appendix 5A
Part 1

Subsidence Assessment (Main Text and Figures)

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West Wallsend Colliery

Subsidence Predictions and General Impact Assessment of the Proposed Western and Southern Domain Longwalls, West Wallsend Colliery

DGS Report No. WWD-012/1

Date: 5 March 2010

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5 March, 2010

Mr Mark Robinson
Environment and Community Co-ordinator
West Wallsend Colliery
The Broadway
Killingworth NSW 2278

DGS Report No. WWD-012/1

Dear Mark,

Subject: Subsidence Predictions and General Impact Assessment of the Proposed Western and Southern Domain Longwalls, West Wallsend Colliery

This report has been prepared in accordance with the brief provided on the above project.

Please contact the undersigned if you have any questions regarding this matter.

For and on behalf of
Ditton Geotechnical Services Pty Ltd

Steven Ditton
Principal Engineer

Executive Summary

West Wallsend Colliery (WWC) is proposing to extract a further thirteen longwall blocks (LWs 38 to 50) in the West Borehole Seam. The longwalls will be located in the Western and Southern Domains of their existing mining leases ML1451, CCL725 and CCL718. The proposed mine workings have been assessed in accordance with Part 3A of the *Environmental Planning and Assessment Act* 1979 (EP&A Act) for New Major Projects.

The Risk Management Zone (RMZ) concepts developed in the recently published report by the NSW Department of Planning on the management of subsidence impacts in the Southern Coalfield (**DoP, 2008**) have also been incorporated in this assessment.

The longwall panels will be approximately 179 m wide (final void width) with cover depths varying from 70 m to 360 m beneath variable topographic relief. One of the proposed panels in the Southern Domain (LW46) will be 168 m wide. The mining height will decrease from 4.7 m along the north-eastern mining lease boundary to 3.3 m along the south-western lease boundary.

The study area is dominated by a north-western orientated ridge line (which forms part of the Sugarloaf Range) in the west of the lease and is now largely State Conservation Area (formerly State Forest). The land is mainly undeveloped bush land with several fire and access trails. The Great North Walk (Department of Lands) is a public walking trail which crosses the Western Domain and follows an unsealed gravel road.

Several geotechnically distinct terrain units exist above the longwall panels and consist of residual and alluvial soil profiles. Ground slopes range between 1^{o} and 35^{o}. Intermittent low-height cliffs < 20 m in height exist along the mid to upper slopes of the ridge which dominates the north-western area of the lease.

Several ephemeral creeks drain the majority of the lease towards the south east and south west and include first order tributaries associated with Cockle, Diega, Ryhope, Central and Bangalow Creeks. The majority of the creek beds consist of broad to incised alluvial sediments, which are actively eroding and transported and deposited downstream during storm events. The upper reaches of the creeks have sandstone and conglomerate exposures along the predominately dry channel bases.

The Western and Southern Domains are intersected by the F3 Freeway (RTA) and a major utilities easement which has inter-city high pressure gas (Jemena) and petroleum pipelines (Caltex) and three Optic Fibre cables (Telstra, Optus and Nextgen). Other features of note within the mining lease are three communications towers (Gencom and Telstra), Aboriginal Heritage sites (Awabakal and Koompatoo), Wakefield Road (Lake Macquarie Council) and a medium-sized farm dam of 5 ML capacity (A. McArthy).

Several risk assessment and planning meetings to-date have been held between the stakeholders so as to identify and minimise impacts deemed to be ‘High’ risk. As a result of this process, some of the proposed longwall panels have been pulled back to a conservative

angle of draw distance from the freeway, services easement, several culturally sensitive aboriginal heritage sites (i.e. axe grinding groove sites, The Wet Soak and a stone 'arch') and an area of low depth of cover in the vicinity of Ryhope Creek. In total, these changes to the mine plan have resulted in the sterilisation of approximately 4.4 million tonnes of coal resource.

The above changes are consistent with the requirements of Risk Management Assessment Zones (RMZs) recently defined for the Southern NSW Coalfield (refer **DoP, 2008**).

Abandoned bord and pillar workings (Awaba Colliery) in the Great Northern Seam exist above the proposed LWs 49 and 50. The Record Tracing of the workings indicates an irregular workings outline within a 100 m by 200 m area. An inspection of the surface around the workings indicates the workings are still standing with no evidence of mine subsidence. The overburden consists of 20 to 30 m of medium to thickly bedded sandstone with semi-cleared bushland on the surface.

Several State Survey control marks (Department of Lands) are likely to be affected by the proposed mining activities and are likely to need re-surveying after subsidence movements have ceased.

The multiple-longwall panel subsidence predictions presented in this study have been based on several empirical and calibrated analytical models of overburden and chain pillar behaviour. The models have been developed and previously applied at WWC and in the Newcastle Coalfield.

There are several massive sandstone channels and conglomerate units within the overburden which will have a subsidence reducing effect above some of the mining lease and proposed panel geometries due to their spanning and bulking properties. The Teralba Conglomerate Member is likely to reduce subsidence above the longwall panels beneath the ridges in the Western Domain by approximately 50%.

The assessment also included estimates of continuous and discontinuous sub-surface fracture heights above the longwalls, the potential for direct hydraulic connection to the surface and the likely increases to rock-mass permeability after mining.

The potential worst-case impacts on surface and subsurface features have been assessed in this study and based on the predicted subsidence parameters and previous experience gained from the Northern Domain longwalls.

The key subsidence impact parameter prediction results are presented below:

(i) Final maximum panel subsidence after extracting multiple longwall blocks in the West Borehole Seam will range from 0.34 m to 2.52 m (9 % to 58% of mining height).

(ii) Maximum chain pillar subsidence after mining 3.3 to 4.8 m high longwall panel faces will range from 0.12 m to 1.0 m above pillars with widths of 30 to 45 m and lengths of

110 m typically. The subsidence will be due to compression of sandstone / siltstone / coal beds above and below the pillars and the pillars themselves. Some load will also be transferred to the goaf if pillars go into yield after mining is completed.

(iii) Yielding of the 35 m wide (solid) chain pillars in the Western Domain is expected where cover depth is > 265 m and double-abutment loads are likely to exceed the strength of the pillars (i.e. an FoS < 1). However, the chain pillars will strain-harden due to high core confinement and transfer of load to compressed goaf will limit subsidence above chain pillars to < 1 m after mining is completed.

(iv) Tilts and curvatures are expected to vary widely over the panels due to the high cover depth range. Maximum panel tilts are estimated to range from 5 to 167 mm/m with concave and convex curvatures ranging from 0.24 to 6.6 km^{-1} (or radii of 4.2 km to 0.15 km).

(v) The maximum tensile strains associated with the curvatures over the panels will range from 2 mm/m to 38 mm/m.

(vi) The maximum compressive strains associated with the curvatures over the panels will range from 2 mm/m to 38 mm/m.

(vii) Subsidence and associated impact parameter contours of principal tilt, horizontal strain have been prepared for surface and sub-surface impact assessment of the natural, archaeological and man-made features.

The results of the subsidence prediction study indicate the following worst-case impacts for the natural, archaeological and man-made features within the study area:

- Surface cracking and shearing will develop within tensile and compressive strain zones above the extracted panels and range in width from 10 mm to 380 mm at cover depths ranging from 360 m to 70 m respectively.

  Repairs to some of the wider and deeper creeks in the vicinity of roads and public access areas may be required. Some remediation of steep slopes and dry creek beds may also be necessary in consultation with stakeholders and government agencies.

- The increase or decrease of surface gradients of up to 3^o (5%) along ephemeral creeks and gullies that exist above the proposed longwall panels. A commensurate increase in erosion and sedimentation is expected to occur along creek beds after several storm events or until a new equilibrium is reached.

- Potential increased ponding depths range from 0.5 m to 1m above the middle-third sections of several of the longwalls and creeks and flatter areas of the site, based on post-mining contour predictions. Any increases of existing ponded areas or development of new ponds are likely to be in-channel and unlikely to cause significant impact to the existing environmental conditions.

- Direct hydraulic connection to the surface due to sub-surface fracturing is considered 'possible' between 70 and 100 m depth, and 'unlikely' where cover depths are > 100 m. Through a risk based assessment of the consequence of potential impacts of connective cracking in this area, 70 m depth of cover has been adopted as the minimum RMZ limit, until further data is available for the Western Domain.

- In-direct or discontinuous sub-surface fracturing could interact with surface cracks where cover depths are < 215 m.

  Creek flows may be re-routed to below-surface pathways and re-surfacing down-stream of the mining extraction limits in these areas. This behaviour usually only occurs where shallow surface rock is present. The temporary loss of surface water flows is unlikely to occur where deep alluvial soil profiles exist. The high level sediment bed load that passes along the creeks during storm events is likely to infill surface cracking.

- For areas with cover depth > 215 m, surface water impacts are likely to be minimal.

- En-masse slip of hills or ridges on weakened bedded partings in thinly bedded siltstones / shale due to the predicted tilts is considered very unlikely.

  Local instability could occur on the steep slopes and cliff lines on the elevated ridges due to mine subsidence deformation and cracking. The situation will also be exacerbated if cracks introduce surface water runoff into the slopes. The high density of trees on the steep slopes is likely to limit potential instability from mining impacts.

- Rock falls from cliff lines could roll down to the base of steep slopes. The potential for roll-out occurrences represents a potential risk to the public and mining personnel and appropriate management measures would be to provide appropriate warning signage, removal of loose boulders from above the Great North Walk and other access tracks, repair cracks and controlling access to the area during mine subsidence development.

- Instability of steep, eroded creek channel banks could be exacerbated by mine subsidence cracking and tilting. Increased erosion (i.e. head-cuts) and sedimentation could develop above chain pillars where surface gradients are predicted to change by more than 1^o to 2^o.

- Minor, localised, sub-surface flow re-routing could occur along creek beds due to the predicted surface cracking along exposed rock bar areas and re-surface downstream of the affected areas. Remedial works may be required where cracks are unable to ‘self-heal’ through natural sedimentation deposits in the cracks over time.

- The ‘Wet Soak’ is a site of historical significance to the Aboriginal community. It is a natural depression located on a south-east facing slope to the north of the proposed LW40. WWC has moved the starting position LW40 to outside the angle of draw to the feature to minimize the potential for long-term impact.

- Several axe grinding grooves, a number of stone arrangements and a stone arch have been identified within the mining lease. Through extensive consultation with the local Aboriginal stakeholders significant changes have been made to the mine plan so as to minimise the potential impacts on these culturally significant sites. The potential for cracking these sites is assessed to be Very Low or < 1% Probability (and based on measured crack data for LWs 22 to 36).

- One axe grinding groove site within the Koompahtoo LALC area exists above the proposed longwall panels, the mine plan has been adjusted so that this site is now located above the centre of a widened chain pillar (which was increased from 30 m to 45 m) to decrease the potential for cracking from 'High' to 'Moderate' or from 32% to 18% cracking probability (based on measured crack data for LWs 22 to 36).

  The chain pillar width between LWs 44 and 45 would need to be increased to 60 and 70 m to provide a 'Low' and 'Very Low' cracking probability (i.e. <10% and <1%).

- Several Awabakal artefact scatters, scarred trees and isolated find sites have been identified within the mine subsidence zone but are unlikely to be impacted directly by mine subsidence. These features could be indirectly impacted by surface cracking and tilting through increased erosion and sedimentation or during crack-remediation activities by WWC. However prior to remediation activities appropriate mitigation measures will be undertaken to reduce the potential impacts.

- The dam on McArthy’s property is likely to be impacted by mine induced cracking and/or shearing resulting in dam wall breach or storage losses through the floor of the dam storage area. Repairs to the dam and temporary supplies of water may be required by the landholder.

- Fences around the study area may be impacted by strains and tilts and require repairs after mining. A mitigation strategy for moving livestock to non-impacted paddocks should be considered whilst mining impacts are occurring or repairs being carried out.

- The Great North Walk is an unsealed gravel road that intersects the project area. The walk follows an east-west orientated ridge spur across the Western Domain and is likely to be subsided by 1.1 m to 2.4 m. The worst case crack width is estimated to range between 30 mm and 140 mm across the road where it passes through the tensile and compressive strain zones above each longwall panel.

  It is estimated that approximately 30 to 50 m of the road above each longwall may require repairs to tensile cracking or compressive shear failures through the road after mining of each panel is completed. Some sections of road above LW39 and 40 may be impacted by local instability on cracked fill slopes.

- The Gencom communications towers (CT1 and CT2) are located outside the limits of LW43 but within the angle of draw. The towers are located on the north-western ridge line crest and could be affected by worst-case subsidence of 0.32 and 0.05m, tilt of 5 and 2 mm/m and tensile strain of 2 mm/m and 0.5 mm/m respectively. Subsidence mitigation strategies may include strengthening of the tower structure or adjustment to the mining layout to minimise potential subsidence impacts.

The poles of the suspended powerline to the Gencom towers are located above the proposed chain pillars between longwalls 38 to 40. Maximum subsidence, tilt and tensile strains at the pole sites range from 0.0 to 0.13 m, 0 to 6 mm/m and 0 to 5.5 mm/m respectively. Surface cracks of up to 60 mm in the vicinity of the poles may also occur. The loss of clearance beneath the conductor catenaries are not expected to exceed the subsidence predictions (i.e. < 0.13 m).

- Telstra communications tower (CT3) will be 146 m from the finishing position of LW47 and outside the angle of draw (50^o). LW47 has been pulled back from the tower to limit far-field displacements and tensile strain of <20mm and <0.25 mm/m respectively. The impacts of the predicted movements are likely to be 'negligible'.

- The Caltex/Jemena Pipelines are located in 2 m deep, backfilled trenches within a services easement located between the Western and Southern domains and west of the F3 Freeway. The shortest distances from the easement to the proposed longwall panels in the Western Domain range from 35^o to 84^o Angle of Draw (AoD) and are therefore unlikely to be subsided.

  The pipelines may be affected by far-field movements of up to 19 mm towards the extracted longwall blocks. The associated ground strains along and across the pipeline and are unlikely to exceed 0.1 mm/m and 0.3 mm/m respectively, if no faulting is present. The presence of faulting may increase the longitudinal and lateral strains up to 0.25 mm/m and 0.7 mm/m respectively.

  Worst-case lateral curvatures along the pipelines are predicted to range between +/- 0.003 km^{-1} (or > 315 km curvature radius).

- The Telstra, Optus and Next Gen Optic Fibre Cables (OFC's) are located in 1 m deep, backfilled trenches within the services easement used for the Caltex and Jemena Pipelines.

  The OFC's may be affected by far-field movements of up to 19 mm towards the extracted longwall blocks. The associated ground strains along and across the OFC and are unlikely to exceed 0.1 mm/m and 0.35 mm/m respectively for normal conditions. The presence of a fault or adverse conditions may increase the above values to 0.25 mm/m and 0.7 mm/m along and across the easement due to slip movements along the fault plane (see **DgS, 2009**).

  Worst-case lateral curvatures along the pipelines are predicted to range between +/- 0.003 km^{-1} (or > 315 km curvature radius).

Monitoring and impact management strategies may require the monitoring of signal loss in the OFC's, as ground movements are predicted to be less than measured by conventional survey techniques

- The F3 Freeway Pavement, cuttings, fill embankments, concrete pipe shafts and culverts are all located outside the angle of draw to the nearest panel ends or corners (of 44^o and 84^o).

  The predicted worst-case far-field displacements at the crests of cuttings and toes of the embankments (including the shaft in embankment No. 3) range from 0 and 27 mm with principal worst-case tensile strains of 0.0 to 0.4 mm/m.

  Cracking or damage to the cuttings, embankments, concrete shafts and culverts is considered very unlikely during mining. Monitoring and impact management strategies may require visual inspections and low frequency surveying.

- The two F3 Freeway Bridges, which form an underpass through Fill Embankment 3, are located outside the angle of draw. Worst-case far-field longitudinal and lateral strain predictions at the bridge location indicate the north and south abutments may be subject to cumulative lateral displacements of 3.5 and 4 mm and longitudinal displacement of 2.0 and 2.5 mm after longwall mining is completed in both domains.

  Worst-case horizontal shear strains (i.e. distortion) at the north and south bridges are estimated to range between 0.11 and 0.17 mm/m (i.e. 1 and 2 mm of shear displacement).

- Palmers Road freeway overpass bridge is 77 m long and 16.5 m wide. It is located approximately 1300 m south of LW50, where cover depth is about 70 m to the WBH seam. It is considered very unlikely that any movement due far-field displacement will have developed at Palmers Road Bridge after mining is completed.

- Wakefield Road and the associated fill embankment are likely to be subsided by between 1.0 m and 2.4 m by LWs 45 and 46 in the Southern Domain. The road is also predicted to be tilted by 24 to 34 mm/m (1.5^o to 2^o increase or decrease in pavement gradients) and cracked by tensile and compressive shear strains ranging from 9 mm/m to 11 mm/m respectively.

  The maximum crack widths are estimated to range between 60 mm and 90 mm respectively over distances of about 10 m to 15 m, and will probably occur across the pavement (and through the embankment) where it crosses the tensile strain zones. Shear cracks or buckling failures are also expected to occur in the pavement and embankment sections where they cross the compressive strain zones in the central areas of the longwalls.

  Timing of the crack development is expected to occur in two phases (i.e. the dynamic and final phases). The first cracking development phase will occur when the LWs 45 and 46 passes underneath the road and arcuate tensile cracks occur up to 30 m behind the longwall face.

The second phase of cracking will occur when the full subsidence trough starts to develop between 0.7 and 1.4 times the panel width behind the retreating longwall face. Buckling and shearing of the sections of road above the middle third area of the subsidence trough (i.e. the compressive strain zone) would be expected to occur as well as the tensile zone cracks at this time. Similar impacts have occurred previously on Wakefield Road and have been managed effectively in consultation with MSB and LMCC.

- The consequences of subsiding the abandoned mine workings in the GN Seam could result in a further increment of subsidence caused by instability of standing pillars or mine working roof.

  Based on the estimated working height of 2.5 to 3 m in the GN Seam and previous cases of multi-seam mining interaction at Newstan Colliery, it is assessed that the additional subsidence due to the abovementioned mechanisms could range from 10% to 60% of the GN Seam thickness or 0.25 m to 1.8 m.

  The total subsidence above this area of the proposed longwall panels could therefore increase to a range of 0.6 m to 3.3 m, which represents (i) a 20% to 50% of the combined seam thicknesses and (ii) a 140% to 210% increase over the subsidence predicted for the longwalls only. It should be noted that a significant proportion of the workings will be located above the chain pillars in the WBH Seam and therefore likely to significantly reduce the potential for increased subsidence due to multi-seam interaction effects. The potential subsidence impacts in this area are predicted to be of minimal consequence.

The above items will require further discussion with stakeholders to enable effective Subsidence Management Plans (SMP) to be developed. A suggested program for monitoring subsidence, tilt and strain at the relevant locations has been provided for the purpose of implementing and reviewing the SMP. The use of remote Aerial Laser Scanning or equivalent is considered an appropriate subsidence monitoring technique in lieu of traditional ground based subsidence survey lines in very steep terrain and the majority of the surface, which is remote undeveloped bushland.

It is concluded that the assessed range of potential subsidence and far-field displacement impacts after the mining of the proposed longwalls LW38 to 50 will be manageable based on the analysis outcomes and discussions with stakeholders to-date.

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Glossary of Terms

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Angle of Draw	The angle (normally no greater than 26.5^{o} from the sides or ends of an extracted longwall block) from the vertical of the line drawn between the limits of extraction at seam level to the 20 mm subsidence contour at the surface. The 20 mm subsidence contour is an industry defined limit and represents the practical measurable limit of subsidence.
Chain Pillar	The pillar of coal left between adjacent longwall panels. This forms a barrier that allows the goaf to be sealed off and facilitates tailgate roof stability.
Compressive Strain	A decrease in the distance between two points on the surface. Compressive strains may cause shear cracking or steps at the surface if > 3 mm/m and are usually associated with concave curvatures near the middle of the panels.
Confidence Limits	A term used to define the level of confidence in a predicted subsidence impact parameter and based on a database of previously measured values above geometrically similar mining layouts.
Cover Depth	The depth from the surface to the mine workings.
Critical Longwall Panels	Longwall panels that are almost as deep (H) as they are wide (W) (ie 0.6 <W/H < 1.4) and is the point where complete failure of the overburden starts to occur and maximum subsidence is likely to develop if the panel widths are increased.
Curvature	The rate of change of tilt between three points (A, B and C), measured at set distances apart (usually 10 m). The curvature is plotted at the middle point or point B and is usually concave in the middle of the panel and convex near the panel edges. i.e. curvature = (tilt between points A and B - tilt between points B and C)/(average distance between points A to B and B to C) and usually expressed in 1/km. Radius of curvature is the reciprocal of the curvature is usually measured in km (i.e. radius = 1/curvature). The curvature is a measure of surface 'bending' and is generally associated with cracking.
CWC Values	The Credible Worst-Case (CWC) prediction for the predicted impact Parameter and normally based on the Upper 95% or U99% Confidence Limit line determined from measured data and the line of 'best fit' used

	to calculate the mean value. The CWC values are typically 1.5 to 2 times the mean values.
Development Height	The height at which the first workings (i.e. the main headings and gateroads) are driven; usually equal to or less than the extraction height on the longwall face.
Extraction Height	The height at which the seam is mined or extracted across a longwall face by the longwall shearer.
Factor of Safety	The ratio between the strength of a structure divided by the load applied to the structure. Commonly used to design underground coal mine pillars.
Far-Field Displacement	Horizontal displacement outside of the angle of draw, associated with movement are due to horizontal stress relief above an extracted panel of coal. The strains due to these movements are usually < 0.5 mm/m and do not cause damage directly. Such displacements have been associated with differential movement between bridge abutments and dam walls in the Southern Coalfield, but generally have not caused significant damage.
First Workings	The tunnels or roadways driven by a continuous mining machine to provide access to the longwall panels in a mine (i.e. main headings and gate roads). The roof of the roadways is generally supported by high strength steel rock bolts encapsulated in chemical resin. Subsidence above first workings pillars and roadways is generally < 20 mm.
Gate Roads	The tunnels or roadways driven down both sides of the longwall block (usually in pairs), to provide airways and access for men, materials, and the coal conveyor to the longwall face. The conveyor side of the block is called the 'maingate' and dust laden air and coal seam gases are exhausted on the opposite side (called the 'tailgate').
Goaf	The extracted area that the immediate roof or overburden collapses into, following the extraction of the coal. The overburden above the ‘goaf’ sags, resulting in a subsidence 'trough' at the surface.
Greenfields Site	Refers to a mining area where no local data of ground response to underground mining exists. Subsidence predictions must therefore be based on experience gained from mining in other areas with similar geological conditions and appropriate engineering models.
Horizontal Displacement	Horizontal displacement of a point after subsidence has occurred above an underground mining area within the angle of draw. It can be

predicted by multiplying the tilt by a factor derived for the near surface lithology at a site (e.g. a factor of around 10 is normally applied in the Newcastle Coalfield).

Inbye An underground coal mining term used to describe the relative position of some feature or location in the mine that is closer to the coal face than the reference location.

Inflexion Point The point above a subsided area where tensile strain changes to compressive strain along the deflected surface. It is also the point where maximum tilt occurs above an extracted longwall panel. It is typically located between 0.25 and 0.4 x cover depth from the panel sides.

Longitudinal Subsidence Profile Subsidence measured (or predicted) along a longwall panel or centre line.

Longwall The method of extracting a wide block of coal (which will be 178.6 m wide in the case of the West Wallsend Colliery) using a coal shearer and armoured face conveyor. Hydraulic shields provide roof support across the face and protect the shearer and mine workers.

The longwall equipment is installed along the full width of the block in an 8 to 10 m wide installation road at the start of the block before retreating back to the finishing end of the block. The shields are progressively advanced across the full width of the face, as shearing continues in a sequence of backwards and forwards motions across the face.

Depending on the geological conditions and longwall performance, the longwall retreats at a typical rate of about 50 to 100 m/week.

Maingate Refers to the tunnels or roadways down the side of a longwall block which provides access for mine operations personnel, power, materials and clean air to the longwall face. It is usually located on the side of the longwall panel adjacent to unmined panels or solid coal.

Mean Values The average value of a given impact parameter value (i.e. of subsidence, tilt and strain) predicted using a line of 'best fit' through a set of measured data points against key independent variables (e.g. panel width, cover depth, extraction height). The mean values are typically two-thirds to half of the credible worst-case values.

Mining Height Refers to the height or thickness of coal extracted along a longwall face.

Outbye An underground coal mining term used to describe the relative position of some feature or location in the mine that is closer to the mine entry point than the reference location.

Outlier A data point well outside the rest of the observations, representing an anomaly (e.g. a measurement related to a structural discontinuity or fault in the overburden that causes a compressive strain concentration at the surface, in an otherwise tensile strain field).

Panel Width The width of an extracted area between chain pillars.

Primary Subsidence The subsidence which occurs that is directly caused by longwall face retreat and the sagging of overburden or compression of chain pillars. Primary subsidence usually continues for three or four longwall panels at an exponential rate of decay after each longwall passes a given site.

Residual Subsidence The last 5% to 10% of subsidence that occurs after primary subsidence is complete. It is not directly linked to the retreating longwall face and is associated with the re-consolidation or re-compaction of goaf and overburden. It is unlikely that any further impact to structures will occur due to residual subsidence.**Shoving**The shortening effect of compressive strains due to mine subsidence on surface terrain, which results in localised shearing movements of soils and rock.

Strain The change in horizontal distance between two points at the surface after mining, divided by the pre-mining distance between the points.

i.e. Strain = ((post-mining distance between A and B) - (pre-mining distance between A and B))/(pre-mining distance between A and B) and is usually expressed in mm/m.

Strain can be estimated by multiplying the curvature by a factor derived for the near surface lithology at a site (e.g. a factor of around 10 is normally applied in the Newcastle Coalfield).

However, discontinuous overburden behaviour can result in local strain and curvature concentrations at cracks, making accurate predictions difficult. A rule of thumb is normally applied to allow for these effects, which is to increase smooth profile strains (and curvatures) by 2 to 3 times. The increase in strain also usually develops at locations with shallow rock profiles, as opposed to areas with deep soil alluvial soil profiles.

Study Area	The area which may have features in it that could be impacted by the proposed mine. It is usually defined by a 26.5^o to 35^o angle of draw to 20 mm of vertical subsidence and up to 3 to 5 times the cover depth to limits of possible far-field horizontal displacement.
Sub-critical Longwall Panels	Longwall panels that are deeper than they are wide (W/H < 0.6) and cause lower magnitudes of subsidence than shallower panels due to natural arching of the overburden across the extracted coal seam.
Subsidence	The difference between the pre-mining surface level and the post-mining surface level at a point, after it settles above an underground mining area.
Subsidence Control	Reducing the impact of subsidence on a feature by modifying the mining layout and set back distances from the feature (normally applied to sensitive natural features that can't be protected by mitigation or amelioration works).
Subsidence Impact	The effect that subsidence has on natural or man-made surface and sub-surface features above a mining area.
Subsidence Management Plan	Refers to the approval process for managing mine subsidence impacts, in accordance with the Department of Primary Industry Guidelines. The mine must prepare a Subsidence Management Plan (SMP) to the satisfaction of the Director-General, before the commencement of operations that will potentially lead to subsidence of the land surface.
Subsidence Mitigation/ Amelioration	Modifying or reducing the impact of subsidence on a feature, so that the impact is within safe, serviceable, and repairable limits (normally applied to moderately sensitive man-made features that can tolerate a certain amount of subsidence).
Subsidence Reduction Potential	Refers to the potential reduction in subsidence due to massive strata in the overburden being able to either 'bridge' across an extracted panel or have a greater bulking volume when it collapses into the panel void (if close enough to seam level). The term was defined in an **ACARP, 2003** study into this phenomenon and is common in NSW Coalfields.
Super-Critical Longwall Panels	Longwall panels that are not as deep (H) as they are wide (W) (ie W/H > 1.4) and will cause complete failure of the overburden and maximum subsidence that is proportional to the mining height (i.e 0.58 to 0.6 T).
Tailgate	Refers to the tunnels or roadways down the side of a longwall block which provides a ventilation pathway for bad or dusty air away from

the longwall face. It is usually located on the side of the longwall panel adjacent to previously extracted panels or goaf.

Tilt The rate of change of subsidence between two points (A and B), measured at set distances apart (usually 10 m). Tilt is plotted at the mid-point between the points and is a measure of the amount of differential subsidence.

i.e. Tilt = (subsidence at point A - subsidence at point B)/(distance between the points) and is usually expressed in mm/m.

Tensile Strain An increase in the distance between two points on the surface. Tensile strains are likely to cause cracking at the surface if > 2 mm/m and are usually associated with convex curvatures near the sides (or ends) of the panels. Tensile strain also usually develops above chain pillars.

Transverse Subsidence Profile Subsidence measured (or predicted) across a longwall panel or cross line.

Valley Closure The inward (or outward) movement of valley ridge crests due to subsidence trough deformations or changes to horizontal stress fields associated with longwall mining. Measured movements have ranged between 10 mm and 400 mm in the NSW Coalfields and are usually visually imperceptible.

Valley Uplift The phenomenon of upward movements along the valley floors due to **Valley Closure** and buckling of sedimentary rock units. Measured movements have ranged between 10 mm and 400 mm in the NSW Coalfields and may cause surface cracking in exposed bedrock on the floor of the valley (or gorge).

1.0 Introduction

This report presents mine subsidence predictions for the proposed Western and Southern Domain Longwalls (LW's 38 to 50) in the West Borehole Seam at West Wallsend Colliery, Killingworth.

The scope of work for the project requires the following to be included in a report for inclusion with a Part 3A Project Application to the NSW Department of Planning (DoP):

(i) Subsidence impact parameter predictions for ten longwall panels in the Western Domain and three panels in the Southern Domain;

(ii) A general impact assessment of natural and man-made surface and sub-surface features within the zone of potential subsidence effects (see **Figures 1a** and **1b**).

(iii) Delineation of subsidence risk management zones and likely requirements for Subsidence Management Plan Development with the relevant stakeholders.

Reference will also be made to relevant information provided in previous reports by Strata Engineering that were prepared for previous Subsidence Management Plans (SMP) in the Western and Southern Domains.

This study has been based on information provided by West Wallsend Colliery and two empirically based mine subsidence prediction models developed for the Newcastle and US Coalfields (refer to **ACARP, 2003** and **SDPS, 2007**).

The proposed mining layouts and management strategies presented herein have been determined based on the requirements of Part 3A of the *Environmental Planning and Assessment Act* 1979 (EP&A Act) for New Major Projects.

Several risk assessment and planning meetings to-date have been held between the stakeholders so as to identify and minimise or practically avoid impacts deemed to be 'High' risk. As a result, of this process, several of longwall panels have been pulled back to a conservative angle of draw distance from the freeway, services easement, several culturally sensitive aboriginal archaeological sites (i.e. axe grinding groove sites, The Wet Soak and a stone 'arch') and the low depth of cover area in the vicinity of Ryhope Creek.

The Risk Management Zone (RMZ) concepts developed in the recently published report by the NSW Department of Planning on the management of subsidence impacts in the Southern Coalfield (**DoP, 2008**) have been incorporated in this assessment.

The control of mine subsidence impacts to acceptable levels may also require further adjustment to the proposed mine plan if early-warning monitoring results and observed impacts indicate that the likely consequences at sensitive sites cannot be managed by mitigatory techniques alone.

2.0 Scope of Study

The work program included the following activities:

(i) Surface inspections of the Western and Southern Domains to identify and characterise existing surface features within the study area.

(ii) Development of a geotechnical model for the study area.

(iii) Prediction of maximum subsidence parameters and profiles using **ACARP, 2003** and completion of model validation/calibration studies using local and regional mine subsidence data and geological information.

(iv) Calibration of the **SDPS®** model to the subsidence profiles generated by the **ACARP, 2003** model.

(v) Generation of subsidence, tilt, strain and horizontal displacement contours for the proposed mining layout.

(vi) Assessment of the impact of the subsidence predictions on the existing surface and subsurface features. The level of uncertainty in the predictions has also been discussed to allow effective risk management assessments to be completed.

(vii) Estimation of sub-surface fracturing heights above the panels for the assessment of the potential of hydraulic connection occurring to surface creeks/alluvium and sub-surface aquifers.

(viii) Recommendations for monitoring and/or mining layout adjustment to control subsidence impacts if mitigation/remediation options are not feasible.

(ix) Estimation of the potential impacts of far-field displacements (FFD) and far-field strains (FFS) on significant items of infrastructure between the Western and Southern Domains. Far-field displacement modelling and the predicted impact of a previous ten panel layout was assessed in **SEA, 2006** and **SEA, 2007** and will be referred to in this study.

3.0 Available Information

The following information was provided by West Wallsend Colliery to prepare this report:

- The proposed longwall panel layout.
- Cover depth contours to the West Borehole (WBH) Seam and seam thickness isopachs.
- Fifty-five surface to seam borehole logs.
- Twenty surface to seam borehole geophysical logs.
- Geological structure (fault and dyke) locations in the study area.
- Surface topographic levels and existing drainage regime locations.
- Locations of surface developments and infrastructure in the study area.
- Surface photography

A plan of the proposed longwalls (LW 38 to 50) with cover depth contours, surface levels, borehole locations, seam thickness isopachs and surface features in the Western and Southern Domains are presented in **Figures 1a-e** to **3a-b**.

4.0 Mining Geometry

The following mine workings details have been assumed in this assessment:

(i) The longwall panels (LWs 38 to 50) are located at a depth of approximately 70 m to 360 m below the surface and will be 178.6 m wide (final void width). One of the proposed panels in the Southern Domain (LW 46) will be 168.2 m wide.

(ii) The first six longwall panels (LWs 38 to 43) will be mined in the Western Domain and will retreat towards the south-south east.

(iii) The next three panels (LW's 44 to 46) will be mined in the Southern Domain and retreat towards the north-north west. Access to the longwall panels will be from south west orientated main headings, which will be driven between the two domains and beneath the F3 Freeway.

(iv) The last four panels (LW's 47 to 50) will be mined in the Western Domain and retreat towards the south-south east.

(v) The longwall panels will have average face extraction heights ranging from 4.5 m to 3.5 m in the Western Domain (decreasing towards the southwest) and from 4.8 m to 4.5 m in the Southern Domain. The face will be ramped back to the gate roads to a height of 3.5 m.

(vi) Chain pillars will be formed between the panels and will be 30 to 45 m wide by 110 m long (solid) with 5.5 m wide nominal roadway widths.

The panel W/H ratio will range from 0.50 to 2.55, indicating both sub-critical and supercritical subsidence behaviour (supercritical behaviour is normally assumed to occur when W/H > 1.4 - see glossary for more term definition details). The chain pillars will have w/h ratios of 8.6 to 10 and will be expected to strain harden if overloaded.

5.0 Landuse

The surface land of the mining area is mainly State Conservation Area (SCA) (formerly State Forest) and is managed by the Department of the Environment and Climate Change (DECC). The SCA is currently used for recreational activities such as bushwalking, mountain bike and trail bike riding. Two small private-rural land holdings are located in the project area and are owned by the McArthy and Corliss residents.

The majority of the Western and Southern Domains is mainly undeveloped native woodland that has been used as a timber logging resource in the past. The land also has numerous unsealed access roads, tracks and fire trails, including the Great North Walk.

The F3 freeway and associated infrastructure (i.e. bridge overpass, cuttings, fill embankments and drainage structures) pass through the middle of the proposed mining area. A utilities easement with high pressure gas and petroleum pipelines and optic fibre cables are adjacent to the western side of the freeway.

There is also an abandoned pit top and bord and pillar workings in the Great Northern (GN) Seam above longwalls LW 49 and 50, known as the former Awaba Colliery; see **Figure 1a** and **1c**. The cover depth above the GN Seam is about 140 to 150 m at this location and the seam is understood to be 3 to 4 m thick.

6.0 Surface Features

6.1 General

In the Western Domain, topographic relief ranges from 40 m AHD to 360 m AHD. The terrain is generally flat to gently undulated in the east and south, with a prominent north-south trending ridge in the west. The ridges have steep to moderate slopes ranging from 15^{o} to 35^{o}, with several low level cliff lines with heights ranging from 2 to 15 m. The headwaters of several ephemeral creeks and tributaries / gullies drain the site towards the east, south-east and south-west.

In the Southern Domain, topographic relief ranges from 40 m AHD to 90 m AHD. The terrain is gently undulated with slopes ranging from 5^{o} to 15^{o} on broad-crested ridges and ephemeral drainage gullies. Several small earth dams (< 1.5 m high) were built along Central Creek during the construction of the F3 freeway, resulting in the development of a pond chain along the watercourse.

The natural and archaeological features of significance within the study area include:

- Steep slopes and low-height cliffs above the Western Domain.
- Several ephemeral creeks and tributaries (Schedule 1) that form the headwaters of Cockle, Diega, Rhyhope, Bangalow, Palmers and Central Creeks.
- Sandy alluvial deposits (up to 3 m deep) exist along the lower reaches of the creeks with no rock exposures evident.
- Silty sand and sandy clay surface soils present on the site are mildly to highly erosive / dispersive if exposed to concentrated runoff during storm flow events.
- Vegetation on the ridges consist of dense stands of dry schlerophyll forest with shrubs, ferns and grasses. The riparian zones along creeks have sparse to dense stands of melaleucas, fallen trees and grasses.
- Flora/fauna habitats within the study area generally and groundwater dependent ecosystems along the watercourses.
- Aboriginal cultural heritage features that are present in the study area include the ‘Wet Soak’ to the north of LW40, axe grinding groove sites, artefact scatters and scarred trees. The sites have been inspected, catalogued and registered with the DECC in liaison with the registered Aboriginal stakeholders.

Existing man-made features within the study area consists of the following:

- McArthy’s water supply dam (1.25 ML capacity) above LW38.
- The Great North Walk, which crosses the area above the Western Domain longwalls.

- Telstra / Nextgen / Optus Optic Fibre Cables between the Western and Southern Domains.
- Caltex Petroleum and Jemena Natural Gas Pipelines existing in the services easement between the Western and Southern Domains.
- RTA F3 Freeway and associated cuttings, embankments, bridges and drainage infrastructure between the Western and Southern Domains.
- Lake Macquarie Council’s Wakefield Road, which traverses LWs 45 and 46 in the Southern Domain.
- Two Gencom communications towers (CT 1 and CT 2) are located on the crest of the western ridge and are located just outside the extraction limits of LW43. Tower CT1 is located 10 m outside the panels western rib-side and 557 m south of its starting position. Tower CT2 is located 54 m outside the north-west corner of LW43’s starting position.
- A suspended power line (11 kV) is currently being constructed across LWs 38 to 43 to supply mains power to the Gencom towers. The power-poles have been located above the centre of the proposed chain pillars between LWs 38 to 40 in the valley below the ridge. The powerline will be installed up to CT1 and then CT2 (note: the powerline had only been installed out as far as LW40 during the time of report preparation).
- One Telstra communications tower 135 m to the south of the finishing position of LW47.
- Transgrid transmission towers are 150 m to 300 m south of the Southern Domain and located above the already extracted Newstan Colliery longwalls. The towers already have cruciform footings and have been fully subsided.
- An Energy Australia domestic power line (11 kV) along Wakefield Road.
- State Survey marks exist in the proposed mining area.

The locations of the man-made features with surface topography and gradients are shown in **Figure 3a**; the aboriginal archaeological site locations and surface topography are shown in **Figure 3b**.

Further details of each feature are provided in the following sections.

6.2 Steep Slopes And Low-Height Cliffs Above The Western Domain

The western domain longwall panels are overlain by areas of steep, rocky slopes (15^o to 30^o) associated with the Teralba Conglomerate Member, with shallow residual soil cover (< 0.5 m thick) - see **Photo 1**. The steep slopes are located to the north and south of the Great North Walk (which follows a ridge spur crest) above the proposed longwalls LW38 to 43 and LWs 47 to 49, see **Figure 3a**.

There are several low-height cliffs ranging from 2 to 15 m high near the crests of the slopes and several large boulders of up to 1 m diameter, which have rolled for distances of up to 100 m downhill of the crests (see **Photo 2**). The boulders appear to have been stopped by tree impacts on the densely timbered slopes. There are several large, fallen trees on these slopes as well.

