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The strength of the pillar-floor system

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ABSTRACT: The strength of the roof/pillar/floor system is controlled by the component with the lowest strength. In some coal seams the floor can be the weakest component and in these situations bearing capacity concepts drawn from foundation engineering can be applied. The low strength floors tend to be clay-rich and can be analysed as behaving in an undrained state (effective friction angle equals zero). A simple thin-layer bearing capacity equation has been found to correctly identify problematic low strength floors. The input variables are the unconfined compressive strength of the layer, its thickness, and the width of the pillar. All reported pillar collapses should be checked against this simple relationship and removed from the pillar collapse database if floor failure is indicated.

INTRODUCTION

A characteristic of many coal seams is the presence of low strength floors. The international coal industry makes reference to seat earths, underclays and fire clays. In Australia, the tuffs in the floor of the Wallarah and Great Northern Seam have sometimes, but not always, been reported to be very weak and have been implicated in unanticipated pillar behaviour. There can be weak claystones in the floor of the Bulli Seam. In the Bowen Basin, some of the early longwalls encountered major difficulties on the longwall face related to low strength floors. Both the South African and Australian pillar strength databases categorically state that there are no weak roof or floor failures in their databases.

There is no definition of what a low strength floor actually is: Is it less than a certain unconfined compressive strength or simply less than the strength of the coal? This lack of a definition, or even an accepted assessment process, can lead to poor mine design. A recent publication on geotechnical engineering in underground coal (Galvin, 2008) leaves this important question unanswered and dismisses earlier attempts to provide a simple assessment tool. This paper reviews the work on low strength floors and provides case study evidence that simple bearing-capacity methods do provide a useful tool: a tool that is in fact more robust than the empirical method for pillar strength itself.

ANALYTICAL AND EMPIRICAL MODELS IN GEOTECHNICAL ENGINEERING

Traditionally, pillar design utilises empirical methods based on a statistical analysis of databases of pillar collapse. The databases contain only pillar geometry and depth - there are no geotechnical parameters such as Unconfined Compressive Strength (UCS) or triaxial strength. The empirical approach does not invoke a failure mechanism and hence does not require the application of laws of physics. The engineering uncertainty in the subsequent design relates directly to the confidence in the data – in the South African database the coal seam and colliery are identified, in Australia not even the seams were identified and now cannot be as the database information has been destroyed (Galvin, 2010). The recommended factors of safety are based on an interpretation of a presumed normally distributed database of pillar collapse.

In analytical engineering approaches, there needs to be a behaviour model that can be interrogated using physical laws. Limit Equilibrium methods seek to calculate driving and resisting stresses at the point of failure. In some case, the arithmetic is very complex and numerical methods are used to estimate stresses - the behaviour model is still an input in terms of the selection of strength properties. Factors of safety are then based on engineering judgement recognising a number of uncertainties.

In an analytical method, uncertainty can be considered in three ways:

Model uncertainty

- Do we really know how the rock behaves?
- How well do the models represent actual behaviour?

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- Are complex models necessarily better?
- Are we applying the right model to the situation?

Parameter uncertainty

- Accepting the model is appropriate, how good are the inputs?
- Should we consider shear, compressive, tensile, or brittle strengths?
- What is the deformation modulus?
- What are the joint strength and stiffness properties?
- What are the joint orientations and spacings?

Human

- Bias, denial, ignorance, jealousy;
- Failure to conduct an appropriate site investigation;
- Arithmetic errors.

It is apparent that the appropriate values for factors of safety change (reduce?) as a project advances and knowledge improves.

It is worthwhile reviewing the history of research into floor behaviour and bearing failures in this framework.

FLOOR STUDIES

Seedsman (1988) studied the low strength floors in the Newcastle coalfield, with particular reference to the Awaba Tuff in the floor of the Great Northern Seam. At the time there had been several unanticipated subsidence incidents in which low strength claystones were implicated (e.g. Awaba, Chain Valley Bay, Gwandalan Point), and the early development of Cooranbong Colliery had hit major obstacles in attempting to enter the underlying Fassifern Seam.

The starting point for the study on the low strength floors was research from the Illinois Basin, which invoked foundation engineering (Stephen and Rockaway, 1981). To explain the failures, either very large factors of safety were required or the floor strengths were massively reduced with no precedent. There were two options - either dismiss the bearing capacity model and seek another failure mechanism or review the parameters. Bearing capacity theory is well established in civil engineering and importantly is scale independent (the magnitudes of the loads and the widths of the pillars are not material to the application of this elastic stress model). Bearing capacity is referenced in standard mining text books, typically Brady and Brown (1985). It was concluded the application of a bearing capacity model was appropriate.

