Rock reinforcement and testing

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David Bigby BSc (Hons), PhD, MIMMM, CEng
Lorraine Kent BSc (Hons), MSc, PhD, FGS, MIMMM, CEng
Rock Mechanics Technology Ltd
Bretby Business Park
Ashby Road
Burton-upon-Trent
Staffordshire
DE15 OQD

Fully encapsulated tendons and rockbolts provide safe and effective reinforcement for the relatively deep coal mines in weak strata generally worked in Europe. However, once a fully grouted tendon/rockbolt has been installed, there is no way of determining whether it remains intact. Reinforced roadways also suffer the risk of deterioration over time and gate roadway stability can be jeopardised in the area of enhanced stress ahead of the retreating/advancing face. If rockbolting is going to fulfil its potential for transforming European coal mining, appropriate instrumentation and strategies need to be developed to overcome these problems. These need to include methods of detecting broken bolts and tendons in-situ; improved instrumentation to detect and indicate stability deterioration with time; and risk assessment techniques to detect and allow repair of areas vulnerable to falls of ground.

The main objective of this Project was to develop further, where necessary, and apply systems for in-situ integrity testing of reinforcement tendons and grout, improved instrumentation for Quality Management of rockbolting and practical risk assessment techniques for identifying vulnerable areas of rockbolted roadway. This will lead to improvement in mine safety through a reduction in falls of ground due to failed tendons, rock deterioration with time and stress change.

Integrity testing of tendons has been progressed significantly and a new radio frequency instrument is currently in use. Using both the ultrasonics and RF instruments intact and broken bolts have been successfully identified in deep coal mines. Instrumentation developments have produced new locally and remotely readable dual and 4 height extensometer(s) systems. Solutions for the detection of roof shear were addressed and the simplest instrument solution implemented. Risk assessment procedures have been addressed, especially with respect to mixed steel and rockbolt systems. An integrated strategy for applying in-situ integrity testing, instrumentation developments and risk assessment procedures has been proposed.

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EXECUTIVE SUMMARY

BACKGROUND
Experience has shown that fully encapsulated tendons and rockbolts with high bearing capacity provide the safest and most effective reinforcement for the relatively deep coal mines in weak strata generally worked in Europe. However, there remain several major safety concerns with such systems. Currently, once a fully grouted tendon/rockbolt has been installed, there is no way of determining whether it remains intact or breaks during its operational lifetime. Several roof falls in European coal mines have been identified as being associated with previously broken bolts and recent underground coring investigations have identified broken bolts at a number of high risk sites. Reinforced roadways also suffer the risk of deterioration over time, due to rock creep and weathering, and gate roadway stability can be jeopardised in the area of enhanced stress ahead of the retreating/advancing face. If rockbolting is going to fulfil its potential for transforming European coal mining, appropriate instrumentation and strategies need to be developed to overcome these problems. These must include methods of detecting broken bolts and tendons in-situ; improved instrumentation to detect and indicate stability deterioration with time; and risk assessment techniques to detect and allow repair of areas vulnerable to falls of ground, particularly ahead of the retreating longwall.

OBJECTIVE
The main objective of this Project was to develop further, where necessary, and apply systems for in-situ integrity testing of grouted reinforcement tendons, improved instrumentation for Quality Management of rockbolting and practical risk assessment techniques for identifying vulnerable areas of rockbolted roadway. This will lead to improvement in mine safety through a reduction in falls of ground due to failed tendons, rock deterioration with time and stress change.

FUNDING
This Project was primarily funded by the European Coal and Steel Community and forms part of research carried out by Rock Mechanics Technology Ltd under project 7220-PR/091 entitled “Improved Understanding of Reinforcement Behaviour/Testing”. A total of 60% of the cost was funded by the ECSC, the HSE contributed 9% and the remainder came from RMT and mine operators.

INTEGRITY TESTING OF TENDONS - ULTRASONIC METHOD
Although relevant research was undertaken there was no success in sourcing a better instrument for the ultrasonic testing of rockbolts. Thus work continued with the Krautkramer USD10 NF, with which there had been some success under a previous research project.

Under this project it was used with some success to survey selected rockbolts at Thoresby Colliery along the length of PG42’s Loader Gate that was being recovered for re-use following 7 years of abandonment. The survey aimed to determine the proportion of roofbolts likely to be intact such that decisions could be made as to a suitable level of additional reinforcement required to maintain roof stability for the gate’s re-use.

The survey was carried out on 50 roofbolts. 19 tests were able to detect successfully the end of the bolts and thus indicate that they were intact. Of the other tests there was no indication that the bolts were either broken or badly bent. It was likely that the
reason that bolt ends were not detected was the unfavourable chamfer angle at the back (distal) end of the bolts.

At another UK colliery, Welbeck, the instrument was used to survey bolts in a gate road where there were known to be bolts broken at approximately 0.6m into the roof. Of the bolts tested, reflections were observed from approximately 0.6m into the roof. However it was not possible to determine, using the ultrasonics method alone, whether these were related to bends or breaks, in this case the survey required the use of the RF system in order to detect the difference.

**INTEGRITY TESTING OF TENDONS – RADIO FREQUENCY METHOD**

Some success had been achieved with the RF NDT system under a previous ECSC project, (7220 AB 149) with the original instrument, (MFJ-259). However success had been very limited in Coal Measures strata and it became evident that the probable reason was that the instrument was unable to read at the lower frequency ranges which were characteristic of tendons in this strata type. The bottom range of the MFJ–259 was originally 1.4MHz. As attempts to increase the range of the MFJ-259 below 1.4MHz were only partially successful, a new instrument was specifically built to read to 0.2MHz.

The new LRF instrument appeared to work well with promising results recorded from datum bolts of different lengths up to 2.4m installed within a coal mine. However there were inconsistencies in results and it was determined that the instrument was sensitive to lead length. Modifications were made and further tests conducted, but results were still inconsistent.

At the time difficulties were being experienced with the LRF a new instrument became available that had potential advantages over the LRF, it had a frequency range as low as that of the LRF but also was able to display the SWR, record the X, R and Z components and was potentially likely to be more stable than the LRF. The new instrument proved to work well although it has only had relatively limited use due to it being obtained somewhat late in the course of the project.

At Thoresby Colliery the RF system worked well and the LRF and new instrument were both able to determine the length of certain 2.4m long original AT rock bolts. However the system did not work on all the original in-situ bolts tested. The probable cause was considered to be a change in ground conditions along the gate road with strata water affecting the results. However when bolts were specifically installed in the area of poor results positive test results were obtained. The reason is still unclear, but could be associated with changes to the conductivity of the rockbolt encapsulating resin with time in wet conditions.

RF testing was also undertaken at Welbeck Colliery in a gate road where shear movement was known to have broken bolts approximately 0.6m into the roof at specific locations. The new instrument was able successfully to determine the length of existing 2.4m long AT bolts and 1.8m long AT lifting bolts. The results correlated very well with those obtained from Thoresby. The instrument was also able to confirm the presence of a broken bolt. At a separate site at Welbeck the RF instrument has indicated that potentially broken bolts are likely to be intact. However, despite this success there have also been inconsistent results when testing SAT bolts and when the mesh is used as the ‘ground plane’ in the orthogonal test geometry. Further work needs to be undertaken to investigate these results.
Following the work under this project the RF testing has now been successfully undertaken within the following rock environments:

(i) Slate,
(ii) Salt,
(iii) Limestone,
(iv) Granite,
(v) Coal Measures limestone,
(vi) Coal Measures sandstone,
(vii) Coal Measures siltstone.

The RF system has now reached a stage where it is a practical tool to locate broken reinforcement in European mines between tendon pairs, though it suffers from the disadvantage of requiring electrical isolation between the tendon pairs tested.

INSTRUMENTATION

Instrumentation developments have led to the production of a portable readout unit for local reading of a remote reading tell tale system. This avoids the need for cabling the system to the mine surface. This system has worked well in an Indian coal mine where roof movement was monitored during pillar extraction.

A blast proof version of the dual height remote reading tell tale has been developed. The transponders are bonded up the hole and there are no bobbins external to the hole for visual reading or which can be damaged by flying rock during blasting. The tell tale is read locally by plugging it into a dedicated readout unit. Once the danger from blast damage has passed the readout cable can be left plugged into the instrument in order to read it from the roadway floor rather than having to access the roof each time. Improvements had to be made to the initial design, including spring loading of the bobbins, to prevent them from sticking at the mouth of the hole, and changing the method for bonding the unit into the hole.

A four height remote or local reading extensometer has been developed based on the dual height version.

An instrument for the detection of roof shear became a priority after a roof fall at Welbeck Colliery where shear had caused the normal dual height telltales to cease functioning and thus give misleading results once the wires were trapped. The simplest solution was to install a telltale with an additional spring that could be used to check for roof shear jamming the wires. This should be used in combination with more rigorous risk assessment, to identify high risk areas, and sentinel bolts in these areas to detect roof shear.

RISK ASSESSMENT

The purpose of a risk assessment is to identify areas of significant risk in relation to roof and rib stability so that the risk can be reduced by applying appropriate additional support or reinforcement.

A risk assessment was carried out for PG42’s Loader Gate at Thoresby Colliery where the gate road had been recovered following abandonment 7 years earlier. The previously developed risk assessment procedure was modified to take into account the additional reinforcement installed and the modified action levels used following the subsequent installation of triple height telltales.
Following a fall of ground at Rossington Colliery in a mixed support system on a faceline an analysis was undertaken of how these support systems were designed and managed in order to develop improvements to the system to reduce the ground control risk.

Examination of statistics on falls of ground indicated that steel supports with an effectively rectangular profile both with and without supplementary rockbolts should be considered relatively high risk with current MASHAM monitoring and inspection procedures. This is because they are significantly weaker than arch profile steel supports and there is an increased risk of relatively sudden collapse, possibly with limited visual warning. It was concluded that steel supported roadways with an effectively rectangular profile without rockbolts should not be used except in special circumstances where reduced risk can be demonstrated, for example narrow roadways.

Where steel and rockbolt support (mixed support) is used in an effectively rectangular profile roadway, the reinforcement system must remain effective as roof support, to maintain safe conditions. The role of steel support, if required, should be to support a limited height of the immediate roof commensurate with the steel support capacity. Where steel support is used with rockbolts in an effectively rectangular profile the reinforcement system should be designed and monitored using the same procedures as for a fully rockbolted roadway. A suitable roof movement monitoring scheme should therefore include appropriate monitoring of the roof measures supported by the steel. This could be achieved by installing triple height telltales or additional telltales to differentiate between deformation in the height of roof that the steel can support and that above, which is effectively supported only by the rockbolt system.

INTEGRATED STRATEGY
One of the objectives of this project was to integrate results of the work undertaken into an overall strategy in order to reduce the risk of falls of ground. This has been undertaken and presented as a flow chart to show how the tools can be used together.

RECOMMENDATIONS FOR FURTHER WORK
With respect to the ultrasonics testing it is recommended that further research should evaluate the potential prototype instrument that was available but was not undertaken within the timescale of this project. Also with improving technology it is possible that a more suitable instrument could still be specifically developed. The method has the disadvantage of requiring end preparation of bolt ends to obtain good signal transmission and further research should address this problem by either attempting to remove the need for end preparation or make it simpler to achieve.

With respect to the RF testing there was considerable success with instrument development and test results. However, despite this success there were also some inconsistent results. Laboratory tests have given some indication to the likely cause and further work is required to test this explanation of anomalous results and examine ways of overcoming the problem when it is encountered. Further work is recommended on investigating the use of the orthogonal method for coal mines in order to reduce the need for bolt isolation. It is recommended that the parallel RF method is now used on a much wider scale as a tool to identify broken reinforcement tendons in mines, tunnels and caverns and that it is integrated into routine, fall of ground risk assessment procedures at UK coal mines.

Instrumentation developments have led to a remote reading dual height tell tale system
with portable readout unit which has worked well for a site specific intensive scientific study in an Indian coal mine, demonstrating excellent stability and accuracy attainable. The experience suggests that it would be worthwhile investigating obtaining permission to use the system under EAWR19 for UK coalmines for site specific studies. A four wire version of this remote reading extensometer has also been developed and now needs field installation in order that its design can be fully assessed. This has the potential for use in UK coal mines to replace the 4 wire extensometers where greater accuracy is required or when access for reading is problematic. A practical combination of instrumentation (modified telltales, sentinel bolts and roof shortening meters) has been developed to identify when excessive shear is occurring in a mine roof. It is strongly recommended that this is now implemented on a much wider basis in UK coal mines.

Routine risk assessment of a district for falls of ground hazards prior to face retreat has been proven to be a practical and effective means of ensuring that potential hazards are dealt with in a timely manner. It is recommended that the procedure is applied much more widely and incorporates NDT testing of tendon integrity where appropriate.

