Weak Floors and their Influence on Pillar Stability in Southern African Collieries

J W Latilla
Ingwe Collieries, Rock Engineering Department

A number of pillar collapse cases have been recorded in southern African collieries where weak floor rock was considered to be influential. In this study an attempt is made to analyse a few of these cases to determine if there are any relationships between them.

Initially a literature survey was carried out which concentrated on local as well as Australian and United States reports and papers. This was followed by collating data from four failed and four stable, Southern African case studies. Three and two-dimensional numerical modelling was done for each of the case studies as well as analysing each from a civil engineering foundation stability point of view.

It became apparent from the literature that pillar settlement and surface subsidence are more prevalent overseas while locally pillar failure with its corresponding safety implications is more likely to occur.

A few relationships were established during analysis of the case studies. However, it appears that only a small number of the currently worked seams in southern Africa have floors which are prone to failure, and that in these cases, it would be prudent to design pillars to higher factors of safety as well as to limit the width of panels. A provisional set of design guidelines have been proposed for the identified problem seams.

Two-dimensional numerical modelling was found to be a useful tool in studying this problem but considerably more geotechnical investigations are needed to evaluate the civil engineering design methods.

Introduction

The widely applied coal pillar design formula derived by Salamon and Munro (1967) has been used by and large successfully over the past 30 years in the design of bord and pillar workings in southern African collieries. However, over the years there have been several cases in which workings with relatively high factor of safety pillars have failed and the presence of soft floor strata has been judged to be a contributory or controlling factor. The most newsworthy of these was the failure at Emaswati Colliery, Swaziland, in 1991 that resulted in the temporary entrapment of 26 workers.

In order to avoid the problem in future there is a need to enable rock engineers to identify potential floor stability problems and adjust their coal pillar design accordingly. To assist in this, cases in which soft floor strata were associated with pillar failure are analysed together with cases in which the pillars remained stable even though soft floor was present. One case in which it was considered that the pillars were punching into weak roof strata is also studied.

Literature Survey

A number of publications and reports dealing with pillar foundation failure were studied to identify any learning points that may assist with understanding and analysing the following case studies. Most of the reports and papers studied were from South Africa, Australia and the United States of America.

The literature indicates that certain seams have floors more likely to fail than others, the 5 seam of the Witbank, and Highveld coalfields as well as the Alfred and Main seams in KwaZulu-Natal being especially prone. Sandier facies generally form weaker pillar foundations than those consisting of shaly material because they are able to absorb more moisture.

Australian and United States experience indicates that while pillar settlement is not atypical in certain areas, pillar failure is more likely to occur in southern Africa. This is probably due to the generally more conservative pillar designs practised in these two overseas countries.

Pillar settlement and floor heave as experienced elsewhere tend to be more of economic consideration than a safety hazard as they lead to surface subsidence and underground roadway closures. In southern Africa, pillar failures resulting from foundation instability can have significant economic consequences but they also potentially pose a greater threat to the safety of underground personnel.

It is relatively simple and inexpensive to determine moisture content as well as slake durability and Duncan free swell indices. As such, this data may be routinely gathered from core samples of the floor, especially in seams with a history of floor failure, such as the Highveld coalfield 5 seam, the 3 seam of the Free State coalfield, the Alfred seam in Kwa-Zulu Natal and the Main seam of the Zululand and Swaziland coalfields.

In three documented floor-monitoring cases, while there was floor heave and even some pillar damage, no mention of subsequent pillar failure is made.

In a number of cases mentioned in the literature, the average pillar width to height ratios are 2.96 and 4.29 for the South African cases and US / Australian cases, respectively. For the failed case study sites the width to height ratio is 2.1 to 5.6.

The presence of water (saturated floor rock) or dolerite intrusions is recorded in a significant number of cases both here and overseas.

While the use of numerical modelling has been evaluated in a number of studies a clear set of guidelines has not been established to assist with the design of pillars standing on soft floors.

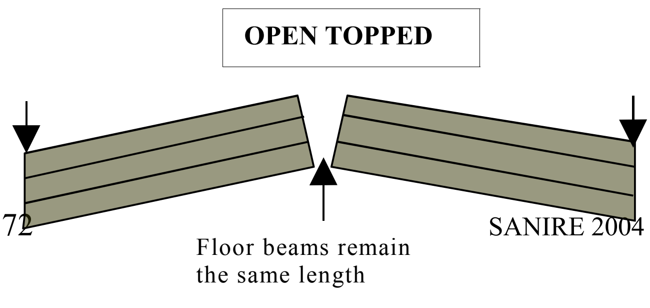
Failed cases

The study of a number of failed cases is considered important to enable the identification of any factors common to the various sites. While only four sites have been chosen it is felt that these represent a good starting point towards a fuller understanding of this problem.