The research then focussed on parameter uncertainty. The first thing to do was to determine whether low strength tuff behaves as massively overconsolidated clay. This means that when loaded quickly, the load is carried by the pore water (Figure 1). The key implication is that for rapid loading the effective friction angle is zero - over time the pore pressures dissipate, the clay consolidates (gains strength) and the friction angle tends to the drained values (about 25^{o}). From a practical viewpoint, the immediate strength is the lowest.

With acceptance of undrained behaviour, it was possible to return to the bearing capacity equations with a friction angle of zero - this was nothing special as a major part of soil mechanics practice is based on the same assumption. Many of bearing factors vanish when the friction angle equals zero.

But there was still a problem with the required factors of safety to explain failure when reference was made to the available core logging. It was known the tuff was layered, although the scale of the layering was not specifically quantified in much of the old logging of the tuffs. A major advance was possible by referencing any of several thin layer equations - the equation of Mandel and Salencon (1969) was used: Bearing capacity = UCS/2*(4.14 + W/2/h), where W is pillar width and h is layer thickness with an unconfined compressive strength (UCS).

Figure 2 provides some examples of what this equation implies. The horizontal axis is UCS, and the diagonal lines give the bearing capacity for different layer thickness for different pillar widths: for example a 1 m thick layer with a UCS of 1 MPa will have a bearing capacity of 7.1 MPa, which is greater than the pillar stress for shallow first workings but less than for deeper pillars in an extraction panel.

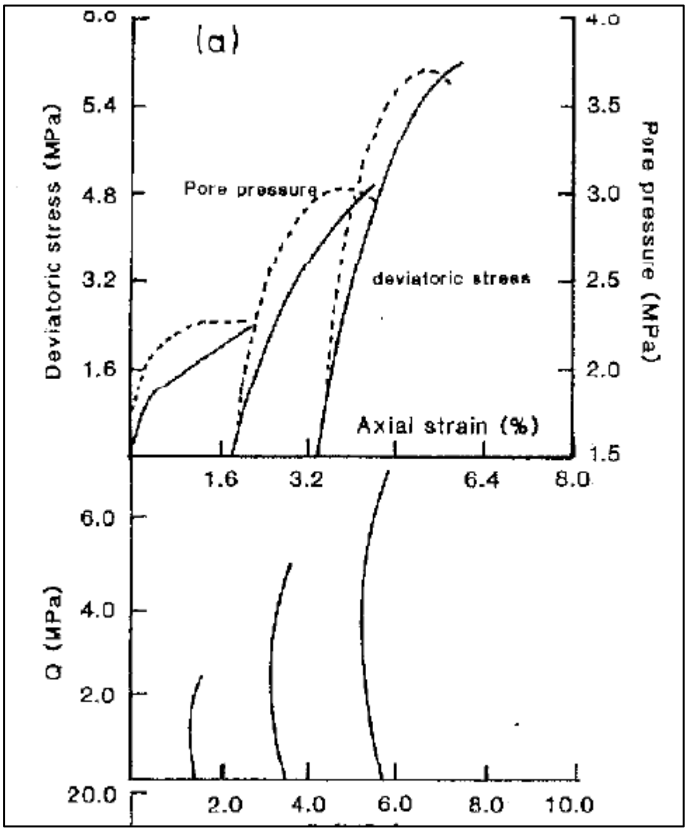

Figure 1 - Undrained triaxial test with pore pressure measurements of a sample of Awaba Tuff