It is recommended that all the techniques and instruments developed under the Project are applied at UK mines in an integrated manner as suggested in the flow chart shown in Figure 64.
1. INTRODUCTION

1.1 BACKGROUND TO PROJECT

Experience has shown that fully encapsulated tendons and rockbolts with high bearing capacity provide the safest and most effective reinforcement for the relatively deep coal mines in weak strata generally worked in Europe. However, there remain several major safety concerns with such systems. Currently, once a fully grouted tendon/rockbolt has been installed, there is no way of determining whether it remains intact or breaks during its operational lifetime. Several roof falls in European coal mines have been identified as being associated with previously broken bolts and recent underground coring investigations have identified broken bolts at a number of high risk sites. Reinforced roadways also suffer the risk of deterioration over time, due to rock creep and weathering, and gate roadway stability can be jeopardised in the area of enhanced stress ahead of the retreating/advancing face. If rockbolting is going to fulfil its potential for transforming European coal mining, appropriate instrumentation and strategies need to be developed to overcome these problems. These must include methods of detecting broken bolts and tendons in-situ; improved instrumentation to detect and indicate stability deterioration with time; and risk assessment techniques to detect and allow repair of areas vulnerable to falls of ground, particularly ahead of the retreating longwall.

The main objective of this Project was to develop further, where necessary, and apply systems for in-situ integrity testing of reinforcement tendons and grout, improved instrumentation for Quality Management of rockbolting and practical risk assessment techniques for identifying vulnerable areas of rockbolted roadway. This will lead to improvement in mine safety through a reduction in falls of ground due to failed tendons, rock deterioration with time and stress change.

1.2 PROJECT FUNDING

This Project was primarily funded by the European Coal and Steel Community and forms part of research carried out by Rock Mechanics Technology Ltd under the following project:

7220-PR/091 entitled “Improved Understanding of Reinforcement Behaviour/Testing”.

It was a targeted collaborative project with the following partners:

(a) UK Coal Mining UK Ltd., (UK)
(b) Houilleres du Bassin de Lorraine, (France)
(c) Deutsche Montan Technologie GmbH, (Germany) and
(d) Deutsche Steinkohle AG, (Germany).

The total value of the project was 2.9 million Euros. The gross cost of the RMT aspect of the ECSC project was 851,000 Euros over 3 years. This was 60% funded by the ECSC and 9% by the HSE with the remainder funded by RMT and mine operators.

The ECSC Project commenced on 1st October 2000 and ran for a period of 3 years. The HSE contract ran from 1st April 2001, with a duration of 36 months until 31st March 2004.
This report covers the aspects of the ECSC/HSE project undertaken by RMT. The final report of the full ECSC report will be published by the EU Commission.

1.3 RESEARCH PROGRAMME UNDERTAKEN

The main aims of this Project were to identify and improve where necessary the most effective NDT systems for detecting failed tendons in roadways supported by rock reinforcement; to analyse surveys using these methods in order to develop methods of reducing the falls-of-ground risk due to broken tendons; to develop a Quality Management approach to Risk Assessment, based on the NDT results; to seek practical methods of measuring bed-shear and to investigate improvements to existing instrumentation such as tell-tales and extensometers.

The main objectives were:

(a) **NDT systems feasibility studies**: Undertake initial feasibility studies of all targeted NDT systems based on information derived under ECSC Project AB149, (ECSC 2000[1]), and determine the most appropriate systems for further development and testing through to application stage;

(b) **NDT systems development and testing**: Undertake necessary further development and testing on chosen targeted systems including certification and approval if required.

(c) **NDT systems application**: Apply appropriate NDT systems to determine prevalence of failed tendons in roadways supported by rock reinforcement.

(d) **NDT systems data analysis**: Analyse results of surveys in terms of the circumstances in which broken tendons have been detected and devise suitable methodologies for reducing the risks of falls of ground due to broken tendons.

(e) **Rockbolting instrumentation: improve and test**: Improve rockbolting instrumentation systems (extensometers, strain gauged bolts and telltale) and test improvements underground.

(f) **Rockbolting instrumentation, shear measurement development**: Identify appropriate systems for bed shear measurement and develop into prototype instrument. Test shear measurement system and develop into practical instrument.

(g) **Risk assessment development**: Develop general risk assessment approach based upon data derived from the use of the NDT systems and previous site specific risk assessments and failure mechanisms identified under the ECSC project AB149, (ECSC 2000[1]).

(h) **Risk assessment application and testing**: Apply and test general risk assessment approach.

(i) **Result integration**: Integrate results of NDT, instrumentation development and risk assessment for overall strategy.
(j) **Disseminate results**

These objectives were met, although, as might be expected, the detailed work programme varied to some extent from that originally envisaged in the detailed objectives. This was due to a combination of factors including changes in priorities, difficulty in obtaining co-sponsorship for some work items, and unexpected results that changed the focus of some investigations. Consequently some important investigations, not originally identified, were undertaken.

The main items of research carried out can be sub-divided as follows:

(a) Continuation of work on the integrity testing methods for rockbolts and cablebolts, namely the Ultrasonics and Radio Frequency methods. This covers objectives (a) to (d). The Ultrasonics work is described in Section 2 and the Radio Frequency in Section 3.

   With respect to the ultrasonic methodology there was no success in sourcing a better instrument than the one currently used, thus work continued with the Krautkrammer USD10 NF. However with the RF method a new instrument was developed and tested which led to a further instrument which had advantages over the previous one. The ultrasonics and RF instruments were applied at various sites and in combination they were able to identify both intact and broken bolts.

(b) Developments and improvements in ground movement instrumentation are detailed in Section 4 (objectives (e) and (f)).

   This work includes developments for the local reading of a remote reading tell tale system, a remote reading blast proof dual height extensometer and a 4 height remote or local reading extensometer. Instrumentation solutions for the detection of roof shear were also addressed and the simplest solution of installing a telltale with an additional spring that could be used to check for roof shear jamming the wires was implemented.

(c) Work with respect to ground control risk assessment is described in Section 5, (objectives (g) and (h)).

   The previously developed risk assessment procedure was modified at one particular site in order to take into account special support circumstances. Risks of falls of ground related to mixed steel and rockbolt systems was also reviewed in detail and guidelines drawn up for risk reduction.

(d) The project conclusions are summarised and integrated in Section 6, (project objective (i)).

   This involves the integration of the work carried out incorporating the integrity testing, instrumentation developments and risk assessment procedures.

In order to meet the project objectives the work was undertaken at a variety of mine sites. The two main sites are described below.
1.4 INTRODUCTION TO MAIN TEST SITES

1.4.1 Thoresby

Thoresby Colliery, owned by UK Coal Ltd., and located in the Nottinghamshire Coal Field was a primary test site for both the NDT and novel risk assessment work. This was due to the unusual nature of PG42’s Loader Gate.

PG42’s Loader Gate was an old roadway that was recovered in order to re-use it as a gate road for a longwall retreat face. It was originally developed as a longwall gate road in 1993 by an adjacent mine, Ollerton Colliery, then part of British Coal. Figure 1 shows the location of the panel with respect to the surrounding workings, and Figure 2 a more detailed plan of the panel. Roadway roof support during development comprised seven 2.4m long resin encapsulated AT rockbolts installed through steel w-straps and steel mesh at 1m strap spacings. The initial 200m of drivage also included steel girder supports.

Following development, the longwall went into production in February 1994. However, the face had only been in production a few weeks when Ollerton Colliery was closed and the face salvaged leaving most of the panel unworked. In 2001 Thoresby accessed the old Ollerton workings with a view to exploiting the remaining reserves in the area adjacent to 42’s Loader Gate.

Between 1994 and 2001 the roadway was subject to partial flooding by strata water originating from a sandstone channel that overlies the outbye part of the panel. Early in 2002 a programme of de-gassing, re-ventilating and pumping was established in order to re-access the roadway. At the time of undertaking the NDT work, the roadway was open for inspection and recovery operations were complete up to the 865m mark. Figure 3 shows the geological variation along the gate.

Initial inspections revealed that roadway conditions up to 860m were good with the roof and rib sides indicating stable conditions and little sign of weathering or deterioration of the strata. Many of the original roof tell tale monitoring indicators, installed at 20m intervals along the roadway, were still operational and these generally indicated low levels of roof movement. However, due to the action of the highly corrosive strata water present in the roadway, significant corrosion was evident on the steel mesh, steel w-straps and the exposed ends of the rockbolts. In places the steel mesh had completely corroded away.

The condition of the roofbolts above the roof line was unknown but, with the presence of a sandstone channel in the roof, there was considered to be a potential for broken roofbolts due to the combined action of lateral roof shear and corrosion, particularly where the sandstone channel thinned. It was therefore determined that a suitable means of assessing the integrity of the rockbolt support was needed.

Two methods were considered, rockbolt recovery and non-destructive testing (NDT). An inspection strategy was devised combining the use of available NDT techniques and rockbolt recovery methods. The object of the NDT testing was to determine whether rockbolts were fully intact, completely broken or contained fracture defects along their length, Section 2.3 and Section 3.5.3.

A general risk assessment for PG42’s Loader Gate stability for face retreat was undertaken as part of the project. Due to the unusual nature of the additional support...
and modified monitoring used this necessitated appropriate modifications to the previously devised risk assessment methodologies used, (Section 5.2).

1.4.2 Welbeck

**PG312’s District**

PG312’s district is in the Parkgate Seam at a depth of 850m and located in the East of the Welbeck reserves. The face is 255m wide and is to have a total retreat of 2260m. Both of the gate roads, the faceline and associated junctions were developed on rockbolted support.

PG312’s district gate roads were developed close to the line of the maximum horizontal stress. The district was developed between Welbeck’s 310’s previously worked retreat face and the previously extracted PG192’s district taken by Thoresby Colliery shown in Figure 4. Consequently the stress regime was modified and this led to floor heave, rib movement and local roof bulking in both gates. The vertical stress regime was affected by the workings in the Top Hard Seam approximately 192m above, also shown in Figure 4. The face was due to retreat under a series of Top Hard pillar remnants, and in these areas increased floor lift and rib movement were expected.

A roof core from the 154RM in the Loader Gate which was considered representative of the area showed that the roof of the Parkgate Seam above the roadway horizon consisted of a grey silty mudstone overlain by siltstone, Figure 5.

**Fall of Ground, PG 312’s Loader Gate**

A fall of ground occurred in the Loader Gate in July 2003 between the 1675 and 1690RM. The face was at the 1694RM. It had previously worked through a poor area under a Top Hard pillar and had passed back under Top Hard waste with a subsequent improvement in conditions.

The fall extended some 16m to a maximum height of 1.3m. It commenced at almost full roadway width at the 1675RM and reduced in width to the left hand side inbye to the 1690RM. Broken roof bolts were observed along with the formation of shear and stress failure planes.

The tell tale installed in the fall area had been read on the morning of the fall and had recorded 12mm of movement in the bolted height and 15mm above it. Following the fall defective telltales were identified locally within the gate due to shear movement, in the immediate roof at approximately 0.6m trapping the wires.

Following this fall of ground, NDT measurements were undertaken within PG312’s Loader Gate in order to assess the condition of selected roofbolts and determine if the NDT ultrasonic and RF methods could identify further broken bolts, Section 2.6 and Section 3.5.4.

Also work was continued on the development of a suitable instrument and/or strategy to detect roof shear and avoid inaccurate telltale readings under shear conditions, (Section 4.5).

**NDT Test Sites, PG312’s Loader Gate**

At the 875RM the standard bolting pattern of 7 x 2.4m AT bolts at 1.0m centres had been supplemented by 2.4m long SAT spotbolts which were being installed within 150m of the face. These were biased to the left and right hand side of the roof due
access logistics associated with the conveyor belt. The roof was dry and reasonably flat with some guttering on the left hand side associated with the face abutment stresses. The nearest tell tale was No. 46 at the 878RM which was indicating 12mm on the A and 15mm on the B.

Between the 523-526RM the standard bolting pattern of 7 x 2.4m AT bolts at 1.0m centres was supplemented by AT spotbolts which had been installed in January 2003 in response to tell tale movement. Wood legs had also been set. 1.8m long AT rockbolts had been installed in the W strap between the No.5 and No.6 bolts for slinging the monorail. The roof had bulked in the centre of roadway. The nearest tell tale was No. 28 at the 524RM and this had been replaced having measured 50mm on the A and 5mm on the B bobbins. The roof was dry in this area.

**Deep Soft Seam**

The Deep Soft Seam at Welbeck Colliery lies approximately 40-50m above the Parkgate Seam. DS217’s was the last district to be worked in the Deep Soft Seam finishing in August 1995 prior to subsequent workings concentrating in the Parkgate Seam below. Re-entry into the Deep Soft Seam occurred early in 2003 with a view to taking the planned adjacent/subsequent panel DS219’s, Figure 6.

The immediate roof geology for the Deep Soft Seam at Welbeck comprises up to a metre of mudstone overlain by variable siltstones and sandy siltstones. In 217’s developments the immediate mudstone was either thin or absent bringing the stronger sandy siltstones down to close to the top of the seam, Figure 7. 217’s Return Gate was developed on primary rockbolt support. Excessive roof lowering in the initial section of 217’s Return Gate Junction in response to the change in geology lead to the requirement for cablebolting.

**NDT Test Site, DS South Return, 217’s Return Gate Junction**

The section of the Deep Soft South Return in the vicinity of 217’s district was also developed on primary rockbolt reinforcement. At the NDT test site at 217’s Return Gate Junction, an initial bolting pattern of 7 x 2.4m AT bolts at 0.5m centres was employed. In response to extensometer and tell tale roof monitoring this was supplemented by cablebolts and then steel support.

