MINING SUBSIDENCE OF AN URBAN AREA IN IPSWICH, QUEENSLAND

By

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ABSTRACT

This paper describes the investigations and results of monitoring of deformations of houses located within the area of influence of a pillar collapse over the Westfalen No. 3 coal mine near Ipswich, Queensland. The subsidence resulted in minor subsidence of at least 18 modern houses and serious damage to 4 others. The history of the events and the measures taken to deal with the physical problems arising are described.

The investigations and results are described and related to the mining methods and geology. The stability of coal pillars up to 9.6 m high was considered and computer modelling undertaken to relate surface subsidence to estimates of the extent of pillar failure. Assessments were made of the likely extent of subsidence during the incident.

Measurements of distortion and tilt were obtained as the ground slowly subsided and criteria established to determine when the structures were deemed to be unsafe. Detailed surveys of houses affected or anticipated to become affected were also undertaken. Four houses were condemned and subsequently demolished as a result of excessive tilt. The methods of assessment are described and the implications of the results related to the various house designs.

INTRODUCTION

On Monday 5th December 1988, a local resident noticed cracks had appeared in McBry Street, Collingwood Park and sought assistance from the local Council. It was quickly realised that a local subsidence trough was developing which had the potential to affect a number of houses.

This paper describes the investigations and results of monitoring of deformations of houses located within the area of influence of a pillar collapse over the Westfalen No. 3 coal mine near Ipswich, Queensland. The subsidence resulted in minor subsidence of at least 18 modern houses and serious damage to 4 others. Levelling surveys were implemented by the Local Authority immediately after minor cracks in the road pavement were first observed and the implications realised. In December 1988, Dr Maconochie was appointed to a steering Committee comprising Government, Local Authority and Mining Company representatives tasked to coordinate investigations, to determine the cause of the subsidence and to predict the future surface effects. Steps were initiated to determine the cause, to preserve public safety and to minimise property damage.

The investigations and results are described and related to the mining methods and geology. The stability of coal pillars up to 9.6 m high was considered and computer modelling undertaken to relate surface subsidence to estimates of the extent of pillar failure.

Detailed surveys of houses affected or anticipated to become affected were also undertaken by structural engineers from the outset. Measurements of distortion and tilt were obtained as the ground slowly subsided and criteria established to determine when the structures were deemed to be unsafe. Four houses were subsequently demolished as a result of excessive tilt.

BACKGROUND

History

The area of interest is situated at Redbank on the West Moreton coalfields in the Ipswich district of SE Queensland. The area lies to the south of Brisbane Road in what has become a well developed good quality suburb between Ipswich and Brisbane.

In the Redbank area, coal was first encountered in a prospecting shaft in 1913 which lead to the establishment of the New Redbank Colliery on the site which now contains the Redbank Plaza shopping centre. Three intervals of what is now identified as a combined section of the Bluff-Four Feet seam were mined between 1921 and 1932. The coal bearing section was 15.2 m thick with 6.5 m of workable coal.

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11th International Conference on Ground Control in Mining, The University of Wollongong, N.S.W., July 1992.
In 1965, the Westfalen company took up an Authority to Prospect over the area of the New Redbank mine and the area to the south. The Department of Mines began drilling in October 1965 when the area was predominantly bushland and continued exploration until August 1969. An econometric seam 7 metres thick containing some 20 million tonnes was identified on the northern side of a major fault running NW–SE near the southern boundary of the lease (Edgar 1976).

Housing Development

As shown in Figure 1, initial steps to rezone the Rural land to Residential A occurred in the early 1970s. A staged development commencing in the area north of Duncom Street (Stages 1 and 2 of Figure 2) saw subdivision commenced in 1975 with subsequent construction of generally brick venner slab-on-ground dwellings after mining had occurred in the area. In January 1974, the mine was flooded and the period of non-production involved in re-establishing the mine saw the surface development underway with a stage by stage the display homes were advertised in Rush Court in November 1977 (Stage 3 of the subdivision), mining of the entry to Panel B under the subsequent subsidence area was just commencing. Development of Stage 4 over the southern half of the Panel B occurred as mining took place in 1979.