The cliffs have 65^{o} to 75^{o} dipping faces with bedding dipping at 5^{o} to 10^{o} towards the SW and into the cliff faces. The cliffs have developed on persistent sub-vertical joint-sets that are parallel and perpendicular to the cliff faces, which strike NW/SE, SW/NE and E/W. The joints are generally open and widely spaced between 0.5 m to 5 m.

The slopes and cliffs above LWs 42 and 43 are similar in terms of geology, but increase in steepness and height (i.e. slopes range from 25^{o} to 35^{o}, with cliff lines ranging from 5 to 15 m in height) along the northern and east facing ridges.

The WBH Seam cover depth below the steep slopes ranges from 140 m to 290 m; see **Figure 1c**.

Photo 1 - Steep Slopes

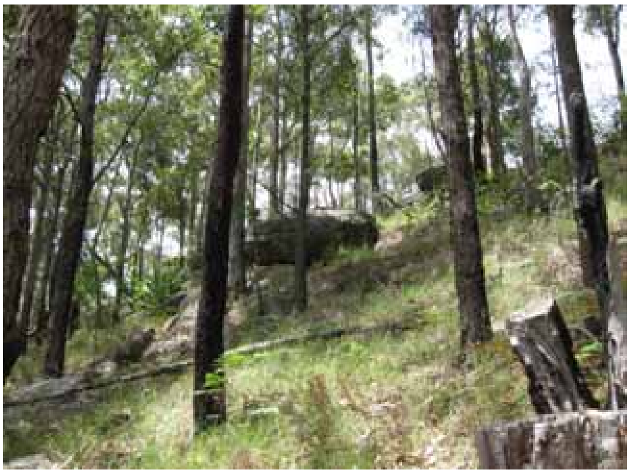
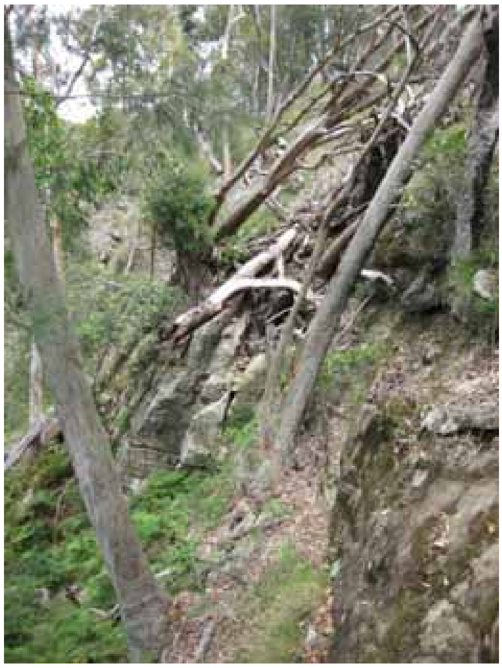


Photo 2 - Low Height Cliffs



6.3 Ephemeral Creeks, Tributaries and Alluvium

There are several first and 2nd order, ephemeral creeks and drainage gullies within the study area that have incised erosion channels with low flow ponding areas - see **Photos 3** and **4**.

The cover depths along the watercourses to the proposed workings range from 70 m to 260 m; see **Figures 1c** and **1d**.

Further details of the creeks are presented in the following sections.

Photo 3 - Ryhope Creek in the Southern Domain

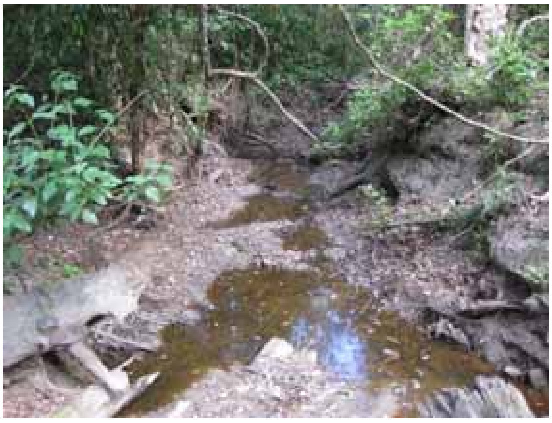


Photo 4 - Central Creek in the Southern Domain

6.3.1 Cockle Creek Tributaries in the Western Domain

Several ephemeral tributaries of Cockle Creek originate within or near the proposed longwall panel limits (LWs 38 to 40) in the Western Domain. The tributaries flow towards the east and into the south flowing Cockle Creek as shown in **Figure 1a**. Two of the four tributaries are located in the northern area of the domains above with two in the southern area. The mine workings cover depth below the northern tributaries ranges from 140 to 160 m and from 90 to 120 m depth below southern tributaries.

All of the tributaries are intermittent/ephemeral 1st and 2nd Order watercourses and are situated in 5 to 10 m wide, semi-actively eroding channels. The channel banks are generally protected by vegetation and are 1 to 1.5 m deep with bank slopes ranging from 10° to 30°. The ground slope along the floor of the tributaries is <5°.

One of the northern tributaries has been dammed on A. McArthy's property.

6.3.2 Diega Creek Tributaries in the Western Domain

Two ephemeral tributaries of Diega Creek (Northern and Southern) flow in a south easterly direction into the creek in the Western Domain, as shown in **Figure 1a**.

The creek then meanders toward the east under the F3 Freeway and into the Southern Domain. Both tributaries are ephemeral 1st and 2nd Order watercourses in this area of the lease.

The cover depth under the northern tributary ranges from 90 to 120 m and from 70 to 100 m under the southern tributary.

6.3.3 Ryhope Creek Tributary in the Western Domain

The head waters of Ryhope Creek drains towards the southeast above the southern ends of LW 49 and 50 in the Western Domain, see **Figure 1b**.

The creek is a 1st and 2nd Order ephemeral watercourse and flows under the F3 Freeway and then into Palmers Creek. The cover depth along the creek above LW50 ranges from 80 to 100 m. The creek flows to the south of LW49.

The tributary within the study area is characterised by a series of small water holes often lacking connection by a well defined channel. In general, the creek is 1 to 2 metres wide and the vertical banks are 1 to 2 metres high. Slopes adjacent to the tributary are long and gentle to moderately sloping. The lower portion of this tributary is covered with dense stands of native and introduced vegetation. Some deep alluvium (> 3 m) exists in the creek channel to the south of the F3 freeway.

6.3.4 Bangalow Creek Tributaries in the Western Domain

Two first order tributaries of Bangalow Creek, which flow towards the west of the Sugarloaf Range, are located above the proposed longwalls 47 to 49 (refer to **Figures 1a and 1b**). The Bangalow Creek tributaries (north and south) proposed to be undermined are ephemeral and only flow for short periods following rainfall,

The tributaries are within the predicted subsidence zone and are defined by small gullies approximately 1 m to 2 m wide. Numerous rock boulders and rock structures exist along the channel bed and banks.

There is extensive riparian vegetation along the creek lines. The creek bed and banks are typically in good conditions with limited evidence of erosion.

6.3.5 Central Creek and the Northern Tributary of Palmers Creek in the Southern Domain

The upper reaches of Central Creek originates within the Southern Domain above LW45 and flows towards the south east across LW46, as shown in **Figure 1b**. Central Creek is a 1st and 2nd Order ephemeral watercourse that drains the north eastern area of the Southern Domain. The creek has been dammed by several small earth bunds, which have created a chain of small ponds along its reach.

The creek itself exists in a broad drainage gully that is 1.5 to 2 m deep and 20 to 40 m wide. The slopes of the gully banks range from 18° to 30° with some recently active bank erosion noted at several locations where the bank had confluence with another tributary. Vegetation in the alluvium filled gully consists of medium to dense stands of semi-mature and mature eucalypts and paper barks (melaeluca's).

Soil exposed along some of the gullies comprises alluvial silty clayey sands and sandy clays that are dark grey with an orange brown mottle. The cover depth under Central Creek ranges from 110 to 155 m.

6.4 Aboriginal Archeological Sites

A total of 67 Aboriginal Heritage Sites have been identified in the study area and consist of the following types:

- Artefact Scatters (12 Sites)
- Axe Grinding Grooves (18 Sites)
- Individual Features (12 Sites)
- Rock Shelters or Stone Arrangements (5 Sites)
- Scarred Trees (10 Sites)
- AIHMS Registered Sites (9 Sites - including the Wet Soak)
- Spring (1 Site)

The location of the sites is provided in **Figure 1b** with a simple numbering system used to identify each site. Further site details are provided in **Appendix F**.

Based on extensive consultation with the Aboriginal stakeholders, the axe grinding grooves, rock shelters/stone arrangements and the Wet Soak will be the most sensitive out of all of the sites identified to mine subsidence impacts. Discussions with Aboriginal stakeholders have identified that eight of the sensitive sites have high cultural significance and will require protection from subsidence impacts.

Subsidence impact reduction strategies have been developed for the following sites:

(i) Wet Soak (Site No. 63)

(ii) Stone Arch, SAH (Site No. 43)

(iii) Stone Arrangement, SAT1 (Site No. 44)

(iv) Stone Canns, STC (Site No. 47)

(v) Grinding Groove Site, GGSD1 (Site No. 27)

(vi) Grinding Groove Site, GGSD1/38-4-1007 (Site No. 28)

(vii) Grinding Groove Site, GGSD2 (Site No.29)

The subsidence impact reduction strategies employed by WWC for the above sites have been to significantly modify the mine plan as follows:

- significant reduction of the length of several longwall panels in the Northern Domain to protect the first six sites listed above,

- the removal of one panel in the southern domain to protect grinding groove Site No. 28, and

- the widening of the proposed chain pillar beneath the ginding groove Site No. 29 to reduce the likelihood of cracking from 'High' to 'Moderate' potential.

The first impact reduction measure mentioned above has now placed the sites outside the angle of draw from the panels and will reduce the potential for surface cracking to develop at the sites.

It has not been possible to completely reduce the potential for cracking from the seventh site (Site No.29), due to its location between two longwall panels. Based on available cracking location data above chain pillars for previous West Wallsend Colliery longwalls, WWC has been able to increase the chain pillar width from 30 m to 45 m beneath the site to minimise the potential for cracking at the site.

A description of the sites is provided in the following sections.

6.4.1 The Wet Soak (Site No. 63)

The ‘Wet Soak’ is a site of high cultural significance to the Aboriginal community. It is a natural depression located on a south-east facing slope that is 81 m to the north of proposed LW40. It is also approximately 151 m and 87 m from the rib-sides of the proposed LWs 39 and 41 respectively, see **Figure 1b**. The cover depth to the WBH Seam at the site ranges from 140 m to 150 m; see **Figure 1e**.

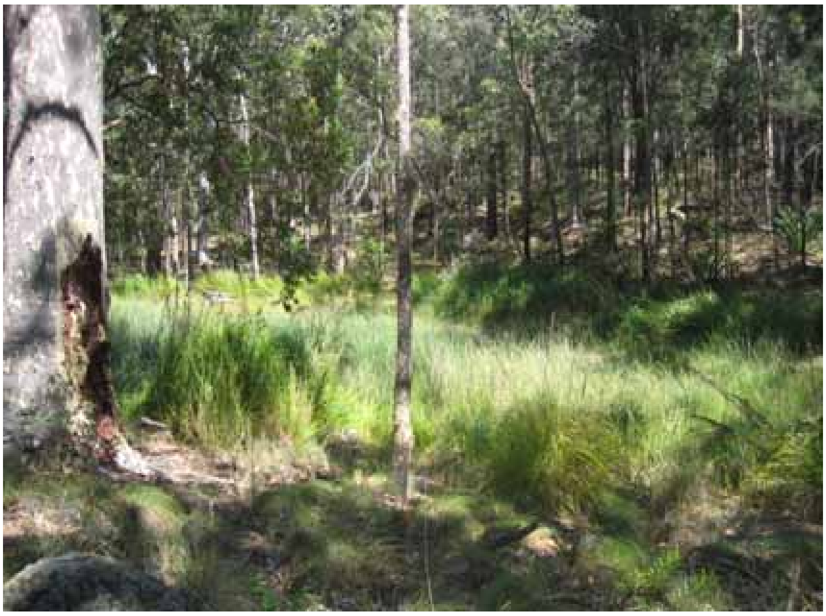
The depression is located within the mid to upper slopes of a WNW - ESE trending ridge-line spur. The depression is oval-shaped with a long axis of approximately 40 m on an NE/SW orientation. The short axis is approximately 30 m in length (see **Photo 5**).

Ground slopes above the depression range from 7^o to 10^o and from 5^o and 15^o below the site. The surface slope through the depression is relatively flat (< 3^o) with a natural embankment height of approximately 1.0 to 1.5 m along the eastern and southern sides.

The slopes above and below the Wet Soak are moderately to densely vegetated, with saplings and mature eucalypts. The Wet Soak itself is overgrown with water resistant grasses and was firm underfoot and no surface water visible at the time of inspection.

Longwall 40 was intentionally shortened to afford protection to this site.

Photo 5 - The 'Wet Soak'



6.4.2 Axe Grinding Grooves (Site No.s 13 to 30)

Fifteen axe grinding groove sites within the Awabakal Local Aboriginal Land Council (LALC) area are located in the Western Domain, see **Figure 1b**.

Three axe grinding groove sites within the Koompatoo LALC area have been located in the Southern Domain, see **Figure 1b**.

The cover depths at the sites are shown in **Figure 1e**.

One of the Southern Domain axe grinding groove sites (Site No. 28) has a set of thirty four axe grinding grooves on the first order tributary of Palmers Creek in the Southern Domain, see **Photo 6**. The site is located within the watercourse on a sandstone outcrop that is approximately 5 to 10 m square. To minimize the potential for subsidence impact to these sites, the proposed longwalls LW44 to 46 have been positioned outside the angle of draw (to the 20 mm subsidence limit) from the site.

Photo 6 - Koompatoo LALC Axe Ginding Groove Site

6.4.3 Rock Shelters and Stone Arrangements (Site No. 43 to 47)

There are five rock shelters or stone arrangements present in the Western Domain, see **Figure 1b**.

The stone arch (Site No. 43) is located along a tributary outside longwall LW49 and is considered to be culturally significant. The sandstone arch is approximately 4.8 m high (above the creek bed) and has a span of 8.8 m, see **Photo 7**. The mine plan has been changed to practically avoid the potential for subsidence impacts to this site.

The stone arrangement (Site No. 44) is also a culturally significant site and is located to the north of the proposed longwalls LW41 and 42.

There are several other rock shelter and stone arrangement sites above the proposed panels that will be undermined.

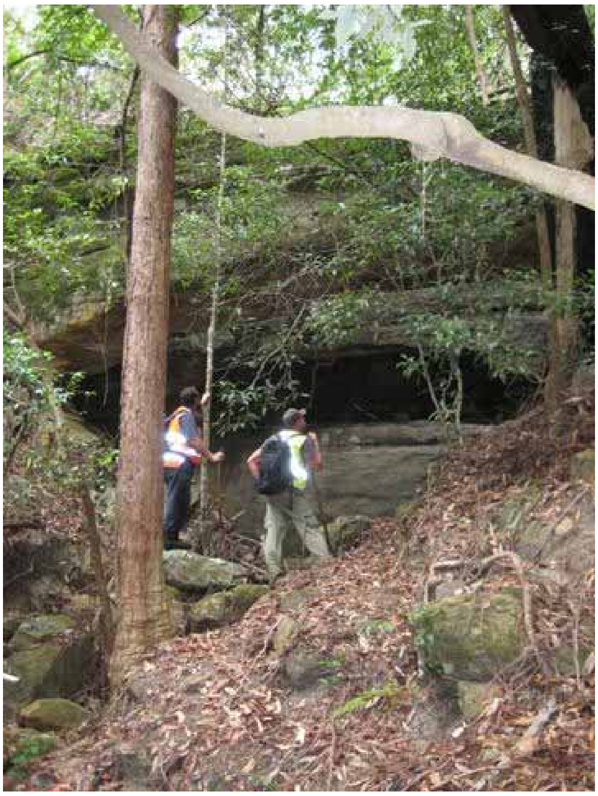
6.4.4 Artifact Scatters and Scarred Tree Sites

There are twelve Awabakal artifact scatters in the Western Domain, see **Figure 1b**.

Artifact scatters are unlikely to be impacted directly by mine subsidence, however, minor impacts due to increased erosion/sedimentation and surface cracking may occur.

The ten scarred tree sites have been identified in both domains (see **Figure 1b**) and consist of tool marks made in standing or fallen trees.

Photo 7 – Stone Arch (Site No. 43)



6.5 McArthy's Water Supply Dam

The dam is privately owned (A. McCarthy) and is located 35 m from the eastern rib side of the proposed LW38, see **Figure 1a** and **Photo 8**.

The dam is about 70 m long (east-west) and 40 m wide (north-south), with a storage volume of approximately 5 ML. The dam has a non-engineered earth fill embankment up to 3 m high with batter slopes of approximately 2H:1V. The batter slopes are well vegetated with grass and semi-mature trees.

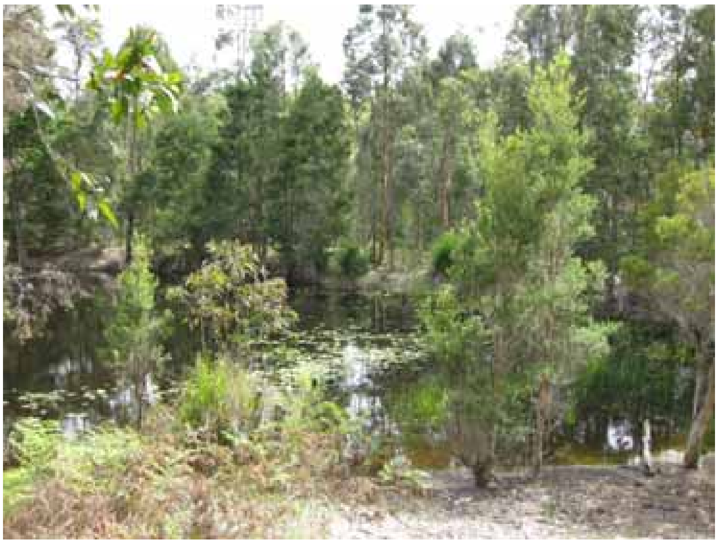
The cover depth to the proposed LW38 at the Dam is 105m; see **Figure 1c**.

The dam embankment and foundation materials consist of silty, sandy clay /clayey sand alluvial soils, borrowed from the storage area. The dam is situated in a broad, ephemeral drainage gully associated with a Cockle Creek tributary with a relatively indistinct channel.

Down-stream terrain of the dam is gently undulated, with ground slopes less than 5^o. The gully has a good coverage of vegetation consisting of grass, trees and shrubs. The gully ultimately drains into Cockle Creek, which is approximately 350 m to the east of the dam wall.

There are no buildings, public access tracks or driveways between the dam and Cockle Creek. Identified scattered artifacts in close proximity to this site have been relocated as part of a salvage program undertaken in consultation with the relevant Aboriginal stakeholders.

Photo 8 - McArthy Dam



6.6 The Great North Walk

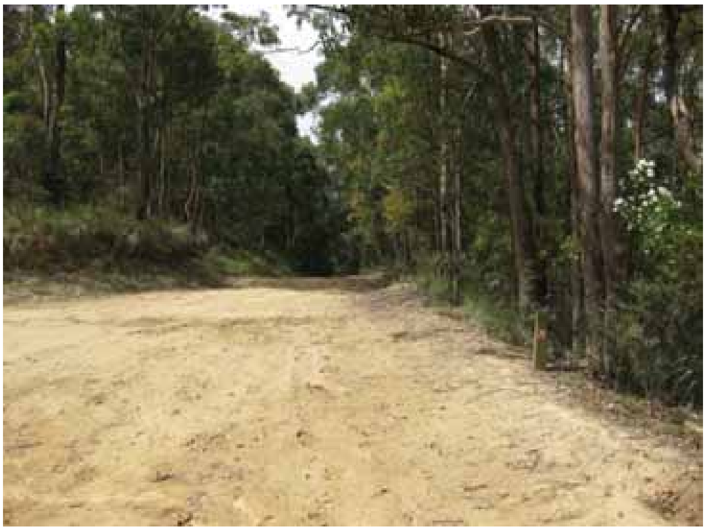
"The Great North Walk is a 250km bush walking track linking Sydney city with the Hunter Valley and Newcastle.

The Great North Walk was initially constructed in 1988 and the Department of Lands continues to undertake its maintenance, construction, enhancement and future development." Ref: Department of Lands website

The Great North Walk is an unsealed gravel road within the project area and crosses the Western Domain longwalls and is coincident with an east-west orientated ridge spur, see **Figures 1a** and **Photo 9**. No improvements have been made to the trail except for several small signs identifying the track.

The cover depth to the workings along the Great North Walk ranges from 140 m to 240 m; see **Figure 1c**.

Photo 9 - Great North Walk and Fire Trail



6.7 Caltex / Jemena Petroleum and Natural Gas Pipelines

The Jemena high pressure natural gas (HPNG) pipeline and Caltex liquid petroleum pipeline easement is adjacent and parallel to the western side of the F3 Freeway, as shown in **Figure 1a** and **Photo 10**.

The services easement is one of the most significant surface features traversing the colliery holding of West Wallsend Colliery. The whole mine layout has been developed to minimise subsidence impact on the services easement. The main development headings and adjacent pillars of coal have been designed to be long-term stable, with negligible subsidence impacts. An additional protective barrier of coal has also been designed adjacent to the main development headings to provide additional long-term support to the easement.

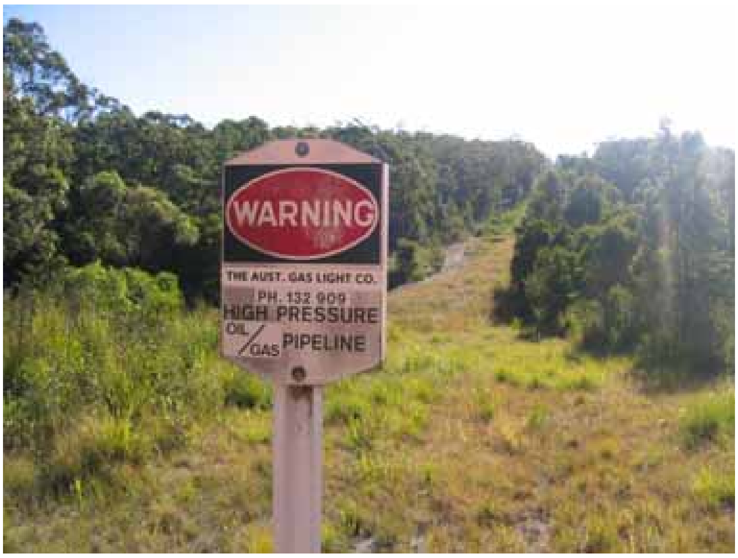
The closest distances from the easement will be to the longwall panel corners, LWs 38 to 50 in the Western Domain, and ranges from 80 to 100 m. The distances to the easement along the longwall panel centrelines will range from 200 m to 250 m and likely to be outside the angle of draw limits. The cover depth ranges from 50 m to 140 m along the easement and is not proposed to be undermined; **see Figure 1d**.

The Jemena pipeline is a 0.5 m diameter mild steel pipe sheathed in low friction coefficient plastic, and is buried in a trench at a depth of approximately 2.0 m. The pipeline is founded in and completely covered by bedding sand. The Caltex pipeline is a 0.3 m diameter mild steel pipe located in the same easement and is adjacent to the Jemena pipeline.

The easement has already been subsided by LWs 27 to 31 up to a maximum subsidence of 376 mm and a horizontal displacement of 168 mm. A maximum longitudinal tensile strain of 1.2 mm/m and a compressive strain of 1.4 mm/m have been measured since the completion of LW31. The transverse strains ranged from -0.3 mm/m (compressive) to 2.6 mm/m (tensile).

Further minor subsidence increases (i.e. <5%) are expected due to the extraction of LWs 32 to 37 in the Northern Domain.

Photo 10 - Services Easement



6.8 Telstra / Nextgen / Optus Optic Fibre Cables

High capacity, fibre optic cables (FOC's) owned by Optus, Nextgen and Telstra are buried in shallow trenches running parallel to the Caltex and HPNG pipelines. The trenches are offset approximately 10 to 20 m to the south-east of the pipelines. The location of the OFC cables in the services easement is shown in **Figure 1a** and **Photo 10**.

The cover depth to the WH Seam along the easement ranges from 100 m to 160 m; see **Figure 1d**.

It is understood that the Telstra cables are very sensitive to movement or differential displacement due to the rocky backfill placed in the trench.

6.9 RTA F3 Freeway

The RTA F3 Freeway is located between the Western and Southern Domains as shown in **Figure 1a** and **Photo 11**.

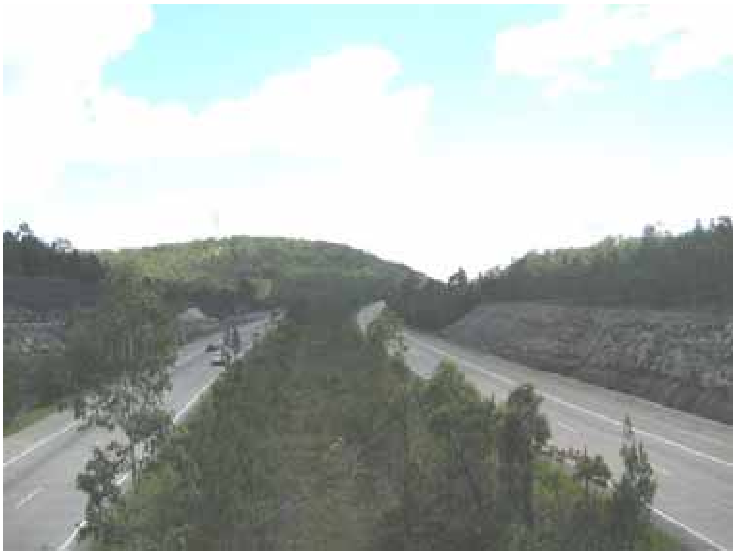
The F3 Freeway is one of the most significant surface features traversing the colliery holding of West Wallsend Colliery. The whole mine layout has been developed to minimise subsidence impact on the F3 Freeway by aligning the main development roadways directly under the F3 Freeway. These main development headings and adjacent pillars of coal have been designed to be long-term stable, with negligible subsidence impacts. An additional protective barrier of coal has also been designed adjacent to the main development headings to provide additional long-term support.

The cover depth to the WH Seam along the easement ranges from 100 m to 160 m; see **Figure 1d**.

The freeway consists of separate north-bound and south-bound carriageways with two lanes each. The carriageways are 7.25 m wide and separated by a 22 m wide nature strip/open drainage reserve.

The freeway passes through four cuttings (No.s 1 to 4) that are 10 m to 40 m deep, and across four earth-fill embankments (No.s 1 to 4) that are about 5 to 10 m high. Details of the cuttings and embankments are presented in the following subs-sections.

Photo 11 - The F3 Freeway (looking north from Palmers Road Overpass)



6.9.1 Carriageways

It is understood that the carriageways each consist of 7.25 m wide rigid pavement with 230 mm thick continuously reinforced concrete (CRC) base and 150 mm thick lean-mix concrete (LMC) sub-base with integrally cast concrete shoulders. A 300 mm thick select fill material forms the sub-grade on the cutting floor, which was ripped to a depth of about 300 mm and re-compacted to RTA specifications. Concrete lined V-drains exist between the pavements and batters along both sides of the cuttings.

The main heading roadways for the Southern and Western Domain longwalls are planned to run beneath the F3 Freeway. The freeway has several sensitive features including several cuttings, fill embankments, culverts and overpass bridges along the 4 km section of the carriageway between the two domains. Details of these features are provided below.

6.9.2 Freeway Cutting No 1

The No.1 Cutting is approximately 350 m long and is located at the Palmers Road overpass. The cutting forms the abutments and access ramps onto and off Palmers Road to the freeway, see **Figure 1b** and **Photo 12**.

The cutting is about 38 m deep on the western batter and 37 m deep on the eastern batter. The distance between the cutting crests is 200 m (see **Section 6.9.6** for design details). The north-western crest of the cutting is approximately 300 m south of the finishing ends of LWs 50 to 51 respectively. The cover depth is approximately 60 m at the cutting; **see Figure 1d**.

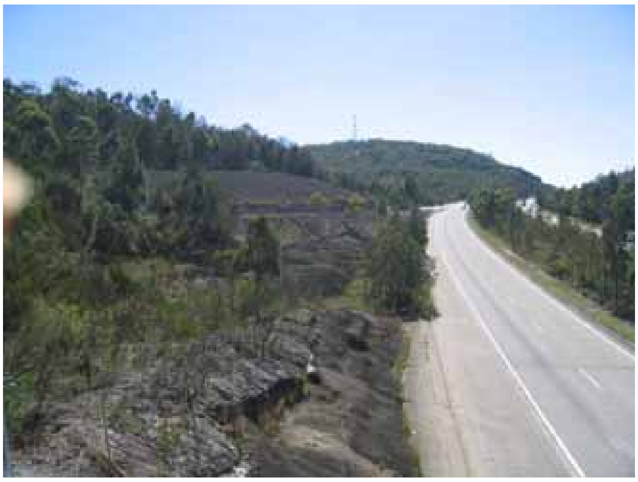


Photo 12 - Freeway Cutting No 1 and North Bound Lane (looking north)

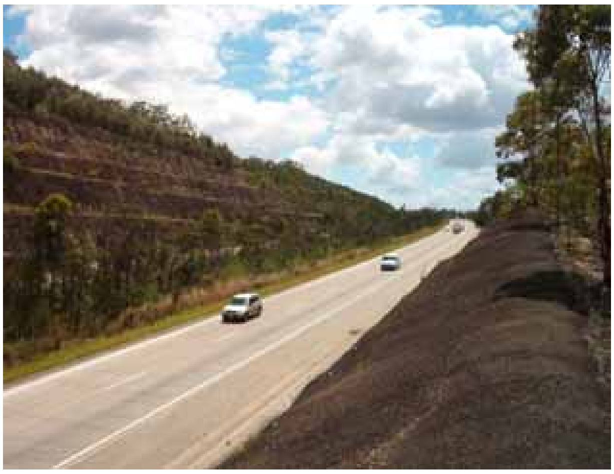


Photo 13 - Freeway Cutting No 2 and South Bound Lane (looking north west)

6.9.3 Freeway Cutting No. 2

Cutting No. 2 is a 330 m long, double-sided cutting and is located 300 m to the south of LW43B in the Western Domain. LW44 is the closest Southern Domain longwall and will be 250 m to the east, see **Figure 1a** and **Photo 13**. The distances between the panels and the cutting are measured from the panel ends to the nearest cutting crest.

The cutting is approximately 40 m deep on the western batter and 30 m deep on the eastern batter. The distance between the cutting crests is 250 m. The cutting is benched every 10 m on the western side (see **Section 6.9.6** for design details).

Wakefield Road is formed on a bench on the eastern side of Cutting No. 2. The bench is located about 10 m above the F3 Freeway carriageway and 20 m below the crest of the cutting. Wakefield Road continues to the north on a 1 km earth fill Embankment No. 2 after it passes through Cutting No.2.

The 20 m deep section of cutting above Wakefield Road is situated predominantly within medium to thickly bedded fine to medium grained sandstone which is grey-brown in colour.

Some honey-cone type weathering is evident in the exposed rock.

A 5 m high mid-section off the cutting has been shotcrete lined with PVC drainage tubes (of 20 mm in diameter) installed through the shotcrete on a 10 m grid spacing. An inspection of the exposed rock slope behind the cutting indicates that the shotcrete lined section consists of extremely weathered siltstone/claystone or a gravelly sandy clay. The shotcrete has been applied to control long-term erosion of this section of the cutting.

The crest of the south-eastern cutting has exposed, distinctly to extremely weathered siltstone and sandstone, which has been battered back at 12°. Some rill erosion has occurred since the formation of the cutting.

Persistent sub-vertical joints are located in the rock strata and strike at 045°:325° (NE:SW) and 100°:280° (E:W). Bedding on the cutting face dips at approximately 3° with a dip direction of 030° (NNE).

6.9.4 Freeway Cutting No. 3

Cutting No.3 is a 525 m long, single sided cutting, and is located 250 to 300 m south of the finishing ends of LWs 40 to 42 in the Western Domain, see **Figure 1a** and **Photo 14**.

The cutting is about 20 to 25 m deep (see **Section 6.9.6** for design details) with 143 m between crests. The bottom 5 m section of the cutting has been shotcrete lined to control long-term erosion and the batters above the shotcrete-lined section are overgrown with vegetation.

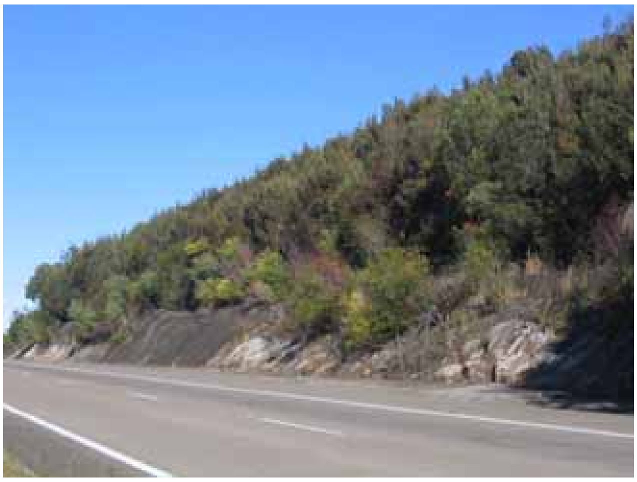


Photo 14 - Freeway Cutting No. 3 and North Bound Lane (looking south west)

Photo 15 - Freeway Cutting No. 4 and North Bound Lane (looking north west)

6.9.5 Freeway Cutting No. 4

Cutting No. 4 is a 500 m long, double sided cutting that is located between LW38 in the Western Domain and LW47 in the Southern Domain, see **Figure 1a** and **Photo 15**. The distances between cutting crests is 143 m. The crests are 240 m and 180 m from the ends of LWs 38 and 47 respectively.

The cutting is about 5 to 10 m deep (see **Section 6.9.6** for design details). The batters are overgrown with vegetation.

6.9.6 Freeway Cutting Design Details

Based on reference to **Leventhal and Stone, 1995**, the cuttings were designed for the worst case scenarios of (i) rock wedge sliding on weak tuffaceous claystone beds (due to groundwater level increases of approximately 3 m above the toe of the embankment), and (ii) earthquake acceleration of 0.1g. For case (i), a design factor of safety (FoS) of 1.2 was adopted for lower bound peak material strengths with 1.5 assumed for average peak material strength conditions.

A minimum FoS of 1.0 was adopted for residual material strength conditions in case (i) and average peak material strength conditions for case (ii).

The possible de-stabilizing effects of future mine subsidence impacts was noted in **Leventhal and Stone, 1995** but not formally included in the stability analysis.

6.9.7 Freeway Embankment No 1

The No.1 Fill is approximately 350 m long and is located across Ryhope Creek. The embankment forms the abutments and access ramps onto and off Palmers Road to the freeway, see **Figure 1a**. The embankment is approximately 110 m wide at the base, 55 m wide at the top and 5 m high (see **Section 6.9.10** for design details).

The north-western toe of the embankment is approximately 400 m and 250 m south of the finishing ends of LWs 49 and 50 respectively. The cover depth is approximately 50 m at the embankment; **see Figure 1d**.

6.9.8 Freeway Embankment No. 2 and 2a

The No.2 Embankment is approximately 780 m long and is located along the southern side of the F3 Freeway, see **Figure 1a** and **Photo 16**. The embankment is approximately 135 m wide at the base, 55 m wide at the top and 5 m high (see **Section 6.9.10** for design details).

The south-eastern toe of the embankment is approximately 200 m and 150 m north of the ends of LWs 44 and 45 respectively. Approximately 50 m length of the embankment is located above LW46 in the Southern Domain. The cover depth at the embankment is about 100 m; **see Figure 1d**.

Embankment 2a turns away from the freeway and forms the section of Wakefield Road which passes over LW46. The embankment is about 48 m wide at the base, 16 m wide at the top and 5 m high (see **Section 6.10** for further details on Wakefield Road).

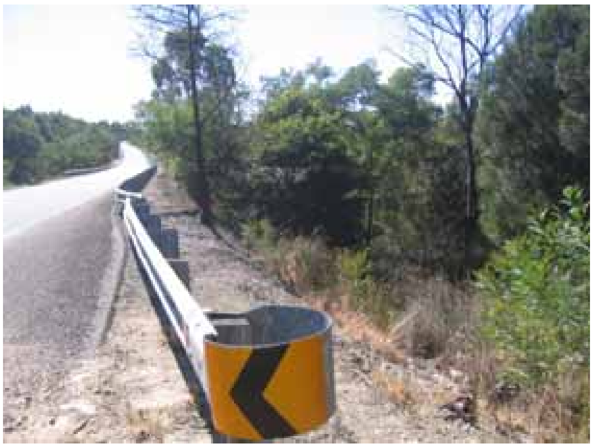


Photo 16 - Embankment No. 2 and Wakefield Road

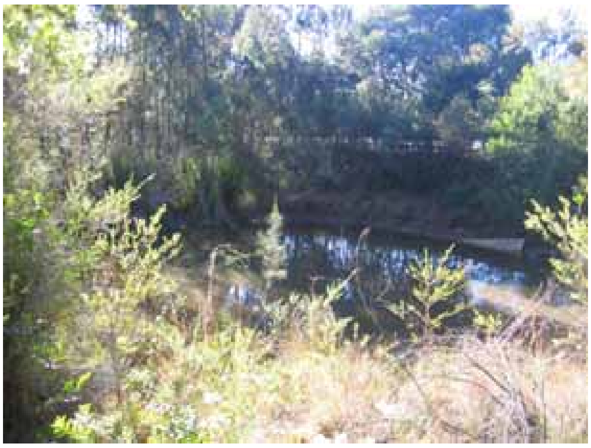


Photo 17 - Embankment No. 3 and Storm Water Detention Basin

6.9.9 Freeway Embankment No. 3

The No.3 Embankment is 600 m long and is located between LW39 in the Western Domain and LW42 in the Southern Domain, see **Figure 1a** and **Photo 17**. The embankment is about 100 m wide at the base, 55 m wide at the top and 5 m high (see **Section 6.9.10** for design details). Lined stormwater detention basins exist on the up-stream and down-stream sides of the embankment.

The distances from the finishing ends of LW39 and LW45 to the embankment are 400 m and 150 m respectively. The cover depth at the embankment is 120 m; **see Figure 1d**.

6.9.10 Freeway Embankment No. 4

The No. 4 Embankment is 300 m long, and is located at approximately 170 m to 200 m east of LW38 in the Western Domain, see **Figure 1a**. The embankment is about 113 m wide at the base, 55 m wide at the top and 5 m high (see **Section 6.9.10** for design details).

The south-eastern toe of the fill embankment is located approximately 20 m from the NW corner of the finishing point of LW30, and was subsided by up to 8 mm.

6.9.11 Freeway Embankment Design Details

The embankments are oriented NE:SW and are approximately 110 m wide at the base and 55 m wide at the top. The fill batters slope at about 26.5° (i.e. 2H:1V).

The embankments are multi-zoned with a predominately low strength claystone core and granular select outer shell. Sandstone boulder rip-rap provide erosion protection to the batters.

The embankments consist of engineered earth fill (i.e. compacted to 95% Standard Compaction in **AS3798**) with sandstone rip-rap boulders placed on the batters, refer to

Leventhal and Stone, 1995.

The earth fill is a very low strength claystone 'core' with higher strength select sandstone 'shell' that was designed to meet stringent slope stability criteria (i.e. they have a minimum and average long term Factor of Safety of 1.2 and 1.5 respectively to allow for material shear strength variations).

The possible de-stabilizing effects of future mine subsidence impacts was noted in **Leventhal and Stone, 1995** but not formally included in the design analysis.

6.9.12 Freeway Bridge (Underpass) in Embankment 3

Two reinforced concrete bridges provide an underpass beneath the freeway through Embankment No. 3. The bridge and the embankment are located between the proposed LW39 in the Western Domain and LW45 in the Southern Domain, as shown in **Figure 1a** and **Photo 18**. The underpass is a bitumen sealed access road and provides access from Wakefield Road to the services easement west of the freeway.