The telltale within the junction at the 1076RM had indicated 112mm within the bolted height and 13mm above it prior to the temporary abandonment of the area. This telltale was reset in July 2003 following re-entry and had recorded negligible movement since that time. The areas that were wet, and which were therefore identified as being at a higher risk of defective roof bolts, were re-bolted with SAT bolts. NDT measurements were undertaken at the site in order to identify whether the original roof bolts were intact, Section 2.7.2 and Section 3.5.4.
2. NON-DESTRUCTIVE TESTING OF ROCKBOLTS, NDT - ULTRASONICS

2.1 INTRODUCTION

The ultrasonic system uses a modified instrument originally designed for crack and flaw detection in the engineering industry. During testing the instrument probes are held against the proximal end of the bolts within which piezoelectric crystals generate sonic compression (longitudinal) waves, which propagate into the rockbolts. Reflected sonic waves are received by the transducer and analysed by the instrument electronics and an echo trace is displayed on the instrument screen.

The instrument continuously transmits and receives sonic pulses and the image is continuously updated whilst the probe is in contact with the bolt, effectively giving a 'live' trace. This 'live' trace enables the user to adjust the probe contact and numerous parameters within the instrument in order to search for the best reflected image.

2.2 ALTERNATIVE INSTRUMENTS

At the outset of the Project it had been hoped that a new portable instrument, the Wavemaker 16, could be a significant improvement on the instrumentation currently available. However, initial tests with a prototype of this instrument were somewhat disappointing. No further progress has been made with this instrument.

Another prototype instrument, developed by NDT Solutions, was planned to be evaluated. This would be designed as a flexible, general purpose instrument with built in pulser, arbitrary function generator, digitiser and disc storage. It was also to be capable of use with both high and low frequency probes. It was anticipated that it would be possible to undertake trials of the prototype NDT Solutions instrument at Middleton mine, however this did not prove possible within the timescale of the project.

During the ECSC project AB149 (ECSC 2000[1]), a new generation instrument was developed by Krautkramer which was modified for low frequency testing and its capability for inspecting rockbolts was assessed. This was the USM20L-RMT. Krautkramer have since produced an intrinsically safe version of this instrument, the USM23EX. However following the disappointing performance of the USM20L-RMT when compared to the USD10NF, as currently used by RMT, this instrument has not been explored further.

For the above reasons, measurements with the Krautkramer USD10NF have continued and the majority of the development effort on NDT methods for rockbolt integrity testing has concentrated on the RF method which is covered in Section 3.

2.3 ULTRASONICS TESTING AT THORESBY COLLIERY

2.3.1 Site Calibration

Ultrasonic testing was used in combination with the RF method to determine rockbolt integrity in 42's Loader Gate, a gate road being recovered following 7 years of abandonment, as described in Section 1.4.

Calibration tests were carried out at the start of the Thoresby project. Three datum bolts of different lengths, 0.6m, 1.2m and 2.4m respectively, were installed into the roof in the roadway under investigation. The embedded ends of the datum bolts were cut
perpendicular to the axis of the bolt before installation in order to maximise signal response during testing, particularly for the High Frequency modes.

The calibration tests indicated that the velocities of the successful Low Frequency (LF) modes were significantly slower than the velocities of the successful High Frequency (HF) modes in bolts embedded in the Thoresby strata, Figure 8. The LF probe indicated the 2.4m bolt end to be at 2975mm for the instrument wave velocity setting at 5920m/sec.

During underground use, the instrument test parameters were set for the HF range. Therefore, as the HF and LF mode velocities are different, traces recorded using the LF probes indicate incorrect distances on the x-axis with a 2.4m bolt shown as approximately 3m long.

As two frequency ranges were used for the testing at Thoresby it would be appropriate to set the instrument display to read in the time domain, that is with the x-axis reading in microseconds. However, it is more meaningful to the operator during testing if the x-axis display is in millimetres.

Figure 9 shows a correlation graph of measured distance versus actual distance based on tests carried out on the datum bolts for the frequency ranges assessed at Thoresby.

The datum bolt tests also indicated that some of the probe transducers were more successful than others in obtaining a clear response from the distal end of the bolts, particularly the 2.4m datum bolt. From this it was possible to determine which HF and LF probes were likely to be successful for measuring the in-situ rockbolts at this site.

2.3.2 Ultrasonic Measurements

Good results were obtained from all three datum bolts using the HF probes. Of the five LF probes used three gave a clear response from the end of the 2.4m bolt and all five successfully detected the end of the shorter datum bolts. The calibration data shown in Figure 8 was derived from these tests.

Following calibration, approximately 50 in-situ rockbolts were prepared and tested between roadway distance marks 215m and 865m. Bolts were tested at approximately 15m intervals along the roadway and tests were carried out working progressively inbye.

Because, when using different ultrasonic probes and the RF system, there was potentially a series of tests required on each roofbolt, a testing strategy was developed to minimise the interrogation time on each bolt. Priority was given to the ultrasonics testing with the High Frequency probes as these had the potential to give the best possible bolt end response, as indicated by Figure 8. They would either be clearly responsive or not. The HF tests would be quick to carry out and would either confirm intact bolt length and/or fracture location or be discounted. The following strategy was adopted.

For each bolt:
1. High Frequency Test First: Far field examination for bolt end or fracture defect followed by near field examination for fractures
Outcomes:
(a) If bolt end response obtained and no earlier fracture defect detected bolt confirmed as intact – no further testing – move to next bolt.
(b) If no bolt end response obtained but earlier defect detected - defective bolt confirmed – test for other defective bolts in same area – recommend appropriate remedial support.
(c) If no response from far field or near field – progress to LF tests on same bolt.

2. Low Frequency Tests. Test a range of Low Frequency modes - far field examination for bolt end reflection or fracture defect followed by near field examination for fracture defects.
Outcomes:
(a) If bolt end response obtained and no earlier fracture defect detected bolt confirmed as intact – no further testing - move to next bolt.
(b) If no bolt end response obtained but earlier defect detected - defective bolt confirmed – test for other defective bolts in same area – recommend appropriate remedial support.
(c) If no response from far field or near field – progress to alternative test method to confirm condition of bolt (RF system or Bolt Recovery).

It was also necessary to devise a methodology for assigning a level of confidence to each test. For tests carried out on old in-situ bolts no clear response was obtained from the bolt end or fracture defects when using the High Frequency probes, most probably due to the unfavourable chamfer angle at the distal end of the bolt. However, considerable success was achieved with the Low Frequency probes and the following results therefore relate to tests carried out with the Low Frequency probes. Table 1 illustrates the classification system employed.

Table 1. Classification of ultrasonic test results for PG42’s Loader Gate, Thoresby Colliery

<table>
<thead>
<tr>
<th>Description of trace [Feature = bolt end or defect]</th>
<th>Classification and level of confidence</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate clear indication of feature by stable distinct peak with amplitude 50% greater than background</td>
<td>Class I &gt;90%</td>
<td>Confirmation of condition of rockbolt – no further testing required</td>
</tr>
<tr>
<td>Feature identifiable by distinct peak with amplitude of peak greater than background</td>
<td>Class II 60-90%</td>
<td>Confirmation of condition of rockbolt – further testing may improve understanding of bolt condition but not necessarily required</td>
</tr>
<tr>
<td>Some indication of feature, peak stable but not distinct from background, or feature indicated by more than 1 probe</td>
<td>Class III 30-60%</td>
<td>Indication of bolt end or defect from test but further examination recommended to confirm result</td>
</tr>
<tr>
<td>No clear response, peaks may coincide with expected bolt end location but none particularly clear or distinct.</td>
<td>Class IV &lt;30%</td>
<td>Bolt condition not confirmed – recommend alternative method of investigation.</td>
</tr>
</tbody>
</table>

Figure 10 shows examples of some of the successful tests obtained using the Low Frequency probes where a reflection from the bolt end was confirmed.
Table 2 summarises the results of the ultrasonic tests on the in-situ roofbolts at Thoresby using the classifications detailed in Table 1. None of the bolts gave any indication of being shorter than 2.4m.

Table 2. Ultrasonic test results from the in-situ bolts tested in PG42’s Loader Gate, Thoresby Colliery

<table>
<thead>
<tr>
<th>Classification</th>
<th>Number of Bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3</td>
</tr>
<tr>
<td>II</td>
<td>16</td>
</tr>
<tr>
<td>III</td>
<td>21</td>
</tr>
<tr>
<td>IV</td>
<td>10</td>
</tr>
</tbody>
</table>

In general, bolts classified as Class III were selected for RF testing whilst bolts identified as Class IV were selected for bolt recovery. However, other factors were also considered when selecting bolts for further investigation including the relative condition of the roadway in the area of each test bolt and the ease of access for bolt recovery. The RF test results are reported in section 3.5.3.

Following the NDT testing 7 bolts were recovered by mine personnel. None of the bolts recovered were broken or had fracture defects along the length of the bolt.

Many of the bolts were still bonded throughout their length to the rock core into which they were installed and when the rock core was removed the sections of rockbolt that were above the roofline and encapsulated in resin were in good condition with little evidence of corrosion. However, some of these bolts displayed bends along their length.

Two of the bolts recovered were found to be not fully encapsulated in resin indicating poor installation quality. One of these was severely corroded over part of the un-encapsulated length significantly compromising their effectiveness as roof support.

Based on the NDT work and the bolt recovery, the Colliery made the decision to place additional bolts in the roof of the roadway throughout.

2.4 ULTRASONIC TESTING GROUND ANCHORS IN A ROAD CUTTING

The ultrasonic instrument and method were assessed for application in seeking defective coupled ground anchors, 36mm diameter, in a UK trunk road cutting in weathered slate, where cases of broken bolts had previously occurred. The work was complicated by the fact that no calibration anchors were available and there was some uncertainty over the original length of at least some of the anchors tested.

2.4.1 Bolt-end preparation for embankment

In order to produce a plane contact surface for the ultrasonic probes, it was necessary to prepare the bolt-ends on site. A 250 mm abrasive cut-off wheel fitted to a 2kW angle-grinder was used to cut about 10 mm off the bolt. The angle-grinder was pivoted from a purpose-built ‘L’ shaped bracket which was securely attached to the hexagonal nut. This allowed a single-pass plane cut to be made.
2.4.2 Initial tests at embankment

The first tested bolt, (RA2 No. 124), was tested with both High and Low Frequency probes, to determine the optimum settings (e.g. frequency filter, amplification level, rectification mode, pulse power) for each probe.

The 15mm diameter 4MHz probe produced a peak in reflected energy at about 100mm, followed by a decay with some additional structure out to 600 – 800mm, Figure 11(a).

Low frequency probe operating at 100kHz and 70kHz revealed some additional features in the 900 – 1000 mm region, Figure 12 (a).

These results indicated that both short and long range features could be detected using the equipment.

Two further bolts were also prepared, (RA2 No. 82 and RA3 No.114) with the angle-grinder equipment, but many others were found to be recessed into the rock, leaving insufficient room to rotate the cutting head through the bolt diameter. It was concluded that an alternative preparation method was necessary for these situations.

2.4.3 Second site visit to road cutting

A small but powerful belt-sander, fitted with the coarsest grade belt, was identified as a possible alternative means of preparing the bolt-ends in confined spaces. It was found that the rapid rate of material removal and the large flat backing-plate produced a reasonably plane surface even when hand-held. The use of a simple jig to steady the tool would no doubt produce an even better contact surface.

Three bolts were prepared using this tool: RA2 No. 123, RA2 No 82 (previously cut with the angle grinder) and the remaining exposed end of a failed bolt RA2 No.126. Ultrasonic tests using the 25mm diameter 4MHz and 30mm diameter 100kHz probes were made on all five prepared bolts:

(i) RA2 No. 124 – prepared on the first visit
(ii) RA2 No. 82 – attempted first visit
(iii) RA2 No.123 – prepared second visit
(iv) RA2 No. 126 - prepared second visit
(v) RA3 No.114 – attempted first visit

The instrument settings were kept constant for each probe on each bolt and Figures 11(b)-(f) and 12(b)-12(f) show the results from the 5 bolts with the 4MHz and 100kHz probes respectively.

The high-frequency reflections were of the same form for all five bolts, with a rapid rise at 200mm followed by a decay to noise levels by 600 – 800mm. Significantly, this form was the same for the failed bolt, showing that it was not due to reflections from the nut and faceplate.

The low-frequency reflections again showed additional structure around the 1000 mm mark, with some strong reflections which varied in relative height and position between bolts. RA2 No. 124 and RA3 No. 114 in particular had strong reflections at 980 – 990 mm.
2.4.4 Interpretation of ultrasonic data from the road cutting

The high-frequency reflections starting at about 200mm correspond to the impedance-change at the air/rock interface. Complete (full cross-section air-gap) breaks would normally produce narrow, strong reflections sometimes with secondary echoes at twice the distance. **Figure 13** shows the reflection obtained from a 15mm deep slot at 300mm in a free standing length of the 36mm diameter anchor. The end-reflection at 750 mm is also visible.

Although the loss of signal into the surrounding material considerably reduced the range compared with the free-standing situation, the absence of similar reflections suggested that no such breaks were present close to the outer ends of the anchors tested.

The severity and distance at which partial breaks or reduced diameters could be detected would need to be established by comparison with known conditions.

The Low Frequency probes have higher power and longer wavelengths for investigating far-field reflections, but with a practical limit of 2 – 3 metres for encapsulated tendons. The strong reflections in some anchors at around 1000 mm could be significant.

