XSTRATA COAL:

Tahmoor Colliery - Longwall 27

Management Plan for Potential Impacts to Wollondilly Shire Council Infrastructure

AUTHORISATION OF MANAGEMENT PLAN

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REFERENCES

AS/NZS 4360:1999 Risk Management

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Drawings

Drawings referred to in this report are included in Appendix B at the end of this report.

1.0 INTRODUCTION

1.1. Background

Tahmoor Colliery is located approximately 80 kilometres south west of Sydney in the township of Tahmoor NSW. It is managed and operated by Xstrata Coal. Tahmoor Colliery has previously mined 25 longwalls to the north and west of the mine's current location. It is currently mining Longwall 26.

Longwall 27 is a continuation of a series of longwalls that extend into the Tahmoor North Lease area, which began with Longwall 22. The longwall panels are located between the Bargo River in the south-east, the township of Thirlmere in the west and Picton in the north. A portion of each longwall is located beneath the urban area of Tahmoor. Infrastructure owned by Wollondilly Shire Council is located within these areas.

Longwall 27 is approximately 283 metres wide (rib-to-rib) and approximately 3.0 kilometres long. The width of the chain pillar between Longwalls 26 and 27 is 40 metres.

This Management Plan provides detailed information about how the risks associated with mining beneath Council infrastructure will be managed by Tahmoor Colliery and Wollondilly Shire Council.

The Management Plan is a live document that can be amended at any stage of mining, to meet the changing needs of Tahmoor Colliery and Wollondilly Shire Council

1.2. Objectives

The objectives of this Management Plan are to establish procedures to measure, control, mitigate and repair potential impacts that might occur to roads, bridges and culverts.

The objectives of the Plan have been developed to:-

- Ensure the safe and serviceable operation of all surface infrastructure. Public and workplace safety is paramount. Disruption and inconvenience should be kept to minimal levels.
- Monitor ground movements and the condition of surface infrastructure during mining.
- Initiate action to mitigate or remedy potential significant impacts that are expected to occur on the surface.
- Provide a plan of action in the event that the impacts of mine subsidence are greater than those that are predicted.
- Provide a forum to report, discuss and record impacts to the surface. This will involve Tahmoor Colliery, Wollondilly Council, Mine Subsidence Board, Department of Trade & Investment, Regional Infrastructure and Services (DTIRIS) and consultants as required.
- Establish lines of communication and emergency contacts.

1.3. Scope

The Management Plan is to be used to protect and monitor the condition of the items of infrastructure identified to be at risk due to mine subsidence. The major items at risk are:-

- Local roads
- Bridges
- Culverts

The Plan only covers infrastructure that is located within the limit of subsidence, which defines the extent of land that may be affected by mine subsidence as a result of mining Longwall 27. The management plan does not include other roads, bridges and culverts owned by Wollondilly Shire Council which lie outside the extent of these areas.

1.4. Proposed Mining Schedule

It is planned that each longwall will extract coal working northwest from the southeastern ends. This Plan covers longwall mining until completion of mining in Longwall 27 and for sufficient time thereafter to allow for completion of subsidence effects. The current schedule of mining is shown in Table 1.1.

Table 1.1 Schedule of Mining

Longwall	Start Date	Completion Date
Longwall 27	November 2012	October 2013

1.5. Definition of Active Subsidence Zone

As a longwall progresses, subsidence begins to develop at a point in front of the longwall face and continues to develop after the longwall passes. The majority of subsidence movement typically occurs within an area 150 metres in front of the longwall face to an area 450 metres behind the longwall face.

This is termed the "active subsidence zone" for the purposes of this Management Plan, where surface monitoring is generally conducted. The active subsidence zone for each longwall is defined by the area bounded by the predicted 20 mm subsidence contour for the active longwall and a distance of 150 metres in front and 450 metres behind the active longwall face, as shown by Fig. 1.1.

Fig. 1.1 Diagrammatic Representation of Active Subsidence Zone

2.1. Maximum Predicted Systematic Parameters

Predicted mining-induced systematic subsidence movements were provided in Report No. MSEC355, which was prepared in support of Tahmoor Colliery's SMP Application for Longwalls 27 to 30.

A summary of the maximum predicted incremental systematic subsidence parameters, due to the extraction of each of the proposed longwalls, is provided in Table 2.1. A summary of the maximum predicted cumulative systematic subsidence parameters, after the extraction of each of the proposed longwalls, is provided in Table 2.2. A summary of the maximum predicted travelling parameters, during the extraction of each of the proposed longwalls, is provided in Table 2.3.

Table 2.1 Maximum Predicted Incremental Systematic Subsidence Parameters due to the Extraction of Each of the Proposed Longwalls 27 to 30

Longwall	Maximum Predicted Incremental Subsidence (mm)	Maximum Predicted Incremental Tilt (mm/m)	Maximum Predicted Incremental Hogging Curvature (1/km)	Maximum Predicted Incrementa Sagging Curvature (1/km)
After LW27	755	6.0	0.07	0.14
After LW28	735	5.9	0.07	0.13
After LW29	735	5.9	0.06	0.13
After LW30	725	5.8	0.06	0.13

Table 2.2 Maximum Predicted Cumulative Systematic Subsidence Parameters after the Extraction of Each of the Proposed Longwalls 27 to 30

Longwall	Maximum Predicted Cumulaative Subsidence (mm)	Maximum Predicted Cumulative Tilt (mm/m)	Maximum Predicted Cumulative Hogging Curvature (1/km)	Maximum Predicted Cumulative Sagging Curvature (1/km)
After LW27	1260	6.3	0.09	0.15
After LW28	1270	6.2	0.09	0.14
After LW29	1270	6.1	0.09	0.14
After LW30	1270	6.3	0.09	0.14

The values provided in the above table are the maximum predicted cumulative systematic subsidence parameters which occur within the general SMP Area, including the predicted movements resulting from the extraction of Longwalls 22 to 30.

Table 2.3 Maximum Predicted Travelling Subsidence Parameters during the Extraction of Each of the Proposed Longwalls 27 to 30

Longwall	Maximum Predicted Travelling Tilt (mm/m)	Maximum Predicted Travelling Hogging Curvature (1/km)	Maximum Predicted Travelling Sagging Curvature (1/km)
During LW27	3.1	0.04	0.03
During LW28	3.0	0.03	0.03
During LW29	3.0	0.03	0.03
During LW30	3.0	0.03	0.03

2.2. Observed Subsidence during the mining of Longwalls 22 to 26

Extensive ground monitoring within the urban areas of Tahmoor has allowed detailed comparisons to be made between predicted and observed subsidence, tilt, strain and curvature during the mining of Longwalls 22 to 26.

In summary, there is generally a good correlation between observed and predicted subsidence, tilt and curvature. Observed subsidence was generally slightly greater than predicted in areas that were located directly above previously extracted areas and areas of low level subsidence (typically less than 100 mm) was generally observed to extend further than predicted.

While there is generally a good correlation between observed and predicted subsidence, substantially increased subsidence has been observed above most of Longwall 24A and the southern end of Longwall 25. This was a very unusual event for the Southern Coalfield.

Observed Increased Subsidence during the mining of Longwall 24A

Observed subsidence was greatest above the southern half of Longwall 24A, and gradually reducing in magnitude towards the northern half of the longwall, which was directly beneath the urban area of Tahmoor. These observations are shown graphically in Fig. 2.1, which shows observed subsidence at survey pegs located along the centreline of Longwall 24A.

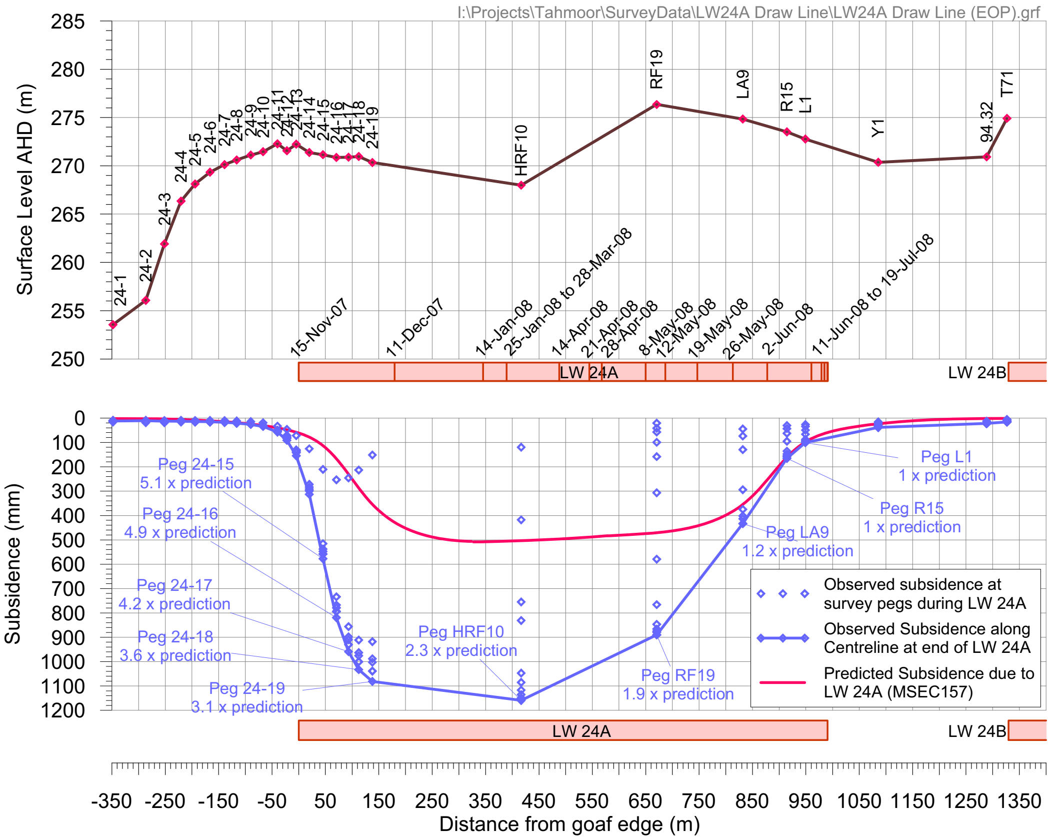


Fig. 2.1 Observed Subsidence along Centreline of Longwall 24A

It can be seen from Fig. 2.1 that observed subsidence was more than twice the predicted maximum value, reaching to a maximum of 1169 mm at Peg HRF10. It is possible that actual maximum subsidence developed somewhere between Pegs HRF10 and RF19, though this was not measured. Observed subsidence was similar to prediction near Peg R15 on Remembrance Drive. Survey pegs RF19 and LA9 are located within a transition zone where subsidence gradually reduced from areas of maximum increased subsidence to areas of normal subsidence.

Observed Increased Subsidence during the mining of Longwall 25

Increased subsidence was observed during the first stages of mining Longwall 25. These observations are shown graphically in Fig. 2.2, which shows observed subsidence at survey pegs located along the centreline of Longwall 25.

It can be seen from Fig. 2.2 that observed subsidence was approximately twice the predicted maximum value, with maximum subsidence of 1216 mm at Peg 25-28.

Observed subsidence is similar to but slightly more than predicted at Peg RE7 and is similar to prediction at Peg Y20 and at all pegs located further along the panel. Survey pegs A6, A7, A8 and A9 are located within a transition zone where subsidence has gradually reduced from areas of maximum increased subsidence to areas of normal subsidence.

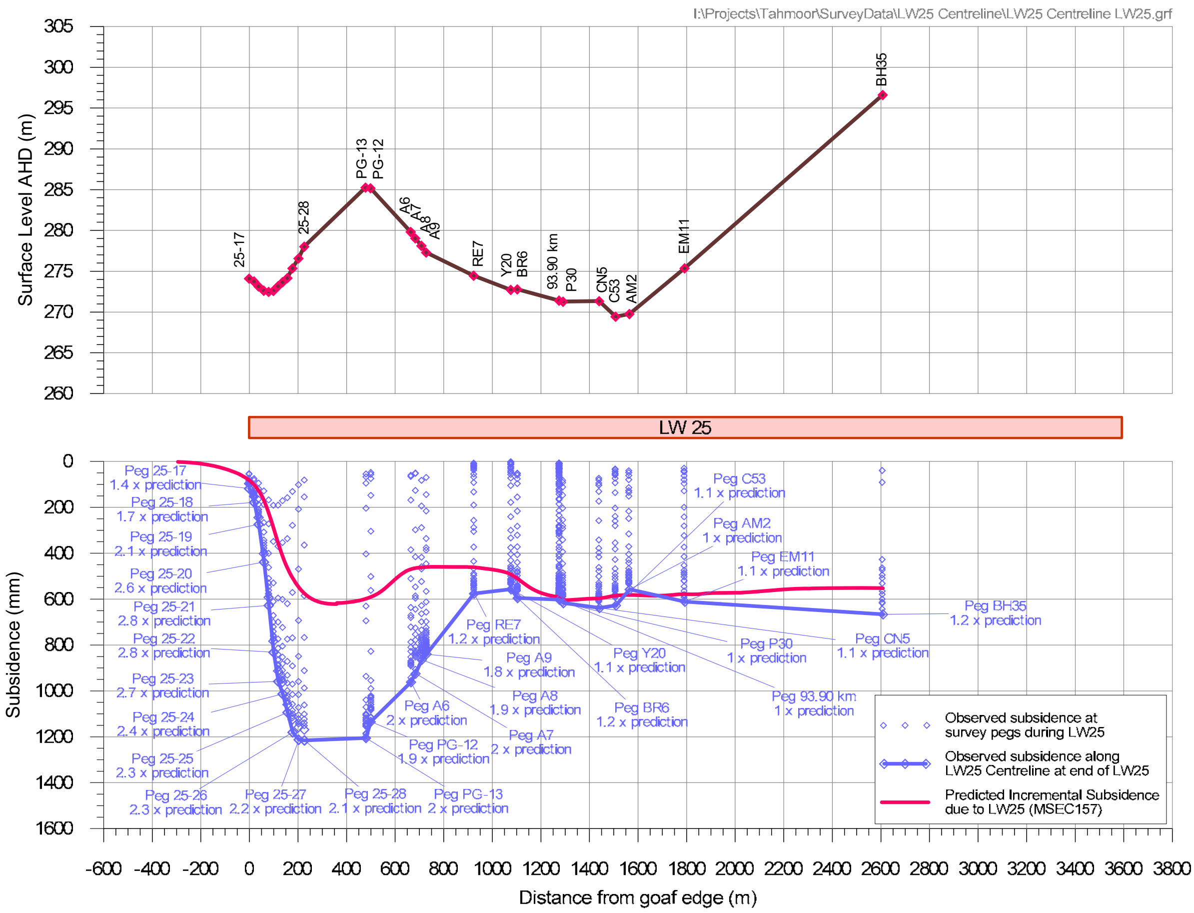


Fig. 2.2 Observed Subsidence along Centreline of Longwall 25

Observed Increased Subsidence during the mining of Longwall 26

Increased subsidence was observed during the first stages of mining Longwall 26, but at a reduced magnitude compared to the subsidence observed above Longwalls 24A and 25. These observations are shown graphically in Fig. 2.3, which shows observed subsidence at survey pegs located along the centreline of Longwall 26. The graph shows the latest survey results for each monitoring line as at August 2012. It is likely that further small increases in subsidence will be observed at these pegs when they are surveyed at the completion of Longwall 26.

It can be seen from Fig. 2.3 that observed subsidence was approximately 1.3 times the predicted maximum value, with maximum subsidence of 867 mm at Peg TM26.

Observed subsidence reduced along the panel until Peg Y40 on York Street, where it was less than prediction. Survey pegs S9, and RE27 are located within a transition zone where subsidence has gradually reduced from areas of maximum increased subsidence between Pegs TM26 and MD4 to areas of normal subsidence at Peg Y40 and beyond.

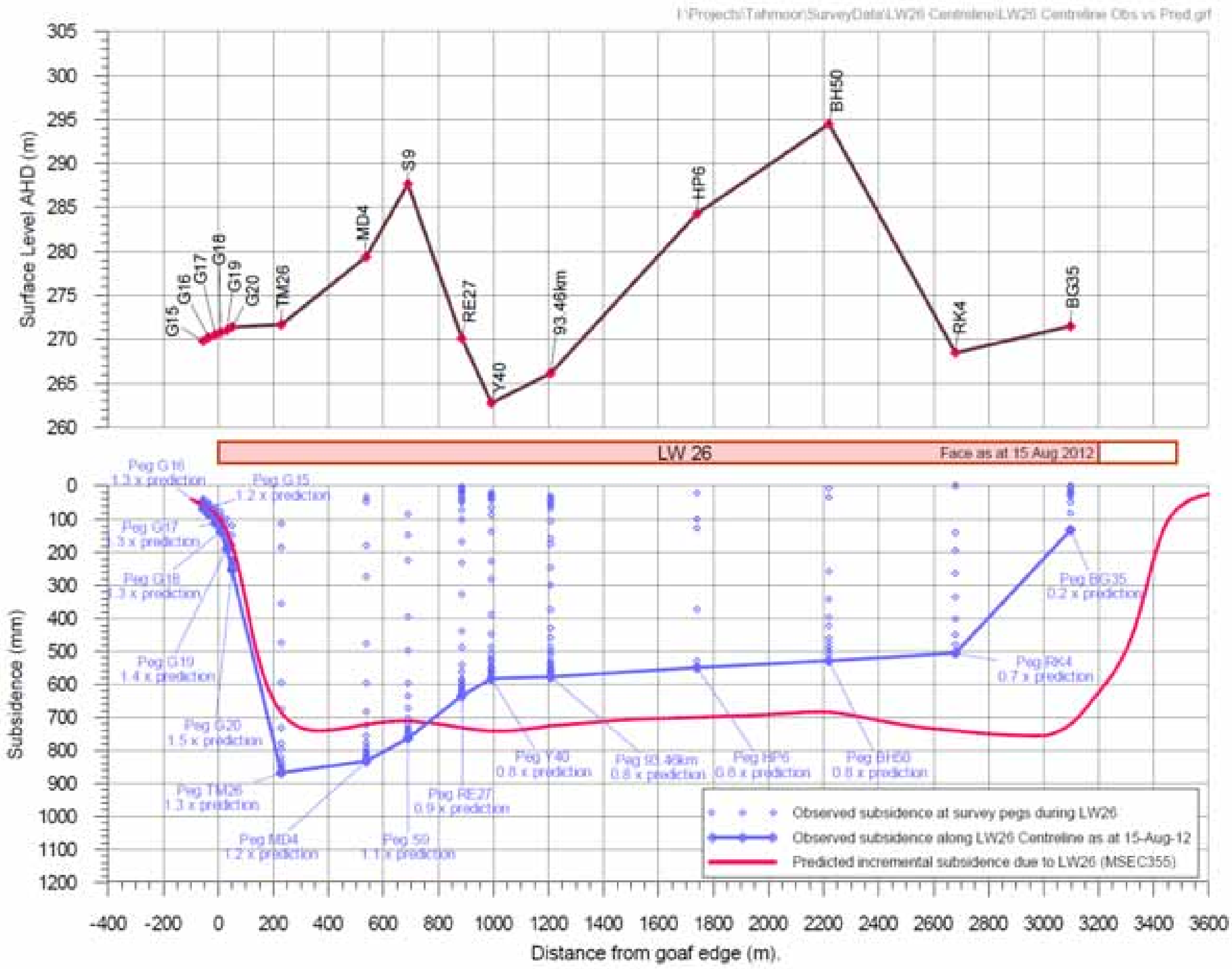


Fig. 2.3 Observed Subsidence along Centreline of Longwall 26 as at August 2012

Analysis and commentary

The cause for the increased subsidence has been investigated by Strata Control Technologies on behalf of Tahmoor Colliery (Gale and Sheppard, 2011). The investigations concluded that the increased subsidence is consistent with localised weathering of joint and bedding planes above a depressed water table adjacent to an incised gorge.

In light of the above observations, the region above the extracted longwalls at Tahmoor has been partitioned into three zones:

1. Normal subsidence zone – where the observed vertical subsidence is within the normal range and correlates well with predictions

2. Maximum increased subsidence zone – where the observed vertical subsidence is substantially greater than predictions but has reached it upper limit. Maximum subsidence above the centreline of the longwalls appears to be approximately 1.2 metres above Longwalls 24A and 25, and 900 mm above Longwall 26.

3. Transition zone – where the subsidence behaviour appears to have transitioned between areas of maximum increased subsidence and normal subsidence.

When the locations of the three zones are plotted on a map, as shown in Drawing No. MSEC567-00-01 (refer Appendix), it can be seen that the transition zone is roughly consistent in width above Longwall 24A, Longwall 25 and Longwall 26. The orientation of the transition zone is also roughly parallel to the Nepean Fault and not the Bargo River.

Prior to the mining of Longwall 26, it was not yet known whether the location of the transition zone was related to the alignment of the Nepean Fault or the Bargo River as both features were aligned approximately parallel to each other adjacent to previously extracted Longwalls 24A and 25.

The Bargo River, however, abruptly turns a sharp bend near the end of Longwalls 25 and 26 and observations during the mining of Longwall 26 were able to provide a first indication that the location of the transition zone was related to the alignment of the Nepean Fault, rather than the Bargo River.

The magnitude of subsidence above Longwall 26 is reduced compared to Longwalls 24A and 25. Given that the alignment of the Nepean Fault moves away from the Bargo River above Longwall 26, it appears that the magnitude of increased subsidence is linked to the proximity of the Bargo River. This observation confirms the findings of Gale and Sheppard that the increased subsidence is linked to localised weathering of joint and bedding planes above a depressed water table adjacent to the incised gorge of the Bargo River.

In summary, it appears that the location of increased subsidence is linked to the alignment of the Nepean Fault and the magnitude of the increased subsidence is linked to the proximity to the Bargo River.

