Appendix A

Technical Information

1. Diagrammatic sections showing mature of an underground void and characteristics of trough subsidence

2. Professor Jim Galvin expert review

3. Moreton Geotechnical Services Pty Ltd Location of pillars analysed by ACIRL
Diagrammatic section - subsidence of a large underground void

Characteristics of trough subsidence.
(left half of profile: vertical components, right half: horizontal components)
COLLINGWOOD PARK GEOTECHNICAL SURVEY

COMMENTS RE:

Review of Existing Information

Commissioned by:

Parson Brinckerhoff

Report No: 0908/1-1a

August 2008
Report to:  Mr A Bulcock
Principal Geotechnical Engineer
Civil Engineering

Project:  Collingwood Park Geotechnical Survey

Title:  Comments Re:
Review of Existing Information Data

Report No:  0908/1-1a

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Date:  *August 2008

Signature:
Emeritus Professor J.M. Galvin
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1.0 INTRODUCTION

In February 1959, the then Department of Development and Mines advised the Qld Housing Commission that no danger from subsidence was expected in the area now known as Collingwood Park. At that time, the area had not been undermined other than in the North East corner by Redbank Colliery many years earlier. The Qld Housing Commission purchased the land and began to develop it. In 1988, the surface subsided in the vicinity of Lawrie Drive over mine workings developed by Westfallen No. 3 Colliery during the 1970s. Another subsidence event occurred in April 2008 resulting in damage to homes in the vicinity of Duncan Street, some 30 years after the area had been undermined.

Parson Brinkerhoff (PB) has been engaged by the Queensland (Qld) Department of Mines and Energy to undertake a scoping study for a geotechnical investigation to determine the cause of past known subsidence and the potential for future subsidence events in the suburb of Collingwood Park. In turn, PB has engaged the author to review and comment on the outcomes of aspects of the scoping study. This report is concerned with Stage 1 – Undertake a review and critique of all existing data.

2.0 DOCUMENTATION REVIEWED

The following documentation provided by PB has been reviewed in compiling this report:

<table>
<thead>
<tr>
<th>Title</th>
<th>Author</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mining Subsidence Assessment Report at Collingwood Park</td>
<td>Moreton Geotechnical Services Pty Ltd</td>
<td>July 1994</td>
</tr>
</tbody>
</table>

3.0 OVERVIEW

Mining in the area of interest was by bord and pillar mining whereby coal pillars are left to support the overlying strata with the intention of restricting mining induced surface movements to imperceptible levels. Surface subsidence over bord and pillar workings can be due to the pillars punching into the roof or floor strata, or failing under load over time, or a combination of both. There is nothing in the information I have reviewed which suggests that pillar punching has been the cause of the surface subsidence events at Collingwood Park. Conversely, there is compelling evidence which indicates that pillar failure has been the cause.
Coal pillars fail when the load acting on them exceeds their strength. The ratio between pillar strength and pillar load is known as safety factor.

\[
\text{Safety Factor} = \frac{\text{Strength}}{\text{Load}}
\]

Coal pillar load increases with increase in depth of mining (because the pillar has to support a greater weight of overburden) and with increase in the percentage of coal extracted from a given area (due to the less coal being left in the pillar to carry the overburden load). The overall strength of a coal pillar is determined by both the strength properties of the coal and by the geometry of the pillar. In the case of geometry, overall pillar strength decreases as pillar height is increased and as pillar width is reduced.

The determination of both the load acting on a coal pillar and the strength of a coal pillar can be quite complex and considerable research has and continues to be devoted to these topics. Advances in computer modelling now permit pillar load to be calculated reasonably accurately in most situations but uncertainty still surrounds the calculation of pillar strength. There is no single formula for calculating coal pillar strength that applies to all situations.

Some pillar design procedures account for uncertainty in the accuracy of pillar load and pillar strength determinations by calculating the probability of pillar failure associated with specific values of safety factor. The most notable and extensively utilised are those of Salamon and Munro (1967) and the University of New South Wales (UNSW), Salamon et al (1996). The three reports under review all make reference to the Salamon and Munro pillar design procedure but predate the development of the UNSW pillar design procedure. Both procedures produce similar pillar strength predictions and probabilities of failure for pillars of the size of those underlying Collingwood Park. One of the main differences between the two procedures is that the UNSW procedure caters for coal pillars that are not square. This advance finds application to Collingwood Park.