The bridge deck has a span and width of 10 m and is supported by two 5 m high Reinforced Earth® concrete façade retaining walls, which are 8 m apart. The north-bound lane bridge abutments are located 146 m from the finishing corner of LW40 in the Western Domain. The south-bound lane abutments are 157 m from the centerline end of LW45 in the Southern Domain. The cover depth beneath these bridges to the WBH Seam is approximately 125 m; **see Figure 1d**.

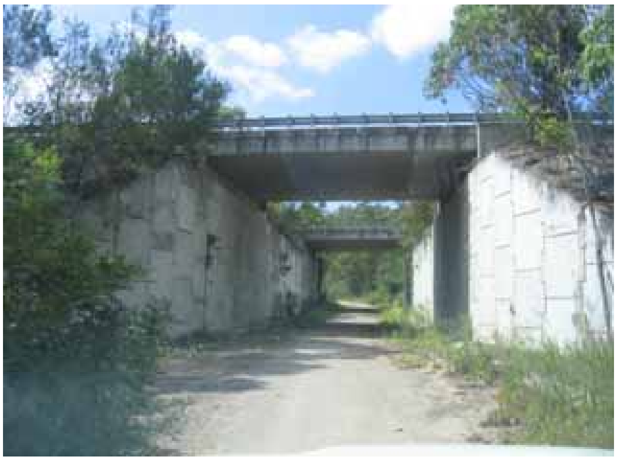


Figure 18 - Freeway Underpass Bridges in Fill 3

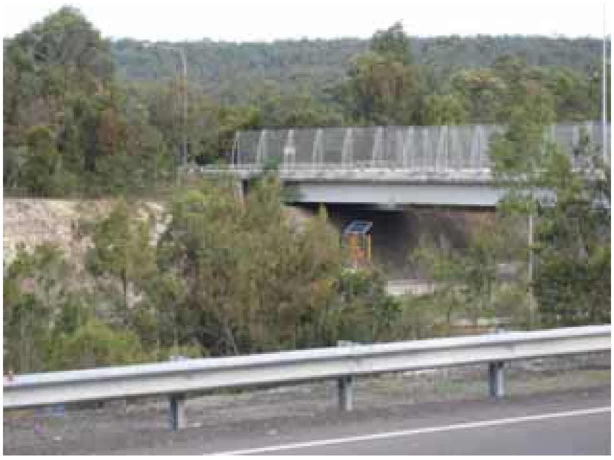


Figure 19 - Freeway Overpass Bridge at Palmers Road

6.9.13 Freeway Bridge (Overpass) at Palmers Road

Palmers Road Bridge is built over the freeway and is approximately 16.5 m wide and 77 m long between the end abutments, see **Figure 1a** and **Photo 19**. The abutments are located in cut and consist of concrete brick lined batters, which slope at 45^o. A central column provides support to the bridge span between the freeway lanes.

The Palmers Road Bridge is located approximately 1200 m to the south of LW50 and 1200 m south-west of LW44 in the Southern Domain. The cover depth beneath the end of LW50 and the bridge is 70 m and 60 m respectively; **see Figure 1d**.

6.9.14 Freeway Stormwater Detention Basins and Drainage Culverts

Other notable structural elements include 6 m lengths of rubber-ring jointed reinforced concrete pipe culverts and associated surface v-drains, entry pits and lined storm water detention basins. The culverts are located through Embankment No.s 1, 3 and 4 and range in length from 92 to 105 m.

A 10 m deep concrete lined shaft forms a storm water entry from the carriageway to the culvert in the southeastern side Embankment No 3. The shaft is approximately 120 m from the northern end of LW47 in the Southern Domain. The cover depth beneath the shaft to the WBH Seam is approximately 140 m; **see Figure 1d**.

6.10 Wakefield Road

Approximately 1.5 km of Wakefield Road (Lake Macquarie Council) is located approximately 150 m north of the finishing end of LW 44 in the Southern Domain. The road then turns towards the west across LWs 45 and 46 in the Western Domain, as shown in **Figure 1a** and **Photo 16**. The cover depth beneath the road to the WBH Seam ranges between 140 m and 160 m; **see Figure 1d**.

The road is a flexible gravel pavement with two-coat bitumen seal. As previously mentioned, approximately 1 km of the road has been formed on Embankment No. 2 and 2a, with another 250 m in cut (Cutting No. 2) and 250 m on grade above LW46. The road appeared to be in good condition with no significant cracking or rutting.

6.11 Gencom Communications Towers

Two Gencom communications towers, referred as CT1 and CT2 in **Figure 1a**, are located on the crest of the northwest ridge and outside the limits of LW43. The cover depth to the WBH seam is approximately 340 m and 360 m below CT1 and CT2 respectively; **see Figure 1c**.

Tower CT1 is located at RL 360 m (AHD) about 10 m outside the solid rib-side and 557 m from the starting position of LW43. Tower CT2 is located at RL 360 m (AHD) and located 54 m north of the north-west corner of LWs 43 starting position.

Tower CT1 is a three-legged steel frame structure that is approximately 30 m high and 2.5 m wide at the base, see **Photo 20**. The structure is supported by concrete encased bored piers and is 14 m east of a fire trail and 20 m upslope of the crest of the ridge. The ground slopes near the tower ranges from 5^o to 10^o and increase from 15^o to 20^o below the ridge crest.

Tower CT2 is a four-legged steel frame structure that is approximately 60 m high and 10 m wide at the base, see **Photo 21**. The tower is located about 30 m upslope from the crest of the steeper ridge slopes.

Photo 20 - Gencom Communications Tower (CT1)

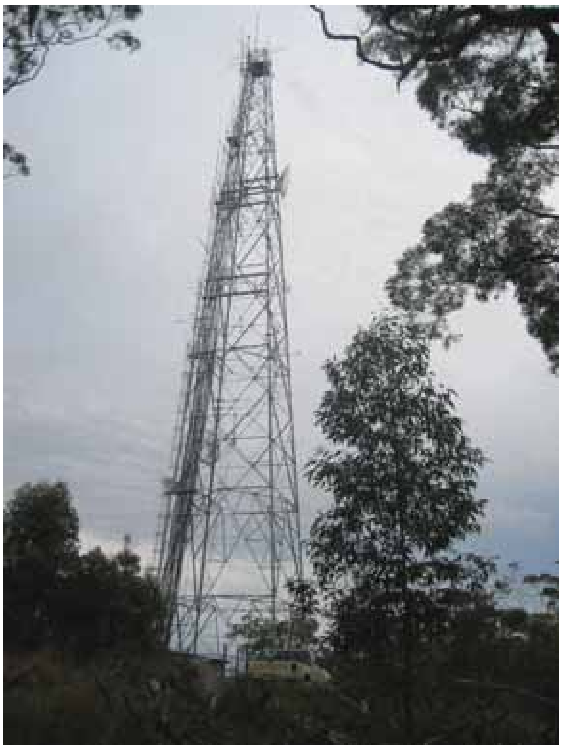


Photo 21 - Gencom Communications Tower (CT2)

A suspended power supply line has been recently installed across McCarthy's property to ultimately connect to the Gencom Towers, see **Figure 1a**. The power line consists of three conductors suspended on six, 18 m high, tapered timber poles with a 300 mm base diameter. The poles have been located above the chain pillars between LWs 38 to 40 to reduce subsidence impact. The cover depth to the WBH Seam ranges from 95 m to 115 m.

The conductors had been installed out to the last power pole at the bottom of the ridge at the time of writing. It is understood that the conductor will be installed up to the ridgeline pole before mining occurs in the area.

6.12 Telstra Communications Tower

A Telstra mobile network services tower is located approximately 135 m to the south of LW47 in the Western Domain, see **Figure 1b**. The cover depth to the WBH Seam is 140 m beneath the tower and 100 m at the end of LW47.

The tower is a four-legged steel frame structure which is approximately 40 m high and 6 m wide at the base, see **Photo 22**. The tower is connected to the network by the Tesltra optic fibre cable in the services easement discussed in **Section 6.8**.

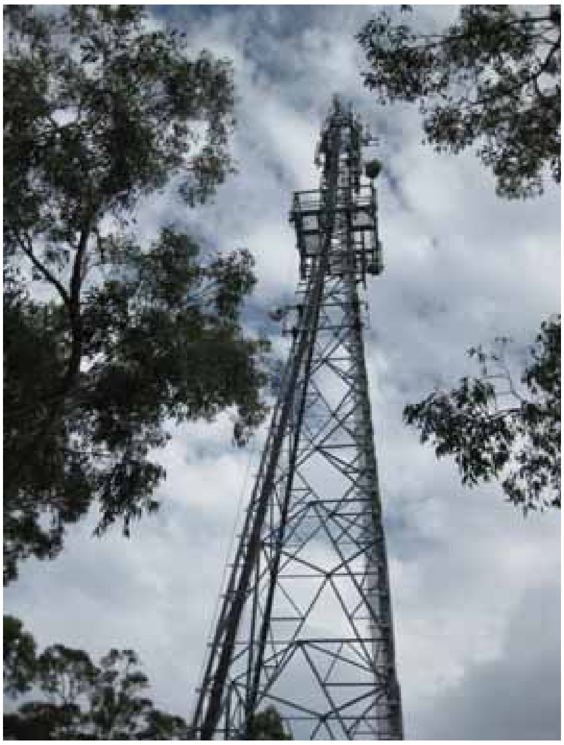


Photo 22 - Telstra Communications (Mobile Network) Tower

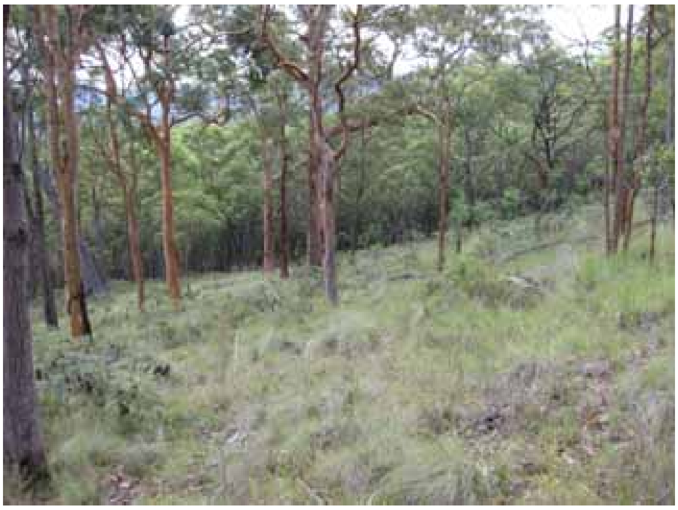


Photo 23 - Bush land Above Abandoned Bord and Pillar Workings

6.13 Abandoned Bord and Pillar Workings

An abandoned underground bord and pillar mine (circa 1880s) exists in the Great North Seam above LWs 49 and 50 in the Western Domain, see **Figure 1b**. The workings are located near the headwaters of Ryhope Creek, where cover depth to the WBH Seam ranges from 150 to 160 m.

The workings appear to have been mined in from the seam outcrop and have an estimated cover depth of 20 to 30 m. Several old track rails were surveyed below the seam outcrop. The workings are likely to have been mined at a height of 2.5 to 3 m. The interburden thickness between the GN and WBH Seam at this location is approximately 130 m.

No sensitive surface features or evidence of mine subsidence exists above the mine workings location, which consists of semi-cleared bushland and moderate slopes of 10^o to 15^o. No evidence of mine subsidence was apparent at the time of the field inspection in December 2008, see **Photo 23**.

6.14 Transgrid Transmission Towers

Several 330 kV Transgrid Towers exist 250 m to 460 m south of the Southern Domain and above previously extracted Newstan Colliery longwalls. The towers have had cruciform footings already fitted and are unlikely to be impacted by the proposed longwalls.

6.15 State Survey Marks

Several state survey control marks (NSW Department of Lands) may be affected by the proposed mining layout. It is anticipated that these marks will need to be resurveyed after subsidence movements have ceased.

7.0 Sub-Surface Conditions

7.1 Geological Setting

West Wallsend Colliery is situated within the western portion of the Newcastle Coalfield. The proposed longwalls are located within the West Borehole (WBH) Seam (i.e. coalesced Nobbys, Dudley, Yard and Borehole Seams), which represents the geological base of the Newcastle Coal Measures (**Hawley and Bunton, 1995**).

Reference to the 1:100,000 Newcastle Coalfield Geology Sheet (**DMR, 1995**) the surface terrain in the Western and Southern Domains are situated within the Moon Island Beach Sub-Group of the Newcastle Coal Measures.

Sandstone and conglomerate of the Teralba Conglomerate Formation dominate the ridges within the study area. Tuffaceous claystone (Awaba Tuff), siltstone and coal seams (Fassifern Seam) generally underlie the near surface conglomerate units. Relatively low-lying terrain with slope wash and alluvium filled gullies exist in both of the domains.

7.2 Overburden

The WBH Seam in the study area is overlain by several massive conglomerate and sandstone channel units that are summarised in **Tables 1** and **Table 2** for Western and Southern Domains respectively.

Due to the steep east-dipping terrain above the Western Domain panels, the Teralba Conglomerate Member will almost certainly outcrop above LW40 through LW50. An interpreted crop line is given in **Figure 1a**.

Table 1 - Summary of Massive Sandstone/Conglomerate Units Above the Western Domain Longwalls

Massive Channel Unit No.	Sandstone/Conglomerate Channel Units*	Strata Unit Thickness Range (m)	Distance above Extraction Horizon, y (m)	Workings Cover Depth, H (m)	y/H
1	Victoria Tunnel	0.1 - 66	0.2 - 28	76 - 293	0.01 - 0.22
2	Fern Valley	1 - 27	6 - 72	76 - 293	0.03 - 0.52
3	Montrose	1 - 45	10 - 92	76 - 293	0.04 - 0.61
4	Lower and Upper Pilot	1 - 30	62 - 143	76 - 293	0.22 - 0.92
5	Teralba Conglomerate	0 - 107	84 - 143	76 - 293	0.38 - 0.49

Note: The channel units are generally named by WWC after the immediate coal seam that they overly (except the Teralba Conglomerate, which immediately overlies the Great Northern Seam).

Table 2 – Summary of Massive Sandstone/Conglomerate Units Above Longwalls in the Southern Domain

Massive Channel Unit No.	Sandstone/Conglomerate Channel Units*	Strata Unit Thickness Range (m)	Distance above Extraction Horizon, y (m)	Workings Cover Depth, H (m)	Y/H
1	Victoria Tunnel	0.2 - 49	2 - 26	55 - 176	0.01 - 0.18
2	Fern Valley	1 - 65	7 - 60	55 - 176	0.05 - 0.38
3	Montrose	1 - 44	12 - 109	55 - 176	0.13 - 0.69
4	Lower and Upper Pilot	1 - 17	54 - 150	55 - 176	0.68 - 0.98
5	Teralba Conglomerate	Nil	N/A	N/A	N/A

Note: The channels are generally named by WWC after the immediate coal seam that they overly (except the Teralba Conglomerate, which immediately overlies the Great Northern Seam).

The interpreted channel thicknesses and their locations above the proposed longwalls are shown on the long section plots in **Figures 4** to **8**. Contours of the individual channel thickness and the distance from the bottom of the channel to the workings are shown in **Figures 9** to **18** for Channels 1 to 5 respectively. The geophysical logs of representative boreholes in the study area that were used to define the massive units are presented in **Appendix B**.

The above information was used to assess the Subsidence Reduction Potential (SRP) and maximum sag or panel subsidence above the proposed longwalls (see **Section 9.0**)

7.3 Immediate Mine Workings Conditions

The thickness of the WBH Seam in the Western Domain decreases in thickness from 4.7 m in the east to 3.5 m in the west. The seam thickness in the Southern Domain decreases from 5.1 m in the east to 4.5 m in the west (see **Figures 2a** and **2b**).

WWC will drive 3.5 m high by 5 m wide development headings in the lower three of four sections of the WBH Seam, which consists of coalesced Borehole, Yard, Dudley and Nobbys Seams (in ascending order).

The immediate roof (i.e. the first 8 m above the gate road roof horizon) within the study area generally consists of low strength Nobbys Seam, Nobbys Tuff and shale with minor mudstone, Fern Valley coal, sandstone and siltstone (UCS ranges from 10 to 25 MPa with an average of 15 MPa).

The immediate mine roof is then overlain by moderate to high strength massive sandstone and conglomerate channels up to 65 m thick (UCS ranges from 30 to 60 MPa). The distance to the first channel above the mine roof generally increases towards the south-west and ranges from 1 m to 26 m.

The floor of the development roadways will comprise 0.3 m of moderate strength carbonaceous siltstone / mudstone and sandstone (UCS ranges from 30 to 40 MPa) with low slaking potential. High strength (UCS ranges from 50 to 60 MPa) Waratah Sandstone underlies the immediate floor strata.

Based on the UCS results, the Young's Modulus for the immediate roof and floor strata is estimated to range between 3 to 15 GPa. The Young's Modulus for coal is normally assumed to range from 2 to 4 GPa.

7.4 Joints

The joints noted in the borehole logs are generally planar, rough and clean with dip angles ranging from 20^o to 80^o. The joint spacing ranges between 0.1 and 3 m, with some crush zones and fault breccia noted. The main joint sets strike at NE:SW (040^o to 060^o) and NW:SE (130^o to 165^o) and generally define the low-level cliff lines in the study area.

7.5 Bedding

Bedding in the study area generally dips towards the south-east at 2^o to 5^o with some localised bedding dips up to 15^o.

7.6 Regional Structure

Regional geological structure in the study area consists of minor to major normal faulting and igneous dyke intrusions. The structures are sub-vertical and strike at NNW:SSE, see **Figures 1a** and **1b**.

7.7 Groundwater

Groundwater inflows are low to very low with typical or average conditions described as dry to damp by OCAL geologists. The source of the groundwater is most likely to be from the Nobbys Coal Unit, which means that it is unlikely to be percolating through the Nobbys Tuff horizon or affecting this potentially moisture sensitive unit.

7.8 Horizontal Stress Regime

Major horizontal stress directions have been previously assessed in LW 28's installation road by roof guttering location observations. The stress was assessed to be orientated at NE: SW.

Triaxial stress cell measurements in 10 C/T of LW28 maingate indicated that the principal horizontal stress (sigma1) orientation was NNE: SSW (i.e. 010°: 190°) at 15 m height above the roof horizon and (sigma2) orientation was ES:NW (i.e. 100°: 280°). This is consistent with the observations of guttering locations in the roadway roof.

The horizontal stress can influence subsidence trough asymmetry and far-field horizontal displacement direction.

8.0 Description of Subsidence Development Mechanism

After the extraction of a single longwall panel, the immediate mine roof usually collapses into the void left in the seam. The overlying strata or overburden then sags down onto the collapsed material, resulting in a subsidence trough developing at the surface.

The maximum subsidence occurs in the middle of the extracted panel and is dependent on the mining height, panel width, cover depth, overburden strata strength and stiffness and bulking characteristics of the collapsed strata. For the case of single seam mining, the maximum subsidence invariably does not exceed 60% of the mining height in the NSW and QLD Coalfields, and may be lower than this value due to the spanning or bridging capability of the strata above the collapsed ground (or the goaf).

The combination of the above factors determines whether a single longwall panel will be sub-critical, critical or supercritical in terms of maximum subsidence. In the Australian coalfields, sub-critical or (spanning) behaviour generally occurs when the panel width (W) is <0.6 times the cover depth (H). If relatively thick and strong massive strata exist, then sub-critical spanning behaviour can occur for panel W/H ratios up to1.5 in the Newcastle Coalfield. The maximum subsidence for this scenario is usually significantly < 60% of the longwall extraction height and could range between 10% and 30%.

Beyond the sub-critical range, the overburden is unable to span and fails or sags down onto the collapsed or caved roof strata immediately above the extracted seam (i.e. the panel is critical or super-critical). Critical panels refer to panels with widths where maximum possible subsidence starts to develop, and supercritical panels refer to panels with widths that cause complete collapse of the overburden. In the case of super-critical panels, maximum panel subsidence does not usually continue to increase significantly with increasing panel width.

The surface effect of extracting several adjacent longwall panels is dependent on the stiffness of the overburden and the chain pillars left between the panels. Invariably, ‘extra’ subsidence occurs above a previously extracted panel and is caused primarily by the compression of the chain pillars and adjacent strata between the extracted longwall panels.

A longwall chain pillar undergoes the majority of life-cycle compression when subject to double abutment loading (i.e. the formation of goaf on either side, after two adjacent panels have been extracted). Surface survey data indicates that an extracted panel can affect the chain pillars between three or four previously extracted panels. The stiffness of the overburden and chain pillar system will determine the extent of load transfer to the preceding chain pillars. If the chain pillars go into yield, the load on the pillars will be mitigated to some extent by load transfer to adjacent fallen roof material or goaf.

The surface subsidence trough extends outside the limits of extraction for a distance assumed equal to half the depth of cover (or an angle of draw to the vertical of 26.5^{o}) in the Newcastle and QLD Coalfields.

The subsidence prediction models used in this study consider the abovementioned processes and will be further described in **Section 9.0**.

9.0 Subsidence Impact Parameter Profile and Contour Prediction Methodology

9.1 Model Background

Two empirically based prediction models (**ACARP, 2003** and **SDPS®**) have been used to generate subsidence impact parameter profiles and contours above the proposed longwall panels after mining is completed.

The subsidence predictions models used in this study are summarised below:

- **ACARP, 2003 -** An empirical model that was originally developed for predicting maximum single and multiple longwall panel subsidence, tilt, curvature and strain in the Newcastle Coalfield. The model database included measured subsidence parameters and overburden geology data which have been back analysed to predict the subsidence reduction potential (SRP) of massive lithology in terms of 'Low', 'Moderate' and 'High' SRP categories.

  The model database also includes chain pillar subsidence, inflexion point distance / subsidence, goaf edge subsidence and angle of draw prediction models, which allow subsidence profiles to be generated for any number of panels and a range of appropriate confidence limits. The Upper 95% Confidence Limit (U95%CL) has been adopted in this study for predictions of the Credible Worst-Case values.

  Sigmaplot® cubic-spline software is then used to generate the subsidence, tilt, horizontal displacement and strain profiles above the panels from the **ACARP, 2003** output.

  The **ACARP, 2003** model has been updated by DgS recently to allow the original model to be applied to other Australian Coalfields (see further below and **Appendix A** for details).

- **SDPS® -** A US developed (Virginia Polytechnical Institute) influence function model for subsidence predictions above longwalls or pillar extraction panels. The model requires calibration to measured subsidence profiles to reliably predict the subsidence and differential subsidence profiles for the assessment of impact to surface features. **Surfer 8®** graphics software is then used to generate the contours of subsidence, tilt, horizontal displacement and strain contours above the panels from the **SDPS®** output files.

  The **SDPS®** model also includes a database of percentage of hard rock (i.e. massive sandstone / conglomerate) that effectively reduces subsidence above super-critical and sub-critical panels, due to either bridging or bulking of collapsed material. An extract from the **SDPS®** User Manual that defines the parameters and terms used, is presented in **Appendix A**.

The modifications to the **ACARP, 2003** model included adjustment to the following key subsidence prediction parameters to improve compatibility with **SDPS®**:

- Chain pillar subsidence prediction is now based on pillar subsidence/extraction height (S_p/T) v. pillar stress (under double abutment loading conditions).

- Distance of the inflexion point from rib sides and inter-panel pillars in similar terms to **SDPS®** software (i.e. d/H v. W/H).

- The horizontal strain coefficient (β_s) is the linear constant used to estimate strain based on predicted curvature and is equivalent to the reciprocal of the neutral axis of bending, d_n used in **ACARP, 2003**. Based on NSW coalfield data, a value of d_n = 7.3 m or a $\beta_s = 0.136\ m^{-1}$ has been applied to predict 'smooth' profile strains using the calibrated **SDPS®** model.

9.2 Multiple-Panel Subsidence Impact Profile Prediction

Representative subsidence profiles for multiple longwall panels have been derived from the following seven key subsidence profile points and cubic spline curve fitting techniques:

(i) maximum subsidence above a longwall panel;

(ii) chain pillar subsidence between adjacent longwall panels;

(iii) inflexion point or maximum tilt location;

(iv) maximum tensile strain or convex curvature location;

(v) maximum compressive strain or concave curvature location;

(vi) goaf edge subsidence;

(vii) angle of draw to the 20 mm subsidence contour.

Multiple-panel effects are determined by adding a proportion of the chain pillar subsidence to the predicted single panel subsidence. Estimates of first and final subsidence above a given set of longwalls use this general approach. The definition of First and Final S_{max} is as follows:

First S_{max} = the total subsidence after the extraction of a longwall panel, including the effects of previously extracted longwall panels adjacent to the subject panel;

Final S_{max} = the total subsidence over an extracted longwall panel, after at least three more panels have been extracted, or when mining is completed.

In the Newcastle Coalfield, First and Final S_{max} for a panel are predicted by adding 50% and 100% of the predicted subsidence over the respective chain pillars (i.e. between the previous and current panel), less the goaf edge subsidence above the maingate.

The subsidence above chain pillars has been defined in this study as follows:

First S_p = subsidence over chain pillars after longwall panels have been extracted on both sides of the pillar;

Final S_p = the total subsidence over a chain pillar, after at least another three more panels have been extracted, or when mining is completed.

A conceptual model of multiple longwall panel subsidence mechanics and the subsidence reducing potential (SRP) of massive strata units are presented in **Figures 19** and **20**.

Residual subsidence above chain pillars and longwall panels tend to occur after extraction and caving of the immediate roof due to (i) increased overburden loading on the pillars and (ii) on-going goaf consolidation or creep. The residual movements can increase subsidence by a further 10 to 30% above chain pillars. A subsidence increase of 20% after double abutment loading occurs (i.e. First S_p) has been assumed in this study to allow for long-term loading effects (i.e. Final S_p). Residual subsidence above longwall panels will decrease exponentially as mining moves further away from a given panel.

Tilts and curvatures have been assessed using the empirical techniques presented in **ACARP, 2003** and by also assessing the first and second derivatives of the predicted subsidence profiles with **Sigmaplot®** for comparative purposes.

Predictions of strain and horizontal displacement were made based on the relationship between the measured curvatures and tilt respectively as discussed in **ACARP, 1993** and **ACARP, 2003**.

Structural and geometrical analysis theories indicate that strain is linearly proportional to the curvature of an elastic, isotropic bending ‘beam’. This proportionality actually represents the depth to the neutral axis of the beam, or in other words, half the beam thickness. **ACARP, 1993** studies returned strain over curvature ratios ranging between 6 and 11 m for NSW and Queensland Coalfields. Near surface lithology strata unit thickness and jointing therefore dictate the magnitude of the proportionality constant between curvature and strain. Similar outcomes are found for tilt and horizontal displacement.

ACARP, 2003 continued with this approach and introduced the concept of secondary curvature and strain concentration factors due to cracking. The mean peak strain / curvature ratio for the Newcastle Coalfield was assessed to equal 5.2 m with strain concentration effects increasing the ‘smooth-profile’ strains by 2 to 4 times.

On-going review of the database has lead to the median value of 7.3 being adopted as a more appropriate value for 'smooth' profile prediction purposes. A strain concentration factor of 2 to 4 may then applied to the ‘smooth profile’ value for estimating worst-case values caused by discontinuous behaviour or 'cracking'.

Cracking is only expected to occur in zones of peak tensile (or compressive) strains or when strain exceeds 1 to 2 mm/m where surface rock exposures are present within 2 to 3 m of the surface.

Alluvial soils within the project area are likely to reduce the potential for strain concentration, resulting in strain profiles close to the predicted 'smooth' subsidence profile strains presented herein.

Surface crack widths (in mm) may be estimated by multiplying the predicted strains by an empirical factor of 10 m, which is based on the distance between the pegs and observed crack widths in the field.

9.3 Validation of Models

Validation of the empirical models has involved the following methods:

(i) The development of simple analytical models of published overburden spanning mechanics and roof-chain pillar-floor system compression. The bearing capacity of the roof and floor strata was also estimated using established shallow footing design theories.

(ii) Comparison of Northern Domain Longwall subsidence data with model predictions.

The results of the studies are presented in **Section 10**.

10.0 Results of Longwall Panel Subsidence Assessment

10.1 General

Total and differential subsidence predictions have been assessed across the study area after (i) each longwall block has been extracted, and (ii) after mining of all of the proposed longwall panels 38 to 50 is complete. The assessment requires the consideration of the following:

- The subsidence reduction potential (SRP) of the overburden and the influence of proposed mining geometry on single panel subsidence development (i.e. whether the panels are likely to sub-critical, critical or super-critical);

- The behaviour of the chain pillars and immediate roof and floor system under double - abutment loading conditions when longwalls have been extracted along both sides of the pillars;

- The combined effects of single panel and chain pillar subsidence to estimate final subsidence profiles and subsidence contours for subsequent environmental impact assessment.

As mentioned previously, it is considered that the development of subsidence will be affected by the spanning potential of the massive sandstone and conglomerate units and the subsidence above the chain pillars between the panels. The outcomes of the subsidence assessment are presented in the following sections.

10.2 Geological Model and Subsidence Reduction Potential of Massive Units

The Subsidence Reduction Potential (SRP) refers to the subsidence reducing effect that massive conglomerate / sandstone units have above longwall panels due to inherent spanning or arching behaviour. The SRP is a function of the cover depth; the width of the panel (or span); the thickness of the massive unit; and the distance of the unit above the workings.

A conceptual model of the spanning potential of a massive strata unit and key parameters used in the assessment are presented in **Figure 20**.

The thickness and location of five massive strata units above the proposed workings have been plotted with the SRP threshold limit lines for the appropriate cover depth categories shown in **Figures 21a** to **21c**.

The outcomes of the assessment are summarised in **Table 3** and indicate that the massive units will have a predominately ‘High’ SRP above most of the proposed 178.6 m wide panels beneath the ridges, with some of the low lying areas (i.e. with cover depths < 120 m and thinner strata units) assessed to have a ‘Low’ SRP, see **Figure 22**.

Table 3 - Minimum Beam Thicknesses Required for 'High' and 'Moderate' SRP based on Empirical Model (ACARP, 2003)

Massive Channel Unit No.	Sandstone/ Conglomerate Channel Units*	Minimum Unit Thickness T for High SRP (West - South Domains)		Minimum Unit Thickness T for Moderate SRP (West - South Domains)		Location	
		H<150m	H>150m	H<150m	H>150 m	y (m)	y/H
1	Victoria Tunnel	27 - 28	39 - 49	27 - 28	30 - 36	0.2 - 28	0.01 - 0.22
2	Fern Valley	26 - 28	34 - 47	27 - 26	25 - 35	6 - 62	0.03 - 0.52
3	Montrose	23 - 23	23 - 32	23 - 25	23 - 35	10 - 109	0.04-0.69
4	Lower and Upper Pilot	13 - 13	12 - 14	13 - 13	10 - 12	54 -150	0.22-0.98
5	Teralba Conglomerate	-	30	-	22	83 - 143	0.38-0.49

Voussoir Beam theory presented in **Diedrichs and Kaiser, 1999** has also been applied to estimate the (i) minimum beam thickness required to span the extracted panels at various heights above the workings and (ii) maximum elastic sag subsidence above the panels.

Calculation details are presented in **Appendix B** and indicate similar outcomes to the empirical values if the following input parameters are assumed:

- a caving angle of 15^o up to the base of the massive unit (to estimate the beam length);
- an abutment angle of 21^o up to the surface (to estimate the load on the massive unit);
- an average rock mass beam strength of 50 MPa;
- a minimum elastic beam factor of safety of 1.5; yielding behaviour between an FoS between 1 and 1.5; and abutment crush or buckling at an FoS of <1.0.
- cover depth range of 70 m to 360 m with 20 m depth increments.

The minimum thickness of each channel required for it to span, based on the Voussoir Beam model used, is summarised in **Table 4**.

Table 4 - Voussoir Beam Model Outcomes

Massive Channel Unit No.	Sandstone/ Conglomerate Channel Units*	Minimum Unit Thickness for FoS > 1.5 for Elastic Beam (i.e. High SRP)		Minimum Unit Thickness for 1.0<FoS<1.5 for Yielding Beam (i.e. Moderate SRP)		Minimum Distance Above Seam	
		H<150m	H>150m	H<150m	H>150m	y (m)	y/H
1	Victoria Tunnel	40 - 47	44 - 48	33 - 39	37 - 40	8.5	0.02 - 0.1
2	Fern Valley	34 - 43	41 - 44	29 - 36	34 - 37	27	0.05 - 0.12
3	Montrose	26 - 38	37 - 40	21 - 31	31 - 33	49	0.21- 0.13
4	Lower and Upper Pilot	18 - 27	29 - 31	14 - 19	24 - 26	100	0.58- 0.83
5	Teralba Conglomerate	-	23 - 28	-	19 - 23	114	0.38- 0.49

The Voussoir Beam model indicates that the minimum thickness of the spanning units is 5 to 7 m thicker than the empirical model outcomes. However, as it would be possible to adjust the Voussoir Beam model inputs to produce a better match between the models, it is considered that the overall trend in subsidence behaviour is what is being assessed, and validates the empirical model outcomes (for worst-case predictions presented later in this study).

10.3 Predicted Maximum Single Panel Subsidence

The maximum subsidence above a single longwall panel will depend upon its width, cover depth, seam thickness, and the SRP of the overburden.

Based on reference to the **ACARP, 2003** model, the SRP categories are then used to select the appropriate subsidence prediction lines from one of three given depth categories. The predictions for single panels in the study area are shown in **Figures 23a** to **23c**.

The depth categories were developed in the **ACARP, 2003** study to cater for the influence of scale on the spanning behaviour of the massive lithological units above panels of a given geometry.

The maximum subsidence (S_{max}) for a single 178.6 m wide longwall panel at 70 to 360 m depth with ‘Low’, ‘Moderate’ and ‘High’ SRP overburden is summarised in **Table 5** for the assumed average face extraction height range from 3.1 to 4.8 m.

The values for each longwall panel were estimated along ten representative crosslines (XL1 to 10) for the Western Domain and three crosslines (XL 11 to 13) for the Southern Domain longwalls respectively, see **Figures 1a** and **1b** for their location.

The maximum subsidence estimated for the Western and Southern Domain Longwall Panels are summarised in **Table 5**.

Table 5 - Predicted Maximum Single Panel Subsidence in the Western Domain

Panel No.	Cross Line No.	Chain From Start (m)	Cover Depth H (m)	W/H	Unit t (m)	Unit Location Above Workings y (m)	Unit Location Factor y/H	SRP	Mining Height (m)	Single S_{max}* (m)	
										Mean	U95% CL
38	**1**	**177.8**	**160**	**1.11**	**25**	**125**	**0.78**	**High**	**4.35**	**1.32**	**1.54**
38	2	177.8	139	1.28	20	110	0.79	High	4.4	1.45	1.67
38	3	177.8	130	1.37	27	7	0.05	Low	4.4	1.97	2.19
38	4	177.8	110	1.62	25	7	0.06	Low	4.5	2.21	2.43
38	5	177.8	100	1.78	35	25	0.25	High	4.5	1.98	2.21
38	6	177.8	130	1.37	45	22	0.17	High	4.5	1.56	1.78
38	7	178.6	150	1.19	44	19	0.13	High	4.35	1.37	1.59
38	8	178.6	125	1.43	25	35	0.28	High	4.35	1.56	1.78
39	**1**	**178.6**	**150**	**1.19**	**27**	**120**	**0.80**	**High**	**4.25**	**1.34**	**1.55**
39	2	178.6	150	1.19	23	107	0.71	High	4.2	1.32	1.53
39	3	178.6	135	1.32	5	117	0.87	Low	4.1	1.84	2.04
39	4	178.6	95	1.88	8	55	0.58	Low	4.3	2.34	2.49
39	5	178.6	95	1.88	25	22	0.23	High	4.3	2.08	2.29
39	6	178.6	130	1.37	33	16	0.12	High	4.35	1.51	1.73
39	7	178.6	150	1.19	30	25	0.17	Low	4.35	1.94	2.16
39	8	178.6	125	1.43	22	35	0.28	Low	4.35	1.97	2.18
40	**2**	**178.6**	**140**	**1.28**	**17**	**103**	**0.74**	**High**	**3.95**	**1.30**	**1.49**
40	3	178.6	160	1.12	5	155	0.97	Low	3.95	1.77	1.97
40	4	178.6	100	1.79	5	95	0.95	Low	4.1	2.15	2.36
40	5	178.6	97	1.84	2	95	0.98	Low	4.1	2.20	2.38
40	6	178.6	125	1.43	22	16	0.13	Low	4.25	1.92	2.13
40	7	178.6	140	1.28	7	133	0.95	Low	4.25	1.90	2.11
40	8	178.6	135	1.32	18	33	0.24	Low	4.2	1.88	2.09
40	9	178.6	110	1.62	13	44	0.40	Low	4.5	2.21	2.44
41	**2**	**178.6**	**160**	**1.12**	**13**	**98**	**0.61**	**Mod**	**3.8**	**1.39**	**1.58**
41	3	178.6	160	1.12	8	93	0.58	Moderate	3.8	1.71	1.90
41	4	178.6	140	1.28	7	90	0.64	Low	3.9	1.74	1.94
41	5	178.6	140	1.28	10	130	0.93	Low	4	1.78	1.98
41	6	178.6	145	1.23	12	133	0.92	Low	4.1	1.83	2.03
41	7	178.6	160	1.12	25	125	0.78	High	4.15	1.27	1.47
41	8	178.6	140	1.28	25	115	0.82	High	4.05	1.33	1.53
41	9	178.6	90	1.98	12	38	0.42	Low	4.3	2.43	*2.49*
42	**2**	**178.6**	**240**	**0.74**	**32**	**142**	**0.59**	**High**	**3.6**	**0.25**	**0.61**
42	3	178.6	235	0.76	30	190	0.81	High	3.7	0.24	0.43
42	4	178.6	220	0.81	30	170	0.77	High	3.7	0.30	0.49
42	5	178.6	215	0.83	25	145	0.67	High	3.9	0.37	0.57
42	6	178.6	205	0.87	25	125	0.61	High	4.0	0.50	0.70
42	7	178.6	180	0.99	35	122	0.68	High	4.0	1.17	1.37
42	8	178.6	90	1.98	15	25	0.28	Low	3.85	2.17	*2.23*

Table 5 (Cont…) - Predicted Maximum Single Panel Subsidence in the Western Domain

Panel No.	Cross Line No.	Chain From Start (m)	Cover Depth H (m)	W/H	Unit t (m)	Unit Location Above Workings y (m)	Unit Location Factor y/H	SRP	Mining Height (m)	Single S_{max}* (m) Mean	Single S_{max}* (m) U95% CL
42	9	178.6	80	2.23	13	33	0.41	Low	4.0	*2.32*	*2.32*
42	10	178.6	78	2.29	15	40	0.51	Low	4.35	*2.52*	*2.52*
43	**2**	**178.6**	**340**	**0.53**	**40**	**143**	**0.42**	**High**	**3.5**	**0.32**	**0.42**
43	3	178.6	335	0.53	40	220	0.66	High	3.5	0.32	0.43
43	4	178.6	315	0.57	40	185	0.59	High	3.65	0.35	0.46
43	5	178.6	290	0.62	40	155	0.53	High	3.7	0.38	0.49
43	6	178.6	225	0.79	35	135	0.60	High	3.8	0.25	0.44
43	7	178.6	140	1.28	*13*	127	0.91	Low	3.7	1.65	1.84
43	8	178.6	120	1.49	*5*	115	0.96	Low	3.7	1.72	1.90
43	9	178.6	95	1.88	12	26	0.27	Low	3.9	2.12	*2.26*
43	10	178.6	75	2.38	15	30	0.40	Low	4.25	*2.47*	*2.47*
47	**4**	**178.6**	**315**	**0.57**	**55**	**180**	**0.57**	**High**	**3.5**	**0.34**	**0.44**
47	5	178.6	280	0.64	60	155	0.55	High	3.6	0.37	0.48
47	6	178.6	275	0.65	38	140	0.51	High	3.7	0.40	0.51
47	7	178.6	200	0.89	38	135	0.68	High	3.7	0.54	0.73
47	8	178.6	150	1.19	30	120	0.80	High	3.7	1.16	1.35
47	9	178.6	120	1.49	18	102	0.85	High	3.9	1.46	1.65
47	10	178.6	85	2.10	12	20	0.24	Low	4.1	*2.38*	*2.38*
48	**4**	**178.6**	**290**	**0.62**	**55**	**170**	**0.59**	**High**	**3.3**	**0.34**	**0.44**
48	5	178.6	260	0.69	60	150	0.58	High	3.5	0.321	0.67
48	6	178.6	240	0.74	40	150	0.63	High	3.6	0.25	0.61
48	7	178.6	250	0.71	40	140	0.56	High	3.7	0.30	0.67
48	8	178.6	190	0.94	35	130	0.68	High	3.7	0.92	1.11
48	9	178.6	110	1.62	*3*	107	0.97	Low	3.9	1.92	2.11
48	10	178.6	140	1.28	22	95	0.68	High	3.9	1.28	1.48
49	**7**	**178.6**	**185**	**0.97**	**40**	**140**	**0.76**	**High**	**3.7**	**1.07**	**1.25**
49	8	178.6	205	0.87	39	138	0.67	High	3.7	0.47	0.65
49	9	178.6	130	1.37	20	110	0.85	High	3.8	1.32	1.51
49	10	178.6	130	1.37	22	95	0.73	High	3.8	1.32	1.51
50	9	178.6	150	1.19	32	115	0.77	High	3.7	1.16	1.35

Note:
XL # - Refer to **Figures 1a** and **1b**.
Bold - Prediction outcomes for first LW panel beneath a given crossline.
SRP = refers to Subsidence Reduction Potential of the assumed strata unit for the purposes of subsidence prediction.
Single S_{max}= maximum surface subsidence predicted for a single, isolated longwall panel.
Italics - Maximum subsidence limited to 58% of mining height (refer to **ACARP, 2003**).