The zones have been projected above Longwalls 27 to 30 from the observed zones above Longwalls 24A and 26, as shown in Drawing No. MSEC567-00-02 (refer Appendix). The projection is based on the orientation of the Nepean Fault. It can be seen that the transition zone extends to sections of Myrtle Creek Avenue, Remembrance Drive, Myrtle Creek and the Main Southern Railway.

Given that Longwall 27 is located further away from the Bargo River than Longwall 26, it is expected that the magnitude of maximum subsidence at the commencing end of Longwall 27 will be less than 900 mm. The amount of reduction in maximum subsidence is difficult to predict. The difference in maximum subsidence between Longwalls 24A and 25 and Longwall 26 is approximately 300 mm. If maximum subsidence at the commencing end of Longwall 27 reduces a further 300 mm, the magnitude of subsidence at the commencing end will return to normal levels.

It is recognised that despite the above analysis and projections, substantially increased subsidence could develop as the mining of Longwall 27 progresses. This Management Plan has been developed to manage potential impacts if substantial additional subsidence were to occur.

2.3. Predicted Strain

The prediction of strain is more difficult than the predictions of subsidence, tilt and curvature. The reasons for this are that strain is affected by many factors, including ground curvature and horizontal movement, as well as local variations in the near surface geology, the locations of joints at bedrock, and the depth of bedrock. The measurements are also affected by survey tolerance. The profiles of observed strain can, therefore, be irregular even when the profiles of observed subsidence, tilt and curvature are relatively smooth.

The relative frequency distribution of maximum observed tensile strains and compressive strains for survey bays located directly above goaf is provided in Fig. 2.4.

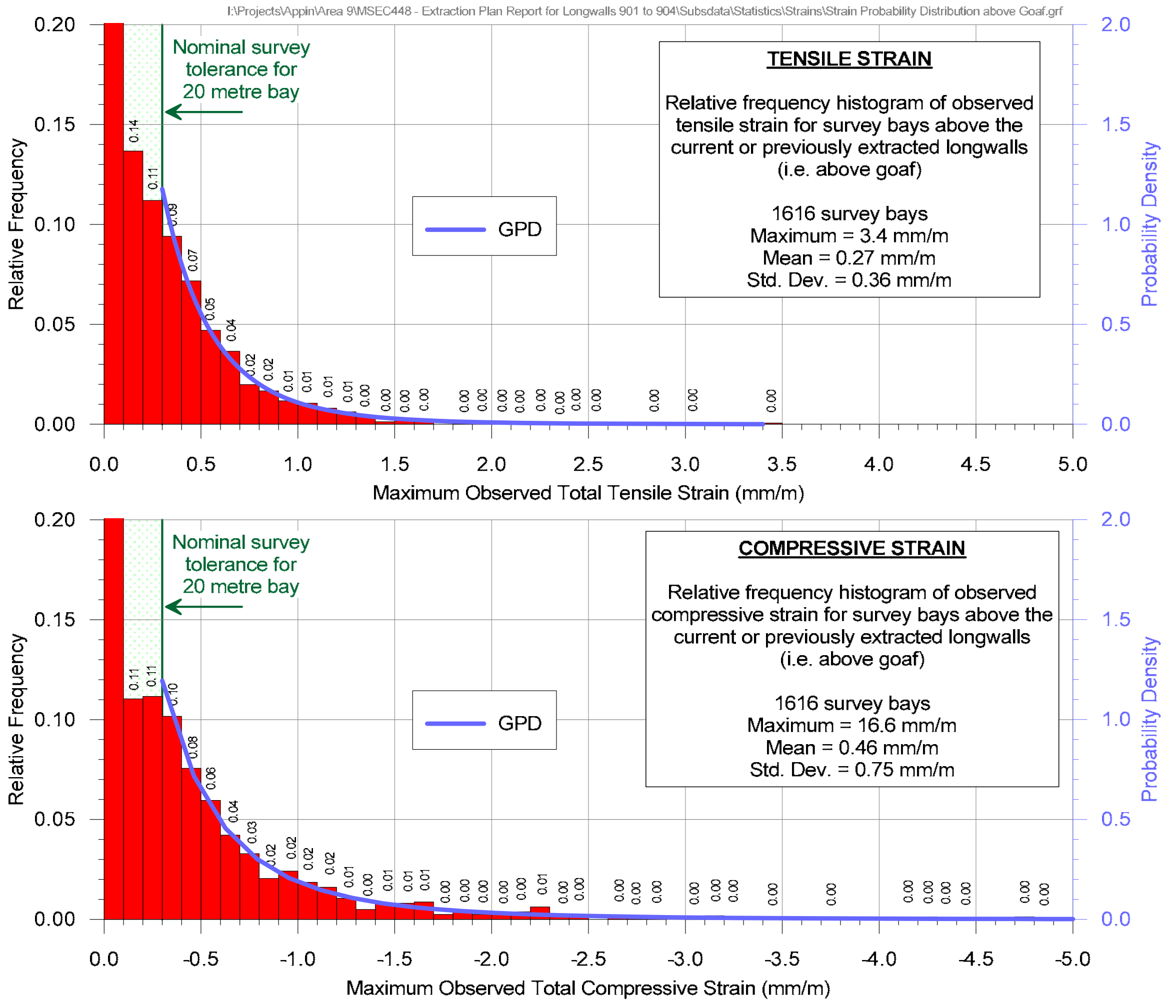


Fig. 2.4 Distributions of Measured Maximum Tensile and Compressive Strains at Any Time for Pegs Located Above Goaf in the Southern Coalfield

While not shown in Fig. 2.4, it is noted that the maximum observed compressive strain of 16.6 mm/m, which occurred along the T-Line above Appin Longwall 408, was the result of movements along a low angle thrust fault within the Cataract Tunnel. All remaining compressive strains in this dataset (which exclude valley related movements) were less than 5 mm/m.

The relative frequency distribution of maximum observed tensile strains and compressive strains above solid coal is provided in Fig. 2.5.

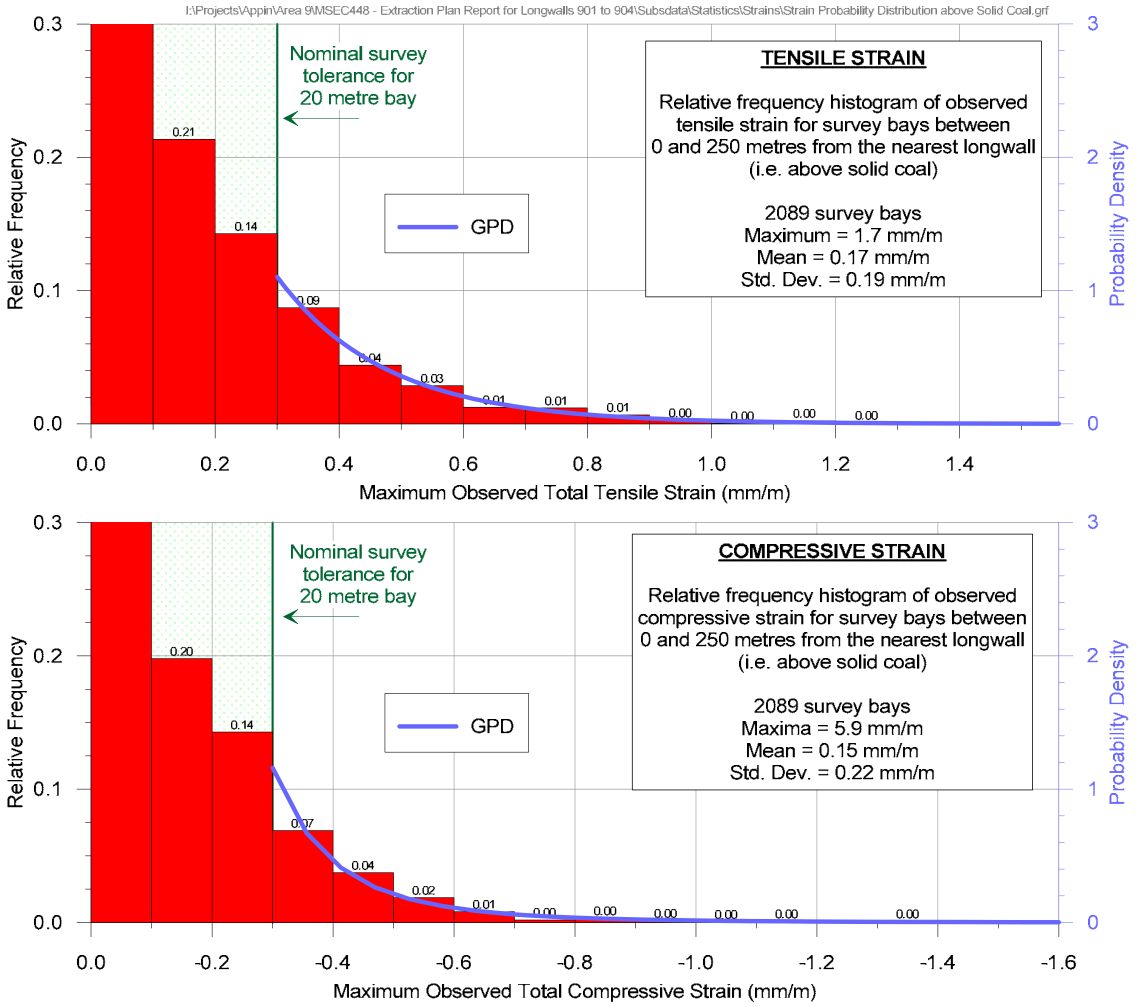


Fig. 2.5 Distributions of Measured Maximum Tensile and Compressive Strains at Any Time for Pegs Located Above Solid Coal in the Southern Coalfield

While not shown in Fig. 2.5, it is noted that the maximum observed compressive strain of 5.9 mm/m, which occurred along the T-Line above Appin Longwall 408, was the result of movements along a low angle thrust fault within the Cataract Tunnel as Longwall 408 approached the monitoring line. A maximum observed compressive strain of 3.1 mm/m was observed across the fault at the completion of Longwall 407. All remaining compressive strains in this dataset (which exclude valley related movements) were less than 5 mm/m.

2.4. Predicted and Observed Valley Closure across creeks

A number of bridges and culverts above Longwall 27 carry road transport over Myrtle Creek and other watercourses. Predictions of valley closure and upsidence at each of these features are provided later in this Management Plan.

A comparison between predicted and observed valley closure movements is provided below.

A map of monitoring lines across Myrtle Creek and a small creek that crosses the Main Southern Railway (called the Skew Culvert) is shown in Fig. 2.6.

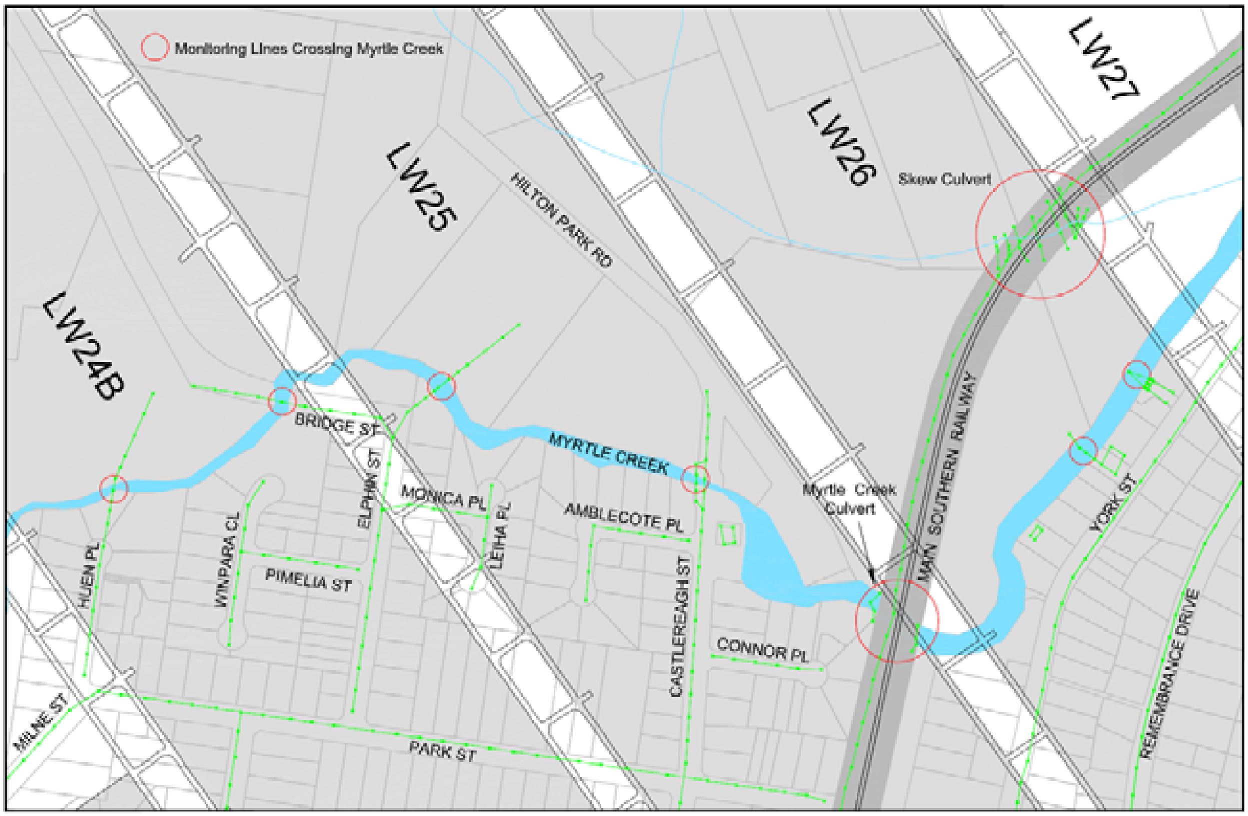


Fig. 2.6 Monitoring lines across Myrtle Creek and Skew Culvert

A summary graph showing the development of valley closure across the Myrtle Creek at each monitoring line is shown in Fig. 2.7.

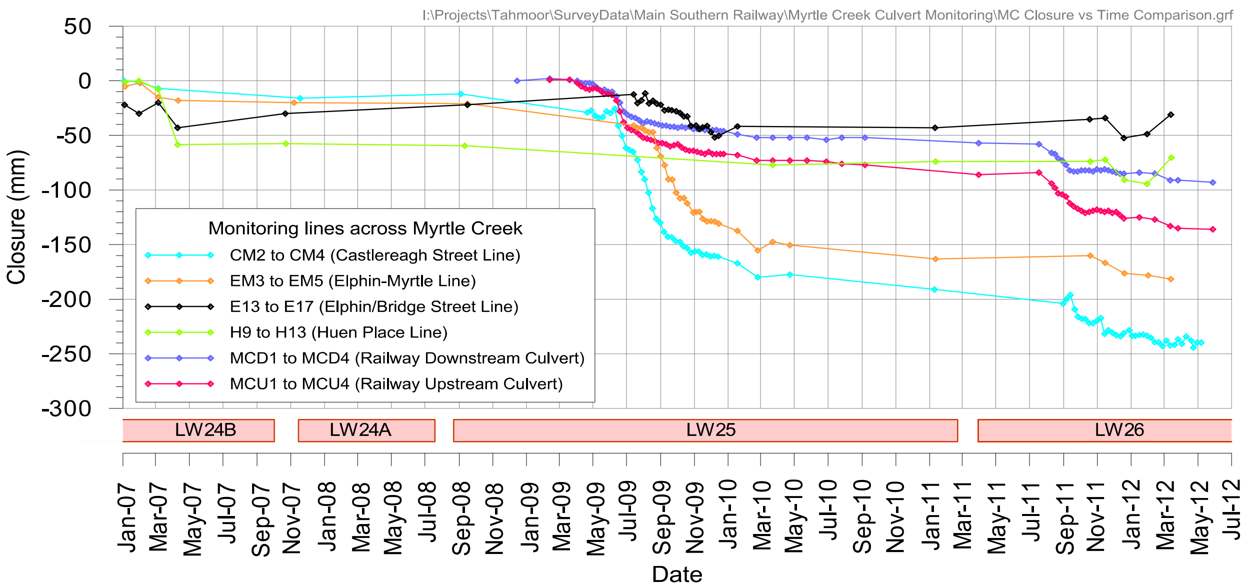


Fig. 2.7 Development of closure across Myrtle Creek during the mining of Longwalls 24B to 26

The development of valley closure across the creek at the Skew Culvert is shown in Fig. 2.8.

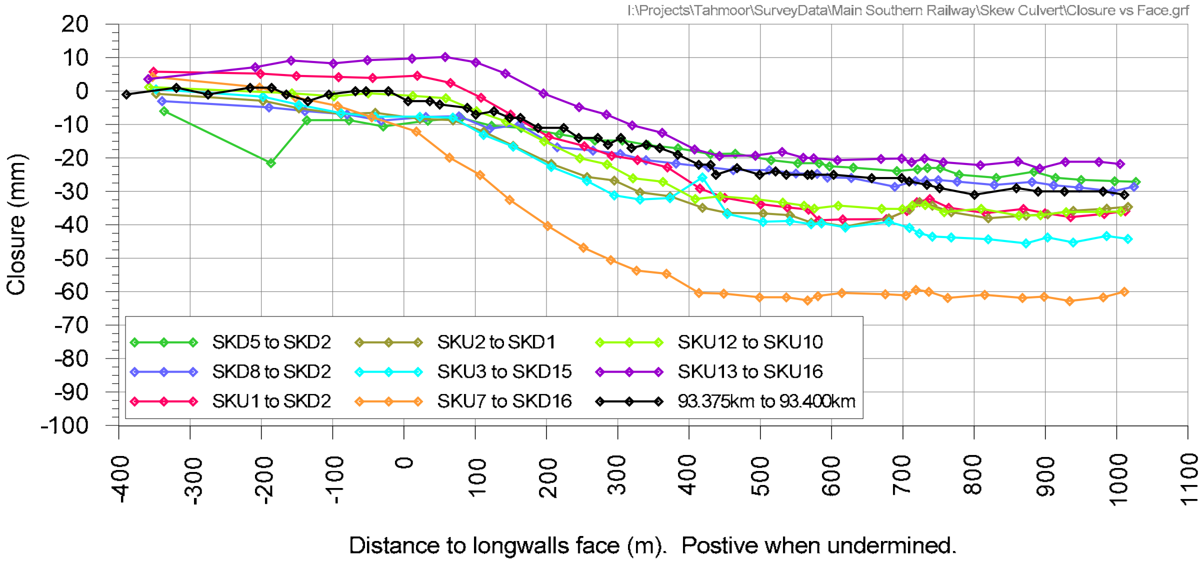


Fig. 2.8 Development of closure across Skew Culvert during the mining of Longwall 26 as at 27 March 2012

A summary of predicted and observed valley closure across Myrtle Creek is provided in Table 2.4. The predictions are consistent with those provided in Report No. MSEC355, in support of Tahmoor Colliery's SMP application to extract longwalls 27 to 30.

Table 2.4 Predicted and Observed Incremental Valley Closure across Myrtle Creek and Skew Culvert at monitoring lines

		Predicted and Observed Valley Closure due to mining of each longwall(s)		
		Due to LW24 (mm)	Due to LW25 (mm)	Due to LW26 (mm)
Castlereagh Street (Pegs C2 to C4)	Predicted	30	55	45
	Observed	12	179	49
Elphin-Myrtle (Pegs EM3 to EM5)	Predicted	60	70	40
	Observed	21	142	22
Elphin Street / Bridge Street (Pegs E13 to E17)	Predicted	75	75	30
	Observed	0	21	6
Huen Place (Pegs H9 to H13)	Predicted	60	35	15
	Observed	58	15	20
Main Southern Railway Upstream (MCU1 to MCU4) Downstream (MCD1 to MCD4)	Predicted	15	30	30
	Observed	-	57 (d/s) to 86 (u/s)	36 (d/s) to 50 (u/s)
Skew Culvert (8 cross sections)	Predicted	< 5	10	25
	Observed	-	-	21 to 60 (avg 36)
13 York Street (Y64-6 to Y64-9)	Predicted	-	-	65
	Observed	-	-	60
9a York Street (Y67-10 to Y67-14)	Predicted	-	-	85
	Observed	-	-	73

It can be seen that observed valley closure has substantially exceeded predictions at the Castlereagh Street crossing, at the crossing of the Elphin-Myrtle monitoring line and to a lesser extent the crossing of the Main

Southern Railway during the mining of Longwall 25. It is considered that the reason for the differences in observations may be linked to the change in orientation of Myrtle Creek as the three above-mentioned monitoring lines are located along the same stretch of Myrtle Creek.

Observed valley closure across the creek at the Skew Culvert has also slightly exceeded predictions, where the differences between predicted and observed closure are relatively small for most cross sections.

3.1. General

The Australian/New Zealand standard for Risk Management defines the terms used in the risk management process, which includes the identification, analysis, assessment, treatment and monitoring of risk. In this context:-

3.1.1. Consequence

'The outcome of an event expressed qualitatively or quantitatively, being a loss, injury, disadvantage or gain. There may be a range of possible outcomes associated with an event.'[1] The consequences of a hazard are rated from very slight to very severe.

3.1.2. Likelihood

'Used as a qualitative description of probability or frequency.'[2] The likelihood can range from very rare to almost certain.

3.1.3. Hazard

'A source of potential harm or a situation with a potential to cause loss.'[3]

3.1.4. Risk

'The chance of something happening that will have an impact upon objectives. It is measured in terms of consequences and likelihood.'[4] The risk combines the likelihood of an impact occurring with the consequence of the impact occurring. The risk is rated from very low to extreme. In this study, the likelihood and consequence are combined via the qualitative risk analysis matrix shown in Table 3.1, to determine an estimated level of risk for particular events or situations.

The Risk Analysis Matrix is similar to the example provided in AS/NZS 4360:1995, Appendix D, p.25.