At the end of surface development housing subdivision extended from Lowrie Drive to Collingwood Park Drive as shown in Figure 2.

Geology

The geology of the area was defined during the exploration programme which permitted correlation of the seams. The succession can be summarised as follows:

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<th>Group</th>
<th>Formation</th>
<th>Coal Zone</th>
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<td>Basalmen</td>
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<td>Basalmen</td>
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<td>Unconformity</td>
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<td>Upper Coal Measures</td>
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<td>Basalmen</td>
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<td>Triassic</td>
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Within the mine area, erosion at the unconformity has removed the upper seams in places. At the mine entrance, the outcrop is almost entirely of the sandstone group and comprises mainly sandstone and conglomerate (Cairns, 1977) with minor beds of carbonaceous mudstone and grey siltstone.

Consequently, the stratigraphy over much of the mine area comprise light grey, fine to medium grained quartzose sandstones with interbedded siltstone and shale of the Redbank Formation and relatively thinly developed pebbly conglomerate, pebbly sandstone and a hard grey shale tilled pink with iron carbonate. Generally however, the unconformity was close to the upper Bluff–Four foot (main) seam within the area of interest.

Near the outcrop, the strata dip approximately 10° to the east and lessen down dip. To the southeast of the intersection of Collingwood Park Drive and Duncansville, the strata then up approximately 50° to the north. Workings were developed east and southeast of the dome with the eventual western limit formed by a NW–SE trending fault whose upper throw was on the western side increased to the SE. Generally, the main seam was flat lying with grades typically 1 in 7 but steepening to the south. Regional faulting is northwest–southeast and is partly pre–Aberdare Conglomerate. Several faults exist to the south of the workings upthrown on the SE side.

Faulting following the regional direction limited workings of the abandoned New Redbank mine, with upthrow on the south western side in each case.

Mine Development

The Department of Mines determined the conditions under which the lease could be mined which included a limit of 40 percent extraction under roads and built up areas with pillar extraction subject to permission of the Minister. Further limitations were placed on the company if access under railways and waterways was to be developed. Edgar (1976) states that the maximum working height of 6.1 m was to be adhered to by the company but no requirements to this effect were included in the lease conditions. Edgar (1976) indicated that the issue of surface ownership, objections and legal arguments delayed granting of the lease until April 1967.

As the mine was subject to a production quota by the Queensland Coal Board, mining was undertaken by broad and pillar methods. Edgar (1976) provides a detailed description of the method of working which included details of thick seam mining using both a 680 AW4 Marietta bore miner and Joy 6CM continuous miner. After forming panels in the top 6.2 m of the seam, the practice was then to mine bottom coal in the tunnels under the supported roof. The Marietta miner cut a profile with a rounded shoulder whilst the Joy cut a rectangular profile. A method of work was evolved which involved a 3 m timber strip with 1 m expansion shell beds, no roof supports but a straight rib for the Marietta. In the 6CM section the wider span of flat roof exposed made it necessary to erect 5 m long timber strips in the roof, on 3 m legs which only allowed mining of the bottom 3 m in a second 5 m wide lift from between the legs set in the first workings.
Kauthge (1980) provides further details of the method of multi-stripe extraction using a wide head Jeffery continuous miner in the panels SE of Laverie Drive and Duncan Street. He describes in detail the method of ramping down to take bottom coal in areas where the seam ranged in thickness from 10.4 m to 17.7 m. After taking a 2.4 m first pass in a 5 heading panel, up to 5 passes between 1.5 m and 1.8 m high were taken mainly in the outer return headings and supply roadways on either side of the central belt roadway. In these areas roadways could be around 7 m wide and varying between 6 m and 11 m high.


date: 1980

Inefficient space precludes a detailed description of the sequence of panel development. The main features may be summarized as follows:

- multi-stripe bend and pillar
- pillars typically 25 m per side, diamond shaped with a 60° included angle (minimum width approximately 20 m)
- extraction rate 40% by plan
- working height variable: typically 6.1 m but multiple pass extraction to occasionally up to 11 m beyond belt roads
- significant stone bands within section worked
- Typically 18% stone in multi-stripe areas
- depth 115 m to 155 m.

