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Report No: DPE/2017-1

From: Dr Ismet Canbulat

RE: Review of the Mine Plan and the Subsidence Risks Associated with the Proposed Mine Plan – Hume Coal

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1 BACKGROUND

Hume Coal Pty Limited has lodged development applications for both the Hume Coal Project and its associated Berrima Rail Project. The NSW Government Department of Planning and Environment (DPE) appointed the author to review the "Mine Plan" and "Subsidence Risks" associated with the proposed mine plan. This report summarises the findings of this review.

The project area is located within the Southern Highlands region of NSW, which is approximately 100km south-west of Sydney, in the Southern Coalfield, Sydney Gunnedah Basin. The area is also within the catchment of the Wingecarribee River, which confines parts of the Upper Nepean and Upstream Warragamba water sources. It is reported that the mine surface infrastructure area will be approximately 7km north-west of the Moss Vale town.

1.1 Information provided

The following information has been provided to the author:

- Hume Coal Project Environmental Impact Statement Main Report, prepared by EMM Consulting Pty Ltd, dated March 2017.
- Appendices A to U of the main report.
- A series of responses to questions from the advisors to DPE.

The subsidence assessment and pillar design studies for the Hume Coal Project were conducted by Mine Advice, which are presented in Appendix L of the main Environmental Impact Statement (EIS) report.

1.2 Proposed mining layout

The EIS states that due to groundwater and surface subsidence constraints, as well as the requirement to emplace the reject tailings from the CHPP, various mining methods were evaluated and a method similar to highwall mining has been adopted. In the proposed method, a series of long drives are formed up using a remote continuous miner in between coal pillars.

In response to questions from advisors to the DPE regarding what previous experience exists in this underground mining layout, Hume Coal has advised there being "none to our knowledge". Although similar layouts have been practised in the past (e.g., highwall mining), this layout is unique as an underground mine.

The proposed web pillar sizes are also unique as the reviewer is unaware of any recent pillar design study that systematically utilised such narrow web pillars for long-term stability.

An important requirement of this project is that the coal pillar system left behind will be stable over the long-term (using a suitably high Factor of Safety (FoS) against pillar failure and ensuring that the low pillar width to mining height ratio pillars are protected by suitably large pillars). Figure 1 shows the proposed layout and the different coal pillar types. Table 1 presents the proposed pillar dimensions for the shallowest to deepest sections of the mine.

Table 1. Proposed pillar dimensions (solid)

Depth of Cover (m)	Web pillar		Intra-panel barrier pillar		Gateroad pillars (m)		Main heading pillars		Solid barrier pillars	
	Width (m)	Length (m)	Width (m)	Length (m)	Width (m)	Length (m)	Width (m)	Length (m)	Width (m)	Length (m)
80	3.5	117.25	14.0	117.25	16.0	24.15	29.5	74.5	50.0	200.0
90	3.5	117.25	14.0	117.25	16.0	24.15	29.5	74.5	50.0	200.0
110	3.8	117.25	16.4	117.25	16.0	24.15	29.5	74.5	50.0	200.0
120	4.1	117.25	16.8	117.25	16.0	24.15	29.5	74.5	50.0	200.0
130	4.4	117.25	18.0	117.25	16.0	24.15	29.5	74.5	50.0	200.0
150	5.1	117.25	20.7	117.25	16.0	24.15	29.5	74.5	50.0	200.0
170	6.0	117.25	22.8	117.25	16.0	44.15	29.5	74.5	50.0	200.0

The web and intra-panel barrier pillars will be formed using a drive width of 4m. A roadway width of 5.5m will be used in the other panels.

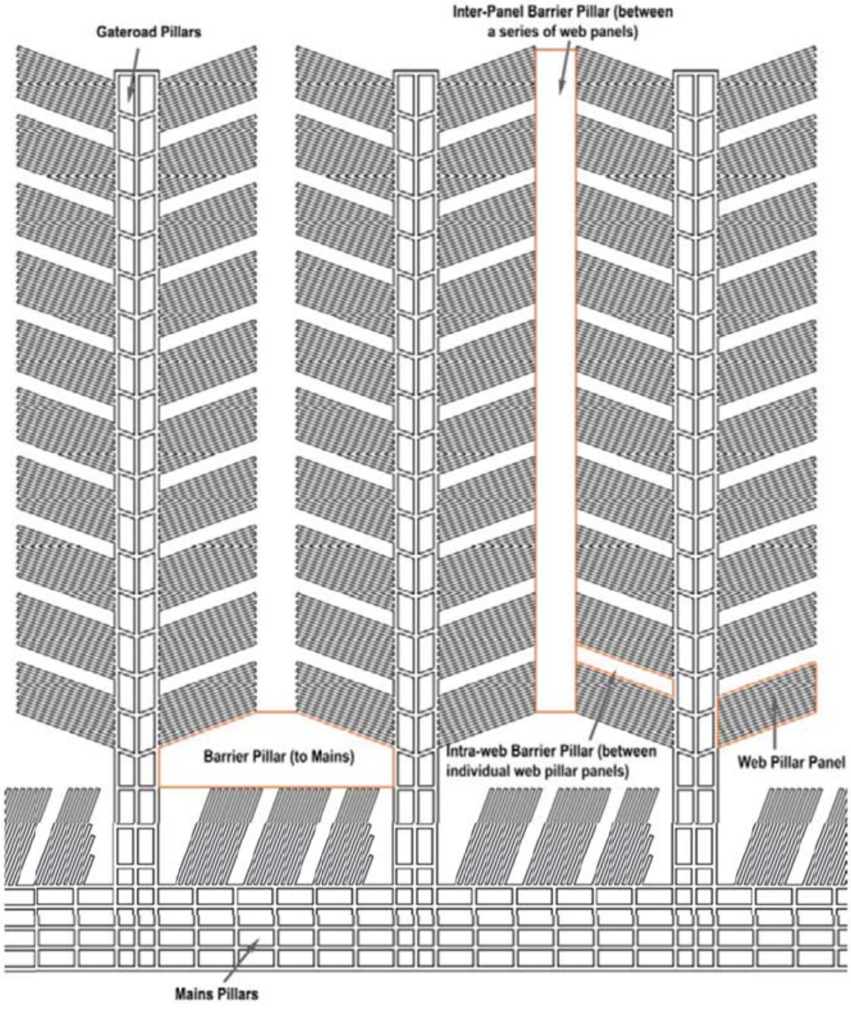

Figure 1. Illustration of the proposed layout showing the different coal pillar types (after Mine Advice, 2017)

The Wongawilli Seam thickness varies from 3m to 7m. The proposed maximum working section height is 3.5m, which contains disseminated mineral matter interspersed with stone bands. Due to quality issues, it is proposed that approximately 3m of coal in the roof and 0.5m of coal in the floor will be left.

In the proposed layout, the web-panel widths are kept relatively narrow (i.e., 54m to 58.6m) in order to ensure that the panels are in a sub-critical state and that the overburden bridges between the intra-web pillars. This concept has been applied in many mining projects to limit the pillar loading and to minimise the surface subsidence, including in highwall mining.

The proposed backfilling allows the mine to store tailings underground and to provide some form of confinement to the proposed web pillars. The Mine Advice reports make no specific reference to the timing of backfilling. In addition, no information regarding the properties of the backfilling is provided in the report.

The Mine Advice report stated that as soon as the panels are mined-out, they will be sealed off to allow the mine workings to flood, which also suggests that the backfill will be emplaced following completion of the web-panels. Mine Advice has also suggested that the maximum generated water head in the mine will be 120m, which is equivalent to a hydrostatic pressure of 1.2MPa. A bulkhead seal design for this purpose was completed by Mine Advice.

1.3 Summary of site characterisation

The Wongawilli Seam in the target area is near the top of the Permian Illawarra Coal measures and is overlain by the Triassic Hawkesbury Sandstone and the Wianamatta Group shale sequence (where present).

An important unit in the overburden is the Hawkesbury Sandstone, which contains conglomerate, fine to coarse grained sandstone and siltstone units in beds ranging from thinly bedded to thickly bedded (0.06m to >2m). This competent unit varies in thickness from 80 to 120m. It is noted by Mine Advice that weathering can be detrimental to the material strength of Hawkesbury Sandstone, as observed in the western part of the mining area within 5 to 10m of the Wongawilli Seam roof. The impact of weathering on the spanning capabilities of Hawkesbury Sandstone is not emphasised in the EIS.

Hawkesbury Sandstone is overlaid by Ashfield Shale, which typically consists of siltstone, shale, mudstone and carbonaceous bands. Laboratory testing results and bedding thicknesses have not been provided for the Ashfield Shale. From descriptions and other references, it can be assumed that this unit is weaker than the Hawkesbury Sandstone.

The Kembla Sandstone has a thickness of 10m to 15m and directly underlies the base of the Wongawilli Seam. It consists of siltstone and sandstone, with occasional mudstone near the lower contact with the underlying American Creek Seam.

The following rock mass characteristics have been summarised in the Mine Advice report:

Table 2. Proposed rock mass properties for the coal, roof and floor

	UCS (MPa)	Young's Modulus (GPa)	Poisson's Ratio
Hawkesbury Sandstone (overburden)	43.0	9.9	0.25
Coal (Wongawilli Seam)	8.5	2.4	0.25
Kembla Sandstone (floor)	68.0	11.0	0.22

Further details with regard to site characterisation and groundwater can be found in the EIS report.

2 PURPOSE AND SCOPE

The required purpose and scope of this review are outlined under a document entitled "*Scope of Work Hume Coal Project – Independent Expert Assessments*", dated 08/06/2017, which summarised the following key deliverables/scope of works for this review:

> The EIS describes the conceptual "pine feather" mining method, which is yet to be used in NSW. The Department requires expert advice:
> - to confirm that the levels of subsidence resulting from this method would be as predicted in the EIS;
> - about the underground safety aspects of using this method; and
> - about the risk of subsidence impacts and environmental consequences to natural and built features, including groundwater aquifers.

The aforementioned scope is not presented in any particular order in this report; yet all aspects are addressed in subsequent sections.

3 REVIEW APPROACH

As part of this review, numerical and empirical modellings have been used to assess the stability of pillars and expected surface subsidence in the cases of stability and failure of pillars.

In order to asses pillar stabilities, the UNSW power-law pillar strength formula has been used. The fundamental principles of the formula were highlighted, and the applicability of the methodology to the Hume Coal Project has been discussed.

Numerical models have been used to assess the loads acting on the proposed pillars, the stability of pillars, surface subsidence and the impact of backfilling on pillar stability.

This study has utilised 2 dimensional (2D) codes, FLAC (Itasca Consulting Group, Inc. v 8, 2017) and RS2 (Rocscience Inc.'s v9, 2017).

In the first stage of the modelling study, the pillar stabilities at different depths were assessed by calibrating the Mohr-Coulomb criterion input parameters to the UNSW pillar strength formula (Salamon et al., 1996). The effect of backfilling on the stability of the pillars has also been assessed using FLAC.

In the second stage of the numerical modelling study, potential surface subsidence impacts were assessed using RS2 as well as elastic and inelastic material properties. Where applicable, input parameters provided by Mine Advice were used.

4 GENERAL COMMENTS ON THE PROPOSED DESIGN

Groundwater and inrush risks

An important consideration regarding the groundwater is the direction of mining and the dip of the seam. Down-dip mining can cause significant problems and may necessitate extensive water management. It is understood that Hawkesbury Sandstone is a productive water source within the project area. The target seam, the Wongawilli Seam, is also an aquifer, which is linked to the overlying Hawkesbury Sandstone. Many Australian mines manage ground water and underground bulkheads effectively. However, in the past there have been a number of bulkhead failures due to piping through rib coal or leakages and hydrostatic pressure build-up, resulting in strata failure around bulkheads.

It is understood from the response of the mine to the reviewers' questions that majority of the panels have been designed so that, where possible, they are down-dip from the mains, not up-dip. However, in certain parts this will not be possible; therefore, a detailed assessment of the risks associated with flooding and the appropriate controls in active and previous panels should be conducted.

Geological discontinuities

Geological structures can play a significant role in determining pillar and roof stabilities. The Mine Advice report suggests that geological discontinuities can simply be mapped underground and then extrapolated for future panels, allowing the mine to manage them.

It is understood from response to the reviewers' comments that the geological mapping will mostly be conducted in surrounding excavations to infer the structures. This technique is mostly accurate for major geological structures. However, in many cases this technique can be ineffective. It is therefore recommended that a management plan should be implemented in order to predict and manage geological structures.

Another critical consideration in determining the rib and pillar stability is the cleat and joint directions, which are not considered in the EIS. As the proposed web pillars are uncommonly narrow, an unfavourable cleat and/or joint direction can impact the rib and the pillar stability. As we will be seen in following section, even minor pillar spalling has the potential to impact the pillar stability.

Roof stability

Whilst not emphasised by Mine Advice in the EIS, another critical consideration in long, unsupported drives is the roof stability. Mining operations can be severely hampered if roof falls occur in the entries. Hence, it is ideal for entries to be fully mined as soon as possible in order to avoid the time dependent deterioration of the roof. Although it is possible for the CM system to tolerate minor roof falls less than 0.1-0.2m in thickness, major roof falls, exceeding 0.3m in thickness, can damage the equipment or even cause it to be buried (Shen and Duncan Fama, 2001). In addition, roof falls cause adjacent web pillars to increase their pillar height, in turn reducing the FoS and pillar width to mining height (w/h) ratio, ultimately resulting in a decrease in pillar stability (Shen and Duncan Fama, 1999). In this respect, unsupported roof stability has been recognised as one of the most important parameters in Australian highwall mining (Duncan Fama, Shen and Maconochie, 2001).

In their response to reviewers comments, Mine Advice stated that "*Given both the low cover depth and sub-critical geometry between intra-panel barrier pillars, it is concluded that the likelihood of coal rib spall resulting in a significant increase in the effective drive width and so adversely affecting roof stability in unsupported drives under normal or background geotechnical conditions, is in the highly-unlikely to practically-impossible risk category*". It is the reviewers' experience that roof falls can occur in the roof, particularly under unsupported coal roof. It is appreciated that the entries are only 4m wide and the likelihood of roof fall occurrence is significantly reduced, it is recommended that potential roof spalling and its consequences should be incorporated into the design.

Off-line cutting

In highwall mining, the actual widths and heights of the web pillars can differ from designed values due to off-line cutting engendered by poor guidance controls (Adhikary, Shen and Duncan Fama, 2002; Zipf and Bhatt, 2004). To prevent off-line cutting, guidance systems were adopted by many companies, which are not the expertise of the author. It is however evident from responses to questions that Hume Coal places reliance on the success of technologies that are under development for guidance control. Since off-line cutting can have a significant impact on the pillar and roof stabilities, an in-depth study into the degree of potential off-line cutting and its impact on the proposed layout is recommended.

