Scoping study for Collingwood Park subsidence 2009

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Department of Mines and Energy

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Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Terms of reference of the scoping study</td>
<td>1</td>
</tr>
<tr>
<td>2. Methodology</td>
<td>3</td>
</tr>
<tr>
<td>2.1 Literature review</td>
<td>3</td>
</tr>
<tr>
<td>2.2 Key Issues</td>
<td>4</td>
</tr>
<tr>
<td>3. Geology</td>
<td>5</td>
</tr>
<tr>
<td>3.1 Structural geology</td>
<td>5</td>
</tr>
<tr>
<td>3.2 Geology and structural geology of the Duncan Street area</td>
<td>5</td>
</tr>
<tr>
<td>4. Collingwood Park subsidence</td>
<td>7</td>
</tr>
<tr>
<td>4.1 Recent subsidence</td>
<td>7</td>
</tr>
<tr>
<td>4.2 Previous subsidence incidents</td>
<td>7</td>
</tr>
<tr>
<td>5. Mining history</td>
<td>9</td>
</tr>
<tr>
<td>6. Pillar stability</td>
<td>12</td>
</tr>
<tr>
<td>6.1 Overview</td>
<td>12</td>
</tr>
<tr>
<td>6.2 Application to Collingwood Park</td>
<td>13</td>
</tr>
<tr>
<td>6.3 Review of strength of coal in pillars</td>
<td>16</td>
</tr>
<tr>
<td>6.4 Hollingsworth Dames and Moore 1990</td>
<td>17</td>
</tr>
<tr>
<td>6.5 Review of relevant reports</td>
<td>18</td>
</tr>
<tr>
<td>6.5.1 ACIRL 1994</td>
<td>19</td>
</tr>
<tr>
<td>6.5.2 MGS 1994</td>
<td>21</td>
</tr>
<tr>
<td>7. Risk assessment</td>
<td>23</td>
</tr>
<tr>
<td>7.1 Normal ground movements affecting houses</td>
<td>23</td>
</tr>
<tr>
<td>7.2 Methodology to distinguish damage to dwellings</td>
<td>24</td>
</tr>
<tr>
<td>7.3 Potential failure mechanisms</td>
<td>27</td>
</tr>
<tr>
<td>7.3.1 Subsidence behaviour type</td>
<td>27</td>
</tr>
<tr>
<td>7.4 Possible failure mechanisms for Collingwood Park subsidence</td>
<td>29</td>
</tr>
<tr>
<td>7.5 Review of subsidence risk in Collingwood Park</td>
<td>30</td>
</tr>
<tr>
<td>7.5.1 Results of risk assessment matrix table</td>
<td>30</td>
</tr>
<tr>
<td>8. Further investigation within the study area at Collingwood Park</td>
<td>32</td>
</tr>
<tr>
<td>8.1 Objectives of further investigation</td>
<td>32</td>
</tr>
<tr>
<td>8.2 Proposed investigation works</td>
<td>32</td>
</tr>
<tr>
<td>8.3 Application to Collingwood Park</td>
<td>33</td>
</tr>
<tr>
<td>8.3.1 Photogrammetric surveys</td>
<td>33</td>
</tr>
<tr>
<td>8.3.2 Resistivity surveys</td>
<td>33</td>
</tr>
<tr>
<td>8.3.3 Continuing ground survey monitoring</td>
<td>33</td>
</tr>
<tr>
<td>8.3.4 Drilling</td>
<td>34</td>
</tr>
<tr>
<td>8.3.5 Back analysis of subsidence trough</td>
<td>34</td>
</tr>
<tr>
<td>8.3.6 Groundwater impacts on long term stability</td>
<td>34</td>
</tr>
<tr>
<td>8.3.7 Seismic surveys</td>
<td>34</td>
</tr>
<tr>
<td>8.4 Summary of investigation methods</td>
<td>35</td>
</tr>
<tr>
<td>8.4.1 Geophysical investigation methods</td>
<td>35</td>
</tr>
<tr>
<td>8.4.2 Geological drilling and geophysical logging</td>
<td>35</td>
</tr>
<tr>
<td>9. Investigation costs and timelines</td>
<td>36</td>
</tr>
<tr>
<td>9.1 Geophysics</td>
<td>36</td>
</tr>
<tr>
<td>9.2 Drilling to verify past mining conditions</td>
<td>36</td>
</tr>
<tr>
<td>9.3 Borehole camera placed into workings</td>
<td>36</td>
</tr>
<tr>
<td>9.4 Site safety</td>
<td>36</td>
</tr>
<tr>
<td>9.5 Groundwater issues</td>
<td>37</td>
</tr>
<tr>
<td>9.6 Laboratory testing</td>
<td>37</td>
</tr>
<tr>
<td>9.7 Reporting (analysis and recommendations for remedial options)</td>
<td>37</td>
</tr>
<tr>
<td>9.8 Summary of indicative costs</td>
<td>37</td>
</tr>
</tbody>
</table>
Scoping study for Collingwood Park subsidence 2009

Contents (continued)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.8.1 Indicative Contractors Costs</td>
<td>38</td>
</tr>
<tr>
<td>9.8.2 Indicative Consulting Costs</td>
<td>38</td>
</tr>
<tr>
<td>10. Summary of work to date</td>
<td>40</td>
</tr>
<tr>
<td>11. Conclusions of the Scoping Study</td>
<td>41</td>
</tr>
<tr>
<td>12. References</td>
<td>42</td>
</tr>
</tbody>
</table>

List of tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-1</td>
<td>Geological Strata</td>
</tr>
<tr>
<td>6-1</td>
<td>Probability of Failure associated with UNSW Power Pillar Design Formulae (Galvin, 2006)</td>
</tr>
<tr>
<td>6-2</td>
<td>Comparison between HDM Safety Factors and those based on UNSW Pillar Design Methodology.</td>
</tr>
<tr>
<td>6-3</td>
<td>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.</td>
</tr>
<tr>
<td>6-4</td>
<td>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%.</td>
</tr>
<tr>
<td>7-1</td>
<td>Collingwood Park -- Risk Table Matrix – Subsidence Risk Assessment prior to Remedial Options</td>
</tr>
<tr>
<td>8-1</td>
<td>Borehole location and purpose</td>
</tr>
<tr>
<td>9-1</td>
<td>Summary or indicative costs</td>
</tr>
</tbody>
</table>

List of figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Surficial extent of mapped subsidence</td>
</tr>
<tr>
<td>2</td>
<td>Timeline of bord and pillar workings at Westfalen No 3</td>
</tr>
<tr>
<td>3</td>
<td>Collingwood Park subsidence monitoring</td>
</tr>
<tr>
<td>4</td>
<td>Collingwood Park subsidence contours and possible location of faults</td>
</tr>
</tbody>
</table>

List of appendices

Appendix A: 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  
Appendix B: Regional Geology  
List of Figures  
  Figure 1 -- Surficial extent of mapped subsidence  
  Figure 2 -- Timeline of bord and pillar workings at Westfalen No 3  
  Figure 3 -- Collingwood Park subsidence monitoring  
  Figure 4 -- Collingwood Park subsidence contours and possible location of faults
1. Terms of reference of the scoping study

In July 2008 Parsons Brinkerhoff (PB) was engaged by the Queensland (Qld) Department of Mines and Energy (DME) to undertake a scoping study for a geotechnical investigation to determine the cause of past known subsidence and the potential for future subsidence incidents over previously undermined areas in the suburb of Collingwood Park, Ipswich, Queensland.

The study area at Collingwood Park is restricted to the approximate area between Reerden Street to the south, Strachan Court to the north, Collingwood Drive to the west and Lawrie and Namatjira Drives to the east. In this report the term “Collingwood Park” means the study area and is restricted to areas previously undermined by the Westfalen No 3 or Old Redbank Collieries.

The objectives as provided by the DME are as follows:

The key aims of the scoping study are:

- Stage 1 – Undertake a review and critique existing data
- Stage 2 – Determine what additional work needs to be undertaken in Collingwood Park
- Stage 3 – Develop a proposal and motivation for future work in Collingwood Park.

**Stage 1 – Undertake a Review and critique existing data**

**Task 1**

Review of previous reports, plans, interviews, discussions, value judgements and professional opinions. Key studies/reports include:

- reports from the 1988 subsidence event in Collingwood Park
- survey data supplied by Ipswich City Council
- latest pillar stability literature/research
- all other relevant reports, plans, maps, data etc.

**Task 2**

Develop a 3D model of the subsidence process.