Table 3.1 Qualitative Risk Analysis Matrix

Likelihood	CONSEQUENCES				
	Very Slight	Slight	Moderate	Severe	Very Severe
Almost Certain	Low	Moderate	High	Extreme	Extreme
Likely	Low	Moderate	High	Very High	Extreme
Moderate	Low	Low	Moderate	High	Very High
Unlikely	Very Low	Low	Moderate	High	High
Rare	Very Low	Very Low	Low	Moderate	High
Very Rare	Very Low	Very Low	Low	Moderate	Moderate

This Management Plan adopts a common system of nomenclature to summarise each risk analysis, which is **"LIKELIHOOD / CONSEQUENCE → LEVEL OF RISK"**.

For example, if the likelihood of a risk is assessed as "**UNLIKELY**", and the consequence of a risk is assessed as "**SEVERE**", the risk analysis would be summarised as **"UNLIKELY / SEVERE → HIGH"**.

[1] AS/NZS 4360:1999 – Risk Management pp2
[2] AS/NZS 4360:1999 – Risk Management pp2
[3] AS/NZS 4360:1999 – Risk Management pp2
[4] AS/NZS 4360:1999 – Risk Management pp3

4.1. Local Roads

There are a number of local roads directly above Longwall 27, as shown in Drawing No. MSEC567-02-01.

The main road is Remembrance Drive (formerly the Hume Highway), which connects Tahmoor with Picton to the north, and Bargo to the south. Some main services infrastructure is located along Remembrance Drive, and includes gas mains, water mains, and optical fibre cables. The main retail and commercial buildings are also located along Remembrance Drive. Remembrance Drive crosses over Longwalls 24A to 27.

The other significant road within the vicinity of Longwall 27 is Bridge Street, which connects Thirlmere with Picton to the northeast. Bridge Street crosses directly over Longwall 27, and it has recently been undermined by Longwall 26.

The network of local roads is spread across Longwall 27, and therefore, they collectively will experience the full range of subsidence impacts, as described in Section 2.1. A discussion on the expected range of tensile and compressive strains during the mining of the proposed longwalls is provided in Section 2.3.

Predictions of systematic subsidence, tilt and strain were made along two major roads, Remembrance Drive and Bridge Street, which are shown in Fig. 4.2 and Fig. 4.3, respectively. A summary of the maximum predicted systematic subsidence, tilt and strain along these roads, due to the extraction of Longwalls LW26 to LW30, is provided in Table 4.1.

Table 4.1 Maximum Predicted Cumulative Systematic Subsidence, Tilt and Curvature along the Alignments of Remembrance Drive and Bridge Street due to the Extraction of Longwalls 22 to 30

Location	Longwall	Maximum Predicted Cumulative Subsidence (mm)	Maximum Predicted Cumulative Tilt (mm/m)	Maximum Predicted Cumulative Hogging Curvature (1/km)	Maximum Predicted Cumulative Sagging Curvature (1/km)
Remembrance Drive	After LW26	890	5.3	0.05	0.08
	After LW27	1135	5.7	0.07	0.11
	After LW28	1150	4.5	0.07	0.11
	After LW29	1155	4.2	0.07	0.11
	After LW30	1155	4.3	0.07	0.11
Bridge Street	After LW26	785	5.4	0.06	0.11
	After LW27	1035	6.0	0.06	0.12
	After LW28	1105	6.0	0.07	0.13
	After LW29	1185	5.9	0.08	0.13
	After LW30	1200	6.1	0.08	0.13

The roads will also be subjected to travelling tilts and curvatures as the extraction faces of the proposed longwalls pass beneath them. A summary of the maximum predicted travelling tilts and curvatures at the roads, during the extraction of each of the proposed longwalls, is provided in Table 4.2.

Table 4.2 Maximum Predicted Travelling Tilts and Curvatures at Remembrance Drive and Bridge Street during the Extraction of Longwalls 27 to 30

Location	Longwall	Maximum Predicted Travelling Tilt (mm/m)	Maximum Predicted Travelling Hogging Curvature (1/km)	Maximum Predicted Travelling Sagging Curvature (1/km)
Remembrance Drive	During LW27	2.9	0.03	0.02
	During LW28	1.9	0.02	0.02
Bridge Street	During LW27	3.0	0.03	0.03
	During LW28	3.0	0.03	0.03
	During LW29	3.0	0.03	0.03
	During LW30	2.9	0.03	0.02

The maximum predicted tilt of 6.1 mm/m, or a change in gradient of 0.6% is very small considering that sealed roads are usually constructed with gradients of approximately 3.0%. The resulting change in road superelevation or gradient is unlikely to affect the serviceability of the road.

Monitoring of road pavements has been undertaken at Tahmoor during the extraction of Longwalls 22 to 26 at Tahmoor Colliery. The monitoring includes a network of ground monitoring lines and weekly visual inspections in areas that are experiencing active subsidence. Approximately 19.3 kilometres of asphaltic pavement lie directly above the extracted longwalls and a total of 39 impact sites have been reported. The observed rate of impact equates to an average of one impact for every 495 metres of pavement. The impacts were minor and did not present a public safety risk.

One of these impact sites, located on Lintina Street above Longwall 24A, was substantially greater than the other 11 impact sites. A selection of photographs is provided in Fig. 4.1. The impacts on Lintina Street were repaired twice by the Mine Subsidence Board as the longwall progressed.

Impacts were also observed during the mining of Longwall 25. A hump was observed on Abelia Street during the mining of Longwall 25 and this has been repaired by the Mine Subsidence Board. A hump was also observed on Remembrance Drive at the roundabout intersection with Thirlmere Way, as shown in Fig. 4.1.

While impacts have been observed to local roads at a number of locations during the mining of Longwall 26, they have not been as severe as those observed on Lintina Street and Abelia Street, though some have required urgent repairs.

More frequent impacts have been observed to concrete kerbs and gutters. The impacts are most commonly focussed around driveway laybacks and involve cracking, spalling or buckling. A typical buckling impact is shown in Fig. 4.1.

A total of 5 drainage pits have been damaged during the mining of Longwalls 24A and 25 in Janice and Abelia Streets. Investigations are currently underway to determine whether impacts have occurred to stormwater pipes in these areas.

Traffic signs and other road infrastructure have not previously experienced any impacts due to mine subsidence.

It is expected that minor impacts will occur to the local roads during mining, similar in frequency and severity to those experienced during the mining of Longwalls 22 to 26.

Lintina Street (most severe to date)

Remembrance Drive (hump at roundabout)

Brundah Road (typical impact to pavement)

Patterson Street (typical impact to kerb)

Photographs courtesy of Tahmoor Colliery and Colin Dove

Fig. 4.1 Photographs of impacts to road pavements and kerbs during the mining of LWs 22 to 26

4.1.1. Risk Assessment

The risk to local sealed roads is that deformation (cracking, buckling or wrinkling) of the road surface may occur. Four levels of impact, in increasing order of severity, have been identified for risk analysis.

1. Minor deformations (cracks less than 2 mm), occurring infrequently within the road network
2. Minor deformations (cracks less than 2 mm), occurring extensively within the road network
3. Major deformations (cracks greater than 2 mm), occurring infrequently within the road network
4. Major deformations (cracks greater than 2 mm), occurring extensively within the road network

Table 4.3 summarises the risk analysis for local sealed roads.

Table 4.3 Risk Analysis for Local Sealed Roads

Level of Impact	Likelihood	Consequence	Level of Risk
Infrequent, minor deformations	LIKELY	SLIGHT	MODERATE
Frequent, minor deformations	UNLIKELY	MODERATE	MODERATE
Infrequent, major deformations	UNLIKELY	MODERATE	MODERATE
Frequent, major deformations	VERY RARE	SEVERE	MODERATE

Any damage to local roads will be repaired at the expense of the Mine Subsidence Board.

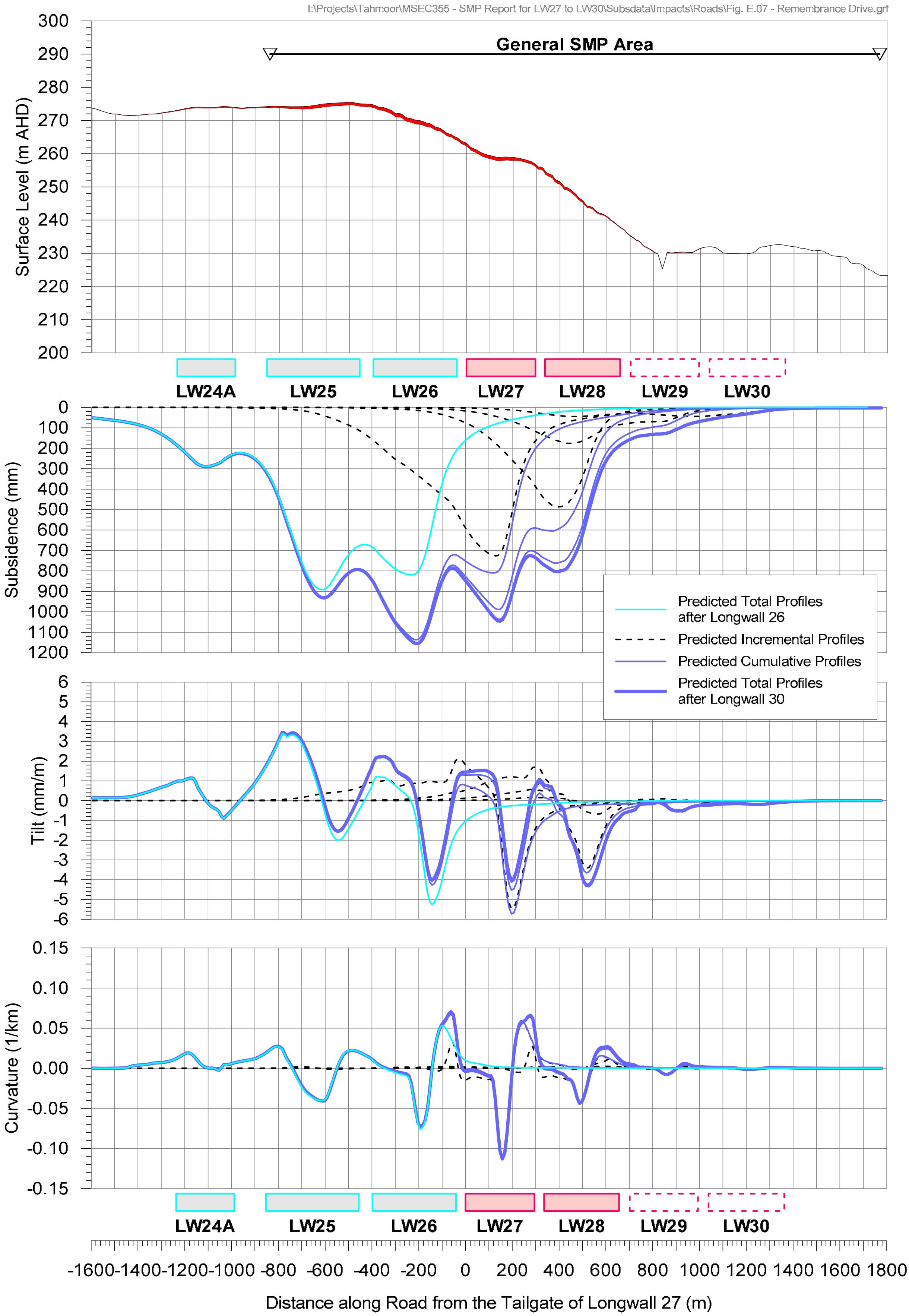


Fig. 4.2 Predicted Subsidence, Tilt and Curvature along Remembrance Drive due to the mining of Longwalls 22 to 30 (Source: Report No. MSEC355)

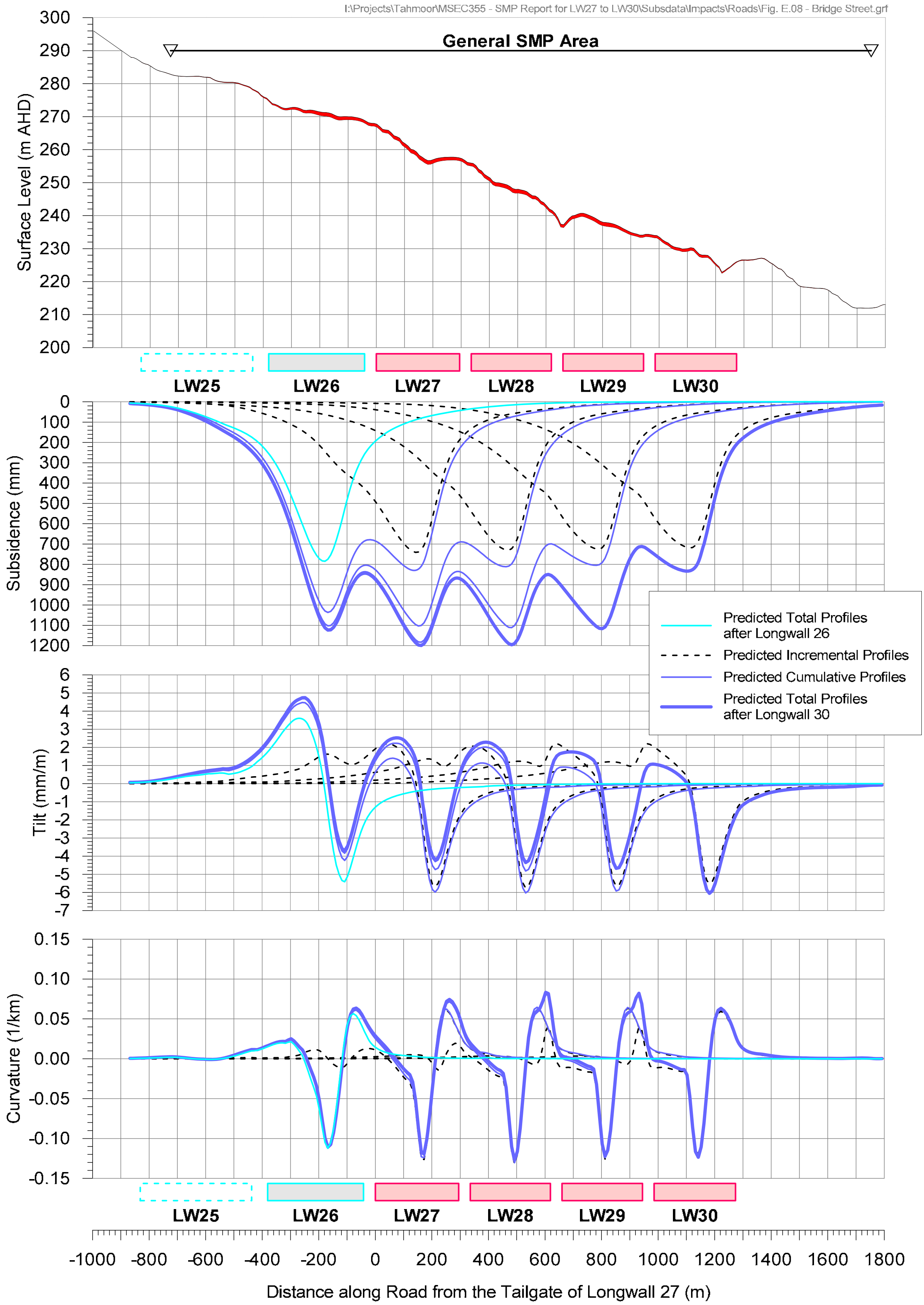


Fig. 4.3 Predicted Subsidence, Tilt and Curvature along Bridge Street due to the mining of Longwalls 22 to 30 (Source: Report No. MSEC355)

4.2. Bridge on Castlereagh Street over Myrtle Creek

The Castlereagh Street Road Bridge over Myrtle Creek is a two-lane bridge that provides access for residents to Hilton Park Road. The single-span bridge is constructed with a concrete deck on concrete abutments, as shown in Fig. 4.4. This bridge is located above Longwall 25.

Fig. 4.4 Castlereagh Street Road Bridge over Myrtle Creek

4.2.1. Predicted Subsidence Movements

The Castlereagh Street Road Bridge over Myrtle Creek is a two-lane bridge that provides access for residents to Hilton Park Road.

Predictions of systematic subsidence, tilt and strain movements have been made at the bridge, and these are shown in Table 4.4.

Table 4.4 Predicted Subsidence Parameters at Castlereagh Street Road Bridge

Stage of Mining	Maximum Predicted Subsidence (mm)	Maximum Predicted Tilt (mm/m)	Maximum Predicted Tension (mm/m)	Maximum Predicted Compression (mm/m)
After Longwall 27	1105	4.7	0.07	0.02
After Longwall 28	1120	4.6	0.07	0.02
After Longwall 29	1120	4.6	0.07	0.02
After Longwall 30	1125	4.5	0.07	0.02

The Bridge will also be subjected to upsidence and closure movements, and these are shown in Table 4.5.

Table 4.5 Prediction of Upsidence and Closure at Castlereagh Street Road Bridge

Stage of Mining	Maximum Cumulative Closure (mm)	Maximum Cumulative Upsidence (mm)
Increment due to LW27 only	25	25
Total due to LWs 22 to 27	160	195
Total due to LWs 22 to 28	170	200
Total due to LWs 22 to 29	175	205
Total due to LWs 22 to 30	175	205

It can be seen from Table 4.5 that the majority of valley closure movements have already occurred at Castlereagh Street Bridge, with 25 mm of valley closure predicted to occur during the mining of Longwall 27.

A comparison between observed and predicted valley closure was provided in Section 2.4. While observed valley closure substantially exceeded predicted closure during the mining of Longwall 25, there was a reasonable correlation between predicted and observed valley closure during the mining of Longwall 26.

4.2.2. Previous experiences at Castlereagh Street Bridge during mining of Longwalls 24B to 26

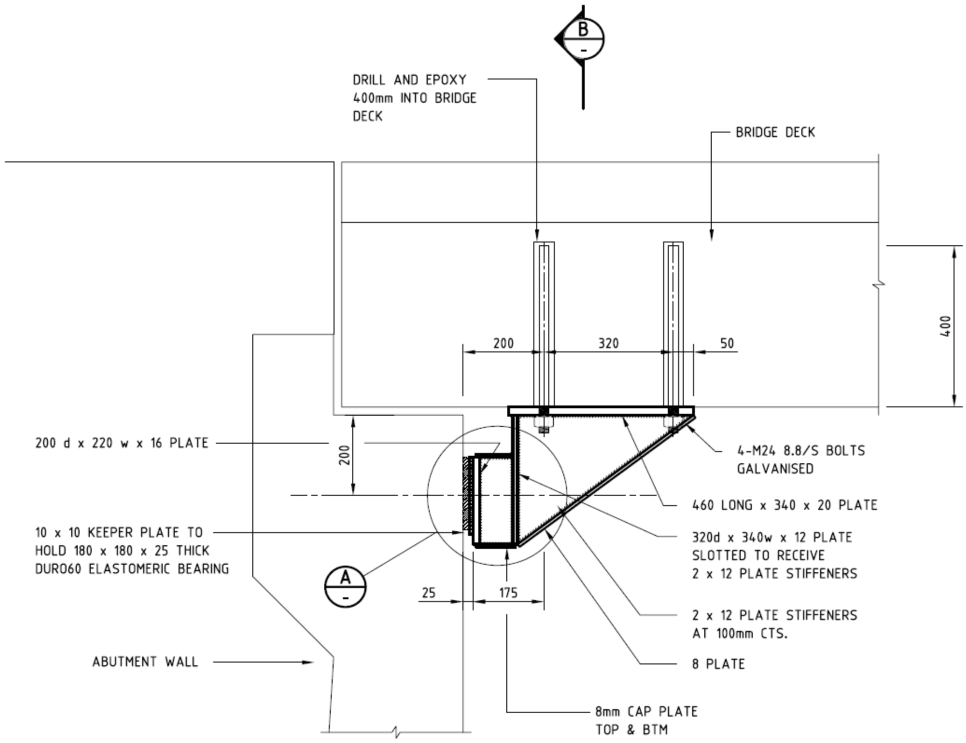
Mitigation measures prior to mining

The single-span bridge is constructed with a concrete deck on concrete abutments. The span of the deck is approximately 12 metres and the heights of the abutments are approximately 6 and 8 metres high.

The deck comprises pre-tensioned bridge units that have been integrated with a reinforced concrete slab. The reinforced concrete abutments and wing walls were dowelled and grouted into the bedrock. Prior to mining, the bridge deck rests on the abutments with rubber bearing pads and was fixed at both ends with vertical dowelled joints spaced at regular centres. There was approximately 5 mm of clearance between each dowel and its hole and a rubber ring had been placed in this gap. Each abutment included a small upstand that prevented the deck from sliding. There was a 20 mm gap between the upstand and the ends of the deck. The gap was filled with polystyrene fillers, rubber buffers and mastic fillers.

Prior to mining, a structural assessment of Castlereagh Street Bridge was undertaken by John Matheson & Associates (JMA, 2006), based on a site inspection and review of structural drawings that were provided by Wollondilly Shire Council. The assessment identified that in the event of abutment closure, it was expected that the concrete hob behind the abutment will initially crack and shear. This impact was not expected to result in any reduction in stability of the bridge. As abutment closure increased the steel dowels that connected the deck to the abutment walls were expected to shear. This impact was not expected to result in any long-term structural concerns as the abutment walls were designed as freestanding cantilevers. However, JMA identified a potential risk where cracking may occur to the abutment corbel at the time the dowels sheared and that this cracking may have occurred suddenly as the abutment support transferred from a propped cantilever to a freestanding cantilever.

JMA recommended that the abutments could be protected at the moment of dowel failure by the installation of steel brackets bolted to the end of each bridge deck girder (13 number girders and 26 number brackets), which bear against the abutment walls (JMA, 2007). The brackets provided additional support to the abutment walls, ensuring that the dowels could shear without breaking the concrete that surround them. The design of the brackets is shown in Fig. 4.5 and a photograph of the completed installation is shown in Fig. 4.6. The galvanised brackets were installed by Tahmoor Colliery prior to the commencement of Longwall 25.