Equipment recovery in drives

An important consideration in this mining method is the recovery of the continuous miner (CM) in cases of breakdown, flooding and/or major fall of ground. These recovery operations require special considerations as they can be regarded as high risk activities, due to the lack of a secondary egress, should the workforce need to enter the drives. Zipf and Bhatt (2004) stated that due to the various reasons about 10 to 15 highwall mining systems became seriously trapped during 2003 and required a substantial retrieval effort such as underground recovery, surface excavation or a major surface retrieval. They also stated that underground recovery is arguably the most hazardous and essentially requires the setup of a small underground coal mining operation. Over the years, a number of failures in highwall mining have also been recorded in Australia.

From the responses to the reviewers' questions it is understood that equipment recovery has already been considered by the mine. It is assumed that a management plan with an underpinning risk assessment for the recovery operations will be developed during and following the equipment design and purchase.

Pillar Load

For a simple layout with a reasonably uniform pattern of pillars and a panel width at least as large as the depth, the average pillar load can be estimated by using the tributary area theory. The theory assumes that each pillar carries a proportionate share of the full overburden load.

The factors influencing pillar load are:

- depth of cover – the deeper the mining, the higher the pillar stress,
- depth to panel width ratio – the narrower the panel in relation to the cover depth, the lower the pillar stress,
- pillar width – the smaller the pillar, the higher the pillar stress,
- roadway width – the wider the roadway, the higher the pillar stress, and
- extraction ratio – the higher the extraction ratio, the higher the pillar stress.

While the tributary area theory provides a simple method of determining the average state of axial stress in a pillar, it has implicit limitations which must be borne in mind. Average pillar stress is calculated by assuming that pillars uniformly support the entire load overlying both the pillars and the mined-out areas. Tributary area theory assumes regular geometry and ignores the presence of abutments. It also ignores the stiffness of the overburden strata and the panel width to depth ratio. The effects of deformation and failure in the roof strata resulting from the mining operation are also disregarded. For practical design purposes however, the suggested equations for average stress calculations are acceptable if the designer appreciates the limitations. If the tributary area theory is not applicable to a particular layout (as in this case), the best way to estimate the pillar loads is to conduct numerical modelling (as presented in this review).

The Mine Advice states that a key design element of the proposed layout is the use of sub-critical panels (i.e., the width of the panels are narrower than the cover depth) to prevent low w/h ratio web pillars ever being loaded under full tributary area loading. This assumption is based on the fact that Hawkesbury Sandstone is capable of bridging long distances to transfer the load from narrow, less stiff pillars onto larger, stiffer intra-panel barrier pillars. This consideration is reasonable under certain environments and it is also possible that in the proposed layout these narrow web pillars will not be subjected to full tributary area load. In response to reviewers' comments Mine Advice also states that the load distribution between the web pillars and intra-panel barrier pillars is almost certainly indeterminate in that it is directly influenced by a number of unquantifiable geotechnical parameters. Therefore, in calculation of pillar loads in the proposed layout, numerical modelling is the most reliable approach. It is not known why a numerical modelling study was not conducted by Mine Advice to assess the bridging capabilities of Hawkesbury Sandstone, the degree of load redistribution and the magnitude of pillar loads. In my opinion this is a major limitation of the pillar design in Hume Coal project. Currently available numerical models are highly capable of estimating pillar loads in the proposed layout, which will be further assessed in subsequent sections of this report.

Pillar stability assessment

The proposed layout relies on different types of pillars. It is assessed that other than the web pillars and the gateroad pillars, the pillar sizes and FoSs' are acceptable. Under development loading, the sizes of the gateroad pillars are also adequate; however, in the case of failure of the web pillars, the gateroad pillars will not have an acceptable FoS for long-term stability. The proposed sizes of the web pillars vary from 3.5m to 6m for depths of 80m to 170m respectively. With the proposed mining height of 3.5m, the pillar width to mining height (w/h) ratio of the web pillars vary from 1.0 to 1.7, which is considered to be *uncommonly* slender by Australian coal mining industry standards. Such slender pillars are known to fail suddenly, causing catastrophic failures. On the contrary, a non-violent pillar squeeze can occur for larger pillars with w/h ratios ranging from 4 to 8 or the strain-hardening behaviour for squat pillars with w/h ratios greater than 10 (Mark, 2006). However, considering the compartmentalisation of the web-panels, it is likely that if the web pillars fail, the intra-web pillars will arrest the failure and confine it within the individual web-panels.

As mentioned before, the proposed mine layout includes a strategy to backfill the drives (only to emplace the reject tailings and for the purpose of generating confinement to the web pillars). This approach appears to be reasonable; however, backfilling will be only effective in ensuring the long-term stability of web pillars by preventing the spalling of pillars by using stiff, cemented backfill material. This will be discussed further in the numerical modelling section of this report.

Mine Advice states that the design of the initial panel layouts has been conducted using Analysis of Retreat Pillar Stability – Highwall Mining (ARMPS-HWM) methodology developed by NIOSH (2012) for the design of the initial panel layouts. To complete the design process to standard that can be considered as part of a mining application in NSW, Mine Advice utilised the UNSW Pillar Design Procedure (Salamon et al., 1996). The NIOSH study is conducted to determine the stability of pillars, roof and rib whilst the mine is active; the reviewer is not aware of any references to long-term assessment of the pillars in the NIOSH program.

The proposed web pillar w/h ratios are within the range of failed cases in the UNSW pillar database, with approximately 17% of failed and intact cases in the database having pillar w/h ratios smaller than 1.7. However, Galvin, as one of the developers of the UNSW pillar design methodology, has stated in numerous public forms, in a number of technical papers and recently in his book (Galvin, 2016), the UNSW pillar strength formulae are not applicable to highwall mining pillars, i.e. similar to those proposed web pillars due to their inherent low w/h ratios.

There are four main reasons for why the UNSW pillar design methodology is not applicable to highwall mining. Firstly, even minor geological structures can play a significant role in the stability of narrow pillars. Although underemphasised by the Mine Advice report, it has been stated by many authors in the past that geological structures can be particularly important in determining the stability of pillars. Considering the size of the web pillars, even cleating and jointing can play a significant role in determining the stability of the proposed web pillars. Secondly, as mentioned above, roof falls are common in highwall mining; even minor roof falls can significantly reduce the strength of those narrow pillars. Thirdly, due to the relatively small size of web pillars, the confinement generated between roof and floor can be relatively low, which may not be represented by the UNSW pillar failed and intact database as 83% of the cases had larger w/h ratios. Lastly, and importantly, Salamon and Munro (1967) stated that the values substituted for the strength and load must be regarded as approximations only, which are subject to error. Therefore, the calculated FoS (using the original Salamon and Munro formula or the UNSW formula as their underlying principles are identical) may not represent the true FoS. Hence, the calculated FoS could deviate to a level either higher or lower than predicted by FoS. An example of this is illustrated in Figure 2. In this figure, the distribution of a pillar FoS of 1.0 using the outcomes of the UNSW study by Salamon et al., (1996) is plotted. As evident, the pillar FoS of 1.0 is only an average value, with the true distribution having any value between 0.65 to 1.9. This uncertainty is caused by:

- natural causes - that is variations in coal strength, pillar loading, structures and the competency of the roof and floor,
- the approximate nature of the strength formula, and
- human error in the data used for parameter estimation.

Irrespective of their size and the FoS, all coal pillars designed using the UNSW formula are subject to the same uncertainty. When pillars are narrow and the FoS hovers around 1.0, this variation becomes critical as the pillars can become rapidly unstable following minor changes in natural environment and in pillar sizes (i.e., height or width); therefore, the behaviour of small pillars, such as in this case, are highly unpredictable.

The probability of pillar stability (PoS) and failure associated with the UNSW pillar strength formulae is summarised in Table 3 and illustrated in Figure 3. As can be seen in this table and also in the figure, when the FoS is 1.0, the probability of the failure (PoF) and the stability (and the failure) of the pillars is only 50%.

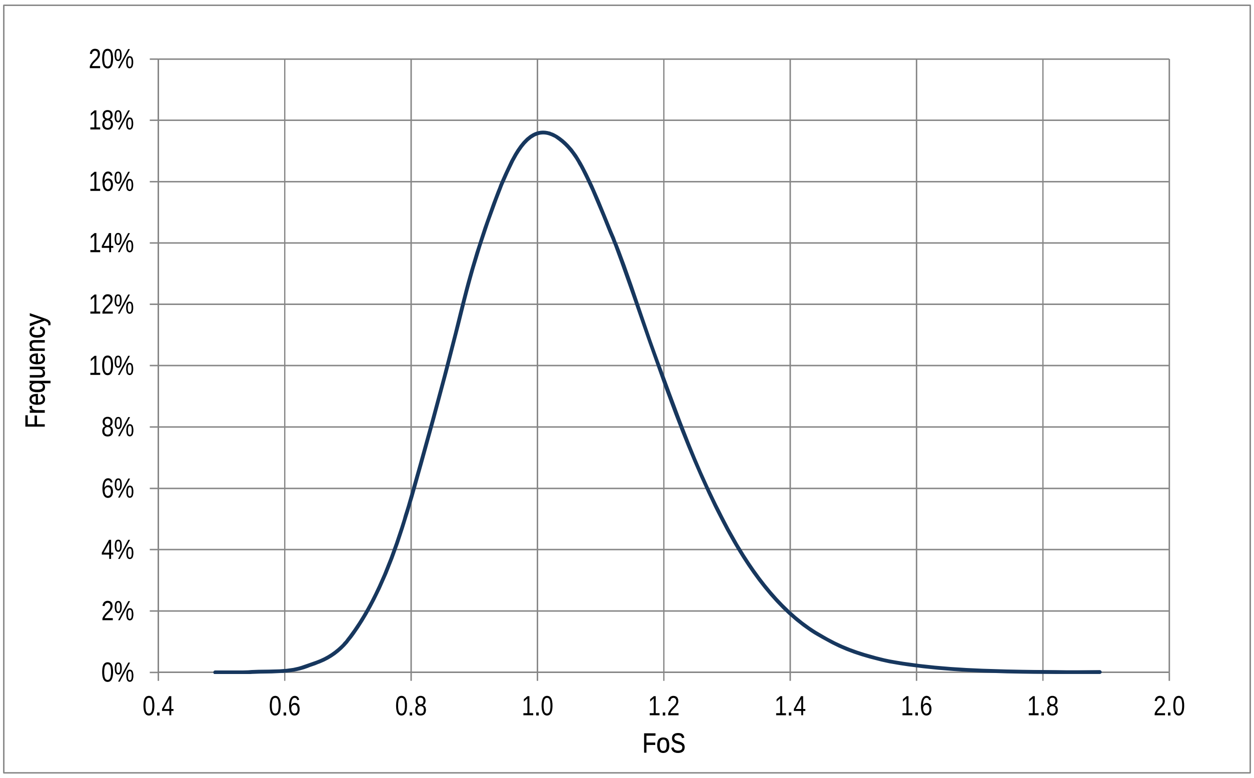

Figure 2. Distribution of pillar FoS of 1.0 using the UNSW pillar strength formula

Table 3. Failure probability associated with Salamon et al., 1996 power law formulae

Safety Factor	Probability of Stability	Probability of Failure in 1 Million
2.1	0.999999	1
2.0	0.999995	5
1.9	0.999978	22
1.8	0.999909	91
1.7	0.999637	363
1.6	0.998622	1378
1.5	0.995097	4903
1.4	0.983949	16051
1.3	0.952650	47350
1.2	0.877237	122763
1.1	0.728098	271902
1.0	0.500000	500000
0.9	0.251083	748917
0.8	0.077615	922385

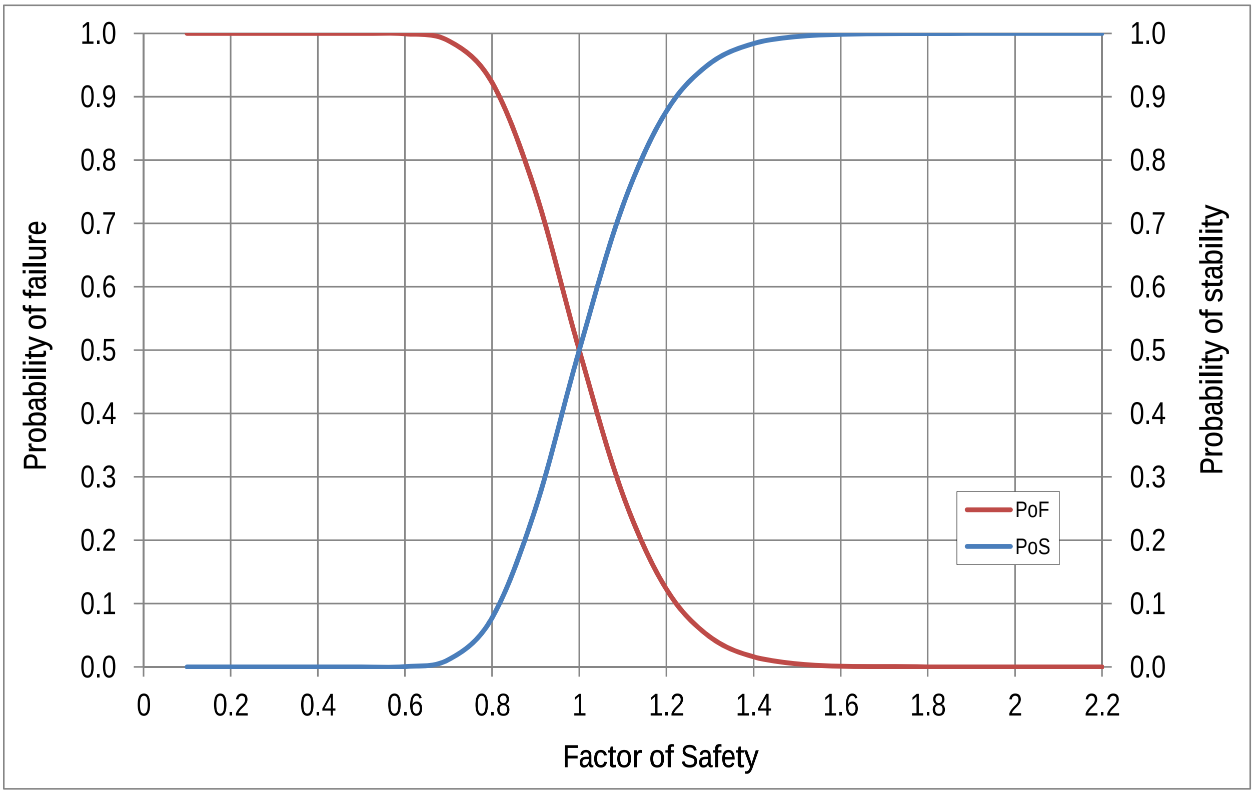

Figure 3. Probabilities of failure and stability associated with the UNSW pillar strength formula

Note that only the power formula is considered in the above calculations. However, the linear formula established by Salamon et al., (1996) is also acceptable, provided that the probability of stability and the safety factor are adjusted accordingly. As the degrees of freedom in the power and the linear formulae are different, the standard deviations calculated in the maximum likelihood analysis are also different. Assuming a lognormal distribution of safety factors (as indicated by Salamon et al., 1996), the calculated probability of stabilities in both formulae will be different; indicating that a higher safety factor will be required in the case of the linear formula to achieve the same probability of stabilities as calculated using the power formula.