- review latest literature and research on pillar stability.

**Task 3**

Summarise and review the available data collated in tasks one and two above.

**Stage 2 – Determine what additional work needs to be undertaken in Collingwood Park Study Area.**

**Task 1**

Determine through a gap analysis what additional data needs to be collected.
Task 2

Task 3
Develop a subsidence risk assessment model for Collingwood Park.

- Develop risk categories such as high, medium and low for short, medium and long term.
- The risk assessment is to be linked to land-use planning and building requirements in the different risk categories.
- Describe likely effects on existing structures in the different risk categories.

Task 4
Investigate and recommend the most appropriate geophysical methods to improve the understanding of existing and potential subsidence events (including seismic techniques).

Task 5
Develop a methodology to distinguish between damage caused by mine induced subsidence versus damage caused by reactive clays, soft soil (including uncontrolled fill), soil subject to erosion, poor building practices, or other causes.

Task 6
Carry out feedback process with DME to determine and refine suitability of data and recommendations.

Stage 3 – Develop a proposal for further geotechnical work in the study area

Task 1
Compile a comprehensive report, based on the tasks described above, with timelines, milestones and indicative costs.
2. **Methodology**

The study proceeded as outlined in the original terms of reference, except where noted. No field work other than a walkover inspection of the site has been undertaken and this report is essentially a desktop study. The location of the subsidence (April 2008) incident relevant to the underlying areas of past mining activity is shown on Figures 1 and 2 in Appendix A.

2.1 **Literature review**

The study began with the collection of all published reports and maps relevant to the study areas which were provided by DME. Copies of plans of the old mine workings were obtained. Aerial photographs and a photogrammetric study map prepared by Queensland Aerial Survey Company Pty Ltd (QASCO) that showed height changes from 1968 to 2005 were examined to identify the extent of cracking and geological features which could influence the extent of mining subsidence. Standard scale base maps were prepared and cadastral, topographic, geological and mining information were obtained from various sources.

Contours of the Duncan Street site have been provided by Ipswich City Council (refer Figure 3). These contours record the continuing subsidence following the initial event after the 26 April 2008. Monitoring results are presented from the 26 April 2008 to 9 July 2008.

As part of this study PB has engaged Dr Jim Galvin of Galvin and Associates (GA) to review and comment on the outcomes of aspects of the scoping study. This report is concerned with Stage 1-Review and critique available data.

Many reports have been provided by DME to PB for review. All reports were reviewed; however, for this study the key reports reviewed in detail were:

- Moreton Geotechnical Services 2007: Five reports for Department of Public Works on potential impacts of reactive clays and past underground mining activities on vacant residential lots.

A list of reports provided by DME is attached.
2.2 **Key Issues**

The key issues identified and to be discussed in this report include:

- geology
- mining history
- mine stability
- subsidence mechanisms
- building damage assessment
- risk analysis
- suggestions for further work and indicative costs.
3. Geology

The geology of the area was defined during the original mine exploration program which permitted correlation of the seams. The geological strata can be summarised in Table 3-1.

<table>
<thead>
<tr>
<th>Group</th>
<th>Formation</th>
<th>Coal Seams</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bundamba Group</td>
<td>Ripley Road, Raceview and Aberdare Formation</td>
<td></td>
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<tr>
<td></td>
<td>Geological Unconformity (time break)</td>
<td></td>
</tr>
<tr>
<td>Ipswich Coal Measures</td>
<td>Blackstone Formation</td>
<td>Bluff Four Feet, Bergin X, Striped bacon, Rob Roy Absent, A</td>
</tr>
<tr>
<td></td>
<td>Tivoli Formation</td>
<td>Cochrane B, Cochrane C, Bottoms D, Unnamed</td>
</tr>
</tbody>
</table>

Within the mine area, erosion at the unconformity has in some areas removed the upper seams. At the mine entrance, the outcrop is almost entirely of the Bundamba Group and comprises mainly sandstone and conglomerate with minor beds of carbonaceous mudstone and grey siltstone. Refer to the attached Regional Geology Plan in Appendix B.

3.1 Structural geology

The strata of the Ipswich Coalfield have a general south to south-east dip of the order of 5° to 15°. The geological strata have been compressed to form a series of folds with fold axes sub-parallel to the regional dip. The workings underneath the Collingwood Park area are located in synclinal trough. On the western side of the basin the strata dips to the south-east at 5° to 10° and on the eastern side the strata dips at about 5° to the south-east.

The top of the coal seam at the northern end of the basin intercepts the base of the Aberdare Conglomerate. A number of north-west trending faults have defined the trend and design of the coal mining layouts. At least three “normal” faults have been recognised in the workings near Duncan Street, as shown on Figures 2 and 4.

3.2 Geology and structural geology of the Duncan Street area

Site geology has been discussed by Moreton Geotechnical Services Pty Ltd (MGS), in a report dated 4 July 1994. The site comprises on average about 1.0 m of residual clay (maximum 3.0 m thick) overlying weathered rock. The rocks, with increasing depth, have been divided into the Raceview Formation (sandstone and siltstone), the Aberdare Conglomerate (coarse grained sandstone and conglomerate) which unconformably overlie the coal bearing Blackstone Formation comprising sandstone, siltstone, shale and coal.
The coal seams of economic interest under the site, in descending order are the combined Bluff-Four Feet Seam, the Bergins Seam and the Rob Roy Seam. Two additional seams (Cochrane and the “D” Seam) in the lower Tivoli Formation are also known to occur.

The combined Bluff-Four Feet Seam is understood to be sub-parallel to the existing topography i.e. to dip to the south-east on average at about 5°.

Inspection of the Westfalen No 3 mine plan appended implies that three and possibly four, north-west to south-east trending faults exist at mine workings level.

A major fault zone which separates the Westfalen No. 3 Colliery workings from the New Redbank Colliery workings has been described in the Peter Hollingsworth and Associated Consultants 1982 report as follows:

The fault consists of a series of closely spaced fault planes which form a fault zone some 20 m wide. Comparison of the plans of the Westfalen No. 3 and New Redbank workings indicate that the fault zone dips NE at about 60 degrees. From drilling results, it is known that the north-east block is downthrown 38 m in the vicinity of the study area. The throw decreases along the fault in both directions away from the study area.

A fault which was known as the ‘Waterline’ from previous communications with the DME (letter appended to the Hollingsworth Dames and Moore (HDM) 1990 report) is understood to have displaced the seam by about 0.5 m. Moderate to strong seepage is known to have occurred down this discontinuity. This fault has however been described as “significant” in the HDM 1990 report.

Another fault is shown at working level under the intersection of Collingwood Drive and Duncan Street. This fault, which has formed the southern and western limit to the workings to the south-east of the intersection of these roads, was also passed through for a main access route, under the same intersections. It is understood from previous discussions that this fault could be described as a ‘scissors’ fault i.e. the displacement which was least under the intersection of Collingwood Drive and Duncan Street increased in a south-east direction towards Lawrie Drive.
4. Collingwood Park subsidence

The suburb of Collingwood Park is underlain by a number of abandoned collieries which were mined by the bord and pillar method. On 25 April 2008, a ground subsidence event occurred in an area near the intersection of Duncan Street, Moloney Street and McLaughlin Streets. A number of houses were badly damaged and evacuated shortly after the event.

In 1988, a similar subsidence event occurred in the nearby Lawrie Drive area.

4.1 Recent subsidence

The April 2008 event is centred on the intersection of Duncan Street, Moloney Street and McLaughlin Streets. It is reported approximately 30 to 40 houses within the immediate subsidence area were damaged to varying degrees and that several houses were damaged beyond economic repair and may be demolished. Houses with lesser damage will be repaired, provided it is economical, however, final assessment of most of these houses will depend on the findings of structural and geotechnical assessments including follow-up inspections. The surficial extent of mapped subsidence at Collingwood Park is shown on Figure 1 and the Bord and Pillar Workings at Westfalen No 3 are shown on Figure 2 in the appendix.

Levelling surveys indicate that subsidence has progressively decreased and is confined to the area as shown on Figure 3 Indicative Subsidence Contours.

4.2 Previous subsidence incidents

On 7 December 1988, the Department of Mines was notified of alleged ground subsidence in the general area bounded by Lawrie Drive, Milgate Street, Reerden Street and McBay Street. A reported up to 570 mm of subsidence was recorded by November 1989 and a number of slab-on-ground houses were damaged, with a number of houses damaged beyond repair and consequently demolished.