Extract of drawing by JMA (Drawing No. 0622-1.00)

Fig. 4.5 Design of Mitigation Measures beneath Bridge Deck

Fig. 4.6 Photograph of Installed Brackets at Castlereagh Street Bridge

Experience during the mining of Longwall 25

The survey results for Castlereagh Street Bridge showed that while the creek sides had closed considerably, the bridge had closed significantly less with the exception of the end of the south-eastern wing wall. The resistance of the bridge structure to closure had resulted in compressive heaving in the road pavement on the southern side of the bridge and damage to the telecommunications conduit at the north-western abutment. Existing cracks on the southern abutment were observed to extend slowly during mining, particularly at the interface between the abutment and south-eastern wing wall.

Differential movements between the bridge deck and the abutments had been observed to gradually increase and they exceeded the BLUE trigger level that had been defined in the Longwall 25 Management Plan. The brackets were cut and set back from the southern abutment on Monday, 7 September 2009. Further small movements were observed after the brackets were removed. At the completion of mining, there was a small air gap between the abutment walls and the bracket supports so that there was no pressure on the brackets, abutments or bridge deck, as shown in Fig. 4.7.

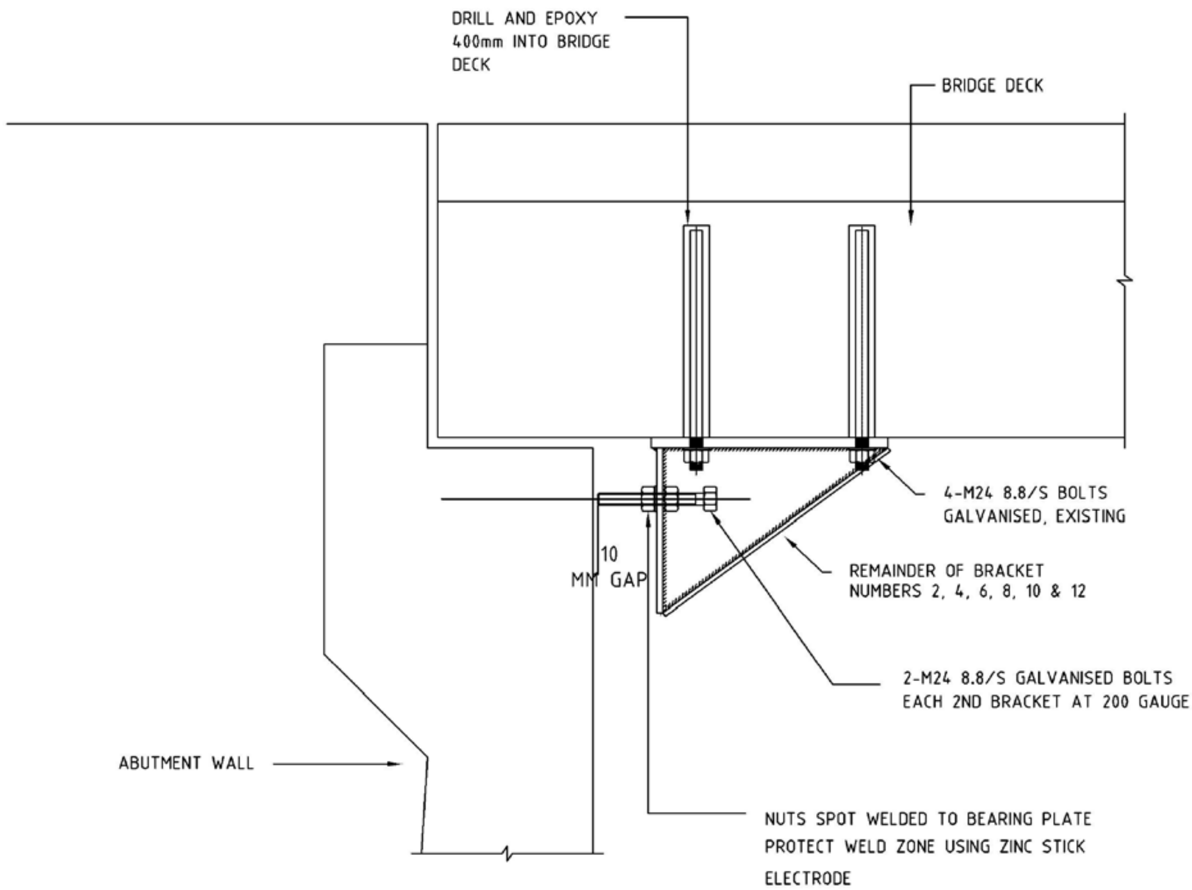


Fig. 4.7 Bracket Adjustment Detail (courtesy John Matheson & Associates)

Experience during the mining of Longwall 26

The development of subsidence and valley closure during the mining of Longwalls 25 and 26 is shown in Fig. 4.8. New cracks were identified by Sunrise Building Property Services (SBPS) for the first time on 23 September 2011 when the Longwall 26 face was approximately square with the Bridge. These consisted of vertical cracks on the south-eastern wingwall / abutment junction and a horizontal crack and concrete spalling at roughly 200 mm below the top of the abutment across the width of the abutment, above and behind the support brackets.

SBPS also noted at the time that the underside support brackets were hard against the abutment, when a gap had been observed during previous weekly inspections. SBPS removed the timber blocks between the brackets and abutments to restore a gap. It was found that the gap between the brackets and abutment closed on average 3mm following this work, with no change to the northern abutment.

SMPS also reported a slight bulging of the road pavement at the approach to the southern abutment, as was observed previously during the mining of Longwall 25.

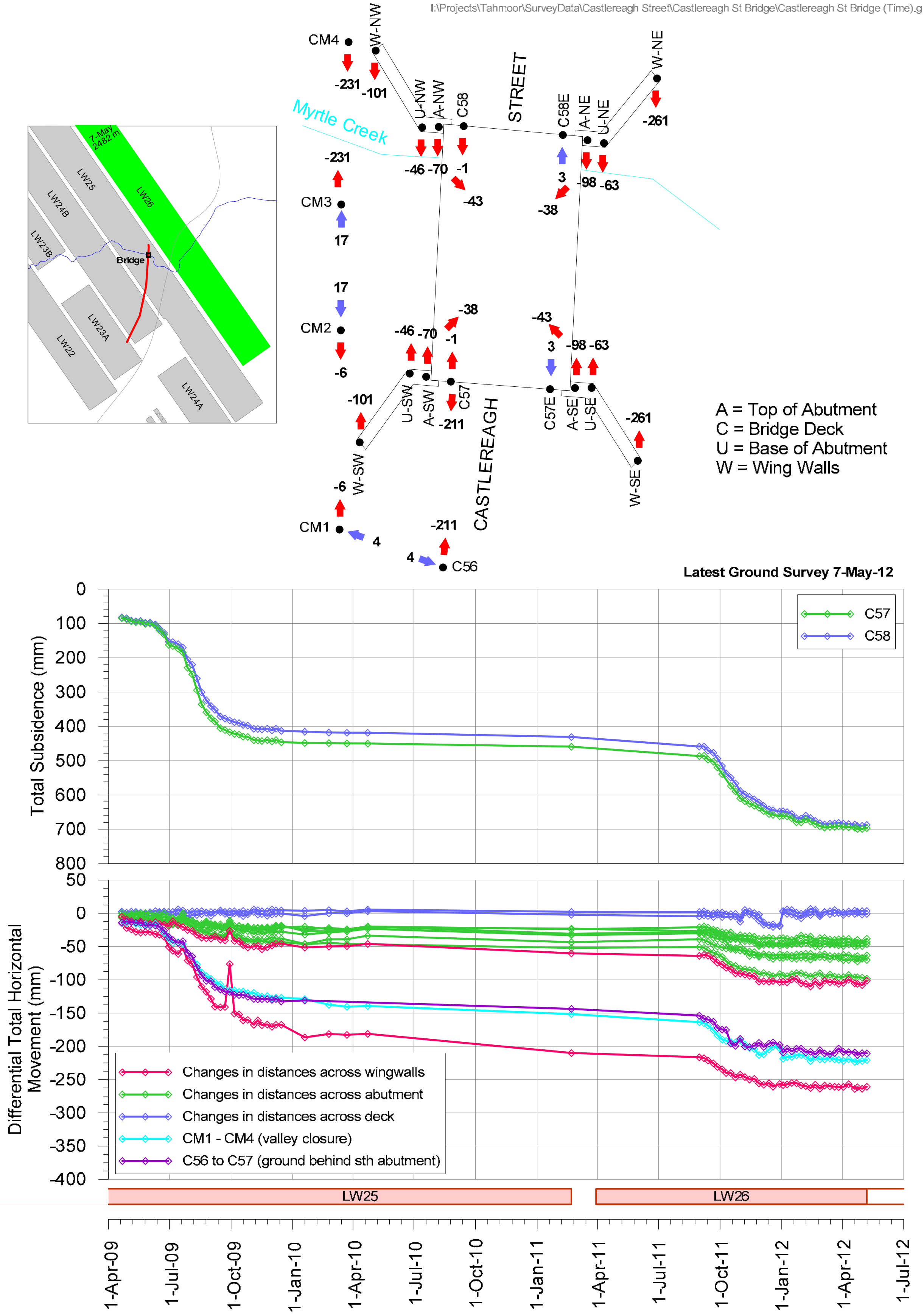


Fig. 4.8 Development of subsidence and valley closure at Castlereagh Street Bridge

Structural engineer, John Matheson & Associates (JMA) inspected photographs of the cracks on 26 September and a teleconference was held between Tahmoor Colliery and Wollondilly Shire Council on 27 September 2011. At the time of the teleconference it was anticipated that further valley closure movements would develop at the Bridge. It was decided at the meeting to undertake repair works in response to the impacts observed and make adjustments to the Bridge to minimise the potential for further impacts on the Bridge. A summary is provided below.

a) The eastern wing walls had closed significantly more than the abutments. This had resulted in flexural bending at the junction of the wingwall and abutment on the south eastern junction such that compressive spalling of concrete was observed.

   Excavations behind the back of the abutment and wingwall on 10 October 2011 uncovered a single tensile crack between the two walls, rather than a series of small tensile cracks as expected. The contractor reported that he could not find any horizontal steel reinforcement on the back face between the two walls for the first 300 mm in depth, contrary to structural design.

   As a result of this discovery, steel tie bars were installed from one abutment to the other to hold JMA Matheson. Soil nails were also installed behind the south eastern wingwall.

b) The horizontal crack across the width of the abutment indicated that the top of the abutment was suffering distress from the deck pressing hard against the abutment either via the small 200mm high and 200mm wide concrete upstand nib and/or the shearing and rotating of the steel dowels that were located between the deck and the top of the abutment, or both.

   It was found from an examination behind the spalled crack that the vertical steel reinforcement in the abutment did not appear to have been extended to the top of the abutment as per the structural design. The crack in the abutment occurs at the top of the steel bars and it is considered possible that the cracks had developed through a section of concrete that is mildly reinforced. Excavations in the week of 7 October 2011 found that the crack was inclined vertically and did not appear to have continued through to the back of the abutment.

   The concrete upstand nib was removed in accordance with recommendations by JMA. The contractor reported that the steel dowels appeared to be 32mm or 36 mm in diameter and not 28 mm, contrary to the structural drawings. This meant that the dowels had a much greater shear capacity than expected and explained why they did not appear to have sheared. The dowels were also located slightly closer to the front face of the abutment than shown in the structural drawings.

   The cutting of the dowels on 13 and 14 October had resulted in an expected change in trends in measurements of the gap between the support brackets and the abutment wall, as shown in Fig. 4.9. The deck was observed to slide further over the southern abutment in response to additional valley closure movements. There was an initial step change in measurements after the dowels were cut, after which the rate of change was gradual and correlated well with changes in valley closure movements.

   The gap between the underside brackets and the abutment reduced as the deck slid over the abutment. This gap has been partially restored by adjusting the brackets.

c) The observed cracking does not appear to be of structural concern at this stage, as the vertical loads continue to be transferred through the abutment. Bars were drilled from the back of the abutment to stitch the cracks, which were also cleaned out and filled with epoxy.

Management strategy during the mining of Longwall 27

The intention of the adjustment works was to allow the bridge deck to slide over the top of the southern abutment without impacting on the abutments or deck. This was observed during the mining of Longwall 26 after the steel dowels were cut and it is expected that the bridge deck will continue to slide over the southern abutment in response to additional valley closure movements during the mining of Longwall 27.

The steel brackets will be left in place as a precautionary measure and a gap will be maintained between the brackets and the abutment wall during the mining of Longwall 27.

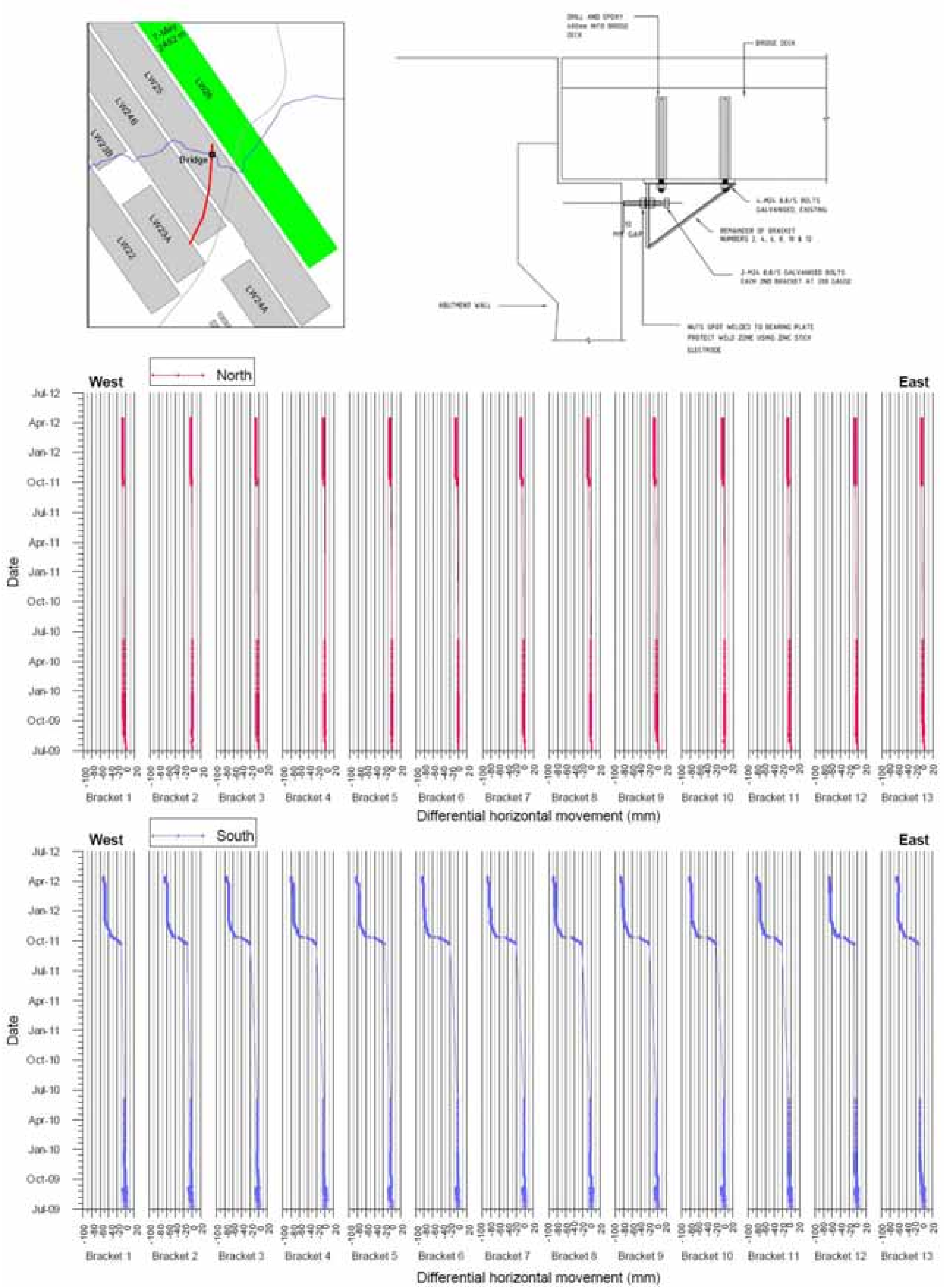


Fig. 4.9 Observed changes in gap between support brackets and abutment wall at Castlereagh Street Bridge

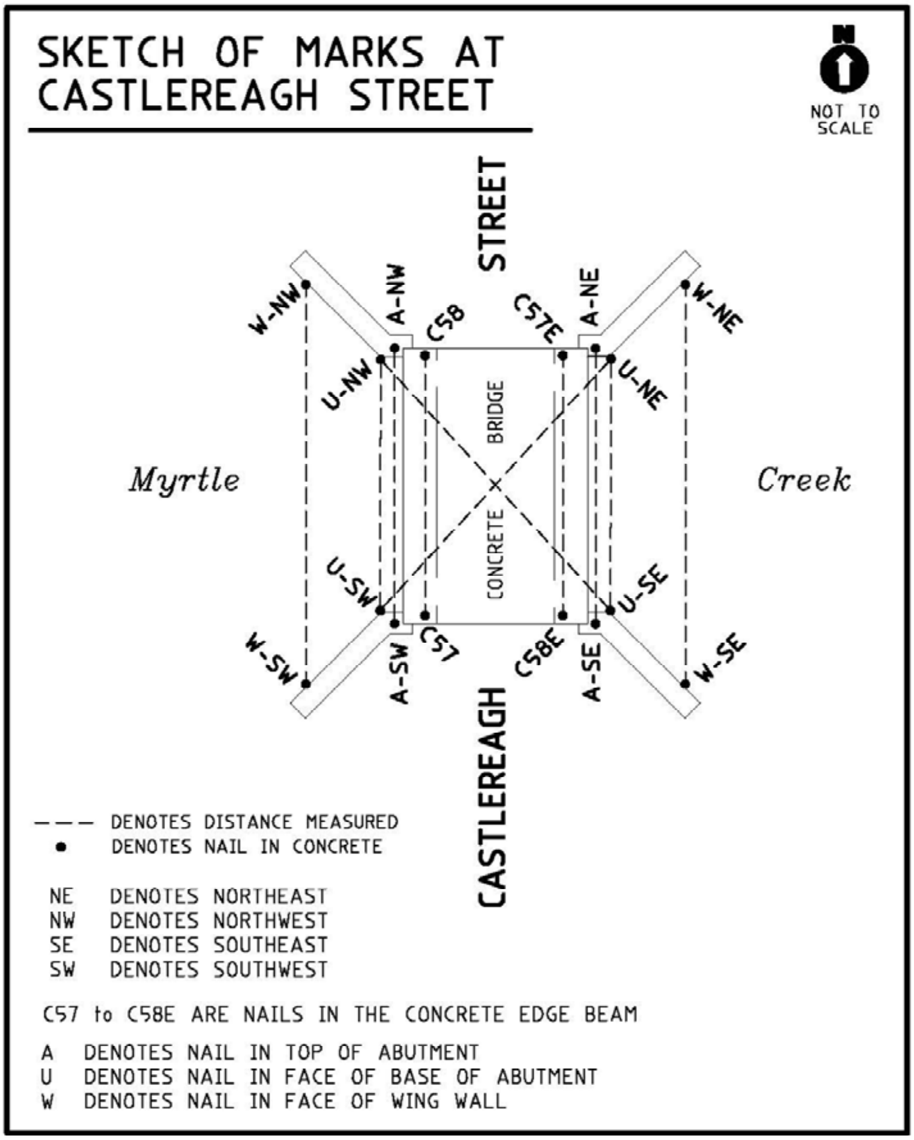
Risk Assessment for Longwall 27

Given the works undertaken at the Bridge during the mining of Longwall 26 and predicted small amount of additional valley closure, the likelihood of bridge damage and collapse due to the extraction of Longwall 27, is assessed as **RARE.** The consequence of this risk is assessed as **SEVERE.** The risk is therefore assessed as **RARE / SEVERE → MODERATE**.

The likelihood of the bridge experiencing additional damage and requiring additional repairs during the mining of Longwall 27 is assessed as **MODERATE.** The consequence is assessed as **MODERATE**. The risk is therefore assessed as **MODERATE / MODERATE → MODERATE**.

4.2.3. Monitoring measures

- Structure survey
  Survey marks have been placed on the top and bottom of the abutment walls, and at the bottom of the wing walls. Marks have also been placed on the bridge deck at each end. The survey marks were installed prior to the influence of Longwall 23A. A sketch of the monitoring mark locations is shown in Fig. 4.10.



Sketch courtesy of Lean & Hayward Surveyors

Fig. 4.10 Location of Survey Marks on Castlereagh Street Bridge

- Bridge Abutment Survey
  Surveys will be conducted to measure differential horizontal movement between the bridge abutment and bridge deck.
- Street survey along Castlereagh Street
  Survey marks have been placed along Castlereagh Street on either side of the Bridge. The street survey will provide general information on subsidence and overall valley closure movements.
- Survey across Myrtle Creek adjacent to Bridge
  Survey marks have been placed on the upstream side of Castlereagh Street Bridge to measure valley ground movements adjacent to the Bridge. Marks have also been placed on the bridge deck at each end. The survey marks were installed prior to the influence of Longwall 24B.
- Measurement of gap between support brackets and abutment walls
  The gap between the support brackets and the abutment walls will continue to be monitored during the mining of Longwall 27 and adjusted when necessary.
- Visual Inspections
  Visual inspections of the Bridge and approaches will be undertaken during mining and report any signs of impact.

4.2.4. Triggers and Responses

JMA (2008) has previously provided advice regarding response measures that can be implemented if triggered by monitoring results. A three stage trigger process was adopted in relation to the Bridge where the level of response increases for increasing trigger levels. The trigger levels relate to crack width.