In addition, it is of note that a recent study conducted by UNSW has established that the strength of pillars in highwall mining is approximately 30% weaker than predicted by the UNSW (or Bieniawski) formula, and approximately 57% weaker than predicted by the adjusted version of Mark-Bieniawski formula (Mark and Chase, 1997). However, it is not expected that Mine Advice can have access to this current UNSW study as it was conducted for CSIRO and a journal paper is currently being published.

Long-term pillar stability

A long-term stable coal pillar is generally defined as a coal pillar system that will not result in failure of pillars over a long period of time (e.g., >100 years), despite rib and roof spalling, which would otherwise cause unacceptable surface subsidence (>20mm in this case). The methodologies presented by Salamon et al., (1998) and Canbulat (2010) can be used to determine the long-term stability of the pillars. These methodologies determine the maximum amount of possible rib spalling (i.e., when the aprons of the spalled material reach the roof) and assess the FoS of the core of a pillar. If the core of a pillar is large enough with a reasonable FoS, pillars can be defined as long-term stable (or *infinitely* stable) using a stochastic analysis. The proposed web pillar system in Hume Coal project is such that the web pillars cannot accommodate any rib or roof spalling. This is illustrated in Figure 4, whereby a pillar FoS reduction occurs as the rib spalling reduces the pillar width. Similarly, Figure 5 shows the pillar FoS as roof spalling and pillar height increases. From these two figures, it can be concluded that even minor rib and/or roof spalling has the potential to reduce the FoS of pillars below 1.0. Considering that the methodology proposed by Salamon et al., (1998) suggests that the ribs of the proposed web pillars can theoretically spall up to 3.2m, the remaining core of the web pillars will have a significantly low FoS. It is therefore reasonable to conclude that the proposed web pillars cannot be considered to be long-term stable, unless they are backfilled. Note that this finding is valid, irrespective of whether the pillars are subject to tributary load or reduced loads due to the stiffness of the overburden.

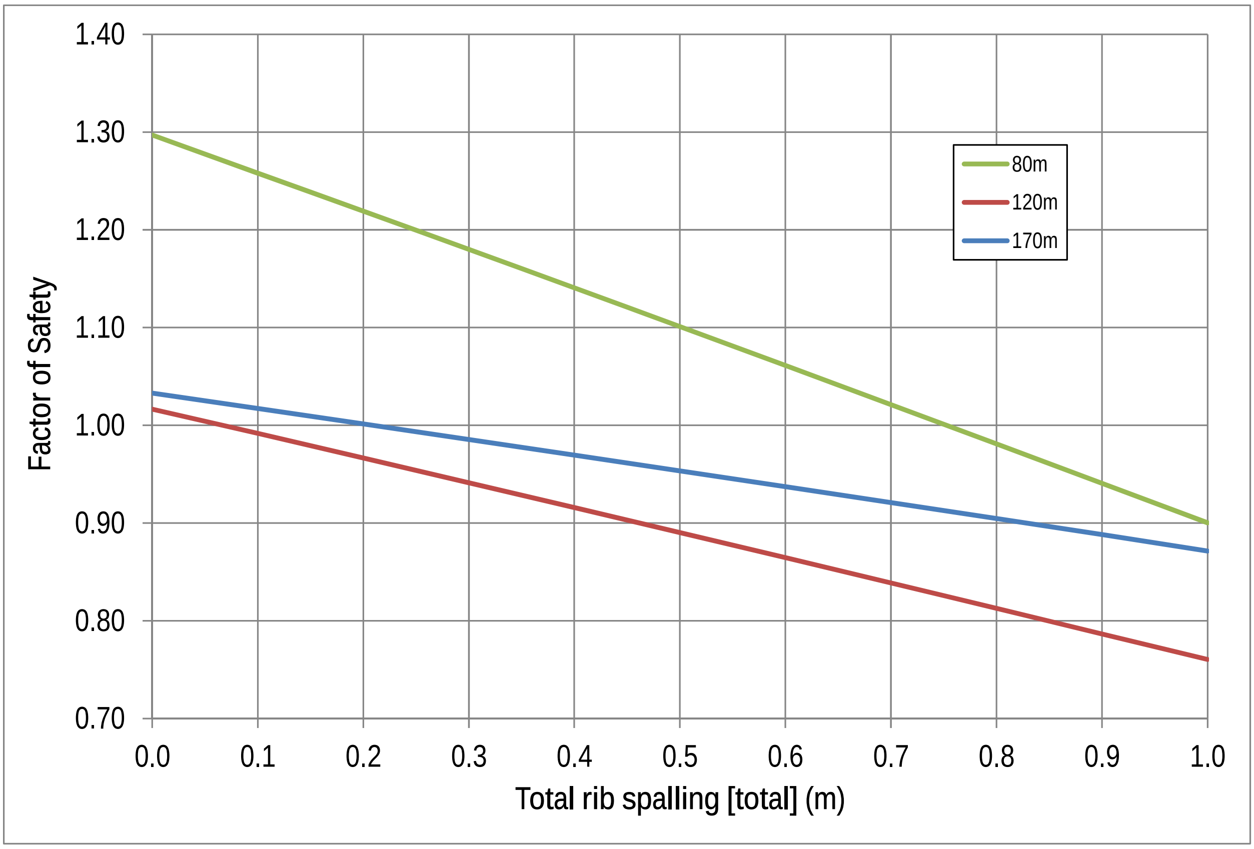

Figure 4. Pillar FoS reduction due to pillar spalling

Elastic settlement calculations

The load, elastic settlement and strength calculations in the Mine Advice report assume that the pillars are in a perfectly elastic state throughout their lives, which may not in fact be the case. The proposed narrow pillars can only accommodate small stress increases in the ribs due to a lack of confinement that is normally evident in larger pillars. Once the ribs start yielding (i.e., possible rib spalling), the yielding propagates into the pillar, which results in pillars being rapidly moved from an elastic to inelastic state. Once a pillar goes into an inelastic state, the elastic subsidence and load estimates, as conducted by Mine Advice, is no longer accurate.

The elastic pillar settlement calculations are more appropriate for larger pillars that mostly remain in an elastic state under load, particularly the core of the pillar. A further assessment of this condition is presented in the numerical modelling section of this report.

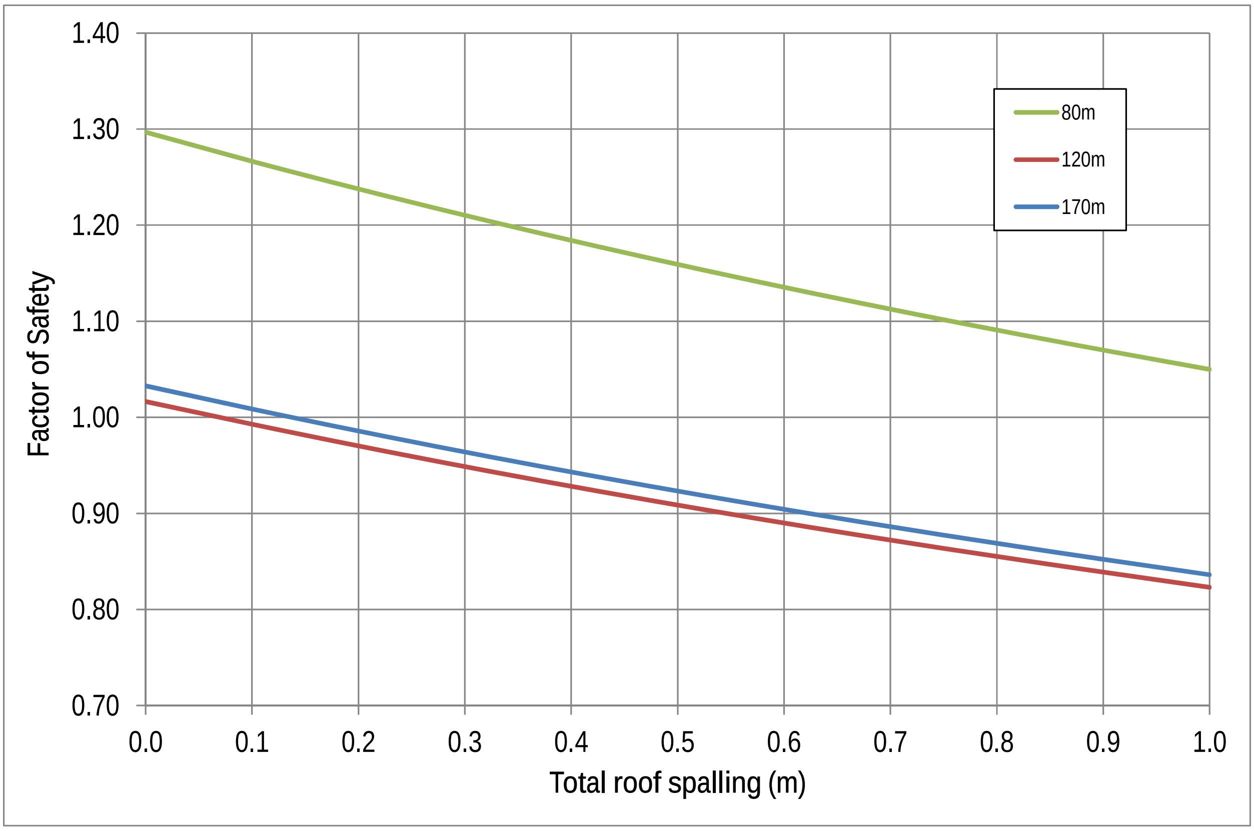

Figure 5. Pillar FoS reduction due to roof spalling

5 ASSESSMENT OF WEB PILLAR STABILITIES USING NUMERICAL MODELLING

Numerical modelling was utilised to investigate the stabilities of pillars in the proposed panel geometry as proposed by Mine Advice. Back-analysis of input parameters for coal was carried out prior to the panel simulations. This back analysis was conducted using the Salamon et al., (1996) coal pillar strength formula. Using the obtained coal parameters and the actual excavation sequence in the panels, the change in stress of coal pillars in the panels were monitored and the factors of safety of the proposed pillars were calculated. Due to the nature of the proposed long pillars, plane strain analysis was conducted using the two-dimensional numerical software FLAC (Itasca Consulting Group, Inc. v 8, 2017).

It is of note that since the input parameters in numerical models are calibrated using the UNSW pillar strength formulae, the uncertainty associated with those formulae should also be considered in analysing the results presented in this section.

5.1 Back-analysis of input parameters for coal

5.1.1 Model calibration against the UNSW formula

Numerical models calibrated against empirical pillar formulae can be a reliable way to explain the average behaviour of pillar models (Esterhuizen, 2014). Therefore, calibration of coal parameters against empirical pillar strength formulae was attempted. The UNSW formula (Salamon et al., 1996), which is widely used for underground coal pillar design, was chosen for calibration. The estimated strength of the coal pillars from the formula is given in Table 4.

The constitutive law for coal pillars was the strain-softening model based on the Mohr-Coulomb failure criterion, while the roof and floor were considered to be an elastic material. Thus, the calibration process aimed to back calculate a set of input parameters including the cohesion, angle of internal friction and corresponding inelastic strain range. The elastic moduli and Poisson's ratios were chosen based on the information provided in the Mine Advice report, as shown in Table 5.

Table 4. Strength of designed coal pillars

	Case 1	Case 2	Case 3
Pillar width (m)	3.5	4.1	6
Pillar height (m)	3.5	3.5	3.5
w/h ratio	1.0	1.17	1.71
UNSW formula: $S_p = 8.6w^{0.51} / h^{0.84}$	5.7MPa	6.2MPa	7.5MPa

where S_p is pillar strength (MPa), w and h are width and height of pillar (m).

Table 5. Material properties used for calibration of model

Material	Elastic modulus (GPa)	Poisson's ratio
Roof	9.9	0.25
Coal	2.0	0.25
Floor	11	0.22

Figure 6 shows the pillar model incorporating half of the coal, roof and floor along the symmetrical centreline of the pillar system, which is a repeating geometry in the panel (i.e., the pillar is under tributary area load). The height of the roof and floor was 20m and the mining height was fixed at 3.5m, while the pillar widths varied to simulate the three cases in Table 4 as well as the pillar with a w/h ratio of 2. The uniform element size of 0.25m was applied to the coal and a smooth variation of zoning from the coal to the boundaries was used for the roof and floor with appropriate aspect ratios to avoid numerical instability. Roller boundaries were applied along the side of the roof and floor, the bottom of the floor and the vertical line. The entry width was fixed at 2m, which is a half of the entry width of 4m. The increasing load was generated by applying a constant velocity of 10^{-6}m/s on the top of the roof in the model.

The coal pillar stress was estimated by taking the average of vertical stresses generated in the elements located at the mid-point of the pillar. The vertical strain was calculated based on the ratio of the difference between the average vertical displacements at the top and bottom of the coal pillar and the mining height. The model was then calibrated against the UNSW formula by monitoring the peak stresses with various sets of input parameters. The calibration results are shown in Table 6 and the stress-strain curve using the calibrated parameters are given in Figure 7.