Table 6 - Predicted Maximum Single Panel Subsidence in the Southern Domain

Panel No.	Cross Line No.	Chain From Start (m)	Cover Depth, H (m)	W/H	Unit t (m)	Unit Location Above Workings y (m)	Unit Location Factor y/H	SRP	Mining Height (m)	Single S_{max}* (m)	
										Mean	U95%CL
44	**7.11**	**425**	**165**	**1.08**	**55**	**17**	**0.10**	**High**	**4.7**	**1.42**	**1.65**
44	7.12	770	125	1.43	55	12	0.10	High	4.7	1.69	1.92
45	**8.11**	**360**	**180**	**0.99**	**50**	**27**	**0.15**	**High**	**4.7**	**1.38**	**1.61**
45	8.12	720	130	1.37	55	23	0.18	High	4.7	1.63	1.87
45	8.13	1160	120	1.49	35	19	0.16	High	4.7	1.75	1.99
46	**9.11**	**320**	**180**	**0.93**	**35**	**40**	**0.22**	**High**	**4.7**	**1.11**	**1.35**
46	9.12	660	145	1.16	40	43	0.30	High	4.7	1.44	1.67
46	9.13	1100	150	1.12	35	30	0.20	High	4.7	1.41	1.65

Note:
XL # - Refer to **Figures 1a** and **1b**.
Bold - Prediction outcomes for first LW panel beneath a given crossline.
SRP = refers to Subsidence Reduction Potential of the assumed strata unit for the purposes of subsidence prediction.
Single S_{max}= maximum surface subsidence predicted for a single, isolated longwall panel.
Italics - Maximum subsidence limited to 58% of mining height (refer to **ACARP, 2003**).

The results of the above assessment indicate that the maximum single panel subsidence is likely to range between 0.25 and 2.03 m (7% to 58% of the mining height) in the Western Domain and between 1.11 and 1.88 m (24% to 42% of the mining height) in the Southern Domain.

The single panel subsidence predictions will be used with the chain pillar and goaf edge subsidence to estimate the multi-panel subsidence in **Section 10.6**.

10.4 Maximum Predicted Subsidence Above Chain Pillars

10.4.1 Empirical Model Development

The predicted subsidence values above the chain pillars have been estimated based on an empirical model and an analytical model of the roof-pillar-floor system.

The empirical model has been developed from measured subsidence data over chain pillars (S_p) divided by the mining height (T) v. the total pillar stress after longwall panel extraction on both sides.

The estimate of the total stress acting on the chain pillars on each side of the panel under double abutment loading conditions is based on the abutment angle concept described in **ACARP, 1998a**. The total stress acting on each pillar of a chain pillar pair after mining was estimated as follows:

σ = pillar load/area = $(P+A_1+A_2)/wl$

where:

P = full tributary area load of column of rock above each pillar;

= $(l+ r)(w + r).\rho.g.H$;

$A_{1,2}$ = total abutment load from each side of pillar in MN/m, and

= $(l+r)\rho g(0.5W'H - W'^2/8\tan\phi)$ (for sub-critical panel widths) or

= $(l+r)(\rho gH^2\tan\phi)/2$ (for super-critical panel widths);

w = pillar width (solid);

l = pillar length;

r = roadway width;

H = depth of cover;

ϕ = abutment angle (normally 21° adopted for cover depths < 360 m at West Wallsend);

W' = effective panel width (rib to rib distance minus the roadway width).

A panel is deemed sub-critical when $W'/2 < H\tan\phi$.

As presented in **ACARP, 1998b** the FoS of the chain pillars were based on the strength formula for 'squat' pillars with w/h ratios > 5 as follows:

$$S = 27.63\Theta^{0.51}(0.29((w/5h)^{2.5} - 1) + 1)/(w^{0.22}h^{0.11})$$

where:

h = pillar height;

Θ = a dimensionless 'aspect ratio' factor or w/h ratio in this case.

The FoS was calculated by dividing the pillar strength, S, with the pillar stress, σ.

10.4.2 Empirical Model Outcomes

The Mean and Upper 95%CL values for the proposed chain pillar subsidence with solid widths of 30 m, 35 m and 45 m under double abutment loading conditions after mining is completed, are summarised in **Tables 7A** and **7B** for the Western and Southern Domains respectively.

The predicted first subsidence over the chain pillars (S_p) between the first and next extracted panels is estimated to range from 0.06 m to 0.68 m +/- 0.048T (Upper and Lower 95% Confidence Limits) for the range of pillar sizes and geometries proposed. The final subsidence over the chain pillars after mining is completed has been estimated by increasing the first chain pillar subsidence by 20% or to a range of 0.07 m to 0.81 m +/- 0.048T.

The empirical model also indicates that for pillar stress over 30 MPa, the subsidence does not increase significantly, which suggests that pillars in the database were yielding and re-distributing load to the adjacent goaf. The same pillar subsidence database was plotted v. 1/FoS (or pillar stress / strength) in **Figures 24b** and **24c** and shows that a pillar stress of about 30 MPa corresponds with an FoS of 1.0 for the West Wallsend chain pillars.

It is also apparent from the measured data **Figure 24a** that the subsidence above the pillars is a function of the strength and stiffness of the coal and surrounding rock mass (i.e. higher subsidence is measured above weak shale roof compared to a strong sandstone one).

Table 7A - Predicted Chain Pillar Subsidence in the Western Domain under Double Abutment Loading (based on Modified ACARP, 2003 Empirical Model)

LW #	XL #	Pillar Width (m)	Cover Depth H (m)	Pillar Stress (MPa)	Pillar FoS	Sp First (mean)	Sp First (U95%CL)	Sp Final (mean)	Sp Final (U95%CL)
38	**1**	**35**	**160**	**11.96**	**2.63**	**0.15**	**0.17**	**0.18**	**0.20**
38	2	35	135	9.31	3.38	0.12	0.14	0.14	0.16
38	3	35	130	8.59	3.66	0.11	0.13	0.13	0.15
38	4	35	110	6.35	4.96	0.08	0.10	0.10	0.12
38	5	35	100	5.61	5.61	0.08	0.10	0.09	0.11
38	6	35	130	8.78	3.59	0.11	0.33	0.13	0.35
38	7	35	155	10.95	2.88	0.14	0.36	0.17	0.39
38	8	35	130	8.09	3.90	0.10	0.32	0.12	0.34
39	**1**	**300**	**155**	**4.53**	**380**	**0.00**	**0.00**	**0.00**	**0.00**
39	2	30	135	10.47	2.48	0.13	0.15	0.15	0.17
39	3	30	125	11.29	2.30	0.13	0.15	0.16	0.18
39	4	30	95	6.10	4.26	0.08	0.10	0.09	0.11
39	5	30	90	5.79	4.49	0.07	0.09	0.09	0.11
39	6	30	130	9.22	2.82	0.11	0.32	0.14	0.34
39	7	30	140	10.86	2.39	0.14	0.34	0.16	0.37
39	8	30	110	8.23	3.16	0.10	0.31	0.12	0.33
40	2	**30**	**140**	**11.83**	**2.21**	**0.14**	**0.16**	**0.16**	**0.18**
40	3	30	170	14.85	1.76	0.19	0.38	0.23	0.42
40	4	30	100	8.00	3.27	0.09	0.11	0.11	0.13
40	5	30	100	8.00	3.27	0.09	0.11	0.11	0.13
40	6	30	120	9.59	2.73	0.11	0.32	0.14	0.34
40	7	30	140	11.83	2.21	0.15	0.35	0.18	0.38
40	8	30	130	10.07	2.60	0.12	0.32	0.14	0.35
40	9	30	110	6.73	3.89	0.09	0.30	0.10	0.32
41	**2**	**35**	**160**	**16.73**	**1.89**	**0.22**	**0.40**	**0.26**	**0.45**
41	3	35	170	16.86	1.88	0.22	0.41	0.27	0.45

Table 7A (Cont…) - Predicted Chain Pillar Subsidence in the Western Domain under Double Abutment Loading (based on Modified ACARP, 2003 Empirical Model)

LW #	XL #	Pillar Width (m)	Cover Depth H (m)	Pillar Stress (MPa)	Pillar FoS	Sp First (mean)	Sp First (U95%CL)	Sp Final (mean)	Sp Final (U95%CL)
41	4	35	140	13.36	2.37	0.16	0.18	0.19	0.21
41	5	35	140	13.36	2.37	0.16	0.18	0.20	0.22
41	6	35	145	13.12	2.42	0.16	0.36	0.20	0.39
41	7	35	160	13.14	2.41	0.17	0.37	0.20	0.40
41	8	35	140	8.31	3.81	0.09	0.29	0.11	0.31
41	9	35	90	4.78	6.63	0.07	0.27	0.08	0.29
42	**2**	**35**	**240**	**29.47**	**1.09**	**0.54**	**0.71**	**0.64**	**0.82**
42	3	35	230	28.52	1.12	0.53	0.70	0.63	0.81
42	4	35	210	25.37	1.26	0.44	0.61	0.52	0.70
42	5	35	210	24.07	1.33	0.42	0.61	0.51	0.69
42	6	35	200	19.29	1.66	0.30	0.49	0.35	0.55
42	7	35	180	12.83	2.49	0.16	0.35	0.19	0.38
42	8	35	95	6.03	5.30	0.07	0.25	0.08	0.27
42	9	35	80	4.74	6.74	0.06	0.25	0.07	0.27
42	10	35	78	4.01	7.98	0.06	0.27	0.07	0.28
43	**2**	**solid**	**330**	**n/a**	**n/a**	**0.00**	**0.00**	**0.00**	**0.00**
43	3	solid	330	n/a	n/a	0.00	0.00	0.00	0.00
43	4	35	310	35.13	0.91	0.68	0.85	0.81	0.99
43	5	35	290	30.96	1.03	0.59	0.77	0.71	0.89
43	6	35	230	24.63	1.30	0.43	0.61	0.51	0.70
43	7	35	140	12.73	2.51	0.14	0.32	0.17	0.35
43	8	35	115	9.00	3.55	0.09	0.27	0.11	0.29
43	9	35	100	6.49	4.93	0.07	0.26	0.09	0.28
43	10	35	75	3.97	8.06	0.06	0.26	0.07	0.27
47	**4**	**35**	**315**	**33.98**	**0.94**	**0.63**	**0.79**	**0.75**	**0.92**
47	5	35	280	11.77	1.11	0.52	0.69	0.62	0.79
47	6	35	270	26.48	1.21	0.47	0.65	0.56	0.74
47	7	35	200	20.59	1.55	0.31	0.48	0.37	0.54
47	8	35	160	13.63	2.35	0.16	0.33	0.19	0.37
47	9	35	120	7.39	4.33	0.08	0.27	0.10	0.29
47	10	35	80	5.73	5.58	0.07	0.27	0.08	0.28
48	**4**	**solid**	**290**	**n/a**	**n/a**	**0.00**	**0.00**	**0.00**	**0.00**
48	5	solid	260	n/a	n/a	0.00	0.00	0.00	0.00
48	6	solid	240	n/a	n/a	0.00	0.00	0.00	0.00
48	7	35	250	21.00	1.52	0.32	0.49	0.38	0.56
48	8	35	190	16.59	1.93	0.21	0.39	0.25	0.43
48	9	35	110	7.45	4.30	0.08	0.27	0.10	0.29
48	10	35	130	8.73	3.66	0.10	0.28	0.11	0.30
49	**7**	**solid**	**180**	**n/a**	**n/a**	**0.00**	**0.00**	**0.00**	**0.00**
49	8	solid	200	n/a	n/a	0.00	0.00	0.00	0.00
49	9	35	130	9.98	3.20	0.11	0.29	0.13	0.31
49	10	300	130	3.74	470.97	0.00	0.00	0.00	0.00
50	**9**	**solid**	**160**	**n/a**	**n/a**	**0.00**	**0.00**	**0.00**	**0.00**

Table 7B - Predicted Chain Pillar Subsidence in the Southern Domain (based on Modified ACARP, 2003 Empirical Model)

LW #	XL #	Pillar Width (m)	Cover Depth H (m)	Pillar Stress (MPa)	Pillar FoS under DA Loading	Sp First (mean)	Sp First (U95%CL)	Sp Final (mean)	Sp Final (U95%CL)
44	**11**	**45**	**165**	**11.47**	**4.07**	**0.16**	**0.18**	**0.19**	**0.21**
44	12	45	125	7.28	6.41	0.10	0.12	0.12	0.14
45	**11**	**45.1**	**180**	**12.45**	**3.79**	**0.18**	**0.20**	**0.22**	**0.24**
45	12	40.3	130	8.55	4.61	0.12	0.14	0.14	0.16
45	13	34.2	120	9.01	3.44	0.12	0.14	0.15	0.17
46	**11**	**solid**	**180**	**n/a**	**n/a**	**0.00**	**0.00**	**0.00**	**0.00**
46	12	solid	145	n/a	n/a	0.00	0.00	0.00	0.00
46	13	solid	150	n/a	n/a	0.00	0.00	0.00	0.00

DA = Double abutment.

The observed behaviour of the chain pillars and roof / floor system has also been used to develop a simple analytical model that includes elastic and post-yielded pillar responses to estimate subsidence based on laboratory testing data and reference to **ACARP, 2005**.

10.4.3 Analytical Model Development

The compression of the chain pillars and immediate roof and floor strata has also been estimated using two relatively simple analytical models. The purpose of this exercise is to check that the empirical model predictions are reasonable compared to analytical predictions, based on the range of measured physical parameters of the rock mass and coal seam.

Given that the stress on the chain pillars may exceed the in-situ strength of the coal and/or roof / floor materials, the analytical models needed to consider both the elastic and post-yield stiffness moduli of the pillar-roof-floor system.

The bearing capacity of the roof/floor strata and chain pillar strength was firstly checked before appropriate rock mass Youngs Modulii values were assigned for subsidence prediction under the assessed loading conditions.

Reference to **Pells *et al*, 1998** indicates that the bearing capacity of sedimentary rock under shallow footing type loading conditions is 3 to 5 times its UCS strength. Based on the estimated range of UCS values of 50 MPa and 15 MPa in the immediate floor and roof strata respectively, the general bearing capacity of the strata is estimated to range between 75 and 150 MPa.

Considering the average chain pillar stress values predicted ranged from 3.7 to 35 MPa, an overall FoS against average roof and floor bearing failure strength of 112.5 MPa ranges between 3.2 and 30 for the range of pillar widths, which is likely to be within the elastic behaviour range for these materials (i.e. if the FoS is > 2.5 or stress is < 40% of pillar strength).

The FoS of the assumed 3.5 m high chain pillars will range between 0.91 and 7.98 for the range of mining geometries and are likely to either behave elastically or go into yield.

Reference to **Figure 24d** indicates that the proposed chain pillars (that will have w/h ratios > 8) would be expected to strain-harden if they are over-loaded and go into yield. The post-yield stiffness of the coal pillars has been assumed to equal 15% of the peak Young's Modulus value of 3 GPa (i.e. 450 MPa) and limit subsidence to within the observed range of subsidence values for Australian longwall mines, as presented in **Figure 24a**. The strain-hardening behaviour will also allow pillar stress to increase beyond 30 MPa (albeit at a lower than elastic rate).

The roof and floor strata FoS values estimated in **Section 10.4.2** indicate that the compression of these materials may be estimated using laboratory test results that have been adjusted to reflect the stiffness of the overall rock mass.

Average rock mass elastic moduli for the floor and roof materials within the significant area of influence of the pillars (i.e. approximately the pillar width or 30 to 35 m above and below the pillars) were estimated based on the laboratory data and the relationship established by **Hoek and Diederichs, 2006** below:

$$E_{rockmass} = E_{laboratory}(0.02+1/(1+e^{(60-GSI)/11})$$

The upper and lower bound Young's Modulus for each of the above have been estimated for an assessed Geological Strength Index (GSI) range of 50 to 60 (blocky to very blocky strata with good bedding party surface quality (i.e. rough, slightly weathered) as follows:

$$E_{rockmass} = 0.4 - 0.5E_{laboratory}$$

E_{roof} = 2 - 7.5 GPa (for an estimated laboratory stiffness range 4.5 to 15 GPa)

E_{floor} = 6 - 7.5 GPa (for an estimated laboratory stiffness range of 15 GPa)

E_{coal} = 2 GPa (for an estimated laboratory stiffness range of 2 - 4 GPa)

In the coal mining industry, strain-hardening response of goaf is also normally assumed to develop, with Young's Moduli increasing exponentially up to and beyond the virgin stress (refer to LaModel® Version 2.1.1 Users Manual extract in **Appendix B**). The stress-strain (σ-ε) curve for the strain-hardening goaf model used in this study is presented below:

$$\sigma_g = a[e^{b\varepsilon} - 1]$$

where

$$a = E_i \sigma_v/(E_f-E_i)$$

$$b = (E_f-E_i)/\sigma_v$$

σ_v = virgin vertical stress or a maximum stress of 27 MPa (refer **Appendix B**)

E_i = initial Young's Modulus

E_f = final Young's Modulus

ε = rubble strain at seam level = c/nT

n = ratio of goaf or rubble thickness/seam thickness or mining height

T = mining height

c = roof convergence at seam level

There is usually a small amount of void between the top of the collapsed roof rubble and overburden, which must be closed before the rubble starts to load up (i.e. system 'slackness'). The author of the LaModel® program suggests a typical initial Young's Modulus (E_i) of 0.7 MPa for the goaf, and a maximum goaf stress limit of 27 MPa to model the field conditions reasonably.

The value of 'n' and Final Young's Modulus assumed are the key variables required for calibrating the goaf model to measured maximum subsidence above extracted longwall panels. For an n = 4, the E_f values for the given depths of cover range between 15 MPa and 900 MPa (see **Figure 24e**) and the chain pillar will not start to shed load to the goaf until the goaf develops similar stiffness to the yielded pillar (i.e. $E_f > 450$ MPa). **Figure 24e** indicates that this won't occur until the cover depth is greater than 280 m. The load shed to the goaf will then increase linearly and limit stress on the chain pillars to about 45 MPa or less, as the values in the empirical model database suggests.

The compression of the pillars in the elastic and post-yielded regimes has been calculated by assuming the pillar will behave like a spring under load and then strain-harden as follows:

$$s_{pillar} = \sigma_{net}T_s/E_c + (\sigma_{max} - S_p)T_s/0.15E_c \qquad (1)$$

where:

s_{pillar} = pillar compression;

σ_{net} = pillar stress increase = total pillar stress - virgin stress;

T_s = Seam thickness;

E_c = Young's Modulus of Coal;

σ_{max} = Maximum stress on pillar after load redistribution to the goaf (if applicable).

S_p = Pillar strength (**ACARP, 1998b**)

The analytical model adopted to estimate the immediate compression of the floor and roof was taken from Boussinesq's elastic pressure bulb theory beneath strip footings of varying aspect ratio, see **Das, 1998**:

$$s_{roof} = \sigma_{net}\ w(1\text{-}v^2)I/E_{roof} \qquad (2)$$

$$s_{floor} = \sigma_{net}\ w(1\text{-}v^2)I/E_{floor} \qquad (3)$$

where:

s_{roof} = roof compression above pillar;

s_{floor} = floor compression below pillar;

σ_{net} = net pillar stress increase (= total stress - effective virgin stress);

w = pillar width;

E_{roof} = average Young's Modulus of roof material for a distance of W above the pillar;

E_{floor} = average Young's Modulus of floor material for a distance of w below the pillar;

v = Poisson's Ratio;

I = Influence function for various footing shape geometries.

The estimate of long-term surface subsidence (s_{total}) above a pillar subject to the assumed loading may be estimated by summing equations (1), (2) and (3):

$$s_{total} = s_{pillar} + s_{roof} + s_{floor}$$

10.4.4 Analytical Model Outcomes

Lower and Upper Bound chain pillar subsidence predictions were determined for the case when pillar height is equal to the face extraction height (i.e. after goaf development) and compared to the empirical model values in **Figures 24g** and **24h**.

The upper bound chain pillar subsidence predictions are presented in **Table 9** with full calculation details presented in **Appendix C**.

Table 9 - Worst-Case Analytical Subsidence Predictions Above the Proposed Chain Pillars

Cover Depth (m)	Virgin Stress (MPa)	Applied Pillar Stress (MPa)	Pillar FoS Under Final Loading	Subsidence Predictions Based on Non-Linear Pillar and Strata System Compression (m)			
				Pillar	Roof	Floor	Total (Lower /Upper Bounds)*
Pillar width = 30 m							
100	2.50	6.5	2.94	0.01	0.04	0.01	0.06 / 0.12
110	2.75	7.5	2.71	0.01	0.04	0.02	0.07 / 0.14
120	3.00	8.6	2.09	0.01	0.05	0.02	0.09 / 0.18
130	3.25	9.8	1.96	0.01	0.06	0.02	0.10 / 0.20
140	3.50	11.0	1.74	0.02	0.07	0.03	0.11 / 0.22
150	3.75	12.3	1.66	0.02	0.08	0.03	0.13 / 0.26
160	4.00	13.6	1.40	0.02	0.09	0.04	0.15 / 0.30
180	4.50	16.6	1.23	0.02	0.11	0.05	0.18 / 0.36
Pillar width = 35 m							
70	1.75	3.55	5.48	0.00	0.03	0.008	0.04 / 0.08
100	2.50	5.94	2.79	0.01	0.04	0.02	0.06 / 0.12
120	3.00	7.83	2.03	0.01	0.05	0.02	0.09 / 0.17
130	3.25	8.86	1.79	0.01	0.06	0.02	0.10 / 0.20
150	3.75	11.09	1.61	0.02	0.08	0.03	0.13 / 0.26
175	4.38	14.20	1.12	0.03	0.11	0.04	0.18 / 0.36
200	5.00	17.67	1.79	0.02	0.14	0.06	0.21 / 0.42
220	5.50	20.71	1.53	0.02	0.17	0.07	0.26 / 0.52
250	6.25	25.71	1.23	0.03	0.21	0.09	0.33 / 0.66
270	6.75	29.33	1.08	0.04	0.25	0.10	0.38 / 0.76
285	7.13	32.19	0.98	0.04	0.27	0.11	0.42 / 0.84

* Upper bound = 2 x Lower Bound

The results of the analytical subsidence prediction analysis for the lower bound material properties and cover depth ranges indicate that the worst-case subsidence over the proposed chain pillars will range between 0.04 and 0.84 m after mining is completed. The results generally plot between the mean and U95%CL values, which are therefore considered reasonable for subsequent impact analysis purposes.

10.5 Goaf Edge Subsidence Prediction

The mean and U95%CL goaf edge subsidence predictions for the proposed panels and cover depth range of 75 m to 340 m are 0.04 to 0.37 m and 0.13 m to 0.61 m respectively.

The goaf edge predictions are based on the prediction curves shown in **Figure 25** and the Maximum Subsidence predictions given in **Section 10.6**.

10.6 Multiple Panel Subsidence Prediction Results

Based on the predicted maximum single panel, chain pillar and goaf edge subsidence values derived from the **ACARP, 2003** model, the empirically derived mean and U95%CL values of first and final maximum multi-panel subsidence and associated impact parameters are presented in **Tables 10A** to **11B** for the western and southern domain longwall (LWs 38 to 50).

The mean subsidence predictions are summarised in **Tables 10A** and **10B**.

The U95%CL subsidence predictions are summarised in **Tables 11A** and **11B**.

Table 10A - Predicted First and Final Subsidence Parameters (Mean Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max[#] Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
38	**1**	**177.8**	**160**	**4.35**	**1.11**	**4**	**25**	**125**	**0.78**	**High**	**35**	**1.32**	**1.43**	**0.15**	**0.18**	**23**	**7**	**5**
38	2	177.8	139	4.4	1.28	1	20	110	0.79	High	35	1.45	1.54	0.13	0.15	26	7	6
38	3	177.8	130	4.4	1.37	1	27	7	0.05	Low	35	1.97	2.04	0.11	0.13	39	9	7
38	4	177.8	110	4.5	1.62	1	25	7	0.06	Low	35	2.21	2.24	0.08	0.10	55	14	11
38	5	177.8	100	4.5	1.78	3	35	25	0.25	High	35	1.98	2.01	0.08	0.09	54	15	12
38	6	177.8	130	4.5	1.37	3	45	22	0.17	High	35	1.56	1.64	0.11	0.13	28	7	6
38	7	178.6	150	4.35	1.19	3	44	19	0.13	High	30	1.37	1.49	0.16	0.19	25	7	5
38	8	178.6	125	4.35	1.43	3	25	35	0.28	High	30	1.56	1.65	0.11	0.13	29	8	6
39	**1**	**178.6**	**150**	**4.25**	**1.19**	**4**	**27**	**120**	**0.80**	**High**	**300**	**1.38**	**1.39**	**0.00**	**0.00**	**22**	**6**	**5**
39	2	178.6	150	4.2	1.19	4	23	107	0.71	High	30	1.35	1.46	0.14	0.17	24	7	5
39	3	178.6	135	4.1	1.32	4	5	117	0.87	Low	30	1.86	1.96	0.14	0.16	37	9	7
39	4	178.6	95	4.3	1.88	3	8	55	0.58	Low	30	2.35	2.37	0.08	0.09	74	19	15
39	5	178.6	95	4.3	1.88	3	25	22	0.23	High	30	2.08	2.11	0.08	0.09	63	17	14
39	6	178.6	130	4.35	1.37	3	33	16	0.12	High	30	1.54	1.63	0.12	0.14	28	7	6
39	7	178.6	150	4.35	1.19	3	30	25	0.17	Low	30	1.97	2.06	0.15	0.18	39	9	7
39	8	178.6	125	4.35	1.43	3	22	35	0.28	Low	30	1.99	2.07	0.12	0.14	41	10	8
40	**2**	**178.6**	**140**	**3.95**	**1.28**	**4**	**17**	**103**	**0.74**	**High**	**30**	**1.34**	**1.46**	**0.14**	**0.16**	**24**	**7**	**5**
40	3	178.6	160	3.95	1.12	5	5	155	0.97	Low	30	1.79	1.89	0.16	0.20	35	9	7
40	4	178.6	100	4.1	1.79	4	5	95	0.95	Low	30	2.16	2.20	0.09	0.11	62	16	13

Table 10A (Cont…) - Predicted First and Final Subsidence Parameters (Mean Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
40	5	178.6	97	4.1	1.84	4	2	95	0.98	Low	30	2.20	2.24	0.09	0.11	67	18	14
40	6	178.6	125	4.25	1.43	3	22	16	0.13	Low	30	1.95	2.03	0.12	0.14	40	10	8
40	7	178.6	140	4.25	1.28	5	7	133	0.95	Low	30	1.93	2.04	0.15	0.18	39	9	7
40	8	178.6	135	4.2	1.32	3	18	33	0.24	Low	30	1.91	1.99	0.13	0.15	37	9	7
40	9	178.6	110	4.5	1.62	3	13	44	0.40	Low	30	2.21	2.25	0.09	0.10	56	14	11
41	**2**	**178.6**	**160**	**3.8**	**1.12**	**4**	**13**	**98**	**0.61**	**Moderate**	**35**	**1.42**	**1.61**	**0.22**	**0.26**	**27**	**7**	**6**
41	3	178.6	160	3.8	1.12	4	8	93	0.58	Moderate	35	1.74	1.90	0.21	0.26	35	9	7
41	4	178.6	140	3.9	1.28	4	7	90	0.64	Low	35	1.75	1.89	0.17	0.21	35	9	7
41	5	178.6	140	4	1.28	5	10	130	0.93	Low	35	1.80	1.93	0.17	0.20	36	9	7
41	6	178.6	145	4.1	1.23	5	12	133	0.92	Low	35	1.85	1.97	0.17	0.20	37	9	7
41	7	178.6	160	4.15	1.12	5	25	125	0.78	High	35	1.31	1.43	0.17	0.20	23	6	5
41	8	178.6	140	4.05	1.28	5	25	115	0.82	High	35	1.37	1.43	0.09	0.11	23	6	5
41	9	178.6	90	4.3	1.98	3	12	38	0.42	Low	35	2.43	2.45	0.07	0.08	84	22	18
42	**2**	**178.6**	**240**	**3.6**	**0.74**	**5**	**32**	**142**	**0.59**	**High**	**35**	**0.34**	**0.96**	**0.55**	**0.66**	**13**	**4**	**3**
42	3	178.6	235	3.7	0.76	5	30	190	0.81	High	35	0.33	0.94	0.55	0.66	13	4	3
42	4	178.6	220	3.7	0.81	5	30	170	0.77	High	35	0.37	0.89	0.47	0.57	12	4	3
42	5	178.6	215	3.9	0.83	5	25	145	0.67	High	35	0.44	0.91	0.44	0.52	12	4	3
42	6	178.6	205	4	0.87	5	25	125	0.61	High	35	0.56	0.86	0.30	0.36	11	4	3
42	7	178.6	180	4	0.99	5	35	122	0.68	High	35	1.21	1.31	0.16	0.19	20	6	5

Table 10A (Cont…) - Predicted First and Final Subsidence Parameters (Mean Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
42	8	178.6	90	3.85	1.98	3	15	25	0.28	Low	35	2.19	2.20	0.07	0.08	72	20	16
42	9	178.6	80	4	2.23	3	13	33	0.41	Low	35	2.32	2.32	0.06	0.07	93	27	21
42	10	178.6	78	4.35	2.29	3	15	40	0.51	Low	35	2.52	2.52	0.06	0.07	108	31	24
43	**2**	**178.6**	**340**	**3.5**	**0.53**	**5**	**40**	**143**	**0.42**	**High**	**239**	**0.53**	**0.56**	**0.00**	**0.00**	**6**	**3**	**2**
43	3	178.6	335	3.5	0.53	5	40	220	0.66	High	239	0.54	0.56	0.00	0.00	6	3	2
43	4	178.6	315	3.65	0.57	5	40	185	0.59	High	35	0.53	1.19	0.68	0.82	18	5	4
43	5	178.6	290	3.7	0.62	5	40	155	0.53	High	35	0.55	1.12	0.59	0.71	16	5	4
43	6	178.6	225	3.8	0.79	5	35	135	0.60	High	35	0.39	0.84	0.42	0.51	11	4	3
43	7	178.6	140	3.7	1.28	5	*13*	127	0.91	Low	35	1.70	1.80	0.14	0.17	32	8	6
43	8	178.6	120	3.7	1.49	5	*5*	115	0.96	Low	35	1.73	1.78	0.09	0.11	35	9	7
43	9	178.6	95	3.9	1.88	3	12	26	0.27	Low	35	2.15	2.17	0.07	0.09	65	18	14
43	10	178.6	75	4.25	2.38	3	15	30	0.40	Low	35	2.47	2.47	0.06	0.07	111	32	25
47	**4**	**178.6**	**315**	**3.5**	**0.57**	**5**	**55**	**180**	**0.57**	**High**	**35**	**0.63**	**1.18**	**0.63**	**0.75**	**18**	**5**	**4**
47	5	178.6	280	3.6	0.64	5	60	155	0.55	High	35	0.63	1.10	0.52	0.62	16	5	4
47	6	178.6	275	3.7	0.65	5	38	140	0.51	High	35	0.57	1.02	0.48	0.58	14	5	4
47	7	178.6	200	3.7	0.89	5	38	135	0.68	High	35	0.59	0.90	0.31	0.37	12	4	3
47	8	178.6	150	3.7	1.19	5	30	120	0.80	High	35	1.18	1.30	0.14	0.17	20	6	5
47	9	178.6	120	3.9	1.49	5	18	102	0.85	High	35	1.47	1.52	0.08	0.10	28	8	6
47	10	178.6	85	4.1	2.10	3	12	20	0.24	Low	35	2.38	2.38	0.08	0.09	88	24	19

Table 10A (Cont…) - Predicted First and Final Subsidence Parameters (Mean Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
48	**4**	**178.6**	**290**	**3.3**	**0.62**	**5**	**55**	**170**	**0.59**	**High**	**solid**	**0.61**	**0.63**	**0.00**	**0.00**	**7**	**3**	**2**
48	5	178.6	260	3.5	0.69	5	60	150	0.58	High	solid	0.55	0.57	0.00	0.00	6	3	2
48	6	178.6	240	3.6	0.74	5	40	150	0.63	High	solid	0.48	0.49	0.00	0.00	5	2	2
48	7	178.6	250	3.7	0.71	5	40	140	0.56	High	35	0.43	0.74	0.32	0.39	9	3	3
48	8	178.6	190	3.7	0.94	5	35	130	0.68	High	35	0.96	1.14	0.22	0.26	17	5	4
48	9	178.6	110	3.9	1.62	5	*3*	107	0.97	Low	35	1.93	1.97	0.08	0.10	46	12	9
48	10	178.6	140	3.9	1.28	5	22	95	0.68	High	35	1.29	1.37	0.10	0.12	22	6	5
49	**7**	**178.6**	**185**	**3.7**	**0.97**	**3**	**40**	**140**	**0.76**	**High**	**solid**	**1.19**	**1.20**	**0.00**	**0.00**	**18**	**5**	**4**
49	8	178.6	205	3.7	0.87	3	39	138	0.67	High	solid	0.55	0.57	0.00	0.00	6	3	2
49	9	178.6	130	3.8	1.37	5	20	110	0.85	High	35	1.34	1.42	0.10	0.12	23	6	5
49	10	178.6	130	3.8	1.37	5	22	95	0.73	High	solid	1.35	1.37	0.00	0.00	22	6	5
50	**9**	**178.6**	**150**	**3.7**	**1.19**	**5**	**32**	**115**	**0.77**	**High**	**solid**	**1.19**	**1.20**	**0.00**	**0.00**	**18**	**5**	**4**

Notes:
XL # - Refer to **Figures 1a** and **1b**.
Bold - Prediction outcomes for first LW panel beneath a given crossline.
SRP = refers to Subsidence Reduction Potential of the assumed strata unit for the purposes of subsidence prediction (i.e. L = Low, M = Moderate, H= High).
First S_{max}= maximum first subsidence for a given panel (including chain pillar compression effects from previously extracted panels).
Final S_{max}= maximum final subsidence for a given panel (including chain pillar compression effects after all longwall panels have been extracted).
Italics - Final S_{max} does not exceed 0.58 x Extraction Height (T).
First Pillar S_p = first subsidence above a chain pillar after longwalls extracted on both sides of it.
Final Pillar S_p = final subsidence above a chain pillar after all longwalls have been extracted.
* - Predicted strains are for a surface with a deep soil cover and likely to have 'smooth' profile strains. A surface with rock exposures is likely to cause strain concentrations, which can range between 2 and 3 x mean 'smooth' profile strains.
#- Mean tilt predictions increased by 10% to reflect additional data from the Northern Domain panels LWs 27 to 37 (see **Section 10.14**)

Table 10B - Predicted First and Final Subsidence Parameters (Mean Values) for LWs 44 to 46 (Southern Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
44	**11**	**178.6**	**165**	**4.7**	**1.08**	**2**	**55**	**17**	**0.10**	**High**	**45.0**	**1.42**	**1.52**	**0.16**	**0.19**	**25**	**7**	**5**
44	12	178.6	125	4.7	1.43	2	55	12	0.10	High	45.0	1.69	1.75	0.10	0.12	32	8	7
45	**13**	**178.6**	**120**	**4.8**	**1.49**	**2**	**35**	**19**	**0.16**	**High**	**34.2**	**1.79**	**1.88**	**0.12**	**0.15**	**38**	**10**	**8**
45	11	178.6	180	4.8	0.99	2	50	27	0.15	High	45.1	1.43	1.55	0.18	0.22	26	7	6
45	12	178.6	130	4.8	1.37	2	55	23	0.18	High	40.3	1.69	1.77	0.12	0.14	32	8	6
46	**13**	**168.2**	**150**	**4.8**	**1.12**	**2**	**35**	**30**	**0.20**	**High**	**solid**	**1.47**	**1.48**	**0.00**	**0.00**	**27**	**8**	**6**
46	11	168.2	180	4.8	0.93	2	35	40	0.22	High	solid	1.18	1.20	0.00	0.00	20	6	5
46	12	168.2	145	4.8	1.16	2	40	43	0.30	High	solid	1.49	1.51	0.00	0.00	27	8	6

Notes:
XL # - Refer to **Figures 1a** and **1b**.
Bold - Prediction outcomes for first LW panel beneath a given crossline.
SRP = refers to Subsidence Reduction Potential of the assumed strata unit for the purposes of subsidence prediction (i.e. L = Low, M = Moderate, H= High).
First S_{max}= maximum first subsidence for a given panel (including chain pillar compression effects from previously extracted panels).
Final S_{max}= maximum final subsidence for a given panel (including chain pillar compression effects after all longwall panels have been extracted).
Italics - Final S_{max} does not exceed 0.58 x Extraction Height (T).
First Pillar S_p = first subsidence above a chain pillar after longwalls extracted on both sides of it.
Final Pillar S_p = final subsidence above a chain pillar after all longwalls have been extracted.
* - Predicted strains are for a surface with a deep soil cover and likely to have ‘smooth’ profile strains. A surface with rock exposures is likely to cause strain concentrations, which can range between 2 and 3 x mean ‘smooth’ profile strains.
#- Mean tilt predictions increased by 10% to reflect additional data from the Northern Domain panels LWs 27 to 37 (see **Section 10.14**)

Table 11A - Predicted First and Final Subsidence Parameters (U95%CL Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max[#] Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
38	**1**	**177.8**	**160**	**4.35**	**1.11**	**4**	**25**	**125**	**0.78**	**High**	**35**	**1.61**	**1.72**	**0.17**	**0.20**	**35**	**10**	**8**
38	2	177.8	139	4.4	1.28	1	20	110	0.79	High	35	1.74	1.83	0.15	0.17	39	11	8
38	3	177.8	130	4.4	1.37	1	27	7	0.05	Low	35	2.26	2.33	0.13	0.15	59	14	11
38	4	177.8	110	4.5	1.62	1	25	7	0.06	Low	35	2.50	2.53	0.10	0.12	83	20	16
38	5	177.8	100	4.5	1.78	3	35	25	0.25	High	35	2.28	2.31	0.10	0.11	82	22	18
38	6	177.8	130	4.5	1.37	3	45	22	0.17	High	35	1.85	1.93	0.33	0.35	43	11	9
38	7	178.6	150	4.35	1.19	3	44	19	0.13	High	30	1.66	1.78	0.37	0.40	37	10	8
38	8	178.6	125	4.35	1.43	3	25	35	0.28	High	30	1.85	1.94	0.32	0.34	44	12	9
39	**1**	**178.6**	**150**	**4.25**	**1.19**	**4**	**27**	**120**	**0.80**	**High**	**solid**	**1.67**	**1.678**	**0.00**	**0.03**	**33**	**9**	**7**
39	2	178.6	150	4.2	1.19	4	23	107	0.71	High	30	1.64	1.75	0.16	0.19	36	10	8
39	3	178.6	135	4.1	1.32	4	5	117	0.87	Low	30	2.14	2.24	0.16	0.18	55	13	10
39	4	178.6	95	4.3	1.88	3	8	55	0.58	Low	30	2.49	2.49	0.10	0.11	111	29	23
39	5	178.6	95	4.3	1.88	3	25	22	0.23	High	30	2.37	2.40	0.10	0.11	94	26	20
39	6	178.6	130	4.35	1.37	3	33	16	0.12	High	30	1.83	1.920	0.32	0.35	42	11	9
39	7	178.6	150	4.35	1.19	3	30	25	0.17	Low	30	2.26	2.35	0.36	0.39	59	14	11
39	8	178.6	125	4.35	1.43	3	22	35	0.28	Low	30	2.28	2.36	0.33	0.35	61	15	12
40	**2**	**178.6**	**140**	**3.95**	**1.28**	**4**	**17**	**103**	**0.74**	**High**	**30**	**1.62**	**1.73**	**0.16**	**0.18**	**36**	**10**	**8**
40	3	178.6	160	3.95	1.12	5	5	155	0.97	Low	30	2.07	2.17	0.35	0.39	52	13	10
40	4	178.6	100	4.1	1.79	4	5	95	0.95	Low	30	2.38	2.38	0.11	0.13	93	24	19