In 1974, the mine was flooded and the upper parts of the mine worked to that time were damaged. Eventually the mine was relanced and worked until closure in 1987.

INVESTIGATIONS

Following initial discovery of tension cracks in several streets in the vicinity of Laverie Drive and McBay Streets and inspections by interested parties and Mines Department inspectors, a meeting was held at the Mines Department to formulate a response and in some respects confirm actions already underway. A steering committee was established to co-ordinate activities between the various bodies who responded to the emergency.

The committee comprised representatives from the following bodies:

- Department of Mines (now Resource Industries)
- Ipswich City Council
- Rhondella Collieries (now PAI Mining)
- SMIRA
- Hollingsworth Consultants (now Hollingsworth Dames & Moore).

Rhondella Collieries were involved due to their acquisition of Westsides Colliery Pty Ltd in 1988. Initial discussions concerned consideration of the development of the subsidence, its possible eventual extent and the various damage and safety scenarios.

The Department of Mines quickly agreed to fund investigations in the interim in order to gather factual data although the legal ramifications regarding responsibility and liability were at that stage not clear as Queensland has no Mine Subsidence Act. In the first few days after subsidence was detected, surface subsidence and crack development continued unabated. At that stage, it could not be determined whether the subsidence was a precursor to the development of a sink hole or widespread collapse of the panel. Consequently, widespread investigations were initiated to cover all eventualities.

As a result of Committee deliberations, the following activities were commenced:

1. Surface levelling and strain monitoring (ICC Survey Department) and crack width monitoring (Department of Mines).
2. Structural inspections of houses (Borahenz and Ward and David Hunt and Associates).
3. Acoustic emission monitoring to provide warning of catastrophic collapses (CSIRO, Division of Geomechanics).

4. Drilling of bores to assess the continuity of strain and condition of pillars in the mine and review of mining records, roof conditions and working height (Department of Mines).

5. Stress analysis of mine pillars and modelling of subsidence (MINCAD).


Hollingsworth Dames & Moore provided advice to the committee, co-ordinated the structural engineers and assisted with interpretation of data as it came to hand.

**Levelling**

A network of pins was installed along the kerbs of Lawrie Drive, Rearden Street, McDay Street and Milgat Street at 10 to 15 m centres on 8 December (Figure 3). The pins were levelled every day by Ipswich City Council surveyors up until 24 December at which time surveys were reduced to twice per week. Initial surveys quickly established that the ground was subsiding at a rate of 116 to 180 mm/week and after 3 weeks this had reduced to approximately 15 to 23 mm/week (Figure 4). The frequency of levelling was reduced as the subsidence rate tapered off.

In addition to the levelling pins, a line of steel fence pickets were installed at 5 m centres along a line through the corner of McDay and Rearden Streets and the Church...
property. Three pegs were levelled and the distance between pegs measured electronically from a datum position in the church grounds.

Structural Inspection of Houses

Domestic residences together with the church in McBay Street were monitored in the area from December 1988 to August 1990. Monitoring of the structures was instigated for the following reasons:

- assessment and maintenance of structural integrity
- safety of the occupants
- to develop comparative data in anticipation of the possible need to assess damage claims
- to assess the performance of modern houses subject to mine subsidence.

Monitoring of the structures entailed visual inspections, notation of any visual signs of distress, taking of levels at selected positions on the structure, measurement of crack widths, measurement of gap widths (e.g. gaps between concrete paths and brick walls of houses), subsequent calculation of tilt (variation of the structure from the horizontal expressed as a ratio of 1 vertical in X horizontal), and discussions with residents regarding new developments/district since last visit. All levels taken on the structures were related back to a datum remote from the site – thus any variations in readings were absolute.

Recording of levels and measurement of cracks and gaps were carried out at decreasing frequency as time progressed. Initially recordings were taken at 1, 2 or 3 day intervals. As a clearer understanding of the extent of the subsidence was known, the frequency was reduced to once a week. This continued until July 1989. Thereafter monitoring was carried out on a monthly basis until cessation of the programme in August 1990.