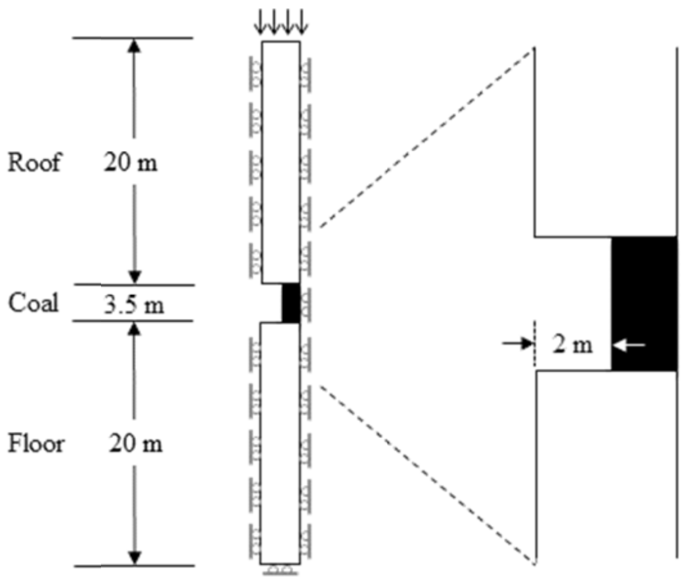

Figure 6. Geometry of coal pillar model

Table 6. Coal properties calibrated against the UNSW formula

Material	Cohesion (MPa)		Friction angle (°)		Plastic strain
	Peak	Residual	Peak	Residual	
Coal	1.8	0.2	21	21	0.025

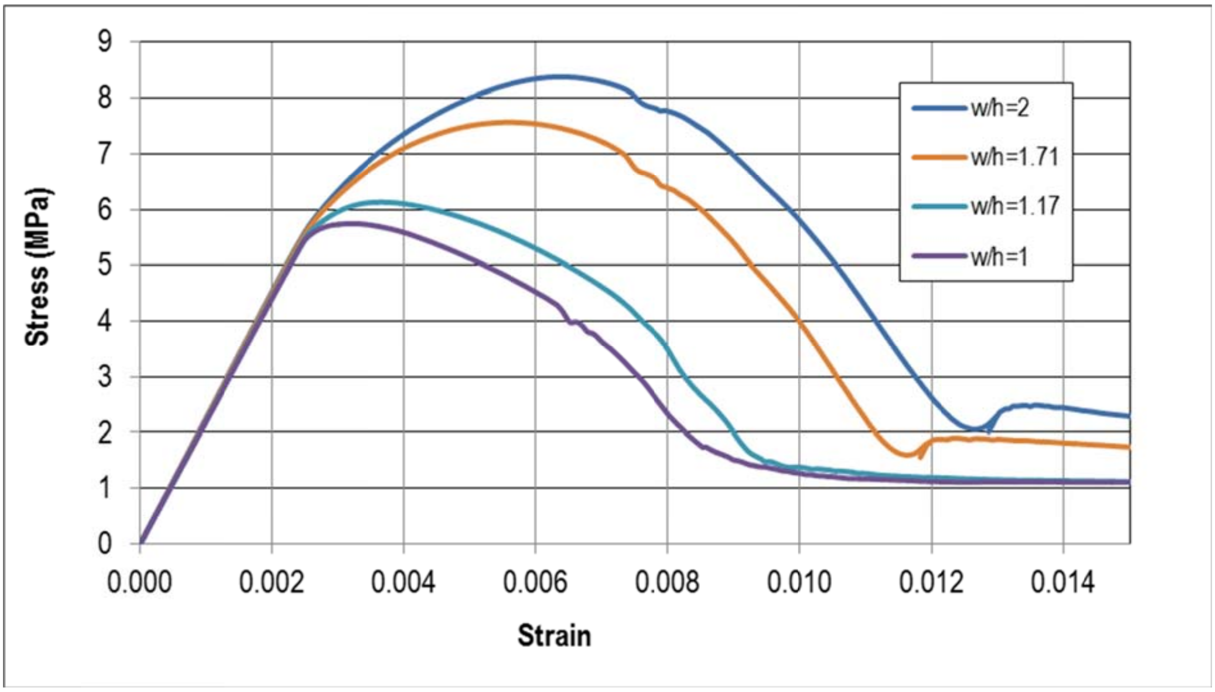

Figure 7. Stress-strain curve

5.1.2 Coal roof and floor

The UNSW formula doesn't apply to cases where a weak roof and floor contribute to pillar system failure (Galvin, 2016). Some studies have been conducted to investigate the effect of weak immediate strata on pillar strength. Gale (1999) suggested the use of the reduced in-situ coal strength of 4MPa for weak surrounding strata using the Bieniawski-based formula, while the in-situ coal strength of 6MPa is generally used for strong surrounding strata.

Whilst the study by Gale (1999) may indicate a possible reduction in pillar strength with the coal roof and floor, no credible study regarding the effect of coal roof and floor on pillar strength has been conducted previously. Therefore, a new model using the same geometry as Figure 6 was built to simulate the 3m coal roof and 0.5m floor as illustrated in Figure 8. Other than the coal roof and floor, the same numerical modelling procedure was conducted as in the preceding section.

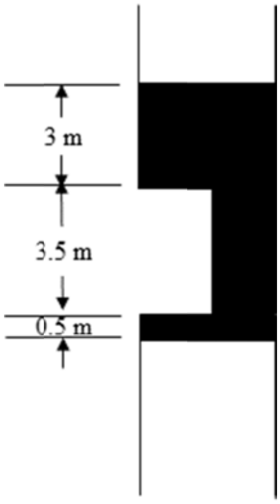

Figure 8. Geometry of coal pillar model with 3 m coal roof and 0.5 m floor

The changes in pillar strength with the coal roof and floor are listed in Table 7. The pillars with w/h ratios of 1 and 1.17 have the same strength values, while those with w/h ratios of 1.71 and 2 show a slight decrease in pillar strength. Based on the results, and when compared to normal roof and floor conditions, the possibility of having a weaker pillar strength for Case 3 (i.e., deeper parts of Hume Coal) should be considered.

Table 7. Comparison of pillar strength

Model result	Case 1 (w/h ratio=1)	Case 2 (w/h ratio=1.17)	Case 3 (w/h ratio=1.71)	w/h ratio=2
Strong roof and floor	5.7 MPa	6.1 MPa	7.6 MPa	8.4 MPa
Coal roof and floor	5.7 MPa	6.1 MPa	7.2 MPa	7.8 MPa

5.2 Stability of the proposed web pillars

Using the above input parameters, the panel simulations of the three cases were conducted as given in Table 8. Figure 9 shows the panel models incorporating the proposed number of web pillars, drives and barrier pillars. In these simulations, the pillars are no longer under tributary area load; the load estimates represent the panel and pillar geometry as well as the input parameters for rock mass, as proposed by Mine Advice. The heights of the overburden and floor were 20m and the mining height was 3.5m. The excavation width was 4m and the barrier width in the model was a half of the actual width, which indicates a repeating geometry of the proposed panels. Roller boundaries were applied along the side and bottom of the model. A stress boundary was applied along the top of the model so that the coal pillars were subjected to the load under the depths of cover. Excavations were made from left to right in sequence and the change in stress of the web pillars were monitored at each excavation step.

Table 8. Panel configurations

	Depth of cover (m)	Web pillar width (m)	w/h ratio	Barrier width (m)	Number of drives	Number of webs
Case 1	80	3.5	1.00	14.0	8	7
Case 2	120	4.1	1.17	16.8	7	6
Case 3	170	6	1.71	22.8	6	5

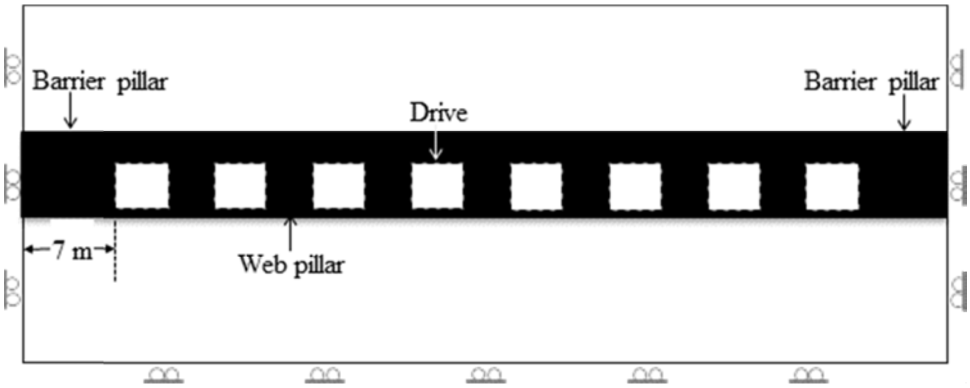

Figure 9. Panel model of Case 1

The panel simulations indicated a relatively stable state for Case 1, whilst showing relatively unstable states for both Cases 2 and 3. In Case 1, the yielded elements were rarely observed within the pillars, as shown in Figure 5. On the contrary, the pillars in the middle of the panels in Case 2 and 3 appeared to yield significantly as shown in Figure 11 and Figure 12..

After the panel simulations, the stresses of each web pillar were recorded and summarised in Table 9. In addition, the FoS and PoF were manually calculated by taking the modelled strength values in Table 7. The web pillars are numbered from the left in the panels. The low FoSs' for Case 2 and 3 correspond to the inelastic states from Figure 11 and Figure 12. Even if the panel of Case 1 appear to be stable, the FoS and the PoFs' of the pillars in the middle of the panel are about 1.4 and 1.6%, which does not indicate long-term stability, considering the potential time-dependent deterioration and strength of pillars (i.e., the long-term stability).

Table 9. Web pillar stress after panel simulation

	Web 1	Web 2	Web 3	Web 4	Web 5	Web 6	Web 7
Case 1 FoS PoF	3.7 MPa 1.6 0.14%	3.9 MPa 1.5 0.5%	4.0 MPa 1.4 1.6%	4.0 MPa 1.4 1.6%	4.0 MPa 1.4 1.6%	3.9 MPa 1.5 0.5%	3.6 MPa 1.6 0.14%
Case 2 FoS PoF	5.3 MPa 1.2 12.3%	5.5 MPa 1.1 27.2%	5.6 MPa 1.1 27.2%	5.6 MPa 1.1 27.2%	5.5 MPa 1.1 27.2%	5.3 MPa 1.2 12.3%	N/A
Case 3 FoS PoF	6.4 MPa 1.1 27.2%	6.5 MPa 1.1 27.2%	6.6 MPa 1.1 27.2%	6.5 MPa 1.1 27.2%	6.4 MPa 1.1 27.2%	N/A	N/A