Failed cases are defined as where there has been total pillar collapse or else severe pillar damage (indicating that the pillars have passed their peak load carrying capacity) and where, at the same time, the floor has shown signs of severe instability.

The following aspects, identified during the literature survey, have been investigated or carried out at the case study sites (where practical):

The case study floor failures have been classified as type I, II or "Newcastle", according to Peng, Wang and Tsang (1995) or Seedsman and Gordon (1991). Type I floor heave is open topped, indicating that the failed floor beams remain the same length, in other words they do not extrude from beneath the pillars (Figure 1). Type II heave is of the closed ridge form and results from weak material flowing from beneath the pillars (Figure 1). "Newcastle" type floor heave has as its central mechanism the shearing of the floor within a 1 to 2m wide "rib spall zone" around the edge of the pillars.



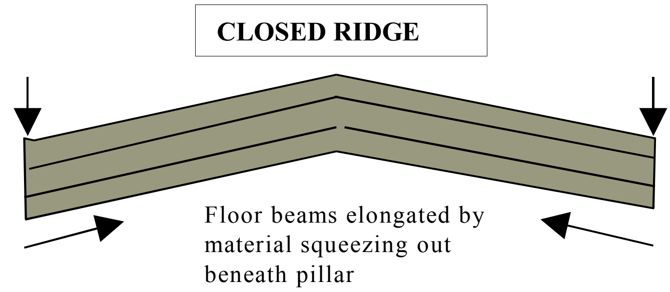


Figure 1. Open and closed ridge floor failure types

Coal mine roof rating (CMRR) developed in the USA (Molinda and Mark, 1994) and the floor rating (FR) developed from it (Riefenberg, 1995) have been determined at most accessible sites.

Floor strata have been classified according to the facies descriptions of Jermy and Ward (1988).

Table 1 gives the mining geometry for each of the failed cases:

Table 1. Failed cases - mining geometry and floor lithology

	Emaswati – main south	**Matla 1 - 5 seam R14 South**	**Welgedacht – Alfred seam stooping**	**ZAC – Mngeni shaft – MEN1**
Cover depth	65m	32.5m	165m	102 to 151m (average 126.5)
Seam thickness	2.5m average (1.9 to 3.1m)	2.4m	3.8m	2.74 to 2.80m (average 2.77)
Pillar centres	16m	11m	24.4m (80ft)	22m
Bord width	6.3m	5.9m	6.3 – 8.0m*	6.11 to 6.41m (average 6.26)
Safety factor (Salamon)	2.3 average (2.0 to 2.6)	2.26	1.18 – 1.51	1.72 to 2.70 (average 2.20)
Pillar width	9.7m	5.1m	16.4 – 18.1m	15.59 to 15.89m (average 15.74)
Pillar width/height ratio	4.1 average (3.1 to 5.1)	2.1	4.32 – 4.76	5.6
Floor lithology	0.5m sandstone and shale, underlain by 1.5m sandstone and 0.5m coal	0.5m sandy shale, very soft and prone to weathering	1.2m interlaminated sandstone and shale	Solid sandstone in one borehole but 0.22m shale and sandy shale underlain by 0.71m sandstone and 0.22m coal in another

*Due to weathering induced deterioration at the top of these old pillars (mined pre-metrication, i.e. before 1971), the actual bord width was as much as 8m in places.

Emaswati Colliery, main haulage south

On Saturday 8th June 1991, a major collapse of ground occurred at Emaswati Colliery situated near Mpaka in the Kingdom of Swaziland. Twenty-six underground employees of the mine were trapped by the collapse and were subsequently all rescued uninjured by the Chamber of Mines Rescue Brigade using the Rescue Drill based at the Colliery Training College, Witbank.

An entire drill and blast section's equipment was lost in this incident but work resumed with equipment obtained from other mines until financial factors resulted in the closure of the mine in 1992.

Convergence rates were in the order of 50mm/annum in 1990 increasing to a maximum of 200mm/annum at one station in May 1991.