The UNSW design procedure has been utilised as a point of reference when evaluating previous findings relating to the stability of pillars beneath Collingwood Park. Table I records the probability of failure associated with safety factors derived from this procedure. It should be noted that these apply only to situations where pillar stability is not affected by adverse geology or soft or weak strata bands in the pillar or in the immediate roof or floor of the pillars. As a guide, it is generally accepted that coal pillars should have a safety factor of at least 1.6 where it is desirable that they remain stable in the long term and the consequences are not too serious should they fail. A safety factor of 1.8 to 2.0 usually applies to situations where the consequences of failure are severe, increasing to 2.2 if the consequences could be catastrophic e.g. inrush of the sea into active mine workings. Higher safety factors are usually required when geological conditions are adverse.
Table 1: Probability of Failure Associated with UNSW Power Pillar Design Formulae (Galvin, 2006).

<table>
<thead>
<tr>
<th>Probability of Pillar Failure</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 in 10</td>
<td>0.87</td>
</tr>
<tr>
<td>5 in 10</td>
<td>1.00</td>
</tr>
<tr>
<td>1 in 10</td>
<td>1.22</td>
</tr>
<tr>
<td>5 in 100</td>
<td>1.30</td>
</tr>
<tr>
<td>2 in 100</td>
<td>1.38</td>
</tr>
<tr>
<td>1 in 100</td>
<td>1.44</td>
</tr>
<tr>
<td>1 in 1000</td>
<td>1.63</td>
</tr>
<tr>
<td>1 in 10 000</td>
<td>1.79</td>
</tr>
<tr>
<td>1 in 100 000</td>
<td>1.95</td>
</tr>
<tr>
<td>1 in 1 000 000</td>
<td>2.11</td>
</tr>
</tbody>
</table>

The calculation of the safety factor and probability of stability of the workings under Collingwood Park is not straightforward and presents challenges to all methods of analysis. Reasons for this include:

1. The pillars are diamond shape, Figure 1. Only the UNSW design methodology caters for diamond shaped pillars and the experience base to validate this methodology is very limited.

2. Both the pillar size and the pillar layout are irregular. Consequently, pillar load is not uniform and can only be calculated with a reasonable degree of confidence utilising computer models.

3. Many of the pillars were formed by mining the coal seam in a series of slices, with the width of many of the pillars and the roadways (bords) changing from slice to slice. Consequently, the pillar sides comprise a series of steps. No analysis techniques exist for reliably calculating the strength of such pillars. In some instances, analysis is complicated further because mining height was variable around the perimeter of individual pillars.

4. The records under review indicate that, in general, there was a degree of variability in mining height throughout the workings, with mining height ranging from about 3m up to around 11m. However, accurate records of this variation appear to be lacking.

5. Even if accurate records of mining height were available, subsequent roof falls would have increased the effective height of the pillars to some unknown extent.

6. The coal seam was dipping in some parts of the mine workings. This can result in pillar load changing across the area and may affect the validity of pillar strength formulae.
Figure 1: A section of the workings of Westfalen No. 3 Collicry beneath Collingwood Park.

7. The acute corners of the coal pillars are very unlikely to have remained intact. In any case, they would have a reduced load carrying capacity in comparison to pillars with right angled corners on which the stability assessment procedures of the past have been based. The spalling of acute pillar corners would result in an increase in bord width, and therefore pillar load, and a decrease in pillar load carrying area, and therefore pillar strength. The net effect is that pillar safety factors computed in the past are likely to be over-estimated.

8. The effect of flooding of the workings is unknown. Flooding can have both a positive and a negative impact on pillar stability. Hydrostatic pressure and buoyancy effects result in a beneficial decrease in pillar load. However, the presence of water sometimes has a detrimental effect on pillar strength.