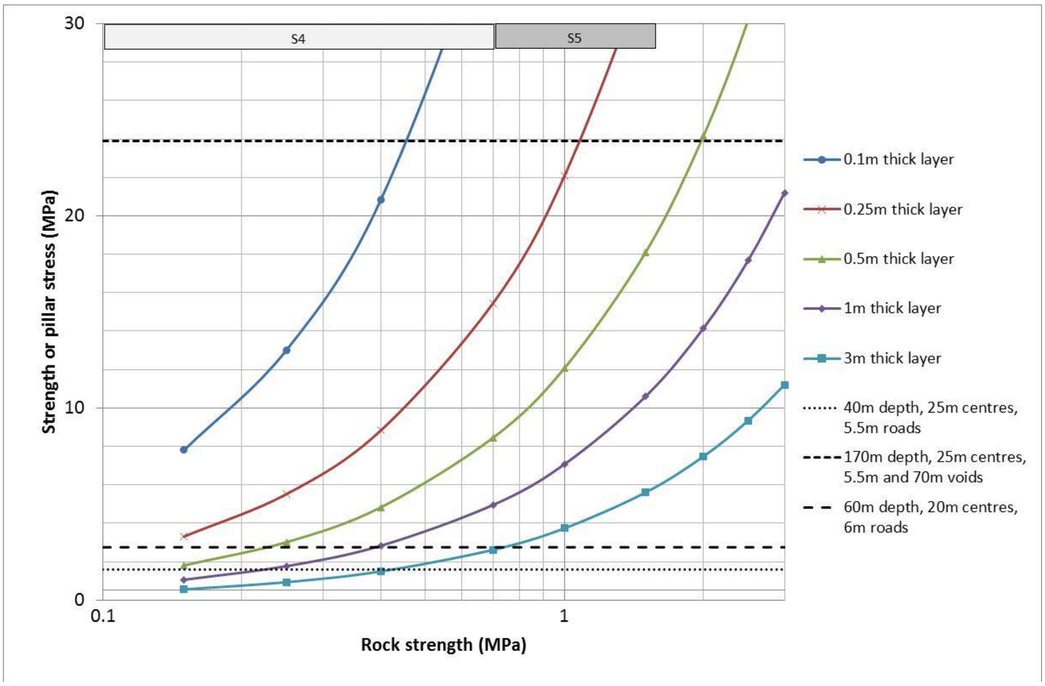

Figure 2 - Bearing capacity under 25 m wide pillars and a range of potential loadings

This thin layer bearing equation also informs the site investigation what may be required. The focus needs to be on thin layers of very low strength. Figure 2 also includes some field strength categories: S4 - readily crumbled by hand, and S5 - trim with knife, thumbnail scratches core. In this context, examples of good logging have been found from 1947 but none subsequently until the last decade once the insight from the research was clear.

There is a strong bias in the Australian coal mining sector against any simple method that requires site investigations – strangely this does not seem to apply to the use of complex numerical codes where the site investigation demands are extreme and many of the input parameters cannot be determined anyhow. Whilst site investigations were not adopted by older mine management processes, it was not because the ground is highly variable and hence cannot be adequately characterised. In fact the opposite applies: diagenetic (rock forming) processes produce more consistent rock masses than weathering processes which produce soils. The failure to commit to site investigations was the result of ignorance, perception of cost, inconvenience, and unfortunately denial.

The reluctance of the industry to accept this bearing capacity approach has been disappointing to say the least. Floor failures have been dismissed because smaller pillars may have been formed – if this logic was applied to pillar collapse we would have no empirical design at all. The different scale of mine pillars compared to civil engineering footings has been invoked even though the relationships and equations of elasticity in general are independent of scale. The most bizarre outcome, and based on denial, was the application of the bearing capacity model using high presumed strengths that not surprisingly showed there was no bearing failure: this was then used to argue there was in fact no hazard. No attempt was made to actually measure the floor strength or even to back-analyse pillar failures and creeps in the adjacent panels. The new panel subsequently collapsed on very low strength material.

SUCCESSFUL APPLICATION

Undrained failure of the floor under a coal pillar can have serious consequences (Figure 3). Localised floor heave can seriously impact roadway serviceability both in terms of loss of roof control and poor trafficability - if not loss of access. For thicker/weaker layers of claystone, lateral extrusion of the clay may cause the collapse of the pillar with consequent loss of access and possibly unacceptable subsidence outcomes. This range of adverse outcomes demands a specific assessment starting early in the project stages – waiting for a set of mine-specific case studies can be too late.