Table 11A (Cont…) - Predicted First and Final Subsidence Parameters (U95%CL Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
40	5	178.6	97	4.1	1.84	4	2	95	0.98	Low	30	2.38	2.38	0.11	0.13	100	26	21
40	6	178.6	125	4.25	1.43	3	22	16	0.13	Low	30	2.23	2.32	0.32	0.35	60	14	11
40	7	178.6	140	4.25	1.28	5	7	133	0.95	Low	30	2.22	2.32	0.35	0.38	58	14	11
40	8	178.6	135	4.2	1.32	3	18	33	0.24	Low	30	2.19	2.27	0.33	0.35	56	14	11
40	9	178.6	110	4.5	1.62	3	13	44	0.40	Low	30	2.51	2.54	0.30	0.32	83	21	16
41	**2**	**178.6**	**160**	**3.8**	**1.12**	**4**	**13**	**98**	**0.61**	**Moderate**	**35**	**1.69**	**1.88**	**0.40**	**0.45**	**41**	**11**	**9**
41	3	178.6	160	3.8	1.12	4	8	93	0.58	Moderate	35	2.01	2.17	0.40	0.44	53	13	10
41	4	178.6	140	3.9	1.28	4	7	90	0.64	Low	35	2.02	2.16	0.19	0.23	52	13	10
41	5	178.6	140	4	1.28	5	10	130	0.93	Low	35	2.07	2.20	0.19	0.22	54	13	10
41	6	178.6	145	4.1	1.23	5	12	133	0.92	Low	35	2.13	2.25	0.37	0.40	56	13	11
41	7	178.6	160	4.15	1.12	5	25	125	0.78	High	35	1.59	1.72	0.37	0.40	35	10	8
41	8	178.6	140	4.05	1.28	5	25	115	0.82	High	35	1.64	1.70	0.29	0.31	35	10	8
41	9	178.6	90	4.3	1.98	3	12	38	0.42	Low	35	2.49	2.49	0.27	0.29	126	33	26
42	**2**	**178.6**	**240**	**3.6**	**0.74**	**5**	**32**	**142**	**0.59**	**High**	**35**	**0.75**	**1.36**	**0.73**	**0.84**	**19**	**6**	**5**
42	3	178.6	235	3.7	0.76	5	30	190	0.81	High	35	0.60	1.20	0.73	0.83	19	6	5
42	4	178.6	220	3.7	0.81	5	30	170	0.77	High	35	0.64	1.16	0.65	0.74	17	6	5
42	5	178.6	215	3.9	0.83	5	25	145	0.67	High	35	0.71	1.18	0.62	0.71	18	6	5
42	6	178.6	205	4	0.87	5	25	125	0.61	High	35	0.84	1.14	0.49	0.55	17	6	5
42	7	178.6	180	4	0.99	5	35	122	0.68	High	35	1.49	1.58	0.35	0.38	31	9	7

Table 11A (Cont…) - Predicted First and Final Subsidence Parameters (U95%CL Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max[#] Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
42	8	178.6	90	3.85	1.98	3	15	25	0.28	Low	35	2.23	2.23	0.25	0.27	108	30	24
42	9	178.6	80	4	2.23	3	13	33	0.41	Low	35	2.32	2.32	0.25	0.26	139	40	32
42	10	178.6	78	4.35	2.29	3	15	40	0.51	Low	35	2.52	2.52	0.27	0.28	163	46	36
43	**2**	**178.6**	**340**	**3.5**	**0.53**	**5**	**40**	**143**	**0.42**	**High**	**239**	**0.75**	**0.77**	**0.00**	**0.03**	**9**	**4**	**3**
43	3	178.6	335	3.5	0.53	5	40	220	0.66	High	239	0.75	0.77	0.00	0.03	9	4	3
43	4	178.6	315	3.65	0.57	5	40	185	0.59	High	35	0.75	1.41	0.86	1.00	27	8	6
43	5	178.6	290	3.7	0.62	5	40	155	0.53	High	35	0.77	1.34	0.77	0.89	24	8	6
43	6	178.6	225	3.8	0.79	5	35	135	0.60	High	35	0.65	1.11	0.61	0.69	16	6	5
43	7	178.6	140	3.7	1.28	5	*13*	127	0.91	Low	35	1.96	2.07	0.32	0.35	49	12	10
43	8	178.6	120	3.7	1.49	5	*5*	115	0.96	Low	35	1.99	2.05	0.27	0.29	52	14	11
43	9	178.6	95	3.9	1.88	3	12	26	0.27	Low	35	2.26	2.26	0.26	0.27	98	27	21
43	10	178.6	75	4.25	2.38	3	15	30	0.40	Low	35	2.47	2.47	0.26	0.28	167	48	38
47	**4**	**178.6**	**315**	**3.5**	**0.57**	**5**	**55**	**180**	**0.57**	**High**	**35**	**0.84**	**1.40**	**0.79**	**0.92**	**26**	**8**	**6**
47	5	178.6	280	3.6	0.64	5	60	155	0.55	High	35	0.84	1.32	0.69	0.79	24	7	6
47	6	178.6	275	3.7	0.65	5	38	140	0.51	High	35	0.789	1.244	0.66	0.76	21	7	5
47	7	178.6	200	3.7	0.89	5	38	135	0.68	High	35	0.850	1.161	0.48	0.54	18	6	5
47	8	178.6	150	3.7	1.19	5	30	120	0.80	High	35	1.448	1.566	0.32	0.35	30	9	7
47	9	178.6	120	3.9	1.49	5	18	102	0.85	High	35	1.74	1.79	0.27	0.29	42	12	9
47	10	178.6	85	4.1	2.10	3	12	20	0.24	Low	35	2.38	2.38	0.27	0.29	132	36	29

Table 11A (Cont…) - Predicted First and Final Subsidence Parameters (U95%CL Values) for LWs 38 to 43 and 47 to 50 (Western Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
48	**4**	**178.6**	**290**	**3.3**	**0.62**	**5**	**55**	**170**	**0.59**	**High**	**solid**	**0.823**	**0.840**	**0.00**	**0.03**	**10**	**4**	**3**
48	5	178.6	260	3.5	0.69	5	60	150	0.58	High	solid	0.947	0.963	0.00	0.03	9	4	3
48	6	178.6	240	3.6	0.74	5	40	150	0.63	High	solid	0.882	0.899	0.00	0.02	7	3	3
48	7	178.6	250	3.7	0.71	5	40	140	0.56	High	35	0.842	1.157	0.50	0.57	13	5	4
48	8	178.6	190	3.7	0.94	5	35	130	0.68	High	35	1.220	1.401	0.40	0.44	25	8	6
48	9	178.6	110	3.9	1.62	5	*3*	107	0.97	Low	35	2.20	2.24	0.27	0.29	69	18	14
48	10	178.6	140	3.9	1.28	5	22	95	0.68	High	35	1.57	1.64	0.29	0.31	33	9	7
49	**7**	**178.6**	**185**	**3.7**	**0.97**	**3**	**40**	**140**	**0.76**	**High**	**solid**	**1.451**	**1.463**	**0.00**	**0.02**	**27**	**8**	**6**
49	8	178.6	205	3.7	0.87	3	39	138	0.67	High	solid	0.815	0.839	0.00	0.02	9	4	3
49	9	178.6	130	3.8	1.37	5	20	110	0.85	High	35	1.608	1.688	0.28	0.30	34	10	8
49	10	178.6	130	3.8	1.37	5	22	95	0.73	High	solid	1.62	1.64	0.00	0.02	33	9	7
50	**9**	**178.6**	**150**	**3.7**	**1.19**	**5**	**32**	**115**	**0.77**	**High**	**solid**	**1.453**	**1.464**	**0.00**	**0.02**	**27**	**8**	**6**

Notes:
XL # - Refer to **Figures 1a** and **1b**.
Bold - Prediction outcomes for first LW panel beneath a given crossline.
SRP = refers to Subsidence Reduction Potential of the assumed strata unit for the purposes of subsidence prediction (i.e. L = Low, M = Moderate, H= High).
First S_{max}= maximum first subsidence for a given panel (including chain pillar compression effects from previously extracted panels).
Final S_{max}= maximum final subsidence for a given panel (including chain pillar compression effects after all longwall panels have been extracted).
Italics - Final S_{max} does not exceed 0.58 x Extraction Height (T).
First Pillar S_p = first subsidence above a chain pillar after longwalls extracted on both sides of it.
Final Pillar S_p = final subsidence above a chain pillar after all longwalls have been extracted.
* - Predicted strains are for a surface with a deep soil cover and likely to have 'smooth' profile strains. A surface with rock exposures is likely to cause strain concentrations, which can range between 2 and 3 x mean 'smooth' profile strains.
#- U95%CL tilt predictions increased by 18% to reflect additional data from the Northern Domain panels LWs 27 to 37 (see **Section 10.14**)

Table 11B - Predicted First and Final Subsidence Parameters (U95%CL Values) for LWs 44 to 46 (Southern Domain)

LW #	XL #	Panel Width W (m)	Cover Depth H (m)	Face Height T (m)	Panel Width/ Cover Ratio W/H	Massive Channel Unit Properties					Chain Pillar Width w_{cp} (m)	Panel First S_{max} (m)	Panel Final S_{max} (m)	Pillar First S_p (m)	Pillar Final S_p (m)	Max# Tilt T_{max} (mm/m)	Max Comp Strain $-E_{max}$ (mm/m)	Max Tensile Strain $+E_{max}$ (mm/m)
						No.	Thickness t (m)	Distance Above Roof y (m)	Location Ratio y/H	SRP								
44	**11**	**178.6**	**165**	**4.7**	**1.08**	**2**	**55**	**17**	**0.10**	**High**	**45.0**	**1.72**	**1.83**	**0.18**	**0.21**	**38**	**10**	**8**
44	12	178.6	125	4.7	1.43	2	55	12	0.10	High	45.0	1.99	2.06	0.12	0.14	48	12	10
45	**13**	**178.6**	**120**	**4.8**	**1.49**	**2**	**35**	**19**	**0.16**	**High**	**34.2**	**2.10**	**2.19**	**0.14**	**0.17**	**57**	**14**	**11**
45	11	178.6	180	4.8	0.99	2	50	27	0.15	High	45.1	1.74	1.85	0.20	0.24	39	11	8
45	12	178.6	130	4.8	1.37	2	55	23	0.18	High	40.3	1.99	2.08	0.14	0.16	48	12	10
46	**13**	**168.2**	**150**	**4.8**	**1.12**	**2**	**35**	**30**	**0.20**	**High**	**solid**	**1.77**	**1.79**	**0.00**	**0.02**	**40**	**11**	**9**
46	11	168.2	180	4.8	0.93	2	35	40	0.22	High	solid	1.49	1.50	0.00	0.02	29	9	7
46	12	168.2	145	4.8	1.16	2	40	43	0.30	High	solid	1.80	1.81	0.00	0.02	41	12	9

Notes:
XL # - Refer to **Figures 1a** and **1b**.
Bold - Prediction outcomes for first LW panel beneath a given crossline.
SRP = refers to Subsidence Reduction Potential of the assumed strata unit for the purposes of subsidence prediction (i.e. L = Low, M = Moderate, H= High).
First S_{max}= maximum first subsidence for a given panel (including chain pillar compression effects from previously extracted panels).
Final S_{max}= maximum final subsidence for a given panel (including chain pillar compression effects after all longwall panels have been extracted).
Italics - Final S_{max} does not exceed 0.58 x Extraction Height (T).
First Pillar S_p = first subsidence above a chain pillar after longwalls extracted on both sides of it.
Final Pillar S_p = final subsidence above a chain pillar after all longwalls have been extracted.
* - Predicted strains are for a surface with a deep soil cover and likely to have ‘smooth’ profile strains. A surface with rock exposures is likely to cause strain concentrations, which can range between 2 and 3 x mean ‘smooth’ profile strains.
#- U95%CL tilt predictions increased by 18% to reflect additional data from the Northern Domain panels LWs 27 to 37 (see **Section 10.14**)

The broad range of cover depths (70 m to 360 m) have resulted in a similarly broad range of subsidence predictions for the study area (see end of results tables for impact parameter ranges).

High SRP overburden is generally present beneath the ridges and Low SRP overburden is located below the valleys.

The predicted mean and U95%CL values for the **first maximum panel subsidence** after mining of LWs 38 to 50 is completed, ranges from 0.33 m to 2.52 m and 0.60 to 2.52 m respectively (i.e. 9% to 58% of proposed extraction heights).

Note: the mean and U95%CL values are assumed to converge for supercritical panel geometries.

The predicted mean and U95%CL values for the **final maximum panel subsidence** after mining of LWs 38 to 50 is completed, ranges from 0.49 m to 2.52 m and from 0.77 to 2.52 m respectively (i.e. 22% to 58% of proposed extraction heights).

The predicted mean and U95%CL values for the **first maximum chain pillar subsidence** after mining of LWs 38 to 50 is completed, ranges from 0.06 m to 0.68 m and from 0.07 to 0.82 m respectively (i.e. 1% to 24% of proposed extraction heights).

The predicted mean and U95%CL values for the **final maximum chain pillar subsidence** after mining of LWs 38 to 50 is completed, ranges from 0.07 m to 0.82 m and from 0.08 to 1.00 m respectively (i.e. 1% to 27% of proposed extraction heights).

The predicted mean and U95%CL values for the **final maximum panel tilt** after mining of LWs 38 to 50 is completed ranges from 5 to 111 mm/m and 7 to 167 mm/m respectively.

The predicted mean and U95%CL values for the **final maximum panel compressive strains** after mining of LWs 38 to 50 is completed, ranges from 2 to 32 mm/m and 3 to 48 mm/m respectively.

The predicted mean and U95%CL values for the **final maximum panel tensile strains** after mining of LWs 38 to 50 is completed, ranges from 2 to 25 mm/m and 3 to 38 mm/m respectively.

The predicted mean and U95%CL values for the **final concave curvatures** associated with the compressive strain zones after mining of LWs 38 to 50 is completed, ranges from 0.31 to 4.42 and 0.46 to 6.64 km^{-1} respectively (i.e. curvature radii of 3.3 to 0.15 km).

The predicted mean and U95%CL values for the **final convex curvatures** associated with the tensile strain zones after mining of LWs 38 to 50 is completed, ranges from 0.24 to 3.49 km^{-1} and 0.36 to 5.23 km^{-1} respectively (i.e. curvature radii of 4.2 to 0.19 km).

10.7 Angle of Draw Prediction

Reference to the **ACARP, 2003** longwall panel angle of draw predictions have been derived from the mean goaf edge subsidence predictions. The AoD to the 20 mm subsidence contour is estimated to range from 8^o to 33^o for the proposed LWs and based on the empirical model presented in **Figure 26a**. A summary of the angle of draw statistics presented in **ACARP, 2003** is given in **Figure 26b**.

Reference to published AoD from the Southern NSW Coalfield in **ACARP, 2002**, also indicates that down-slope movements may increase the AoD by up to 45^o.

An AoD of 26.5^o is still considered typical for the study area, however isolated values of up to 33^o (or even 45^o) should be considered for mine planning and impact management purposes near sensitive surface features.

Detailed monitoring studies should be conducted in non-sensitive areas during mining where a sensitive surface feature is at risk of being damaged and will be discussed further in **Section 13**.

10.8 Inflexion Point and Peak Strain Locations

The subsidence development process causes tensile and compressive strains to develop above an extracted longwall panel, due to the sagging and bending of the overburden strata.

Tensile strains are generally located in the outer third zone above an extracted longwall panel and the compressive strains will occur above the central or middle third area. The point where the tensile strains become compressive is called the inflexion point.

The relative locations of the peak surface impact parameters above an extracted longwall panel are shown schematically in **Figure 27a**. The Newcastle Coalfield database of longwall inflexion point and tensile/compressive strain or convex/concave curvature peak locations are shown in **Figure 27b**.

The predicted locations of the above points for the proposed panels are given in **Table 12**.

Table 12 - Predicted Inflexion and Strain Peak Location Summary

Cover Depth H (m)	Panel W/H	Inflexion Point Location Factor d/H	Inflexion Point Location from Panel Rib-side d	Tensile Strain Peak Location Factor d_t/H	Tensile Strain Peak Location From Panel Rib Side d_t	Compressive Strain Peak Location Factor d_c/H	Compressive Strain Peak Location from Panel Rib-Side d_c
75 - 340	0.53 - 2.38	0.15 - 0.39	29 - 57	0.08 - 0.28	21 - 41	0.22 - 0.5	38 - 75

10.9 Subsidence Profile Prediction Results

Based on the modified **ACARP, 2003** empirical model and **Sigmaplot®** cubic-spline curves, predictions of the maximum subsidence, tilt and strain profiles along cross lines XL 2, 5, 7, 9, 10 and 11 after each panel is extracted and on the completion of mining of LWs 38 to 50 are shown in **Figures 28a** to **33c**.

10.10 Predicted v. Measured Subsidence Data for the Northern Domain Panels

A review of the predicted and measured subsidence parameter data was undertaken along several crosslines and centrelines in the Northern Domain, see **Figure 34a**. The panel geometry and inferred overburden lithology for the extracted longwall panels LWs 27 to 37 are summarised in **Table 13**.

The data was subsequently compared to predicted values derived from the empirical database in **ACARP, 2003**.

Table 13 - LW Panel Geometry and Geology for the Northern Domain Panels

Parameter	Units	LW27	LW28	LW31	LW32	LW33	LW34	LW35	LW36	LW37
Survey Cross Lines	name	WN	WN, WO	WN, WO	WO	WO	WO, WP	WO, WP	WP	WP
Panel Void Width, W	m	175	175	175	172.5	175	178.6	178.6	178.6	178.6
Cover Depth, H	m	157	153, 190	137, 200	230	190	190, 180	190, 155	143	145
Panel W/H	m/m	1.11	1.14, 0.92	1.28, 0.83	0.75	0.92	0.94, 0.99	0.94, 1.15	1.25	1.23
Extraction Height, T	m	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
Development Height, h	m	3.1	3.3	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Massive Unit Thickness, t*	m	33	27, 28	25, 35	35	33	25, 43	18, 30	10	10
Unit Height above Workings, y*	m	45	38, 50	38, 43	43	41	38, 40	37, 46	45	45
Massive Unit Location Factor, y/H	m/m	0.29	0.25, 0.26	0.28, 0.21	0.19	0.22	0.20, 0.22	0.20, 0.30	0.31	0.31
SRP^	**L,M,H**	**H**	**H, M**	**H, H**	**M**	**H**	**M, H**	**L, H**	**L**	**L**
Chain Pillar Width, w_{cp}	m	30	32.5	175, 35	35	35	35	35	35	35
Chain Pillar Length, l_{cp}	m	110	92.6	110	110	110	110	110	110	110
Measured Maximum Panel Subsidence S_{max}	First m	**1.28**	**1.67, 1.50**	**1.68, 0.93**	**1.05**	**1.43**	**1.83, 1.34**	**1.35**	**2.42**	**2.41**
	Final m	**1.43**	**1.74, 1.58**	**1.68, 1.25**	**1.16**	**1.50**	**1.83, 1.42**	**1.40**	**2.46**	**2.41**
Measured Chain Pillar Subsidence, S_p	First m	**0.15**	**0.06, 0.18**	**-, 0.55**	**0.13**	**0.045**	**-, 0.095**	**0.099**	**0.062**	**-**
	Final m	**0.17**	**0.09, 0.25**	**-, 0.61**	**0.14**	**0.12**	**-, 0.099**	-	**-**	-

Note:
* - The 'worst case' combination of values for y and t has been assumed.
^ - L = Low; M = Moderate; H = High SRP.

10.11 Subsidence Reduction Potential of the Massive Units Above the Northern Domain Panels

The thickness and location of several sandstone and conglomerate strata units above the Northern Domain Panels have been assessed from investigation borehole logs along four geological long-sections shown in **Figures 34b** to **34e**.

The SRP of the units has been assessed using **Table 3** and relevant curves presented in **Figures 21a,b**.

The outcomes of the SRP assessment are summarised in **Table 13** and indicate that the Young Wallsend channel (Unit 1) and Fern Valley Channel (Unit 2) have 'Moderate' to ‘High’ SRP. The other channels are assessed to have a ‘Low’ SRP. The Unit 1 and 2 channel thickness and their location above the workings contours are presented in, **Figures 34f, g** and **34h, i** respectively.

10.12 Comparison Between Measured and Predicted Subsidence

First and Final maximum panel and chain pillar subsidence values over the Northern Domain panels were then determined. The predictions have been compared to the measured values along Crosslines WN, WO and WP in **Table 14** and shown in **Figures 34j** to **34l**.

Table 14 - Subsidence Predictions vs. Measured Outcomes along Cross lines in the Northern Domain

LW	Survey Line (see Fig. 34a)	Predicted First S_{max} (m)	Measured First S_{max} (m)	Predicted S_P (m)	Measured S_P (m)	Predicted Final S_{max} (m)	Measured Final S_{max} (m)
27	WN	1.46 - 1.72	1.28	0.19 - 0.34	0.15 - 0.17	1.53 - 1.79	1.43
28		1.79 - 2.06	1.67	0.16 - 0.30	0.06 - 0.09	1.84 - 2.11	1.74
31		1.62 - 1.89	1.68	-	-	1.63 - 1.90	1.68
28	WO	1.51 - 2.00	1.50	0.32 - 0.50	0.18 - 0.25	1.64 - 2.13	1.58
31		0.61 - 1.14	0.93	0.39 - 0.69	0.55 - 0.61	0.80 - 1.33	1.25
32		0.79 - 0.97	1.05	0.37 - 0.55	0.13 - 0.14	0.95 - 1.13	1.16
33		1.14 - 1.63	1.43	0.26 - 0.43	0.05 - 0.12	1.24 - 1.74	1.50
34		1.65 - 2.14	1.83	0.26 - 0.43	-	1.74 - 2.14	1.83
34	WP	1.41 - 1.90	1.35	0.20 - 0.36	0.09 -	1.47 - 1.97	1.42
35		1.55 - 1.81	1.35	0.15 - 0.30	0.10 -	1.68 - 1.87	1.40
36		2.17 - 2.44	2.42	0.14 - 0.28	0.06 -	2.21 - 2.48	2.46
37		2.16 - 2.43	2.41	0.10 - 0.24	-	2.18 - 2.44	2.41

Notes:
- The predicted values are the Mean and Upper 95% Confidence Limits that have been determined from the database.
Bold - Measured values exceed predicted U95%CL values by > 10%.

All of the predicted values of First and Final S_{max} are between the predicted Mean and Upper 95% Confidence Limits and 0.86 times the U95%CL values.

The measured chain pillar values, however, are only 39% of the predicted U95%CL and 60% of the predicted mean values respectively.

The survey data also indicates that the subsidence above the chain pillars accounts for approximately 3% to 16% of the Final S_{max} that occurred over each panel. The remaining 97% to 84% of the measured subsidence is almost certainly a result of the sag or deflection of the overburden between the chain pillars.

10.13 Comparison Between Predicted and Measured Maximum Tilts, Curvatures and Strains

A similar validation exercise was also conducted on the transverse differential subsidence predictions and measurements for LWs 27 to 37, and is summarised in **Tables 15** and **16** and presented in **Figures 34j** to **34l**.

Table 15 – Differential Subsidence Predictions v. Measured Values along Cross lines in the Northern Domain

LW	Survey Line (see Fig. 34a)	Predicted Maximum Tilt (mm/m)	Measured Maximum Tilt (mm/m)	Predicted Maximum Curvature (km^{-1}) {'smooth' profile}		Measured Maximum Curvature (km^{-1})	
27	WN	25 - 34	17 - 26	Convex	0.78 - 1.17	Convex	0.31 - 0.66
				Concave	0.99 - 1.48	Concave	0.66 - 1.05
28		32 - 45	30 - 35	Convex	0.94 - 1.41	Convex	0.76 - 1.26
				Concave	1.19 - 1.78	Concave	1.26 - 1.59
31		27 - 38	25 - 36	Convex	0.83 - 1.24	Convex	0.70 - 0.95
				Concave	1.05 - 1.58	Concave	0.91 - 1.13
28	WO	27 - 38	33 - 41	Convex	0.83 - 1.25	Convex	1.04 - 1.17
				Concave	1.06 - 1.59	Concave	**2.55 - 2.65**
31		12 - 17	11 - **22**	Convex	0.47 - 0.70	Convex	0.34 - **0.79**
				Concave	0.59 - 0.89	Concave	0.67 - 0.70
32		13 - 18	10 - 18	Convex	0.50 - 0.74	Convex	0.25 - 0.79
				Concave	0.63 - 0.94	Concave	0.46 - 0.95
33		18 - 26	23 - **35**	Convex	0.63 - 0.95	Convex	0.83 - **1.11**
				Concave	0.80 - 1.20	Concave	1.11 - **1.54**
34		29 - 40	36 - **48**	Convex	0.85 - 1.27	Convex	1.15 - 1.23
				Concave	1.08 - 1.61	Concave	1.39 - **4.07**
34	WP	23 - 32	23 - 28	Convex	0.72 - 1.08	Convex	0.86 - 0.92
				Concave	0.92 - 1.37	Concave	0.75 - 0.92
35		25 - 36	21 - 26	Convex	0.78 - 1.17	Convex	0.92 - 0.97
				Concave	0.99 - 1.49	Concave	1.00 - **1.92**
36		40 - 56	43 - **69**	Convex	1.08 - 1.62	Convex	**1.94** - **2.87**
				Concave	1.37 - 2.06	Concave	1.54 - **3.65**
37		39 - 55	59 - **68**	Convex	1.07 - 1.60	Convex	**2.12** - **4.04**
				Concave	1.35 - 2.03	Concave	**3.62** - **3.73**

Notes:
- Predictions are the Mean and Upper 95% Confidence Limits determined from 'best fit' regression analysis with measured data.
- Bold - Measured values exceed predicted U95%CL values by > 10%.
- Predicted tilts are smooth profile values and could increase by 1.3 to 1.5 times due to 'discontinuous' overburden behaviour.
- Predicted curvatures are smooth profile curvatures and could increase by 2 or 3 times due to strain concentration effects.

Seventy-nine percent (79%) of the measured maximum tilts were within the Mean and Upper 95% Confidence Limits +/- 10% for the predicted tilts (with a range from 0.72 to 1.08 and an average of 0.88 times the U95%CL value). The remaining 20% of measured maximum panel tilts exceeded the U95%CL values by 1.20 to 1.35 times (average of 1.26) and likely to be due to discontinuous overburden behaviour.

Seventy percent (70%) of the measured maximum curvatures were within the Mean and Upper 95% Confidence Limits +/- 10% for the predicted curvatures (with a range from 0.56 to 1.06 and an average of 0.84 times the U95%CL). The remaining 30% of measured maximum panel curvatures exceeded the U95%CL values by 1.12 to 2.53 times (average of 1.70). The exceedences were also probably due to discontinuous behaviour of the overburden.

Table 16 - Horizontal Strain Predictions vs. Measured Outcomes along Cross lines in the Northern Domain

LW	Survey Line (see Fig. 34a)	Predicted* Tensile Strain $+E_{max}$ (mm/m)	Measured Tensile Strain $+E_{max}$ (mm/m)	Max Strain Ratio M/P	Predicted* Compressive Strain $+E_{max}$ (mm/m)	Measured Compressive Strain $+E_{max}$ (mm/m)	Max Strain Ratio M/P
27	WN	5.7 - 8.5	5.7 - 8.9	1.06	7.2 - 10.8	5.3 - **16.1**	1.49
28		6.8 - 10.3	6.2 - 7.2	0.70	8.7 - 13.0	5.8 - 11.3	0.87
31		6.1 - 9.1	9.7 - **13.9**	1.53	7.7 - 11.5	11.0 - **38.3**	3.33
28	WO	6.1 - 9.1	1.5 - 6.5	0.71	7.7 - 11.6	4.9 - **21.0**	1.81
31		3.4 - 5.1	4.5 - 4.6	0.90	4.3 - 6.5	2.8 - **9.8**	1.51
32		3.6 - 5.4	2.3 - 4.6	0.85	4.6 - 6.9	4.0 - 4.2	0.61
33		4.6 - 6.9	3.7 - **12.0**	1.74	5.9 - 8.8	5.7 - 8.6	0.98
34		6.2 - 9.3	4.3 - **15.1**	1.62	7.9 - 11.8	9.2 - 11.7	0.99
34	WP	5.3 - 7.9	3.3 - **11.9**	1.51	6.7 - 10.0	4.6 - 5.2	0.52
35		5.7 - 8.6	4.0 - 4.6	0.53	7.2 - 10.9	9.0 - 9.7	0.89
36		7.9 - 11.8	17.9 - **32.2**	2.73	10.0 - 15.0	**17.5** - **30.5**	1.17/2.03
37		7.8 - 11.7	7.9 - 11.7	1.00	9.9 - 14.8	15.0 - 15.2	1.03

Notes:
* - The predicted values are the Mean and Upper 95% Confidence Limits that have been determined from the database.
Bold - Measured values exceed predicted U95%CL values by > 10%.
M/P - Ratio of measured maximum strain over predicted U95%CL strain.

Seventy-seven percent (77%) of the measured maximum strains were within the Mean and Upper 95% Confidence Limits +/- 10% for the predicted strains (with a range of 0.52 to 1.06 and an average of 0.84 times the U95%CL). The remaining 23% of measured maximum panel strains exceeded the U95%CL values by 1.17 to 3.33 times (average of 1.83) and likely to be due to discontinuous overburden behaviour.

The concentration of curvatures and associated horizontal strains generally starts to occur when longwall panels are in the critical to supercritical W/H ratio range of 1 to 2 and in zones where the radius of curvature is less than 5 km (i.e. curvatures 0.2 km^{-1}). The phenomenon is thought to be caused by secondary curvature effects or surface 'hump' development in the compressive zones due to buckling or shear failure of surface strata; or crack development/opening of pre-existing jointing in the tensile regions of the subsidence profile.

The surface topography and geology (i.e. lithology and defects) are also governing factors with regard to the distribution of curvature and strain over the subsiding area.

Two examples of the significant increase to measured tilts, curvatures and strains that can occur due to a discontinuity developing at the surface is shown in **Figures 34m** and **34n** above the starting ends of LWs 34 and 37.

10.14 Review of ACARP, 2003 Databases for Tilt, Curvature and Strain Prediction

It is apparent from the comparison between predicted and measured tilt, curvature and strain for the Northern Domain that there were a higher proportion of exceedences than expected of the predicted U95%CL values for tilts (21%), curvatures (30%) and strains (23%). Some of the exceedences were clearly associated with discontinuities associated with the faults and dykes near the northern ends of the panels, but not for all of the cases.

The results for the Northern Domain crosslines were subsequently plotted with the **ACARP, 2003** databases in **Figures 34o**, **34p** and **34q**.

The outcomes of this exercise indicate that the tilt prediction model could be improved by including the Northern Domain data and revising the regression equations for the expanded database (see **Figure 34r**). The revised regression curves will increase the predicted mean tilts by 10% and the U95%CL values by 18%.

The exceedences of the convex and concave curvature however, are considered to be due to discontinuous behaviour as shown in **Figures 34p** and **34q**. No changes to the curvature prediction curves are considered necessary at this stage.

The "K" ratio between the maximum measured strain and curvature for each panel in the Northern Domain has also been determined. Regression analysis on the data indicates the mean K values ranged between 4.7 and 10.8, giving an average ratio of 7.4, as shown in **Figure 34s**. The assumed K value of 7.3 is therefore still considered reasonable for subsequent predictions of strain from curvature profiles in the Western and Southern Domains. The similarity between the measured/predicted strain and the curvature ratios also indicate the K factor is reasonable.

10.15 Comparison Between Measured and Predicted Goaf Edge Subsidence and Angle of Draw

The predictions of goaf edge subsidence and angle of draw for the Northern Domain Panels have been compared to the measured values along Crosslines WN, WO and WP and Panel End Centrelines in **Table 17**.

Table 17 - Goaf Edge Subsidence and Angle of Draw Predictions vs. Measured Outcomes along Cross lines and Centrelines in the Northern Domain

LW	Survey Line (see Fig. 34a)	Predicted* Goaf Edge Subsidence S_{goe} (m)	Measured Goaf Edge Subsidence S_{goe} (m)	Predicted* Angle of Draw (o)	Predicted* Angle of Draw+ (z/H)	Measured Angle of Draw (o)	Measured Angle of Draw+ (z/H)
27	WN	0.08 - 0.21	0.12 - 0.16	13 - 22	0.23 - 0.40	14 - 17	0.25 - 0.31
28		0.09 - 0.24	0.14	14 - 23	0.25 - 0.42	19	0.34
31		0.06 - 0.17	-	11 - 20	0.19 - 0.36	-	-
28	WO	0.13 - 0.33	0.07 - 0.11	18 - 26	0.32 - 0.49	28	0.53
31		0.05 - 0.20	0.08 - 0.11	14 - 23	0.25 - 0.42	15	0.27
32		0.10 - 0.28	0.075	17 - 26	0.31 - 0.49	13	0.25
33		0.09 - 0.25	0.08	15 - 24	0.27 - 0.45	24	0.45
34		0.13 - 0.34	0.07	18 - 26	0.32 - 0.49	5	0.09
34	WP	0.10 - 0.26	0.07	15 - 24	0.27 - 0.45	10	0.18
35		0.08 - 0.21	0.10	13 - 22	0.23 - 0.40	21	0.38
36		0.09 - 0.25	0.12	14 - 23	0.25 - 0.42	14	0.25
37		0.091 - 0.25	0.09	14 - 23	0.25 - 0.42	15	0.27
34	WJ	0.064 - 0.18	0.070	11 - 20	0.19 - 0.36	11	0.19
37	WM	0.065 - 0.18	0.050	11 - 20	0.19 - 0.36	8	0.13

Notes:
* - The predicted values are the Mean and Upper 95% Confidence Limits that have been determined from the database.
+ - Measured distance to 20 mm subsidence (z) over the cover depth (H).
Bold - Measured values exceed predicted U95%CL values by > 10%.

The measured goaf edge subsidence and angles of draw to the 20 mm subsidence contour were generally between the predicted mean and U95%CL values. The one exceedence that did occur was 1.08 times the U95%CL.

10.16 Summary

Overall, the above exercise was considered to be a necessary step in the empirical model validation process for using the model to make subsidence predictions for the Western and Southern Domain longwalls.

The measured maximum subsidence above the panels in the Northern Domain were generally between the predicted mean and U95%CL of First and Final Maximum panel subsidence values with no exceedences occurring. This indicates that the claimed reliability of 95% for the maximum subsidence predictions is likely to continue in the Western and Southern Domains.

Updating the **ACARP, 2003** database indicates that mean and U95% Confidence Limits for the maximum panel tilt predictions should be increased by 10% and 18% respectively for the future mining areas at West Wallsend Colliery. The revised tilt model has been adopted for the predictions presented in **Tables 10A, B and 11A, B** for the Western and Southern Domain panels respectively (see **Section 10.6**).

It is not considered necessary to amend the curvature and strain prediction models at this stage.

11.0 Prediction of Subsidence Impact Parameter Contours

11.1 Calibration of the SDPS® Model

The **SDPS®** model was then calibrated to the ACARP model profiles to derive U95%CL subsidence contours. The outcome of the SDPS model calibration exercise is summarised in **Table 19**.

Table 19 - SDPS® Model Calibration Summary

Input Parameters	**Value**	**Model Database**
Panel No.s (refer to Figures 1a and 1b)	LWs 38 to 50	Includes LWs 11 to 37
Panel Void Width, W (m)	168.2 to 178.6	34 - 264
Cover Depth, H (m)	70 - 360	71 - 516
Mining Height, T (m)	3.3 - 4.8	1.05 - 4.9
W/H range	0.5 - 2.38	0.2 - 2.0
Massive Strata Unit Thickness, t (m)	2 - 60	<5 - 80
Strata Unit Distance Above Workings, y (m)	7 - 120	1 - 350
SRP for Mining Area	Low to High	Low to High
Strata Unit Location Ratio (y/H)	0.05 - 0.98	0.0 - 0.9
Maximum Final Panel Subsidence Range, S_{max} (m)	0.49 - 2.61	0.02 - 2.5
S_{max}/T Range for Panels	0.22 - 0.58	0.01 - 0.58
Chain Pillar Width (m)	30, 35 and 45 m	18 - 49
Chain Pillar Lengths (m)	110 m	60 - 110
Development Road or Chain Pillar Height (m)	3.5	1.8 - 3.5
Gate road Heading and Cut-through Widths (m)	5.5	4.8 - 6.0
Chain Pillar Subsidence (m)	0.07 - 0.99	0.03 - 1.00
Theoretical Maximum Chain Pillar Stress (MPa)*	3.7 -34.6	4.8 - 81
Chain Pillar FoS	0.90 - 7.98	0.56 - 9.40
Chain Pillar Width/Development Height	6.7 - 10.0	7.4 - 15.8
Modified ACARP, 2003 Inflection Point Location (d) from Rib-side/Cover Depth (H): d/H	0.15 - 0.39	0.03 - 0.50
Modified ACARP, 2003 Inflection Point Location from Rib-side, d (m)	29 - 57	5 - 99
Goaf Edge Subsidence (m)	0.04- 0.62	0.02 - 0.38
Angle of Draw (degrees)	8 - 33	0 - 33
Calibration Results for Best Fit Solution to the Modified ACARP, 2003 Model Predictions^	**Optimum Value***	
Influence Angle (Tan(beta))	1.8	
Influence Angle (degrees)	61	
Supercritical Subsidence Factor for Panels and Pillars (S_{max}/T)	66 - 100	
Distance to Influence Inflexion Point from Internal Chain Pillar or Solid Rib-Sides (m)	29 - 57	

Notes:
^ - See SDPS manual extract in **Appendix A** for explanation of methodology and terms used.
* - These values provide best fit to Modified **ACARP, 2003** profiles only and are due to the effect of calibrating SDPS to multiple panels with compressing chain pillars (i.e. they should not be used as predicted values alone).

Based on reference to **Table 19** the geometry and geology of the proposed longwall panels (LWs 38 to 50) are generally within the limits of the current database.

The modified **ACARP, 2003** model and **SDPS®** model subsidence impact parameter profiles have been compared in **Figures 35a** to **35c** for XL 3 and **Figures 35d** to **35f** for XL11 respectively.

The predicted **SDPS®** subsidence and tilt profiles were generally located within +/- 10 to 20% of the predicted modified **ACARP, 2003** models Upper 95% Confidence Limits. This outcome is considered a reasonable fit considering that the **ACARP, 2003** profiles represent measured tilt profiles that are invariably affected by 'skewed' or kinked subsidence profiles.

The results of the analysis indicate that the majority of the predicted convex curvature (and tensile strain) and concave curvature (and compressive strains) predicted by the **SDPS®** model would fall within +/- 50% of the modified **ACARP, 2003** model predictions. This result is also considered reasonable in the context that the **ACARP, 2003** model represents measured profile data that includes strain concentration effects such as cracking and shearing. As mentioned earlier, this 'discontinuous' type of overburden behaviour can increase 'smooth' profile strains by 2 to 4 times locally.

11.2 Predicted Subsidence and Associated Impact Parameter Contours

Based on the calibrated SDPS® model, predictions of final, worst-case subsidence contours for LWs 38 to 50 are shown in **Figures 36a, b**.