A geotechnical assessment was commissioned to investigate the cause of the subsidence. Investigations methods previously undertaken included:

- investigation of mine plans and mining methods
- investigation of underlying strata
- pillar stability
- acoustic monitoring
- seismic survey
- pillar stability calculations
- numerical modelling of pillar failure.
A summary of results from the investigation include:

- It was almost certain that the observed subsidence was the consequence of the yielding of a group of between 10 and 20 pillars in the panel B workings of Westfalen No 3 Mine.

- The non-symmetrical subsidence profile over panel B probably resulted from the irregular pillar geometry and the local geological features.

- The area of yielded pillars appeared to be confined on three sides by substantial areas of intact coal or relatively large coal pillars.

Diagrammatic sections showing a generalised profile relating to subsidence of a “large” underground void and trough subsidence is shown in Appendix A.
5. Mining history

In the Redbank area, coal was first encountered in a prospecting shaft in 1913 which led to the establishment of the New Redbank Colliery on the site that now contains the Redbank Plaza shopping centre. Coal intervals, 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 a 6.5 m thick section of workable coal.

In 1965, the Westfalen Company took up the 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 economic seam 7 m thick containing some 20 million tonnes was identified on the northern side of a major fault trending north-west to south-east near the southern boundary of the lease (Edgar 1976).

Maconochie (1992) states the Department of Mines determined the conditions under which the lease could be mined, which included a limit of 40% extraction under roads and built up areas, with pillar extraction subject to the permission of the Minister. Edgar (1975) states that a maximum working height of 6.1 m is to be adhered to, but it is understood no requirements to this effect were included in the lease conditions.

The Duncan Street area is underlain by economic coal from the Bluff-Four Feet seam which is at 80 m to 115 m depth and averages 10 m thickness. This seam has been extensively worked and this area forms part of mining lease (ML) 4618 (formerly ML 568 Ipswich) and by ML 4656 (formerly ML 741 Ipswich) in the south-east corner (refer to memorandum December 1990 from P.E. Balfe, Manager Coal and Oil Shale Resources).

The coal seams were worked by bord and pillar workings and multi-slice extraction methods were undertaken. A Marietta miner was used to mine the panels in the central area (refer panels G and H in Figure 6-2 in section 6.2)

Kathage (1980) describes these methods which included ramping down to take bottom coal in areas where the seam ranged in thickness from 10.4 m to 17.7 m. A wide head Jeffrey continuous miner was used to take panels south-east of Lawrie Drive and Duncan Street. After taking a 2.4 m first pass in a five heading panel, up to five passes between 1.5 m and 1.8 m high were then taken, mainly in the outer return headings and supply roadways on either side of the central belt roadway. In these areas, roadways could be 7 m wide and vary between 6 m and 11 m high.

The main features of the mine may be summarised as follows:

- multi-slice bord and pillar
- pillars typically 25 m per side, and are diamond shaped with a 60° included angle to give a minimum pillar width of about 20 m
- extraction ratio 40% by plan
- working heights typically 6.1 m, but multiple pass extraction occasionally up to 11 m beyond belt roads
- significant stone bands within section worked; typically 18% stone in multi-slice areas
- depth 115 m to 155 m.
The mine was flooded in 1974 and the upper parts of the mine workings at that time were damaged.

After the 1974 floods, the mine was pumped out and mining recommenced. It is believed that some mining also recommenced down the drift under Duncan Street at the same time development works were being undertaken along the belt and ventilation drives under Herman Avenue and Perryman Court on the western side of the Redbank fault.

We understand that the direction of workings under Lawrie Drive was from the south and stopped under Rush Court in 1978. Before retreating the pillars tops may have been split and then during retreat mining the floors were stripped to leave a total mining height of 9 to 10 m and up to 11 m high.

It is understood mining continued on the eastern side of Goodna Creek until 1987 when the mine closed. The timeline of the Bord and Pillar workings at Westfalen No 3 is indicated on Figure 2.

Further communication from Safety in Mines Testing and Research Station (SIMTARS) has been received and is included as anecdotal notes below.
Anecdotal notes \(^1\) have been obtained after discussions with persons that worked at the Wesfalen mine. Neither PB or DME have been able to properly verify these comments at the time of completing this report.

Discussions with Westfalen staff obtained the following points of note. (no anecdote is ascribed to any person as per agreement with them at the time).

- In the 1974 flood, damage was done to the entry passages but not deeper into the mine
- There were incidences of the support being washed away by the water, but as far as can be determined very little pillar damage was done
- During and after this period there were areas where roof falls occurred
- After the floods the mine was in financial difficulties and workers were let go
- The mine was reopened and new passages in older workings were established
  The best way to determine the sequence of the progress is by the number on the roof survey pegs that are entered on the plan
- There is an area to the south of the main entry that was mined shortly after the mine was opened. This might have provided access to the older workings to the north of Duncan Street
- It has been alleged that cheaper coal was obtained from further bottoming of these older workings
- On the whole efforts were directed at maintaining a safe environment for the workers by ensuring roof stability. No efforts can be discerned to maintain pillar stability over the longer term in the early post flood areas
- In areas where the mining came to dead ends (pegs 525 to 642) it is alleged that the maximum amount possible of coal was extracted by deepening the floor levels. This is believed to have occurred in the Lawrie Drive area
- This deepening of the roadways could have happened in other area. These roadway heights were not surveyed due to safety concerns
- After this initial area was worked out another area was started to the west of Kruger Parade (rifle range area).

There is talk suggesting mine plans in other areas existed but it would seem that plans with offsets and mining heights were not required at the time and were therefore not drawn up.

---

\(^1\) Oberholzer JW  Personal communications with previous staff of the Westfalen mine. May 2008
6. Pillar stability

6.1 Overview

The attached Figure 2 shows the mine plans for the bord and pillar layout for both the older New Redbank Colliery “Top” seam workings and the more recent Westfalen No. 3 Colliery Bluff-Four Foot Seam workings. The likely direction of workings after the workings were pumped out in 1974 is estimated and shown on Figure 2 and this shows that the workings ceased about below Rush Court in 1978. At this point the pillars changed from diamond to square pillar shapes.

Mining in the area of interest was by bord and pillar mining, in which coal pillars are left to support the overlying strata with the intention of restricting mining-induced surface movements to imperceptible levels.

In general, the factors that influence pillar design can include:

- geology and in situ stress magnitude
- roof and floor strength and influence of geological structures
- the shape of the pillars and the overall mine layout
- height of the workings, including mining procedures and extent of any floor stripping or pillar splitting.

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. 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.

Coal pillar load will increase with 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 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.

The University of New South Wales (UNSW) design procedure has been used as a point of reference when evaluating previous findings relating to the stability of pillars beneath Collingwood Park. Table 6-1 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 stone 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 6-1: Probability of Failure associated with UNSW Power Pillar Design Formulae (Galvin, 2006)

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<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 1 000</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>

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 the safety factor. The most notable and extensively utilised are those of Salamon and Munro (1967) and the UNSW, Salamon et al (1996). In Section 6, the three reports reviewed all make reference to the Salamon and Munro pillar design procedure. These references 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 can be applied to Collingwood Park.

6.2 Application to Collingwood Park

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

1. The pillars are diamond shaped (refer to Figures 2 and 4 in the Appendix). The UNSW design methodology caters for diamond shaped pillars but the experience base to validate this methodology is very limited; the earlier reference used the minimum width of diamond shaped pillars rather than the length of each side.

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 using 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 which may also affect pillar strength. 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 3 m up to around 11 m. However, some of this data is anecdotal and accurate available 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.

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.

7. The widths of the panels are typically slightly wider than the depth, resulting in some of the pillar load being shared to the surrounding abutments.

**Figure 6-1: Section of the workings of Westfalen No.3 Colliery beneath Portion 55, Collingwood Park (refer to HDM report)**

Other risks to pillar stability include:

1. The acute corners of the coal pillars are very unlikely to have remained intact and 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 overestimated.

2. 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.

In summary, the factors that influence the mining subsidence failure at Collingwood Park, and generally in Australia, occur when the width to height ratios are 3 or less and depth is less than 200 m. At Collingwood Park the pillars are both square and diamond shaped; the effective width of the diamond shaped pillars was 21 m, and where the pillar height exceeded 7 m the width to height ratio was less than 3. On this basis, the mine would have been at risk in these areas.

Both the Lawrie Drive and Duncan Street failures have centred over areas where retreat mining, including floor stripping and pillar splitting, may have occurred.