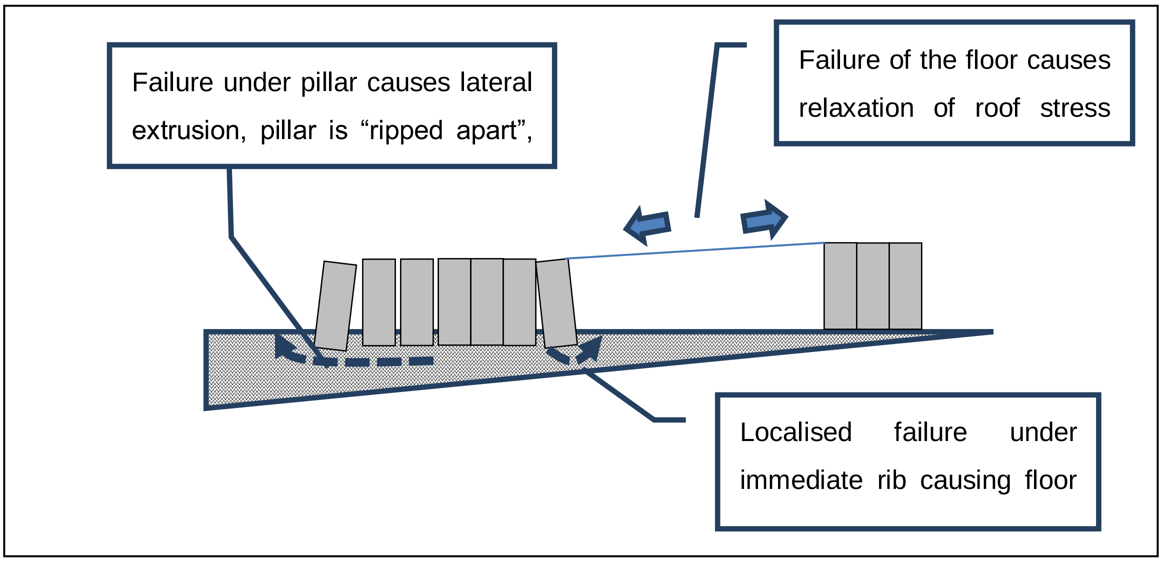

Figure 3 - Models for roadway distress associated with a bearing failure in the floor

Awaba

The last of the longwalls at Newstan Colliery extracted the West Borehole Seam under previous Awaba workings, close to the creep documented in Galvin (2008). Site investigations into the Great Northern

Seam, and a review of very old core logging, revealed the presence of low strength Awaba Tuff layers. In 1961 Cliff McElroy logged: SILTSTONE (?TUFF) very friable and easily powdered between fingers, thickness of 3'1" (0.94 m).

Using accepted field strength estimates, this would have a UCS of 150-700 kPa (S4). For pillars on 20 m centres with roads of 6 m width, this would imply a bearing capacity of between 0.9 and 4.1 MPa. The pillar stress would be 1.5 MPa at 30 m depth. With this knowledge, it is not surprising the pillar system failed. At Awaba, the collapses were delayed until the panel span allowed the failure of the massive overlying Teralba Conglomerate - once the critical panel span was exceeded, the collapse was almost instantaneous.

From 2007 to closure in late 2011, Awaba successfully extracted wide panels (less than 100 m) without generating a pillar creep. A key part of this success was the use of the thin layer equation applied to the results of coring underneath standing pillars. The rigorous assessment process gave confidence to the workforce, the mine owner, and government regulators that another creep would not be induced.

South Bulli

Roadways involved in the creep in W and T Mains in the 1980s were recovered about 10 years ago. The creep began during roadway development and before the longwalls were retreated. Removal of the floor material did not initiate new movements. The edge of the creep was clearly identifiable within about 5 m of roadway length. The inbye and outbye roadways either side had the same pillar and abutment dimensions. Site investigations revealed a 380 mm thick tuff unit with a strength of about 400 kPa. The bearing strength on development was 7.7 MPa compared to a vertical stress of 16.5 MPa.

More recently Wongawilli extraction was conducted along strike of the creep zone. When floor heave developed under a fender, there was also a tendency for the intersection roofs to unravel. Poor roof conditions had been observed during the recovery of the Mains but in that case it had been ascribed to the use of very early roof bolts. The roof destabilisation is probably related to the abutment relaxation mechanisms proposed by Diederichs and Kaiser (1999) such that yield of one side of a roadway can lead to de-stressing of the roof and the possible onset of tensile stresses. It has also been speculated that the same mechanism applies to pre-driven roadways on low strength floors.

THE INTEGRITY OF PILLAR COLLAPSE DATABASE

Of particular concern is the claim in the pillar collapse databases that there are no instances of floor failure. Without a specific assessment, how is the claim made? In the South African data, knowledge of the seam and the colliery allows local users in that country to assess the validity of the claim (van der Merwe, 2006). This is not the case for the Australian data base, which was always confidential and has now been destroyed (Galvin, 2010).

The SC3 data point

Colwell (2010) proposed the SC3 case in the Australian pillar collapse database (Salamon, *et al*, 1996) was drawn from the Great Northern Seam at Wyee Colliery. The floor of the Great Northern Seam at Awaba Colliery has been discussed above. Galvin (2008) also includes a separate discussion on the low strength floor Great Northern Seam and its association with seven unexpected subsidence events.