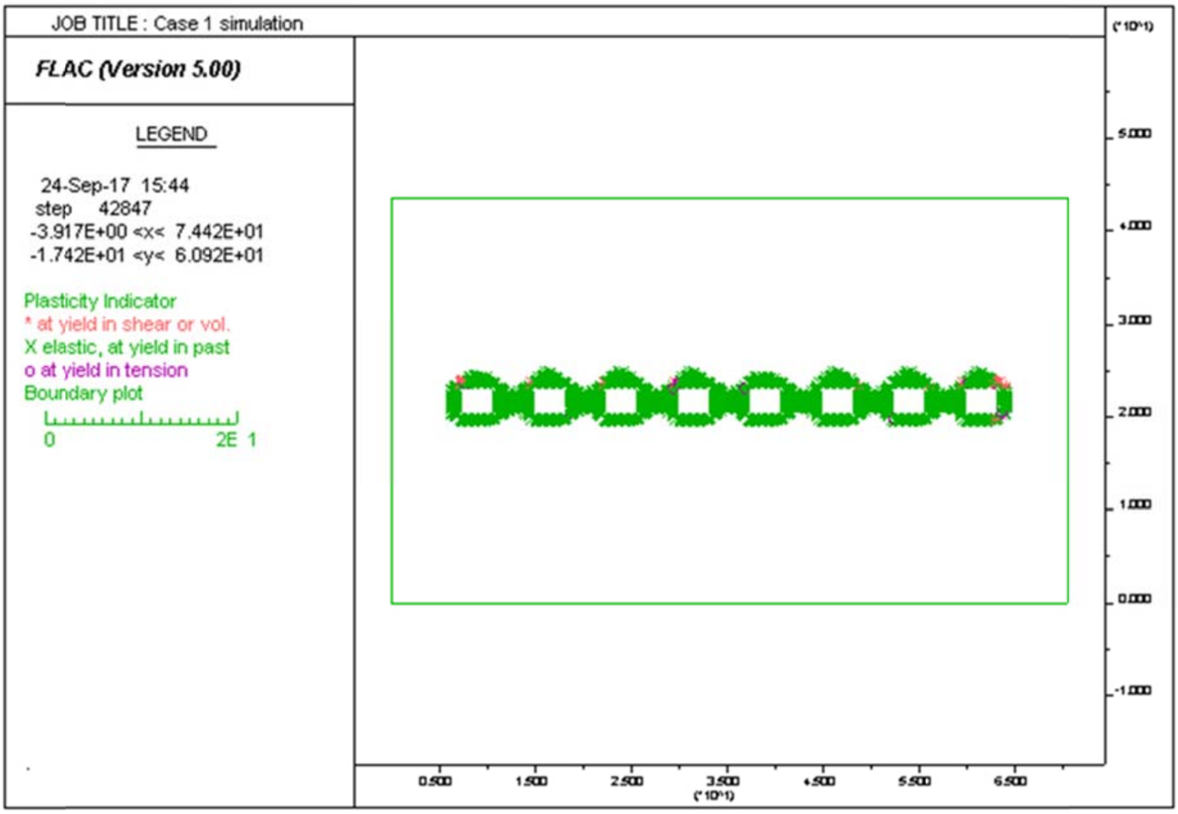

Figure 10. Plastic indicator in Case 1

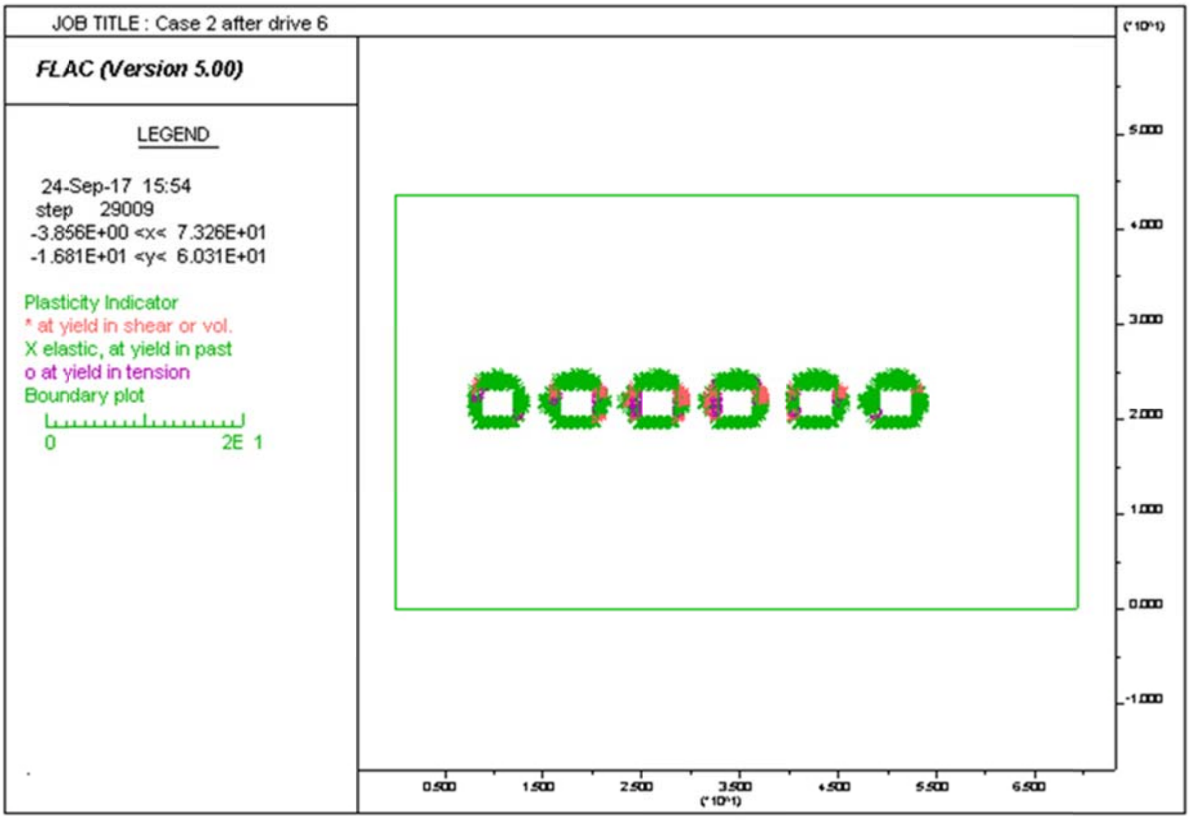

Figure 11. Plastic indicator in Case 2

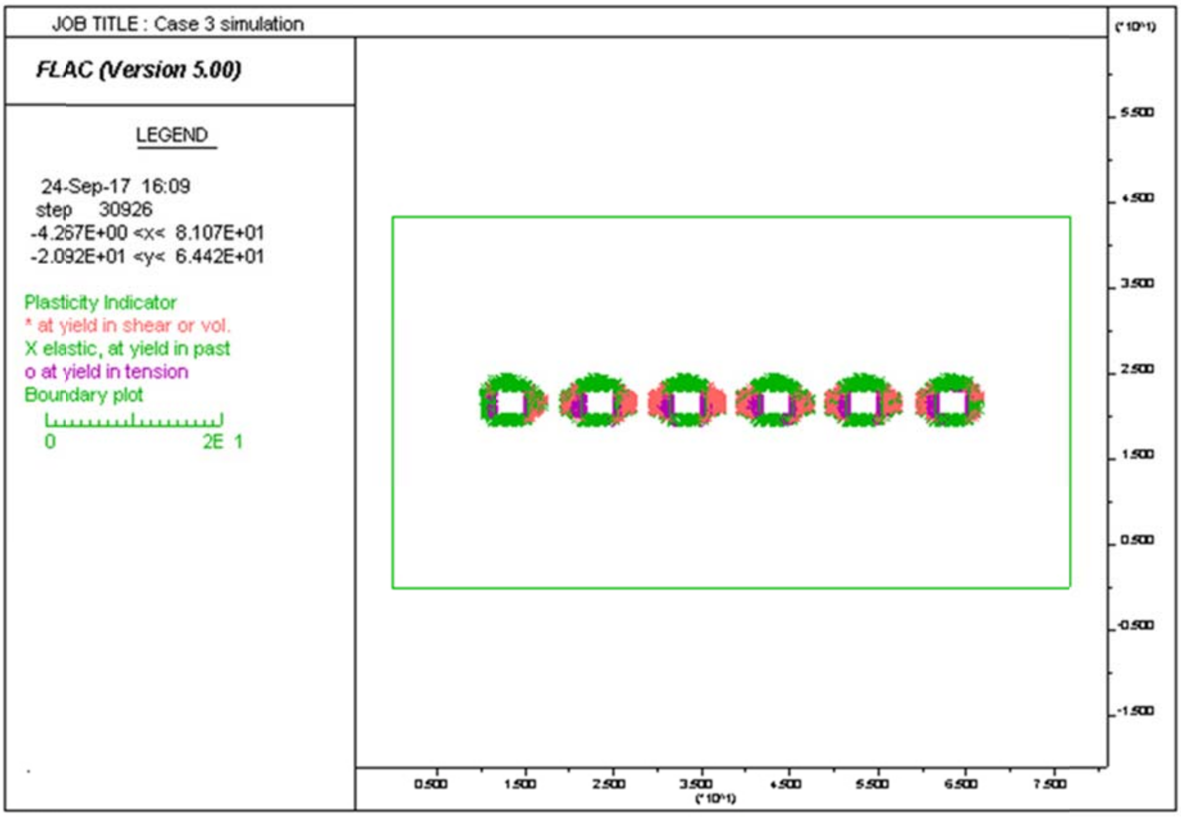

Figure 12. Plastic indicator in Case 3

Another assessment was also conducted to determine the depth of material yielding in the ribs. The results are summarised in Table 10. In Case 1, the responses of the pillars were elastic. However, in Cases 2 and 3, the depths of yielding into the pillars (i.e., depth of yielded elements) in the middle of the panels were 1.3m and 2.0m respectively.

Table 10. Depth of yielding (per rib)

	Depth of cover (m)	Pillar width (m)	Depth of yielding without backfilling (m)
Case 1	80	3.5	0.0
Case 2	120	4.1	1.3
Case 3	170	6.0	2.0

The above numerical results indicate that:

- The proposed pillars are not in complete failure. However, the pillar FoSs' using the inputs proposed by Mine Advice are not significantly greater than the tributary area estimates.
- As a result, these pillars cannot be considered to be long-term stable.

The below sections assess the effectiveness of backfilling.

5.3 Case 3 with backfilling

In this section, the geometry of Case 3 is utilised, as given in Figure 10. The vertical boundary on the left hand side was placed 30 m away from the first drive so that the behaviours of the pillars were not to be affected by the boundary. However, the barrier on the right hand side was a half of the width of the actual barrier pillars. Overall, this geometry was considered for the possible influence of stress path history on the state of stress and/or deformation. As in the previous section, the roof and floor heights were 20m, including the 3.5m coal roof, 0.5m coal floor and the mining height of 3.5m. The excavation width was 4m. Roller boundaries were applied along the side and bottom of the model. A stress boundary was applied along the top of the model so that the coal pillars were subjected to the load under a depth of cover of 170m.

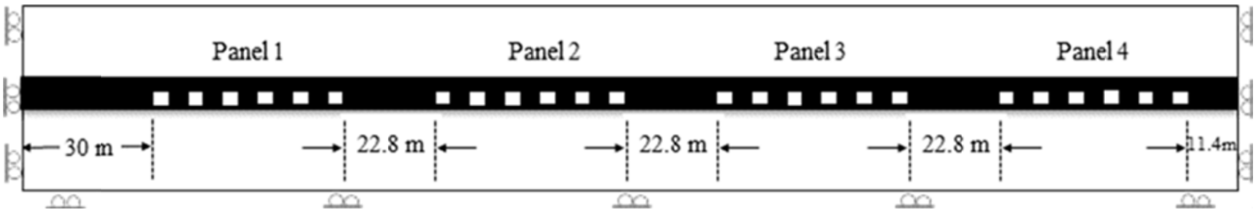

Figure 13. Multiple panel model for Case 3

The excavation and backfilling sequence used are illustrated in Figure 14. The first two panels are mined and then Panel 1 is backfilled. Following this, the third panel is mined and then Panel 2 is backfilled, and so on. The change in average vertical stress and vertical strain of the web pillar in the middle of Panel 3 was monitored. A height of 2.5 m of the drives was backfilled, which is approximately 70% of the mining height.

The results revealed an influence of stress path on deformation as shown in Figure 12, which shows the stress-strain curve of the web pillar in the middle of the panel. While the pillar stress reached a similar level, approximately 6.5MPa. From this result it is concluded that emplacing a <u>cemented</u> backfill material into the drives will not change the stress profile significantly. However, it will prevent spalling of the web pillars over a period of time. This in turn will provide long-term stability of the panels.

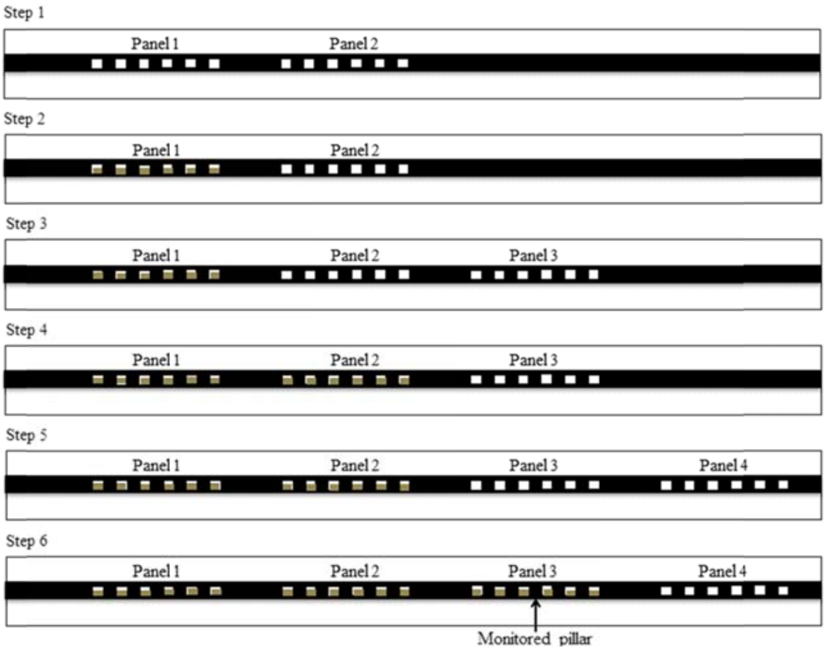

Figure 14. Excavation and backfilling sequence

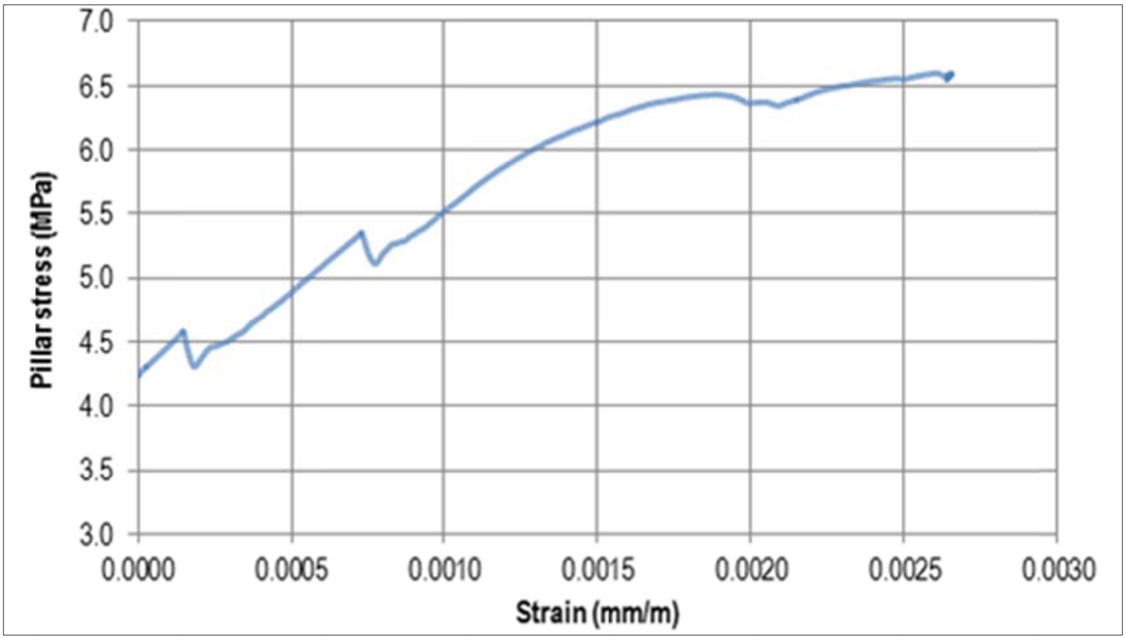

Figure 15. Stress-strain curve of backfilling

6 ELASTIC AND INELASTIC SETTLEMENT OF WEB PILLARS AND SURFACE SUBSIDENCE

As indicated above, a critical assumption made by Mine Advice is that the pillars will stay in an elastic state for their life. However, it is evident in the previous numerical modelling section that the pillars at a shallow depth can be stable and mostly remain in an elastic state. However, once the pillars go into the inelastic state, their response to loading can change, which may result in noticeable subsidence on the surface.

Another assumption made by Mine Advice in the pillar settlement calculations is that the web pillars are subjected to tributary area load. As evidenced in the previous section, it is highly likely that the ribs of the proposed web pillars will yield. Once the pillars start yielding, the elastic settlement calculations cannot provide accurate subsidence estimations.. In order to demonstrate this behaviour, a study has been conducted using RS2. As RS2 cannot represent the strain-softening behaviour of materials as in FLAC, a calibration of pillar strength, similar to the FLAC modelling study presented in the previous section, has been conducted to determine the representative cohesion and frication angle values.

The layout used in this study is presented in Figure 16, which includes 20 of the 6m wide web pillars and 21 of the 4m wide drives at a depth of 170m. This layout excludes the intra-web pillars to ensure that the web pillars are subjected to tributary area loading (as assumed by Mine Advice). As seen in Figure 16, Ashfield Shale is also modelled using the assumption that 60% of the overburden consists of Hawkesbury Sandstone (i.e., 102m in this case) and 40% Ashfield Shale (i.e., 68m in this case). The overburden and floor have been assumed to be elastic. For a realistic loading and stress distribution in the overburden, joints have also been included in these models as representatives of bedding planes. Joint densities of 10m and 20m in Ashfield Shale and Hawkesbury Sandstone have been assumed respectively. Since the Mine Advice report suggests that Hawkesbury Sandstone contains conglomerate, fine to coarse grained sandstone and siltstone units in beds ranging from thinly bedded to thickly bedded (0.06m to >2m), a modelled 20m bedding thickness is considered to be reasonable. It is noted that considering the distance of the base of the Ashfield Shale, it is assessed that a 10m bedding thickness of the shale will have an insignificant impact on the results. Coal has been assumed to be elastic and inelastic in subsequent models. As suggested by Mine Advice, a 3m thick coal in the roof and a 0.5m thick coal in the floor have also been included in these models.

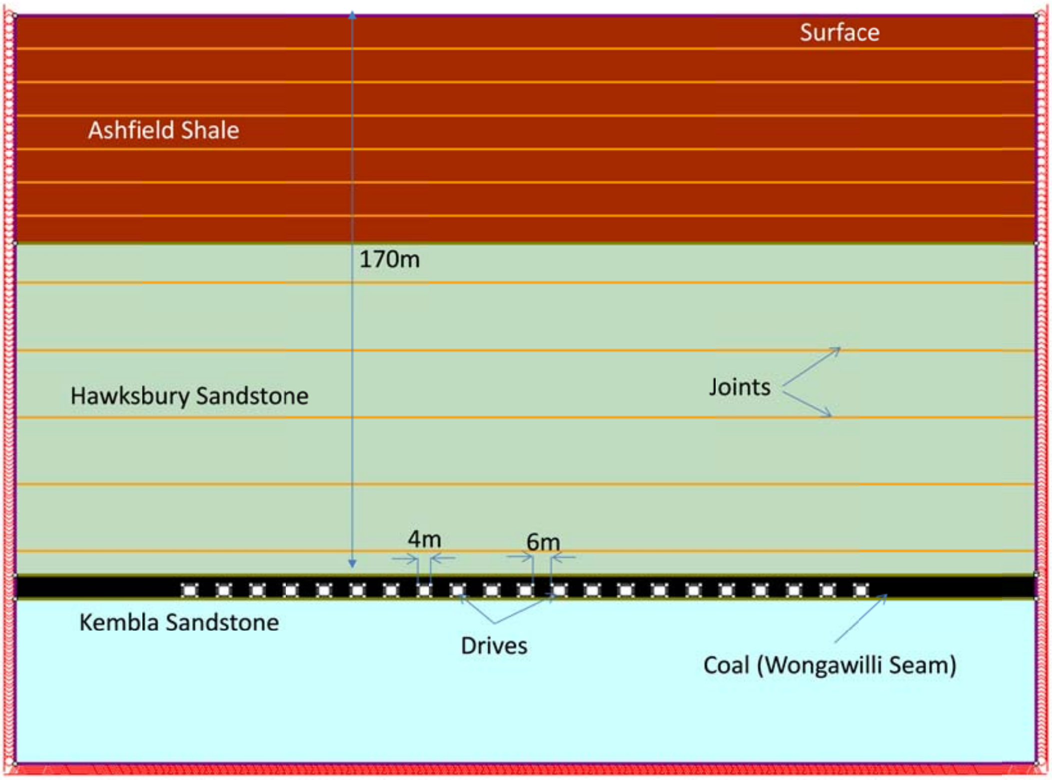

Figure 16. Phase2 layout to assess the elastic settlent of we pillars

Similar to the previous section, all critical input parameters used in this modelling study have also been obtained from the Mine Advice report. Information not included in the Mine Advice report has been obtained from relevant literature and/or previous numerical modelling studies conducted in NSW. Table 11 shows the input parameters used in these models. As evidenced by this table, the strength parameters for coal are somewhat greater than the FLAC models presented in the previous section. This is because the failure of material is simulated as an elastic–brittle–plastic material, i.e., as soon as material reaches peak strength, the strength drops to residual value without a post-peak modulus.