It is considered that the worst-case scenario in regards to surface impacts would probably be associated with maximum panel subsidence and chain pillar subsidence, and has been used in the impact assessment sections of this report.

Associated subsidence impact parameter contours of principal tilt and horizontal strain have been subsequently derived using the calculus module provided in **Surfer8®** and the Worst-case subsidence contours. The outcomes are shown in **Figures 37a, b** and **38a, b** respectively.

It should be understood that the predicted tilts and curvatures will not change significantly if the minimum chain pillar subsidence were to occur.

The pre and post mining topography have been generated from the aerial survey and the predicted subsidence contours. The results are given in **Figure 39**.

12.0 Subsidence Impact Assessment and Management Strategies

12.1 General

The **DoP, 2008** report provides a comprehensive summary of the range of potential mine subsidence effects on the environment and impact management techniques that have been considered in this document.

DoP, 2008 recommends that subsidence Risk Management Zones (RMZs) be defined around sensitive features within a mining lease before subsidence occurs. The RMZs in the Southern NSW Coalfield may be defined by either an AoD of 40^o or 400 m distance from the feature (whichever is the greater) to the limits of mining where significant subsidence is likely to occur (i.e. longwall or pillar extraction panels). The above setback criteria are based on the limits of observed crack impacts to creek rock bars, and were associated with 'unexpected' valley closure and uplift movements.

The RMZs are intended to provide focus for future impact management of sensitive features such as:

- 3rd and higher order streams and creeks
- significant rock bars
- significant cliff lines and overhangs
- ground dependant eco-systems
- valley infill swamps
- significant Aboriginal heritage sites (that may be associated with the above features)
- sensitive man-made developments

The location of an RMZ is considered to be the first step in managing prediction uncertainties and the potential impacts associated with subsidence, valley uplift and closure, and far-field displacements. It will then be necessary to determine what constraints on mining may be required within the RMZ to reduce subsidence effects to 'repairable' or ALARP (*'As Low As Reasonably Practicable')* levels. For the longwall mining case the RMZs are deemed to be mining exclusion zones unless local data is available that allows the limits of mining to encroach inside the RMZ without significant impact on the feature.

Based on the recommendations of **DoP, 2008** and consultation with stakeholders, the natural and man-made features within the WWC mining lease that will require an RMZ to be applied include:

- significant Aboriginal heritage sites (associated with creek rock bars and overhangs)
- first and second order creeks with shallow cover depths of < 70 m
- F3 Freeway infrastructure and the adjacent services (high pressure gas and petroleum pipelines and several Optic Fibre cables)
- three communications towers (CT1 to CT3)

Based on available cracking data in similar terrain in the northern and southern domains of WWC, an RMZ defined by an AoD of 26.5^{o} from the limits of mining is considered appropriate for the sensitive features listed above.

RMZs are not proposed for the following site features that may be either repaired after mining impact or accepted as being likely to sustain cracking damage:

- Aboriginal archaeological sites
- first and second order creeks with cover depths > 70 m
- low height cliffs < 20 m high and steep slopes
- McArthy's Dam
- The Great North Walk
- Wakefield Road

The following sub-sections provide an assessment of the worst-case subsidence impacts that could occur to the existing features within the mine lease (with or without an appropriate RMZ applied) and suggested impact management strategies that will be required to minimise long-term impacts to the feature after mining is completed.

Due to the uncertainties associated with mine subsidence prediction and associated impacts for a given mining geometry and geology etc, a credible range of impact outcomes (based on probabilistic design methodologies) have been provided to assist with the development of effective subsidence management plans for the existing site features.

Discussions of likelihood of impact occurrence in the following sections generally refer to the qualitative measures of likelihood described in **Table 20**, and are based on probabilistic terms used in **AGS, 2007** and **Vick, 2002**.

The terms ‘mean’ and ‘credible worst-case’ used in this report generally infer that the predictions will be exceeded by 50% and 5% of panels mined with similar geometry and geology etc. Using lower probability of exceedence values (i.e. <5% probability of exceedence) may result in potentially uneconomic or marginal mining layouts with a negligible gain to impact reduction.

The consequences of an exceedence will therefore need to be considered when selecting an appropriate probability of exceedence likelihood.

Table 20 - Qualitative Measures of Likelihood

Likelihood of Occurrence	Event implication	Indicative relative probability of a single event
Almost Certain	The event is expected to occur.	90-99%
Very Likely	The event is expected to occur, although not completely certain.	75-90%
Likely+	The event will probably occur under normal conditions.	50-75%
Possible	The event may occur under normal conditions.	10-50%
Unlikely*	The event is conceivable, but only if adverse conditions are present.	5-10%
Very Unlikely	The event probably will not occur, even if adverse conditions are present.	1-5%
Not Credible	The event is inconceivable or practically impossible, regardless of the conditions.	<1%

Notes:
+ - Equivalent to the mean or line-of-best fit regression lines for a given impact parameter presented in **ACARP, 2003**.
* - Equivalent to the worst-case or U95%CL subsidence impact parameter in **ACARP, 2003**.

12.2 Surface Cracking

12.2.1 Potential Impacts

The development of surface cracking above a longwall panel is caused by the bending of the overburden strata as it sags down into the newly created void in the coal seam. The sagging strata are supported by collapsed roof material (goaf) that slowly compresses to final maximum subsidence. The stiffness and bulking characteristics of the overburden, and vertical stress acting on the goaf, will influence the final maximum subsidence magnitude.

Tensile fractures generally occur between the panel ribs and the point of inflexion where convex (i.e. hogging) curvatures and tensile strains develop. The point of inflexion is assessed to be located 29 to 57 m from the panel ribs for the range of mining geometries proposed. Tensile fractures can also occur above chain pillars that are located between extracted panels.

The surface cracks in the tensile strain zones will probably be tapered and extend to depths ranging from 5 m to 10 m, and possibly deeper in near surface rock exposures and ridges.

Based on the predicted range of maximum transverse tensile strains from 2 to 38 mm/m for cover depths of 360 m to 70 m, maximum surface cracking widths of between 20 mm and 380 mm may occur within the limits of extraction.

The location and frequency of surface cracking tends to occur within 10 m to 15 m from the tensile strain peaks and may consist of one to five cracks, depending on whether the near surface lithology comprises soil or rock.

Compressive shear fractures will generally develop in the central area above a longwall panel and between the inflexion point locations. This zone is where concave (i.e. sagging) curvatures and compressive strains will develop. Cracks within compressive strain zones are generally low-angle shear cracks caused by failure and shoving of near surface strata. Some tensile cracks can also be present, due to buckling and uplift of near surface rock in the base of gullies or man-made cuttings (see section on valley closure also).

Based on reference to **ACARP, 2003**, the cracking described above will probably have developed by the time the longwall face has retreated for a distance equal to 1 to 2 times the cover depth past a given location.

Predicted maximum surface crack width and shear displacement contours associated with post-mining tensile and compressive strains are presented in **Figures 40a** and **40b**.

Arcuate, tensile cracks will also probably develop up to 20 or 30 m behind the advancing goaf edge of the longwall panels. The majority of these cracks are likely to range in width between 10 mm and 50 mm and will generally close in the central, compressive strain areas of the longwall panels after the subsidence trough has fully developed.

Cracking is may also occur above the chain pillars and just outside the limits of extraction, but only within the outer 16.5 m of the pillar or solid ribs (see below). A database of surface cracking location has been compiled from LWs 22 to 36 for assessing the probability of crack development (see **Appendix F**).

Twenty two cracks have been observed above 30 m to 45 m wide chain pillars between LWs 22 to 36. The cracks were 10 mm to 100 mm wide and occurred within 16.5 m of the panel extraction limits (see **Figure 40c**).

Undermining ridges can also result in surface cracks occurring up-slope and outside the limits of extraction for significant distances due to rigid block rotations. This phenomenon has been previously observed up to 50 m outside the extraction limits of LW29, which is located to the northeast of the Southern Domain. The effective angle of draw to the cracks was 11^{o} or 0.2 x the cover depth. Further details on this issue are provided in **Appendix F**.

In regards to the creeks and watercourses in the study area, the following potential impacts are expected to occur due to mine subsidence:

- Transient surface cracking is likely to develop behind the retreating longwall face and along and across creek beds or watercourses that are undermined (see Figures 38a, b).

- Final surface crack width distribution will be influenced by the strain contours shown in **Figures 40a** and **40b**. The location of the cracks observed to date is given **Figure 40c**.

- Cracks that occur within the drainage gullies or creek beds could result in sub-surface re-routing of surface flows during storm periods. The impacts in most cases should be self-healing, due to the high sediment bed load that will accumulate in the cracks after several storm events.

- The depth of surface cracking in alluvial soils along creek beds will be affected by the depth to rock and width of cracking at rock head. Where shallow rock or bedrock is exposed, the maximum crack depth would be expected to range between 2 and 10 m.

- It is likely that there will be a short term increase of existing pre-mining erosion rates and head cuts along the creeks inside the up-stream ribs of the proposed longwall panels. The erosion rates are expected to reach a new equilibrium after several storm events have occurred, with sediment likely to accumulate where net surface gradients have been decreased after mining (see **Figure 45**)

Cockle Creek has been previously undermined by Northern Domain longwall panels (LWs 27, 28, 31 and 32) with no significant impacts observed or measured to-date, after subsidence of up to 1.5 m and cover depths > 130 m (**Umwelt, 2004**).

12.2.2 General Impact Management Strategies

Surface crack repair works in accessible terrain may need to be implemented around the affected areas of the mining lease, and in particular, across any public (or private) access roads or ephemeral watercourses that do not infill naturally with sediment due to natural geomorphic processes.

All remediation will be undertaken in consultation with the relevant stakeholders and may involve either the ripping / tilling of small to moderate sized cracks or pouring crushed rock, gravel, concrete or grout into larger sized cracks.

12.2.3 Impact Management Strategies on Ridges and Steep Slopes

It is unlikely that cracks that develop on the steep side slopes and ridges will be able to be practically and safely inspected and repaired due to poor access and safety issue constraints. It has therefore been necessary to consider what the likely consequences will be if surface cracks due to mine subsidence cannot be safely repaired by WWC (see **Section 12.3**).

The assessment of potential cracking impact of the proposed longwall mining beneath the steep slopes is considered to be primarily an issue of slope instability. It is likely that the stability of the slopes will be dependent on the extent and magnitude of cracking and whether the reinforcing effect of existing tree density and vegetation will resist the destabilising force of water percolating deeper into the slopes.

It is noted that there has been a lack of general slope instability events on the already steep slopes to-date at WWC and is probably due to the shear strength of the residual soils and rock as well as the tree density and vegetation present on the ridges above the subsided areas.

12.2.4 Impact Management Strategies in Creeks and Watercourses

Remediation of subsidence impacts within creeks and watercourses will be undertaken in accordance with the existing subsidence management strategies that have been employed by WWC in the past. These strategies have been successfully developed in close consultation with the relevant stakeholders.

The existing strategies to address subsidence crack impacts in creeks and watercourses have been to undertake pre-mining and post-mining inspections. The results of these inspections are then communicated to the respective stakeholders. Should a significant impact be identified during these inspections, an appropriate remediation strategy will be developed.

Due to the variation in type, extent and location of potential remediation required, each site will require a specific remediation strategy. Consultation with the relevant stakeholders will determine the specific remediation strategy for each specific site. Consultation with DCCW has suggested that natural regeneration may be the favoured management strategy in most scenarios, due to the likely level of disturbance caused by other remediation strategies.

12.3 Sub-Surface Cracking

12.3.1 Sub-Surface Fracturing Zones

The caving and subsidence development processes above a longwall panel usually results in sub-surface fracturing and shearing of sedimentary strata in the overburden, see **Figure 41**. The extent of fracturing and shearing is dependent on mining geometry and overburden geology.

International and Australian research on longwall mining interaction with groundwater systems indicates that the overburden may be divided into essentially three or four zones of surface and subsurface fracturing. The zones are generally defined (in descending order) as:

- Surface Zone
- Continuous or Constrained Zone
- Fractured Zone
- Caved Zone

Starting from the seam level, the Caved Zone refers to the immediate mine workings roof above the extracted panel, which has collapsed into the void left after the coal seam has been extracted. The Caved Zone usually extends for 3 to 5 times the mining height above the roof of the mine workings.

The Fractured Zone has been affected by a high degree of bending deformation, resulting in significant fracturing and bedding parting separation and shearing. The Fractured Zone is supported by the collapsed material in The Caved Zone, which usually has a bulked volume equal to 1.2 to 1.5 times its undisturbed volume.

The Continuous or Constrained Zones refer to the section of overburden which has also been deformed by bending action, but to a lesser degree than the Fractured Zone below it.

The Surface Zone includes the tensile and compressive surface cracking caused by and is assumed to extend to depths of 5 to 10 m in the Newcastle Coalfield.

Based on reference to **Whittaker and Reddish, 1990** and **ACARP, 2003**, the impact of mining on the sub-surface aquifers and surface waters, requires an estimate of the '*Continuous*' and '*Discontinuous*' heights of fracturing or the A and B Zones - shown schematically in **Figure 41**.

Continuous sub-surface fracturing (A-Zone) refers to the zone of cracking above a longwall panel that is likely to result in a direct flow-path or hydraulic connection to the workings, if a sub-surface (or shallow surface) aquifer was intersected.

Discontinuous sub-surface fracturing (B-Zone) refers to the zone above the A-Zone where there could be a general increase in horizontal and vertical rock mass permeability, due to bending or curvature deformation of the overburden. This type of fracturing does not usually provide a direct flow path or connection to the mine workings like the A-Zone; however, it is possible that B-Zone fracturing may interact with surface cracks, joints, or faults. This type of fracturing can therefore result in an adjustment to surface and sub-surface flow paths, but may not result in a significant change to the groundwater or surface water resource in the long-term.

In regards to the general zones of fracturing mentioned earlier, the A-Zone may be assumed to include the Caved and Fractured Zones, and the B-Zone will develop in the Constrained Zone, Both A and B-Zones can extend to the Surface Zone and will depend on the mining height, cover depth, geology and panel width.

Two empirically-based models (**Forster, 1995** and **ACARP, 2003**) and have been used in this study to predict the A and B-Zone heights of sub-surface fracturing within the study area.

The **Forster, 1995** model was developed from deep multi-piezometer data from subsided overburden in the Central-Coast area of the Newcastle Coalfield and in-directly defines the A and B-Zones as a function of the mining height (the model refers to the A and B-Zones as the tops of the Fractured and Confined Zones respectively - see **Figure 42** for the model fracture zone definitions).

The **Forster, 1995** model predicts that the height of the Fractured or A-Zone will generally range between 21 and 33 times the mining height (T). The predicted extent or height of the

Confined or B-Zone and its thickness will be dependent on the cover depth and height of A-Zone fracturing.

The **ACARP, 2003** model was derived from the **Forster, 1993** Model data, and supplemented with drilling fluid loss records from surface to seam drilling logs in subsided, fractured overburden from the NSW Southern Coalfield and Oaky Creek Mine in the Bowen Basin.

The **ACARP, 2003** model includes several of the key parameters defined by **Whittaker and Reddish, 1989** and referred to in **Mark, 2007**. The additional parameters include the panel width, cover depth, maximum single panel subsidence and geological conditions (i.e. Subsidence Reduction Potential). The mining height is not applied directly, but indirectly through the subsidence prediction (further model development details may be found in **Appendix A**).

The measured data in **ACARP, 2003** has been plotted as the height of A or B-Zone fracturing /cover depth v. S_{max}/Effective Panel Width^2. A log-normal regression line has subsequently been derived to give predictions of mean and U95%CL values for both fracture zones.

12.3.2 Sub-Surface Fracture Height Predictions

The predicted mean (average) and Upper 95% Confidence Limit (i.e. worst case) values for the **ACARP, 2003** model's *Continuous and Discontinuous sub-surface fracturing heights* above longwall panels are summarised in **Tables 21a** and **21b** and presented in **Figures 43** and **44a**. The results for the **Forster, 1995** model are also provided.

The tables also indicate in bold and italic font where A and B-Zone fracturing could develop to within 10 m from the surface respectively, and is the depth where interaction with surface cracking may start to occur.

Table 21a - Summary of Predicted Sub-Surface Fracturing Heights above the Proposed LWs 38 to 43 and 47 to 50 in the Western Domain

Cross Line #	Long wall Panels	Cover Depth, H (m)	Mining Height, T (m)	Single Panel S_{max} (mean) (m)	Predicted Fracture Heights (m)				
					Continuous (A-Zone Horizon)			Discontinuous (B-Zone Horizon)	
					ACARP, 2003 Model (mean - U95%CL)		Forster, 1995) 21-33T	ACARP, 2003 Model (mean - U95%CL)	
9	43	75	4.25	2.47	59	**79**	**89 - 140**	*85*	*98*
10	42	78	4.35	2.52	61	**81**	**91 - 144**	*87*	*101*
9	42	80	4.00	2.32	60	**81**	**84 - 132**	*88*	*102*
10	47	80	4.10	2.38	60	**82**	**86 - 135**	*88*	*102*
9	41	90	4.30	2.43	63	**87**	**90 - 142**	*96*	*111*
9	41	95	4.30	2.49	63	**89**	**90 - 142**	*101*	*119*
8	42	100	3.60	1.91	56	82	76 - **119**	*96*	*113*
9	40	110	4.50	2.55	65	94	95 - **149**	*108*	*127*
8	38	120	4.60	2.07	59	91	97 - **152**	*109*	*130*
6	40	130	4.25	2.32	63	98	89 - **140**	*118*	*140*
7	39	140	4.35	2.35	69	106	91 - **144**	*127*	*151*
7	38	150	4.50	1.80	63	103	95 - **149**	128	*154*
7	41	160	4.05	1.67	62	105	85 - 134	133	*161*
9	50	175	3.70	1.39	65	112	78 - 122	143	*174*
7	49	190	3.70	1.14	52	103	78 - 122	142	*175*
8	48	205	3.80	1.01	30	85	80 - 125	134	170
8	49	215	3.80	0.72	16	74	80 - 125	129	167
7	48	265	3.70	1.26	27	99	78 - 122	165	211
6	47	290	3.70	1.22	33	111	78 - 122	183	234
4	43	320	3.50	0.79	23	116	74 - 116	197	253
2	43	360	3.65	0.80	30	119	77 - 120	213	276

Bold - Direct hydraulic connection to the surface is considered 'likely' to 'possible' if A-Horizon prediction within 10 m of the surface.
Italics - Discontinuous fracturing may interact with surface cracks if B-Horizon within 10 m of surface, resulting in surface flow re-routing.
Underlined - model results are questionable.

Table 21b - Summary of Predicted Sub-Surface Fracturing Heights above the Proposed LWs 44 to 46 in the Southern Domain

Cross Line #	Long wall Panels	Cover Depth, H (m)	Mining Height, T (m)	Single Panel S_{max} (mean) (m)	Predicted Fracture Heights (m)				
					Continuous (A-Zone Horizon)			Discontinuous (B-Zone Horizon)	
					ACARP, 2003 Model (mean - U95%CL)		**Forster, 1995**) 21-33T	**ACARP, 2003** Model (mean - U95%CL)	
13	44	125	4.7	1.69	58	92	99 - **155**	111	*133*
12	44	130	4.7	1.63	58	93	99 - **155**	114	*137*
12	46	135	4.7	1.58	60	96	99 - **155**	118	*141*
11	44	145	4.7	1.51	63	102	99 - **155**	125	*151*
12	46	145	4.7	1.44	65	104	99 - **155**	127	*152*
11	44	150	4.7	1.42	63	103	95 - **155**	128	*154*
13	46	150	4.7	1.41	67	107	99 - **155**	131	*157*
11	45	155	4.7	1.45	66	107	99 - **155**	133	*160*
11	46	180	4.7	1.11	70	118	99 - **155**	150	*181*

Bold - Direct hydraulic connection to the surface is considered 'likely' to 'possible' if A-Horizon prediction within 10 m of the surface.
Italics - Discontinuous fracturing may interact with surface cracks if B-Horizon within 10 m of surface, resulting in surface flow re-routing.

12.3.3 Discussion of A-Zone Horizon Model Predictions

The **ACARP, 2003** model predicts that heights of *Continuous sub-surface fracturing* (or A-Zone) is estimated to range between 52 m and 119 m for cover depths from 75 to 360 m and mining heights of 3.3 m to 4.7 m. Based on the mean and U95%CL values, the height of A-Zone is likely to interact with the surface cracking zone for cover depths < 70 m and possible for cover depths < 100 m (see **Figure 43a**).

The outcomes of the **Forster, 1995** model predicts that *Continuous fracture heights* could extend between 74 m and 155 m above the proposed workings, and therefore indicates that sub-surface cracks could intersect with surface cracks for cover depths <165 m.

However, the experience in the Northern Domain at WWC to-date, where cover depths ranged from 130 to 250 m with longwall mining heights of 4.5 to 4.7 m, has not resulted in continuous fracturing to the surface watercourses. It is therefore considered that the upper limit of the A-Zone fracture heights (based on the **Forster,1995** model) are likely to be < 120 m or <26 x mining height, if a 10 m surface crack depth is assumed.

A similar US version of the **Forster, 1995** model (refer **Mark, 2007**) indicates that the height of *Continuous fracturing* could range between 10T and 24T (i.e. 35 m and 113 m for the proposed mining heights of 3.5 m and 4.7 m), which is slightly lower than the WWC predictions.

12.3.4 Discussion of B-Zone Horizon Model Predictions

The **ACARP, 2003** model predicts that heights of *Discontinuous sub-surface fracturing* is likely to range between 67 and 276 m for the given mining geometries. The *Discontinuous sub-surface fracturing* is therefore considered 'likely' to interact with surface cracks, where cover depths are < 213 m, and 'possible' up to 276 m.

Creek flows may be re-routed to below-surface pathways and re-surfacing down-stream of the mining extraction limits in these areas. For areas with cover depths > 276 m, surface water impacts from *Discontinuous sub-surface fracturing* interaction are ‘unlikely’ to occur.

A similar US version of the **Forster, 1995** model (refer **Mark, 2007**) indicates that the height of *Discontinuous fracturing* could range between 24 T and 60T (112 m to 282 m). A comment is made in **Mark, 2007**, that the “variation is also probably due to differences in geology and panel geometry”.

12.3.5 Discussion of Prediction Model Uncertainties

Due to the complexity of the issue overall, it is difficult to ascertain which of the two Newcastle Coalfield-based models presented herein is likely to be the most accurate. It has therefore been considered necessary to review the assumptions made in each model.

Both models indicate that the height of continuous fracturing is fairly insensitive to depth of cover (see **Figure 44a**). It is apparent however, that the **Forster, 1995** model predicts a higher continuous fracture height than the **ACARP, 2003** model generally.

The heights of continuous (and discontinuous) sub-surface fracturing are also probably influenced by the panel width and overburden spanning capability to some degree. Other subsidence workers in the Southern Coalfield claim that fracture heights could extend as high as 1.4 x Panel Width, which would indicate a fracture height of 250 m is possible for the West Wallsend panels. That particular model however, does not distinguish between continuous and discontinuous fracturing and is therefore considered to be a discontinuous sub-surface fracture height model only.

The height of continuous fracturing data presented in **Forster, 1995** and **ACARP, 2003** infers that the fracture height is *not significantly* influenced by the panel width (see **Figure 44b**).

This would seem to contradict arching theory, where the height of the ‘arch’ or fractured zone would be expected to increase as the panel width increases. However, as the effective width of the panel decreases with increasing height above the workings, the spanning capability of the rock ‘beams’ will also increase and limit the height of fracturing. The presence of spanning massive strata would also have a limiting affect on fracture height development.

What is clear is that there a high degree of uncertainty in predicting the A and B-Zone horizons using either of the models and that the available management strategies will need to carefully consider the consequences of the predictions if they are exceeded (see **Section 12.3.7**).

12.3.6 Impact on Rock Mass Permeability

In regards to changes to rock mass permeability, **Forster, 1995** indicates that horizontal permeabilities in the *Continuous sub-surface fracture* zone above longwall mines (see **Figure 42**) could increase by 2 to 4 orders of magnitude (e.g. pre-mining $k_h = 10^{-9}$ to 10^{-10} m/s; post-mining $k_h = 10^{-7}$ to 10^{-6} m/s).

Vertical permeability's could not be measured directly from the boreholes but could be inferred by assuming complete pressure loss in the 'A Zone', where direct hydraulic connection to the workings occurs. Only a slight increase in the B-Zone or indirect / discontinuous fracturing develops (mainly due to increase in storage capacity) from bedding parting and joint separation. It is possible that minor vertical flows will occur from B-Zone into A-Zone (and workings) as well.

Discontinuous fracturing would be expected to increase rock mass storage capacity and horizontal permeability without direct hydraulic connection to the workings. Rock mass permeability is unlikely to increase significantly outside the limits of extraction.

12.3.7 Impact Management Strategy

It is understood that there are no subsurface aquifers of potential resource significance within the overburden that could be affected by *continuous and/or discontinuous fracturing* above the extracted longwall panels. Subsequent groundwater and surface aquifer impact studies have considered the high level of uncertainty in regards to predicting the height of each zone of sub-surface fracturing.

Based on **Table 20**, the **ACARP, 2003** model outcomes have been assessed in accordance with the Likelihood of Occurrence that continuous fracturing will intersect with surface cracks that extend to 10 m depth, and is summarised in **Table 22a** and **Figure 44a**.

Table 22a - Event Likelihood Assessment for Continuous Fracturing Above the Proposed Longwall Panels Intersecting with Surface Fractures

Likelihood of Occurrence*	Cover Depth (m)	Indicative Relative Probability of a Single Hazardous Event
Likely	50 - 70	25% - 75%
Possible	70 - 100	5 - 25%
Unlikely	100 - 165	1 - 5%
Very Unlikely	165 - 360	<1%

* - refer to **Table 20** for definitions of likelihood of occurrence.

The practical options available for controlling subs-surface fracturing are limited to the following (in order of increasing impact to proposed mining layouts):

(i) repair large surface cracks as soon as possible if they occur along the creeks,

(ii) decrease mining height to limit continuous sub-surface fracture heights to 10 m below the surface (*note: the minimum practical longwall mining height is 3.3 m*).

(iii) pull the longwall panel back to a minimum cover depth contour to achieve a similar outcome as item (ii).

Regardless of which prediction model is adopted, the confidence in the predictions may be increased by measuring the height of fracturing above areas where any adverse impact is unlikely to occur in deeper areas with cover depths > 130 m. The A-Zone heights are assessed to range between 21 and 26 T until local fracture height data becomes available.

The proposed mining layout presented in this report has already been adjusted during various risk management assessments using both options (ii) and (iii) after consideration of impact consequences (see further below for details). The minimum recommended Option (ii) mining heights for minimising the likelihood of seam to surface connective cracking is summarised in **Table 22b**.

Table 22b - Recommended Mining Height Limits to Minimise the Likelihood of Connective Cracking to the Surface

Cover Depth (m)	Assumed Continuous Fracture Height which could intersect with surface cracks (m)	Recommended Average Longwall Face Height (m)*
130	120	5.7 - 4.6
120	110	5.2 - 4.2
110	100	4.8 - 3.8
100	90	4.3 - 3.5
90	80	3.8 - *3.1*
80	70	*3.3 - 2.7*
70	60	*2.9 - 2.3*

Notes:
* Continuous Fracture Height = 21 to 26 x Average Mining Height (based on **Forster, 1995**).
Italics - Mining Height < Practical mining height limit of 3.3 m.

Clearly, for cover depths < 100 m, it may not be possible to prevent connective cracking from developing by reducing the mining height (T) to the practical minimum of 3.3 m, if the A-Horizon exceeds 26 T. As significant areas of the Western Domain Area will have cover depths between 70 and 100 m, it has therefore been necessary to consider the consequences of connective cracking impacts to the creeks.

Based on a risk assessment of the potential impacts to Ryhope Creek above the Western Domain, it was decided by WWC to pull back several of the proposed finishing points for LWs 49 and 50 to the 70 m cover depth contour to limit the effective cover depth within the

180 m from the end of the 178.6 m wide panels to approximately 100 m (i.e. this is the location wher maximum panel subsidence is likely to occur). Longwall 51 was eliminated altogether due to the shallow cover depth issue for this creek.

Several sections of the proposed panels 39 to 43 and 47 to 48 have cover depths ranging from 70 to 100 m, and may therefore result in connective cracking developing to the surface (even if the mining height is limited to 3.3 m) and along Diega and South Diega Creeks (see **Figure 1b**).

Based on discussions with the specialist groundwater consultant for the project, the absence of surface alluvium and ephemeral nature of the creeks/gullies is unlikely to result in significant degradation of these particular creeks or inrush event into the underground workings. It is considered more likely that any re-directed surface flows will be manageable underground and able to be repaired at the surface.

The above assessment is of course dependant on our limited understanding of the continuous fracture heights in this area of the mine until monitoring/measurement data becomes available.

12.4 Slope Stability

12.4.1 Potential Impacts

The following potential impact on the cliff lines, steep slopes, watercourses, developments and public access ways due to natural weathering processes and mine subsidence have been identified:

- General slope instability (translational / rotational sliding) of cliff lines and steep slopes;
- Local instability of cliff lines and steep slopes due to cracking, toppling failures and erosion;
- Rock fall movements from cliff lines and down slopes (falling, bouncing and rolling boulders) from cliffs and steep slopes.

The likely impacts of the proposed mining layout have been assessed in the following sections. Options to manage these impacts appropriately have also been provided.

12.4.2 General Ridge Stability

The likelihood of *en-masse* sliding (i.e. a landslip) of the ridges or hills over basal siltstone beds that have been cracked and tilted by subsidence has been assessed based on reference to **Das, 1996** and the landslide risk assessment terminology presented in **AGS, 2007**.

The impact of subsidence on local stability of steep soil slopes, cliff lines on the ridges and incised channel beds in the watercourses are assessed in **Section 12.4.3**.

It is considered that the stability of cliff lines and steep soil slopes will be dependent on the following key changes to the surface topography due to mine subsidence:

(i) existing slope magnitude and change in gradient due to tilt;

(ii) orientation and depth of cracking due to by tensile strain;

(iii) presence of water in and on-going erosion of cracks;

(iv) depth of soil cover;

(v) stabilising effect of vegetation;

(vi) destabilising effect of seismic vibrations of overburden or earthquake events;

(vii) the limited access surface inspections and crack repair works.

The predicted post mining surface slope gradients are presented in **Figures 45a** and **45b**. The predicted subsidence, tilt, strain, and crack width contours are presented with the post-mining topography and relevant surface features in **Figures 46** to **49** respectively.

Based on reference to **Fell et al, 1992,** any siltstone and mudstone units that may be present at the base of massive conglomerate units on the the ridges on the site have been assumed to have a lower bound, drained angle of friction (Ø') of 15°. Saturated slopes with water filled joints or mining-induced cracks have been assumed representative of worst-case conditions.

The predicted tilts for the slopes above the proposed longwalls are expected to change existing gradients by between 1° and 2° (i.e. 10 and 35 mm/m tilt). This would indicate that any near-surface rock beds will have their dip increased from about 3° to 5° to a range of 4° to 7° on east and south facing slopes within the mining lease.

The predicted strains for the slopes above the proposed longwalls are expected to range between +/- 3 and +/- 5 mm/m. This would indicate that any near-surface cracking of between 30 and 50 mm. The cracks are likely to occur at a spacing of 1 m to 3 m within the tensile strain zones above the longwall panels and extend to depths of 2 to 10 m.

The Factors of Safety against *en-masse* sliding of a natural slope in the study area due to the predicted bedding dip increase and surface cracking effects mentioned above are estimated for the worst-case condition by the method presented in **Das, 1998** as follows:

Before mining: FoS = (u_b/u_r) tan(Ø')/tan(theta) = 0.6 tan (15°)/tan(5°) = 1.8.

After mining: FoS = (u_b/u_r) tan(Ø')/tan(theta) = 0.6 tan (15°)/tan(7°) = 1.3.

where:

u_b = buoyant unit weight of sandstone above the mudstone = 14 kN/m^3
u_r = dry unit weight of sandstone above the mudstone = 24 kN/m^3

Based on a recommended minimum FoS of 1.2 to 1.3 (**Levanthal and Stone, 1995**) for the worst-case scenario, it is assessed that it is 'very unlikely' that a large scale instability or landslip will occur in the long-term due to mine subsidence within the study area.

12.4.3 Steep Slope Stability

A similar exercise was completed on the stability of the 20 to 35 degree slopes below the cliff lines in the study area. The slopes were been assessed for dry and wet (saturated) conditions before and after the effects of longwall mining.

The factor of safety (FoS) for translational sliding of the sandy clay soils over the sandstone and shale strata units has been calculated using a simple force balance model defined in **Das, 1998**. The weight force of a unit width of soil and water (if present) acting down the slope and the frictional resistance against sliding has been calculated as follows:

W = $(d_s g)bh$ = weight of a 1 m wide soil block with density d_s, gravity constant, g, length b, and depth h.

T = (W+U)sin(a) = driving force along potential failure plane of slope, a.

V = $d_w gz/\cos(a)$ = uplift force of water (with density d_w) in a saturated soil of depth z on the slope.

U= $d_w gz^2/2$ = driving force of water (with density d_w) filled crack of depth z on the slope.

S= c'b/cos(a) + (Wcos(a)-V-Usin(a))tan(p') = sliding resistance along potential failure plane with drained cohesion, c' and drained friction angle, p'.

FoS = S/T = factor of safety against sliding.

The drained soil strength parameters c' and p' were back calculated for the slopes before mining impacts of cracking and tilting. A conservative thickness of the soil profile on the steep slopes was assumed to be 0.5 m, based on the road cuttings present on the site and the knowledge that there have been no sliding failures on the slopes to-date. The above theory indicates that the stability of the slopes will be most sensitive to (i) soil cover thickness and (ii) water filled cracks.

Based on assumed soil parameters of c' of 5 kPa and p' of 28^o for the stiff clayey sands/sandy clays and reference to **Lambe and Whitman, 1969**, an FoS range of 2.09 to 1.19 was estimated for worst-case, pre-mining conditions with saturated, uncracked, 20^o to 35^o slopes .

Based on the predictions of principal tilt and strain on the slopes after mining, the steep slopes were considered likely to be subject to full soil profile cracking at some stage during or soon after mining. The stability assessment was therefore completed for the steep slopes for the range of climatic (i.e. dry or wet) and worst-case mine subsidence impacts.

A summary of the stability assessment is presented in **Table 23** and shown in **Figures 50a** and **50b**.

Table 23 - Summary of Sliding Potential Assessment of the Steep Slopes

Case	Conditions	Driving Forces (kN/m)	Resisting Forces (kN/m)	Factor of Safety
Maximum Slope Angle = 35^o				
Pre-Mining	Dry Slope	6.18	10.80	1.75
	Saturated Slope	6.75	8.05	1.19
Post Mining (Tilt = 20 mm/m)	Dry Slope	6.36	10.82	1.70
	Saturated Slope	6.94	8.01	1.16
	Saturated Slope + water filled cracks	7.93	8.01	1.01
Maximum Slope Angle = 20^o				
Pre-Mining	Dry Slope	3.69	10.71	2.90
	Saturated Slope	4.02	8.42	2.09
Post Mining (Tilt = 35 mm/m)	Dry Slope	5.65	12.83	2.27
	Saturated Slope	6.17	9.58	1.55
	Saturated Slope + water filled cracks	8.39	9.58	1.14

Details of the stability analysis and schematic drawing of the force system assumed are presented in **Appendix D**.

The potential or likelihood of slope failure may then be considered based on reference to **Luo and Peng, 1999,** which provides the following assessment of 'sliding potential' categories for the predicted FoS values:

FoS > 1.8 'Low Potential' for slope failure

1.25< FoS > 1.8 'Medium Potential' for slope failure

FoS < 1.25 'High Potential' for slope failure

The above values are consistent to values often used to design cuttings and fill embankments in civil works, with long and short-term stability criteria set at 1.5 and 1.2 to 1.3 for average and lower bound peak material strengths (refer to **Leventhal and Stone, 1995**). An FoS as low as 1.0 may also be acceptable for short-term adverse loading conditions due to water filled cracks and earthquakes. Other mitigation measures such as drainage and repair works are generally required to control these short-term, high failure potential conditions.

The slopes in the Western Domain (in their current condition) are assessed to have a 'Low' to 'Medium' Sliding Potential over an extreme range of climatic conditions (i.e. Dry to Saturated) with an FoS range of 2.9 to 1.2.

Based on the above, it is considered that the proposed mining impacts on the slopes could result in marginally stable conditions developing at locations where tensile cracking has occurred and prolonged rainfall events have saturated the soil and filled the cracks to the surface (i.e. there will be 'High' potential for instability).

However, it is considered that the high density of tree and vegetation coverage on the slopes will allow a minimum design FoS of 1.0 for this impact scenario and therefore considered acceptable in risk management terms.

In summary, it is considered that the potential for steep soil slope failure after mining would be 'High' for the predicted tilts, strains and cracks but may be reduced to 'Medium' potential overall, due to the high density of trees and vegetation. The consequence of a slope failure is likely to be localised and unlikely to impact on slope aesthetics or public safety.

12.4.4 Down-Slope Soil Movements due to Subsidence

The FoS against sliding (of a dry slope) may also be used to determine the potential worst-case increase in vertical and horizontal displacements due to mine subsidence effects in hilly terrain, using the empirical model presented in **Luo and Peng, 1999**. The reciprocal of the FoS value (i.e. stress/strength) for a soil slope under additional stress from mining has been found to provide a good indication of down-slope soil movements.

The model calculates the additional displacement increments for soil slopes when subject to mine subsidence as follows:

dV = G.S.sin(a) = additional vertical subsidence increment on a steep slope;

dH = G.S.cos(a) = additional horizontal subsidence increment on a steep slope;

where

a = slope angle;

G = empirically derived proportionality coefficient = (1/FoS); see **Table 24**.

S = V.sin(a) + U.cos(a) = the down slope displacement and V and U are predicted subsidence and horizontal displacements for a flat terrain model.

A summary of expected increases in the soil movements are presented in **Table 24**.

Table 24 - Predicted Worst-Case Down-the-Slope Soil Movements after Mine Subsidence

Maximum Slope Angle	Predicted Subsidence V (m)	Predicted Horizontal Displacement, U (m)	FoS (dry slope)	G (1/FoS)	S (m)	dV (m)	dH (m)
20^o	1.0 - 1.6	0.10 - 0.35	2.27	0.44	0.92	0.30	0.12
35^o	0.6 - 1.0	0.05 - 0.20	1.69	0.59	0.75	0.42	0.32

The results indicate that an additional 300 to 420 mm of subsidence (19 to 26% increase) and 120 mm to 320 mm (60% to 90% increase) of horizontal displacement on the steep slopes above the Western Domain is expected due to topographical effects after the predicted mine subsidence occurs.

Based on the above, predicted strains may increase locally along the ridge crests and toe of the slopes by 100%, with tilts on the slopes themselves increased by 10 to 15%. The overall stability assessments remain unchanged.

12.4.5 Cliff Stability

The FoS of the cliffs to resist sliding movements when subject to mine subsidence, tilt and strain has been assessed using a similar analytical approach to that prepared for the steep soil slopes in **Section 12.4.3**. The model used to calculate the FoS is also a simple force-strength balance model discussed in **Hoek, 2000**.

The bedding beneath the cliffs dip towards the south east at 2 to 3 degrees and may consist of low strength shales and sandstone. Cliff heights of up to 15 m have been assessed for pre-mining and post-mining conditions.

Predicted FoS values of sliding on these beds after the affects of mining (with a range of tilts from 5 to 20 mm/m and water filled cracks behind the cliff crests) have been assessed and summarised in **Table 25**. Residual shear strengths of $c_r = 0$ kPa and $p_r = 15^o$ have been assumed based on **Leventhal and Stone, 1995**. The results are also shown graphically in **Figures 51a** and **51b** in terms of FoS, cliff height and tilt.

Table 25 - Summary of Sliding Potential Assessment of the Cliffs

Case	Conditions	Driving Forces (kN/m)	Resisting Forces (kN/m)	Factor of Safety
Cliff Face Slope = 75^o, Bedding Slope = 3^o, cliff height = 15 m				
Pre-Mining	Dry Cliff	575	2940	5.11
	Wet + Cracked Cliff	1431	2338	1.63
Post Mining (tilted 20 mm/m)	Dry Cliff	637	2354	3.70
	Wet + Cracked Cliff	1462	1864	1.28

Details of the stability analysis and schematic drawing of the force system assumed are presented in **Appendix D**.