Any review of past reports needs to be conscious of these limitations and of the state of knowledge existing at the time that they were produced.
4.0 HOLLINGSWORTH DAMES & MOORE 1990

Section 1 - Introduction to this HDM report notes that As per the request of the Queensland Housing Commission, a review has been undertaken of a 1981 mining subsidence assessment report (on the above site). The HDM report provides no information as to the title or authors of this 1981 report. The Introduction goes on to make reference to a Hollingsworth Consultants (HC) report in 1982 and to state that the HDM review (of 1990) considers the results provided by Schlanger et al (1983)....

Reference is also made to considerable research into the strength of coal in the Westfalen No. 3 mine and to a report by the Australian Coal Industry Research Laboratory (ACIRL). The title and authors of the Schlanger et al report are noted in Section 5.0 - References. A CSIRO report concerning a hypothesis for predicting the strength of banded coal pillars at Westfalen No. 3 Colliery is also listed in that section but the date of the report and its authors are not recorded. The ACIRL report is not referenced. I have not been provided with a copy of any of these four reports (HC, Schlanger et al, CSIRO and ACIRL) and have had to rely on information contained in the HDM report of 1990.

HDM reports that No detailed records of working height have been revealed despite intensive investigations and that a review of working heights was undertaken through interviews with previous mine managers....No significant differences with regard to working height were identified compared with previous information supplied and reported in our previous reports, as long as the reports from one manager are discounted as not all his observations were substantiated by independent advice from other managers..

Two approaches have been applied to assessing pillar strength, namely that of Salamon and Munro (1967) and that of Schlanger et al (1983). The Salamon and Munro approach has been modified by reducing the value of the coal material strength parameter from 7.2 MPa to 5.6 MPa as a result of studies by Maconochie and Colwell on pillar strength in the West Moreton coalfield contained in an unpublished report for the CSIRO.

The information provided on the pillar design formula of Schlanger et al (1983) is sufficient to be able to conclude that this approach no longer has currency, albeit that it might produce reasonably accurate predictions in some circumstances. HDM report that it produced a pillar strength that was 45% higher than when the Salamon and Munro formula was applied with a modified strength parameter of 5.6 MPa. This means that had the Salamon and Munro formula been used in its unmodified form, the CSIRO formula would have produced a strength increase of 13%. HDM go on to state that: Consequently, the experience at Lawrie Drive and the back analysis of the failure suggest that the weak coal strength formula for pillar strength in Westfalen No. 3 is reasonable and that the Salamon and Munroe (sic) constant for pillars in this mine is probably greater that the 5.6 we used in section 2.1 and could be as high as 7.2. I presume in the absence of the associated reports that the term weak coal

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1 MPa = 100 tonnes/m²
**strength** formula is used to distinguish the Salamon and Munro formula from the CSIRO formula.

More recent research has produced a strength formula for coal pillars in Australia (UNSW power formula) that agrees closely with the unmodified Salamon and Munro formula (Salamon et al, 1996) for pillars having a width to height ratio less than 5. However, as width to height ratio reduces towards a value of 2, the strength of the coal material rather than the geometry of the coal pillar primarily determines the overall strength of a pillar (Galvin, 2008). Given that the width to height ratio of most of the pillars beneath Collingwood Park is typically in the range of only 2.5 to 3, the Maconochie and Colwell approach of using a modified (reduced) coal mass strength value in the Salamon and Munro formula is quite plausible and cannot be discounted.

The HDM report goes onto to state that: *Typically the pillar widths in the mine are 18 to 25m wide and range in height from 3 to over 9m. Average width/height ratios are probably about 3.5 to 4.5...* Based on back analysis of results in other reports under review and my independent analysis, I suspect that HDM has calculated pillar width to height ratio on the basis of the minimum side length of the pillars. This approach is appropriate for square or rectangular shaped pillars but results in effective width to height ratio being over-estimated in the case of diamond shaped pillars. The effective minimum pillar width (measured at right angles to the pillar sides) for pillar beneath Collingwood Park is only 0.87 times the minimum side length. If minimum side length rather than effective minimum pillar width was also applied in calculating pillar strength, it will have also resulted in pillar strength being over-estimated.