Table 11. Input parameters used in the models

	Unit weight (MN/m^3)	Young's Modulus (GPa)	Poisson's Ratio	Tensile strength (MPa)	Peak cohesion (MPa)	Peak friction angle (⁰)	Residual cohesion (MPa)	Residual friction angle (⁰)	Dilation Angle (⁰)
Coal (Wongawilli Seam)	0.014	2.4	0.25	0	2.4	35	0.1	35	10
Overburden (Hawkesbury Sandstone)	0.025	9.9	0.25	N/A	N/A	N/A	N/A	N/A	N/A
Floor (Kembla Sandstone)	0.025	11	0.22	N/A	N/A	N/A	N/A	N/A	N/A
					Peak cohesion (MPa)	Peak friction angle (⁰)	Normal stiffness (GPa/m)	Shear stiffness (GPa/m)	Initial Joint Deform.
Joints	N/A	N/A	N/A	0	0.1	30	26	2,6	Yes

A gravity loading type has been assumed. The stress states in different units have been assumed as proposed by Mine Advice using the Tectonic Stress Factors (TSF) for major and minor horizontal stresses.

6.1 Elastic settlement of the web pillars

Figure 17 shows surface subsidence due to the settlement of pillars. As evidence in this figure, the maximum surface settlement over the entire panel is approximately 7.5mm, which is comparable to the estimate of Mine Advice.

Another finding of this study is that despite the 204m wide panel, the load acting on the pillar located in the centre of the panel is approximately 7MPa, which is approximately 96% of the full tributary area load.

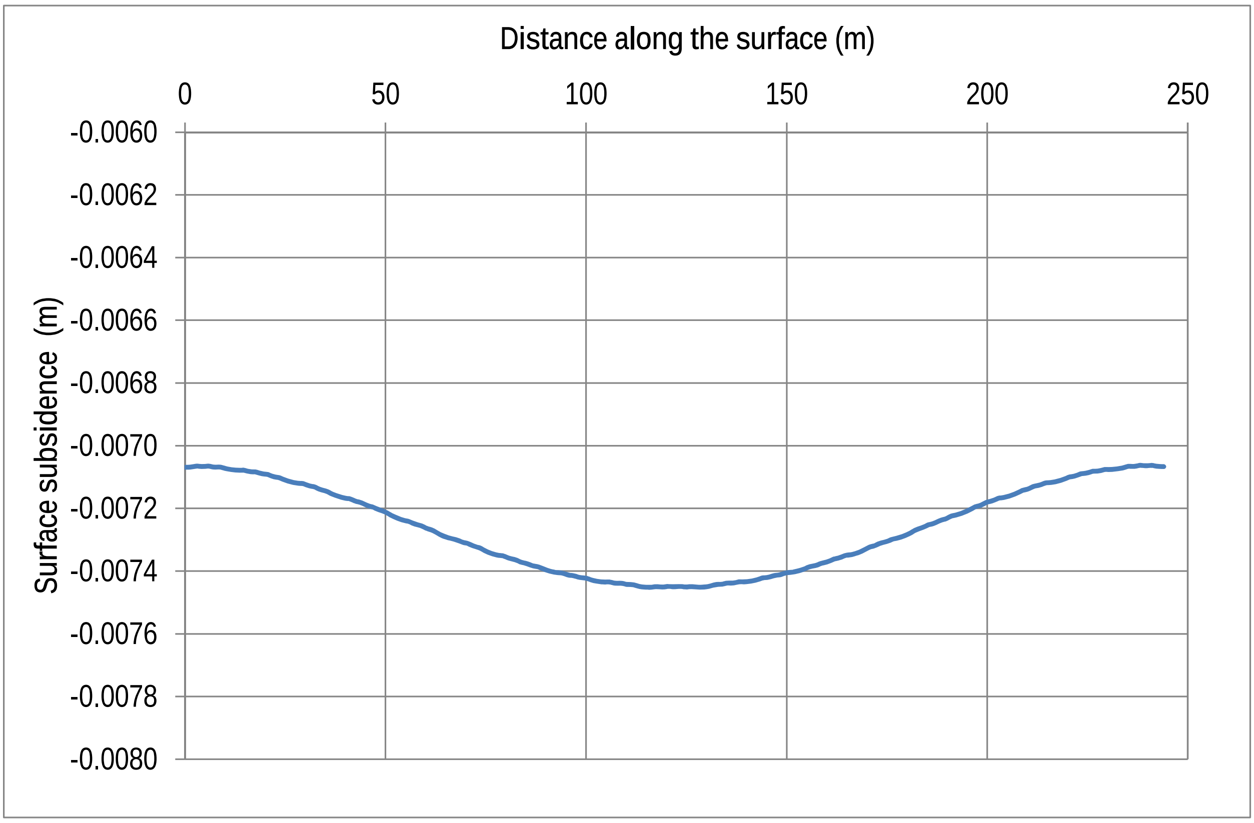

Figure 17. Elastic settlement of an elastic pillars on surface

As indicated in Table 10, the ribs of the pillars located at depths of >80m will be in a state of yielding. In order to assess the expected surface settlement in inelastic state, the same models were run using inelastic pillars in the following section.

6.2 Inelastic settlement of web pillars

Using the input parameters presented in Section 6 and the layout presented in Figure 16, a further study has been conducted to estimate the pillar settlement and surface subsidence in the case of inelastic coal pillars. Figure 18 shows the surface subsidence above the panel in the case of inelastic pillars. It is apparent that up to 200mm of surface subsidence can be expected if the panel is as wide as it is modelled.

From the results above it can be concluded that when the web pillars are in a completely elastic state, the calculations provided by Mine Advice are acceptable. However, as seen in the numerical modelling section, it is likely that the proposed web pillars will not be in an elastic state at depths greater than 80m. Therefore, the inelastic settlement of the pillars using inelastic coal material properties is assessed for the proposed geometries in the following section.

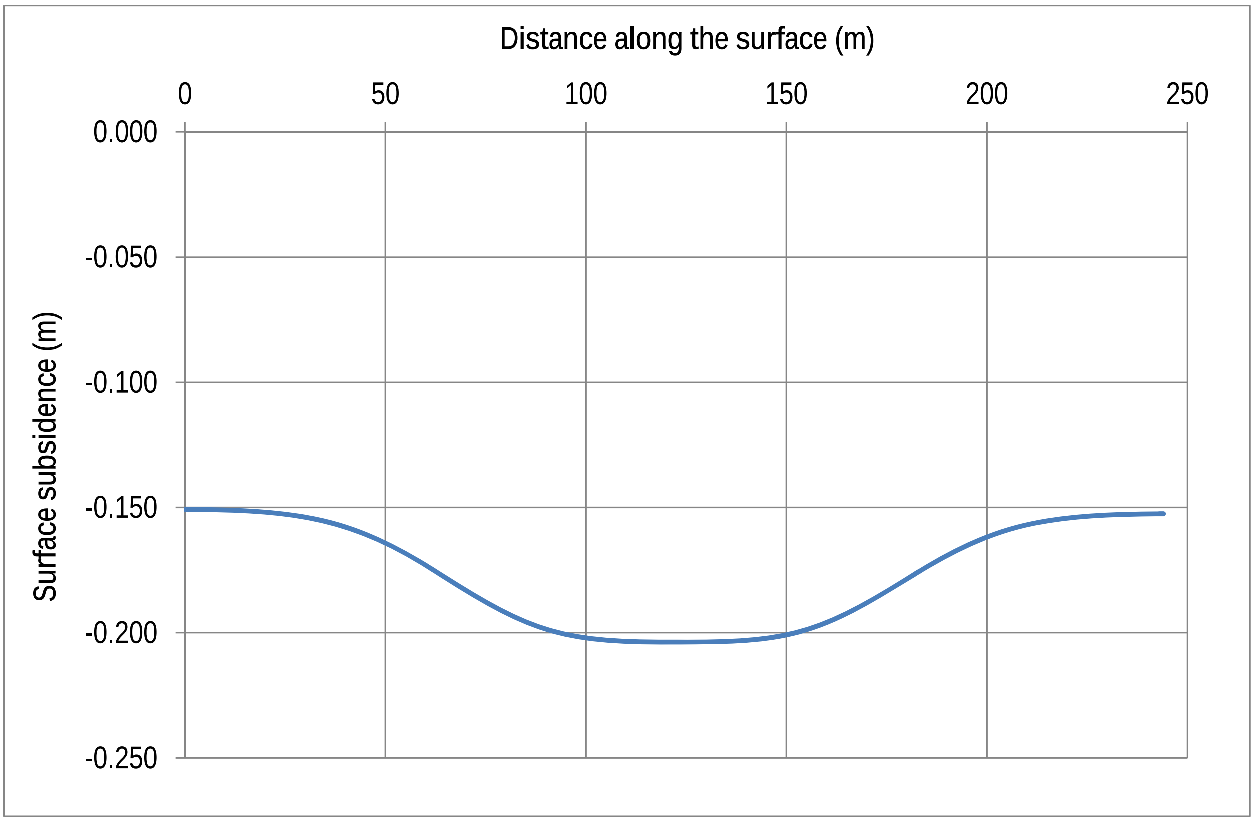

Figure 18. Surface subsidence due to settlement of inelastic pillars

6.3 Surface subsidence of web pillars in proposed layouts

Following on from the above, a further modelling study has also been conducted to estimate the likely subsidence on the surface for the layouts proposed by Mine Advice. In this study, identical RS2 input parameters have been used, but the layouts and dimensions at depths of 80m, 120m and 170m have been assessed, as presented in Table 8. In these models, four web-panels are modelled, as shown in Figure 13, and each panel is mined in subsequent mining steps.

Results from this study are presented in Figure 19 to Figure 23Figure 22. These figures highlight that at greater depths the surface subsidence can reach up to 45mm. Although this level of subsidence is not significant, it is nevertheless greater than 20mm.

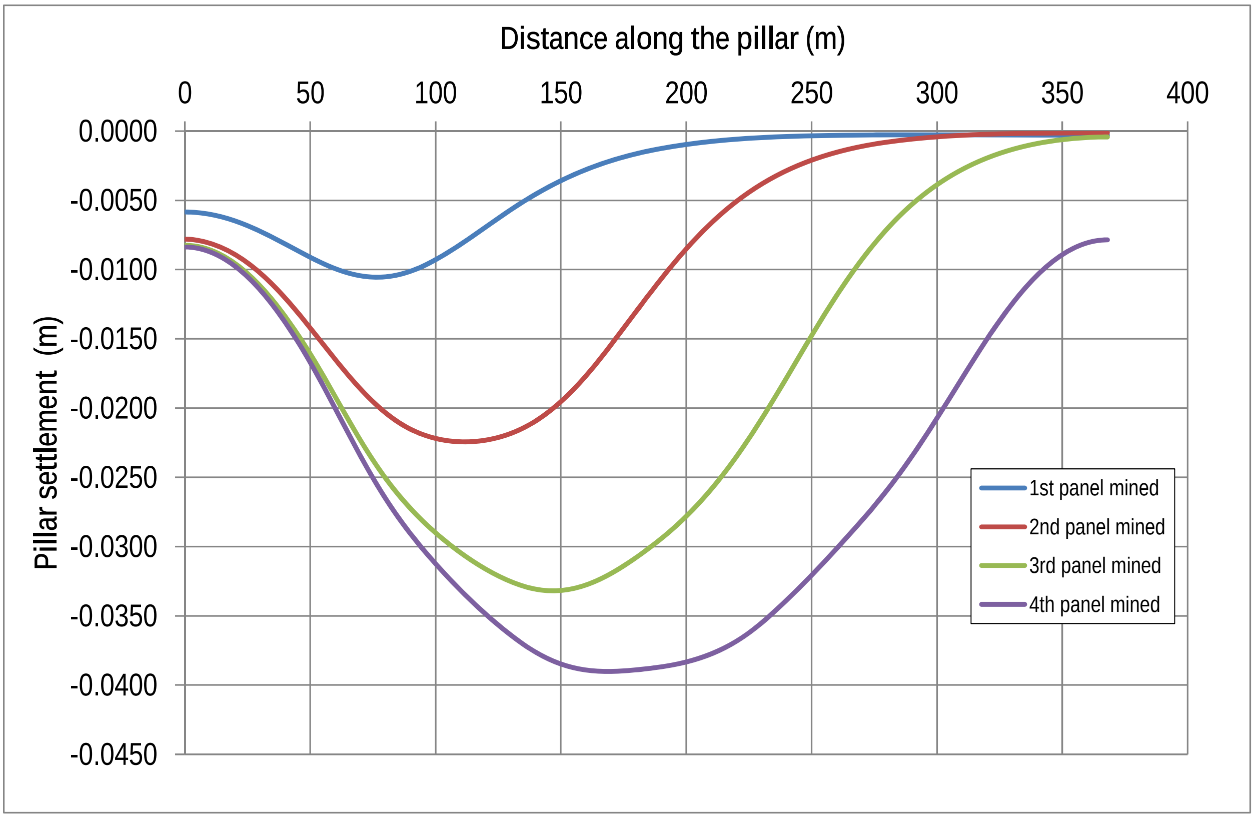

Figure 19. Surface subsidence at 80m depth following the extraction of four panels