It was concluded that the ultrasonic method would be viable to detect near end features and bolt failures and that it could be used in conjunction with the RF method for a full survey at the site.

2.5 ULTRASONIC MEASUREMENTS AT AN EVAPORITE MINE

A series of measurements were carried out underground at an evaporite mine using the ultrasonics and RF method to test whether each system would be successful in determining the intact length of in-situ rockbolts in salt.

For these tests, 6 x 20mm diameter resin encapsulated bolts were specially installed into a dry salt ribside. These comprised three bolt pairs of lengths, 1.4m, 1.0m and 0.6m as shown in **Figure 14**.

A variety of probes was tried. It was found that, by using a high frequency probe, it was possible to determine the length of each test bolt to within 6% of its actual length. For these initial investigative tests the instrument calibration used was only approximate for the particular bolt/resin/rock configuration and it is likely that, by undertaking precise calibration checks on the system the reading accuracy could be improved to +/- 10mm of the actual bolt length.

**Figures 15 and 16** show the instrument traces produced during the tests.

**Figure 15(a)** shows traces for tests carried out on the two 1.4m bolts and these indicate the lengths to be 1480mm in each case.

**Figure 15(b)** shows traces for the tests carried out on the two 1m bolts and these indicate the lengths to be 1040mm in each case. The reflected signal for these two tests was much stronger than for the 1.4m bolts. This is partly the effect of signal attenuation with bolt length and partly a result of the difference in the end condition of
the bolts. The condition of the end of the bolt is critical in order to achieve good signal transmission into the bolt. The two traces shown in Figure 15(b) also vary slightly both in the degree of background noise and the amplitude of the reflected signal with the top trace indicating less noise and a stronger reflected signal from the far end of the bolt. In this case the difference is related mainly to the variation in the quality of the bolt end preparation.

Figure 16 shows traces for the tests carried out on the two 0.6m bolts. The top two traces are the same test but with the distance indicator gate moved to pick out different reflected signals. The trace on the right highlights a second reflection from the end of the bolt, indicated as being twice the length of the bolt. In this case the ultrasonic wave has travelled from the probe to the end of the bolt and back, then is reflected from the near end of the bolt back to the far end and back again for a second time. This indicates good signal transmission with relatively low signal attenuation at this frequency. Tests carried out with the low frequency probes were less satisfactory. This was partly due to the bolt diameter being much smaller than the probe diameter thereby giving poor signal transmission into the bolt and partly due to the higher signal attenuation normally experienced at the lower frequencies.

In summary, the ultrasonic system was successful in determining intact bolt length on the in-situ test bolts but bolt end preparation is critical to the successful application of the system. The mine have indicated that the bolt end preparation aspect of this method would be a distinct disadvantage for application of this system for routine testing. It was therefore decided to concentrate on the RF system for further tests and development for this application.

2.6 ULTRASONIC TESTING OF LARGE DIAMETER BOLTS AT A POWER STATION

The same equipment was successfully used to determine the in-situ length of 24 x 86mm diameter machine anchorage bolts embedded in concrete at a power station. Tests were undertaken with 4 and 2 MHz probes in the high frequency range and 100kHz, 70kHz, 50kHz and 24kHz probes in the low frequency range. The high frequency probes and, in particular the 2MHz probe, gave excellent results indicating that all the bolts were straight and confirmed their expected lengths to +/- 0.05m (16 x 2.76m, 16 x 2.47m). Example traces are shown in Figure 17. In the low frequency range, the 24kHz probe gave the clearest results and indicated some of the structure within the mounting block. However the low frequency probes did not generally give good end reflections.

2.7 ULTRASONICS TESTING AT WELBECK COLLIERY

2.7.1 PG312’s Loader Gate

Tests were conducted at the 523/524RM, in an area where broken bolts were suspected, on the following pre-prepared roof bolts:

(i) A potentially broken 2.4m long AT bolt in the poor roof zone,
(ii) A 1.8m long AT lifting bolt,
(iii) A 2.4m long potentially intact AT spotbolt.
**High Frequency Tests, 2, 4 Lg., 4 Sm. MHz probes**
Numerous High Frequency tests were undertaken with the range of High Frequency probes on the 1.8m bolt, the potentially intact 2.4m spotbolt and the potentially broken 2.4m replacement bolt adjacent a poorly installed bolt in strap. Following the initial results from these three bolts additional testing was undertaken on a different potentially broken bolt at 527RM also within a bellied zone of roof.

**Figure 18** shows the results obtained with the 4 MHz large headed probe which are considered representative of the results from the other two HF probes, for the potentially broken 2.4m AT bolts at the 524RM and 527RM and the potentially intact 2.4m spotbolt. In all cases a reflection indicative of the bolt end was not seen, and there were no convincing peaks along the length of the bolt that could be associated with a break or severe bend.

**Low Frequency Tests, 70kHz, 850kHz, Pundit (150kHz)**
All three probes indicated potential peaks in the area of 0.5-0.6m, but the confidence level was medium to low. **Figure 19** indicates the results from the 70kHz probe. All three bolts including the intact bolt indicated peaks that could be indicative of bends and/or breaks at approximately 0.55m. A trace of the probe characteristic when in fresh air confirms that this not likely to be a feature of the probe.

**Figure 20** shows the results from the 850kHz probe which has characteristics that fall between the High and Low Frequency modes. Here potential bends/breaks are indicated at approximately 0.55m in the two potentially broken bolts but not in the potentially intact bolt. The probe characteristic indicates that there is a peak at 0.41m, however this is seen at a much higher amplitude setting on the instrument, 69dB as opposed to 47dB, for the tests done in-situ.

**2.7.2 Deep Soft Seam: 217’s Return Gate Junction**
The ultrasonics tests were conducted on 4 pre-prepared bolts labelled No. 1 - 4. Bolts 1–3 were existing 2.4m AT while Bolt 4 was a potentially intact SAT bolt installed a few months earlier.

**High Frequency Tests**
Numerous HF tests were undertaken using the 2MHz, 4MHz large headed and 4MHz small headed probes on one of the existing bolts, Bolt 1, (potentially broken) and Bolt 4 (potentially intact SAT bolt). An end reflection or reflection representing a bend/break within the length of the bolt could not be seen with any of the three probes on either of the 2 bolts.

It was decided to use the 4MHz large headed probe to interrogate all 4 bolts using the same machine settings in order to try to define any subtle differences between them. **Figure 21** shows the results from the 4 bolts with the 4MHz probe. These results show no significant peaks associated with either the bolt end or a break. Interestingly, for the same instrument settings the SAT bolt records slightly higher background noise, this could be related to the more pronounced ribbed profile of the bolt.

**Low Frequency Tests**
The 850kHz, 70kHz, 50kHz, 100kHz, ICON(70kHz) and Pundit(150kHz) probes were used on Bolt 1 and Bolt 4 in a similar methodology as for the High Frequency tests described immediately above. None of the probes gave definitive reflections indicative
of the bolt end. Various peaks were observed between 0.40 and 1.00m for both bolts, dependant upon probe, which could have been related to bends.

It was decided to use the 850KHz probe to obtain comparative traces from all 4 bolts. Figure 22 shows the results. These results show no significant peaks associated with either the bolt end or a break. Interestingly, as with the HF 4MHz probe for the same instrument settings, the SAT bolt records slightly higher background noise. This could be related to the more pronounced ribbed profile of the bolt.

2.8 SUMMARY OF ULTRASONIC RESEARCH

Although relevant research was undertaken there was no success in sourcing a better instrument for the ultrasonic testing of rockbolts. Thus work continued with the Krautkramer USD10 NF, with which there had been some success under the previous research project.

Under this project it was used with some success to survey selected rockbolts at Thoresby Colliery along the length of PG42’s Loader Gate that was being recovered for re-use following 7 years of abandonment. The survey aimed to determine the proportion of roofbolts likely to be intact such that decisions could be made as to a suitable level of additional reinforcement required to maintain roof stability for the gate’s re-use.

The survey was carried out on 50 roofbolts. 19 tests were able to successfully detect the end of the bolts and thus indicate they were intact. Of the other tests there was no indication that the bolts were either broken or badly bent. It was likely that the reason that bolt ends were not detected was the unfavourable chamfer angle at the back end of the bolts.

At a separate UK colliery, Welbeck, the instrument was used to survey bolts in a gate road where there were known to be bolts broken at approximately 0.6m into the roof. Of the bolts tested reflections were observed from approximately 0.6m into the roof. However it was not possible to determine, using the ultrasonics method alone whether these were related to bends or breaks, in this case the survey required the use of the RF system in order to detect the difference.
3 NON DESTRUCTIVE TESTING OF ROCKBOLTS, NDT – RADIO FREQUENCY METHOD

3.1 INTRODUCTION

The method uses electro-magnetic waves as opposed to ultrasonics and involves direct connection of a tuneable RF source to a circuit including the installed rockbolt or cablebolt, which must be an electrical conductor, in order to measure its effective length. Two basic circuit geometries have been used, referred to as ‘perpendicular’ and ‘parallel’.

The perpendicular (or ‘orthogonal’) circuit uses a single rockbolt installed in the rock, with a conducting ‘ground plane’ arranged perpendicularly to, but not in electrical contact with, the exposed end of the rockbolt. The RF generator in the readout instrument is connected to the ground plane and the circuit to the rockbolt is completed through the rock. The rockbolt acts as a radiating antenna (aerial) within the rock, but has a maximum efficiency at one frequency, the resonant frequency \( f(0) \), which depends on the length of the bolt. Recent work with this method has investigated the use of the mesh as a ground plane rather than using a purpose built ground plane.

In the parallel method the RF instrument is connected to adjacent, but electrically isolated rockbolts. The resonant frequency in this case is a function of the mean length of the two rockbolts. The RF method relies on the bolts under test being electrically isolated from each other and from other steel in the roadway such as the ‘W’ straps, mesh and other bolts.

The method is patented under UK patent 2 304 417, ‘A method and apparatus for monitoring reinforcing tendons’.

3.2 INSTRUMENT DEVELOPMENT, MFJ TO LRF

3.2.1 MFJ-259 Instrument

Under the ECSC project AB149 (ECSC 2000[1]), considerable success was achieved with this system in roofs composed of materials other than Coal Measure mudstone/siltstones. In particular, success was achieved in coal measures limestone and slate. A major goal of this project was to extend the technique to coal measure mine roof more typical of Northern Europe, (mudstones, siltstones and sandstone). It was thought that, in order to achieve this, an instrument with a lower frequency range was required. Initial work under this project obtained the components necessary to build a lower frequency instrument.

The target range for the new instrument, the LRF, was 20kHz to 20 MHz in 3 switched and tuneable ranges, with the option of phase change detection for identifying resonant frequencies.

It had become necessary to extend the range of the current RF resonance detection meter, the MFJ-259, below 1 MHz, in view of experimental results from Middleton Mine and HBCM (Meyreuil Colliery) in particular, (both with limestone roofs). The resonant frequencies for 10 – 12 metre tendons at Middleton and 5 – 7 metre tendons at Meyreuil were such that only the higher frequency parts of the resonance peaks could be observed. It was also possible that the lack of experimental data from some other coal-measure rocks may have been due to insufficient low-frequency range.
Early attempts to extend the range of the original MFJ-259 meter below its minimum frequency of 1.4 MHz were only partially successful. The MFJ uses inductance-capacitance (L-C) resonant circuits for tuning, with discrete switched inductors being tuned by a vane-type variable capacitor. Adding larger switched inductors extended the range down to about 0.4 MHz, but the L-values were necessarily quite high and caused some problems with current surges through the null-detector. Also, a strong internal resonance was encountered when even larger L-values were used to extend the range below 0.4 MHz. This effect swamped that of the tuning capacitor, reversed its action and made the unit unstable.

It was decided to replace the L-C oscillator by a resistance-capacitance (R-C) based oscillator, available as an integrated circuit. This oscillator used discrete switched capacitors, tuned by a variable resistor, and had a range of 20 kHz – 20 MHz using reasonable values of C and R.

A prototype oscillator was constructed and tested in the laboratory, using an open-circuit co-axial cable to simulate a tendon pair, and an oscilloscope to detect phase and amplitude changes at resonance. The oscillator output was stable and of sufficiently constant amplitude throughout the frequency range. Three overlapping ranges were established:

(i) “Low”: 25 – 380 kHz,
(ii) “Medium”: 220 kHz – 3.15 MHz, and
(iii) “High”: 1.6 – 19.8 MHz.

Tuning within each range was by a 10-turn potentiometer, which allowed sensitive adjustment.

The MFJ ‘s on-board frequency meter was found not to function well at lower frequencies, so rather than incorporate the new oscillator into the MFJ, a new unit was constructed based around a proprietary hand-held digital frequency meter and a simple RF diode bridge as the resonance null-detector. The out-of-balance bridge voltage was indicated by a 0 – 1 volt digital LCD voltmeter with a simulated analogue meter output. This provided a more robust indicator than a sensitive moving-coil current-meter and was visually easier to balance than a purely digital meter.

3.2.2 LRF Instrument

Figure 23 shows photographs of the original MFJ –259 instrument and the new LRF instrument.