Apparently, ACIRL utilised numerical modelling to determine that the load acting on some pillars was in the range of 6.0 MPa to 7.8 MPa. Suffice to state that, in the absence of the associated documentation, these values appear sensible. HDM undertook further analysis based predominantly on a mining height of 6.1m, a percentage areal extraction of 55% and a coal material strength factor of 7.2 MPa. No allowance was made for changes in pillar width and roadway width between successive mining slices. I have re-run this HDM analysis twice using the UNSW pillar design methodology, taking into account the diamond shape of the pillars. Given the irregular shape and pattern of the mine workings, it is not possible to produce a unique value for safety factor in each zone; the outcome depends on which pillar is chosen within each zone. Suffice to state that my first analysis produced values of percentage areal extraction in the range of 40 to 53%. In theory, the HDM of 55% extraction is over-estimated, but in practice it is quite reasonable when allowance is made for reduction in the effective load carrying area of pillars due to collapsed pillars corners and rib (sidewall) spall.

My second analysis utilised HDM’s values for mining height and percentage areal extraction. Table 2 compares these outcomes with those of HDM. Given the uncertainties associated with factors such as mining height, state of pillar corners, roof falls, effective roadway width and percentage extraction, there is no practical significance between the outcomes of the two sets of analysis in respect of risk of surface subsidence. The risk of pillar failure, and therefore surface subsidence, is high to very high. The HDM analysis indicates that it is at least 30% for some panels, corresponding to the collapse of 3 in every 10 mining panels, whilst the UNSW
analysis indicates it is at least 50% for these panels (Table 1). Little consolation should be drawn from panels with higher safety factors given that:

- There is considerable uncertainty in the parameters that the safety factor derivations have been based on;
- No allowance has been made for any increased load that a collapse may induce on pillars surrounding the collapse area;
- The probability of failure is still unacceptably high by established standards.

If the reduction in roadway width that is reported to have occurred between some slices could be quantified, the pillars may have a higher safety factor. On the other hand, it might be lower if the effective height of some pillars has been increased as a result of roof falls.

<table>
<thead>
<tr>
<th>Area</th>
<th>HDM Salamon &amp; Munro Safety Factor</th>
<th>UNSW Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>B(2)</td>
<td>0.8</td>
<td>0.62</td>
</tr>
<tr>
<td>C1</td>
<td>2.0</td>
<td>2.04</td>
</tr>
<tr>
<td>C2</td>
<td>1.3</td>
<td>1.12</td>
</tr>
<tr>
<td>D</td>
<td>1.4</td>
<td>1.25</td>
</tr>
<tr>
<td>E</td>
<td>1.2</td>
<td>0.99</td>
</tr>
<tr>
<td>F</td>
<td>1.1</td>
<td>0.93</td>
</tr>
<tr>
<td>G</td>
<td>1.2</td>
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<tr>
<td>H</td>
<td>1.1</td>
<td>0.99</td>
</tr>
<tr>
<td>I</td>
<td>1.1</td>
<td>0.94</td>
</tr>
</tbody>
</table>

Table 2: Comparison between HDM Safety Factors and those based on the UNSW Pillar Design Methodology.

HDM concluded from their analysis that:

- ...the factor of safety for the various areas are better than those in the collapsed area. This shows that the factors of safety are however, low and would not be regarded as being satisfactory for long term stability.
- .....most of Portion 55 is considered to be subject to considerable risk of mining subsidence in the medium to long term.
- Subsidence is considered to have developed at Lawrie Drive through the combined effects of roof falls affecting pillar stability possibly accelerated by water inflow.
- It is considered that despite the installation of roof support during mining, in the long term other falls will occur throughout the mine possibly with more likelihood in the vicinity of geological structures.
The back analysis at Lawrie Drive supports the view that if one pillar fails through the effects of roof falls, then adjacent pillars can be over loaded leading to collapse over a wider area.

I consider all these conclusions reasonable and still relevant to the designated areas today. HDM go on to conclude that long term surface stability will be consequently largely determined by preservation of roof stability. I have insufficient information to be able to concur fully with this conclusion. Certainly, it is plausible. However, it is also plausible given the low safety factors of the pillars, pillar failure is also due in part or whole to the deterioration (spalling) of the pillars themselves over time.