Assessing pillar stress:

In the absence of valid mathematical theories for the three dimensional behaviour of mine pillars, the probable load is generally obtained by calculating the total load of the superincumbent strata over the working area and placing this load onto the pillars left (Tributary Area Theory).

\[
\sigma = \frac{\rho g H}{1-ER} \text{ (MPa)}
\]

where:

- \( \sigma \) is Average Pillar Stress
- \( \rho \) is density of overburden (\( \approx 2500 \text{ kg/m}^3 \))
- \( g \) is gravity (\( \approx 9.8 \text{ m/s}^2 \))
- \( H \) is cover depth (m)
- \( ER \) is extraction ratio (%) divided by 100

Obviously Tributary Area Theory assumes that the stress increase, due to extraction, is added to the initial vertical stress (prior to mining) and evenly distributed within the remaining coal pillars and is independent of their size and location and panel width to depth.

At Collingwood Park, using a cover depth of 127 m for calculating pillar stress and considering a number of large barrier pillars still in place, it is suggested by Australian Coal Industry Research Laboratories (ACIRL) that an extraction ratio of 45% is quite sufficient (and conservative) for determining the average pillar stress.

Average pillar stress then is:

\[
\sigma = \frac{(2500 \times 9.8 \times 127)}{1-.45}
= 5.66 \text{ MPa}
\]
6.3 **Review of strength of coal in pillars**

Coal is generally not considered to be homogeneous and isotropic. Consequently it is not possible to determine the compressive strength of test specimens or to use the average value for pillars of any size and width to height ratio.

Basic coal strength laboratory data and the relationship between strength, pillar size and pillar dimensions are required to accurately evaluate pillar stability. Unfortunately very few empirical investigations have been undertaken on this subject in Australia.

In 1982, the ACIRL and the Division of Applied Geomechanics of Australia’s Commonwealth Scientific and Industrial Research Organisation (CSIRO) carried out laboratory strength and elasticity tests on coal for the Westfalen No. 3 mine. Both sets of results are proprietary information belonging to the Westfalen Collieries and were consequently unavailable.

However, it is understood that results of a limited amount of testing carried out on small core specimens of coal from the general area gave a compressive strength of 13 MPa. When due allowance was made for the size of the pillars a compressive strength of 7 MPa for pillar design was adopted.

The relationship between strength, size and height to width ratio of pillars that is widely used was determined in an extensive program of tests by Salamon and Munro (1967). Similar formulae have been derived by Bieniawski.

The pillar compressive strength (S) is given by:

\[
\text{Pillar strength (MPa)} = k \, w^{.46} / h^{.56}
\]

where:

\(k\) is the mass strength of a unit cube of coal (7.2 MPa)
\(w\) is the effective pillar width \((4A_p/C)\)
\(h\) is the mining height.

The above strength equation suggests all South African coals have a mass strength (based on a statistical analysis) of 7.2 MPa; similarly the USBM suggests a value of 6.0 MPa for chain pillars. The reviewer of this report was also co-author of an investigation commissioned by CSIRO entitled ‘A Review of Colliery Failures in the Ipswich Coalfield, Qld.’, conducted by Maconochie and Colwell (1983). This study concluded 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. This report did not take into account inaccuracy of mine plans (i.e. bord widths mined wider than plan) and pillar spall/fret common to pillars within steeply dipping seams.

Pillar shape has been analysed in a variety of ways which are all very similar over the range for which calibration data exist. At width to height \((w/h)\) ratios greater than about 8, pillars do not fail (i.e. they never reach peak load carrying capacity), but in fact ‘work harden’. The pillar width used is the average pillar width, which equals four times the area of the pillar divided by the circumference (i.e. \(4A_p/C\)). The modified Salamon and Wagner (1985) equation can be used to more correctly estimate the strength of squat pillars \((w/h \geq 5)\), rather than the original Salamon and Munro (1967) equation. This modified equation is an extension of Salamon’s original research.
6.4 Hollingsworth Dames and Moore 1990

A review has been undertaken of a Hollingsworth Dames and Moore (HDM) 1990 mining subsidence assessment report (on portion 55 of the Queensland Housing Commission site). The introduction makes reference to two Hollingsworth Consultants (HC) reports (Maconochie and Forster, 1982 and Queensland Housing Commission, 1981). 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).

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, because 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. This reduction is the result of studies by Maconochie and Colwell on pillar strength in the West Moreton coalfield contained in an unpublished report for the CSIRO.

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 states 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…”

HDM concluded from its analysis that:

- ....the factors 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.

\[1 \text{ MPa} = 100 \text{ tonnes/m}^2\]
It is considered that all these conclusions are reasonable and still relevant to the designated areas today. HDM concludes that long term surface stability will be largely determined by preservation of roof stability. There is 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, that pillar failure is also due in part or whole to the deterioration (spalling) of the pillars themselves over time.

There is compelling evidence which indicates that pillar failure has been the cause of subsidence.

6.5 Review of relevant reports

This review is a summary of the comments provided by Professor Galvin of Galvin and Associates (GA). A copy of this report is provided in the Appendix.

 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. HDM undertook further analysis based predominantly on a mining height of 6.1 m, 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. GA has 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 the safety factor in each zone; the outcome depends on which pillar is chosen within each zone. This review confirms that the percentage areal extraction in the range of 40% to 53%. In theory, the HDM value 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.

A second analysis utilised HDM’s values for mining height and percentage areal extraction. Table 6.2 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 significant difference 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 6-2). 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 (refer Figure 6.2)</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>
<td>1.03</td>
</tr>
<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>

**6.5.1 ACIRL 1994**

Australian Coal Industry Research Laboratories (ACIRL) was commissioned by Moreton Geotechnics Services Pty Ltd 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 practise 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 and going possibly to more conservative factors of safety (say 1.4 and 1.8 respectively)

This advice is still valid.

ACIRL recognised and discussed 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 is some doubt as to the accuracy of the mine plan and actual workings.

PB and GA agree with this reasoning.
The six pillars analysed by ACIRL were located in areas C1, C2, E and G. The location of the pillars designated A,B,C,D,E and F is shown attached in the Appendix as an MGS Figure.

The pillar analysis differed from that of HDM in that depth of mining was fixed at 127 m 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 fall 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 6-3 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>
<thead>
<tr>
<th>PILLAR</th>
<th>FACTOR OF SAFETY FOR MINING HEIGHT OF:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5 m</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>
</tr>
<tr>
<td>D</td>
<td>1.71</td>
</tr>
<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 6-3. 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 reflecting a weaker coal material strength.

An alternative and more robust approach is to base the analysis on an increased percentage extraction. Table 6-4 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 5 m and no roof falls subsequently occurred, they would have had a factor of safety 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 7 m.

Table 6-4: 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>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5 m</td>
</tr>
<tr>
<td></td>
<td>ACIRL</td>
</tr>
<tr>
<td>A</td>
<td>1.34</td>
</tr>
<tr>
<td>B</td>
<td>1.19</td>
</tr>
<tr>
<td>C</td>
<td>1.38</td>
</tr>
<tr>
<td>D</td>
<td>1.33</td>
</tr>
<tr>
<td>E</td>
<td>1.33</td>
</tr>
<tr>
<td>F</td>
<td>1.33</td>
</tr>
</tbody>
</table>

The ACIRL report comments generally on subsidence prediction and the positive and negative implications of flooding of the workings. These are considered reasonable assessments.

6.5.2 MGS 1994

The Moreton Geotechnical Services (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 5.1 m and 8.4 m 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 7.6 m and 5.6 m high whereas nearby B1 and B2 were measured equally at 8.4 m 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 400 m 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 provides predictions of subsidence components. Additional site specific information is available if comment is required on the reliability of these predictions. Suffice it 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. Risk assessment

7.1 Normal ground movements affecting houses

The movement of footings and slabs of residential building and subsequent damage from that movement is often similar regardless of cause. The damage is often represented as cracking in concrete floor slabs, cracking in brickwork and cracking in plasterboard and cornices. Movement of slabs and footing can also be seen when concrete slabs tilt, windows and doors become misaligned to their framework, and plasterboard walls and ceilings exhibit out of plane buckling.

The major causes of movement of footings and slabs of residential dwellings are as follows:

- shrinking and swelling of reactive soils (sagging or hogging)
- low bearing capacity of soil (settlement)
- tilting of the ground surface and collapse of soils (subsidence)
- footing or slope instability.

In shrinking and swelling of reactive soils, concrete slabs will exhibit movement due to the seasonal variation of moisture, especially in areas that have high seasonal rainfall variations. Concrete slabs will also be subject to movement when foundation soils become wetter or drier than the original condition of the soil when the slab was placed. If this variation is homogeneous throughout the site, the subsequent pattern of movement will be greater at the perimeter of the slab and less in the centre of the slab.