After the investigation the following conclusions were made:

The surrounding dolerite intrusions are considered to be the primary cause of the pillar failure in this case, in particular, the underlying sill that resulted in the seam becoming devolatilised. Tectonic forces due to the dolerite intrusions induced slickensided vertical joints in the laminated floor strata. As the floor heave was predominantly open topped and not closed it is evident that little or no flow of weak floor from below the pillars occurred in this case. The overall failure mode was more akin to that experienced in the USA and Australia in that the pillars largely remained intact and the roof failed around them. However, in those two countries this type of failure is normally associated with closed ridge floor heave whereas in this case the floor heave was of the open topped variety. The pillars exhibiting the greatest damage were mostly in the central third of the panel, which would be the position of highest deflection of the massive sandstone roof beam.

Convergence rates were highest where less or no remedial work had been carried out. This would indicate that the relatively low restraint offered to both the pillars and floor by the limited sand filling that had been completed, had a significant effect in slowing down convergence.

Matla 1, 5 seam, panel R14 South

Surface subsidence of 1 to 1.3m was observed at the N° 1 shaft at Matla Colliery (Matla 1) on 26th January 1995 during an airborne gas pipeline inspection carried out on behalf of Gascor. It was found to have been caused by a collapse of pillars in the abandoned 5 seam bord and pillar workings. There was also an Eskom power line pylon over the affected area.

The area was mined in 1981 by continuous miner. Due to roof control problems, operations on the 5 seam were curtailed in 1983, from which time, only the underlying 4 seam was mined. Since the pillar collapse, the 5 seam workings have been sealed off close to shaft bottom (Latilla, Oldroyd and Wevell, 1997).

It was concluded that:

The soft sandy shale floor was further weakened by water and dykes as well as (possibly) the underlying dolerite sill. This weak floor material was then squeezed out from beneath the harder coal pillars. The floor damage has been classified as “closed ridge” type heave (Figure 1).

As the pillars were only 5.1m wide the effect of 200mm to 500mm wide slabs being torn off is significant (Figure 2). This was caused by the pillar foundations flowing into the bords (Jack and Madden, 1995). In the worst case, this would leave pillars with an effective width of only 4.1m. The resultant safety factor and width to height ratio of the reduced pillars would have been 1.3 and 1.7 respectively.

The initial failure occurred between the south-eastern barrier and the long pillars left along the dyke. This would be consistent with the load being applied by an elastic beam resting on the previously mentioned barrier and long pillars.

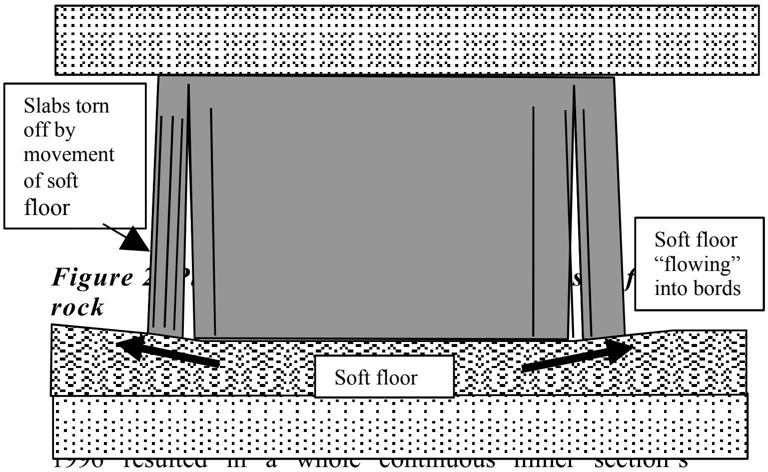


Figure 2 ... *rock*

1996 resulted in a whole continuous miner section's equipment being trapped during stooping operations on the Alfred seam.

The causes for the collapse were considered to be as follows:

The load on the pillars in this case was greatly increased by the following:

The goaf was suspected not to have reached surface.

The resulting bridging would have increased the load on the pillars awaiting extraction.

The collapsed area lay beneath the highest point of the mountain.

Coupled to this was the pillar weathering problem mentioned in the footnote to table 1, resulting in the actual safety factor of the pillars being as low as 1.18 even before the effect of additional loading by the bridging goaf is taken into account.