5.0 ACIRL 1994

ACIRL was commissioned by Moreton Geotechnical Services (MGS) to provide estimates for strength, stress and factor of safety against pillar failure for six (6) nominated pillars beneath Collingwood Park, to comment on the effect that water can have on the long term stability of the pillars, and to provide some broad comments on the procedure for predicting maximum subsidence effects at the surface.

ACIRL advised that:

assuming accurate estimates (of input parameters) are available, as is the case in South Africa using the Salamon and Munro (1967) equation, good engineering practice uses factors of safety of 1.2 or greater if failure is to be avoided in the short term, and factors of safety greater than 1.6 for long term stability. Using South African equations in Australia requires consideration go possibly more conservative factors of safety (say 1.4 and 1.8 respectively)

This advice is still valid.

ACIRL went on to recognise and discuss the difficulties in calculating pillar load for the irregular pillar layout beneath Collingwood Park. ACIRL noted that:

In discussions with ACIRL prior to commissioning this work, it was decided that refinement of the estimates for pillar stress using 3-D numerical modelling was not warranted in this situation as there was some doubt as to the accuracy of the mine plan and actual workings.

I concur with this reasoning.

The six pillars analysed by ACIRL were located in areas C1, C2, E and G. The pillar analysis differed from that of HDM in that depth of mining was fixed at 127m and percentage extraction was fixed at 45%. These input values gave an average pillar stress of 5.66 MPa, as compared to 7.0 to 7.2 MPa in the HDM analysis in these areas. Factors of safety were calculated for effective pillar heights of 5, 7, 9 and 11m and for coal material strength factors of 5.6 MPa and 7.2 MPa. The analysis was based on equating the effective width of the diamond shape pillars to the width of square pillars using a concept known as ‘hydraulic radius’. This concept has the potential to introduce considerable error when applied to diamond shaped pillars that fail within particular size ranges. Re-analysis for this review indicates that the error introduced
was minor and not significant when compared to all the other potential sources of error. Table 3 compares the outcomes of the ACIRL analysis for a material strength factor of 7.2 MPa with that of the UNSW pillar design methodology (using the simple power formula).

Table 3: Comparison between ACIRL safety factors for a coal material strength factor of 7.2 MPa and those based on the UNSW Pillar Design Methodology - percentage areal extraction = 45% in both cases.

<table>
<thead>
<tr>
<th>PILLAR</th>
<th>FACTOR OF SAFETY FOR MINING HEIGHT OF:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5m</td>
</tr>
<tr>
<td></td>
<td>ACIRL</td>
</tr>
<tr>
<td>A</td>
<td>1.73</td>
</tr>
<tr>
<td>B</td>
<td>1.53</td>
</tr>
<tr>
<td>C</td>
<td>1.77</td>
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<td>D</td>
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<tr>
<td>E</td>
<td>1.71</td>
</tr>
<tr>
<td>F</td>
<td>1.71</td>
</tr>
</tbody>
</table>

The ACIRL safety factors for a coal material strength factor of 5.6 MPa are 77% (three quarters) of the values listed in Table 3. The ACIRL report notes that previous work by Maconochie and Colwell in 1983 relating to pillar failure in the Ipswich Coalfield concluded that the relationship developed by Salamon and Munro (1967), in conjunction with using a strength value of 5.6 MPa, was appropriate for a number of seams within the Ipswich Coalfield. However, the report did not take into account inaccuracy of mine plans (bord widths mined wider than plan) and pillar spall/fret common to pillars within steeply dipping seams. ACIRL appear to be suggesting that the reduced strength value for the Ipswich Coalfield compensated for reductions in the size of the pillars from those shown on the mine plan rather than reflected a weaker coal material strength.