In total, some 45 houses in Lawrie Drive, Milton Street, McBay Street, Roncea Street, Rush Court, and Flinn Street were inspected of which approximately 22 were monitored as described above. Of the 22 houses monitored, 18 of these were purchased by the State Government, 4 of which were demolished in Lawrie Drive as shown on Figure 3.

Acoustic Emission

Soon after subsidence was confirmed to be continuing, Mines Department engaged Dr Brian Wood of CSIRO Division of Geomechanics to install two noise detectors to monitor the rate of acoustic emissions from below the surface. Acoustic emissions are produced by a number of mechanisms associated with the release of energy from slippage and crushing of rock. The objective was to compare the rate of emissions and provide early warning of accelerating surface subsidence.

Three boreholes, numbered BH1 to BH3 were drilled over the period December 13, 1988 to 26 January 1989, as shown on Figure 3. A total of 273.7 m of open hole using a downhole percussion hammer and 201.5 m of HQ diamond core were drilled. Field supervision for all holes was done by the Coal and Oil Shale section of the Queensland Department of Mines who also logged the open hole drilling. All core was logged by Hollingworth Danes & Moore.

All holes intersected about 4 m of clay soil from the surface, followed by 4-5 m of sandy alluvium. Beneath the alluvium, the holes passed through sandstone, siltstones, and mudstones from the Triassic Raceview Formation and Aberdare Conglomerate then siltstone, claystone, mudstone and coal from the Blackstone Formation.

BH1 was targeted to drill into the intersection of two mine roadways, while BHs 2 and 3 were targeted at pillar areas.

Lithologies

Cores from the Raceview Formation show that the sequence comprises very thinly bedded sandstone, siltstones and mudstone distinguished mainly through sections having higher proportions of a particular lithology.

Figure 3 - Site Plan and Contours of Subsidence to 31 Jan 89

Drilling

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thick was intersected at 88–91 m.

The Aberdare Conglomerate, the top of which was identified at 134 m in BH2, comprises a coarse and fine sandstone conglomerate and also mudstone. The lower boundary with the Blackstone Formation which comprises mainly claystone, silstone, mudstone and coal is not clear.

The coal was intersected in only BH2 from a depth of 155.0 m. The best coal was intersected to a depth of 160.9 m.

Bands of other lithologies increase down the seam and no coal was intersected past about 171 m. Silstone and sandstone were intersected below the coal seams.

Detailed logging assessed rock quality and defect type. In BH2 broculation and shattering of the core generally increased towards the coal seam, with the exception of relatively unfractured brittle siliciclastic beds immediately above the seam (150–154.4 m). Core in the coal section of BH2 is mostly completely shattered and stress breaks are present in the interbedded sandstone bands.

The core from BH3 is similar to the corresponding section of BH2 but from 141 m, fracturing increases markedly. All drilling fluid circulation was lost at that depth and the loss could not be sealed off despite strenuous efforts. Advancing with no circulation, increasingly shattered core was recovered before the hole was abandoned at 150.44 m, estimated to be 1–5 m above workings level.

In the non-cored BH1 borehole, a void was intersected at a depth of 144.6 m. Apparently harder material was found at a depth of 149.3 m as the drill tentatively probed further. The roof of the workings should have been intersected at a depth of 152.0 m, based on mine survey data.

**SUBSIDENCE**

Surface levelling up until October 1991 has showed that the centre of the trough was located along a compression line shown on Figure 3 which parallels the regional structural trend of NW–SE faulting. Tension cracks between 3 to 5 cm wide but up to 8 cm in places were first measured in McBay Street and the Church car park.

A compression ridge 30 cm wide and 10 cm high was recorded in the park and appeared in the road pavement in Lawrie Drive as a narrow ridge approximately 7 cm high. This ridge may have developed overnight on the 4 December. Subsequent tilt measurements along the base line of pegs set up at right angles to the general crack direction in McBay Street are shown in Figure 5 with the recorded level changes. The trough is approximately 50 m wide terminating in the vicinity of the northern kerb of Lawrie Drive and continuing to the SSE an unknown extent. The area between BH2 and the corner of McBay and Recorden Streets comprised a series of blocks which moved en masse separated by tension cracks.