The immediate floor, consisting of 2m laminated sandstone and shale, was not capable of supporting the additional pillar load and started to flow out from beneath the pillar resulting in “closed ridge” type floor heave (Figure 1). Subsequently severe floor heave and accelerated pillar spalling occurred.

As the spalling was generally not cleaned up it provided considerable lateral constraint. The confinement, coupled with the pillar width to height ratio of 4.32, resulted in the pillars failing in a slow and controlled fashion.

ZAC, Mngeni shaft, panel MEN1

Pillar deterioration was first noticed during June 2000 in panel MEN1 at the Mngeni shaft of ZAC soon after completion of chequerboard pillar extraction of a block of ±40 pillars on the eastern side of the area under discussion. Chequerboard extraction entails the mining of every second pillar in a panel on a take-one leave-one pattern. Development was planned in-bye of this area and a "B10" (stone work development) crew was busy some distance in-bye of the pillar deterioration area. In light of the initially measured rapid closure of between 4 and 15mm over a two-day period, it was decided to suspend the "B10" operations ahead of this area until further investigations had been carried out.

Subsequently it was decided that the convergence was steady enough to allow subsequent development to proceed in-bye of the pillar damage area (PDA), subject to regular convergence monitoring and pillar support work. The pillars developed were subsequently chequerboarded on the retreat up to and including the PDA. Very tight control was exercised during pillar extraction of the PDA with "go / no-go" decisions made virtually on a daily basis depending on the maximum measured convergence rates.

Convergence ranged between 17 and 68mm/annum for the first seven months of monitoring, dropping to between 5 and 10mm/annum until secondary extraction began in the area. During chequerboarding convergence rates were generally in the order of 5 to 10mm/day although in one case 23mm/day was measured.

The pillar damage was found to be due to the following factors:

Chequerboarding of the eastern portion of the panel was the prime cause of the pillar deterioration as the increased load on the pillars caused pillar dilation as well as overloading of the core of the pillars. This was borne out by a reduction in the convergence rate after the initial chequerboarded area (on the eastern side) goafed.

Furthermore, the floor was weak in places and had been displaced along bedding planes. Observations indicate that the pillars had both punched into the floor and slid slightly down dip towards the bottom of the chequerboarded area. Displacement was indicated by the opening up of floor cracks and floor ride ridges. No signs of floor material squeezing from beneath the pillars was noticed. Zones of shattering in the pillars indicate that the floor cracks developed quickly. The floor damage can best be described as "open topped" floor heave but with occasional "closed ridge" features probably caused by pillar ride (Figure 1). Floor heave was more prevalent in bords than intersections.

The massive sandstone roof was the primary driving mechanism and the greatest damage was close to the edge of the chequerboarding area that was where the greatest roof deflection would be expected.

The high width to height ratio of the pillars in this area ensured that the pillar deterioration occurred in a slow and controlled manner.

It is possible that accumulations of water on the floor, in faces on the eastern half of the panel, may have further weakened the strata.

Stable Cases

While the study of failed cases is self-evidently important, it is considered equally important to investigate a number of stable cases in order that a comparison could be made between the two types. This is in order to improve prediction of floor and pillar behaviour. Studying only failed cases could lead to a one-sided appraisal of the problem.

Stable cases are defined as instances where, although the soft adjacent strata showed signs of damage, there were no signs of associated pillar deterioration.

Table 2 gives the mining geometries for the stable cases:

Matla 1, 5 seam, Panel R14 West

Panel R14 West, developed to the north west of the failed R14 South panel, was stable at the time of the failure event and indications are that it remains so to this day. Cover depth and mining height in these two panels were similar at around 34.9 and 2.3m respectively. In addition, the centres in panel R14West were 12m as opposed to 11m in the failed area.

Matla 2, 5 seam, Panel M12 North

This is a seven-road bord and pillar panel, of which eight of the twelve boreholes along it, or close to it, recorded the presence of soft floor. This panel was last inspected in the early part of 2001 when the pillars, roof and floor were found to be in good condition.

Two different types of floor heave were observed, one being low, narrow compression humps, generally less than 0.1m high and 0.5m wide, while others were the more classic form of floor heave where the floor cracked and slabbed in plates. The floor damage can best be described as predominantly the "closed ridge" type. Pillar conditions throughout the panel were good to excellent with the worst damage being occasional cracked pillar corners and ±0.1m of spalling.