Old core records including those near the possible site of SC3 have been examined. The logging is not ideal from a geotechnical perspective but units with S4 and S5 strength can certainly be confidently identified. Based on that experience and as a default position, it is anticipated that a floor layer of 1 m thickness and 500 kPa strength exists and evidence from site investigations to demonstrate otherwise is sought.

There are additional problems with SC3 as over the recent years the reported depth, the goaf width, and the time to failure have all changed, and there is now no possible verification. If the stated SC3 dimensions are 170 m deep, a 20 m pillar, and 5.5 m and 70 m voids are accepted, then the extraction ratio of 83% will results and hence a pillar stress of 23.9 MPa can be calculated. Invoking 70 m voids gives more credence to the proposition that SC3 is from the Great Northern Seam with its massive Teralba Conglomerate roof. By contrast, the default bearing capacity would be 3.5 MPa. Floor failure is indicated, and at such low bearing strength extrusion of the floor and destruction of the pillar would be expected, which is consistent with the observations in Colwell (2010).

According to the rules of the database, there should be no failure of the roof or the floor. Removing SC3 will cause a major problem with the statistical analysis because this point is an outlier. The pillar strength design equation basically passes through this point. If it is removed, the pillar collapse databases for both Australia and South Africa are truncated at a width/height (W/H) ratio of 4.8 (Figure 4). The recent South African database also reveals a disturbing trend for collapsed pillars to have factors of safety well in excess of 1.2 while still constrained by the W/H ratio. Statistically there is zero probability of failure for pillars with width/height greater than 4.8. Pillar failure at greater aspect ratios could be due to unique combination of conditions, but it is wrong to extrapolate statistical trends and hence probabilities beyond a data base.

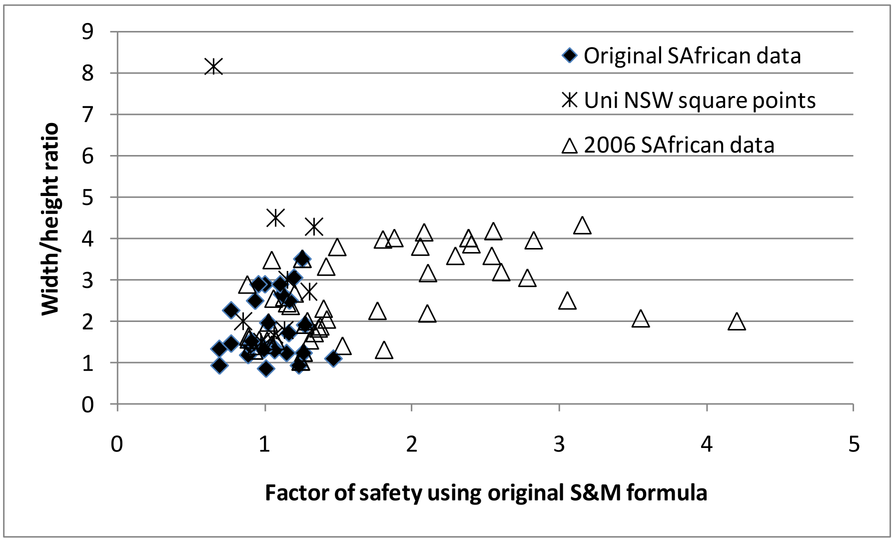

Figure 4 - Summary of South African and Australian pillar collapse databases

The question to be asked is why is the database truncated at a width to height ratio of 4.8? This should be the subject of more research. The author's view is based on the kinematics of pillar collapse (Figure 5). A highly structured coal can have a low UCS and hence low cohesion, but the friction angle must always be finite. It is necessary to separate the concept of pillar collapse from pillar crushing and compression. If collapse requires shear through the body of the pillar (Figure 5) than for a 15^{o} friction angle, kinematic failure cannot develop for W/H greater than 3.73. A ratio of 4.8 implies a friction angle of 12^{o}.