Figure 4 Subsidence of selected pegs since 9 Dec 1988

Maximum tilt occurred in a right arc around the edge of the pronounced trough. Subsequent levelling and distance measurements indicate that the centre of the trough is steadily subsiding at the rate of about 0.25 m per year. Strain and level measurements taken in the centre of the trough indicate ground heave is developing which is indicative of strain crushing due to high local compressive stresses.

However, when the subsidence recorded by the 31 January 1989 is contoured in Figure 3, it is apparent that the effects are not symmetric with a pronounced wide zone of influence and tensile strain to the SW of the centre of the trough. At that time, the maximum subsidence recorded of 400 mm reduced very sharply to the north and northwest.

The mining beneath this area was limited to the SW by a steep roll upwards in the seam which has facilitated the development of a wide and very pronounced tensile zone (Figures 3 and 6).

Comparisons of levels recorded on a truck sewer main crossing the subsidence trough indicate that around the trough perimeter at least 0.15 m and 0.6 m in the centre had occurred before levelling commenced. Consequently the maximum gross subsidence to date is approximately 1.3 m.

**House Monitoring**

The Government set down guidelines for assistance to property owners. To be eligible, the property had to be wholly or substantially within the “affected area” and had suffered damage as a result of the subsidence of the “affected area”. The “affected area” being defined at being within the 10 mm subsidence contour detailed in Hollingworth, Damen & Moore’s report of February 1989. For a property to be considered for purchase by the Government, the following apply:

- it cannot be economically repaired, or
- it is the subject of a demolition order, or

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- the owner/occupier can provide evidence that he has been offered employment elsewhere, or
- the circumstances are accepted by the Minister.

An offer to purchase being based on the market value at the time of offer assuming no damage due to subsidence.

The houses in the subsidence area were generally of the two following types of construction:
- single storey, brick veneer external walls, timber framed internal walls, concrete slab on ground, concrete or terra cotta roof tiles
- two storey, upper storey as for single storey except timber framed floor, lower storey walls being single leaf brickwork with engaged brick piers, concrete slab on ground

In addition, there were basically three types of footing/slab systems which affected the performance of the houses during the subsidence event. The two storey houses had strip footings supporting external load bearing walls with pad footings to internal posts or piers. The lower storey floor slab was a flashing slab, i.e. totally independent of the strip footings and load bearing walls. The single storey houses had either timber stiffened raft type footings/slab or independent strip footings to external load bearing walls with concrete slab not tied to footing beams.

Understandably, the performance of the stiffened raft system was far better than the other two in all locations of the subsidence profile. Although a house on such a raft slab may have undergone an unacceptably high tilt, the extent of distress/damage to the walls/cladding/brickwork was comparatively minor.

One of the most badly affected houses in Lawrie Drive was a single story house with strip footings and concrete floor slab not tied to footing beams. Being of brick veneer construction, the brickwork was supported on the footing beam and the load bearing timber framing of the external walls positioned on the edge of the slab. This house was located in the high tensile ground strain area with a ground crack located diagonally under the house. This house was virtually torn apart as the strip footing moved away from the edge of the slab taking the brick wall with it. The load bearing timber framed wall on the edge of the slab was in danger of losing the support previously offered by the footing under.

Another house which was closer to the centre of the subsidence on Lawrie Drive, and of two storey construction as described above was located in an area of high tilt and high compressive ground strain. In this instance, strip footings and the pad footings to brick piers supporting an upper level concrete patio slab were being moved inwards relative to the opposite side of the house. This resulted in cracking of the brickwork and severe loss of verticality of the load bearing walls and piers – eccentric loading.

As monitoring of the houses progressed and the volume of data increased, it became evident that, with regard to tilt, certain thresholds persisting to the assessment of the structural integrity could be established in general terms. Those thresholds were:
- 1 in 150 to 1 in 100 tilt. Minor damage requires to be considered as possible potential loss of structural integrity. Tilt is becoming visibly noticeable.
- 1 in 100 to 1 in 75 tilt. Tilt is visible, crack widths increasing. Structural integrity needs serious evaluation by calculation. Closer visual inspection required at monitoring times.
- Greater than 1 in 75 tilt. Possible loss of structural integrity. Temporary works may be necessary to maintain such integrity.