Table 2. Stable cases - mining geometry and floor lithology

	Matla 1 - 5 seam R14 West	**Matla 2 - 5 seam M12 North**	**ZAC – Mngeni MN**	**ZAC – Maye, poor roof case**
Cover depth	34.2 to 36.4m	57 to 63.7m (average 59.3)	100.1m	98 to 111m
Seam thickness	2.2 to 2.4m	1.7 to 2.0m (average 1.8)	2.57m	1.47 to 2.04m
Pillar centres	12m	15m	22m	17m
Bord width	5.9 to 6.1m	6.2 to 6.5m (average 6.4)	5.98m	5.67 to 5.89m
Safety factor (Salamon)	2.4 to 3.0	2.6 to 3.1 (average 2.9)	2.93 (0.55 after chequerboarding)	2.1 to 3.5
Pillar width	5.9 to 6.1m	8.5 to 8.8m (average 8.6)	16.02m (10.44m after chequerboarding)	11.11 to 11.33m
Pillar width/height ratio	2.5 to 2.8	4.4 to 5.0 (average 4.7)	6.23 (4.06 after chequerboarding)	5.44 to 7.07

Floor lithology	0.16m sandy shale underlain by 0.26m interlaminated shale and sandstone	0.3m shaly sandstone	0.62m laminated sandy shale and shaly sandstone underlain by 0.19m coal and 0.51m shale. Total potential soft floor 1.32m	1.25m shaly sandstone and sandstone

ZAC, Mngeni, panel MN

Floor conditions were cause for concern during development of this panel, which at the time was mining into virgin ground. The floor was broken to a depth of ±0.15m.

After "turbo chequerboarding" (which is where additional slipes are taken from the remaining pillar to reduce it's final width to height ratio to just above 4), severe floor heave was reported in the "goaf" as was the fact that numerous mine poles had broken towards the centre of the panel. Fractures and cracks were opening up in the pillars and hairline roof cracks were forming as the extraction line approached. As a result, it was decided to change to normal chequerboarding for a couple of rows of pillars due to concerns of a failure similar to that at Welgedacht, where additional pillar loading caused the failure of the weak floor. This proved successful and the panel was chequerboarded back beyond this point without incident.

ZAC, Maye, poor roof case

This case has been included as it was assumed at the time of mining in this area that the coal pillars were punching into the roof. As such, this case is considered similar to the other cases of weak floors being punched by the pillars, only in this case, the problem has been "turned on its head".

Of most interest in this case is that a number of roofbolt bearing plates (washers) were observed to have buckled. As the thickness of soft roof was ±1.2m, consisting of laminated, micaceous, sandy shale, overlain by ±10m of massive sandstone, the 1.5m long, M16 rebar bolts used would have anchored in the sandstone. Tests on intact washers showed that they collapse at 105kN at which stage the deformation was 7.5 to 9mm. based on the average bolt spacing throughout the area the load on the washer from the dead weight loading of the soft roof was 49.74kN. It was considered reasonable to speculate that a portion of the excess stress responsible for overloading the roofbolts was due to horizontal stresses and buckling of the soft roof beam.

Case Study Comparisons

Table 3 shows ranges of values for the failed and stable cases:

Although the number of stable and failed case studies is admittedly low, a number of promising relationships are suggested. They are listed below in their perceived order of significance:

Table 3. Comparison between failed and stable cases

Aspect	Failed	Stable	Remarks
Ave. mining ht. (m)	2.4 to 3.8	1.47 to 2.57	Only 0.17m overlap
Pillar w/h ratio	2.1 to 5.6	2.5 to 7.07	No apparent relationship
Pillar safety factor (Salamon)	1.51 to 2.3	2.1 to 3.5	Only 0.2 overlap
Panel width (m)	116 to 271	74 to 205	For stable cases, 3 out of 4 were less than 130m. Slight agreement
Seam dip (°)	0 to 6	0 to 5	More stable cases where seam is relatively flat
Roof lithology	Massive sandstone	Generally sandy shale overlain by massive sandstone	Behaviour of softer band between top of seam and massive sandstone roof to be further investigated
Floor lithology	Sandy shale to interlaminated sandstone/shale	50/50 split sandy shale/shaly sandstone	No discernable difference between floor litho types
Soft floor thickness (m)	0.49 to 1.15	0.3 to 1.32	No apparent relationship
Geological structure	Dykes and faults present	Faults and /or dykes often present	No simple relationship between presence of dykes / faults and performance of floor / pillars.
Cover depth (m)	32.5 to 165	34.2 to 111	No apparent relationship
Pillar centre distance (m)	11 to 24.4	12 to 22	No apparent relationship
Average pillar stress (MPa)	3.8 to 7.5	3.3 to 6.5	In 3 of the stable cases it is over 4.4 MPa but even then, there is no strong relationship
Pillar joint spacing (m)	0.6 to 2m	None to ±1	No apparent relationship
Coal mine roof rating (CMRR)	60.3 to 64.7	72	Insufficient data at present
Impact splitting rating of roof	98 to 214	88 to 116.05	Roof stronger in failed cases
Width to height ratio of soft floor	10.2 to 30.2	9.3 to 28.7	No apparent relationship
Slake durability index **Weighted SD for 0.5m floor**	14.8 to 62.7 7.5 to 58.7	4.6 to 31.6 9.2 to 47.7	Lower range for stable cases but large overlap of values
Floor rating (FR)	4.5 and 20.5	17.5	No apparent relationship
Class of floor heave	2 x type I. Type II and "Newcastle" / 2 x closed ridge and 2 x open topped	Type I / 2 x closed ridge and 1 x open topped.	More type I than any other but fairly evenly split between open topped and closed ridge types.
Max. floor heave (m)	0.1 to 2.0	0.01 to 0.3	Overlap only 0.2m
Convergence rate mm/day	0.24 to 7.5	Not measured	No comparison possible
Surrounding workings	Virgin ground in 2 cases and adjacent to secondary extraction in another 2.	Generally in virgin ground	No apparent relationship
Pillar width (m)	5.1 to 18.1	5.9 to 16.02	No apparent relationship
Moisture content (%)	1.36 to 10.38	1.53 to 2.09	Limited data but there appears to be a relationship when compared on a seam for seam basis
Panel width / cover depth (m)	1.21 to 3.56	1.03 to 3.68	Similar range for stable and failed cases
MAP3D numerical modelling results	Tensile stresses in bords only in 75% of cases	Tensile stresses in bords only in 50% of cases	No apparent agreement between failed and stable cases. May be better when compared on a seam for seam basis with tensile stresses in bords and intersections only modelled for the number 5 seam.
PHASE2 numerical modelling results	Tensile stresses present in floor beam in all cases	No tensile stresses present in floor beam in all cases	Very good agreement
Vesic's modified equation (HF)	0.74 to 2.45	1.68 to 2.89	Indicated performance in 87% of cases

Pillars with a safety factor in excess of 2.3 (as calculated using Salamon's formula) were found to be stable.

Mining height - taller (not necessarily less squat) pillars being more prone to failure as a result of foundation damage. This is possibly due to taller pillars having a greater propensity to buckle in the near vertical axis.

Impact splitting (IS) ratings (Latilla *et al*, 2002) and composition of the seam **roof**, failure being associated with stronger roof conditions. Deformation of a thick and strong roof beam may be influential in loading of the floor through the pillars. It could also be that the floor damage is only the outward manifestation of already occurring pillar failure and that the floor does not fail first

Panels with a width of 130m or less seem less prone to failure.

Relatively flat seams (dip 1° or less) are less prone to failure.

Where floor heave is low (typically less than 0.1m) the pillars appear to be more stable.

It appears that the weighted slake durability (SD) index for the immediate 0.5m of floor may show promise in predicting the type of floor heave that may be expected (i.e. “open topped” or “closed ridge”). This is an interesting finding and while not relevant at present to predicting stability, it may be useful when applied to a larger database. Note that conventionally, a high swell index implies poor rock while, conversely, a high slake durability index implies good rock. For this study, to avoid confusion, it was decided to present the slake durability index as 100 - I_{d2}. This results in poor rock being assigned higher values for slake durability.

Numerical Modelling

Both three and two-dimensional numerical modelling was carried out to simulate the stress conditions for the failed and stable case study sites. While the lack of *in situ* stress field measurements precluded an analysis of the absolute values obtained from the numerical modelling outputs their value as tools for gaining a better understanding of the problems is recognised. As may be expected from elastic modelling, deformations were an order of magnitude smaller than those measured in the field.

The most important findings from the modelling are as follows:

The two-dimensional PHASE^2 model was successful in indicating zones of tensile minor principal stresses in the floor beams of the failed cases.

