available data on rockmass density and Hoek-Brown friction parameters, (Hoek & Brown, 1988)

Uncertainty however still remains in definition of many of the key geotechnical factors controlling stability state. In particular, the role of in situ stress within the crown pillar zone is still far from clearly understood. In certain situations, lateral clamping stresses are significant and in other cases they seem to be ineffective or completely absent. Theoretically, the presence of clamping stresses and the development of a compression arch within a crown pillar will significantly enhance the stability of the pillar. As it is of such importance, there is a case to be made that determination of the in situ stress state in a stable thin crown pillar would provide invaluable data to further current understanding of crown pillar behaviour in a marginal stability state.

11 ACKNOWLEDGEMENTS

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Over the past almost two decades of use of the procedures, studies for many of these same mining companies have contributed to advances in current understanding. While specific thanks is due to these organizations that have supported the work by supplying unpublished, often confidential information to form a case study data base, acknowledgements must also go to many colleagues at Golder Associates and other consultancies and universities who have added additional case records or made significant contributions to improvements in the formulation and testing of some of the methods outlined herein.

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