An alternative and more robust approach is to base the analysis on an increased percentage extraction. Table 4 shows a comparison between this approach utilising the UNSW methodology based on 55% extraction and the ACIRL results based on 45% extraction and a material strength value of 5.6 MPa. In terms of the predicting pillar stability, the outcomes are in close agreement. Effectively, even if the pillars had only been extracted to a height of 5m and no roof falls subsequently occurred, they would have had a safety factor that was inadequate for reducing the risk of failure in the long term to an acceptable level. The probability that the pillars will fail in the long term is of the order of 50% if, as a result of mining and roof falls, their effective height approaches 7m.
Table 4: Comparison between ACIRL safety factors for a coal material strength factor of 5.6 MPa and a percentage extraction of 45% and those based on the UNSW Pillar Design Methodology for a percentage extraction of 55%.

<table>
<thead>
<tr>
<th>PILLAR</th>
<th>FACTOR OF SAFETY FOR MINING HEIGHT OF:</th>
<th>5m</th>
<th>7m</th>
<th>9m</th>
<th>11m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ACIRL</td>
<td>UNSW</td>
<td>ACIRL</td>
<td>UNSW</td>
<td>ACIRL</td>
</tr>
<tr>
<td>A</td>
<td>1.34</td>
<td>1.28</td>
<td>1.08</td>
<td>0.96</td>
<td>0.92</td>
</tr>
<tr>
<td>B</td>
<td>1.19</td>
<td>1.06</td>
<td>0.95</td>
<td>0.80</td>
<td>0.81</td>
</tr>
<tr>
<td>C</td>
<td>1.38</td>
<td>1.26</td>
<td>1.10</td>
<td>0.95</td>
<td>0.93</td>
</tr>
<tr>
<td>D</td>
<td>1.33</td>
<td>1.26</td>
<td>1.07</td>
<td>0.95</td>
<td>0.90</td>
</tr>
<tr>
<td>E</td>
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<td>F</td>
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<td>1.26</td>
<td>1.07</td>
<td>0.95</td>
<td>0.90</td>
</tr>
</tbody>
</table>

The ACIRL report goes on to make some general comment on subsidence prediction and the positive and negative implications of flooding of the workings. These are considered reasonable assessments.

6.0 MORETON GEOTECHNICAL SERVICES, 1994

The MGS report is premised in part on the outcomes of the analytical work it subcontracted to ACIRL. Significant points of note in the report are:

- The western edge of the area under the intersection of Collingwood Drive and Duncan Street experienced many roof falls and was affected by the 1974 floods.

- The CSIRO report shows that the pillars measured under the site.....ranged from between 5.1m and 8.4m high. In addition the CSIRO work has shown that large height differences can occur around the same pillar and between nearby pillars. For example, locations A1 and A2 were measured at 7.6m and 5.6m high whereas nearby B1 and B2 were measured equally at 8.4m high.

- Borehole 4 and redrill (Borehole 6) failed to encounter workings even though the drilling of Boreholes 1, 2, 3 and 5 had indicated that the workings were in the correct position in relation to the surface road boundaries.
• A factor of safety (FOS) of 1.6-1.8 is usually adopted for long term stability although there has been at least one recent move to adopt 2.0 as the FOS for long term stability for the Ipswich area.

• Subsidence has already occurred in similar mine workings within 400m of the site to the south.

MGS concluded that:

.....the analysis has shown that pillars 5.0m and higher may or may not be stable in the long term depending on the actual strength of the coal and also the effect of any additional stresses generated on neighbouring pillars if one or more pillars crush. On balance, and in view of the subsidence event that previously occurred close to this site, it is our belief that the worst case scenario should be considered, i.e. that all pillars are potentially unstable in the long term ............

.....

On balance, it is therefore our opinion that it should be assumed that subsidence will occur at some time in the future, particularly when mine water backs up the workings (sic), when roof falls occur or for other reasons such as earth tremors.

Subsequent events confirm the conclusions in respect of pillar stability. However, subject to the nature of the material comprising the floor, pillars and roof strata, it is possible that flooding of the workings could have a stabilising effect rather than a destabilising effect.

The MGS report goes on to provide predictions of subsidence components. I require additional site specific information if comment is required on the reliability of these predictions. Suffice to state that:

• The predictions appear sensible.

• Detailed analysis of their accuracy appears pointless since the magnitudes of the subsidence movements are so large that their order of accuracy is unlikely to make much difference to whether they impact adversely on surface structures. Adverse impacts are inevitable.