The three-dimensional MAP3D model was, in some cases, able to indicate where damage is likely to appear in the floor and, to a lesser extent, the pillars.

Civil Engineering Approach

The following conclusions have been reached with respect to the two floor bearing capacity methods assessed:

Terzaghi’s formula (Terzaghi and Peck, 1964), in its current form, does not appear to predict foundation behaviour for soft coal mine floor material. Further work is needed to determine the actual value for cohesion and angle of internal friction for weak coal floors.

Vesic’s equation, as modified by Speck (1981), predicted stability successfully in 75% of failed cases and 100% of stable cases. However, the number of case studies is small and considerable additional effort will be required in evaluating the applicability of this formula for South African conditions. With more accurate determination of moisture content, this approach may be suitable for the prediction of coal mine foundation stability.

Vesic’s modified equation shows promise as a tool to be used in determining the likely behaviour of soft foundations and may be included in the design guidelines.

Stability Rating

With no strong relationships evident, and a number of aspects showing weak agreement, it was decided to investigate the use of a rating based approach to the floor performance. This method works on the premise that as the number of factors identified with failure increases so too does the likelihood of instability.

As parameters such as safety factor and panel width are outcomes of the method they were not used in the development of the Floor Stability Rating (FSR) as shown in Table 4.

Table 4. Floor stability rating matrix

	Range of values				
Mining height	Thickness (m)	<2.3	2.3 - 2.8	2.8 - 3.2	>3.2
	Description	Low seam		Medium seam	High seam
	Rating value	**2**	**4**	**6**	**8**
Dip	Dip (°)	<1	1 - 3	3.1 - 5	>5
	Description	Flat	Slight dip	Moderate dip	Steeply dipping
	Rating value	**1**	**2**	**3**	**4**
Weighted average SD rating for 0.5m*	0.5m floor SD rating	<10	10 - 30	31 - 50	>50
	Description	Low slaking potential. Good floor	Moderate slaking potential. Moderate to poor floor	Moderate slaking potential. Very poor floor	High slaking potential. Extremely poor floor
	Rating value	**1**	**2**	**3**	**4**
Roof IS rating	IS rating for 2m of immediate roof	< 88	89 - 113	114 - 130	>130
	Description	Moderate to good roof	Good roof	Very good roof	Very good to excellent roof
	Rating value	**1**	**2**	**3**	**4**

The following characteristics were used:

Mining height – as this aspect appears to have the strongest relationship it is weighted as double that applied to any of the other three aspects. Note that mining height and seam thickness are not necessarily the same. It may be necessary to add the thickness of soft floor likely to be mined. As this aspect has the strongest perceived relationship it carries a weighting double that of the other factors as shown in Table 4.

Weighted average slake durability (SD) rating for 0.5m of floor. While no apparent relationship exists between this parameter and pillar failure it is influential in determining the type of failure expected. It is also considered a useful and simple characteristic to determine relative weakness of the floor material.

Impact splitting rating for the roof.

Seam dip.

Even though PHASE^2 numerical modelling (Rocscience) gave very encouraging results, in that the minor in-plane stress (σ_3) was found to be tensile in all the failed case studies, it is not used for this stability analysis as it cannot always be assumed that persons needing to design pillars on soft foundations will have access to the programme.

Applying the FSR to the case studies (and using average values where actual values were not available – shown in bold type in Table 5) the following values were obtained:

Table 5. Floor stability rating values for case studies

Case Study	Values from stability rating matrix				Floor Stability Rating
	Mining height	Dip	SD	IS	
ZAC, Maye - stable	2	1	**2.5**	1	6.5
Matla 2, 5 seam - stable	2	1	3	2	8
Matla 1, 5 seam - stable	4	1	3	2	10
ZAC, Mngeni - stable	4	3	1	**2.5**	10.5
Matla 1, 5 seam - failed	4	1	4	2	11
Emaswati - failed	4	2	2	3	11
ZAC, Mngeni - failed	4	4	1	4	13
Welgedacht - failed	8	2	**2.5**	**2.5**	15

Design Procedure

The following rudimentary design guidelines are suggested as an interim measure pending further work on the subject.

These guidelines are only applicable to the following seams:

Witbank and Highveld coalfields; number 5 seam.

Kwa-Zulu Natal and Swaziland; Alfred and Main seams.