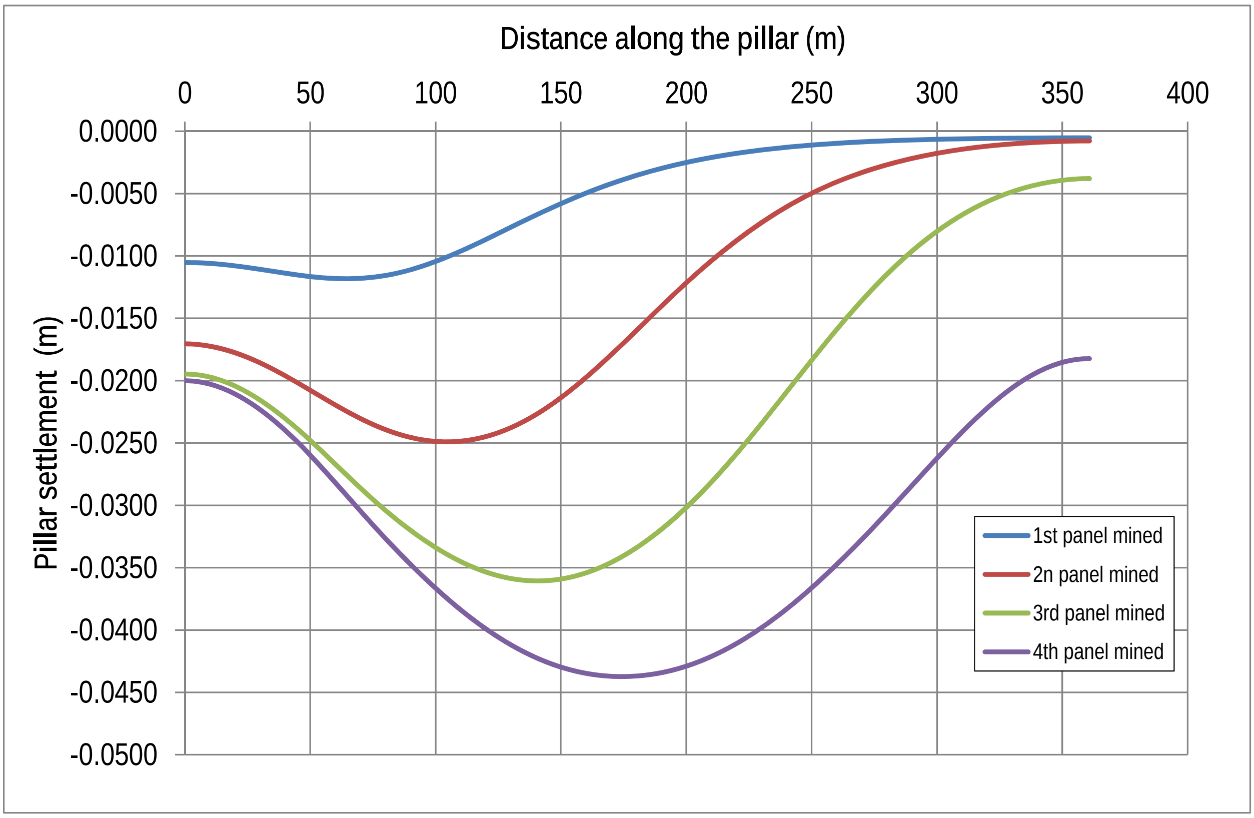

Figure 20. Surface subsidence at 120m depth following the extraction of four panels

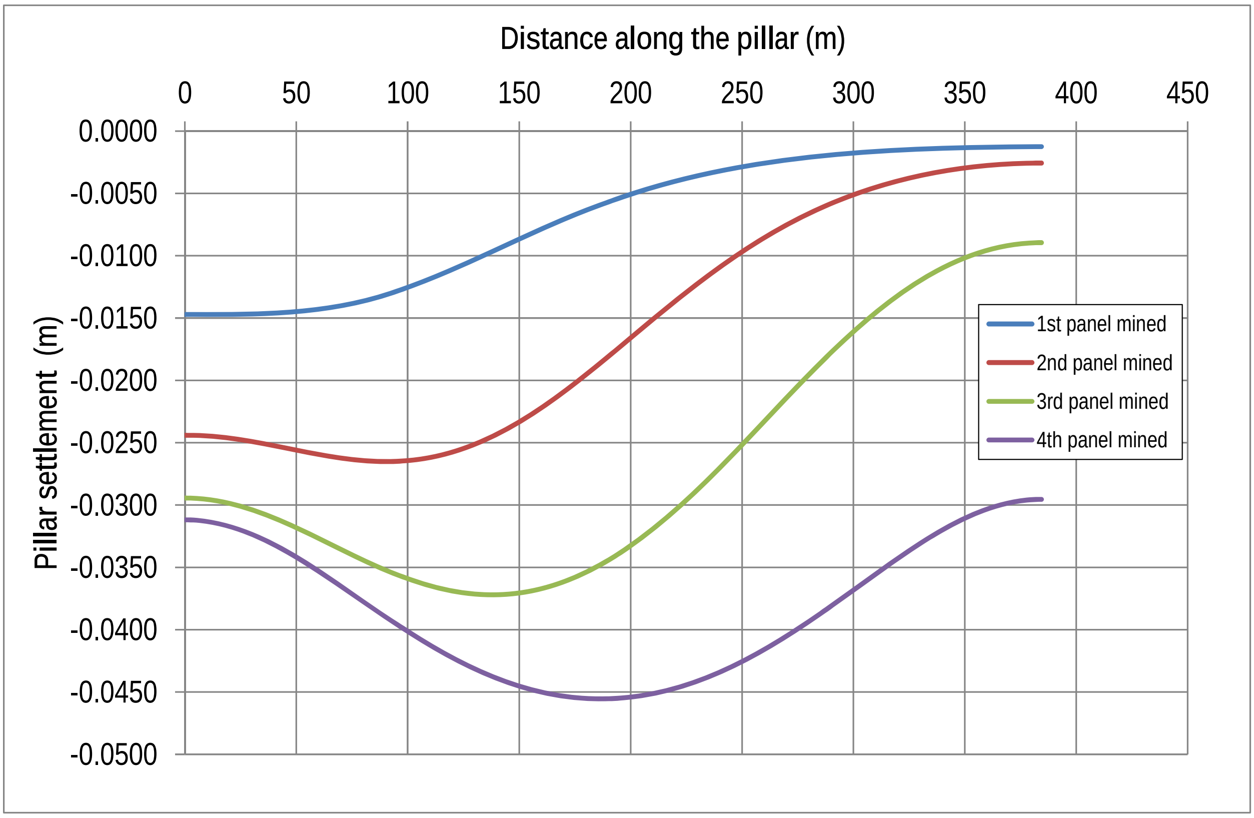

Figure 21. Surface subsidence at 170m depth following the extraction of four panels

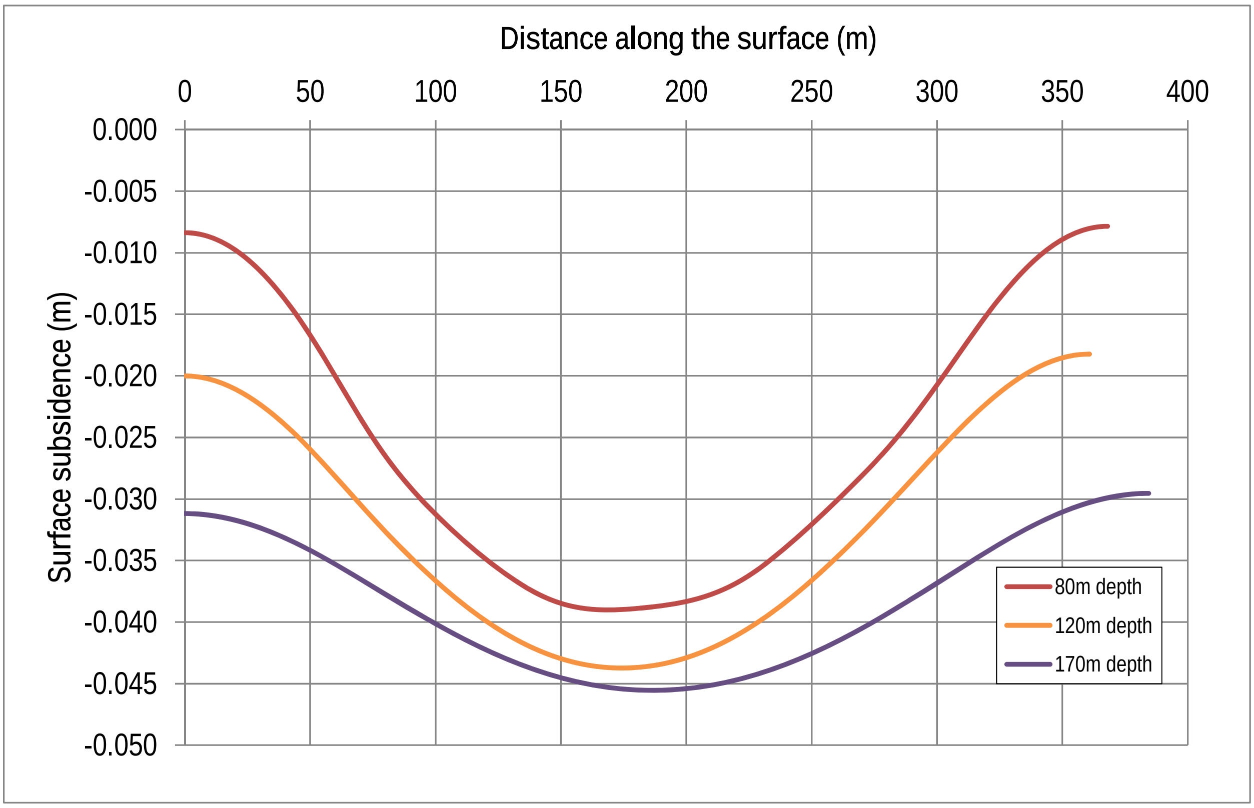

Figure 22. Comparison of surface subsidence at 80m, 120m and 170m depth following the extraction of the four panels

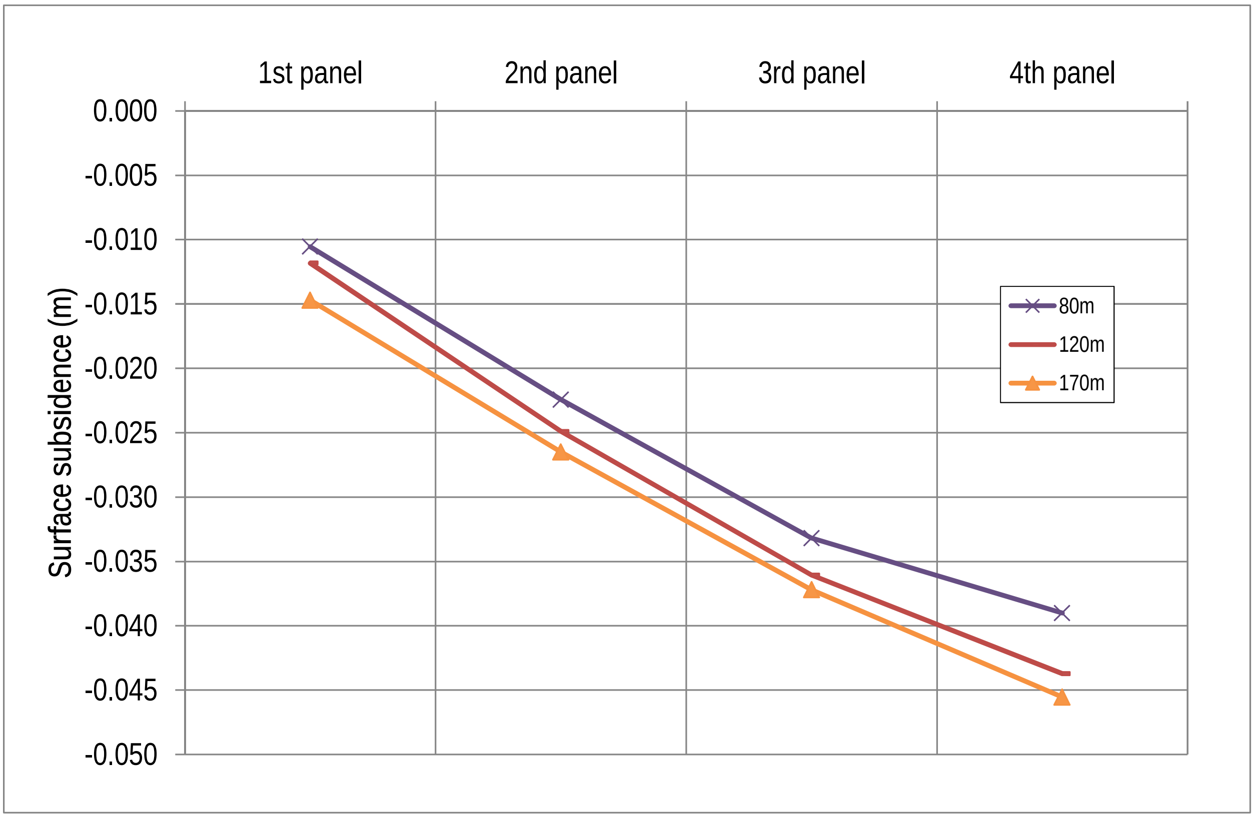

Figure 23. Comparison of maximum surface subsidence at 80m, 120m and 170m depth following the extraction of the panels

6.4 Worst-case surface subsidence in the case of complete failure of web pillars

In order to determine the maximum possible surface subsidence in the case of a complete failure of the web pillars, an additional model has also been constructed at 170m. Similar to the preceding section, all four panels were extracted in a sequence and in an additional stage all web pillars located in the panels were removed, representing a complete failure of web pillars in all web-panels. Figure 24 shows the subsidence along the surface. This figure indicates that in a case of failure of web pillars, the surface subsidence can be in the range of 80mm. Although the tilts and strains caused by this maximum level of surface subsidence will not be significant on natural features, the expected impacts on man-made features should be assessed. Therefore, a subsidence management plan is recommended to manage the expected subsidence and measure its potential impacts on public safety, the environment, community, land use, surface improvements and infrastructure.

6.5 Best-case estimate of pillar loading

Another model was also constructed to determine the stress on the web pillars when they are in an elastic state. This model can be considered as the best-case model as all materials (i.e., coal overburden and floor) in the model are elastic. The modelling results indicate that the average stress acting on the centre pillar is approximately 6.6MPa, which is approximately 90% of the tributary area load of 7.25MPa in the proposed layout. This finding suggests that even under the elastic state (i.e., the best case scenario), the load acting on the web-pillars will be approximately 10% lower than the tributary load; therefore, the FoS and the associated PoS of the web pillars will not be significantly lower than the tributary are load estimate in the proposed narrow panel widths.