This pattern of movement is called differential movement and often described in terms of sagging or hogging. It should be noted that patterns of movement are complex and depend upon a number of factors, including the shape of the slab, whether it is elongated or has re-entrant corners.

Watering gardens close to foundations, and/or low lying areas of poor drainage can cause localised swelling of clays and subsequent localised movement of the slab. Similarly large trees or clumps of trees may draw significant moisture from the soil and create movement from drying of the soils. Other external causes to movement of reactive soils include broken sewerage and stormwater pipes or removal of vegetation. Also, soil reactivity may not be uniform throughout the site and as such can cause differential movement of the slab. Levelled building platforms obtained by cutting and filling are likely to exhibit different reactivity rates in the cut and fill areas and can often be described as ‘tilting’.

The resultant movement due to reactive soils is generally vertical; however, horizontal movement can also be seen especially where there are deep slab edge beams or strip footings. The effect of both vertical and horizontal movement can cause buckling in walls and curvature in slabs. Observations on the surface are mostly likely to reveal irregular shrinkage cracks.

Low bearing capacity of soils can cause settlement of isolated footings. This is often seen in houses with porches and verandas which are supported on columns which are in turn supported on individual footings. The load of the columns can be greater than the bearing capacity of the soil for that size of footing, especially on soft or alluvial soils. Differential movement can also be seen in houses that are high set and supported on numerous
columns and isolated footings. Uncompacted fill can also cause localised settlement and can result in columns twisting or being out of alignment. It can also cause ‘tilting’ of slabs.

7.2 Methodology to distinguish damage to dwellings

The following is a methodology developed to distinguish damage from subsidence compared to reactive soils.

The word subsidence is often used to describe any movement in soil. With respect to damage in buildings, subsidence is generally used to specifically describe the movement due to the collapse of soils due to mine shafts of underground workings. Movement due to mine subsidence is often severe and includes lateral strain, settlement, tilting of the slope and curvature.

Unless the movement of the footings and slabs and damage to the residence is obvious, an investigation may be required since a number of factors can cause similar movement and damage to dwellings.

In ascertaining the underlying cause of movement of slabs and footings of a residential dwelling some or all of these items will be required:

1. pattern of slab and footing movement over time
   - extent of damage established with respect to ‘normal conditions’.

2. geotechnical parameters
   - soil reactivity
   - soil bearing capacity
   - soil moisture content.

3. adequacy of footing and slab design for ‘normal’ conditions
   - investigation of original engineering drawings and geotechnical investigation.

4. identification of site features
   - location of trees and gardens
   - historical knowledge, including location of mine tunnels and shafts
   - historical knowledge of the methods of underground working including depth and height of working, pillar sizes, dip of coal seam, presence of faults
   - adequacy of existing site drainage
   - adequacy of existing site pipe work.

Pattern of slab and footing movement over time

In investigating the pattern of slab and footing movement over time, a baseline value for the as-constructed slab and walls can be established based upon a ‘zero’ value and known construction tolerances. A survey of the concrete slab and/or brick line can establish a pattern of movement since construction. A number of periodic surveys completed after a change is season, i.e. wet/dry, summer/winter, can establish the difference between seasonal variations of soil movement.
The pattern of damage including indicative crack widths and the extent of damage should be recorded. AS2870 Residential Slabs and Footings classifies performance criteria for walls and floors with respect to movement and damage for ‘normal’ conditions.

**Geotechnical investigation**

A geotechnical investigation of the site in accordance with AS2870 Residential Slabs and Footings should be conducted to establish a site classification and the existing conditions. The soil profile would be obtained by a drilling auger and shrink-swell tests would be conducted.

The allowable bearing capacity of the soil and soil moisture content would also be obtained. Typically, two investigation bores would be required, one at the front and one at the rear of the dwelling, to ensure that a representational site profile could be obtained to a minimum of 2.3 m. Any areas in which the site has been filled should be defined.

**Adequacy of footing/slab design and construction for ‘normal’ conditions**

Original engineering drawings and soil tests should be reviewed to ensure that the footing and slab design conform to AS2870 and that the site classification was correct.

**Identification of site features**

A visual inspection of the site and the identification of sources of damage are required. Investigation of existing pipework by a licensed plumber may be required if it is suspected that leaking pipes may be the cause of moisture ingress. Investigation of historical works, including assessment of past mining activity and location of mine shafts and adits, should also be undertaken.

Based on this observation data a proposed flow diagram showing a methodology has been provide in Figure 7-1 below.
Figure 7-1: Proposed methodology to assess reason for ground movement.

**Stage 1**
**Desktop Study**
- Check if there is known past history of mining subsidence.
- Request soil test data, engineering footing and slab plans from house construction – previous building inspection reports (have purchased).
- Check current and historical topographic plans and mine plans.

**Stage 2**
**Site Visit**
Observe and plot on house plan evidence of:
- distribution of movements of house and site
- evidence of hogging of slab, vertical or horizontal movements
- cracking of natural ground, concrete pavements
- interview relevant persons who could provide data on observations in the past and recently
- relevant external data including trees, gardens, drainage etc.

**Stage 3**
**Analysis**

Reactive Site if:
- no underground mining in area
- soil shrinkage crack (pattern)
- hogging of slab
- similar distribution of cracking through house
- soil test indicate reactive Classification as per AS2870.
- Plumbing report

Underground mine subsidence if:
- previously undermined
- no evidence of reactive clays
- distinct ground cracking with no Gilgai structures
- severe distortion, tilting
- known pillar instability in area

**If inconclusive**
Further site investigation required
- Conduct soil test to assess site conditions.
- Conduct level survey of house and slab.
- Check Pillar stability.

**Reactive Site**
**Subsidence**
**Recommendations**
7.3 Potential failure mechanisms

Figures 3 and 4 in the Appendix show the 2008 subsidence area, plus the layout of the bord and pillar workings and proposed interpretation of the location of faults at the level of the workings.

The subsidence incidents as shown on Figure 4 appear to be bounded by faults on both sides and square pillars of larger cross sectional area on one other side south of Fowler Street. Depending on the dip of the seam, the surface topography and the nature of the faulting, 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 area of pillar collapse which leads to subsidence. 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 an area of underground workings.

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 within the study area 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.

At Collingwood Park, pillar collapse confined by inclined faults could also produce unacceptable differential settlements near the surface trace of the faults.

The surface traces of faults can define areas which may impact differently for housing development.

7.3.1 Subsidence behaviour type

A further discussion of subsidence behaviour is provided below:

Mining induced subsidence can be described as the ground movement above mine workings due to particle or total displacement of the overburden into the mine workings. Subsidence of the surface results in the land sinking, rotating or tilting, stretching to make up the extra area in the strain zone and crumpling or compacting in the compression zone.

The tilts, strains and compressions can be uniform where pliable rock such as shale exists between the surface and the workings, or irregular where brittle rocks such as sandstone exist.
Significant angular distortions also occur where major discontinuities such as faults exist in the rock mass between the surface and the workings. Subsidence in its mildest form resulting in settlement, tilts, strains and compressions, can damage structures. Sinkholes (potholes) are the most severe form of subsidence. Sinkholes, which can take years or decades to migrate to the surface, usually open up quickly once the surface is reached. Sinkholes can be life-threatening, as well as result in damage to structures.

Subsidence in coal mining can essentially be divided into two main categories, which can be described as controlled and uncontrolled.

**Controlled subsidence** is planned or predicated in terms of amount, extent and time whereas uncontrolled subsidence is not. Controlled subsidence occurs while bord and pillar mining is in progress and is due to the elastic compression of the pillars experiencing extra imposed weight. The resulting subsidence is generally low in magnitude and evenly distributed above the working area.

Controlled subsidence also occurs when a mining company uses shortwall or longwall (not used within the study area) mining techniques or opts to extract the pillars that are left supporting the roof on completion of the main mining operation or when the mine reaches the end of its lease area etc. Subsidence at the surface due to shortwall mining or pillar extraction is usually predictable in terms of the amount of settlement, tilt, strain and compression, as well as the time taken to subside.

The exception is where pillar extraction has occurred at very shallow depths (less than 20 m) and where the water table fluctuates in the remaining mine voids. In this case, the land surface can move up and down due to the buoyancy effects of major drought and flood cycles. Detailed studies on surface impacts of controlled subsidence have been carried out in New South Wales and in other parts of the world but not in the Ipswich area. Controlled subsidence, which has mainly occurred over pillar-extracted areas in the Ipswich region, is not considered in this report.