Table 4.6 Trigger Levels for Castlereagh Street Bridge

Trigger Level	New Cracking to Abutment Walls	Triggered Response
GREEN	Cat 0 crack width <0.3mm	No response other than standard monitoring and inspection procedures
BLUE	Cat 1 single cracks 0.3 mm to 1.0mm spaced at approx. 500 mm centres	Consider the following measures: - increase monitoring frequency - structural inspection of Bridge - install crack gauges and measure growth
YELLOW	Cat 2 single cracks 1.0 mm to 5.0mm spaced at approx. 500 mm centres	Consider the following measures: - increase monitoring frequency - structural inspection of Bridge - install temporary props to strut the abutment walls to reduce earth pressure
ORANGE	Cat 3 single cracks of width > 5.0 mm spaced at approx. 500 mm centres	Consider the following measures: - increase monitoring frequency - structural inspection of Bridge - install temporary props to strut the abutment walls to reduce earth pressure - limit bridge traffic

Trigger levels for changes in distance between the abutments had previously been applied during the mining of Longwalls 25 and 26. As the dowels on the southern abutment have been cut off, these triggers no longer apply and they have been removed from the Management Plan for Longwall 27.

In the rare event of the Bridge becoming unserviceable, the Emergency Management Plan for Castlereagh Street Bridge (EMP, 2012) will be activated to provide access for residents on Hilton Park Road via an access road and level crossing on the Main Southern Railway and a private property on York Street.

4.3. Remembrance Drive Road Bridge over Myrtle Creek

The Remembrance Drive Road Bridge over Myrtle Creek is a two-lane bridge, which is located on the northern edge of the Tahmoor urban area. The bridge is located just beyond the commencing (southern) end of Longwall 29. The bridge is located approximately 500 metres to the side of Longwall 27.

The Roads and Traffic Authority have provided a copy of the structural design drawings, which show that the dual-span bridge is constructed with a concrete deck on concrete abutments and central pier, as shown in Fig. 4.11. The span of the deck is approximately 18 metres and the heights of the abutments are approximately 7 metres.

The bridge units have been integrated with a reinforced concrete slab. The reinforced concrete abutments appear to rest on pad and strip footing foundations. The pre-tensioned bridge deck units are connected to the central pier with dowels. The drawings do not include the abutment connections, but it appears that the bridge units rest on a corbel at each end. It is likely that a concrete upstand has been constructed at the ends of the deck.

Fig. 4.11 Remembrance Drive (Myrtle Creek) Road Bridge

The design of the bridge is not conducive to upsidence and closure movements because it is partly supported by a central pier. Upsidence may cause the central pier to move upwards, relative to the abutments. It is likely that the upstand at the ends of the bridge units will prevent the deck from sliding over the abutments as they close towards each other.

Predictions of systematic subsidence, tilt and strain movements have been made at the bridge, and these are shown in Table 4.4.

Table 4.7 Predicted Subsidence Parameters at Remembrance Drive Road Bridge and Pedestrian Bridge

Stage of Mining	Maximum Predicted Subsidence (mm)	Maximum Predicted Tilt (mm/m)	Maximum Predicted Tension (mm/m)	Maximum Predicted Compression (mm/m)
After Longwall 27	< 20	< 0.2	< 0.01	< 0.01
After Longwall 28	25	< 0.2	< 0.01	< 0.01
After Longwall 29	100	0.9	0.01	0.01
After Longwall 30	145	1.3	0.02	0.01

The Bridge will also be subjected to upsidence and closure movements, and these are shown in Table 4.5.

Table 4.8 Prediction of Upsidence and Closure at Castlereagh Street Road Bridge

Stage of Mining	Maximum Cumulative Closure (mm)	Maximum Cumulative Upsidence (mm)
Increment due to LW27 only	< 10	<10
Total due to LWs 22 to 27	< 10	15
Total due to LWs 22 to 28	20	25
Total due to LWs 22 to 29	40	80
Total due to LWs 22 to 30	55	125

It can be seen from Table 4.8 that very little valley closure and upsidence is predicted to occur during the mining of Longwall 27, as the Bridge is located approximately 500 metres from the side of Longwall 27.

Survey marks were installed on the Remembrance Drive Road Bridge prior to the extraction of Longwall 24A. While the Bridge has experienced approximately 25 mm of subsidence, measured changes in horizontal distances between the abutments are very small and close to survey tolerance. No closure has been detected and instead, a small opening has been measured. Vertical subsidence is relatively consistent across all survey marks, indicating that no measureable upsidence has occurred to date.

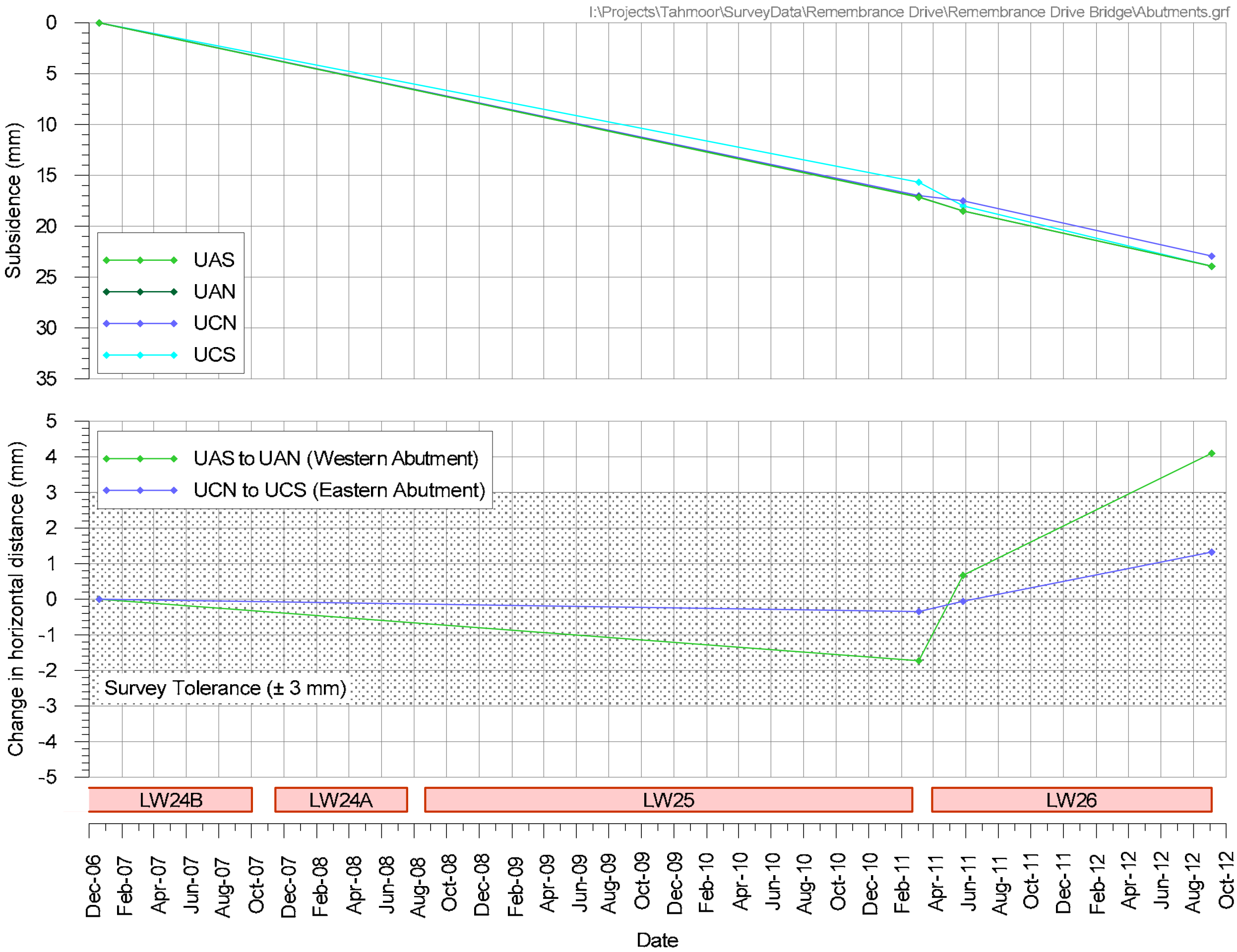


Fig. 4.12 Observed subsidence and changes in horizontal distances across the abutment of Remembrance Drive (Myrtle Creek) Road Bridge

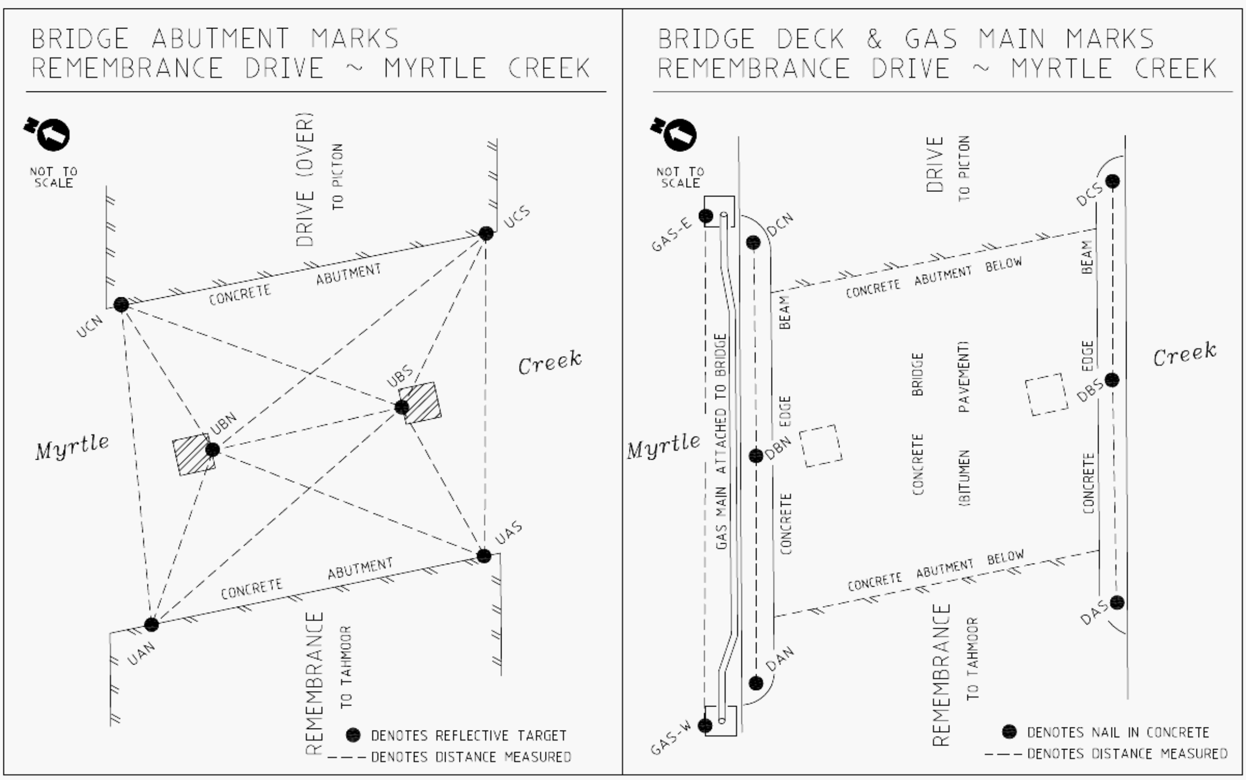
The Remembrance Drive survey line crosses Myrtle Creek between the Remembrance Drive Road Bridge and Pedestrian Bridge. Measured changes in horizontal distances between survey pegs within the Myrtle Creek valley are very small and within survey tolerance.

The bridge has been inspected by structural engineer John Matheson & Associates (JMA) who advises that mitigation measures can be designed, if required, to reduce the potential of impacts to the bridge (JMA, 2009). JMA recommends a structural analysis be conducted on the bridge to assess its ability to withstand differential ground movements and we concur with this recommendation. If mitigation measures are required, it is recommended that they be installed prior to the mining of Longwall 29.

Given the offset distance of the Bridge from Longwall 27 and the anticipated very small amount of movement that is expected to occur, the likelihood of bridge damage and collapse due to the extraction of Longwall 27, is assessed as **VERY RARE.** The consequence is assessed as **SEVERE**. The risk is therefore assessed as **VERY RARE / SEVERE → MODERATE**.

The likelihood of the bridge being damaged and requiring repairs during the mining of Longwall 27 is assessed as **VERY RARE.** The consequence of this risk is assessed as **MODERATE.** The risk is therefore assessed as **VERY RARE / MODERATE → LOW**.

The Bridge will be surveyed and visually inspected on a weekly basis upon commencement of Longwall 27. The surveys include monitoring of survey points on the Bridge, which were installed prior to the commencement of Longwall 24A. A map of survey points is shown in Fig. 4.13.



Sketch courtesy of SMEC (Urban)

Fig. 4.13 Survey marks on Remembrance Drive (Myrtle Creek) Road Bridge

This information will complement survey data of pegs that are located in the ground and pegs that are located on the Pedestrian Bridge.

4.4. Remembrance Drive Pedestrian Bridge over Myrtle Creek

The Remembrance Drive Pedestrian Bridge over Myrtle Creek is a single-lane bridge, which is located on the northern edge of the Tahmoor urban area. The bridge is located on the commencing (southern) end of Longwall 29. The bridge is located approximately 500 metres to the side of Longwall 27. The Bridge is listed as an item of environmental heritage in Wollondilly Shire Council's Local Environmental Plan.

The Roads and Traffic Authority have provided a copy of the structural design drawings for the renewal of the bridge in 1926. The pedestrian bridge was a dual lane bridge, at that point in time. Half of the bridge was demolished when the road bridge was constructed. The structural drawings show that the dual-span bridge was constructed with a concrete deck on sandstone abutments and central pier. The span of the deck is approximately 17 metres and the heights of the abutments are approximately 6 metres, and is shown in Fig. 4.11. The span of the deck is approximately 18 metres and the heights of the abutments are approximately 7 metres.

The bridge units have been integrated with a reinforced concrete slab. The reinforced concrete abutments appear to rest on pad and strip footing foundations. The pre-tensioned bridge deck units are connected to the central pier with dowels. The drawings do not include the abutment connections, but it appears that the bridge units rest on a corbel at each end. It is likely that a concrete upstand has been constructed at the ends of the deck.

Fig. 4.14 Remembrance Drive (Myrtle Creek) Pedestrian Bridge

The design of the bridge is not conducive to upsidence and closure movements because it is partly supported by a central pier. Upsidence may cause the central pier to move upwards, relative to the abutments. It is likely that the upstand at the ends of the bridge units will prevent the deck from sliding over the abutments as they close towards each other.

The bridge was inspected in May 2009 by structural engineer John Matheson & Associates (JMA, 2009) who advises that the bridge deck is in poor condition and the timber balustrade posts are severely dilapidated due to rot and/or termite damage.

Provided the deck and balustrade are restored to good condition, JMA advises that mitigation measures can be designed, if required, to reduce the potential of impacts to the bridge (JMA, 2009). These may include the provision of slip bearings and hydraulic jacks beneath each end of the bridge above both abutment walls. It is recommended that they be installed prior to the mining of Longwall 29. These works are unlikely to impact on the masonry abutments and pier of the heritage listed bridge.

Please refer to previous Section 4.3 for information on predictions and observations of subsidence movements.

Given the offset distance of the Bridge from Longwall 27 and the anticipated very small amount of movement that is expected to occur, the likelihood of bridge damage and collapse due to the extraction of Longwall 27, is assessed as **VERY RARE.** The consequence is assessed as **SEVERE**. The risk is therefore assessed as **VERY RARE / SEVERE → MODERATE**.

The likelihood of the bridge being damaged and requiring repairs due to subsidence during the mining of Longwall 27 is assessed as **VERY RARE.** It is noted, however, that the Bridge is currently in poor condition and currently requires repair. The consequence of this risk is assessed as **MODERATE**. The risk is therefore assessed as **VERY RARE / MODERATE → LOW**.

The Bridge will be surveyed and visually inspected on a weekly basis upon commencement of Longwall 27. Survey marks will be installed on the Bridge at each abutment and at the central pier. Vertical subsidence and changes in horizontal distance will be measured. This information will complement survey data of pegs that are located in the ground and pegs that are located on the Road Bridge.

4.5. Culverts

There are many culverts in the vicinity of Longwall 27, though only two minor culverts are directly above Longwall 27. The majority of the culverts are located above or near previously extracted Longwalls 22 to 26. Eleven (11) culverts have been identified that carry water from creeks and watercourses under local roads and railways, as shown in Drawing No. MSEC567-02-02.

There is a small concrete culvert over Myrtle Creek (Ref. C40), which provides vehicular access to Property Y58 from York Street, Tahmoor. The single lane low-level crossing consists of two 600 mm diameter pipes, which have been encased in concrete. The culvert is located above Longwall 26.

The remaining culverts are generally small in size, and typically range between 450 mm and 900 mm in diameter.

The risk of impacts to the culverts is considered low. No impacts to road culverts have been reported during the mining of Longwalls 22 to 26.

The hazard associated with culverts is that they could be damaged and/or rendered unserviceable from mine subsidence impacts.

The likelihood of extensive damage is assessed as **VERY RARE**. The consequence of this risk is assessed as **MODERATE**. The risk is therefore assessed as **VERY RARE / MODERATE → LOW.**

The likelihood of minor damage is assessed as **UNLIKELY**. The consequence of this risk is assessed as **SLIGHT**. The risk is therefore assessed as **UNLIKELY / SLIGHT → LOW.**

5.1. Structures Management Group (SMG)

The SMG is responsible for taking the necessary actions required to manage the risks that are identified from monitoring of structures. The SMG's key members are:

- Tahmoor Colliery
- Wollondilly Shire Council
- John Matheson and Associates
- Mine Subsidence Engineering Consultants
- Mine Subsidence Board

5.2. Mitigation Measures

Mitigation measures have been undertaken for Castlereagh Street Bridge, as described in Section 4.2.

5.3. Monitoring Measures

Monitoring lines have been installed along all streets within the urban area above Longwall 27, as shown in Drawing No. MSEC567-00-03. The monitoring lines have been initially surveyed to provide a baseline reference. Monitoring of street survey lines will be conducted for every 200 metres of longwall travel as a minimum for pegs located within the active subsidence zone.

Additional surveys will be conducted for the Castlereagh Street Bridge and Remembrance Drive Road Bridge and Pedestrian Bridge, as described in Table 5.1.

5.4. Emergency Management Plans for Bridges

In the rare event of the Bridge becoming unserviceable, the Emergency Management Plan for Castlereagh Street Bridge (EMP, 2012) will be activated to provide access for residents on Hilton Park Road via an access road and level crossing on the Main Southern Railway and a private property on York Street.

Given that the Remembrance Drive Road and Pedestrian Bridges are located more than 500 metres from Longwall 27, the likelihood of the Bridges becoming unserviceable is considered barely credible. It is recognised, however, that an Emergency Management Plan for these Bridges will be developed during the mining of Longwall 27, in preparation for finalisation prior to the commencement of Longwall 28.

5.5. Risk Control Procedures

Risk control procedures are provided in Table 5.1. The procedures include responses if triggered by monitoring results.

In relation to triggers associated with the Castlereagh Street Bridge, please refer to Section 4.2.4.

Table 5.1 Risk Control Procedures

Infrastructure	Hazard / Impact	Risk	Trigger	Control Procedure/s	Frequency	By Whom?
Local Roads	Impacts to Roads Impacts to Culverts	**MODERATE**	None	Conduct visual inspection for surface deformations along local roads.	Detailed inspection once a week Vehicle based inspection once a week within active subsidence area	Tahmoor Colliery
				Conduct surveys along survey lines to provide some early warning for potentially damaging subsidence events	Every 200 metres of longwall face movement	Tahmoor Colliery (SMEC Urban / MSEC)
			Impacts occur	Notify all stakeholders, including Council, Tahmoor Colliery, Mine Subsidence Board and DTIRIS	Within 24 hours	Tahmoor Colliery / WSC
				Repair road	As required	WSC
Remembrance Drive Road Bridge and Pedestrian Bridge	Impacts to Bridge(s)	**MODERATE / LOW**	None	Conduct surveys of Remembrance Drive Road Bridge and Pedestrian Bridge, and survey of ground pegs located in the valley sides between the two bridges	Marks installed on Road Bridge Install marks on Pedestrian Bridge prior to start of LW27 Survey weekly commencing after start of LW End of LW27	Tahmoor Colliery (SMEC Urban / MSEC)
				Visual inspections of Remembrance Drive Road Bridge and Pedestrian Bridge	Weekly commencing after start of LW	Tahmoor Colliery
			Impacts occur	Notify all stakeholders, including Council, Tahmoor Colliery, Mine Subsidence Board and DTIRIS and in the case of the Pedestrian Bridge, the Heritage Council	Within 24 hours	Tahmoor Colliery / WSC
				Repair Bridge	As required	WSC

Infrastructure	Hazard / Impact	Risk	Trigger	Control Procedure/s	Frequency	By Whom?
Bridge on Castlereagh Street over Myrtle Creek	Impacts on Bridge	**MODERATE**	**GREEN** (None)	Install mitigation measures	Complete	Tahmoor Colliery
				Conduct surveys at Bridge, including: • Structure surveys • Castlereagh Street ground survey (all pegs within 80 metres of Bridge – Pegs C54 to C62) • Myrtle Creek cross line adjacent to Castlereagh Street Bridge)	Weekly within active subsidence zone until agreed to reduce End of LW27	Tahmoor Colliery (SMEC Urban)
				Conduct visual inspections of Bridge	Weekly within active subsidence zone until agreed to reduce End of LW27	Tahmoor Colliery
				Adjust brackets if they are observed to press against the abutment wall	As required	Tahmoor Colliery
				Assess monitoring results and report	Weekly when bridge is within active subsidence zone until agreed to reduce	Tahmoor Colliery (MSEC)
			BLUE (refer Table 4.6)	Convene meeting of SMG to consider additional monitoring and mitigation measures based on observed monitoring results, which may include: • Increase monitoring and inspection frequency • Structural inspection of Bridge • Repair cracks • Install crack gauges and measure growth • Adjust position or remove galvanised steel brackets	Within 48 hours	SMG
			YELLOW (refer Table 4.6)	Convene meeting of SMG to consider additional monitoring and mitigation measures based on observed monitoring results, which may include: • Increase monitoring and inspection frequency • Structural inspection of Bridge • Repair cracks • Install temporary props to strut the abutment walls to reduce earth pressure	Within 48 hours	SMG
			ORANGE (refer Table 4.6)	Convene meeting of SMG to consider additional monitoring and mitigation measures based on observed monitoring results, which may include: • Increase monitoring and inspection frequency • Structural inspection of Bridge • Repair cracks • Install temporary props to strut the abutment walls to reduce earth pressure • Limit bridge traffic	Within 24 hours	SMG

6.0 MANAGEMENT PLAN REVIEW MEETINGS

The monitoring of natural surface features and surface infrastructure which forms an integral part of this Management Plan will be carried out by Tahmoor Colliery. SMG Meetings will be held between Tahmoor Colliery and Wollondilly Shire Council for discussion and resolution of issues raised in the operation of the Management Plan. The frequency of meetings shall be as agreed by the parties.