Underground trials on pairs of tendons at Middleton Mine showed that the new LRF unit worked well. The resonances observed coincided with the frequencies determined using the MFJ instrument, and it was found possible to plot them to lower frequencies using the LRF meter. Figure 24 compares the resonances for a tendon-pair (10m and 12m) using (a) the extended-range MFJ and (b) the new LRF prototype.

The resolution of the resonant frequencies was however restricted by the 8-segment structure of the LCD voltmeter. In order to obtain maximum accuracy, it was necessary to note the transition between segments in terms of frequency for both rising and falling directions of tuning. This proved to be time-consuming and could have been confusing when recording results.
The voltmeter was therefore replaced by a 100 micro-amp moving coil meter. This greatly improved the ease of operation, but introduced an internal resonance, due to the inductance of the meter, centred around 280 kHz. Further underground trials showed that this could be mistaken for, or interfere with, a real external resonance in some circumstances.

It was found to be possible to reduce the internal resonance or move it out of the frequency range of the LRF by modifying the capacitance and/or the resistance of the bridge detector circuit, but only at the expense of unacceptable reductions in sensitivity.

Two methods of avoiding the problem were explored:

(a) retain the simulated analogue voltmeter for coarse visual balance, but add a small digital panel-meter for final balance and resonance-plotting;

(b) include a transformer isolation of the moving-coil meter from the bridge circuit.

A facility for changing the internal bridge-balance resistor was added to the LRF. This was to accommodate resonant systems with Characteristic Impedances other than the 50 Ohms normally encountered in the antenna and feed-lines for which the MFJ was originally intended. A rotary switch enabled the bridge options of 20, 50 or 100 Ohms to be selected. It was found that selecting the 100 Ohm option at Middleton Mine produced a more prominent resonance than either 20 or 50 Ohms. This suggested that Characteristic Impedances of at least 100 Ohms needed to be accommodated.

The prototype instrument was then repackaged with a dedicated frequency meter, and wider range of Characteristic Impedance options. The unit is battery powered (4 x AA).

3.3 LRF INSTRUMENT TESTS AT THORESBY COLLIERY

3.3.1 Initial Tests with the LRF Meter

Comparative tests using the LRF and MFJ meters were undertaken on the 0.6m, 1.2m and 2.4m long datum bolts installed at the 177MM at Thoresby Colliery in 42’s Loader Gate. (The tests results from 42’s Loader Gate are described in section 3.5.3 below and are discussed here only in terms of instrument developments). When the datum bolts at the 177 mark at Thoresby were tested with the MFJ the longest datum pair (2.4m and 1.2m long bolts, mean length 1.8 m) had a resonant frequency, \(f(0)\), of 1.9 MHz which was close to its lower frequency limit and this meant that the complete resonance trough could not be followed, Figure 25(a). However when the LRF instrument was used the complete resonance trough for the longer datum pair, 1.2m long and 2.4m long was recorded, Figure 25(b).

3.3.2 Effect of Lead-Length

During the comparative tests using the LRF and MFJ meters on the 0.6m, 1.2m and 2.4m long datum bolts installed at the 177MM at Thoresby Colliery in 42’s Loader Gate the testing revealed an apparently large discrepancy between the indicated values of \(f(0)\) for each tested bolt pair when using the MFJ with its 2.7m long lead set and the LRF meter with its 4.0m long lead set. This is illustrated in Columns 1 and 2 in Table 3 below.
Table 3. Effect of lead length on measured resonant frequency (MHz)

<table>
<thead>
<tr>
<th>Datum Bolt Pair</th>
<th>MFJ + 2.7 m long leads</th>
<th>LRF + 4.0 m long leads</th>
<th>LRF + 2.7 m long leads</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6 + 1.2 m</td>
<td>3.862</td>
<td>3.123</td>
<td>3.892</td>
</tr>
<tr>
<td>1.2 + 2.4 m</td>
<td>1.935</td>
<td>1.629</td>
<td>1.892</td>
</tr>
<tr>
<td>0.6 + 2.4 m</td>
<td>3.232</td>
<td>2.741</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Laboratory experiments showed that this could have been due to the different lengths (2.7 m and 4.0 m) of connecting leads used with each instrument. Repeating the underground measurements confirmed that the MFJ and LRF frequencies were in fact comparable, Column 3 of Table 3, provided the same leads were used. However, the LRF was found to be much more sensitive to lead length than the MFJ in the laboratory.

All further measurements with the LRF were made using the 2.7 m leads.

3.3.3 Improved connection methods

Improved methods of making electrical connections to existing, often corroded bolts were examined alongside the LRF instrument developments. The original method entailed removing sufficient corrosion from the tops of the thread to reveal a stripe of bare metal, using an abrasive pad or wire brush, then to clamp a braided copper wire with an attached electrical socket to the clean area, using a "Jubilee" pipe-clip or similar device. This was a time-consuming operation if a large number of bolts were to be tested, partly because of the need to align the clean stripe and the wire. Two alternatives were tested. The first was to fit electrical sockets to large "crocodile" battery clips. These are quickly applied to a cleaned area and allow multiple contact points without critical alignment. The second alternative consisted of a drilled-out nut, with three radial, hard-point, set-screws operated by hexagon-wrench, and an electrical socket. This also provided multiple contacts and was reasonable quick to install. Figure 26(a) shows a photograph indicating the evolution of the connection methods.

In practice, the "crocodile clip" method was found to be much less reliable in terms of obtaining a good connection at first attempt when compared to the "nut connector". In addition, the physical size of the crocodile clips and possible variations in their alignment was found to be an additional factor in the sensitivity of the LRF to lead-length in these conditions. Figure 26(b) shows the effect of connector type, particularly the large "crocodile clips".

3.3.4 Effects of “stray” capacitance and inductance

The resonant frequency of a bolt-pair may be regarded as a function of the capacitance and inductance of the circuit formed by the pair and the material between them. The presence of other "stray" capacitances and inductances due to, for example, the connection between the pair and the meter, can unduly affect the measured frequency, unless they are small in comparison.

The effect of the connecting lead on the measured resonant frequencies appears not to be confined to its length alone. The "V" formed by the individual wires between the end of the co-axial section and the bolts being tested has capacitance and inductance values which depend on the distance apart of the bolts. This had not previously presented a problem, but the combination of relatively short bolt-pairs with low
capacitances and inductances and a more lead-sensitive meter was found to accentuate the effect. Thus a scatter of resonant frequencies has been observed for different bolt-pairs with the same mean length but with different separations. This is illustrated in Figure 26(b), which also showed an effect of connector type, particularly the large “crocodile clips”.

3.3.5 Modifications to LRF meter

As a result of the effects noted in Section 3.3.2 and 3.3.4 above, the bridge circuit of the LRF meter was modified using an alternative connection to the balancing bridge to reduce the effect of external capacitances and inductances. This measure also reduced the sensitivity of the null-balance indicator, so an additional amplification stage was incorporated in compensation.

Further tests on the outbye datum bolts at the 177MM at Thoresby however showed that an effect of lead-length was still present and that the resonance signatures had changed considerably. The resonance troughs for the modified instrument, Figure 27(b), were much shallower and wider than before, Figure 27(a).

Tests with the modified LRF instrument on datum bolts installed inbye in 42’s Loader Gate at the 456-457MM were unable to detect the fundamental resonant frequencies observed previously. Instead, wider, shallower troughs were observed at higher frequencies. These are consistent with being the open-circuit harmonic at 3 times the fundamental frequency which was considered to be related to the modification of the LRF rather than a poor connection to the test bolts.

The modified LRF meter was used at Middleton Mine to check for lead-length dependence and sensitivity. The lead-length variation was found to be much smaller than at Thoresby (typically 5% compared with 30%) and there was no difficulty in observing fundamental resonances at a range of frequencies, including the 1 – 3 M Hz band which was missing at Thoresby.

3.3.6 Further Development of LRF Instrument

The modifications to the original circuit, intended to reduce the effect of connecting leads, were removed because of undesirable side-effects. Although the modified LRF had been found to perform satisfactorily in the limestone of Middleton Mine, the results from the coal-measures at Thoresby Colliery showed considerable increases in the frequency-width of resonances, with some becoming undetectable.

In order to try to understand the lack of resonances at Thoresby with the LRF the MFJ was used and the ‘X’ and ‘R’ values recorded, an option not available on the LRF, so as to ascertain the cause for the lack of valid resonances with the LRF. However results using the MFJ instrument also appeared to show wider resonances than before in some circumstances. It was therefore considered possible that these changes could be due to deterioration of the dielectric properties of the roof at Thoresby.

In order to investigate the possibility that there could be a time dependant change at Thoresby the re-modified LRF instrument was used to re-measure the datum bolts and compare the results with those obtained using the original circuit.

Figure 28(a) shows the observed resonances for the pair of 2.4m datum bolts at Thoresby, using the original, modified and re-modified LRF instruments. The broad
resonances at 0.03 and 5 MHz introduced by the modification can be seen, as well as the lack of resonance at the original position. The re-modified version shows a resonance of similar width to the original, but centred on a slightly lower frequency.

The pattern is repeated for the other datum bolt pairs, with the shift to a lower frequency becoming more pronounced as the mean length increases. This is illustrated in Figure 28(b), which is a plot of 1/frequency versus mean length for the original version (“Th3&4”) and the re-modified version (“Th7”). Since the gradient of the 1/f vs L plot depends on the effective dielectric constant of the material between the pair of bolts, the difference in gradient between the two sets of measurements may be evidence for a time-dependent effect. “Th3” and “Th4” were obtained on 19th April 2002 and 3rd May 2002, and “Th7” on 20th February 2003.

The apparent increases in the width and position of some resonances recorded by the MFJ instrument at Thoresby were found to be due to an intermittent fault in one of the cables used to connect the instrument to the roof-bolts. The effect of this was to disconnect one of the pair of bolts, leaving the other connected. Examination of the relevant “X” and “R” signatures recorded at the same time as the SWR data allowed this condition to be identified and spurious data discarded. This fault could not however explain the change in performance of the LRF instrument after modification because of the similarity between readings taken with both sets of connecting cables. Further analysis has indicated that the modification to the LRF reversed the sense of the null detector so that troughs became peaks and vice versa. If the ‘modified’ data in Figure 28(a) is plotted as (100-null)%, the peaks and troughs closely coincide with the ‘original’, with a resonance at 1-2MHz.

3.4 NEW INSTRUMENT – VIA ANALYSER

The remaining problems with lead-length sensitivity of the LRF, and the potential usefulness of “X” and “R” measurements with the MFJ at the lower frequencies covered only by the LRF, prompted a renewed search for a commercial instrument with a low frequency range.

It was discovered that a North American manufacturer, had recently introduced a new version of one of their instruments, with the frequency range extended down from the previous 1 MHz to 0.1 MHz. Their original had been used on a previous investigation of the RF method. The original version’s main advantage was the automatic scanning and graphical display of SWR over a selected frequency range, but the lower frequency limit of 1 MHz and the lack of “X” and “R” measurements made it less useful than the MFJ device.

The new instrument provides a graphic display of “X” and “R” in addition to SWR over a selectable frequency range between 0.1 and 54 MHz. The instrument will calculate and display the position of the strongest resonance in terms of minimum SWR and resonant frequency. There are four non-volatile memories for storing data scans and data may be transferred through a serial port to a PC for further analysis by the included software package or customised spreadsheets.

The operation of the VIA instrument was checked on a range of previously tested tendon pairs at Middleton Mine. Figure 29 shows the results indicating a close correlation of the VIA results with those from the LRF and MFJ instruments. The instrument was also used and checked on the datum bolts at Thoresby Colliery. Figure 30(a) shows the plot for the 0.6m datum and 1.2m datum bolt combination, with
a well defined SWR curve with confirmatory minima in the Z, X and R values. **Figure 30(b)** shows the results from the 3 datum bolt combinations plotted as mean length against 1/(f). The results show that there was good correlation for the 0.9m and 1.5m mean length datum bolt combinations but the 1.2m and 2.4m long datum bolts with a mean length of 1.8m recorded a lower frequency than expected leading to a higher 1/f(0) value. The reason for this is not clear and further testing is required to understand why. **Table 4** below shows the tests plotted in **Figure 30(b)** (prefix THOR8), alongside those from a subsequent visit, (pre-fixed THOR10) which indicate the repeatability of the results.

**Table 4. RF test results from the datum bolts at Thoresby Colliery to indicate repeatability of results**

<table>
<thead>
<tr>
<th>Test Ref.</th>
<th>Test Combination</th>
<th>Dist Apart (m)</th>
<th>Resonant Freq. (MHz)</th>
<th>Valid Resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td>THOR802</td>
<td>177MM, 0.6m Datum – 1.2m Datum</td>
<td>0.9</td>
<td>4.00</td>
<td>Y</td>
</tr>
<tr>
<td>THOR1004</td>
<td>177MM, 0.6m Datum – 1.2m Datum</td>
<td>0.9</td>
<td>3.95</td>
<td>Y</td>
</tr>
<tr>
<td>THOR801</td>
<td>177MM, 1.2m Datum – 2.4m Datum</td>
<td>0.5</td>
<td>1.3</td>
<td>Y</td>
</tr>
<tr>
<td>THOR1005</td>
<td>177MM, 1.2m Datum – 2.4m Datum</td>
<td>0.5</td>
<td>1.24</td>
<td>Y</td>
</tr>
<tr>
<td>THOR804</td>
<td>177MM, 0.6m Datum – 2.4m Datum</td>
<td>1.4</td>
<td>3.6</td>
<td>Y</td>
</tr>
</tbody>
</table>

### 3.5 RF MEASUREMENT RESULTS

#### 3.5.1 RF Measurements at an Evaporite Mine

A series of measurements was carried out underground at an evaporite mine using both the ultrasonics and radio frequency systems to test whether each system would be successful in determining the intact length of in-situ rock bolts in salt. The ultrasonics test results were given in Section 2.5 above.