**Uncontrolled subsidence** generally results over time from the deterioration of mine working (roof, pillar or floor) where pillars are left in place. Mining necessitates that the workings are only kept safe for the relatively short duration while extraction is occurring.

Uncontrolled subsidence can also occur during the operation of the mine where pillars unexpectedly crush or pillars punch through the floor or roof. Sinkholes are the result of the collapse of roof rock over relatively shallow underground roadways or above the intersection of underground roadways. Sinkholes in the Ipswich area generally occur to a maximum cover depth to worked height ratio of about 10. Sudden settlement of backfill in vertical shafts or steep tunnel entries can also form holes which can be called sinkholes.

As part of this discussion, the mining-associated mechanisms that may affect a number of the coal mining operations at Collingwood Park might include:

- **Floor heave** – migration of floor rocks into the mine openings as a result of punching of the pillar into floor rock; creep/heave of the floor rock into the opening; or a combination of both. Heave can completely fill fine voids and provide support for pillars.

- **Crushing of pillars** occurs when the overburden stress carried by the pillar exceeds the pillar strength. Stress concentrates at pillar edges, causing them to fail and spall, which in turn leads to further stress concentration and deterioration of the pillar. Long-term stability of the pillar is achieved when the pillar is large enough to ensure a restrained
central core where behaviour of the coal is typified by the angle of internal friction of the coal mass.

- Roof collapse – disintegration and deformation of the roof rocks over mine voids, resulting in the collapse of the crown between pillars. The roof often continues to fail until bulking of the collapsed material chokes the void; until strata of sufficient strength to span the opening is reached; or until the surface is reached.

- Subsidence due to consolidation, associated with convergence of rock mass into broad areas of goaf (such as formed during long-wall operations) or as a secondary mechanism following collapse of pillars.

- Collapse of shafts – generally a local effect around shafts that have not been backfilled, or poorly backfilled shafts; or poorly capped shafts.

The expression of each of these mechanisms at surface is expected to take one of the following forms:

- Sinkholes – subsidence is characterised by an abrupt sinking of the surface, resulting in a circular steep-sided, craterlike feature that has an inward drainage pattern. Sinkholes are generally associated with roof collapse in shallow mines. Shaft collapse would be expected to form a similar feature. This has not been recognised on site to date.

- Sag or trough subsidence – characterised by a gentle, gradual settling of the surface. It is associated with pillar crushing, pillar punching (floor heave) or widespread consolidation. Only sag or through subsidence has currently been recognised at Lawrie Drive and Duncan Street.

- Stepped subsidence – characterised by abrupt change in surface elevation. It is associated with pillar collapse but is usually restricted to either relatively shallow depths or associated with elevation changes near to or along geological structures.

### 7.4 Possible failure mechanisms for Collingwood Park subsidence

On 25 April 2008 the subsidence at Duncan Street occurred and based on the discussion above, it is suggested that the contributing factors to the failures at Lawrie Drive and Duncan Street in Collingwood Park are as follows:

- Both failures have occurred above mined areas where pillars have width to height ratios of about 3 or less. That is, workings may be 6 m to 8 m high and effective pillar widths may be from 18 m to 22 m. Pillars with a width to height ratio below 3 are often considered to give rise to the possibility of uncontrolled or sudden pillar collapse.

- Anecdotal evidence suggests that, particularly in Lawrie Drive mining may have involved pillar splitting and extraction in a retreat mode. Further mining after 1974 is likely to have occurred under Duncan Street.

- Both subsidence events are centred between two faults. A review of the patterns and voids sizes produced by the bord and pillar workings suggests that past mining development drives ceased at the fault or possibly progressed parallel to the faults in some areas. This results in the roof strata cantilevering along a failure plane. This increases the stress on the pillars and results in pillar failure.
Other factors contributing to pillar stability, as noted in Section 6.6 include, past mining activities (that is, floor stripping) creating diamond shaped pillars and changes in effective stress should variations in the groundwater regime occur.

7.5 **Review of subsidence risk in Collingwood Park**

Reviewed reports have indicated considerable risk of mining induced subsidence occurring in some areas overlain by the Westfalen No 3 workings, in the medium to long term.

It is likely that the subsidence in the Lawrie Drive area was caused by pillar collapse due to a combination of pillar dimensions, roof falls which affected pillar stability possibly triggered by groundwater inflow or changes to the groundwater regime, and effects of major geological structures (faults).

The proximity to the Lawrie Drive area and the similarity of events suggest that the recent subsidence at Duncan Street was caused by mechanisms similar to those proposed for the Lawrie Drive area.

7.5.1 **Results of risk assessment matrix table**

PB has created a risk assessment matrix table (refer Table 7.5) for the study area at Collingwood Park. This table forms the basis for a risk assessment which relates primarily to a brick veneer type of housing construction. The risk assessment is described in terms of Probability and Consequence of a particular subsidence/settlement behaviour type occurring. Remedial measures can change the probability of a behavioural type occurring from ‘almost certain’ to ‘unlikely’.

The Risk Table Matrix implies a high risk of structural damage to buildings from localised step-and-trough subsidence resulting from an almost inevitable future event(s) if no remedial works are undertaken.
Table 7-1: Collingwood Park -- Risk Table Matrix – Subsidence Risk Assessment prior to Remedial Options

<table>
<thead>
<tr>
<th>Settlement Behaviour type</th>
<th>Probability</th>
<th>Consequence</th>
<th>Risk</th>
<th>Effect on land use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
<td>P</td>
<td>C</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Minor settlement, slow comparable to expansive soil (shrink/swell), time dependent.</td>
<td>A</td>
<td>5</td>
<td>Medium</td>
<td>Superficial damage, repairable by filling cracks and painting.</td>
</tr>
<tr>
<td>Trough subsidence, bowl shaped depression, economically repairable, localised differential settlement.</td>
<td>B</td>
<td>4</td>
<td>Medium</td>
<td>Superficial damage, repairable by filling cracks and painting.</td>
</tr>
<tr>
<td>Step subsidence, open cracks with height difference across surface.</td>
<td>C</td>
<td>1</td>
<td>High</td>
<td>Building is uninhabitable and would require purchase, demolition and revision to parkland.</td>
</tr>
<tr>
<td>Trough subsidence, bowl shaped depression, non-repairable because of excessive tilt (1 in 80).</td>
<td>B</td>
<td>3</td>
<td>Medium</td>
<td>Partial structural damage, repairable by localised grouting or underpinning.</td>
</tr>
<tr>
<td>Sink hole with vertically sided shaft from shallow depth workings.</td>
<td>D</td>
<td>1</td>
<td>Medium</td>
<td>Building is uninhabitable and would require purchase, demolition and revision to parkland.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Probability</th>
<th>Consequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>ALMOST CERTAIN to happen</td>
</tr>
<tr>
<td>B</td>
<td>LIKELY to happen at some point</td>
</tr>
<tr>
<td>C</td>
<td>MODERATELY possible, it might happen</td>
</tr>
<tr>
<td>D</td>
<td>UNLIKELY to happen</td>
</tr>
<tr>
<td>E</td>
<td>RARE, practically impossible</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Consequence</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
</tr>
<tr>
<td>1</td>
<td>High Risk</td>
<td>High Risk</td>
<td>High Risk</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
</tr>
<tr>
<td>2</td>
<td>High Risk</td>
<td>High Risk</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
</tr>
<tr>
<td>3</td>
<td>High Risk</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
<td>Low Risk</td>
</tr>
<tr>
<td>4</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
<td>Low Risk</td>
<td>Low Risk</td>
</tr>
<tr>
<td>5</td>
<td>Medium Risk</td>
<td>Medium Risk</td>
<td>Low Risk</td>
<td>Low Risk</td>
<td>Low Risk</td>
</tr>
</tbody>
</table>
8. Further investigation within the study area at Collingwood Park

8.1 Objectives of further investigation

The principle objectives of undertaking further investigations in the context of the conclusions from the study are as follows:

- Investigate the condition of the pillars (workings) in both the known failed areas (Duncan Street and Laurie Drive incident areas) and the balance of the underground Collingwood Park area.
- Identify the groundwater regime to better understand the role that water may have on the stability of the workings.
- Assess the extent of the failed areas and the influence of faulting on the subsidence behaviour.
- Assess the results of the investigations proposed above and develop stability models to further assess the options for mitigating the risks of more mine failures and resulting subsidence.
- Prepare cost estimates for the mitigation options (if any can be recommended) and compare with cost estimates from other subsidence management alternatives.