A secretary will be appointed at the SMG Meeting. All documentation, distribution of meeting minutes and organising of meeting times will be undertaken by the secretary.

SMG Meetings will discuss any incidents reported in relation to the relevant surface feature, the progress of mining, the degree of mine subsidence that has occurred, and comparisons between observed and predicted ground movements.

It will be the responsibility of the meeting representatives to determine whether the incidents reported are due to the impacts of mine subsidence, and what action will be taken in response.

In the event that a significant risk is identified for a particular surface feature, any party may call an emergency SMG Meeting, with one day's notice, to discuss proposed actions and to keep other parties informed of developments in the monitoring of the surface feature.

7.0 AUDIT AND REVIEW

All Management Plans within this document have been agreed between parties. The Management Plan will be reviewed following extraction of each longwall.

Should an audit of the Management Plan be required during that period, an auditor shall be appointed by the Tahmoor Colliery to review the operation of the Management Plan and report at the next scheduled Plan Review Meeting.

Other factors that may require a review of the Management Plan are:-

- Observation of greater impacts on surface features due to mine subsidence than was previously expected.
- Observation of fewer impacts or no impacts on surface features due to mine subsidence than was previously expected.
- Observation of significant variation between observed and predicted subsidence.

8.0 RECORD KEEPING

The secretary will keep and distribute regular minutes of each Plan Review Meeting for each surface feature. The minutes will include reports on the condition of the relevant surface feature, the progress of mining, the degree of mine subsidence that has occurred, comparisons between observed and predicted ground movements, agreements reached between parties, and a log of incidents that have occurred on the surface feature.

9.0 CONTACT LIST

Organisation	Contact	Phone	Email / Mail	Fax
NSW Department of Trade and Investment, Regional Infrastructure and Services, Division of Resources and Energy (DTIRIS)	Phil Steuart	(02) 4931 6648	phil.steuart@industry.nsw.gov.au	(02) 4931 6790
	Gang Li	(02) 4931 6644 0409 227 986	gang.li@industry.nsw.gov.au	(02) 4931 6790
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John Matheson & Associates (JMA)	John Matheson	(02) 9979 6618	jma.eng@bigpond.net.au	(02) 9999 0121
Mine Subsidence Board	Darren Bullock	(02) 4677 1967	d.bullock@minesub.nsw.gov.au	(02) 4677 2040
Mine Subsidence Engineering Consultants (MSEC)	Daryl Kay	(02) 9413 3777	daryl@minesubsidence.com	(02) 9413 3822
Xstrata Coal Tahmoor Colliery – Environment and Community Manager	Ian Sheppard	(02) 4640 0156 0408 444 257	isheppard@xstratacoal.com.au	(02) 4640 0140
Xstrata Coal Tahmoor Colliery – Community Coordinator	Belinda Clayton	(02) 4640 0133	bclayton@xstratacoal.com.au	(02) 4640 0140
Wollondilly Shire Council Manager - Works	Justin Nyholm	(02) 4677 8247	justin.nyholm@wollondilly.nsw.gov.au	(02) 4677 2339
Wollondilly Shire Council Technical Officer - Works	Barry Allen	(02) 4677 8232	barry.allen@wollondilly.nsw.gov.au	(02) 4677 2339

APPENDIX A.

Please refer to the following documents:

- Drawings
- JMA, (2009). *Myrtle Creek Bridges on Remembrance Drive: Condition Report.* John Matheson & Associates, Report No. R0116-Rev 02, May 2009.
- JMA (2011). Castlereagh Street Bridge Structural Investigation Report. Report No. R0171, John Matheson & Associates, 27 September 2011.
- EMP (2012). Emergency Management Plan – Castlereagh Street Bridge. Tahmoor Colliery, 2012.

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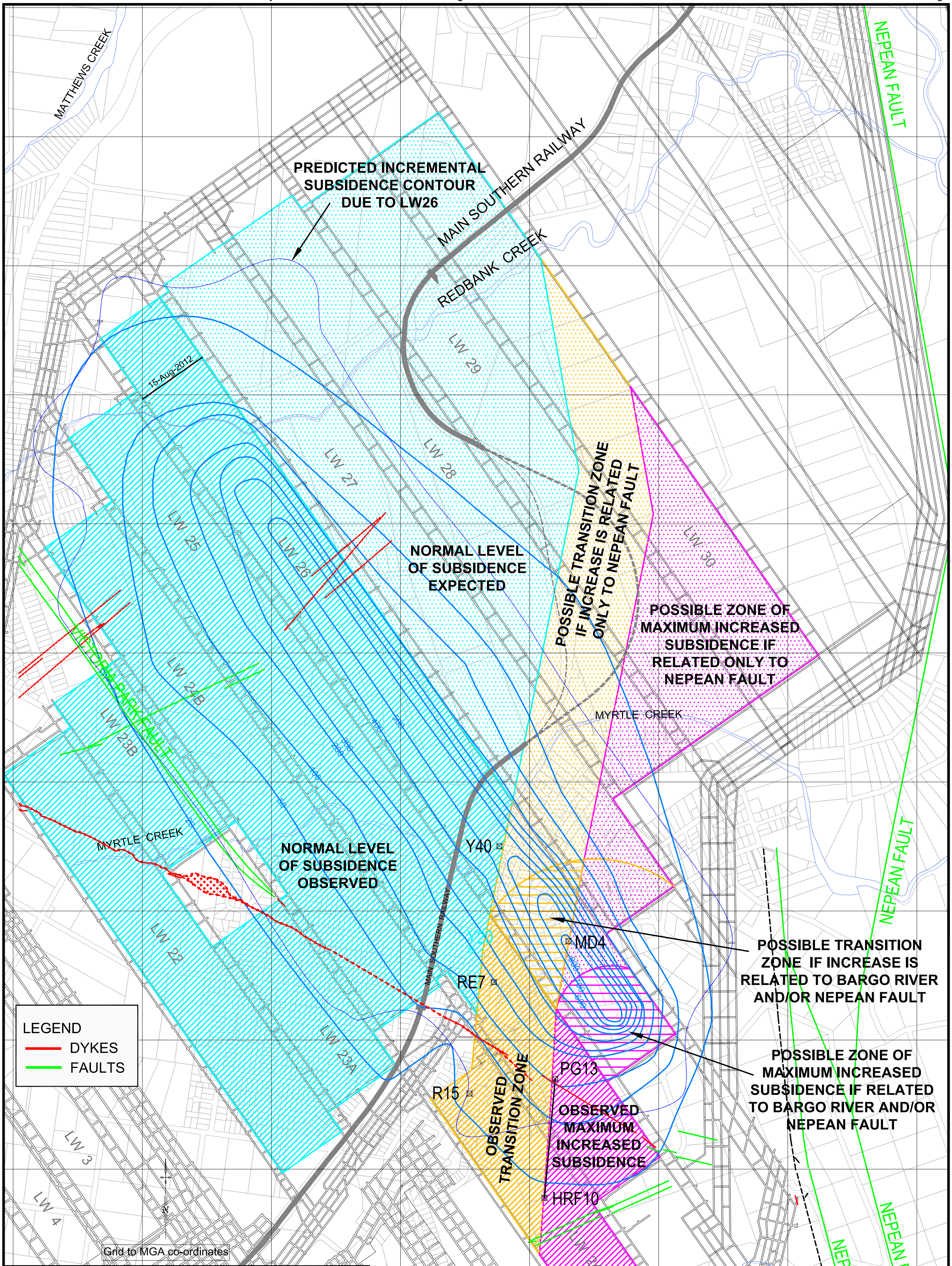

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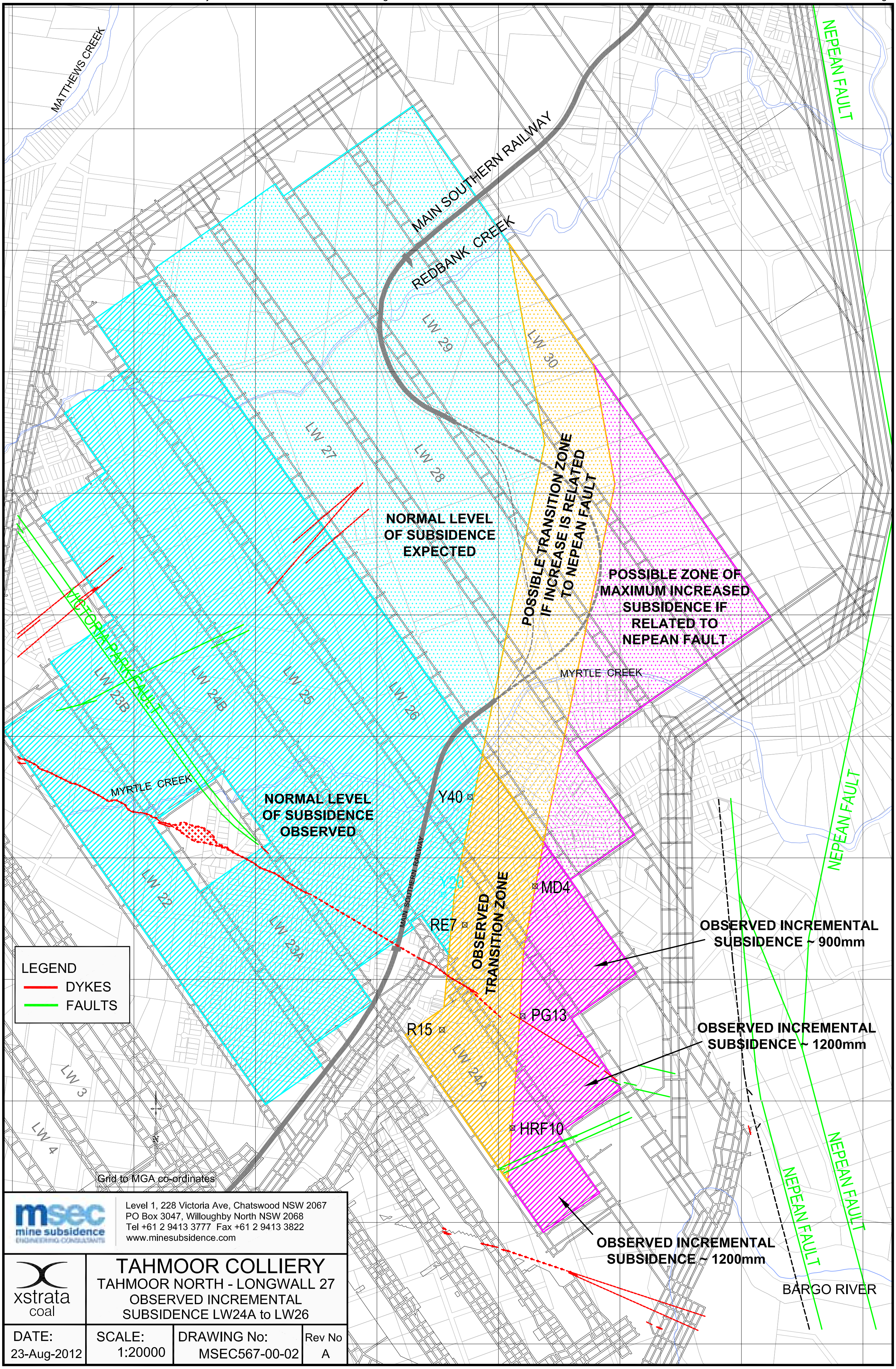
xstrata coal

TAHMOOR COLLIERY

TAHMOOR NORTH - LONGWALL 27
OBSERVED INCREMENTAL SUBSIDENCE
DUE TO LW26 AS AT 15-AUG-2012

DATE:	SCALE:	DRAWING No:	Rev No
23-Aug-2012	1:20000	MSEC567-00-01	A



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TAHMOOR COLLIERY
TAHMOOR NORTH - LONGWALL 27
OBSERVED INCREMENTAL
SUBSIDENCE LW24A to LW26

DATE:	SCALE:	DRAWING No:	Rev No
23-Aug-2012	1:20000	MSEC567-00-02	A

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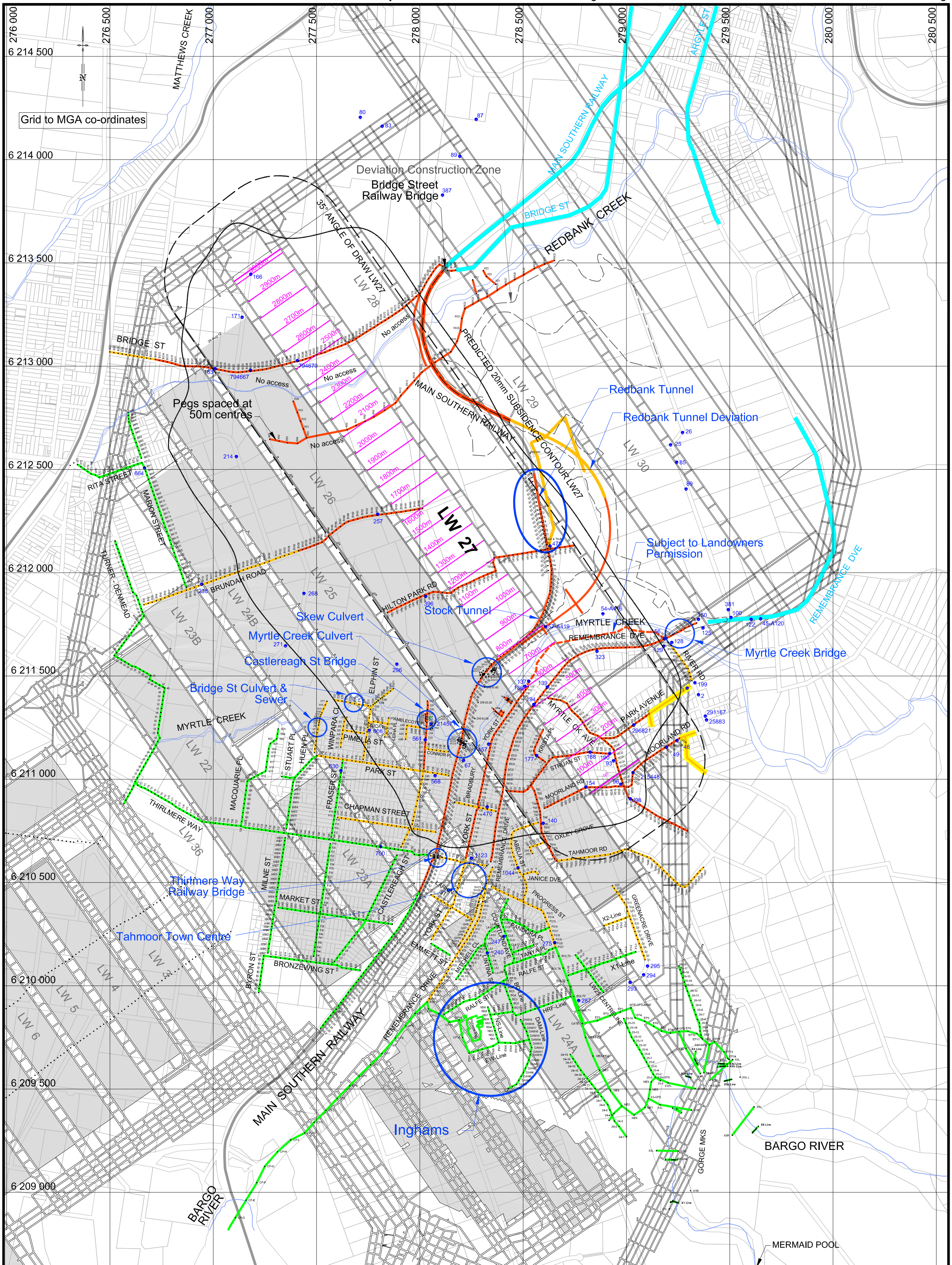

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TAHMOOR COLLIERY

TAHMOOR NORTH - LONGWALL 27
MONITORING OVER LW27

DATE:	SCALE:	DRAWING No:	Rev No
13-Sep-2012	1:25000	MSEC567-00-03	A

LEGEND

- Existing Monitoring Lines
- Future Monitoring Lines
- Future Monitoring Lines Prior to LW27
- Monitoring Lines Before & End of LW27
- Surveys During LW27
- Critical Power Poles
- Specific Structure Inspections

Refer to Management Plans for Timing & Frequencies

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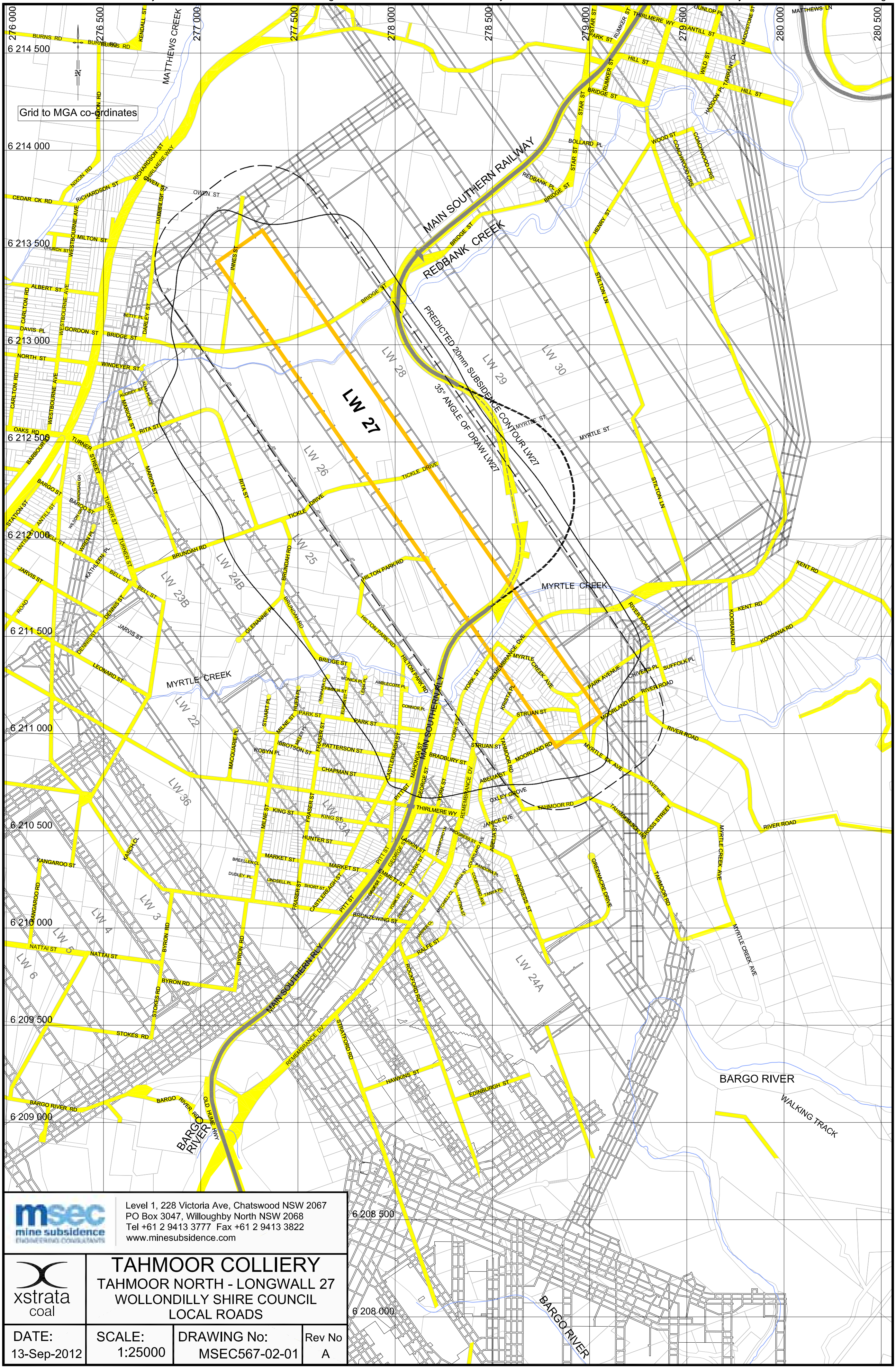

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TAHMOOR COLLIERY
TAHMOOR NORTH - LONGWALL 27
WOLLONDILLY SHIRE COUNCIL
LOCAL ROADS