For these tests, 6 x 20mm diameter resin encapsulated bolts were specially installed into a dry salt ribside. These comprised three bolt pairs of lengths, 1.4m, 1.0m and 0.6m as previously shown in **Figure 14**. As the bolt lengths were relatively short in these tests, the MFJ instrument was used.

The resonant frequency was successfully measured for each of the pairs of equal length bolts and for two pairs of bolts of differing lengths, bolts 2 and 3 and bolts 4 and 5. **Figure 31** shows the resulting calibration curve developed from these tests and indicates the frequency values changing consistently with bolt length. These results compare well with the database of information already obtained for a range of tendon systems and rock types measured at sites worldwide.

In order to develop a user friendly calibrated system for the mine the following variables were recommended to be quantified using the RF system :-

(a) The effect of variations in rock type and rock conditions, ie dry salt, damp or wet salt, dry potash and wet potash.

(b) The effect of variation in bolt spacing.
As the mine uses tendons up to 6m in length, tendons ranging from 0.6m up to 8m were recommended to be tested. However this follow up work was not completed within the timescale of the project.

3.5.2 RF Measurements on Large Diameter Anchor Bolts at a Power Station

A trial of the RF system at a power station to determine the in-situ length of 84mm diameter machine anchorage bolts in concrete was unsuccessful due to an electrical short circuit between the test bolts within the embedment concrete. This was probably due to reinforcing steel within the concrete. This is an inherent problem with the method, which prevents it being applicable in some circumstances, and illustrates the need to have more than one method available for determining the in-situ length of reinforcement tendons.

3.5.3 RF Measurements on Ground Anchors in Road Cutting

The RF method was assessed for application in seeking defective coupled ground anchors, 36mm diameter, in a UK trunk road cutting in weathered slate, where cases of broken bolts had previously occurred. The work was complicated by the fact that no calibration anchors were available and there was some uncertainty over the original length of at least some of the anchors tested.

**Ground-plane method used at the road cutting**

In view of the wide spacing of the anchors, relative to previous experience of common underground reinforcement, the ground-plane technique was selected for single, isolated tendons. A larger than normal, 1 metre square, ground-plane was constructed with specific attachments and electrical connections for the anchors. The results were disappointing with only broad, high-frequency resonances due to connecting cables and the ground-plane itself. In addition, the recessed anchors would not allow direct connection of the ground-plane. An extension-piece was constructed to solve this problem and a second site-visit made, but with the same results.

**Parallel method used at the road cutting**

Connecting anchors to the instrument in pairs did however produce deep, narrow resonances in the frequency range which would be expected for long tendons in this type of rock. *Figure 32(a)* shows resonance data from Bolt RA2 No. 124 in parallel with Bolt RA2 No. 82. The resonant frequency f(0) is taken as that corresponding to the minimum in the SWR trace, and was 0.650 MHz in this case. The Z, X and R parameters are automatically recorded by the instrument and may give additional information related to the dielectric properties of the materials between the anchors. The value of 1/f(0) is proportional to the mean length of the two tendons, so for an array of tendons with the equal nominal lengths, a short or broken member of a pair will be detected by an anomalous value of f(0). The f(0) values for the pairs tested are given in *Table 5*.

RB1#231 and RB1#230 are the upper and lower respectively of two “spot” bolts to the W of RA2#108.

The nominal lengths of the anchors and bolts where known and are given in *Table 6*.

The lengths of the RB1 bolts are variously quoted as “0.5m” and “1-5m”.

30
Table 5  Resonant frequencies of ground anchor pairs at the road cutting

<table>
<thead>
<tr>
<th>Pair</th>
<th>Anchors Reference Numbers</th>
<th>f(0), (MHz)</th>
<th>1/f(0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>RA2#124 + RA2#82</td>
<td>0.650</td>
<td>1.67</td>
</tr>
<tr>
<td>B</td>
<td>RA2#124 + RA2#125</td>
<td>0.310</td>
<td>3.23</td>
</tr>
<tr>
<td>C</td>
<td>RA2#125 + RA2#113</td>
<td>0.225</td>
<td>4.44</td>
</tr>
<tr>
<td>D</td>
<td>RB1#231 + RB1#230</td>
<td>0.850</td>
<td>1.18</td>
</tr>
<tr>
<td>E</td>
<td>RB1#230 + RA2#108</td>
<td>0.600</td>
<td>1.58</td>
</tr>
<tr>
<td>F</td>
<td>RA2#108 + RB1#231</td>
<td>0.450</td>
<td>2.22</td>
</tr>
</tbody>
</table>

Table 6  Nominal lengths (m) of the anchors tested in the road cutting

<table>
<thead>
<tr>
<th>Anchor Component</th>
<th>RA2#113</th>
<th>RA2#124</th>
<th>RA2#82</th>
<th>RA2#125</th>
<th>RA2#108</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inner Length ‘fixed’ (m)</td>
<td>5.5</td>
<td>7</td>
<td>7</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>Outer Length ‘free’ (m)</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>Total Length (m)</td>
<td>10.5</td>
<td>12</td>
<td>12</td>
<td>12.5</td>
<td>10.5</td>
</tr>
</tbody>
</table>

Interpretation of RF data from the road cutting

Figure 32(b) shows the values of 1/ f(0) plotted against the mean nominal lengths for the pairs of tendons tested. The theoretical linear relationship between 1/f(0) and nominal mean length of the anchor pairs has been drawn passing through the origin and the highest measured value of 1/f(0).

This form of relationship has been confirmed experimentally in a range of rock types, but the gradient and intercept are site-specific and must be determined using several pairs of tendons with known lengths.

However, since the highest value of 1/f(0) most probably corresponds to the longest intact pair (Pair C), with a mean length of 11.5m, it is clear that Pairs A and B have anomalously low values of 1/f(0) and have actual mean lengths of less than their mean nominal values.

This suggests that RA2 #82, and RA2 #124 are not electrically continuous for their full lengths. Using the approximate linear relationship, and solving the relevant simultaneous equations, gives estimated lengths of 4.5m and 4.2m, respectively.

This could be due to a lack of electrical continuity between the free (5m) section and the fixed section in each anchor.

The set of measurements (Pairs D, E & F) using RA2#108 and the two “spot” bolts suggest that the latter are not the same lengths, since they each resonate at different frequencies when paired with the adjacent anchor. Using the same method of analysis gives the bolt lengths as 1.4m and 4.7m, and the electrical length of the anchor as 6.8m.

It was concluded that, with the installation of some calibration anchors of known length the RF method could be used to obtain useful data on the condition of the installed anchors at the site.
3.5.4 RF Measurements at Thoresby Colliery

Calibration Tests
Prior to testing the in-situ bolts at Thoresby, calibration tests were conducted on three datum bolts installed in 42’s Loader Gate at the 178m mark. These were tested to establish the relationship between Radio Frequency and bolt length in the Thoresby roof strata.

The following three combinations of the datum bolt pairings were installed and tested:

(a) 0.6m + 1.2m datum bolts, 0.93m apart, mean length = 0.9m
(b) 0.6m + 2.4m datum bolts, 1.5m apart, mean length = 1.5m
(c) 1.2m + 2.4m datum bolts, 0.57m apart, mean length = 1.8m.

The results from these bolts are shown in Figure. Figure 33(a) reproduces the results from the LRF instrument and Figure 33(b) plots the frequency of the SWR defined by the base of the curve against mean bolt length. For standard tendon spacing this is normally a simple inverse relationship, however, this graph shows a variance in the data points which may be related to the difference in distance between the tested pairs. The 2.4m bolt and the 0.6m bolt were significantly further apart than the other pairings and there was a bolt in the roof between, and in line with, the two bolts tested. This may have influenced the results.

The gradient of the calibration graph (1/f(0) versus L) is a function of the electrical permittivity and magnetic permeability of the strata, relative to free space. The larger these values, the lower the resonant frequency for a given mean length of tendon. Figure 34 shows the comparative plots for other strata previously obtained. The gradient for the Thoresby strata is considerably greater than that encountered in Middleton limestone, slate or halite and even greater than the previous highest at Meyreuil (carboniferous limestone). As a result, the resonant frequencies of even the relatively short bolt-pairs at Thoresby are unusually low compared with previous experience. It is important to note that these measurements at Thoresby were the first to be successfully obtained in a coal mine with a more typical siltstone/sandstone roof and, as such, provided encouragement to continue development of the method.

In-Situ Bolt Results
Once the relationship between resonant frequency and bolt length had been established for 42’s Loader Gate a number of tests were carried out on selected in-situ rockbolts.

As stated earlier, one of the requirements for testing rockbolts with the RF system is that the bolt under investigation has to be electrically isolated from other conducting elements in the vicinity. In 42’s Loader Gate at Thoresby most of the rockbolts were installed through W-straps and steel mesh. The steel mesh was found to be highly corroded and was easily removed from around the bolt but the ‘W’-straps had to be cut in order to isolate the bolts under test.

Table 7 lists the in-situ bolts which were tested including the test results recorded at the 178m mark at the site of the datum bolts where a 2.4m datum bolt was tested with a 2.4m in-situ spot bolt and two 2.4m in-situ bolts, one of which was isolated and the other connected to an adjacent ‘W’ strap and the roof mesh.
The test results were mixed. Whilst tests on rockbolts towards the outbye end of the roadway gave valid resonances, only a weak resonance was obtained from the test bolt at 245m mark and bolts tested inbye of 389m mark did not give valid resonances at all.

Table 7  RF tests conducted on selected in-situ rockbolts at Thoresby Colliery

<table>
<thead>
<tr>
<th>Bolt Reference</th>
<th>Roof Conditions</th>
<th>Comments</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>178MM, 2.4m Datum Bolt and 2.4m Spot Bolt beside No.2 Bolt (connected to adjacent W strap)</td>
<td>Dry, Siltstone roof</td>
<td>Re-tested after no positive results inbye</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>178MM, 2.4m Isolated Spot beside No.4 Bolt @179.4MM and 2.4m Bolt in adjacent W strap</td>
<td>Dry, Siltstone roof</td>
<td>Two Old Bolts tested after no resonances on paired old bolts inbye</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>Immediate roof geology visibly changes at approx. 197MM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>245MM, Bolt 2 in Strap (Isolated) with Bolt 1 in Strap</td>
<td>Damp Roof Sandstone Roof</td>
<td>Not previously tested – tested where geology/water conditions ‘intermediate’</td>
<td>Weak resonance</td>
</tr>
<tr>
<td>389MM, Bolt 7 in Strap (Isolated) with Bolt 6 in Strap</td>
<td>Damp Sandstone roof</td>
<td>Short Circuit Indicated -Tried various combinations of the following: 1. Croc clips versus jubilee clips 2. Add. cleaning of bolt ends to ensure good contact 3. Checks on connection leads 4. Different lead set</td>
<td>No resonance</td>
</tr>
<tr>
<td>445MM, Bolt 3 in Strap (Isolated) with Bolt 4 in Strap</td>
<td>Damp Sandstone Roof</td>
<td></td>
<td>No resonance</td>
</tr>
<tr>
<td>464MM, Bolt 6 in Strap (Isolated) with Bolt 5 in Strap</td>
<td>Damp Sandstone Roof</td>
<td></td>
<td>No resonance</td>
</tr>
<tr>
<td>865MM, Bolt 7 in Strap (Isolated) with Bolt 6 in Strap</td>
<td>Damp Sandstone Roof</td>
<td></td>
<td>No resonance</td>
</tr>
<tr>
<td>851MM, Bolt 7 in Strap (Isolated) with Bolt 6 in Strap</td>
<td>Damp Sandstone Roof</td>
<td></td>
<td>No resonance</td>
</tr>
<tr>
<td>830MM, Bolt 3 in Strap (Isolated) with 831MM, Bolt 2 in Strap also Isolated</td>
<td>Damp Sandstone Roof</td>
<td></td>
<td>No resonance</td>
</tr>
</tbody>
</table>

**Figure 35** shows the successful test results from the in-situ/datum and in-situ/in-situ bolts tested at the 178mm. This indicates a variation in the resonant frequency between the two different combinations of 2.4m bolts tested; the 2.4m datum and 2.4m in-situ and the 2x2.4m in-situ, one of which was isolated. This highlights that bolt condition, connection quality and bolt spacing can influence the result. The sharper deeper resonance curves recorded from the two in-situ bolts is probably associated with the fact they are closer together than the other bolts.

A re-test on the 2.4m datum bolt and 2.4m in-situ spot bolt using the LRF meter, was carried out following the lack of valid resonances inbye to check that the RF system was still functioning correctly and to check on repeatability of the test technique. This test confirmed that the system was indeed functioning correctly as indicated in Figure 35(a).