In the latter instance, the house at the corner of Lawrie Drive and Milgate Street was in an area of high ground tilt. Before demolition this house attained a tilt of 1 in 47. Immediately prior to this, it was recommended that certain exterior wall be provided with additional lateral support. The combination of lateral wind loads and out of plane vertical dead loads exceeded the capacity of the bearing walls in the house which were originally provided for lateral wind loads only.

Houses in the locations of high ground strain required individual assessment. No thresholds were universally applicable. Houses on adjacent allotments south of McBay Street in Lawrie Drive were in an area of high tensile ground strain, low tilt, and low subsidence. The house described earlier was torn apart whereas the adjacent house to the north underwent no significant movements and suffered no distress. This was a result of two factors – different footing/slab systems and the location of tensile ground cracks. The entire area of lot 41 was located between parallel ground cracks and was undergoing any subsidence effects en masse.
In general terms, the visible effects to houses as a result of the subsidence were:

- cracking of brickwork and interior linings
- jamming of doors and windows
- walls visibly out of plumb
- window frames out of parallel with jamb
- separation of joints in paving slabs
- roof ridge lines not horizontal
- cracking of ceramic tiles
- cracking of glass in windows
- broken drainage pipes at point of connection to house
- water level in pools not parallel with coping tiles.

In appraisal of all the measurements recorded on the houses, (i.e. subsidence, tilt and crack widths) it is evident that when each parameter is plotted graphically against time, the resulting graph is exponential in nature (For example Figure 4). The major proportion of the total change in any parameter occurred within 2 to 3 months after the commencement of recordings. It is also considered that significant subsidence effects had already taken place by the time monitoring systems could be instigated and readings taken. For example, at the time of the first readings being taken on the house at Mitigate Street on the 19.12.88 the tilt in the house was already 1 in 74 - at the time of the last reading prior to demolition on the 06.04.89, the tilt was 1 in 47. Similarly, the gap between the concrete patio slab and brick wall of the house at 4 McBay Street was already 42 mm on the 10.12.88 and 106 mm on the 06.07.90.

**PILLAR STABILITY AND SUBSIDENCE MODELLING**

The aim of the modelling was to back-analyse the observed subsidence and assess pillar stability. The likely pattern of pillar deformation and load transfer from the yielded pillars to surrounding pillars and abutment areas could then be determined.

The analysis was performed with the three-dimensional displacement discontinuity method program MINLAY (Waddle, 1984; Waddle, 1986). The detailed three-dimensional extraction layout was modelled in plan by a uniform rectangular grid of elements. The area modelled consisted of 120 x 120 elements each of size 2 m by 2 m, covering 240 m x 240 m.

MINLAY models the rock mass as a number of homogeneous, cross-anisotropic parallel layers. A better match to observed subsidence profiles is obtained with appropriate anisotropic rock properties than is possible with isotropic models. A relatively low shear modulus is used to represent gross slip on bedding planes.

A simplified stratigraphic section that incorporates the significant material property contrasts was derived from logs obtained from the boreholes in the subsided area. The model material parameters were scaled to approximate the observed subsidence characteristics.

A series of alternative configurations of yielded and non-yielded pillars were modelled and the subsidence predictions for the two modelled pillar geometries compared with the observed subsidence profile. The subsidence bowl was best approximated by an areal extent of yielded pillars consisting of 3 or 4 rows of pillars. These yielded pillar configurations were used to study the redistribution of vertical stress at seam level following pillar yield and compared with pillar strengths estimated by a variety of theories.

**Pillar Stability**

The ultimate strength of a pillar is a function of many factors including its physical dimensions, the properties of the coal and the surrounding roof and floor rock and so on. It is beyond the scope of this paper to either discuss these numerous equations that have been proposed to calculate ultimate strength of pillars or to determine likely pillar strengths. However, it should be noted that pillar strength estimates based solely on laboratory coal strength tests have generally been found to be unreliable. Pillar strengths should be estimated using empirical data from actual pillar performance if at all possible as this represents the best available data.