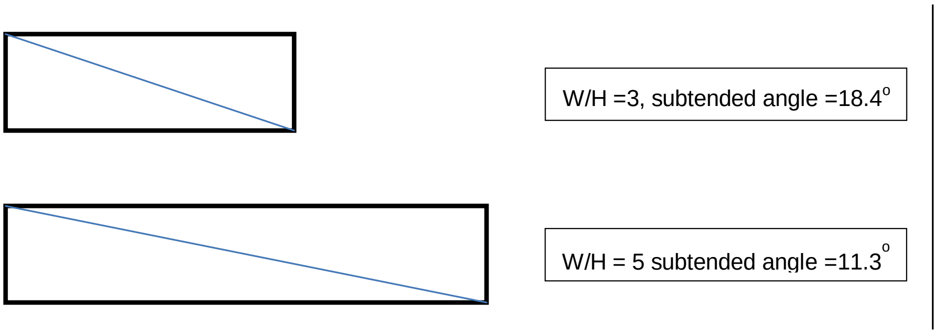

Figure 5 - Kinematically acceptable mechanism for pillar collapse

Implications to pillar design

Colwell *et al.* (1999) uses the Mark Bieniawski pillar strength equation in an empirical method for determining the requirements for tailgate roof support. Seedsman (2001) has argued that the success of the method is related to onset of yield in the pillar leading to de-stressing of the roadways. The Mark Beniawski equation may be a relationship for the onset of yield and not ultimate strength. The University of NSW method (Galvin, *et al.*, 1999) should not be used for chain pillars.

The University of NSW method is relied on by subsidence regulars searching for long term stable pillars. The method is conservative for pillars with W/H greater than 4.8. For subsidence, the design issue becomes one of considering both collapse and allowable deformations. The empirical data indicates collapse will not happen for W/H ratios greater than 4.8. There will be additional deformations. Figure 6 compares two approaches to pillar design for subsidence control: in both cases the pillars would have long term stability, but the design for 300 m depth of cover based on a W/H ratio of 5: would have a 12.5 m pillar with 52% reserve recovery versus a 22.1 m pillar with 36% recovery. There would be major improvements in mining rates if a place change system could be adopted.

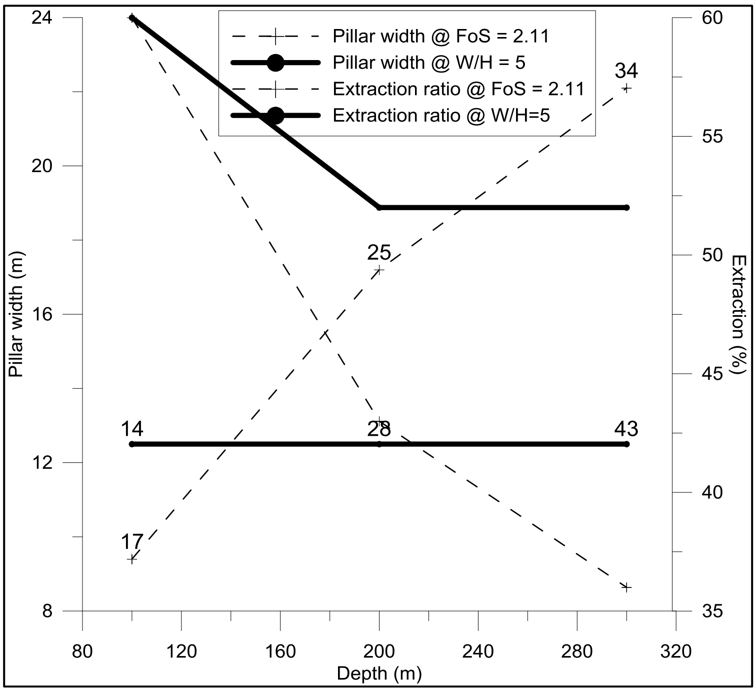

Figure 6 - Pillar width, extraction ratio, and surface subsidence (posted numbers in mm) for a bord and pillar operation in a 2.5 m thick seam for two different definitions of long term stability

CONCLUSIONS

The strength of the pillar system will be the strength of the weakest unit. The floor should always be characterised and the potential instability checked with the simple thin layer equation. Expert/more detail advice should be sought if the factors of safety are less than about 2.0 for greenfield sites or less than 1.5 where there is some precedent practice.

This is a remarkably simple test for weak floor (and by implication weak roof) that should be standard in every geotechnical toolbox. Its limitations are overridden by the simplicity of the calculation and the ability to do sensitivity studies. There is no justification for not collecting the data which will need to include a component of core or test pitting so as to check for thin very low strength layers.

Pillar collapse databases need to be exposed to this simple objective test. Currently, there is no basis for UNSW pillar design methodology for pillars with width to height ratio greater than 4.8. The method is massively conservative and probably results in unnecessary sterilisation of coal. Only the power relationship should be used, and not extrapolated beyond a W/H of 4.8. More research is required on pillar performance and especially the definitions of collapse, failure, and deformation.

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