8.2 Proposed investigation works

To assist with the assessment of site remedial options the following methods should be considered:

- geophysical and survey methods
- site drilling.

These methods comprise two types.

- methods to assess the relatively broad scale sections of the site along line traverses, that is, geophysical and survey methods.
- methods to sample and assess the strata at selected locations, that is, borehole drilling and testing.

Each of the methods may provide information to help develop a ground model so that a better understanding of the subsidence processes occurring on site can be obtained.

Geophysical and survey methods

- Photogrammetric survey and surface levelling surveys should be undertaken to assess the surface extent and possible amount of displacement that has occurred, and ongoing extent of surface movements.
- Resistivity surveys have previously been undertaken to define the surface location of faults that may delimit the extent of subsidence induced by pillar failure.
Seismic surveys were used at Lawrie Drive in 1989 to define the extent of subsidence within the coal workings and/or the extent of trough subsidence via mapping displacement in the rock strata. These surveys can also predict the extent of faulting in the strata. Techniques have been refined since 1989 and a new survey should be undertaken.

Site Drilling

Based on the geophysical results discussed above plus detailed site surveys to locate the planned extent of the old coal workings a borehole can be placed at the optimal locations. The object of this drilling is to assess the nature of the workings including the height of the mined section and the presence of gas and/or water.

As part of the drilling process additional valuable information can be obtained from placing a visible light camera or sonic camera (where visibility is a problem) into the workings. This can identify the existing condition of the workings including whether the pillars or roof has collapsed.

Downhole geophysical logging, including the density, gamma and calliper sonde and a sonic sonde can be used to assess the nature and present condition of the roof strata above the workings. This may detect the extent of the underground failures— that is, the extent of any bedding separation which may be related to roof collapse above intersections.

8.3 Application to Collingwood Park

8.3.1 Photogrammetric surveys

Photogrammetric surveys were undertaken in 2005 by QASCO. Survey updates to 2008 are suggested to assess relative movements and to separate level differences arising from the original Stage 2 subdivision earthworks since about 1985.

8.3.2 Resistivity surveys

To supplement the available information, geophysical studies using resistivity techniques were undertaken in selected areas. Resistivity methods cannot detect coal seams at depth but are capable of detecting subsurface discontinuities and, in certain circumstances, may distinguish large underground voids. The prime purpose of this field investigation was to define potential barriers to mine development. In addition, trials were carried out to assess the usefulness of resistivity methods in locating the limits of abandoned workings.

The results of these studies showed that resistivity methods were effective in locating faults, provided the fault extended to the surface or was covered by only a thin layer of younger sediments. However, the resolution of the method was insufficient to accurately define the extent of underground workings.

8.3.3 Continuing ground survey monitoring

Ipswich City Council (ICC) is understood to be undertaking continuing ground survey monitoring which is a very useful tool to predict rates of subsidence. Further detailed subsidence monitoring via survey is recommended for Collingwood Park areas at risk of subsidence.
8.3.4 Drilling

Drilling to penetrate the workings is suggested to define geotechnical parameters for analysis of factors of safety, to access whether the workings are collapsed and to assess groundwater conditions. It is anticipated that the bottom section of the test bore would be cored and piezometers installed after downhole geophysical logging which may include caliper, short and long spaced density and possibly gamma plus neutron logs. This data would be matched to the test borehole log to provide a detailed log of the strata, and provide an indication of whether pillar failure has occurred in particular areas.

RAAX imaging could also be considered, to provide an image of the borehole and access defects. One of the aims of this work would be to assess the height and extent of fracturing above the collapsed workings.

During the drilling operations and as part of the assessment fractures in the rock core will be noted and compared. This is to provide a guide as to which areas have “goaf cracking” and which areas appear not to have subsided. The use of downhole cameras will be considered where drilling holes penetrate into the bords.

Groundwater levels and piezometric levels in the bores will be measured using standpipes and piezometers. The presence of methane should be assessed.

8.3.5 Back analysis of subsidence trough

Further review of the settlement data and the results of the drilling investigation are recommended, followed by back analysis of the mine plan to identify the extent of collapse underground. The model of the mine thus developed can be used to further review the stability of the remaining panels and to develop possible mine stabilisation options.

8.3.6 Groundwater impacts on long term stability

The effects of either groundwater or surface water can have both positive and negative effect on the long term stability of coal pillars. The effects on coal pillar strength can be neutral if porewater pressures are balanced but coal pillar strength can be reduced if the water level in the working is drawn down or is imbalanced. The establishment of a steady state groundwater regime is preferred for long term stability.

8.3.7 Seismic surveys

The following types of seismic survey could be used:

- Standard seismic refraction surveys
- MiniSOSIE surveys
- Multichannel analysis of surface waves (MASW) techniques.

Each of these approaches have limitations. The MASW technique has been used for shallow coal seams near the Ipswich Motorway with some success, but at depths of greater than 100 m its capabilities will be stretched and it is unlikely to see the coal seam.

Standard seismic refraction surveys may pick up the faults: however, they may have limited depth penetration to the coal and may not be able to differentiate the coal seams because of the possible presence of high velocity refractors at shallow depths.
The MiniSOSIE or similar seismic reflection technique involves an enhanced reflection seismic survey using a controlled impact device that vibrates 6-7 times per second. The technique is suitable for the depth range 100–150 m at Collingwood Park. A trial would be necessary to confirm its usefulness.

8.4 Summary of investigation methods

8.4.1 Geophysical investigation methods

The geophysical survey methods that could be considered for Collingwood Park are:

- Downhole logging and piezometer installation to assess the nature of fracturing in the boreholes, and to monitor groundwater levels and/or the presence of hydrocarbon gas.
- MiniSOSIE or seismic reflection to assess, subject to a trial, the extent of the coal workings, the possible extent of roof collapse and the extent of faulting.
- Resistivity surveys to better trace the surficial extent of faulting.

8.4.2 Geological drilling and geophysical logging

It is suggested that a series of geotechnical boreholes be drilled into the workings and coal pillars to verify the working locations, and that the pillars be examined with optical and/or a sonic cameras, as appropriate. As technically appropriate, downhole geophysical logging should be undertaken in the test boreholes.

As noted in Table 8.1 between 12 and 20 boreholes may be needed, depending on the progressive outcomes of the investigation.

Table 8-1: Borehole location and purpose

<table>
<thead>
<tr>
<th>Suggested borehole numbers</th>
<th>Location</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 to 3</td>
<td>Redbank Colliery</td>
<td>To identify if the existing workings have collapsed. If they have there is a reasonable chance that this area has stabilised.</td>
</tr>
<tr>
<td>4 to 6</td>
<td>Vicinity of the Strachan Court cul-de-sac and Herman Avenue and Perryman Court area</td>
<td>To assess condition of workings to assess risk of extension of existing Duncan Street collapse. Install acoustic monitoring devices and check nature of belt drive access.</td>
</tr>
<tr>
<td>2 to 4</td>
<td>Within the Duncan Street failure area</td>
<td>To assess the nature of the present collapse. Install access fire monitoring devices.</td>
</tr>
<tr>
<td>2 to 3</td>
<td>Near the Lawrie Drive failure area</td>
<td>To assess if void filling of the workings is complete.</td>
</tr>
<tr>
<td>2 to 4</td>
<td>Between failures of Duncan Street and Lawrie Drive</td>
<td>To assess the condition of pillars in this area</td>
</tr>
</tbody>
</table>
9. Investigation costs and timelines

An indication of possible future costs for the following components of work has been attempted. PB has obtained quotes from selected contractors. It is suggested that ideally that the geophysics should be undertaken first followed by drilling and laboratory testing.

9.1 Geophysics

A proposal for surface seismic and electromagnetic (EM) induction surveys has been obtained from Ecophyte Pty Ltd. At this stage surface resistivity to check for the surficial locations of the faults has been deleted because of access problems.

Downhole geophysics has been costed by Weatherford Downhole Logging.

Reflection Seismic survey costs have been received from Marine and Earth Sciences.

Estimated time to undertake these surface geophysical works is about 6 weeks.

9.2 Drilling to verify past mining conditions

The drilling of up to 12 to 20 bores is proposed, including percussion drilling to the top of and into the workings. Some core drilling would also be planned, and the sides of the bores would be inspected using a suitable camera. A suitable drilling rig is expected to take about 2 months to obtain once a decision has been made to undertake this work.