**Tests on the ‘Additional’ Datum Bolts in ‘Wet’ Ground**

The reasons for the lack of valid resonances for the inbye bolts were not clear. One possibility was that there was a change in the dielectric properties of the roof strata.
The efficiency of the RF system is related to the dielectric constant of the rock mass. A rock with high conductivity will effectively short circuit the system, preventing a successful test. This may be caused by an increase in the moisture content in the strata. In order to test this hypothesis further datum bolts were installed inbye where the roof conditions were ‘wet’; those that were damper were denoted by drippers at various intervals along the roof.

The three new datum bolts were installed at the 456m mark, two 2.4m long and one 1.8m long. It was requested that they were installed in a pattern creating an equilateral triangle with distances between the bolts of 0.64m (the same distance as between bolts in the normal strap pattern along the gate). These were to be installed such that one of the 2.4m long bolts should be installed approximately 0.64m from the isolated test bolt at the installation site. Unfortunately due to the location of air and water services it was not possible to install the 3 bolts adjacent to a pre-tested isolated in-situ bolt. The distances between the 3 datum bolts and the tests carried out with them are given in Table 8 below.

Table 8. RF tests conducted with the LRF instrument on the additional datum bolts at 456/457MM, 42’s Loader Gate, Thoresby Colliery

<table>
<thead>
<tr>
<th>Bolt Reference (bolt position in strap L to R inbye)</th>
<th>Roof Conditions</th>
<th>Comments</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 2.4 m Datum bolt, 0.6m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘new nut connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 2.4 m Datum bolt, 0.6m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘croc. clip connectors’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 1.8m Datum Bolt, 0.82m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘new nut connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 1.8m Datum Bolt, 0.78m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘new nut connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 2.4 m No. 2 Bolt in 457MM Strap, 0.56m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘new nut connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 2.4 m No. 2 Bolt in 457MM Strap, 0.45m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘new nut connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 2.4 m No. 2 Bolt in 457MM Strap, 0.45m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘croc clip connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>456/457MM, 2.4m Datum Bolt with 2.4 m No. 2 Bolt in 457MM Strap, 0.45m apart</td>
<td>Damp Sandstone Roof</td>
<td>With ‘jubilee clip connector’</td>
<td>Valid resonance</td>
</tr>
<tr>
<td>464MM, Bolt 6 in Strap, (Isolated) with Bolt 5 in Strap</td>
<td>Damp Sandstone Roof</td>
<td>With ‘new nut connector’</td>
<td>No resonance</td>
</tr>
</tbody>
</table>

Figure 36 shows the results of the RF tests on these additional datum bolts (circles) and includes the results from the original datum bolts at the 178m mark (diamonds).
The results show that despite the ‘wet roof strata’ valid resonances were recorded for the newly installed bolts.

The suite of tests conducted between the inbye datum bolts and the existing 2.4m bolts in ('W' Straps) in the vicinity of the datum bolts are shown as squares on Figure 36. These indicate valid resonances but at variable frequencies. This could be attributed to:

(a) distance between the tested bolts  
(b) nature of connection to bolt.

Further tests were also conducted with the VIA instrument on Datum bolts when in combination with either the newly installed additionals bolts in the gate road (1.8m long), with the existing (original) 2.4m long bolts, and existing to existing bolts. As with the LRF instrument valid resonances could be obtained from Datum to existing bolts, Figure 37(a) but not when existing to existing bolts were tested for example Figure 37(b). Figure 37(b) shows that although there is broad inflection in the SWR the confirmatory X, Z and R values have been shifted toward the very bottom end of the frequency range of the instrument.

The two sets of results with the LRF and VIA indicate that, under certain circumstances (i.e. inbye of 177MM), valid resonances can be obtained where at least one of the bolts is newly installed, but not necessarily when both the bolts are ‘old’.

The reason why there should be valid resonances from newly installed bolts (datums or replacements) but not the older in-situ bolts inbye in 42’s Loader Gate is still unclear and further research is required to resolve this issue.

The addition of a bolt plate to one of the datum bolts did not prevent the resonance being detected, in combination with either another datum bolt or with an existing bolt, Figure 38. This eliminates short-circuiting of bolts via the plates and roof surface as a mechanism for lack of resonance. (The datum bolts were installed without plates).

The probability is that new resin effectively insulates the rockbolt from the surrounding rock, but the insulation becomes less effective with time. Thus a resonance may be obtained even if only one bolt of the pair is insulated from the strata by recently installed resin encapsulation.

**Rib - Rib Bolts**

The RF system was also used at Thoresby Colliery in PG42’s Loader gate in order to investigate its potential for detecting the integrity of the 1.8m long steel rib bolts installed in coal. Table 9 below summarises the tests undertaken. In 42’s Loader Gate 3 rib bolts were installed in a vertical row within the seam at nominal spacing of 1.0m along the roadway.

The tests were taken at 3 different locations but the rib condition was considered to be similar in all cases and it was anticipated that the bolts were intact.

The results in Table 9 indicate a high proportion of valid test results (7 of 8) with clear resonances with confirmatory minima in R, Z and X, of which an example is shown in Figure 39(a).
Table 9. RF test results from the VIA instrument on selected steel rib bolts in PG42’s Loader Gate, Thoresby Colliery

<table>
<thead>
<tr>
<th>Test Ref.</th>
<th>Test Combination</th>
<th>Dist Apart (m)</th>
<th>Resonant Freq. (MHz)</th>
<th>Valid Resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td>THOR0903</td>
<td>389MM, Top – Middle</td>
<td>0.50</td>
<td>1.01</td>
<td>Y</td>
</tr>
<tr>
<td>THOR0904</td>
<td>389MM, Middle - Bottom</td>
<td>0.85</td>
<td>0.62</td>
<td>Y</td>
</tr>
<tr>
<td>THOR0906</td>
<td>245MM, Bottom – Bottom</td>
<td>1.00</td>
<td>1.70</td>
<td>?</td>
</tr>
<tr>
<td>THOR1010</td>
<td>254MM, Top – Middle</td>
<td>0.35</td>
<td>2.25</td>
<td>Y</td>
</tr>
<tr>
<td>THOR1011</td>
<td>254MM, Middle - Bottom</td>
<td>0.35</td>
<td>1.90</td>
<td>Y</td>
</tr>
<tr>
<td>THOR1012</td>
<td>254/253MM, Middle - Middle</td>
<td>1.00</td>
<td>2.55</td>
<td>Y</td>
</tr>
<tr>
<td>THOR1013</td>
<td>254/253MM, Top - Middle</td>
<td>0.90</td>
<td>2.20</td>
<td>Y</td>
</tr>
<tr>
<td>THOR1014</td>
<td>254/253MM, Bottom - Middle</td>
<td>0.95</td>
<td>1.85</td>
<td>Y</td>
</tr>
</tbody>
</table>

The resonant frequencies vary from 0.62 MHz to 2.55 MHz. Excluding the first 2 tests they fall within the frequency range of 2.1MHz +/- 0.4MHz. This range of frequencies could be due to a variety of reasons including:

(i) bolt location,
(ii) bolt spacing,
(iii) bolt geometry, or
(iv) bolt integrity.

Figure 39(b) plots the results with bolt location and spacing. There appears to be little correlation with these factors. Provided that there is no other steel in the way the geometry between the tested bolts would be expected to have no influence. As no previous tests have been conducted on bolts in coal there is no calibration data available in order to determine what expected frequency and therefore if any of the bolts are potentially broken (although the resonant frequency would be expected to increase if the bolts were broken).

Use of Mesh as the Ground Plane
The recent installation of new mesh in the roof of PG42’s Loader Gate allowed the “ground-plane” geometry to be tested as a possible alternative to the tendon-pair (or “parallel”) geometry. The tests were undertaken using the VIA instrument and the bolts were connected to the mesh with either a small crocodile clip, or on more recent tests a small terminal block connector in order to increase confidence in the electrical connection to the mesh. The resonant frequency results are given in Table 10.

The results indicate a poor success rate with only 2 of the 12 tests giving a valid resonance. The valid resonance for the 0.6m datum bolt, 4.2 MHz, correlates well with that expected from when it is tested in parallel with a longer bolt. However the valid resonance from the 1.8m datum, 0.65MHz, was lower than expected, as it would be expected to be closer to 2MHz. Despite these results being poor the success of 2 tests indicates that it could have potential and consequently warrants further testing to determine the potential of using the mesh as a ground plane.

3.5.5 RF Testing at Welbeck Colliery

PG312s Loader Gate – 874RM
The objective at this site was to test the newly installed 2.4m rockbolts that were almost certainly intact. This would allow comparison with the Thoresby datum results and
provide a ‘benchmark’ for the tests on potentially broken bolts further outbye in PG312s Loader Gate.

Table 10. RF tests with the VIA instrument with mesh as a ground plane in 42’s Loader Gate, Thoresby Colliery

<table>
<thead>
<tr>
<th>Test Ref.</th>
<th>Test Combination</th>
<th>Dist. to Mesh (m)</th>
<th>Resonant Freq. (MHz)</th>
<th>Valid Resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td>THOR0805</td>
<td>177MM, 0.6m Datum – Mesh</td>
<td>0.60</td>
<td>4.2</td>
<td>Y</td>
</tr>
<tr>
<td>THOR0806</td>
<td>177MM, 1.2m Datum – Mesh</td>
<td>0.62</td>
<td>0.2</td>
<td>??/N</td>
</tr>
<tr>
<td>THOR0807</td>
<td>177MM, 2.4m Datum - Mesh</td>
<td>0.65</td>
<td>0.6</td>
<td>?</td>
</tr>
<tr>
<td>THOR1018</td>
<td>456MM, 2.4m Datum LHS - Mesh</td>
<td>0.15</td>
<td>-</td>
<td>N</td>
</tr>
<tr>
<td>THOR1019</td>
<td>456MM, 2.4m Datum RHS - Mesh</td>
<td>0.15</td>
<td>-</td>
<td>N</td>
</tr>
<tr>
<td>THOR1020</td>
<td>456MM, 1.8m Datum - Mesh</td>
<td>0.15</td>
<td>0.65</td>
<td>Y</td>
</tr>
</tbody>
</table>

All the RF tests were conducted with the new instrument, the VIA Analyser. The standard lead set was used and all connections to bolts were via the special reamed out nuts with grub screws. All the bolts were tested with their nuts and plates removed, mesh isolated if necessary, and those bolts tested in the ‘W’ strap were in the centre of the hole and definitely isolated from the strap. All tests where the mesh was used as the ground plane made use of a small crocodile clip for electrical connection. Table 11 summarises the results.

Table 11. RF results PG312’s Loader Gate at 874RM, Welbeck Colliery

<table>
<thead>
<tr>
<th>Test Ref.</th>
<th>Test Combination</th>
<th>Dist. Apart (m)</th>
<th>Resonant Freq. (MHz)</th>
<th>Valid Resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEL31201</td>
<td>2.4m Spot-SAT to 2.4m Spot-SAT</td>
<td>0.53</td>
<td>0.74</td>
<td>?</td>
</tr>
<tr>
<td>WEL31202</td>
<td>2.4m Spot-SAT to Mesh</td>
<td>0.53</td>
<td>0.11</td>
<td>No</td>
</tr>
<tr>
<td>WEL31203</td>
<td>2.4m Spot-SAT to 2.4m Spot SAT</td>
<td>1.00</td>
<td>0.32</td>
<td>?</td>
</tr>
<tr>
<td>WEL31204</td>
<td>2.4m Spot-SAT to Mesh</td>
<td>0.54</td>
<td>0.10</td>
<td>No</td>
</tr>
<tr>
<td>WEL31205</td>
<td>2.4m Spot SAT to 2.4m Spot SAT, N.B. another Spot SAT directly in between</td>
<td>0.87</td>
<td>0.18</td>
<td>?</td>
</tr>
<tr>
<td>WEL31206</td>
<td>2.4m Spot SAT to Mesh</td>
<td>0.87</td>
<td>0.07</td>
<td>No</td>
</tr>
</tbody>
</table>

The results shown in Table 11 were somewhat disappointing with no definite valid resonances. An example is shown in Figure 40. The reason for the lack of resonances is not clear. This was the first set of tests run on SAT as opposed to conventional AT rockbolts. Potential reasons for this are further discussed in section 3.5.5.

PG312’s Loader Gate – 523-527RM

The test combinations are schematically illustrated in Figure 41 and the results are given in Table 12 below.
The test results shown in Table 12 illustrate a relatively successful test suit. The questionable results are from bolts tested in the orthogonal mode with mesh. The reasons for this are unclear but are in line with testing to mesh at Thoresby.