Where the FSR exceeds 10, the safety factor of pillars must be designed so that the FSR / safety factor ratio does not exceed 4. This may be determined with the following simple formula:

$$SF_{FSR} = \frac{FSR}{4}$$

Equation 1

Where:

SF_{SR} = New safety factor adjusted for stability rating.

FSR = Floor Stability Rating as determined from the matrix (Table 4).

Panel widths should not exceed 150m with the continuous inter-panel barrier pillars being at least as wide as the pillars in the adjacent panel. Where the pillars in two adjacent panels are of differing sizes, then the inter-panel barrier pillar must be the width of the wider of the two pillar sizes. Note that the above equation is only valid for FSR ≥10.

Where the FSR is between 7 and 10 further investigation must be carried out. This should take the form of modelling the problem numerically, for example with PHASE^2, or alternatively, using the modified Vesic's formula (Speck, 1981) to analyse foundation stability.

Where the minor in-plane stress is tensile at any point in the floor when PHASE^2 is used then FSR must be taken as 11.

Where the modified Vesic's formula indicates a heave factor of less than 1.5 then FSR must be taken as 11.

A FSR of less than 7 indicates no likely foundation problems and normal design methods can be adopted.

Note that the above is a "first pass" assessment of a complex problem and due to the limited cases studied cannot be viewed as comprehensive. It does however provide a starting point in the design of pillars with potentially soft foundations. With more data it may be possible to simplify the approach to allow analysis by means of the matrix only. In this case, negative values may be assigned for high safety factors to effectively reduce the FSR to below 10.

Acknowledgements

This paper is a summary of a Research Report (Latilla, 2003) produced by the author towards an MSc at the University of the Witwatersrand, Johannesburg under the mentorship of Prof. TR Stacey.

The support and assistance of the management of Ingwe Collieries (a division of BHP Billiton Energy Coal) and Eyesizwe Mining is gratefully acknowledged as is their permission to publish this work.

References

Jack, B.W. and Madden, B.J. (1995). Matla Colliery, No 5 seam pillar collapse. SIMRAC interim project report. December.

Jermy, C.A. and Ward, J.R. (1988). The application of geotechnical testing to the mining of coal. SANGORM symposium. Rock Mechanics in Africa. November. pp 267 – 271.

Latilla, J.W., Oldroyd, D.C. and Wevell, E. (1997). Pillar failure associated with weak floor rock. Three case studies. Coal Pillar Performance Workshop, SANGORM (Coalfields Branch). August.

Latilla, J.W., van Wijk, J.J., Wevell, E. and Neal, D. (2002). Evaluation of the impact splitting technique used for predicting geotechnical conditions in underground coal mines. SANIRE 2002 Symposium. Re-Defining the Boundaries. Maccauvlei. September.

Latilla, J.W., (2003). Influence of weak floor on the stability of pillars in southern African collieries. Project Report. Dept Mining Engineering, University of the Witwatersrand, Johannesburg. March.

Molinda, G.M. and Mark, C. (1994). Coal mine roof rating (CMRR): A practical rock mass classification for coal mines. USBM IC 9387. 83pp.

Peng, S.S., Wang, Y.J. and Tsang, P. (1995). Analysis of floor heave mechanisms. Presentation at the Annual Meeting of the Society for Mining Metallurgy and Exploration, Inc. Denver, Colorado. March. pp 1 – 12.

Riefenberg, J. (1995). Towards a method of determining floor quality in an underground coal mine. 14th Conference on Ground Control in Mining. Morgantown, WV. pp 200 – 208.

Rocscience. (2003). Phase^2 v. 5.0 – Finite Element Analysis for Excavations. http://www.rocscience.com. Cited January 2003.

Terzaghi, K and Peck, R.B. (1964). Theoretical soil mechanics. Engineering Practice. John Wiley and sons, Inc. pp 137 – 1999.

Salamon, M.D.G. and Munro, A.H. (1967). A study of the strength of coal pillars. Journal of the SAIMM. September.

Seedsman, R.W. and Gordon, N. (1991). Methods to optimise mining operations affected by weak claystone materials. NERDDC End of Grant Report. Project No 1336. December.

Speck, R.C. (1981). The influence of certain geologic and geotechnical factors on coal mine floor stability – a case study. Proc. 1st Annual Conference on Ground Control in Mining. Morgantown. WV. pp 44-49.