Installing the full complement of boreholes suggested could take from 60 to 70 working days and requires geophysical supervision. There are considerable risks associated with drilling in disturbed ground as could exist above the workings and a contingency amount of at least 20% is suggested.

Estimated time to undertake surface drilling with no hold ups for weather or site clearances is 60 to 70 working days plus a further 10 days to complete downhole geophysical studies and install monitoring piezometers.

9.3 Borehole camera placed into workings

A borehole camera would also be used to inspect the workings and the amount of filling within the voids. Simtars would be required to verify that the camera is intrinsically safe.

A quote based on daily rates has been received from Geotechnical Systems Australia Pty Ltd (GSA). Use of the downhole camera is subject to a risk assessment and standard operating procedure in accordance with the GSA operating manual.

9.4 Site safety

The safety of all subsurface operations must be ensured at locations when dealing with hydrocarbon gas. Site operations will include geotechnical drilling (where Washington diverters will be employed) and downhole logging sondes and cameras, which will have to comply with DME safety guidelines. All work on site by contractors and consultants must be subject to risk assessments prior to commencement.
9.5 **Groundwater issues**

It is suspected that groundwater pulses relating to possible intensive rainfall events may be a trigger for subsidence events at Collingwood Park. Collection of groundwater data during drilling is essential to determine this and to assess remedial options. Monitoring bores should be constructed as part of the drilling program.

Local data on rainfall events would also be required in order to review whether subsidence movements could be related to rainfall events.

9.6 **Laboratory testing**

Laboratory tests would be undertaken at a NATA registered laboratory. They would assess the strength and modulus of the strata to complete a geotechnical model of the strata at Collingwood Park. This work should be discussed with Simtars when the results of their 3-D modelling are available.

It is estimated that laboratory testing would be completed in 25 working days.

9.7 **Reporting (analysis and recommendations for remedial options)**

The outcomes for the site investigations should consider and discuss future options, and include comments on the investigation objectives as set out in Section 8.1.

Depending on the scope and availability of subcontractors and used of peer reviewers indicative costs of this work to provide positive recommendations is discussed in the following sections.

9.8 **Summary of indicative costs**

A summary of the items discussed above, together with indicative costs, is provided in Table 9.1.
Table 9-1: Summary or indicative costs

<table>
<thead>
<tr>
<th>Work Phase</th>
<th>Surface Geophysical (Possible Contractor)</th>
<th>Work Description and aim</th>
<th>Estimated cost based on rates provided*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>Trial Seismic Reflection Study (Marine and Earth Sciences)</td>
<td>Aim is the location of faults at depth and delimit possible areas of subsidence disturbance</td>
<td>Trial allow $44,000</td>
</tr>
<tr>
<td>1B</td>
<td>Trial resistivity/electromagnetic study (Ecophyte Pty Ltd)</td>
<td>Proposed trial to assess use of Resistivity and/or EM4 for surface location of faults</td>
<td>Trial only allow $16,000</td>
</tr>
</tbody>
</table>

| Cost estimate surface geophysics | Subtotal | $60,000 |

<table>
<thead>
<tr>
<th>Drilling and Downhole geophysics(Possible contractors)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Drilling of up to 20 bores by drilling contractor at locations selected and based on the results of the surface geophysical studies.</td>
</tr>
</tbody>
</table>

| 3A | Provision of Downhole Camera Geotechnical Systems Australia Pty Ltd | To view borehole sides and condition of workings | Rates only provided. Indicative cost allow $60,000 |

| 3B | Downhole geophysics Waterford Downhole Logging | Assume 7 days work on site up to 20 holes logged with calliper, gamma and 10 only holes with acoustic televiewer for structure. (Bores may need to be topped up with water). | Based on rates per day and per hole. Indicative cost allow $40,500 |

| Cost Estimate drilling and logging | Subtotal | $345,500 |

| Total Estimated cost | $405,500 |

*All Rates as at December 2008 and all estimated costs do not include GST.

9.8.1 Indicative Contractors Costs

An indicative budget estimated costs for the above work for up to 20 boreholes from contractors is $405,500 plus GST subject to any rates increase in 2009.

9.8.2 Indicative Consulting Costs

The contractor's costs do not include any project management or geotechnical supervision, laboratory testing, analysis and reporting. As a guide professional costs would be based on hourly rates and for the possible scope of the work noted above could amount to an indicative cost of $250,000 to $260,000 (+ or – 20 %)

Further detailed discussions to assess feedback are desired with DME to review the scope of the work and outcomes.
These costs do not include:

- a contingency sum for possible delays due to weather conditions and any downtime or costs associated with traffic control plus DBYD costs. DME may wish to consider employing the contractors directly.
- survey of boreholes. It is envisaged that all testing and borehole locations will have to be surveyed by a qualified cadastral and mine surveyor before drilling and surface geophysics is undertaken.
- costs associated with public liaison/consultation.

In summary, for up to 20 bores to depth to about 140 m plus geophysics, the indicative total cost is about $670,000 plus GST.

Should the program be achieved in about 12 bores instead of the 20 bores, these indicative costs may reduce to a lower amount of about $525,000 plus GST.
10. Summary of work to date

Based on the presently available data the subsidence events at Collingwood Park area can be attributed to pillar failure.

The areas where subsidence has occurred are located at:

- Lawrie Drive area, which subsided in 1988; reported total subsidence to date is 2.2 m.
- Duncan Street area which subsided in April 2008: recorded subsidence to August 2008 is understood to total about 1.3 m.

Underground mining of the Westfalen No 3 mine which commenced in 1965 was interrupted by the 1974 floods. The mine was pumped out and mining recommenced along the line as shown on Figure 2 in the Appendix. Mining stopped in 1987; however, an area along Rush Court is suggested to be the extent of working in 1978. These 1978 diamond shaped workings stopped to the south of square shaped workings centred below Milgate and McInnerney Streets. Subsidence below Lawrie Drive is centred on these diamond shaped workings. A personnel communication from Taylor Mining Services Pty Ltd suggests that the floors in this area were stripped to provide a bord height of at least 9 to 10 m or greater.

It is also reported by DME that, whilst retreat mining occurred that the pillars were also split. Pillar stability calculations imply that diamond shaped pillars with these bord heights have a factor of safety of about 1.0 and a probability of failure of 5 in 10, which is unacceptable in both the short and long term. These calculations do not take into account the effects of pillar splitting, which would have further reduced long term stability.

Maconochie 1992 suggests that below Lawrie Drive pillar failure occurred in one pillar and progressed to failure of about 20 pillars.

Both subsidence areas lie between two faults, which have been mapped at the level of the workings. The surface trace of the Waterline fault, interpreted from surface resistivity survey data is shown on Figure 2. It is evident that the most continuous surface cracking is sub-parallel to the trend of these faults.

Within the Duncan Street area, the pit entry was sub-parallel to Duncan Street and was flooded in 1974. It is understood that this flooding destroyed the integrity of some pillars and removed the roof supports. Also it is reported by DME, that when the mine was being pumped out after the 1974 floods, further mining may have occurred under the Duncan Street region.

It is considered for both areas that factors contributing to subsidence failures were retreat or further mined (pillar splitting) and /or the possible effects of flooding. It is not possible to confidently predict the amount and timing of future subsidence. Further subsidence is possible as its theoretical vertical extent has not been reached. It is considered that the recognised geological structure (faulting) will influence the lateral extent of the present subsidence.

Further work, as detailed in Section 9, is necessary to detail risks and future options for Collingwood Park.
11. Conclusions of the Scoping Study

The Duncan Street (April 2008) incident has many similarities with the previous 1988 Laurie Drive incident in that:

- the major subsidence was manifest without warning over a period of a few days.
- subsequent recorded settlement rates gradually reduced and apparent stability appears to be occurring.
- significant damage occurred to a few buildings and utilities as a result of step subsidence, tilting and cracking.
- localised damage appears to be accumulated in the vicinity of faults and within the limits of the underground workings.
- pillar failure in the bord and pillar workings has been identified as the principal cause of the subsidence.

Further review of pillar stability of the mine indicates that further similar incidents are likely to occur unless practical mitigation options can be identified and undertaken.

These works may include mine filling with a suitable flowable fill at selected locations, however, the extent of present failures and condition of the workings must be understood before any recommendations can be made to reduce the risk of future mine subsidence. These stabilisation techniques are being considered in relation to construction of the nearby Ipswich Motorway Upgrade construction.

Recommendations for further investigations are provided in accordance with the requirements of the brief. This will enable further assessment of the feasibility and potential risks of the mitigation options.
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