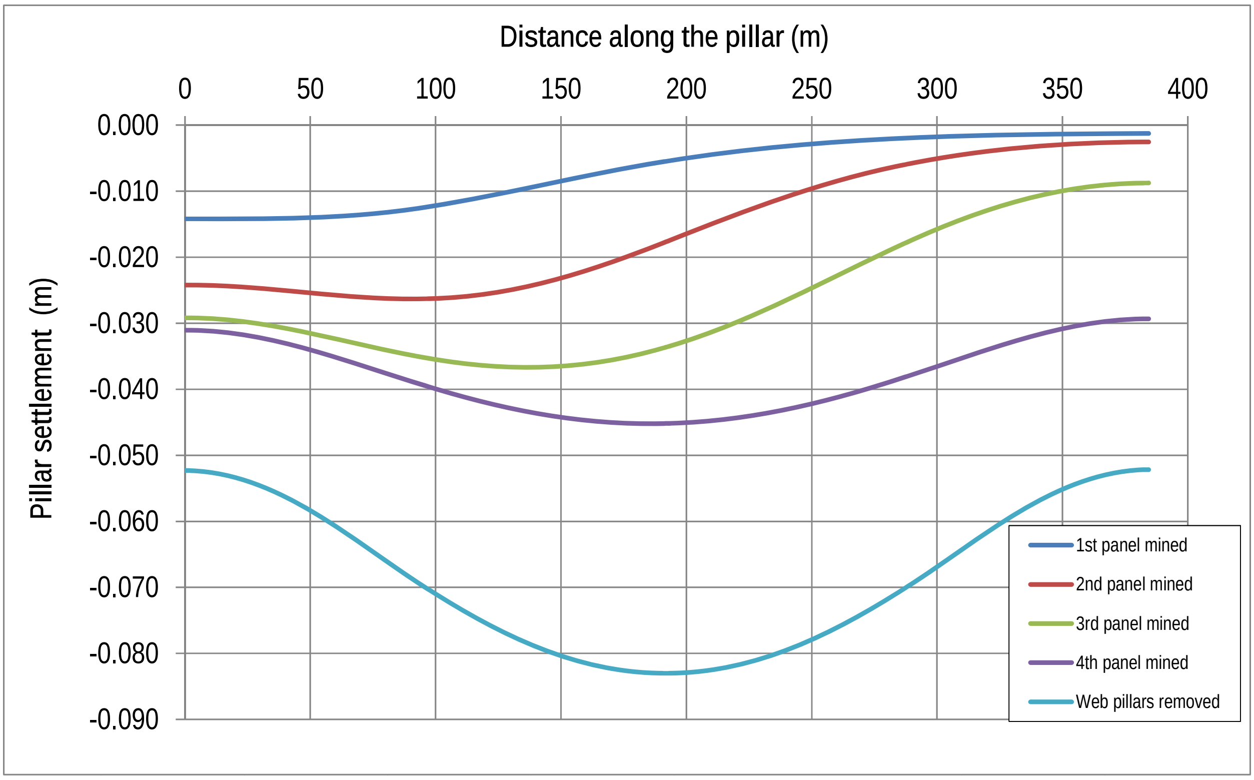

Figure 24. Surface subsidence in worse case, i.e. all web pillars have failed with no remnenat

6.6 Assessment of the stability of the intra-web pillars

In the case of a failure of web pillars, the load will be redistributed onto the intra-web pillars. Therefore, their stability is critical. Their stability is also assessed in these models using the above inelastic coal material properties. Figure 25 indicates that in the case of a complete failure of the web pillars, the Strength Factor (i.e., strength/load) of the core of the intra-web pillars is approximately 1.6.

Figure 25 also shows that the shear and tensile failures in the ribs of the intra-web pillar will extend approximately 7m into the intra-web pillars; however, the core of the pillar will remain relatively stable.

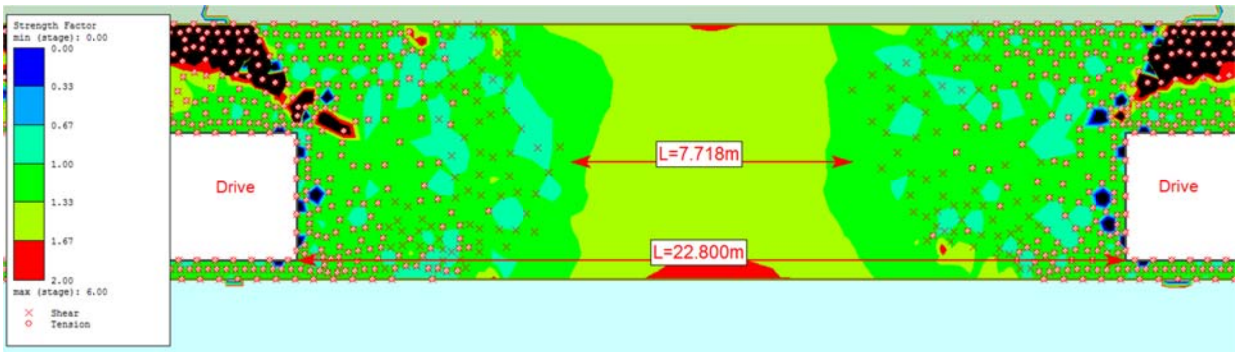

Figure 25. Strength factor of the intra-web pillar in the case of complete failure of web pillars

7 ASSESSMENT OF GATEROAD PILLAR STABILITY

It is assessed that the FoS of the gateroad pillars are acceptable under development loading with the assumption that the web pillars will be long-term stable. However, in the case of failure of the web pillars, the gateroad pillars will be subjected to abutment loads, which can be estimated using the simple single side abutment-loading model presented by Mark (1990) rather than numerical modelling, due to the simplicity of the calculations.

Figure 26 shows the FoSs' of the gateroad pillars at different depths under single abutment loading (i.e., web pillars located only on one side of the gateroad pillars failed). This figure indicates that when the gateroad pillars are subjected to abutment loads (using an abutment angle of 21^{o}), they can start yielding and become unstable at depths of greater than 130m. The yielding and failure of the web and gateroads pillars can certainly cause surface subsidence significantly greater than 20mm.

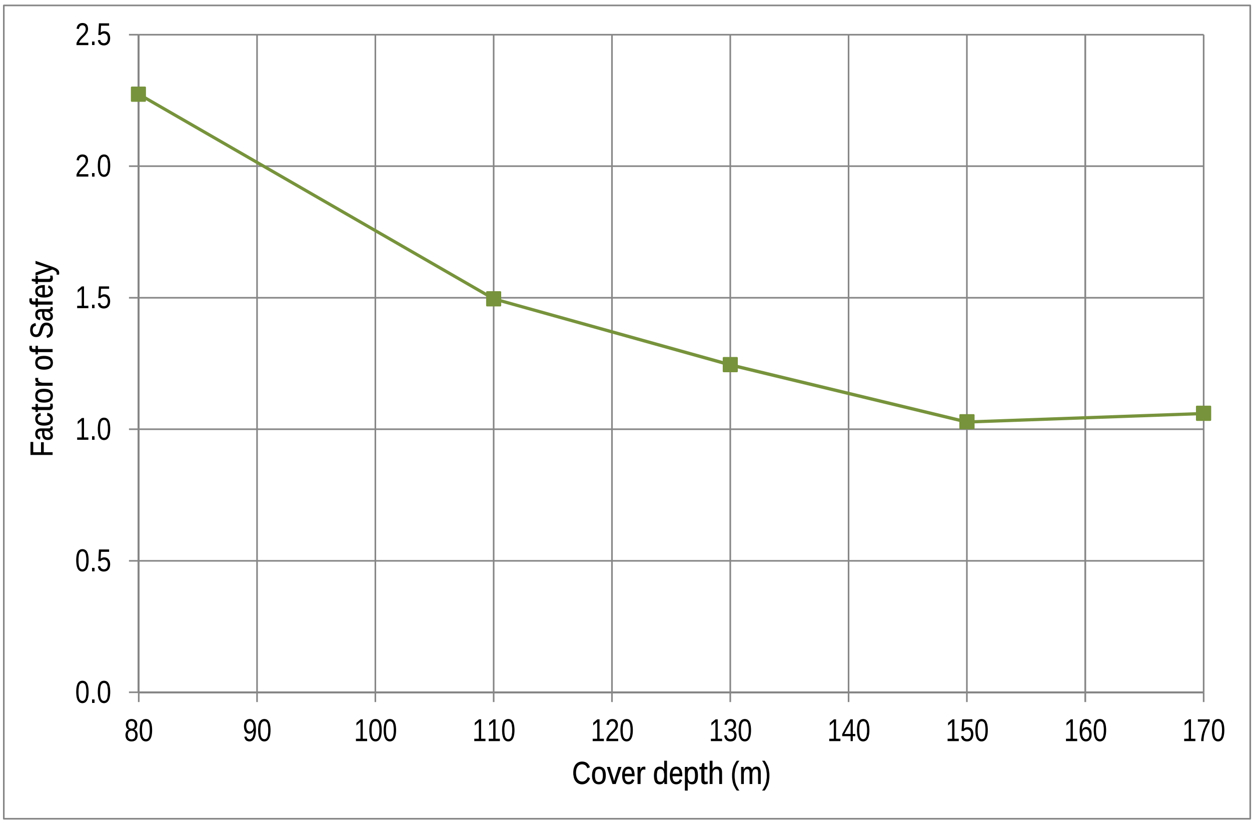

Figure 26. Surface subsidence in worse case, i.e. all web pillars are extracted

It is believed that, for practical reasons, the gateroad pillar width was kept constant by Mine Advice and the length of the pillars increased at 170m cover depth. It is recommended that depending on the final layout and the changes of the cover depth in each panel, the gateroad pillars should be redesigned.

8 SUMMARY AND DISCUSSION

In the past, similar mining layouts which utilise the bridging capabilities of the overburden strata have been practised. However, in this instance the proposed web pillar sizes are unique; the reviewer is unaware of any recent pillar designs that have utilised such narrow, systematic web pillars to ensure long-term stability.

The following operational considerations are relevant to the Hume Coal Project:

- Groundwater and inrush risks
- Geological discontinuities
- Equipment recovery in the drives

- Roof stability
- Off-line cutting and its impact on pillar and roof stability

Inrush has been identified as a potential risk. Another consideration regarding the groundwater is the direction of mining with regard to the dip of the seam. In such conditions where the seam is an aquifer and other aquifers are in close proximity to the target seam, down-dip mining can pose a risk to the underground workforce and cause significant operational problems necessitating extensive water management.

Geological discontinuities can adversely affect pillar and roof stabilities. It is understood that most of the geological mapping will be conducted in surrounding excavations to infer the structures. This technique is mostly accurate for the identification of major geological structures. However, in many cases this technique can be ineffective. In my opinion, in this proposed layout, even minor geological discontinuities, including cleats, joints and rolls, can cause rib spalling and/or roof falls that can adversely affect web pillar stability.

In highwall mining, the actual widths and heights of web pillars can differ significantly from the designed values as a result of off-line cutting brought about due to poor guidance controls. It is understood that Hume Coal relies upon the success of technologies related to guidance control that remain in development. Since off-line cutting can have a significant impact on pillar and roof stabilities, an in-depth study into the degree of potential off-line cutting and its impact on the proposed layout is highly recommended.

In the USA and Australia, a number of reported underground highwall mining failures have trapped mining equipment. Therefore, another critical consideration in highwall mining is the recovery of the CM in the case of a breakdown, flooding or fall of ground, particularly if an entry to the drives is required. It is understood that equipment recovery has already been considered by the mine. However, a management plan with an underpinning risk assessment for recovery operations will only be developed during and following the design and purchase of equipment.

As stated by Mine Advice, the load distribution between web pillars and the intra-panel barrier pillars is almost certainly indeterminate, in that it is directly influenced by a number of unquantifiable geotechnical parameters. Therefore, numerical modelling is the most reliable approach to calculating pillar loads for the proposed layout. Currently available numerical models are highly capable of assessing the bridging capabilities of Hawkesbury Sandstone, the degree of load redistribution and the magnitude of pillar loads. In my opinion this is a major limitation of the pillar design study conducted in EIS.

Mine Advice utilised the ARMPS-HWM method for initial pillar designs followed by the UNSW pillar design methodology to complete the design process to a standard that can be considered as part of a mining application in NSW. In the authors opinion, the NIOSH study was conducted to determine the stability of pillars and roof whilst the mine is active (i.e., to ensure the stability in entries for a short period of time); the reviewer is not aware of any references to the assessment of the long-term stability of pillars in the NIOSH method.

The results from the assessment of web pillar stability using numerical modelling showed that at depths of 120m and 170m, the FoSs' of the web pillars are low, approximately 1.1. At shallower depths (i.e., 80m), the FoS of the web pillars are approximately 1.4. It is of note that the input parameters regarding the strata stiffness and the stress environment were extracted from the Mine Advice report in this study. The results revealed that the proposed web pillars will not be long-term stable.

This numerical study also indicates that the web pillars with a w/h ratio of 1.71 at 170m may have a weaker strength with the 3m coal roof and 0.5m floor. A further study will be required to identify the effect of a thick coal roof and floor on pillar strength. In the meantime, a more conservative design approach using a higher FoS may be considered.

Backfilling is found to be appropriate for Hume Coal. Whilst backfilling has occasionally been used to stabilise old workings, it has rarely been used to increase the FoS during extraction. Backfilling will prevent spalling of the web pillars over a period of time. This will, in turn, ensure the long-term stability of the panels. However, as the numerical modelling of backfilling is not conducted on a real time basis and the property of a backfill is assumed to be settled over time, the effect of backfilling needs to be further studied with field trials. The material properties of the backfill material should also be determined.

The numerical modelling study has also found that the ribs of the web pillars will not be in an elastic state as assumed by Mine Advice. The results showed that at maximum depths of 170m the expected subsidence would be approximately 45mm. The absolute worst-case subsidence in the case of failure of web pillars (modelled as no web pillars) at a depth of 170m would be approximately 80mm. Although the tilts and strains caused by the maximum level of surface subsidence will not be significant on natural features, impacts to man-made features should be reassessed. Therefore, a subsidence management plan is recommended to manage the expected subsidence and its potential impacts on public safety, the environment, community, land use, surface improvements and infrastructure.

Another model was also constructed to determine the stress on pillars when they are in an elastic state as assumed by Mine Advice. This model is considered to be the best-case model, as all materials in the model were elastic. These models indicated that the stress acting on the centre pillar is approximately 90% of the tributary area load (6.6MPa at a depth of 170m). This suggests that even in the best state of pillars with bridging of Hawkesbury Sandstone, the FoS of the pillars will not be significantly greater than estimated by the tributary area theory.

Simple abutment loading models revealed that if the gateroad pillars are subjected to abutment loads, they can start yielding and become unstable at depths greater than 130m. The failure of the web pillars in conjunction with the gateroad pillars in this case can certainly cause surface subsidence greater than 20mm. It is therefore recommended that depending on the final layout and the variation of cover depth in each panel, the gateroad pillars should be redesigned.

9 OVERALL CONCLUSIONS

A pillar design, which uses the bridging capabilities of Hawkesbury Sandstone to limit pillar loads and surface subsidence is credible. However, the degree of load distribution and surface subsidence depends on many geotechnical and geological factors. The most appropriate pillar design tool in this case is numerical modelling that can accurately reveal pillar loads. Without a detailed numerical modelling study to design the pillar system stabilities, an appropriate assessment of the project is not achievable.

A long-term stable coal pillar is generally defined as a coal pillar system that will not result in the failure of pillars over a long period of time, despite rib and roof spalling, which can otherwise cause unacceptable surface subsidence. Once the pillar dimensions are determined, the long-term stability of the pillars can be evaluated using an industry standard or the methodologies proposed by Salamon et al., (1998) and Canbulat (2010). Without this assessment, an appropriate assessment of the project is not achievable.

Even if the proposed web pillars fail, the expected subsidence would be relatively low. However, this low level of subsidence can impact man-made features. Therefore, a subsidence management plan is recommended to manage the expected subsidence and its potential impacts on public safety, the environment, community, land use, surface improvements and infrastructure. The subsidence management plan should be conducted following the above recommended pillar design study.

In my opinion, the Hume Coal's EIS should consider the recommendations given in this report to elicit an appropriate assessment of the proposed mine design.

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