Additional subsidence monitoring information would be useful in any assessment of whether collapsed areas have stabilised and whether a new collapse is likely to cause additional movement over areas that have already collapsed.

7.0 OTHER POINTS OF NOTE

The 2008 collapse appears to be bounded by faults on two sides and pillars of larger cross sectional area on one other side. Depending on the dip of the seam and the surface topography, it is possible that variations in overburden load may also have played a role in determining the extent of the collapse.
Faulting can influence both the initiation and termination of a collapse. A fault may be the site of initiation of a pillar failure if adverse ground conditions are associated with the fault. Weaker material, sympathetic micro faulting, increased jointing and water ingress due to faulting are some of the factors that can result in a reduction in pillar strength in the vicinity of faulting. Faulting can also be associated with the initiation of a pillar collapse because the roof beam (or plate) is effectively turned into a cantilever along the fault line, thereby destroying the capacity of the overburden to bridge or span across an area. Consequently, the pillars are exposed to full deadweight loading in circumstances where this may not have previously been the case. This latter effect is more likely to materialise during or soon after mining whilst the former tends to be more time dependent and so is more likely to be associated with the subsidence events at Collingwood Park.

Once a pillar collapse is initiated, load is transferred onto the pillars surrounding the collapse area. This additional ‘abutment’ load then causes these surrounding pillars to collapse, thereby setting up what is termed a ‘pillar run’ or a ‘domino failure’. Faulting can influence the termination of such a collapse because it can act as a barrier to the transmission of the travelling abutment stress front.

8.0 CONCLUSIONS

1. Although the knowledge and experience base concerning pillar stability in Australia has advanced considerably since the three Collingwood Park subsidence reports under review were compiled, the conclusions contained in these reports are still appropriate.

2. These advances highlight that the risk of failure of the Westfalen No. 3 Colliery pillar layout beneath Collingwood Park is high and may continue to increase with the passage of time.

3. Some of this risk is associated with uncertainty regarding the reliability of the mine plans and mining dimensions being used in stability assessments and the suitability of the pillar stability assessment methods to the given circumstances.

4. Whilst further field investigations could improve the reliability of the stability assessment outcomes by providing confirmation of the mine plan, mining dimensions and the current condition of the pillars, this is considered unwarranted. Unless large areas of low working height (~3m) exist, it is extremely unlikely to result in long term pillar stability throughout the area being assigned to a lower risk category, albeit that areas which have not yet collapsed may never collapse. Both theoretical analysis and the extent of the collapsed areas to date indicate that the stability of all the Westfalen No. 3 mine workings beneath Collingwood Park is marginal. The 2008 collapse appears to have terminated against faults and some pillars of larger cross sectional area. Whether this collapse area extends in the future is problematic.
Borehole drilling to obtain additional information to better assess this risk is likely to be cost prohibitive and may still prove inadequate.

5. At this point in the process, it seems to me that there are three way forward for the future:

i. Resign to the fact that the area has a high potential for future mine subsidence and implement a plan to manage the associated risk (e.g. do not accept the risk and relocate residents, accept the risk and develop a plan for managing any future subsidence event).

ii. Backfill those areas that have not already failed, with the primary intention being to prevent failure and the secondary intention being to mitigate risk should the areas still fail. Backfill would probably need to be placed to at least two-thirds of the pillar height to be effective in confining the pillars sufficiently to prevent them from collapsing in the future. Consideration could be given to exempting areas (but not isolated pockets) of pillars having a height of less than 3m.

iii. Evaluate if there is any potential for ongoing subsidence over those areas that have already collapsed. If the potential is minimal, these areas could be released for development/redevelopment. Consideration should be given to implementing a building code which provides structural tolerance to some residual surface subsidence.

REFERENCES

Galvin JM: 2006

Galvin, J.M. 2008
Geotechnical Engineering in Underground Coal Mining – Principles, Practices and Risk Management. ACARP Project C14014. UNSW.

Salamon MDG, Galvin JM, Hocking G & Anderson I: 1996

Salamon, M.D.G. and Munro, A.H: 1967