DATE:	SCALE:	DRAWING No:	Rev No
13-Sep-2012	1:25000	MSEC567-02-01	A

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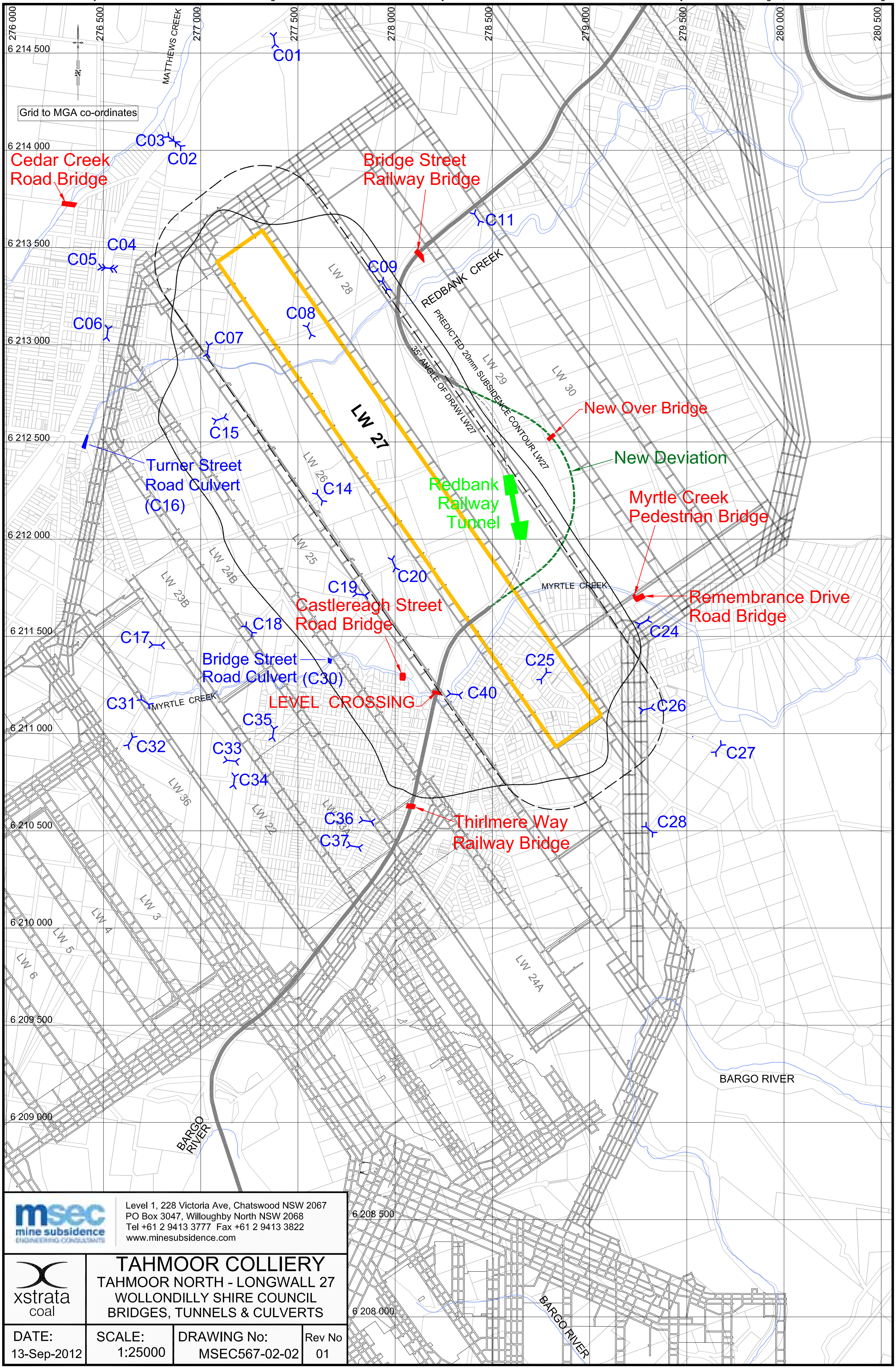

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TAHMOOR COLLIERY
TAHMOOR NORTH - LONGWALL 27
WOLLONDILLY SHIRE COUNCIL
BRIDGES, TUNNELS & CULVERTS

DATE:	SCALE:	DRAWING No:	Rev No
13-Sep-2012	1:25000	MSEC567-02-02	01

Myrtle Creek Bridges on Remembrance Drive: Condition Report.

Report: R0116-Rev 02

Prepared by John Matheson

Date: 18th May 2009.

John Matheson & Associates Pty Ltd

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Introduction

This structural condition report has been prepared at the request of MSEC concerning the two bridges that cross Myrtle Creek on Remembrance Drive, which are located to the north of Tahmoor. The original bridge crossing served as a vehicular bridge and in more recent times as a pedestrian bridge after a new vehicular bridge was constructed in the early 1970's. The bridges have not been assigned any particular MSEC number to identify them and they are known as the Remembrance Drive Pedestrian Bridge and the Remembrance drive Road Bridge respectively.

1.0 Remembrance Drive Pedestrian Bridge Structure

The abutments and central pier have been constructed using sandstone masonry construction with a sand/cement mortar as can be seen in figures 1 and 5. The masonry abutments walls have been constructed with wingwalls returning back into the embankment and the existing drawings indicate that the foundations have been constructed below the riverbed and are likely to have been founded upon rock. The masonry abutment walls and central pier appeared to be in fair condition at the time of the inspection.

The bridge deck consists of steel rolled steel joists spanning between both abutment walls and the central masonry pier. The beams do not appear to haven joined above the central masonry pier. The bridge deck has been constructed as a tar macadam surface over steel buckle plates spanning between double angle secondary beams and the primary rolled steel joists. The drawings show transverse steel reinforcing rods were placed within the tar macadam to enable the tar macadam to arch between the rolled steel joists in conjunction with the steel buckle plates.

1.1 Remembrance Drive Pedestrian Bridge Structural Assessment

The pedestrian bridge is no longer in use and barrier s have been erected across the both ends of the bridge to prevent pedestrian access as the bridge deck is in poor condition and the timber balustrade posts are severely dilapidated due to rot and/or termite damage and the dilapidated state is clearly visible in figures 2, 3, 6 & 7 in particular. The steel buckle plates show evidence of general corrosion; however, it was not possible to carry out an inspection of the tie rods that were embedded within the tar macadam. The edges of the tar macadam pavement have eroded and fretted due to exposure to wind and rain as can be seen in figures 2, 3 & 6.

Non systematic valley closure and upsidence are likely to occur as a result of subsidence associated with the extraction of coal from longwalls LW 27-LW30. The action of valley closure will cause each abutment wall to move inwards toward the centre of the valley as a rigid body whereas valley upsidence is likely to cause the abutment walls to tilt backwards away from the valley about the base of the wall as a rigid body. The upsidence will also cause the central pier to rise relative to the abutment walls.

Generally speaking, the valley closure will cause compression of the bridge deck unless it is released from the ground in the form of directional sliding bearings. The degree to which upsidence and the associated back slope of the abutment wall will offset valley closure at bridge deck level is uncertain and therefore it is recommended that the bridge deck be allowed to slide relative to the valley sides in order to avoid serious damage to the bridge deck, the abutment walls and the central pier. The ends of the bridge above the abutment walls can be jacked to maintain the planar surface of the bridge deck to reduce the effects of curvature on the bridge deck. It is recommended that the dilapidated timber balustrade posts and rails be removed from the bridge deck to prevent them becoming unstable during the proposed subsidence period and access to the bridge deck be prohibited.

Based upon a methodology of providing directional slip bearings and hydraulic jacks beneath each end of the bridge above both abutment walls the bridge is expected to remain in equilibrium during the proposed subsidence period. The bridge could be rehabilitated after the proposed subsidence where a new deck and balustrade could be constructed upon the existing beams or an entirely new bridge could be erected upon the existing abutment walls and pier after conclusion of the active subsidence period.

It is expected that intervention measures, if required, can be designed to protect the abutment walls, piers and the bridge deck from the most damaging impacts of valley closure and upsidence and that, furthermore, that trigger levels can be established for crack width and structure tilt to maintain the safety and serviceability of the bridge structure and limit the impact of mine subsidence.

1.2 Remembrance Drive Pedestrian Bridge Photographs

Figure 1

Figure 2

Figure 3

Figure 4

Figure 5

Figure 6

Figure 7

2.1 Remembrance Drive Road Bridge Structure

The abutment walls and central piers and headstock have been constructed using reinforced concrete construction based upon a review of the work as executed drawings. The bridge deck has been constructed using simply supported pre-cast pre-tensioned bridge deck units with infill concrete. The drawings show both bridge spans are supported upon elastomeric bearings upon the central headstock and are connected to the central headstock by grouted dowels. A vertical bitumastic strip forms a vertical separation between both bridge spans above the central headstock.

The bridge deck appears to be supported partly upon a reinforced concrete corbel that has been constructed monolithically with the abutment wall and partly upon the abutment wall itself as the barrier post in each corner of the bridge appears to have been constructed upon the bridge deck, refer figure 14. The road surface has cracked behind the abutment wall behind the barrier posts (figure 12), which supports the supposition that the bridge deck continues onto the wall proper. The structural drawings do not show whether or not the deck has been dowelled to both abutment walls. However, experience at the Castlereagh Street Bridge suggests that this is likely.

The drawings call for footings to be founded 450mm into solid rock including the abutment/wing walls and the footings to the concrete piers. The abutment walls do not appear to rely upon cantilever action other than what may be derived from the self-weight of the abutment walls.

2.2 Remembrance Drive Road Bridge Structural Assessment

The vehicular bridge is expected to be impacted by non-systematic valley closure and upsidence as a result of subsidence associated with the extraction of coal from longwalls LW 27-LW30. The action of valley closure will cause the abutment walls to move inwards toward the centre of the valley as a rigid body whereas valley upsidence is likely to cause the abutment walls to tilt backwards away from the valley about the base of the wall as a rigid body. There is some uncertainty as to the extent that closure and upsidence will coexist during subsidence and therefore the bridge structure will require detailed analysis as part of the SSSMP.

Generally speaking, the valley closure will cause compression of the bridge deck since it is unlikely that the bridge deck can be isolated from the abutment walls due to the likelihood that the deck is connected to the abutment wall by grouted dowels. Valley upsidence is expected to relieve the effects of valley closure due to possible back tilt of the abutment walls away from the valley centre as appears to be occurring at the Castlereagh Street Bridge currently being impacted by the subsidence effects of LW25 but the central piers and headstock are expected to rise relative to the abutment walls.

If the bridge deck cannot be isolated from the abutment walls it is possible that earth pressures near the top of the abutment wall may approach passive pressure levels and this may tend to generate significant tensile stresses in the vertical reinforcement near the external face of the wall. This is expected to cause horizontal tensile cracking in the abutment wall unless intervention measures are undertaken to remove the backfill from behind the retaining wall and install a temporary steel back span above the excavated backfill on one or both sides of the bridge to disengage the bridge structure from the effects of compressive ground strain upon the backfill caused by valley closure.

A detailed structural analysis of the abutment walls and the bridge deck should be carried out to assess the structural impacts of valley closure and upsidence upon the structure to determine the extent of any intervention measures that may be required.

It is expected that intervention measures, if required, can be designed to protect the abutment walls, piers and headstocks and the bridge deck from the most damaging impacts of valley closure and upsidence and that, furthermore, that trigger levels can be established for crack width and structure tilt to maintain the safety and serviceability of the bridge structure and limit the impact of mine subsidence.

2.3 Remembrance Drive Road Bridge Photographs

Figure 8

Figure 9

Figure 10 Gas pipeline emerging from the ground next to the southwest corner of bridge

Figure 11 Gas pipe support along bridge.

Figure 12

Figure 13

18th May 2009.

Figure 14

Yours faithfully,

John Matheson & Associates Pty Ltd

John Matheson

John Matheson & Associates Pty Ltd

Castlereagh Street Bridge

Structural Inspection Report: R0171

John Matheson & Associates Pty Ltd
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Email: jme.eng@bigpond.net.au
27/9/2011

DOCUMENT HISTORY			
Revision	Date	Amendments	Author
	27.09.2011	Issued to Tahmoor Colliery	JM

TABLE OF CONTENTS

INTRODUCTION

The photographs taken by John Schwarz on Friday 23rd September show that some new cracking has occurred and existing cracking has re-activated with subsidence impacts now being experienced at the Castlereagh Street Bridge.

A number of photographs were taken of the abutment wall between the steel brackets, which appear to show a near horizontal crack running along the top of the abutment wall approximately 200mm below the level of the bridge bearings but above the level of the steel brackets. Coincidentally, the steel dowels that tie the deck to the abutment wall terminate at approximately this depth below the abutment corbel.

The longwall 25-subsidence records show that the measured closure between the top of the abutment walls was substantially less than that measured between the ends of the wing walls. This indicates that the joint between the abutment and wing walls appeared to be opening in plan at the rear face (the obtuse angle formed by the intersection of the wing wall and the abutment wall appears to have opened). The vertical cracks that can be observed on the external face of the wall near the junction of the southeastern wing wall and the southern abutment wall appear to be spalling cracks.

The cracks shown in figure 1 were taken of the top of the southeastern wing wall where the wing wall joins the main abutment wall. Figure 2 shows the designated concrete section through the abutment. These cracks appear to reflect a continuation of the joint opening at the rear of the abutment/wing wall with the wing wall above corbel level.

The drawings show that the front face of the abutment should be reinforced with C20 at 100mm centres rising vertically in the wall, which are splice with C16 at 300mm centres at the top of the wall and the horizontal wall reinforcement should be C16 at 300mm centres. The original reinforcement layout and the proposed concrete repairs are shown in a separate drawing SK1-2011-09-27.

It is apparent from the photographs that the bridge deck has been resisting continues to resist the inwards movement of the abutment walls toward the creek with valley closure. The horizontal reaction that has been developed in the bridge deck has generated an equal and opposite horizontal shear force at the top of the abutment wall. This shear force is being resisted by the concrete nib behind the bridge deck and the steel dowels. However, the dowels may be deforming and causing the embedded portion of the dowel to rotate outwards. The outwards rotation of the dowels in conjunction with the shear force may be contributing to the development of a shear-flexure crack near the top of the abutment.

The original calculation of the shear capacity of the abutment wall where the bridge deck is supported was based on the original engineering drawings and the section considered being of least shear capacity. The actual construction process may have resulted in a significantly greater nib section than documented and this may be the cause of the shear crack appearing in the face of the abutment wall rather than through the nib as anticipated.

The designated wall reinforcement should maintain ductile behaviour up to a point. However, a 10mm horizontal stepped crack is significant given the amount of wall reinforcement shown on the drawings and it should be investigated further. Figure 4, shows the location where the 10mm crack was measured and it appears that the 10mm step in this location may be the result of spalling near the junction with the east wing wall. The crack width is of lesser magnitude elsewhere with 3mm horizontal step being recorded in figure 5.

The photographs indicate that horizontal crack has occurred generally above the level of the bracket restraints. The bracket restraints were installed to provide an alternative horizontal load path for the restraint of the top of the abutment walls if the shear strength of the rear corbel behind the bridge deck was exceeded by valley closure effects. Therefore, if a sudden release of energy occurs at the top of the abutment wall then the steel brackets remain in position to provide lateral support to the abutment wall.

Additional intervention measures are proposed to ease the restraint force that has developed in the bridge deck. These measures include the following:

i. Provide temporary traffic lights and speed control signs to manage the local traffic.

ii. Provide New Jersey Barriers to delineate traffic and work areas.

iii. Excavate a works access trench behind the southern abutment.

iv. Provide steel cover plates over the access trench for the full width of the bridge.

v. Inspect the structure as the trench is excavated.

vi. Reduce travel between the abutment wall and bracket restraints prior to demolition of concrete nib.

vii. Carefully demolish the concrete nib behind the bridge deck to release load from bridge deck.

viii. Sever nib reinforcement projecting above bridge corbel level.

ix. Epoxy inject the crack from the front face of the abutment wall from a scaffold already in place.

x. Drill and epoxy crack stitching bars across the crack.

xi. Provide sand: cement skim coat over the remnant nib surface to finish flush with the corbel level.

The intention of the proposed works is to release the compression load in the bridge deck and the corresponding shear force at the top of the abutment wall whilst maintaining safety and serviceability of the abutment wall structure using the steel brackets to control large displacements. The crack repairs are intended to restrict further crack growth at this location.

Yours faithfully
John Matheson & Associates Pty Ltd

John Matheson BE (HON II) MIEAust CPEng
Director

Report Number: R0171
Tuesday, 27 September 2011

APPENDIX A

FIGURE 1

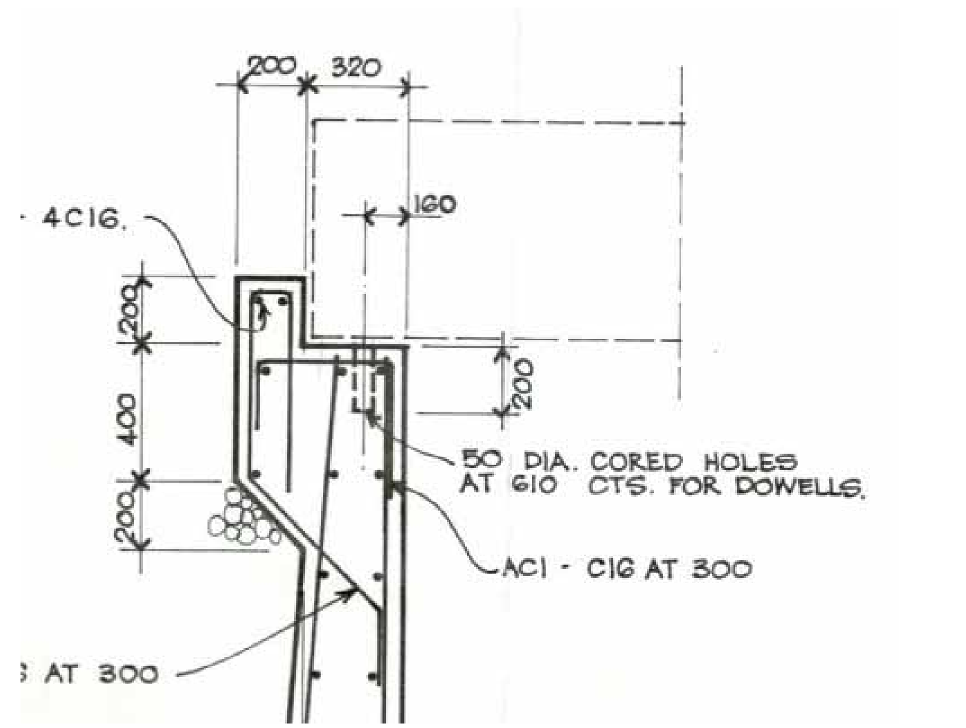
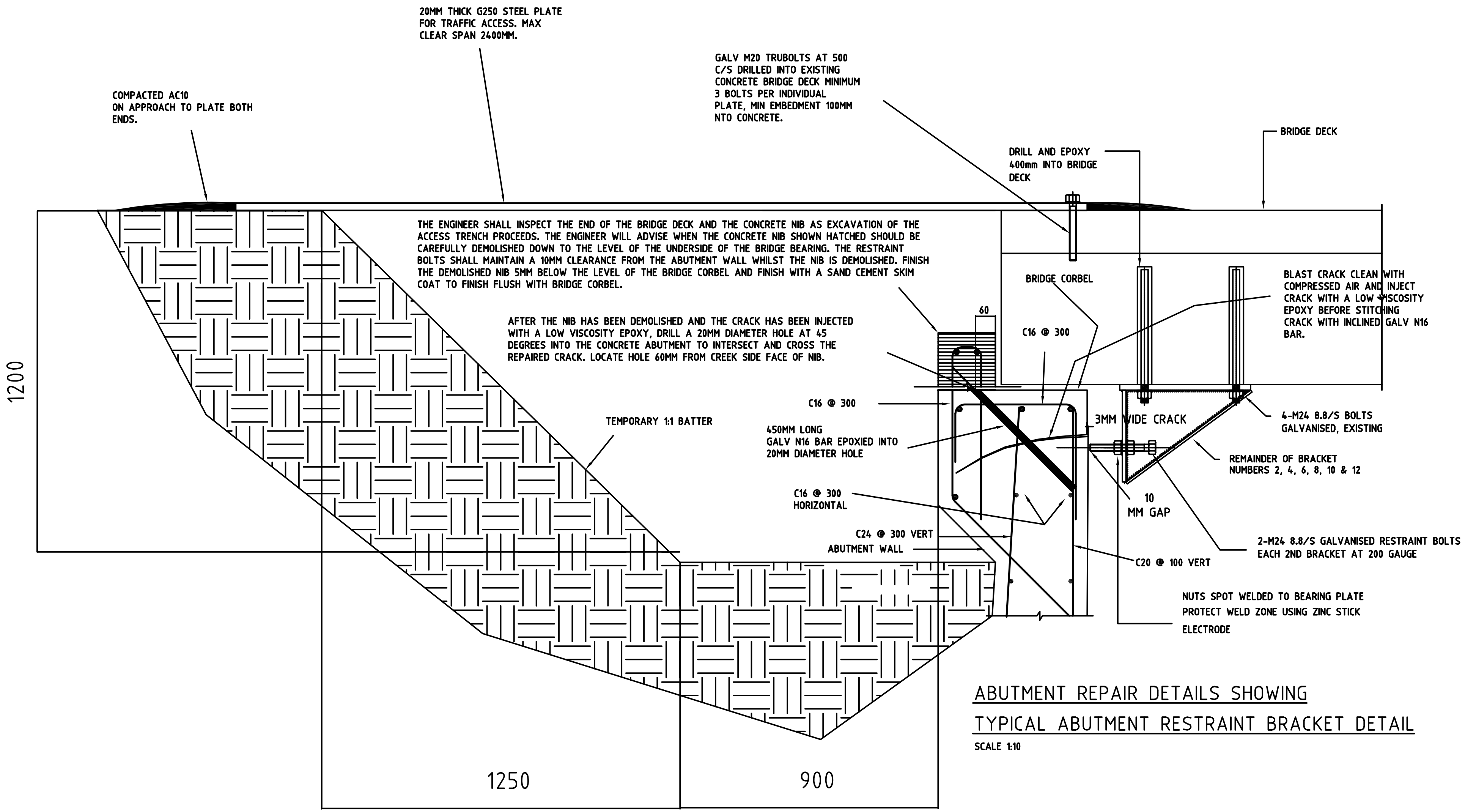