Based on the FoS range of 1.28 to 5.11 before and after mining impacts of cracking and a maximum tilt of 20 mm/m (1^o), it is estimated that the cliffs are 'unlikely' to slide *en-masse* after the predicted mine subsidence.

12.4.6 Cliff Line Damage Classification and Ranking System

The impact of longwall mining on the cliffs in the study area has also been assessed based on reference to the damage rating and ranking system presented in **ACARP, 2002**.

The system is an empirical model that was developed based on similar stability and risk assessment methods used by the RTA on managing man-made and natural slopes adjacent to the NSW road network. The ACARP model was developed to take into account the measured responses of cliff lines due to mine subsidence in the Southern and Western Coalfields in NSW. The model also incorporated the method of assessment developed for cliffs in the Western Coalfield by Radloff and Mills (refer **ACARP, 2002**).

The cliff heights in the **ACARP, 2002** model's database range between 10 m and 150 m and are generally significantly greater in height than the cliffs at West Wallsend. The authors of the model also suggest that for cliffs that are deemed to be outside the limits of the database (or in a different coalfield), it may be necessary for the impact parameter limits in the model to be re-calibrated or adjusted upon review of local mining experience. It is therefore considered that the use of the **ACARP, 2002** model could result in conservative assessments of the subsidence impact on the cliffs at this study location.

The relevant extracts from **ACARP, 2002**, which describe the assessment methodology, are presented in **Appendix E**.

The **ACARP, 2002** model essentially allows a holistic approach to the response of cliff faces to mine subsidence, and includes the following three impact categories:

(i) the impacts of mining induced deformation (i.e. expressed in terms of the % length of cliff line affected by rock falls),

(ii) exposure of the public (and mining personnel) to rock falls and the potential loss of aesthetic appeal of the cliffs, and

(iii) the contribution of the natural instability of the cliffs (i.e. the on-going weathering and cliff adjustment processes).

There are a number of factors assigned to each impact category, which are then multiplied by a weighting value to provide a score for each factor. The scores are then summed and ranked as a proportion of the maximum possible score for each category.

It should be noted that it is claimed by the model authors, that any attempt to assess the likelihood of a cliff collapse or rock fall at a particular location is not possible, since the actual stability of the rock face cannot be determined by the appearance of it before mining (this is based on the **ACARP, 2002** author's experiences of cliff rock fall patterns observed during the development of mine subsidence beneath them).

It should also be understood that the predicted % length of cliff line affected by rock falls due to mining are worst-case values, and also include rock falls due to natural weathering processes. It is therefore possible to calculate the background level or percentage of rock falls along a cliff line due to 'natural' causes only by assessing the % of falls for the lowest possible mining impact category at a given site.

*Note: As previously mentioned, the assessment of the mining impact on the cliffs at West Wallsend using the values presented in Table 10.1 from **ACARP, 2002** (see **Appendix C**) are likely to require review after mining, however, it is considered prudent at this stage to adopt them until local data can be obtained.*

A summary of the average and worst-case cliff line impact ranking assessment due to the proposed longwall panels is presented in **Table 26**.

The average and worst-case input values for each impact category factor have been adopted in this study; details of the analysis and results are presented in **Appendix E** for the cliffs in their current condition.

Predicted values of subsidence, tilt, strain and horizontal displacement (at the crest of the cliff) are included in the **Appendix D** tables and were derived from the subsidence contours presented in **Figures 46** to **49**.

Table 26 - Summary of Average and Worst-Case Overall Cliff Line Impact Rankings due to Mine Subsidence from Proposed Longwalls

Cliff Line #	LWs	Cliff Face Height (m)	Range of Subsidence at Cliff Toe (m)	Mining Impact: Category 1		Public Exposure/ Aesthetics: Category 2		Natural Instability: Category 3		Overall Cliff Impact Ranking
				Rating	Ranking	Rating	Ranking	Rating	Ranking	
1	42 - 43	3 - 15	0.6 - 1.4	0.56	VH	0.11	VL	0.29	L	**Moderate**
2	47 - 49	3 - 15	1.0 - 1.4	0.67	EH	0.09	I	0.29	L	**Moderate**

Notes:
I = Insignificant; VL = Very Low; L = Low; M = Moderate; H = High; VH = Very High; EH = Extremely High.

The results indicate that the cliff's mining impact rating is 'Very High' to 'Extremely High'; the aesthetics and public exposure impacts is 'Very Low' to 'Insignificant'; with natural instability having a 'Low' impact. The overall impact rating is 'Moderate' after consideration of all three impact categories.

Figure 10.1 in **ACARP, 2002** (see **Appendix E**) indicates that for the assessed mining and natural instability impact category ratings for the cliffs, rock falls are expected to affect 35% to 42% of the 2.2 km of cliff lines present. The 'background' or natural weathering processes are estimated to account for 10% to 15% of the assessed impact, representing a net damage increase of 20 to 32% after extraction of the proposed longwall panels .

Reference to **AGS, 2007** indicates that an overall impact rating of 'Moderate' or less would normally be acceptable to stakeholders, provided appropriate impact management strategies were implemented by WWC.

12.4.7 Local Instability and Erosion

Local instability refers to the following impacts due to subsidence:

(i) toppling failures or rock falls from cliff lines;

(ii) erosion / deposition adjustment of terrestrial / watercourse channel soil slopes.

The predicted impacts of the tilts are also considered 'very unlikely' to cause localised surface instability to soil slopes and low-height cliff lines (< 20 m high) unless mining-induced cracking and increased erosion rates also affect them.

The above assessment particularly applies to the steeply eroded banks present within the drainage gullies and cliff crests along the ridges, which may slump or topple if cracks develop through them.

The potential for rock-fall roll out should also be considered in regards to the development of public safety management plans in areas where cliff lines and steep slopes are present. The trees present below the cliff lines will probably limit the distances at which boulders will be able to roll down slope from the cliff. Boulders of up to 1 m in diameter were observed approximately 100 m below cliff crests and steep, 20^o to 35^o slopes around the study area.

The rate of soil erosion is expected to increase significantly on crack affected slopes that have gradients > 10° and are subjected to the estimated tilt increases of 1^o to 2^o and have exposed dispersive/reactive soils.

Areas with slopes < 10° are expected to have low erosion rate increases, except for the creek channels, which would be expected to re-adjust to any changes in gradient; see **Figures 52a,b** and **Figures 53a** to **53h** for predicted surface level and gradient changes along the Cockle Creek Northern and Southern tributaries, Diega Creek's Northern and Southern tributaries, Ryhope Creek, Bangalow Creek's Northern and Southern tributaries (all in the Western Domain) and Central Creek (Southern Domain). The impact results are summarised in **Table 27**.

The proposed longwalls are not expected to change the creek bed gradients by more than +/- 3^o. Head-cuts would be expected to develop above chain pillars between the panels and on the side where gradients increase. Sediment would be expected to accumulate where gradients decrease.

Table 27 - Potential Worst-Case Gradient Change Assessment for Creek Beds in the Western Domain

Creek	LWs #	Fig. #	Pre-Mining Slope (o)	Predicted Subsidence (m)	Post-Mining Slope (o)	Gradient Change Increase After Mining (o)	Gradient Change Increase After Mining (%)
Cockle Ck North Trib.	38 - 40	53a	3.5 - 25.4	0 - 1.92	3.5 - 24.1	-1.0 - 1.1	**-4.8 - 4.6**
Cockle Ck South Trib.	38 - 40	53b	1.3 - 8.3	0 - 2.65	2.9 - 8.3	-0.7 - 2.2	**-3.4 - 6.5**
Diega Ck	40-42	53c	2.1 - 27.4	0 - 2.15	2.5 - 27.4	-2.4 - 2.1	**-6.7 - 6.5**
Diega Ck South Trib.	41 - 43, 47	53d	1.4 - 22.0	0 - 2.45	1.2 - 21.7	-1.0 - 1.4	**-4.5 - 5.5**
Bangalow Ck North Trib.	48 - 49	53g	9.9 - 36.9	0 - 1.56	9.9 - 36.9	-0.7 - 0.5	**-3.7 - 2.5**
Bangalow Ck South Trib.	49	53h	8.7 - 30.8	0 - 0.9	8.8 - 30.8	-0.8 - 0.9	**-1.0 - 4.1**
Ryhope Creek	49 - 50	53e	1.0 - 24.1	0 - 1.67	1.0 - 24.1	-1.3 - 0.0	**-5.2 - 0.0**
Central Creek	44 - 46	53f	1.1 - 6.1	0 - 2.21	1.1 - 6.3	-0.4 - 1.3	**-2.2 - 5.1**

12.4.8 Impact Management Strategy

To minimise the likelihood of slope and cliff line instability and increased erosion due to cracking or changes to drainage patterns after mining, the management strategy should include:

(i) Surface slope and cliff line displacement monitoring (combined with general subsidence monitoring along cross lines and centre lines);

(ii) Removal of potentially unstable boulders from cliff lines above public access-ways.

(iii) Placement of signs along public access ways warning of rock fall dangers and mine subsidence impacts.

(iv) Infilling of surface cracking, where possible, to prevent excessive ingress of run-off into the slopes and cliffs after each panel is completed.

(v) Areas that are significantly affected by erosion after mining may need to be repaired and protected with mitigation works such as re-grading and re-vegetation of exposed areas.

(vi) On-going review and appraisal of any significant changes to surface slopes such as cracking along ridges, increased erosion down slopes, foot slope seepages and drainage path adjustments observed after each longwall is extracted;

To-date, longwall mining experiences in undulating terrain with ground slopes up to 25^o has not resulted in any large scale, *en-masse* sliding instability due to mine subsidence (or other natural weathering processes etc). The steeper slopes of 25^o to 35^o in the study area are a concern due to lack of access to effect repairs; however, any impacts are likely to be limited by the high tree density.

It is recommended that any stabilisation works to creeks and vegetation affected by rock-falls or erosion should be based on consultation with the relevant government agencies.

12.5 Valley Closure and Uplift

12.5.1 Potential Impacts

Closure and uplift movements can be expected between valley crests whenever longwalls are mined beneath them, based on reference to **ACARP, 2002**.Valley closure and uplift movements can also occur along broader drainage gullies and man-made cuttings, where shallow, interbedded surface rock of moderate to high strength is present.

As discussed in **ACARP, 2002**, when creeks and river valleys are subsided, the observed subsidence in the base of the creek or river is generally less than would normally be expected in flat terrain. This reduced subsidence is due to the floor rocks of a valley buckling upwards when subject to compressive stresses generated by surface deformation. In most cases in the Newcastle and Southern NSW Coalfields, the observed uplift has extended outside steep sided valleys and included the immediate cliff lines and the ground beyond them.

It should also be understood that these movements are strongly dependent on the level of 'locked-in' horizontal stress immediately below the floor of the gullies and more importantly the bedding thickness of the floor strata (i.e. thin to medium bedded sandstone is more likely to buckle than thicker, massive beds). The influence of the aspect ratio (i.e. valley width/depth) is also recognised as an important factor, with deep, narrow valleys having greater upsidence than broad, rounded ones, due to higher stress concentrations.

High horizontal stresses have been measured along the F3 Freeway cuttings (10 MPa at 27 m depth in Cutting 2) and uplift movements of 230 mm have occurred (Cutting 7A) after LW28 was extracted within the angle of draw. Cutting 7A was up to 30 m deep and 50 m wide and had massive conglomerate strata in the cutting walls and thin to medium bedded sandstone in the floor of the cutting. The uplift movements damaged the reinforced concrete pavements and shotcrete lining on the batters.

As a result of the above, subsequent longwalls were mined end-on and towards the freeway (LWs 29 and 30) in the Eastern Domain with no impact to cuttings or embankments. The proposed longwalls in the Western and Southern Domains have similar end-on orientation and will finish further from the freeway than the previous panels.

To-date, closure and uplift measurements across several drainage gullies above extracted panels in the Northern and Eastern Domains (LWs 27 to 37) have not been conclusive, as measured movements have been similar to the available survey accuracy for total station observations (i.e. +/- 20 mm).

If 'closure' and 'upsidence' does occur, it is unlikely to exceed 230 mm at WWC (which has been measured in Cutting 7A on the F3 Freeway) or cause more than minor cracking of the near surface rocks.

The development of upsidence or subsidence cracking may cause localised deviation of surface flows in rocky, ephemeral creek beds into sub-surface routes. The re-routed surface flows would be expected to re-surface downstream of the impacted area.

12.5.2 Impact Management Strategy

The impact of valley bending effects due to mine subsidence may be managed as follows:

(i) Install and monitor survey lines along ephemeral drainage gullies and along gully crests during and after longwall undermining. Combine with visual inspections to locate damage (cracking, uplift).

(ii) Review predictions of upsidence and valley crest movements after each longwall.

(iii) Assess whether repairs to cracking, as a result of upsidence or gully slope stabilisation works are required to minimise the likelihood of long-term degradation to the environment or risk to personnel and the general public.

12.6 Ponding

12.6.1 Potential Impacts

Ponding refers to the potential for closed-form depressions to develop at the surface above longwall panels. They could affect drainage patterns and flora, fauna and groundwater dependent ecosystems.

Ponding locations are generally expected to occur along the creeks and tributaries above the proposed longwall panels with gentle slopes and low-lying areas. The actual ponding depths will depend upon several other factors, such as rain duration, surface cracking and effective percolation and evapo-transpiration rates.

The potential ponding depths have been estimated along several creeks in the Western and Southern Domain and are based on the 1 m post-mining topography contours shown in **Figures 54a** to **54f**.

Reference has also been made to the pre and post mining creek bed profiles presented in **Figures 53a** to **53h** and the pre-mining contours presented in **Figures 54.1a** to **54.1f**. Pre-mining surface contours indicate that some of the locations are already depressions or potential 'ponds'. The increase in ponded depths and volumes have therefore been provided for mining impact assessment purposes.

The potential worst-case pond depths, affected area and volume along each creek, before and after mining, are summarised in **Tables 28a** and **28b** for the Western and Southern Domains respectively. The net increases in potential pond volumes are also given in the table.

Table 28a - Potential Worst-Case Ponding Assessment for Creek Beds in the Western Domain

Creek	**LW #**	**Site #**	**Fig. #**	**Pre-Mining Pond**			**Post-Mining Pond**			**Ponded Area Increase After Mining# (m^2)**	**Ponded Volume Increase After Mining# (m^3)**
				Max. Depth (m)	**Area (m^2)**	**Vol.* (m^3)**	**Max. Depth (m)**	**Area (m^2)**	**Vol.* (m^3)**		
Cockle Creek North Trib.	38	1.1	54a	0.5	59	15	0.8 2.0	137	55	**78**	**40**
		1.2		2.0	196	196		196	196	**0**	**0**
	39	2.1	54a	1.5	103	77	1.5	103	77	**0**	**0**
		2.2		1.8	172	129	2.0	196	196	**0**	**0**
	40	3	54a	1.8	188	170	1.8	188	170	**0**	**0**
Cockle Creek South Trib.	38	4	54b	0.0	0	0	0.3	1,885	282	**1,885**	**282**
	39	5	54b	0.0	0	0	1.0	2,827	990	**2,827**	**990**
	40	6	54b	0.8	236	94	1.0	942	471	**706**	**377**
Diega Creek	40	7	54c	1.0	1,570	785	1.5	5,500	4,125	**3,930**	**3,340**
	41	8	54c	0.7	295	103	1.0	1,885	942	**1,590**	**839**
	42	9	54c	1.0	785	392	1.5	1,100	825	**315**	**433**
Diega Creek South Trib.	41	10	54d	1.0	471	235	1.0	942	471	**471**	**236**
	42	11	54d	0.5	314	79	0.8	1,571	628	**1,257**	**549**
	43	12	54d	0.7	275	96	1.0	1,100	550	**825**	**454**
	47	13	54d	0.15	157	12	0.6	942	565	**785**	**553**
Bangalow Creek North	48	-	-	0	0	0	0	0	0	0	0
	49	-	-	0	0	0	0	0	0	0	0
Bangalow Creek South	49			0	0	0	0	0	0	0	0
Ryhope Creek	49	14	54e	0.0	0	0	0.0	0	0	**0**	**0**
	50	15	54e	0.0	0	0	0.0	0	0	**0**	**0**

Notes:
^ - Area = π x pond width x pond length/4;
* - Volume = Area x Maximum Pond Depth/2.
# - Net increase = Post-mining pond - pre-mining pond.

Table 28b - Potential Worst-Case Ponding Assessment for Creek Beds in the Southern Domain

Creek	LW #	Site #	Fig. #	Pre-Mining Pond			Post-Mining Pond			Ponded Area Increase After Mining# (m^2)	Ponded Volume Increase After Mining# (m^3)
				Max. Depth (m)	Area (m^2)	Vol.* (m^3)	Max. Depth (m)	Area (m^2)	Vol.* (m^3)		
Central Creek	44	16	54f	0.0	0	0	0.6	314	94	**314**	**94**
	45	17	54f	0.0	0	0	1.0	2,121	1,060	**2,121**	**1,060**
	46	18	54f	0.35	314	55	0.6	1,767	530	**1,453**	**475**

Notes:
^ - Area = π x pond width x pond length/4;
* - Volume = Area x Maximum Pond Depth/2.
# - Net increase = Post-mining pond - pre-mining pond.

Several other ponded area increases have also been identified above several low-lying gully areas in the Western Domain and are presented in **Table 28c**.

Table 28c - Potential Worst-Case Ponding Assessment for Other Low-Lying Gully Areas in the Western Domain

LW #	Site #	Fig. #	Pre-Mining Pond			Post-Mining Pond			Ponded Area Increase After Mining# (m^2)	Ponded Volume Increase After Mining# (m^3)
			Max. Depth (m)	Area (m^2)	Vol.* (m^3)	Max. Depth (m)	Area (m^2)	Vol.* (m^3)		
38	19	54b	0.0	0	0	0.7	2,749	962	**2,749**	**962**
38	20	54ab	0.0	0	0	0.8	1,571	628	**1,571**	**628**
38	21	54a	0.0	0	0	0.8	707	282	**707**	**282**
39	22	54a	0.0	0	0	0.7	235	82	**235**	**82**
39	23	54b	0.0	0	0	0.4	706	141	**706**	**141**
39	24	54ab	0.0	0	0	0.6	2,750	824	**2,750**	**824**
39	25	54a	0.0	0	0	0.3	628	94	**628**	**94**
41	26	54c	0.0	0	0	0.3	589	88	**589**	**88**
43	27	54d	1.0	358	178	1.0	471	236	**113**	**58**
47	28	54d	0.0	0	0	1.0	118	59	**118**	**59**

Notes:
^ - Area = π x pond width x pond length/4;
* - Volume = Area x Maximum Pond Depth/2.
# - Net increase = Post-mining pond - pre-mining pond.

The existing surface slopes in the ridge affected areas range between 5° and 25° and up to 35°. They are unlikely to be affected by ponding as the surface slopes are great enough to "absorb" the subsidence without altering the surface flow paths.

12.6.2 Impact Management Strategy

An appropriate management strategy would include:

(i) The development of a suitable monitoring and response plan, based on consultation with the DCCW and regulatory authorities to ensure ponding impacts on existing vegetation do not result in long-term environmental degradation.

(ii) The on-going review and appraisal of changes to surface drainage paths and surface vegetation in areas of ponding development (if they occur), after each longwall is extracted.

Overall, the impact of the increased ponding along the creek beds is likely to be 'in-channel' and therefore the potential effects on existing flora and fauna is likely to be minimal. Further discussion on the ponding impacts are provided in the specialist ecological consultants reports.

12.7 Aboriginal Archaeological Sites

12.7.1 Impact Potential Criteria

The potential for damage to the 67 Aboriginal Heritage Sites identified above the Western and Southern Domains has been estimated based on the predicted final subsidence, tilt, strain and surface gradient change contours presented in **Figures 36b** to **38b** and **40b** and the risk ranking criteria in **Table 29**. The probability of cracking has been assessed in **Appendix F**.

Table 29 – Impact Potential Criteria for Aboriginal Heritage Sites

Key	Cracking Potential	Indicative Probabilities of Occurrence	Predicted 'Smooth profile' Tensile Strain (mm/m)
VL	Very Low	Very Unlikely (<5%)	<1
L	Low	Unlikely (5 - 10%)	1 - 2
M	Moderate	Possible (10 - 25%)	2 - 3
H	High	Likely (>25%)	>3
Key	**Erosion Damage Potential**	**Indicative Probabilities of Occurrence**	**Predicted Surface Gradient Change**
VL	Very Low	Very Unlikely (<5%)	<0.3%
L	Low	Unlikely (5 - 10%)	0.3-1%
M	Moderate	Possible (10 - 25%)	1-6%
H	High	Likely (>25%)	>6%

The 'cracking potential' is considered the primary damage potential indicator and the 'Erosion Damage Potential' a secondary indicator of damage (i.e. the presence of erosion and sedimentation increases at a site may result in unacceptable long-term degradation of a site).

12.7.2 Potential Impacts to Grinding Groove Sites

The predicted worst-case subsidence parameters at the Awabakal and Koompatoo ALCA Grinding Groove sites are presented in **Table 30**.

Table 30 - Predicted Worst-Case Subsidence Impacts at Grinding Groove Sites

Site No.	Site Name	LW #	Subsidence (m)	Tilt (mm/m)	Final Strain (mm/m)	Dynamic Strain^ (mm/m)	Cracking Potential
13	GG(38-4-0461)	47	-0.54	14	2.3	1.2	H
14	GG(38-4-0462)	48	-0.08	4	1.1	0.6	M
15	GG1(AR Rd)	40	-0.29	15	4.7	2.4	H
16	GG1(BC)	43	-0.78	2	2.1	1.1	H
17	GG1(CC)	42	-0.37	11	2.4	1.2	H
18	GG1(DC)	48	-1.79	15	-6.7	3.4	H
19	GG2(BC)	43	-0.82	2	1.3	0.6	M
20	GG2(DC)	42	-0.03	1	1.9	1.0	M
21	GG3(BC)	43	-1.12	8	-0.1	0.1	VL
22	GG3(DC)	43	-1.56	22	-5.8	2.9	H
23	GG4(BC)	43	-1.07	9	0.5	0.2	VL
24	GG5(BC)	43	-0.76	1	1.6	0.8	M
25	GG6(BC)	48	-0.95	7	-2.3	1.1	M
26	GGR1	43	-0.79	6	2.4	1.2	M
27	**GGSD1**	**44**	**0.00**	**0**	**0.0**	**0.0**	**VL**
28	**GGSD1 (38-4-1007)**	**44**	**0.00**	**0**	**0.0**	**0.0**	**VL**
29	**GGSD2**	**44/45**	**-0.06**	**2**	**2.3**	**0.0**	**M#**
30	GNW1(38-4-0995)	48	-1.09	8	-2.1	1.0	M

Note:
Bold - key sites requiring RMZ impact protection.
Predictions based on **Figures 36b** to **38b** and **40b**
* - negative crack widths indicate low-angle shearing.
^ - refers to transient tensile strains that may occur as subsidence develops at the site.
# - Cracking potential based on the findings presented in **Appendix F** in regards to chain pillar width and distance from rib-side.

Based on the review of chain pillar crack location data presented in **Appendix F**, it is assessed that the proposed mining layout controls instigated for two of the three key grinding groove sites (except Site29: GGDS2) are likely to provide adequate protection from cracking with 'Very Low' cracking potential at these sites assessed.

The cracking potential for Site 29 (GGSD2) was initially rated as having 'High' cracking potential or a 32% cracking probability, due to its location above a 30 m wide chain pillar between the previously proposed LWs 45 and 46 (now changed to LW44 and 45).

The cracking data above chain pillars for West Wallsend Colliery (see **Appendix F**) indicates that increasing the width of the chain pillar by 15 m (from 30 m to 45 m) will significantly reduce the potential for cracking of this site from 'High' to ' Moderate' (i.e. 18% probability of cracking).

For the GGSD2 site to have its cracking potential reduced to 'Low' (i.e. 1 - 10% probability of cracking), the pillar would need to be widened a further 15 m to give a chain pillar width of 60 m. This adjustment would locate LW44's rib-side 20.5 m from the edge of the 20 m diameter grinding groove site, and 19.5 m from the rib-side of LW45.

For 'Very Low' cracking potential, the site would need to be setback to 70 m based on an 26.5^{o} angle of draw to 20 mm subsidence and a cover depth of 140 m.

The above assessment assumes that the coordinates provided are at the centre of the groove site.

The Erosion Potential at the key sites is assessed as 'Very Low' (i.e. < 0.3% gradient increase) and ranges from 'Very Low' to 'Moderate' for the other sites. Further impact assessment details are presented in **Appendix F**.

12.7.3 Potential Impacts to Stone Arrangement and Arch Sites

The predicted worst-case subsidence parameters at the stone arrangements and arch sites above the Western and Southern Domains are presented in **Table 31**.

Table 31 - Predicted Worst-Case Subsidence Impacts at Stone Arrangement and Arch Sites

Site	Site Name	LW#	Subsidence (m)	Tilt (mm/m)	Final Strain (mm/m)	Dynamic Strain^ (mm/m)	Cracking Potential
43	**Stone Arch**	**49**	**0.00**	**0**	**0.0**	0.0	**VL**
44	**Stone Arrangement**	**41**	**0.00**	**0**	**0.0**	**0.0**	**VL**
45	Stone Arrangment2	48	-1.01	7	-1.4	0.7	L
46	Cockle Creek Shelter with Artefacts	42	-0.94	17	0.0	0.0	L-M
47	**Stone Canns**	**49**	**0.00**	**0**	**0.0**	0.0	**VL**
43	**Stone Arch**	**49**	**0.00**	**0**	**0.0**	0.0	**VL**

Note:
- predictions are based on **Figures 36b** to **38b** and **40b**.
Bold - key sites requiring RMZ impact protection.
* - negative crack widths indicate low-angle shearing.
^- refers to transient tensile strains that may occur as subsidence develops at the site.

Based on the predictions, it is assessed that the proposed mining layout controls instigated for the key stone arrangement and arch sites are likely to provide adequate protection (i.e. 'Very Low' cracking potential). The other sites however, are assessed to have a 'Low' to 'Moderate' cracking potential as indicated in the table.

The Erosion Potential at the key sites is assessed as 'Very Low' (i.e. < 0.3% gradient increase) and ranges from 'Very Low' to 'Low' for the other sites. Further impact assessment details are presented in **Appendix F**.

12.7.4 Potential Impacts to the Wet Soak Site and Spring

The predicted worst-case subsidence parameters due to LW40 at the Wet Soak and Spring are presented in **Table 32**.

Table 32 - Predicted Worst-Case Subsidence Impacts at the Wet Soak

Site	Site Name	LW#	Subsidence (m)	Tilt (mm/m)	Final Strain (mm/m)	Dynamic Strain^ (mm/m)	Cracking Potential
63	**Wet Soak (WD 5 AHIMS Registered)**	**40**	**0.00**	**0**	**0.0**	0.0	**VL**
48	Spring	45,46	0.00	0	0.0	0.0	VL

Note:
- predictions are based on **Figures 36b** to **38b** and **40b**.
Bold - key sites requiring RMZ impact protection.
* - negative strains and crack widths indicate compression and shearing displacements respectively.
^ - refers to transient tensile strains that may occur as subsidence develops at the site.

Based on the predictions, it is assessed that the proposed mining layout controls instigated for both sites are likely to provide adequate protection (i.e. 'Very Low' cracking potential).

The Erosion Potential at the sites is also assessed as 'Very Low' (i.e. < 0.3% gradient increase). Further impact assessment details are presented in **Appendix F**.

12.7.5 Potential Impacts to Scattered Artefact, Individual Features and Registered Sites (other than the Wet Soak)

The predicted worst-case subsidence parameters at the Awabakal and Koompatoo ALCA Scattered Artefact Sites, Individual Features and Registered Sites (other than The Wet Soak) are presented in **Table 33**.

Table 33 - Predicted Worst-Case Subsidence Impacts at Scattered Artefact, Scarred Trees and Other Registered Sites

Site	Site Name	LW#	Subsidence (m)	Tilt (mm/m)	Final Strain (mm/m)	Dynamic Strain (mm/m)	Cracking Potential
1	38-4-0097 AS	38	0.00	0	0.0	0.0	VL
2	38-4-0098 AS	40	-0.16	13	5.4	2.7	H
3	Artefact Scatter1	41	-0.13	6	1.6	0.8	M
4	Aubes Ridge Rd AS1	41	-0.52	18	2.9	1.4	H
5	Brunkerville Trail AS1	50	-0.05	2	0.3	0.2	VL
6	Artefact Scatter 2	41	0.00	0	0.0	0.0	VL
7	Artefact Scatter 3	41	0.00	0	0.0	0.0	VL
8	Artefact Scatter 4	47	-1.01	9	0.0	0.0	VL
9	Artefact Scatter 5	43	-1.43	4	-1.4	0.7	L
10	Artefact Scatter 6	na	0.00	0	0.0	0.0	VL
11	Artefact Scatter 7	48	-0.66	22	2.9	1.5	H
12	Artefact Scatter8	47	-0.15	6	3.0	1.5	H
31	Individual Find 1	42	-1.20	7	-2.5	1.3	M
32	IF10	43	-0.23	12	3.4	1.7	H
33	IF11	43	-0.76	23	1.6	0.8	M
34	IF2	42	-0.73	10	1.0	0.5	L
35	IF3	38	0.00	0	0.0	0.0	VL
36	IF4	38	0.00	0	0.0	0.0	VL
37	IF5	48	-0.83	10	-0.6	0.3	VL
38	IF7	50	0.00	0	0.0	0.0	VL
39	IF8	38	0.00	0	0.0	0.0	VL
40	IF8	48	-0.12	7	3.2	1.6	H
41	IF9	48	-0.07	2	2.7	1.3	H
42	Pigment in Creek?(near GG38-4-0461)	*47*	*-0.54*	*14*	*2.3*	*1.2*	*M*
49	Scarred Tree 1	41	-0.01	0	0.2	0.1	VL
50	Scarred Tree 2	41	-0.77	25	3.0	1.5	H
51	Scarred Tree 3	45	-0.34	17	4.4	2.2	H
52	Scarred Tree 4	43a	0.00	0	0.0	0.0	VL

Note:
- predictions are based on **Figures 36b** to **38b** and **40b**.
* - Negative crack widths indicate low-angle shearing

Table 33 (Cont...) - Predicted Worst-Case Subsidence Impacts at Scattered Artefact, Scarred Trees and Other Registered Sites

Site	Site Name	LW#	Subsidence (m)	Tilt (mm/m)	Final Strain (mm/m)	Dynamic Strain (mm/m)	Cracking Potential
53	Scarred Tree 5	43b	-0.04	5	4.3	2.1	H
54	Scarred Tree 6	51	0.00	0	0.0	0.0	VL
55	Scarred Tree7	43a	-0.74	2	2.3	1.1	H
56	Scarred Tree8	na	0.00	0	0.0	0.0	VL
57	Scarred Tree9	49	-0.88	5	-2.0	1.0	L
58	Scarred Tree10	49	-1.01	7	-1.4	0.7	L
59	Western Domain 1 AHIMS registered	38	-0.01	1	0.8	0.4	L
60	Western Domain 2 AHIMS registered	38	-0.06	6	2.8	1.4	H
61	Western Domain 3 AHIMS registered	38	-1.26	41	1.4	0.7	M
62	Western Domain 4 AHIMS registered	38	0.00	0	0.0	0.0	VL
64	Western Domain 6 AHIMS registered	39	-0.16	14	6.9	3.4	H
65	Western Domain 7 AHIMS registered	39	-2.30	18	-7.7	3.9	H
66	Western Domain 8 AHIMS registered	40	-1.23	39	-0.3	1.0	M
67	Western Domain 9 AHIMS registered	38	-0.62	31	6.4	3.2	H

Note:
Predictions based on **Figures 36b** to **38b** and **40b**.
* - Negative crack widths indicate low-angle shearing

It is assessed that the artefact scatter and scarred tree sites are unlikely to be affected directly by cracking and increased erosion due the predicted tilts and strains. Impact to these sites however, are more likely to be impacted by remediation works unless clearly defined in the field.

12.7.6 Impact Management Strategies

An appropriate management strategy for the archaeological sites would include:

(i) The development of a suitable monitoring and response plan based on consultation with the relevant ALCA and regulatory authorities, to ensure potential impacts to the sites are discussed prior to mining impact.

(ii) It is recommended that any damage to archaeological sites and subsequent stabilisation/erosion protection works to cracks and vegetation, should be based on consultation with the relevant ALCA's, government agencies and rehabilitation works consultants.

12.8 Gencom Communications Towers and the Proposed Power line

12.8.1 Potential Impacts to the Towers

The Gencom communications towers are located on the flatter ridge crest and upslope of the proposed LW43 in the Western Domain. Both towers are likely to be within the angle of draw.

The predicted worst-case subsidence impact parameters due to LW 43 at the Gencom Towers are presented in **Table 33**. Reference to **Section 12.2**, indicates that the potential for tensile cracks migrating up the slope to the ridge crests and in the vicinity of the towers should be considered.

Table 33 - Predicted Worst-Case Subsidence Impacts at the Gencom Towers

Site	Subsidence (m)	Tilt (mm/m)	Strain* (mm/m)	Maximum Crack Width* (mm)
CT1	0.32	5	2	20
CT2	0.05	2	0.5	2

* - Migration of cracks up-slope of the extracted area to ridge crests is possible if toe of slope subsided. (i.e. due to rigid body rotation effects).

12.8.2 Potential Impacts to the Power line

The proposed timber powerpole locations (PP05 to PP08) are shown in **Figures 1b** and **36a**. Three of the timber poles proposed above the longwall panels in the Western Domain will subject various magnitudes of subsidence, tilt and strain as shown in **Table 34**.

Table 34 - Predicted Subsidence Impacts at the Gencom Power Poles

Pole#	LW#	Subsidence (m)	Tilt* (m/m)	Strain (mm/m)	Maximum Crack Width (mm)
PP9	LW39/40 Chain Pillar	0.13	+/-6	5.5	55
PP8	LW38/39 Chain Pillar	0.04	+/-2	4	40
PP7	LW38 Rib Pillar	0.06	4	1.5	15
PP6	Outside LW38	0.00	0	0	0
PP5	Outside LW38	0.00	0	0	0

* - Positive tilts are towards the west.

12.8.3 Impact Management Strategies

Impact management strategies for sensitive structures (such as communications or transmission towers) near the crests of undermined ridges should consider the potential for cracks to develop up-slope and outside the limits of extraction as previously noted. Protection of the towers with mining encroaching inside the RMZ may include the following impact minimisation strategies:

(i) Reinforcement of footings similar to the Transgrid Towers cruciform style, which may be designed to resist ground strains and structural damage due to footing spread and/or closure. The cruciform footings also allow the towers to be re-levelled if residual tilts are excessive.

(ii) Relocation of the towers (where possible) outside the RMZ to reduce the predicted subsidence impacts to within tolerable levels;

(iii) Moving the starting position of the longwall to an appropriate RMZ limit.

Consultation between stakeholders will be necessary to select the most appropriate tower protection option.

Appropriate impact management strategies for the Gencom powerline would be:

(i) The development of a suitable monitoring and response plan based on consultation with the owners of the power line, to ensure the impacts on the towers and powerlines do not result in unsafe conditions, bush fires or loss of serviceability during and after mining.

(ii) Management of impacts would include maintaining the integrity of the power poles and preventing potential damage to conductors and surrounding bush land (e.g. in the event of a conductor break sparking a bush fire) and/or providing an alternate supply of power to the communications towers until subsidence has fully developed.

(iii) Suitable responses to subsidence impacts to the power poles and conductors would be to provide appropriate sheathing on the poles to control the tension in the conductors during/after mining impacts.

(iv) Damage from subsidence (i.e. cracking and tilting) can manifest quickly after mining (i.e. within hours). The appropriate management plan will therefore need to consider the time required to respond to an impact exceedence if it occurs.

12.9 The Privately Owned Dam (A. McArthy)

12.9.1 Potential Impacts

The predicted worst-case subsidence deformations (subsidence, tilt and horizontal strain) at the dam site in the study area are shown in **Figures 36a** to **38a** with potential crack widths presented in **Figure 40a**.

The predicted credible worst case subsidence impact parameters for the dam site is summarised in **Table 36**.

Table 36 - Predicted Subsidence Impact Parameters for the McCarthy Dam

Site	Subsidence (m)	Tilt (m/m)	Strain (mm/m)	Maximum Crack Width* (mm)
Dam Wall	1.0 - 1.6	30 - 45	1 - 5	10 - 50
Storage Area	1.4 - 2.6	4 - 35	-12 - 1	-120 - 10

* - Negative strains and crack widths indicate compression and shearing displacements respectively.

Non-engineered farm dams and water storages will be susceptible to surface cracking and tilting (i.e. storage level changes) due to mine subsidence. The tolerable tilt and strain values for the dams would depend upon the materials used, construction techniques, foundation type and likely repair costs to re-establish the dam’s function and pre-mining storage capacity.

The expected phases of tensile and compressive strain development may result in breaching of the dam walls or water losses through the floor of the dam storage area. Loss or increase of storage areas may also occur due to the predicted tilting.

Based on the predictions for maximum tensile and compressive strain, the worst-case crack width in the dam wall and storage areas is estimated to range between 10 mm and 50 mm at the surface and taper to a depth of about 5 m. This would probably result in the loss of storage through the floor of the storage area or by a breach through the dam embankment itself as previously discussed.

12.9.2 Impact Management Strategies

Appropriate impact management strategies would be:

(i) The development of a suitable monitoring and response plan based on consultation with the owners of the dams and regulatory authorities, to ensure the impacts on the dams and fences do not result in unsafe conditions or loss of access to water during and after the effects of mining.

(ii) Management of impacts would include maintaining the integrity of the dams and preventing potential downstream flooding or erosion damage and/or providing an alternate supply of water to the affected stakeholder, until the dams can be reinstated to pre-mining conditions (including re-filling the dams). Threats to public / personnel / livestock safety should also be managed by good communication and keeping downstream areas clear until mining impacts to the dam is restored or controlled.

(iii) Damage from subsidence (i.e. cracking and tilting) can manifest quickly after mining (i.e. within hours). The appropriate management plan will therefore need to consider the time required to respond to the impact in a controlled manner, when it occurs. It will also be possible to identify the dams likely to be impacted significantly, based on their location above the mine panels and predicted subsidence contours.

(iv) Suitable responses to subsidence impacts would be to either i) drain the dam storage area before subsidence occurs and repair the dam with an impermeable clay liner after mining, or ii) allow the dam wall to breach or storage areas to crack and drain during mining and repair the dam and storage area after the majority of subsidence has occurred.

The management of the mining impacts on the dam, flora and fauna and potential downstream flooding impacts, should also be addressed in the management plan and developed in consultation with the owner, MSB and relevant government agencies.

It should be noted that dams like the ones in the mining area have been undermined by longwalls elsewhere in Australia impact have been effectively managed. The dams were reinstated in a timely manner and an alternative supply of water was provided by the mine during the interim period.

12.10 Great North Walk

12.10.1 Potential Impacts

The predicted subsidence impact parameters for the Great North Walk are summarised in **Table 37**.

Table 37 - Predicted Subsidence Impact Parameters for the Great North Walk

Longwall	Cover Depth (m)	Mining Height (m)	Maximum Subsidence (m)	Maximum Tilt (m/m)	Maximum Strain (mm/m)	Maximum Crack Width* (mm)
38	150	4.5	1.80	33	-10 - 8	-100 - 80
39	140	4.35	2.35	51	-14 - 11	-140 - 110
40	140	4.2	1.82	34	-10 - 8	-100 - 80
41	160	4.05	1.67	29	-9 - 7	-90 - 70
42	170	3.9	1.49	24	-8 - 7	-80 - 70
43	*150*	*0.0*	*0.0*	***0***	***0***	***0***
47	210	3.7	1.07	14	-5 - 4	-50 - 40
48	235	3.75	1.14	14	-5 - 4	-50 - 40
49	215	3.8	0.72	6	-3 - 3	-30 - 30

* - Negative strains and crack widths indicate compression and shearing displacements respectively.
Italics - No mining beneath this section of road.

Based on the predictions for maximum tensile and compressive strain, the worst case crack width is estimated to range between 30 mm and 140 mm across the road where it passes through the tensile and compressive strain zones above each longwall panel.