The best known empirical pillar strength equation is that of Salamon and Munro (1967), who analysed statistically pillar failures in room-and-pillar mines in South Africa and derived the following formula:

\[ q_p = k \cdot w^{-0.6} \cdot h^{-3.7} \]
where $c_p$ is the pillar strength, $k$ is a constant and $w$ and $h$ are the width and height of the pillar.

Earlier studies by Dr Macconochie for CSIRO of pillar failures in the West Moreton Coalfield had indicated that a value of $k$ was typically 5.6. Schlanger et al (1983) reported on a research program into the strength of coal and pillars within the West Moreton No. 3 mine. Their approach was to develop a theory which considered the weighted cohesion of stone and coal acting as a continuous medium in conjunction with a pillar shape function which considered the width to height ratio of the pillar. The pillar shape function has a similar effect as the Salamon and Munroe equation except that it is linear. The proposed design formula was Schlanger's Weak Coal Model.

The pillar strength predictions from the Salamon and Munroe equations as modified and applied by Hollingsworth, Dunns & Moore were compared with the predictions obtained from the CSIRO coal strength model by plotting pillar strength as a function of pillar width to height ratios. Typically the pillar widths in the mine are 18 to 25 m wide and range in height from 3 to over 9 m. Average width/height ratios are probably about 3.5 to 4.5 which is well within the relevant data range afforded by both strength theories. Comparisons of predicted pillar strength showed that for a width to height ratio of 3, the CSIRO strength is 45% higher than the Salamon and Munroe result for a strength constant of $k=5.6$.

This difference is significant in the determination of whether the pillar factors of safety are adequate or not for the land under review. On average, however, the pillars should have been adequate which indicates that a trigger failure mechanism is required.

Consequently, a simple hypothesis for characterising the time-dependent variation of pillar strength was developed. On formation the pillar will be surrounded by sound roof and floor conditions and have a maximum ultimate strength value. If, for example, one or more intersections bounding the pillar collapse, the ultimate strength will decrease. The likely change in pillar strength due to changes in pillar dimensions was estimated by one of the theories and compared with estimates of pillar load.

Analyses of the panel following mining of the roadways and cross-cuts showed that the average pillar stresses in the area of interest were approximately 6 MPa, compared to the normal premining vertical stress of 2.6 MPa. The resultant pillar strengths using Schlanger's methodology were 8.4 MPa (10 m high) to 10.4 MPa (6.4 m high).

Further analyses were undertaken to consider the transfer of load onto adjacent pillars if several pillars yielded as a result of weakening arising from roof falls by making reasonable assumptions regarding the strength of yielded pillars. Generally, the analyses suggested that the pillars would have to weaken through roof falls for the panel to yield due to the presence of larger barrier pillars around the northern, eastern and western sides. However, sufficiently high stresses were calculated for the pillars to the south of the yielded zone to permit propagation of the yield zone in this direction.

CONCLUSIONS

The rock strata material properties and areal extent of yielded pillars were 'fitted' to match the available subsidence monitoring data. Based on present data it appears that the observed subsidence is the consequence of yield of a group of about 10 to 20 pillars brought about by roof falls.

The resulting surface subsidence caused serious damage to the relatively few houses located within the area of maximum tilt. Given the depth of the workings, local geological anomalies probably contribute to the severe disassociation to the overlying strata.

The present zone of yielded pillars appears to be confined on three sides by substantial areas of intact coal or relatively large pillars. However, the area immediately to the south has a similar pillar layout and is also bounded by the same geological structures. The possibility exists for propagation of the yield zone in this direction.

The potential for further subsidence episodes in the vicinity of this site and elsewhere in the region is high.

Previous experience, including the U.S.A. (Gray and Brune, 1982), suggests that subsidence events can recur at a variable frequency over a time span of the order of decades. If the triggering mechanism is related to the predominant structural features, and with the inevitable degradation of pillars in the mine, the possibility of further pillar failures cannot be ruled out.

ACKNOWLEDGMENTS AND DISCLAIMER

This paper represents the interpretations and views of the authors. The information provided does not necessarily represent the full extent of knowledge or opinion and no inferences are intended to be drawn with regard to other parts of the mine.

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