FIGURE 2

FIGURE 3

FIGURE 4

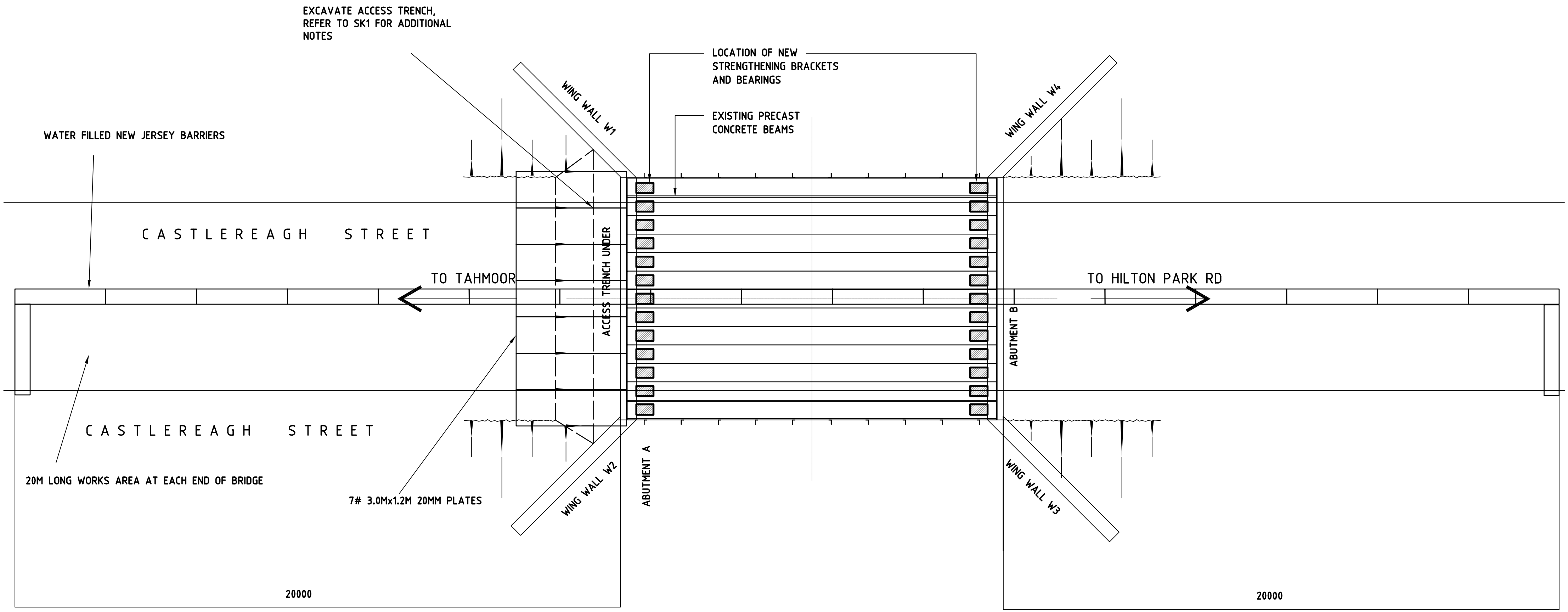
FIGURE 5



ABUTMENT REPAIR DETAILS SHOWING

TYPICAL ABUTMENT RESTRAINT BRACKET DETAIL

SCALE 1:10



MYRTLE CREEK BRIDGE PLAN

SCALE 1:100

NOTE

THE CONTRACTOR SHALL PROVIDE & MAINTAIN TEMPORARY TRAFFIC LIGHTS AT BOTH ENDS OF THE BRIDGE TO MANAGE LOCAL TRAFFIC IN ACCORDANCE WITH ALL RTA & STATUTORY REQUIREMENTS.

THE EXCAVTION OF THE ROAD PAVEMENT, AND MOVEMENT OF THE ACCESS TRENCH DECK PLATES AND THE TRAFFIC BARRIERS SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR DURING THE CONTRACT PERIOD.

Emergency Management Plan (EMP)

Castlereagh Street Bridge

Longwall 27 – Tahmoor Colliery

Emergency Management Plan

Castlereagh Street Bridge

Contents

Belinda Clayton | Community Coordinator | Xstrata Coal Tahmoor

1. Introduction

1.1 Purpose

The primary purpose of this Emergency Management Plan (EMP) is to implement an evacuation plan for the residents of Hilton Park Road Thirlmere. Castlereagh Street Bridge is the only access point for residents in vehicles via road. In the event that Castlereagh Street Bridge is closed due to the impacts of longwall mining and an incident (eg fire) occurs, triggering the evacuation of the Street, the EMP for Hilton Park Road Thirlmere will be triggered.

The Castlereagh Street Road Bridge over Myrtle Creek is a two-lane bridge that provides access for residents to Hilton Park Road Thirlmere. The single-span bridge is constructed with a concrete deck on concrete abutments, as shown in Fig. 1. This bridge is located above Longwall 25.

Figure 1 – Castlereagh Street Bridge over Myrtle Creek.

1.2 Background

The Castlereagh Street Road Bridge over Myrtle Creek is a two-lane bridge providing access for approximately 22 residential properties in Hilton Park Road. The single-span bridge is constructed with a concrete deck on concrete abutments.

Subsidence impacts from longwalls 25 and 26 have caused minor cracking and movement to the Castlereagh Street Bridge. John Matheson and Associates (JMA) have completed mitigation works during the extraction of both longwalls. At no point was there a concern with the surface safety and serviceability of the Bridge (refer to longwall 27 Wollondilly Shire Council Management Plan for further details).

Longwall 27 will extract beneath Hilton Park Road Thirlmere. In the event that the Castlereagh Bridge becomes inaccessible due to the impacts from longwall 27 and residents are required to evacuate the Street, the following EMP will be activated.

1.3 Scope

The plan applies to residents of Hilton Park Road Thirlmere and any person that is in Hilton Park Road Thirlmere at the time of an Emergency (due to the closure of Castlereagh Street Bridge from the impacts of longwall mining).

1.4 Objective

The objectives of this plan are to:

- Protect life
- Prevent or minimise injury
- Provide an evacuation point during an emergency

1.5 Related Documents

This EMP should be read in conjunction with the *Tahmoor Colliery Longwall 27 Wollondilly Shire Management Plan.*

1.6 Hilton Park Road EMT

The EMP for Hilton Park Road Thirlmere is implemented through emergency notification actions and supported by an organised team to control and manage the emergency and return to normal operations.

The role of the EMT is to notify residents, protect residents and re-establish access to Hilton Park Road Thirlmere via Castlereagh Street Bridge. The EMT will also provide rescue and response services during the event of an emergency.

2. Emergency Management

An emergency is an event which:

- Has occurred, or has the imminent potential to occur and
- Has or is likely to endanger persons, the environment or infrastructure and
- May or does require urgent action to treat injured persons or prevent further injury/damage.

If in the event that the Castlereagh Street Bridge becomes out of operation due to the impacts from mining and Hilton Park Road residents are required to be evacuated to a safe environment, Tahmoor Colliery will implement this Plan.

3. EMT Roles and Responsibilities

The EMP is implemented through emergency notification and supported by teams to control and manage the emergency.

The role of the Red Group is to notify, coordinate and manage the broader ramifications of the incident.

The role of the Yellow Group is to deal with the immediate impact of the emergency.

The Green Group are the residents of Hilton Park Road Thirlmere.

Code	EMT	Person/s	Roles and Responsibilities
Red	Tahmoor Colliery	Environment & Community Manager Community Coordinator Environment Coordinator	Strategic direction of the Group and residents. Provision of ongoing updates to the Community. Coordinate residents with Railcon and emergency services. Control the situation. Manage broader ramifications of incident.
Yellow	Emergency Services	Police Fire	Deal with immediate impact of emergency.
Orange	Wollondilly Shire Council	Works Department	Manage Castlereagh Street Bridge works with John Matheson and J&J Earthmoving.
Orange	Mine Subsidence Board	Picton Manager	Provide approval to proceed with costs approved by MSB.
Orange	John Matheson & Associates	John Matheson	Priority is coordinate J&J Earthmoving with works program for Castlereagh Street Bridge.
Orange	Railcon	Chris Bloor	Coordinate Rail Protection Officer/s.
Orange	J&J Earthmoving	Manager Works Team	Manage works provide program directed by Wollondilly Shire Council and John Matheson.
Green	Residents	As per table	Coordinated by Tahmoor Colliery and emergency services. Assist emergency services were practicable.

Table 1 – Hilton Park Road Thirlmere EMT Roles and Responsibilities.

4. Process Chart for Activation

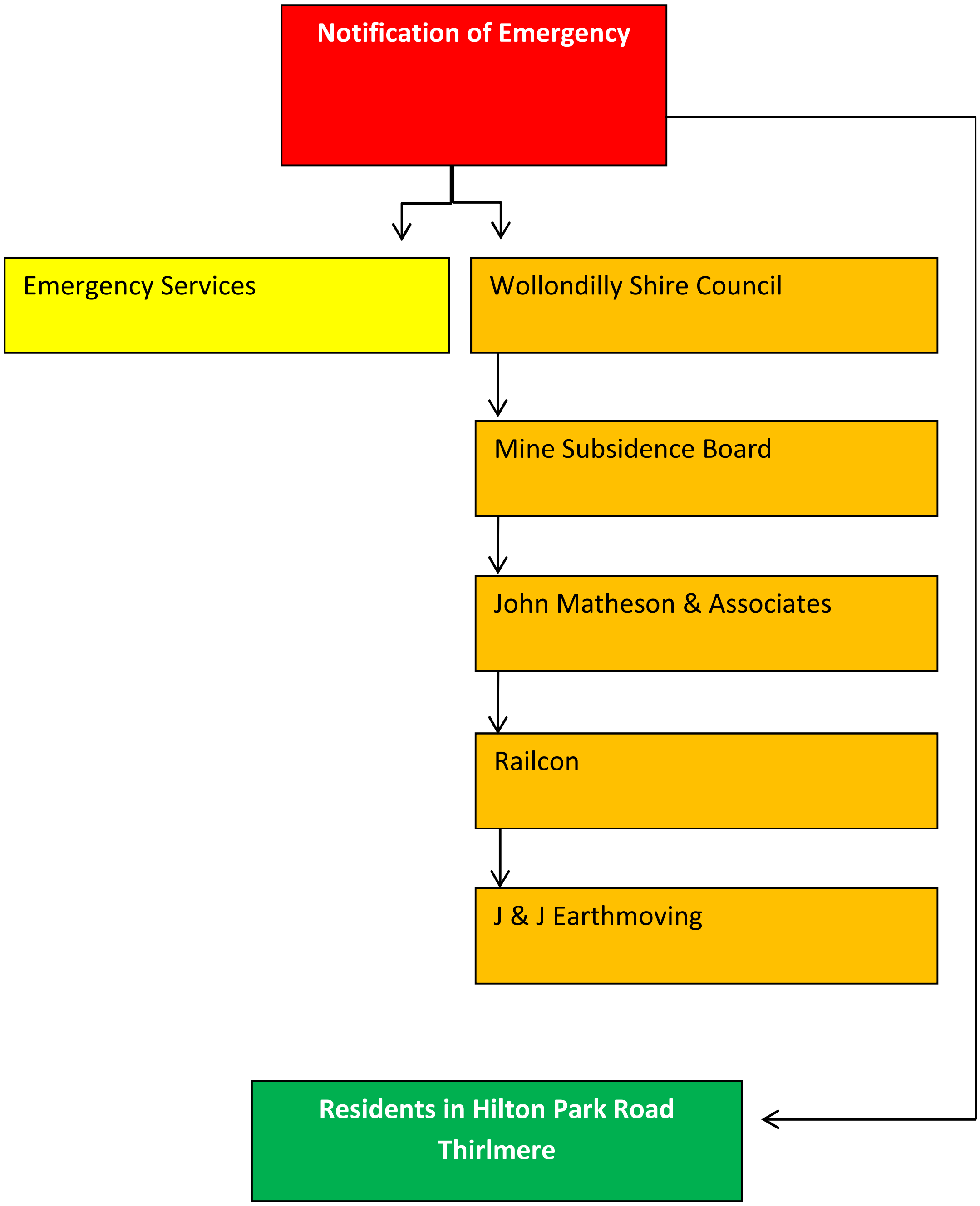


Figure 2 – Hilton Park Road Thirlmere EMT Process Chart for Responsibility Activation.

5. Emergency Procedure

5.1 Response to Notification

In the event that Castlereagh Street Bridge becomes inaccessible for any period due to longwall mining impacts, Tahmoor Colliery will implement the EMP in consultation with emergency services.

Tahmoor Colliery will incorporate this document into the *Tahmoor Colliery Longwall 27 Wollondilly Shire Council Management Plan.*

On notification of the emergency, the Environment and Community Manager of Tahmoor Colliery will contact the emergency services and all persons in the Orange Group. The Orange Group will meet at Castlereagh Street Bridge as soon as possible to determine works required. A Rail Protection Officer or Officers will be arranged to attend Rail Corridor for the duration of road closure.

Residents will be notified by the Community Coordinator and Environmental Coordinator of Tahmoor Colliery by phone and door knocking.

5.2 Emergency Evacuation

An emergency evacuation point has been designated for the use in the event of an emergency. The emergency assembly point is identified in Figures 2.

Site Protection Officers will position at Point 2 and 3 (Figures 2) and will remain on site during Castlereagh Street Bridge closure.

5.3 Procedure

The emergency evacuation procedure will take place as follows:

- Residents of Hilton Park Road Thirlmere are to meet at the Emergency Assembly Point
- Evacuation will be by pedestrians only down the dirt lane to the rail corridor and wait/meet the Rail Protection Officer
- The Rail Protection will escort residents along the rail corridor to point 4
- An access gate will allow residents to exit safely into Mahonga Street Tahmoor

5.4 Emergency Evacuation Route

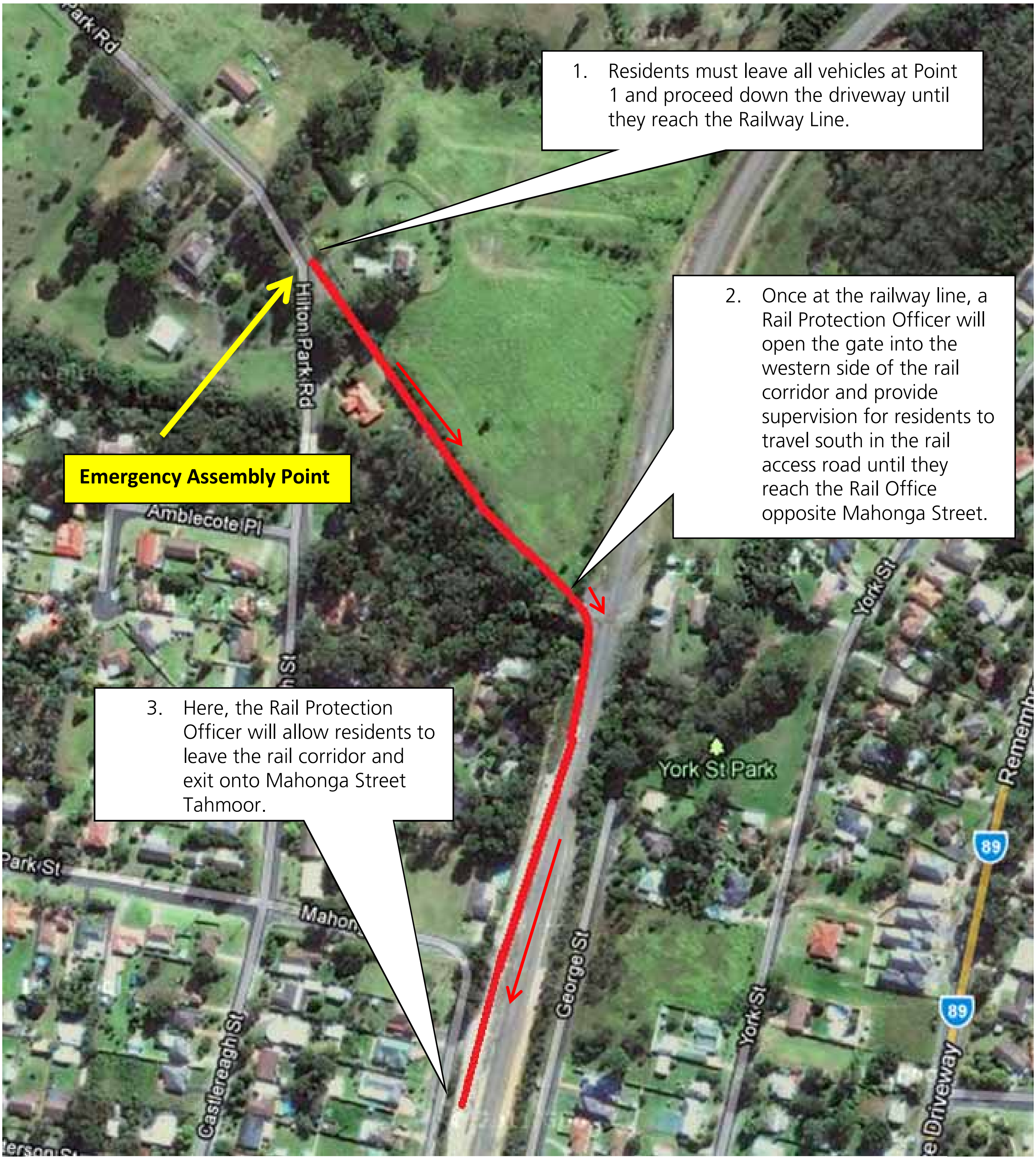


Figure 3 - Emergency Evacuation Route.

6. Contact Details

Contact details for notification of the EMT and residents in the event that the EMP is implemented.

Group	Members	Team
Red	Xstrata Coal Tahmoor	EMT
Yellow	Emergency Services	Emergency Services
Orange	Specialist Operators	EMT
Green	Hilton Park Road Thirlmere Residents	Residents

6.1 Red Group

Name	Company	Contact by phone	Contact by email
Belinda Clayton	Xstrata Coal Tahmoor Community Coordinator	Work 4640 013 3 Mobile 0428 260 899 24 hour 1800 154 415	bclayton@xstratacoal.com.au
Nick Wandke	Xstrata Coal Tahmoor Environment Coordinator	Work 4640 0158 Mobile 0409 318 219 24 hour 1800 154 415	nwandke@xstratacoal.com.au
Ian Sheppard	Xstrata Coal Tahmoor Environment and Community Manager	Work 4640 010 0 24 hour 1800 154 415	tahmoorenquiries@xstratacoal.com.au

6.2 Yellow Group

Name	Contact by phone
Police	000
Fire	000
Ambulance	000
Gas	131 909
Electricity	131 002
Sydney Water	132 090
Telstra	132 203

6.3 Orange Group

Name	Company	Contact by phone	Contact by email
Justin Nyholm	Manager - Works Wollondilly Shire Council	Direct 4677 2339 Work 4677 1100 Mobile 0418 602022	justin.nyholm@wollondilly.nsw.gov.au
Barry Allen	Coordinator - Works Wollondilly Shire Council	Direct 4677 8232 Work 4677 1100 Mobile 0408 979046	barry.allen@wollondilly.nsw.gov.au
Darren Bullock	District Manager Mine Subsidence Board	Work 4640 0158 Mobile 0409 318219 24 hour 1800 154415	nwandke@xstratacoal.com.au
John Matheson	Civil Engineer John Matheson and Associates	Work 4640 0100 24 hour 1800 154415 Mobile 0418 238777	jma.eng@bigpond.net.au
Chris Bloor	General Manager Rail Con	Work 4256 0162 Mobile 0422 807231	chris@railcon.com.au
Jack Doosey	Manager J&J Earthmoving	Mobile 0410 309178	jandjearthmoving@bigpond.com

6.4 Green Group

Address	Name	Contact Number
4 Hilton Park Road	Laurence and Erica McCormack	
10 Hilton Park Road	Yvonne Bowden	(02) 4681 8226
15 Hilton Park Road	Martin and William Gatt	(02) 4683 0795
25 Hilton Park Road	Andrew Crawford	(02) 4681 8062
35 Hilton Park Road	Deborah Ramage	
45 Hilton Park Road	Andrew and Christine Anderson	(02) 4683 0845
49 Hilton Park Road	Peter and Robyn Wells	(02) 4683 0106
55 Hilton Park Road	William and Majorie Thompson	
65 Hilton Park Road	Mark and Christina Dench	(02) 4681 0686
75 Hilton Park Road	Thomas Anderson	(02) 4681 8002
85 Hilton Park Road	David Jolliffe	
95 Hilton Park Road	Marc Webb	
70 Hilton Park Road	Wayne and Jeanette Cook	(02) 4683 1443
99 Hilton Park Road	Denis Colburn	

Address	Name	Contact Number
100 Hilton Park Road	Marhold Pty Ltd	
20 Hilton Park Road	Joseph Touma	
50 Hilton Park Road	Peter and Jacqueline Roser	(02) 4681 9699
70 Hilton Park Road	Wayne Cook and Jeanette Cook	
74 Hilton Park Road	Michael and Jennifer Blount	(02) 4681 0138
80 Hilton Park Road	Geoffrey and Lyn Stalling	(02) 4681 8395
84 Hilton Park Road	Brian and Christine Duncan	
90 Hilton Park Road	Deborah Sandars and David Pescud	(02) 4681 8850

7. Aerial Plan

Figure 4 – Location Map of Hilton Park Road Residents.