It is estimated that approximately 30 to 50 m of the road above each longwall will require repairs to tensile cracking or compressive shear failures through the road after mining of each panel is completed. Some sections of road above LW39 and 40 may be impacted by local instability on cracked fill slopes.

Some erosion damage may also occur due to changes in drainage paths along the sides of the road and the installation of new table drains or possibly culverts across the road may be necessary. As the road is located along a ridge crest, no ponding impacts are expected to occur.

12.10.2 Impact Management Strategies

Appropriate impact management strategies would be:

(i) The development of a suitable monitoring and response plan, based on consultation with DCCW and regulatory authorities, to ensure the management of impacts on the walk does not result in unsafe conditions during and after the effects of mining.

(ii) Management of impacts would include visual inspections of the road on a weekly basis just prior to and after undermining of the road until 90% of subsidence has developed (usually occurs when the longwall face has retreated 1.4 x the cover depth past the road). The inspections should be completed above each panel and any impacts repaired promptly in accordance with the subsidence management plan.

(iii) Erection of signage along the affected area which cautions drivers / riders of vehicles / motorbikes / mountain bikes of the hazards associated with mine subsidence. A contact phone number should be provided if subsidence impacts are encountered.

(iv) Emergency response plans to close the road temporarily at short notice is also recommended if slope instability affects the road.

12.11 Wakefield Road

12.11.1 Potential Impacts

Wakefield Road has been undermined previously by several longwalls to the north. Some damage from subsidence development was repaired and managed successfully using the SMP between WWC and Lake Macquarie Council. The damage to the road consisted of tensile cracking with widths of up to 60 mm and compressive shearing and buckling of a similar magnitude to the tensile crack widths.

A section of Wakefield Road is located on a 5 m high earth embankment, which is beyond the predicted 20 mm subsidence limits from the corner and the ends of the first panel (LW 44). This section of Wakefield Road and embankment is unlikely to be impacted by vertical subsidence. A far-field subsidence impact assessment has been previously completed in **SEA, 2007** and also indicates negligible impact is likely.

The road crosses the full width of the last two panels, LWs 45 and 46 and is located 110 m to 355 m from their finishing points respectively. Predicted worst-case subsidence, tilt and strain along the road due to LWs 44 to 46 are presented in **Table 38**.

Table 38 - Predicted Subsidence Impact Parameters for Wakefield Road

Longwall	Cover Depth (m)	Mining Height (m)	Maximum Subsidence (m)	Maximum Tilt (m/m)	Maximum Strain (mm/m)	Maximum Crack Width* (mm)
44	*120*	*0.0*	*0.00*	*0*	*0*	*0*
45	125	4.7	1.73	24	-3 - 6	-30 - 60
46	150	4.7	1.76	34	-11 - 9	-110 - 90

* - Negative strains and crack widths indicate compression and shearing displacements respectively.
Italics - No mining beneath this section of road.

Where Wakefield Road traverses LWs 45 and 46, the maximum crack widths are estimated to range between 60 mm and 90 mm respectively over distances of about 10 m to 15 m, and will probably occur across the pavement (and through the embankment) where it crosses the tensile strain zones (see **Figures 38a** and **40a**).

Shear cracks or buckling failures are also expected to occur in the pavement and embankment sections where they cross the compressive strain zones in the central areas of the longwalls.

Timing of the crack development is expected to occur in two phases (i.e. the dynamic and final phases). The first cracking development phase will occur when the LWs 45 and 46 pass underneath the road and arcuate tensile cracks occur up to 30 m behind the longwall face.

The dynamic cracks generally in cycles equal to the 0.5 panel width (i.e. ~90 m) and may close again after full subsidence develops at the road. The second phase of cracking will occur when the full subsidence trough starts to develop between 0.7 and 1.4 times the panel width behind the retreating longwall face. Buckling and shearing of the sections of road above the middle third area of the subsidence trough (i.e. the compressive strain zone) would be expected to occur as well as the tensile zone cracks.

The cross falls or super-elevation for the road may increase and decrease slightly due to the road crossing the subsidence troughs at a high angle (refer to **Figure 36a**). The longitudinal slope of the road, however, is likely to increase and decrease by between 2^o and 3^o (i.e. about 0.3 to 0.5 m over a distance of 10 m) where the maximum tilt locations occur above the panels.

Ponding is also not expected to develop along the subsided sections of road (see **Figure 54f**).

12.11.2 Impact Management Strategies

Appropriate impact management strategies would be:

(i) The development of a suitable monitoring and response plan, based on consultation with LMCC and regulatory authorities, to ensure the management of impacts on the road does not result in unsafe conditions during and after the effects of mining.

(ii) Due to public safety concerns, 24-hour surveillance of the road (and embankment) by a LMCC roadwork crew should be present while the road is being undermined, as cracking will develop rapidly.

(iii) The stability of the embankment will also need to be monitored along the crests and toes, with cracks repaired as soon as possible to prevent excessive moisture ingress into the embankment.

(iv) It is recommended that the 24-hour surveillance of the road be provided for 3 to 4 weeks from the time where LWs 46 and 47 begin to retreat beneath the road (i.e. the cracks are not likely to develop ahead of the longwall face). Work crews may need to repair new and existing cracks several times to ensure the road remains safe and serviceable during and after mining.

Surveying of the road and embankment by WWC will also be necessary at a frequency yet to be decided but would probably range between once/week (i.e. every 7 days) for the first four weeks and then to once/month until the subsidence development period has finished.

12.12 Abandoned Bord and Pillar Workings

12.12.1 Potential Impacts

The bord and pillar workings are assumed to be still standing in the Great Northern Seam.

Predicted worst-case subsidence, tilt and strain in the vicinity of the workings, due to LWs 49 and 50 are presented in **Table 39**.

Table 39 - Predicted Subsidence Impact Parameters for Wakefield Road

Longwall	Cover Depth (m)	Mining Height (m)	Maximum Subsidence (m)	Maximum Tilt (m/m)	Maximum Strain (mm/m)	Maximum Crack Width* (mm)
49 - 50	130 - 150	3.7 - 3.8	0.35 - 1.56	23 - 28	-9 - 7	-90 - 70

* - Negative strains and crack widths indicate compression and shearing displacements respectively.

Reference to **Mark, 2007** indicates that regardless of interburden thickness and geology, existing pillar workings that are undermined, can become unstable if the subsided by high extraction workings in the seam below. Interactions have occurred between 30 m and over 170 m in the US. The instability arises when upper workings are subject to bending and shearing deformation and loss of confinement. The instability described has referred to roof falls, pillar rib spalls and floor heave; all of which, have resulted in additional surface subsidence.

The low cover depth (< 30 m) above the abandoned workings infers that the subsidence could manifest itself as either (i) pot-hole subsidence due to roof instability or (ii) trough subsidence due to pillar instability and floor heave.

The consequences of subsiding the old workings could therefore result in a further increment of subsidence from the upper seam workings.

Based on the estimated working height of 2.5 to 3 m in the GN Seam and previous cases of multi-seam mining interaction at Newstan Colliery, it is assessed that the additional subsidence due to the abovementioned mechanisms could range from 10% to 60% of the Upper Seam thickness or 0.25 m to 1.8 m.

The total subsidence above this area of the proposed longwall panels could therefore increase to a range of 0.6 m to 3.3 m, which represents (i) a 20% to 50% of the combined seam thicknesses and (ii) a 140% to 210% increase over the subsidence predicted for the longwalls only.

12.12.2 Impact Management Strategies

Appropriate impact management strategies would be:

(i) The development of a suitable monitoring and response plan, based on consultation with DECC and regulatory authorities, to ensure the management of impacts of additional trough and pothole subsidence does not result in unsafe conditions during and after the effects of mining.

(ii) The development of subsidence cracks, steps or pot hole should be infilled or repaired in accordance with the subsidence crack management plan and consultation with the MSB.

(iii) The erection of warning signs around the perimeter of the area of concern.

It is understood that the location of the abandoned workings has not been proven by field inspection and survey at this stage. It is therefore recommended that this work be completed prior to mining of LWs 49 and 50.

12.13 Property Fences

12.13.1 Potential Impacts

Wherever post and wire fences are located above the limits of mining, they are likely to be subject to potentially damaging curvatures and strains in excess of 0.2 km^{-1} and 2 mm/m respectively.

Impacts would be expected to include tilting of posts, distortion of gates and breakage of wires.

12.13.2 Impact Management Strategies

Appropriate impact management strategies would be:

(i) The development of a suitable monitoring and response plan, based on consultation with owners and regulatory authorities, to ensure the management of impacts of fence damage does not result in loss of property or injury to livestock during and after the effects of mining.

(ii) The prompt repair of surface cracking in grazing paddocks and fences and temporary transfer of livestock to undamaged paddocks prior to mining.

12.14 Far-Field Horizontal Displacements

12.14.1 Background to Prediction Model Development

Far-field displacements (FFDs) generally only have the potential to damage long, linear features such as pipelines, bridges and dam walls.

Horizontal movements due to longwall mining have been recorded at distances well outside of the angle of draw in the Newcastle, Southern and Western Coalfields (**Reid, 1998, Seedsman and Watson, 2001**). Horizontal movements recorded beyond the angle of draw are referred to as far-field horizontal displacements.

For example, at Cataract Dam in the Southern NSW Coalfield, **Reid, 1998**, reported horizontal movements of up to 25 mm when underground coal mining was about 1.5 km away. Seedsman reported movements in the Newcastle Coalfield of around 20 mm at distances of approximately 220 m, for a cover depth ranging from 70 to 100 m and a panel width of 193 m. However, the results may have been affected by GPS baseline accuracy limitations.

Based on a review of the above information, it is apparent that this phenomenon is dependent on (i) cover depth, (ii) distance from the goaf edges, (iii) the maximum subsidence over the extracted area, (iv) topographic relief and (v) the horizontal stress field characteristics.

An empirical model for predicting far-field displacement (FFDs) in the Newcastle Coalfield is presented in **Figure 55**. The model indicates that measurable FFD movements (i.e. 20 mm) generally occur in relatively flat terrain for distances up to 2 to 3 times the cover depth.

The direction of the FFD movement is generally towards the extracted area, but can vary due to the degree of regional horizontal stress adjustment around extracted area and the surface topography. The movements also appear to decrease around the corners of longwall panels.

Any sensitive surface feature, such as a bridge or culvert, within 5 times the cover depth (i.e. 250 m to 750 m from the proposed longwalls) should therefore be assessed and monitored for FFD movements during mining.

An empirical model for predicting far-field strains (FFSs) in the Newcastle Coalfield is presented in **Figure 56**. The model indicates that measureable (but diminishing) strains can also occur outside the limits of longwall extraction for distances up to 2 times the cover depth (based on the Upper 95% Confidence limit curve). It should be noted that the model was based on steel tape measurements which did not extend further than a distance equal to the 1.5 times the cover depth from the extraction limits. Any FFS predictions that are >1.5 times the cover depth from the panels in this report are therefore an extrapolation of the regression lines for the database and likely to be conservative.

A numerical modelling exercise estimating the worst-case horizontal movements along the freeway and its associated infrastructure and the utilities easement, was completed for the first five longwall panels in the Western Domain and then five panels in the Southern Domain (refer to **SEA, 2006** and **SEA, 2007**).

The Map-3D® model used was an elastic boundary-element program which was used to model the surface displacements and strains caused by horizontal stress relief into the caving zones above all of the longwall panels mined and proposed at West Wallsend Colliery. The model was calibrated to (i) measured far-field displacements at several surface features after several longwalls were extracted and (ii) horizontal stress measurements in investigation boreholes that were drilled in ridges proposed for the F3 freeway cuttings.

The Map-3D® modelling was done to assess the cumulative impacts of mining on both sides of the freeway and possible divergences between the empirical model estimates. The numerical modelling also indicated that the horizontal stress tended to decrease in the overburden between the Western and Southern Domains due to stress relief effects in the caving zones above the panels.

One of the outcomes from the study was the assessment that the prediction of cumulative affects was considered too complex for single panel empirical models used by Strata

Engineering. As the model was being used to predict the deflected profiles of the freeway pavement, pipelines and Optic Fibre Cables and relative movements between bridge abutments then this conclusion is understandable.

However, in regards to mine planning predictions, the use of superimposition techniques and the empirical models developed by DgS has resulted in good agreement between the calibrated numerical model and empirical model values. The issue of prediction uncertainty can also be handled better by the empirical models, as the numerical model was calibrated to observed displacements to provide an ‘expected’ prediction response to mining. The uncertainty in the numerical model predictions then needs to refer to empirical models.

The empirical model’s mean and U95%CL strain curves have been adopted to provide reasonably conservative outcomes when superimposition techniques are applied for multiple panel impact predictions. In some instances, the U99%CL curves have been used where additional conservatism is warranted (i.e. the Services Easement).

Discontinuous FFD response due to the presence of NW striking fault was also assessed in the Strata Engineering study with the numerical model. Very low shear strength and stiffness fault plane properties were assumed.

The modelling results indicated that shearing at bridges between the longwall blocks was ‘very unlikely’ to damage these structures or render them ‘unsafe’. A similar pair of bridges were also unaffected by the extraction of LW26 to the north of the study area.

The empirical DgS models for displacement and strain predictions have therefore been used in this study for initial assessment of the additional five longwall panels to the south west of the previous study area. It is recommended however, that the numerical model be extended to include the additional panels once the mining layout has been finalised, for subsequent stakeholder review of deflected profiles etc. This additional study will also require further field displacement information to re-calibrate the model with survey data obtained closer to the proposed mining area, as was recommended in the Strata Engineering study.

The following sections discuss predicted impact and management strategies for infrastructure and utility features located outside the predicted vertical subsidence area limits, but within the potential far-field horizontal displacement area limits.

12.14.2 Potential Impacts to RTA F3 Freeway

The nearest points of each proposed longwall panel to the freeway are the south-east corners of the panels in the Western Domain and the north-west corners of the panels in the Southern Domain (see **Figure 1b**).

The shortest distances from the freeway pavements (north and south bound lanes) to the proposed longwall panel finishing ends range from 54^{o} and 87^{o}, which indicates that the freeway is ‘likely’ to be located outside the angle of draw limits to measurable vertical subsidence. Therefore, the subsidence impact upon the freeway pavements only need to be discussed in terms of far-field horizontal displacement impacts.

The predicted final far-field displacements and strains (Upper 95%CL) at the freeway pavements (north and south bound carriageways) due to LWs 38 to 50 are summarised in **Tables 40a** and **40b**.

The displacements and strains shown are located where the longwall panel centrelines have been projected out to intersect with the freeway. Principal displacements and strain directions were assumed to act towards the centre of the panel ends. The cumulative effect of the longwalls at a given location on the freeway was subsequently assessed.

The principal movements were transformed into lateral (y-axis) and longitudinal (x-axis) components relative to the freeway axes at a given location. Positive lateral movement was assumed acting towards the Western Domain (i.e. NW to NNW) and positive longitudinal movement generally towards the NNE to NE along the freeway itself.

Table 40 - Predicted Far-Field Displacements and Strains (U95%CL Values) at the F3 Freeway Pavement (North Bound Lane)

LW #	Chain (m)	Distance to Pavement from Panel End z (m)	Cover Depth H (m)	z/H	Angle of Draw (o)	Max Panel Subs. (m)	Predicted Cumulative Horizontal Displacement u (mm) 1=principle x=longitudinal y=lateral			Predicted Horizontal Strain, e (mm/m) x=longitudinal y=lateral	
							u_1	u_x	u_y	e_x	e_y
38	136	248	130	1.9	62	2.4	9	-2	9	0.03	0.09
38(46)	500	385	130	3.0	71	2.4	7	-5	4	0.14	0.20
39(45)	825	401	135	3.0	71	2.2	7	-6	-2	0.23	0.28
40(44)	1166	325	95	3.4	74	2.4	9	6	-7	0.06	0.09
41	1451	313	100	3.1	72	1.8	4	4	-2	0.01	0.01
42	1732	386	80	4.8	78	2.4	0	0	0	0.00	0.00
43	1984	415	100	4.2	76	1.8	0	0	0	0.00	0.00
47	2240	449	120	3.7	75	1.8	1	0	1	0.00	0.00
48	2506	455	100	4.6	78	2.4	0	0	0	0.00	0.00
49	2793	331	80	4.1	76	2.2	0	0	0	0.00	0.00
50	3126	542	70	7.7	83	2.4	0	0	0	0.00	0.00

Table 40b - Predicted Far-Field Displacements and Strains (U95%CL Values) at the F3 Freeway Pavement (South Bound Lane)

LW #	Chain (m)	Distance to Pavement from Panel End z (m)	Cover Depth H (m)	z/H	Angle of Draw (o)	Max Panel Subs. (m)	Predicted Cumulative Horizontal Displacement u (mm) 1=principle x=longitudinal y=lateral			Predicted Horizontal Strain, e (mm/m) x=longitudinal y=lateral	
							u_1	u_x	u_y	e_x	e_y
38	315	270	130	2.1	64	2.4	7	-2	7	0.00	0.00
38(46)	500	430	130	3.3	73	2.4	10	-10	0	0.25	0.26
39(45)	823	445	135	3.3	73	2.2	14	-3	-14	0.31	0.36
40(44)	1165	367	95	3.9	75	2.4	18	7	-17	0.11	0.13
41	1445	350	100	3.5	74	1.8	8	7	-5	0.00	0.01
42	1727	423	80	5.3	79	2.4	0	0	0	0.00	0.00
43	1977	449	100	4.5	77	1.8	1	0	1	0.00	0.00
47	2234	483	120	4.0	76	1.8	0	0	0	0.00	0.00
48	2500	493	100	4.9	79	2.4	0	0	0	0.00	0.00
49	2794	377	80	4.7	78	2.2	0	0	0	0.00	0.00
50	3130	391	70	5.6	80	2.4	0	0	0	0.00	0.00

The far-field displacement and strain profiles for the north and south bound lanes are presented in **Figures 57a** to **58b** respectively. **Figure 59** shows the plot of predicted lateral pavement curvature, with peak curvatures ranging between -0.0002 km^{-1} and 0.0003 km^{-1} (curvature radii of 5,000 to 3,333 km) along the freeway between the two domains.

The worst-case damage to the frreway pavements after mining is assessed to be 'negligible'.

12.14.3 RTA F3 Freeway Cutting No.s 2, 3 and 4 and Fill Embankment No.s 1 to 4

The proposed distances from the cuttings to the nearest panel ends or corners range from 44^o and 88^o from the panel finishing end centrelines, which indicates that the freeway is located outside the angle of draw limits to measurable vertical subsidence.

As discussed in the previous section, the subsidence impact upon cuttings and fill embankments therefore, need only be assessed in terms of far-field horizontal displacement.

The predicted worst-case far-field displacements and strains at the crests of cuttings and toes of the embankments (including the shaft in embankment No. 3) are summarised in **Table 41**.

Table 41 - Predicted Far-Field Displacements and Strains (U95%CL) at the F3 Freeway Cuttings and Fills

Longwall	LW #	Distance to Feature from Centre of Panel End, z (m)	Cover Depth at Easement, H (m)	z/H	Angle of Draw (o)	Maximum Panel Subsidence (m)	Predicted Horizontal Displacement u1 (mm)	Predicted Horizontal Strain, e1 (mm/m)
Fill 4	38	208	130	1.60	58	2.4	15	0.16
Cut4	38	236	135	1.75	60	2.4	12	0.12
Cut4	46	178	140	1.27	52	1.8	17	0.23
Fill3	40	316	95	3.33	73	2.2	1	0.01
Fill3	45	182	145	1.26	51	2.2	22	0.29
Cut3	41	228	100	2.28	66	1.8	4	0.03
Fill2a	44	250	145	1.72	60	2.2	11	0.12
Cut2	43	300	100	3.00	72	1.8	2	0.01
Fill1	48	1200	100	12	85	2.4	0	0.00
Fill1-Palmers Rd Bridge	49	1900	80	23	88	2.4	0	0.00
Shaft-Fill3	46	133	140	0.95	44	1.8	27	0.42

The results indicate that the cuttings and embankments may be displaced towards the extracted areas between 0 and 27 mm with principal worst-case tensile strains of 0.0 to 0.42 mm/m (based on 95% Confidence Limits).

The worst-case 'spread' of the 48 m to 113 m wide fill embankments ranges between 5 and 48 mm. Cracking is unlikely to occur in the embankments due to FFDs.
The worst-case 'opening' of the 143 m to 250 m wide cuttings ranges between 0 and 42 mm between crests. Cracking is unlikely to occur in the cuttings due to FFDs.

12.14.4 Vertical Shaft in Embankment No. 3

Based on the results presented in **Table 41**, the vertical shaft in Embankment 3 could be subject to strain of 0.42 mm/m and 'open' 0.1 mm across a diameter of 0.3 m after extraction of LW46. Cracking of the shaft is considered 'very unlikely'.

12.14.5 RTA F3 Bridge Underpass in Embankment No. 3

The predicted far-field displacements and strains at the RTA Bridge between the finishing points of LWs 39 and 45 are summarised in **Table 42**.

Table 42 - Predicted Far-Field Displacements and U99%CL Strains at the RTA Bridge

Longwall	Distance to Feature from Centre of Panel End, z (m)	Cover Depth, H (m)	Normalised Location z/H	Angle of Draw (degrees)	Maximum Panel Subsidence (m)	theta (o)	Predicted Horizontal Displacement u (mm)		Predicted Horizontal Strain, e (mm/m)	
							u_x	u_y	e_x	e_y
North Bound Bridge										
39	469	130	3.61	75	2.4	33	1	1	0.01	0.00
40	210	110	1.91	62	2.4	91	0	5	0.05	0.16
45	186	140	1.33	53	2.2	244	-9	-18	0.19	0.35
46	203	135	1.50	56	1.8	309	14	-18	0.14	0.18
Cumulative displacements and U99%CL strains							**7**	**-30**	**0.39**	**0.69**
South Bound Bridge										
39	485	130	3.73	75	2.4	36	1	0	0.00	0.01
40	240	110	2.41	65	2.4	91	0	3	0.03	0.10
45	161	140	1.15	49	2.2	239	-13	-22	0.28	0.46
46	180	135	1.33	53	1.8	309	19	-24	0.20	0.24
Cumulative displacements and U99%CL strains							**7**	**-41**	**0.52**	**0.80**

Notes:
Superposition techniques applied to predictions for total strain after mining.
Theta - anti-clockwise angle subtended between x axis of F3 and principal displacement direction towards a given longwall panel (finishing point at end centreline or perpendicular to rib side).
x - movement along freeway (north-east movement is positive).
y - movement across freeway (north-west movement is positive).

The worst-case predicted longitudinal and lateral strains at the bridge location in **Table 42** are based on the U99% Confidence Interval derived from empirical data and indicate the north and south bridge abutments may be subject to cumulative lateral displacements of 3.5 and 4 mm and longitudinal displacement of 2 and 2.5 mm respectively after longwall mining is completed in both domains. The 10 m x 10 m spans are parallel to the freeway or longitudinal directions.

Worst-case horizontal shear strains (i.e. distortion) at the north and south bridges are estimated to range between 0.11 and 0.17 mm/m, which indicates 1 to 2 mm of shear displacement between the abutments.

It is understood that the predicted displacements are within the tolerable range of 5 mm for the bridge abutments. The worst-case damage to the bridges is therefore assessed to be ‘negligible’ to ‘slight’ after mining is completed.

12.14.6 RTA F3 Bridge (Palmers Road) Overpass

The predicted far-field displacements and strains at the Freeway Over Pass for Palmers Road are summarised in **Table 43**.

Table 43 - Predicted Far-Field Displacements and U99%CL Strains at the Palmers Road Overpass Bridge

Longwall	Distance to Feature from Centre of Panel End, z (m)	Cover Depth, H (m)	Normalised Location z/H	Angle of Draw (degrees)	Maximum Panel Subsidence (m)	theta (o)	Predicted Horizontal Displacement u (mm)		Predicted Horizontal Strain, e (mm/m)	
							u_x	u_y	e_x	e_y
50	1300	70	18.5		2.2	38	0	0	0.00	0.00

It is very unlikely that any movement due far-field displacement will develop at Palmers Road Bridge after mining of LWs 38 to 50.

12.14.7 Culverts Beneath the Freeway

The predicted far-field displacements and strains at the culverts beneath the freeway are summarised in **Table 44**.

Table 44 - Predicted Far-Field Displacements and Strains (U95%CL) at the F3 Freeway Culverts

Longwall	LW #	Distance to Feature from Centre of Panel End, z (m)	Cover Depth at Panel End H (m)	z/H	Angle of Draw (degrees)	Maximum Panel Subsidence (m)	Predicted Horizontal Displacement u (mm)	Predicted Horizontal Strain, e (mm/m)
Fill4	38/46	208	130	1.60	58	2.4	15	0.16
Fill3	39/45	182	145	1.26	51	2.2	22	0.29
Shaft-Fill3	45	133	140	0.95	44	1.8	27	0.42
Fill1	48	398	120	12	85	2.4	0	0.00

The rectangular section-shaped culverts were constructed in 6 m segment lengths. Based on the predicted strains in **Table 44**, it is assessed that the joints may open by up to 2.5 mm if full strain transfer occurs between the fill and embankments. Shearing of culvert joints is estimated to be <1.5 mm for this scenario.

Damage to the culverts due to the above displacements is likely to be 'negligible'.

12.14.8 Easement with Buried Jemena Gas and Caltex Liquid Petroleum Pipelines and Optus, Nextgen and Telstra Fibre Optic Cables

The easement is located between the Western and Southern domains along the F3 Freeway as shown in **Figure 1b**. Jemena high pressure gas and Caltex petroleum pipelines and fibre optic cables (FoC's) have been buried in various trenches along the easement.

Since the easement is positioned in the west side of the freeway, only subsidence from the panels in the Western Domain need be assessed for any impact on the easement. The shortest distances from the easement to the proposed longwall panels in the Western Domain range from 34^o to 87^o.

The predicted far-field displacements and strains at the utilities easement without faulting present are summarised in **Table 45a**. The predicted displacement and strain profiles along the services easement are presented in **Figures 60a** and **61a** respectively.

Table 45a - Predicted Far-Field Displacements and U95%CL Strains at the Utilities Easement without Faulting Present

LW #	Chain (m)	Distance to Feature from Centre of Panel End, z (m)	Cover Depth, H (m)	Normalised Location z/H	Angle of Draw (degrees)	Maximum Panel Subsidence (m)	Predicted Horizontal Displacement u (mm)		Predicted Horizontal Strain, e (mm/m)	
							u_x	u_y	e_x	e_y
38	149	99.5	127	0.78	38	1.9	0	5	0.10	0.31
38	369	215	127	1.69	59	1.9	7	8	0.07	0.08
39	477	102	133	0.77	37	2.2	0	6	0.12	0.37
39	759	298	133	2.24	66	2.4	5	4	0.04	0.03
40	870	74	108	0.69	34	2.4	0	7	0.16	0.47
40	1087	260	108	2.41	67	2.2	3	3	0.02	0.02
41	1190	67	105	0.64	33	1.8	0	6	0.13	0.39
41	1338	140	105	1.33	53	2.4	10	19	0.13	0.23
42	1431	73	83	0.88	41	2.4	0	5	0.11	0.32
42	1577	140	83	1.69	59	1.8	4	9	0.05	0.09
43	1669	70	103	0.68	34	1.8	0	6	0.12	0.36
43	1819	142	103	1.38	54	2.4	9	18	0.12	0.21
47	1888	135	120	1.13	48	1.8	0	3	0.05	0.15
47	**2033**	**144**	**120**	**1.20**	**50**	**1.8**	**2**	**19**	**0.09**	**0.26**
48	2136	245	100	2.45	68	2.4	0	0	0.01	0.01
48	2307	508	100	5.08	79	2.4	0	0	0.00	0.00
49	2487	856	93	9.20	84	2.4	0	0	0.00	0.00
49	2537	730	93	7.85	83	2.2	0	0	0.00	0.00
50	2634	1180	70	16.86	87	2.2	0	0	0.00	0.00

Note:
x - refers to movement along the axis of the easement.
y - refers to movement across the axis of the easement.
Bold - Telstra Tower location

The presence of NW-SE trending faults through the end of LW 38 and the easement may result in lateral shear displacements developing at the surface after mining. The magnitude of the displacements along the fault will be dependent on the strength of the fault, distance from the panel extraction limits, the depth of cover and maximum panel subsidence. The shear displacement along a weak, NW-SE trending fault through the easement at the end of LW38 has been assessed in **DgS, 2009** and indicates potential slip across the easement of up to 8 mm.

The predicted far-field displacements and strains at the utilities easement with faulting present through the end of each longwall panel are summarised in **Table 45b**.

Table 45b - Predicted Far-Field Displacements and U95%CL Strains at the Utilities Easement with Faulting Present

LW #	Chain (m)	Distance to Feature from Centre of Panel End, z (m)	Cover Depth, H (m)	Normalised Location z/H	Angle of Draw (degrees)	Maximum Panel Subsidence (m)	Predicted Horizontal Displacement u (mm)		Predicted Horizontal Strain, e (mm/m)	
							u_x	u_y	e_x	e_y
38	149	100	127	0.78	38	1.92	0	6	0.1	0.4
38	369	215	127	1.69	59	1.92	9	10	0.24	0.73
38	379	215	127	1.69	59	1.92	14	16	0.10	0.11
39	477	102	133	0.77	37	2.20	0	6	0.17	0.42
39	759	298	133	2.24	66	2.40	5	4	0.24	0.74
39	769	298	133	2.24	66	2.40	10	7	0.04	0.03
40	870	74	108	0.69	34	2.40	0	7	0.17	0.49
40	1087	260	108	2.41	67	2.20	3	3	0.31	0.94
40	1097	260	108	2.41	67	2.20	8	7	0.02	0.02
41	1190	67	105	0.64	33	1.80	0	6	0.14	0.40
41	1338	140	105	1.33	53	2.40	10	19	0.26	0.77
41	1348	140	105	1.33	53	2.40	15	28	0.13	0.23
42	1431	73	83	0.88	41	2.40	0	5	0.17	0.44
42	1577	140	83	1.69	59	1.80	4	9	0.22	0.65
42	1587	140	83	1.69	59	1.80	8	16	0.05	0.09
43	1669	70	103	0.68	34	1.80	0	6	0.14	0.40
43	1819	142	103	1.38	54	2.40	9	18	0.24	0.72
43	1829	142	103	1.38	54	2.40	14	26	0.12	0.21
47	1888	135	120	1.13	48	1.80	0	3	0.11	0.26
47	**2033**	**144**	**120**	**1.20**	**50**	**1.80**	**2**	**19**	**0.10**	**0.30**
47	2043	144	120	1.20	50	1.80	3	30	0.09	0.26
48	2136	245	100	2.45	68	2.40	0	0	0.05	0.14
48	2307	508	100	5.08	79	2.40	0	0	0.02	0.02
48	2317	508	100	5.08	79	2.40	2	2	0.00	0.00
49	2487	856	93	9.20	84	2.40	0	0	0.00	0.00
49	2537	730	93	7.85	83	2.20	0	0	0.00	0.00
49	2547	730	93	7.85	83	2.20	1	1	0.00	0.00
50	2634	1180	70	16.86	87	2.20	0	0	0.00	0.00

The far-field displacement and ground strain profiles predicted along the services easement due to weak fault displacements are presented in **Figures 60b** and **61b** respectively.

Figure 62 shows the plot of predicted lateral curvature, with peak curvatures ranging between -0.003 km^{-1} and 0.0025 km^{-1} (curvature radii of 315 to 400 km) along the easement after mining.

A study by Worley Parsons (refer **Worley Parsons, 2007**) on the predicted lateral distortion of the pipelines and FOC's due to far-field horizontal displacement (refer **SEA, 2006**) concluded that the potential for significant damage is very low. It will however be necessary to conduct 3-D surface monitoring along the pipeline within the zone of influence of mining to confirm the above modelling outcomes.

The impact of the worst-case strains of <0.7 mm/m across and < 0.25 mm/m along the Telstra Optic Fibre cable is also likely to be 'negligible' based on the significantly higher strains that occurred along and across the easement next to LW27.

12.14.9 Telstra Mobile Network Services Tower

The Telstra mobile network services tower (CT 3) is located on the crest of a hill and 146 m south of the finishing point of LW 47. Based on a cover depth at the tower of 135 m, the tower is located at a draw angle of 50^o. Whilst is assessed that the tower is likely to be outside the measureable limits of vertical subsidence (i.e. < 2 mm), it is however, within the likely distance for far-field horizontal displacement (< 3 times the cover depth) and tensile strain.

Based on the empirical database of measured horizontal displacements and strain outside the ends of longwall panels (see **Figures 55** and **56**), the worst-case maximum horizontal displacement and U95%CL strain at the centre of the Telstra tower due to the Western Domain longwalls is assessed to be <20 mm and 0.25 mm/m respectively (see **Table 38** and **Figures 60a,b** to **61a,b**).

The impact of far-field horizontal displacements and strains on the tower are likely to be 'negligible' regardless of whether a NW-SE striking fault is present or not between the tower legs.

12.14.10 Far-Field Displacement Impact Management Strategy

Based on the study, the maximum cumulative horizontal displacement and tensile strain of the freeway and utility infrastructure is likely to be < 40 mm and < 0.8 mm/m respectively after the completion of the proposed longwalls in the Western and Southern Domains (including any localised shear displacements along faults). This result is also consistent with the **SEA, 2006** numerical model assessment outcomes.

For effective management of far-field displacement impacts, it is important to have a good understanding of (i) the tolerable movements of the long or spanning structures present within the zone of influence and (ii) the accuracy of the survey or monitoring techniques that will be required to provide real-time management information.

It is understood that the monitoring of utility infrastructure signal loss or decay (i.e. optical fibre cable) is a more appropriate (and direct) indicator of system distress than standard survey measurement techniques. The accuracy of current survey techniques is discussed further in **Section 13**.

13.0 Suggested Monitoring Program

13.1 Surface Monitoring

The following subsidence and strain monitoring program is suggested for providing adequate information to monitor and implement appropriate subsidence impact management plans in the study area:

(i) Install a minimum of two cross lines in the Western Domain and one cross line in the Southern Domain to monitor subsidence and strain development across some or all of the panels (access permitting). The location of cross lines required will depend upon steep slope access and environmental or stakeholder item impact issues.

A cross line located near XL 2 and along the Great North Walk in the Western Domain (see **Figure 1a**) and one near XL 12 in the Southern Domain (see **Figure 1b**) is suggested at this stage. The lines should be installed to at least the middle of the next adjacent longwall prior to undermining.

A half-panel width crossline to the north of McArthy's Dam above LW38 is also suggested.

(ii) Install centre lines at the start and end of all panels to monitor subsidence and strain development over the ends of the panels. The longitudinal lines should extend out from the ends of the panels for a minimum distance equal to the cover depth.

A 50 m section of centreline located adjacent to the McCarthy Dam is also suggested.

(iii) A survey line along and across the banks of one or more creeks (refer to surface water consultants).

(iv) Conduct visual inspections and surveys of surface cracking (width and depth), slope instability and significant erosion during and after longwall extraction.

(v) Conduct low frequency subsidence monitoring of RTA Cuttings/ F3 Freeway pavements, the utility easement and the section of Wakefield Road along the freeway, Great North Walk, and subject to review after the completion of each longwall panel. Tilt and strain monitoring of the Gencom communications towers will also probably be required.

(vi) Conduct high frequency subsidence monitoring for the section of Wakefield Road above LWs 45 and 46 (including 24 hour surveillance by Council work crews over a 3 to 4 week period when the road is being subsided).

(vii) A minimum of 3 pegs spaced 10 m apart in a line or triangle at any isolated feature of interest (i.e. towers, archaeological sites) to measure subsidence, tilt and strain.

(viii) Establish a survey line and monitor total displacements (X, Y, Z) across the NW striking fault, which passes through the end of LW38. The line is proposed along the Great North Walk above LW38 to 39 to provide early warning for discontinuous strata movements at the RTA Underpass Bridge in Fill Embankment No. 3.

Discussions with RTA should include the range of available pre-mining and impact response actions that can be taken to ensure the serviceability of the bridges are not compromised.

(ix) Establish a 'signal strength loss' monitoring plan with owners of optic fibre cables to provide real-time response data for management of far-field displacements. A similar monitoring approach may be required for the Gencom and Telstra Towers as well.

The above monitoring program proposed is intended to allow the comparison between predicted and measured subsidence parameters for a given feature. The survey pegs should be spaced at a minimum of 10 m for reasonable tilt and curvature measurement accuracy. A minimum of two baseline surveys of subsidence and strain is recommended before mine subsidence effects occur.

Survey frequency will be dependent upon mine management requirements for subsidence development data in order to implement subsidence and mine operation management plans.

Reference to **ACARP, 2003** indicates that measurable subsidence at a given location above the longwall panel centreline is likely to commence at a distance of about 50 to 100 m ahead of the retreating longwall face; accelerate up to rates from 50 to 300 mm/day when the face is 0.2 to 1 times the cover depth past the point; and decrease to < 0.020 m/week when the face is > 1.5 times the cover depth past the point (see **Figure 63**). Further subsidence is likely to develop due to compression of chain pillars when adjacent panels are subsequently mined.

Subsidence and strains may be determined using total station techniques to determine 3-D coordinates, provided that the survey accuracy is suitable. Survey accuracy using EDM and traverse techniques from a terrestrial base line is normally expected to be +/- 2mm for level and +/- 10 to 20 mm for horizontal displacement (i.e. a strain measurement accuracy of +/- 1 to 2 mm/m over a 10 m bay-length).

Strain measurements using the steel tape method generally improve the accuracy to +/- 2mm (or 0.2 mm/m strain over 10 m) and would be the preferred method for measuring strain impacts on dams, watercourses and sensitive archaeological sites.

Alternatively Aerial Laser Scanning (ALS) techniques (or equivalent) will allow a reduction in ground monitoring to key baseline monuments and provide subsidence data to within +/- 0.15 m, based on published information. The ALS scans will also provide a more thorough picture of the subsidence development along creeks and steep surface terrain generally and without the need for intrusive surveys or monitoring pegs (which can be a hazard to the general public and livestock).

13.2 Sub-surface Monitoring

Sub-surface monitoring is recommended for providing information required for management of surface and groundwater resource impacts in areas with shallow depth of cover (< 130 m).

The following subs-surface monitoring program is suggested:

(i) Installation of a multi-anchor surface to seam level extensometer above the centre of a suitable longwall panel. The exto should be located where maximum panel subsidence is expected and cover depth is > 130 m.

(ii) Installation of multi-level vibrating wire piezometer cluster and screened-well system adjacent to the extensometer to estimate continuous and discontinuous fracture heights and complement the exto data. It may also be used to characterise groundwater quality and sub-surface aquifer system.

(iii) Monitoring and sampling of mine water makes to determine source and quantity of groundwater inflows.

Due to the uncertainties of overburden behaviour, the measurement of sub-surface fracture heights and groundwater levels above longwalls is a risky activity in regards to losing the borehole before sufficient data is obtained.

The extensometers and piezometers should therefore be installed with real-time monitoring and remote data transfer devices. It is also recommended that lockable, vandal proof covers or containers with methane venting be installed over the instruments.

An alternative to the above program would be to conduct open hole surface to seam drilling using wash bore techniques and measure depths where partial and complete water losses occur. A line of boreholes could be drilled between the rib-side and centreline to establish the overburden caving and fracture zone profiles using this technique.

Drilling through subsided overburden also represents significant risk to the drilling equipment and personal due to fractured ground and methane emissions. An activity hazard and personal safety management plan will therefore need to be developed for this type of drilling program.

14.0 Conclusions

It is concluded that the assessed range of potential subsidence and far-field displacement impacts after the mining of the proposed longwalls LW38 to 50 will be manageable for the majority of the site features, based on the analysis outcomes and discussions with the Stakeholders to-date.

Provided the proposed impact management strategies is acceptable to the relevant stakeholders, the proposed mining layout is considered reasonable overall.

If the estimated worst-case impacts cannot be reasonably managed if exceedences occur (however unlikely) through mitigation or amelioration strategies, then it will be necessary to adjust to the mining layout further to provide a more acceptable risk to the stakeholders.

The extent of mining layout adjustment will also require further discussions (and review of monitoring data) after the completion of a given panel with stakeholder and government agencies. Any subsequent changes to the mine layout should not be attempted part-way through a panel due to underground operational and safety issues.

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