Table 12. RF results from PG312’s Loader Gate at 523/524RM and 527RM, Welbeck Colliery

<table>
<thead>
<tr>
<th>Test Ref.</th>
<th>Test Combination</th>
<th>Dist. Apart (m)</th>
<th>Res. Freq. (MHz)</th>
<th>Valid Resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>523/524RM</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WEL31207</td>
<td>2.4m ‘No. 6’ AT in strap to 2.4m AT Spot</td>
<td>0.70</td>
<td>1.16</td>
<td>Yes</td>
</tr>
<tr>
<td>WEL31208</td>
<td>2.4m AT Spot to 1.8m AT in strap</td>
<td>0.70</td>
<td>1.30</td>
<td>Yes</td>
</tr>
<tr>
<td>WEL31209</td>
<td>1.8m AT in Strap to 2.4m AT replacement for poor installation, potentially broken</td>
<td>0.75</td>
<td>1.30</td>
<td>Yes</td>
</tr>
<tr>
<td>WEL31210</td>
<td>2.4m AT replacement for poor installation, potentially broken, to 2.4m AT in strap</td>
<td>1.40</td>
<td>1.15</td>
<td>Yes</td>
</tr>
<tr>
<td>WEL31211</td>
<td>2.4m AT replacement for poor installation, potentially broken, to Mesh</td>
<td>0.63</td>
<td>0.28</td>
<td>?</td>
</tr>
<tr>
<td>WEL31212</td>
<td>1.8m AT in Strap to Mesh</td>
<td>0.80</td>
<td>0.46</td>
<td>?</td>
</tr>
<tr>
<td><strong>527RM</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WEL31213</td>
<td>2.4m potentially broken AT bolt in Strap to Mesh</td>
<td>0.37</td>
<td>6.8</td>
<td>Yes</td>
</tr>
<tr>
<td>WEL31214</td>
<td>2.4m potentially broken AT bolt in Strap to potentially intact 2.4m AT Spotbolt</td>
<td>0.66</td>
<td>6.3</td>
<td>Yes</td>
</tr>
<tr>
<td>WEL31215</td>
<td>Potentially intact 2.4m AT Spotbolt to Mesh</td>
<td>0.27</td>
<td>0.38</td>
<td>?</td>
</tr>
</tbody>
</table>

Figure 42(a) shows one of the typical valid results obtained for tests WEL31207-10 where the SWR and X troughs are coincident in terms of frequency and the X values have reached zero. Tests WEL31207 and WEL31210 for the 2.4m long bolt combinations recorded minimum resonant frequencies of 1.15MHz and 1.16MHz. This contrasts with the 1.30MHz recorded for the shorter bolt combination with the 1.8m long AT bolt installed for lifting. The higher frequency for the shorter bolt length is as would be expected. These results are shown in Figure 42(b) in comparison with the result from Thoresby, showing good correlation.

Figure 43(a) shows one of the typical valid results obtained for tests at 527RM. Tests WEL31213 and WEL31214 recorded minimum resonant frequencies of 6.8MHz and 6.3MHz respectively. This contrasts markedly with the 1.15MHz and 1.30MHz for the 2.4m/2.4m and 1.8/2.4m combinations at the 523/524RM. This frequency shift strongly suggests the presence of a broken bolt. One of these results, WEL31213, is tested in the orthogonal method with mesh. This appears to be giving a valid resonance whereas WEL31215 is the 2.4m long bolt at 527RM tested with mesh which is giving a questionable result. As indicated above with the testing at the 523/524RM, testing to mesh has given mixed results.
Figure 43(b) plots the result from test WEL31214 on the same graph as the results for the 523/524RM and those from Thoresby. Using the straight line fit of the Thoresby/Welbeck combined data it indicates a mean length of about 0.6m for the pair of bolts. It is therefore almost certain that one of them, probably the one in the disturbed roof, is broken. In order to calculate the actual length of the broken bolt, it is necessary to assume:

(i) that the other bolt is intact, at 2.4m long,
(ii) that the relationship between 1/(f) and mean length is valid for mean lengths less than 0.9m (the gradient calibration point,
(iii) that the Welbeck data obeys the Thoresby relationship throughout.

This gives \((2.4 + x)/2 = 0.6\) for the length, \(x\), of the broken bolt, where \(x = -1.2m\), which is not possible. Therefore one or more of the above assumptions must be invalid, i.e. both bolts could be broken (in-situ a total length of 1.2m) and/or one of the bolts is so short that the relationships (b) and (c) break down. Further measurements of pairs including the supposed intact bolt and also data from pairs of shorted calibration bolts would be required to resolve the question.

These results from the RF testing in PG312’s Loader Gate indicate that this NDT test method can successfully indicate bolt lengths, 1.8m, 2.4m and potential broken bolts <0.9m. As described in section 2.7.1 this was not feasible with the ultrasonics testing, due to either bending or the chamfered end at the distal end of the bolt. However the ultrasonic testing did indicate features at certain points along the length of the bolt and thus in combination with the RF testing it can now be reasonably concluded that these peaks are likely to be due bends not breaks.

Deep Soft South Return, 217’s Return Gate Junction

For this suite of RF tests in the Deep Soft the VIA Analyser was used with the standard lead set. All bolts tested with the straps had their nuts and plates removed and were in the centre of strap holes and thus definitely isolated.

Figure 44 shows the bolt locations and the test combinations and the results are summarised in Table 13.

The results shown in Table 13 were less successful than anticipated. Tests WELDS06-08 used the orthogonal method with the mesh. The highly corroded nature of the mesh meant it was difficult to get a positive connected circuit. As stated in Section 3.5.3 mixed success has been achieved with mesh and further research is needed to investigate the causes.

The reason for the lack of a valid resonance for WELDS05 is unknown, but the 2 bolts were 1.00m apart with a bolt between them, and both these factors could have affected the result.

Tests WELDS03 and 04 both involved connection to a SAT bolt. The reason for the lack of resonance here could be the same as the lack of resonances with those SAT bolts tested in PG312’s Loader Gate.

Valid resonances would be expected from tests WELDS01 and 02. The results from test WELDS02 are plotted in Figure 45(a). This indicates a weak resonance which is possibly valid. When plotted with the Thoresby data and PG312’s Loader Gate data, Figure 45(b), this indicates that the mean length for the 2 bolts is 2.4m and thus the two AT bolts numbered 1 and 2 are both intact. The result from WELDS01 also

39
indicates a probably valid resonance. It is less well defined than for WELDS02 and consequently the certainty that Bolt 3 is intact, which the weak frequency resonance indicates, is lower.

**Table 13. RF Test results from DS217’s Return Gate Junction in the South Return, Welbeck Colliery**

<table>
<thead>
<tr>
<th>Test Ref.</th>
<th>Test Combination</th>
<th>Dist Apart (m)</th>
<th>Resonant Freq. (MHz)</th>
<th>Valid Resonance</th>
</tr>
</thead>
<tbody>
<tr>
<td>WELDS01</td>
<td>Bolts 2 and Bolt 3 both 2.4m long AT bolts potentially broken</td>
<td>0.62</td>
<td>0.08</td>
<td>?</td>
</tr>
<tr>
<td>WELDS02</td>
<td>Bolts 2 and 1 both 2.4m long AT bolts potentially broken</td>
<td>0.62</td>
<td>1.20</td>
<td>Yes</td>
</tr>
<tr>
<td>WELDS03</td>
<td>Bolt 2 the 2.4m long AT bolt potentially broken and Bolt 4 the 2.4m long potentially intact SAT spot bolt</td>
<td>0.67</td>
<td>0.17</td>
<td>?</td>
</tr>
<tr>
<td>WELDS04</td>
<td>Bolt 1 the 2.4m long AT bolt potentially broken and Bolt 4 the 2.4m long potentially intact SAT spot bolt</td>
<td>0.21</td>
<td>0.18</td>
<td>?</td>
</tr>
<tr>
<td>WELDS05</td>
<td>Bolts 2 and 5, both 2.4m long AT bolts potentially broken (Bolt 5 in adjacent strap)</td>
<td>1.00</td>
<td>0.02</td>
<td>?</td>
</tr>
<tr>
<td>WELDS06</td>
<td>Bolt 5 the 2.4m potentially broken AT bolt to mesh, BUT Mesh highly corroded, poor connection, hence use of the crocodile clip to ‘W’ strap</td>
<td>0.10</td>
<td>?</td>
<td>No</td>
</tr>
<tr>
<td>WELDS07</td>
<td>Bolt 5 the 2.4m potentially broken AT bolt to mesh, Mesh highly corroded, poor connection, mesh then re-snipped and a fresh surface exposed for positive connection with terminal block</td>
<td>0.10</td>
<td>?</td>
<td>No</td>
</tr>
<tr>
<td>WELDS08</td>
<td>Bolt 1 the 2.4m potentially broken AT bolt to mesh, BUT Mesh highly corroded, poor connection, hence use of the crocodile clip to ‘W’ strap</td>
<td>0.10</td>
<td>1.85</td>
<td>?</td>
</tr>
</tbody>
</table>

In order to determine if the 3 tested bolts were intact a decision was made to remove the bolts from the roof via overcoring. A 3 inch core barrel system was employed in order to do this. However on the first bolt attempted, No. 3, the core barrel was unable to core further than 0.6-0.8m due to a bend in the bolt. Consequently the 5 inch core barrel system was employed for the No. 1 bolt. Although the drilling was slow it was successfully recovered. **Figure 46** shows a photograph of the recovered bolt which, although slightly bent was, intact.

These results from the RF testing in the Deep Soft, as in PG312’s Loader Gate, indicate that this NDT test method can successfully indicate intact bolt lengths. As described in section 2.7.2, for the same bolts in the Deep Soft, this was not feasible with the ultrasonics testing, (due to either bending or the chamfered end at the distal end of the bolt). In these cases none of the probes gave definitive reflections indicative of the bolt end, various peaks were observed between 0.40 and 1.00m for both bolts, and it can now be reasonably concluded that these peaks are likely to be due bends not breaks.
3.5.6 Analysis of the Welbeck and Thoresby Field Data

Where anomalous results were observed at the Welbeck and Thoresby sites they were generally characterised by a lower-than-expected resonant frequency, as determined by the minimum (trough) in the SWR versus frequency plot. This was often accompanied by a minimum in the Z (total impedance) plot at a different frequency, for example Figure 40. (In an ideal system, the SWR and Z minima would occur at the same frequency).

Laboratory tests have shown that these effects may be due, in the field, to the presence of low electrical resistance (leakage) between the tendons under test.

A simple electrical circuit was constructed to simulate a tendon-pair with a resonant frequency of approximately 3000 kHz and a series (internal) resistance of 55 Ohms.

The response of the VIA Analyser when connected to the simulator using the standard leads used on site was the SWR and Z minima were well defined and at practically the same frequency, with X (reactive component of Z) close to zero and R (real or resistive component of Z) about 60 Ohms, as expected. Figure 30(a) illustrates typical field result from the datum bolts at Thoresby.

When progressively lower resistances were connected across (in parallel with) the terminals of the simulator circuit, the SWR minimum was first broadened at 15,000 Ohms, then distorted from its normally symmetrical shape when an 82 Ohm resistor was in parallel. Eventually it was made practically undetectable, Figure 47(a) at 10 Ohms in parallel and formed an anomalous broad minimum at a lower frequency, Figure 47(b) with 2.2 Ohms in parallel.

Corresponding changes in the Z response were also observed, again producing a low-frequency minimum for small parallel resistances.

These effects on SWR and Z were small, provided the parallel resistance was large compared with the internal resistance of the system being measured. The internal resistance of tendon pairs depends on the geometry and the dielectric properties of the intervening material. Measurement of this parameter requires access to the far ends of the pair, which is not normally practicable. Indications and theoretical estimates suggest that it is at least 100 Ohms and probably much larger.

This means that the effect of a given parallel resistance is likely to be greater for a real tendon pair than for the simulator.

Further work is required to test the above explanation for anomalous results and examine ways of overcoming the problem when it is encountered. For example, direct measurement of the leakage resistance between tendon pairs or modifications to the instrument to allow for a range of internal resistances. However the availability of the X, R and Z parameters on the new instrument means that it is now considerably easier to differentiate between valid and invalid results when using the RF bolt integrity method.

3.6 SUMMARY OF RF RESEARCH

Success had been achieved with the RF NDT system under the ECSC project AB 149 (ECSC 2000[1]), with the original instrument, (MFJ-259). However this had been very
limited in Coal Measures strata and it was suspected that the probable reason was that the instrument was unable to read at the lower frequency ranges which were characteristic of tendons in this strata type. The bottom range of the MFJ–259 was originally 1.4MHz. As attempts to reduce the range of the MFJ-259 were only partially successful, a new instrument was specifically built to read to 0.2MHz.

The new LRF instrument appeared to work well with promising results recorded from datum bolts of different lengths up to 2.4m installed within a coal mine. However there were inconsistencies in results and it was determined that the instrument was sensitive to lead length. Modifications were made and further tests conducted, but results were still inconsistent.

At the time difficulties were being experienced with the LRF a new instrument became available that had potential advantages over the LRF, it had a frequency range as low as that of the LRF but also was able to display the SWR, record the X, R and Z components and was potentially likely to be more stable than the LRF. The new instrument proved to work well although it has only had relatively limited use due to it being obtained somewhat late in the course of the project.

At Thoresby Colliery the RF system worked well and the LRF and new instrument were both able to determine the length of certain 2.4m long original AT rock bolts. However the system did not work on all the original in-situ bolts tested. The probable cause was considered to be a change in ground conditions along the gate road with strata water affecting the results. However when bolts were specifically installed in the area of poor results positive test results were obtained. The reason is still unclear, but is probably associated with changes to the conductivity of the rockbolt encapsulating resin with time in wet conditions.

RF testing was also undertaken at Welbeck Colliery in a gate road where shear movement was known to have broken bolts approximately 0.6m into the roof at specific locations. The new instrument was able successfully to determine the length of existing 2.4m long AT bolts and 1.8m long AT lifting bolts. The results correlated very well with those obtained from Thoresby. The instrument was also able to confirm the presence of a broken bolt. At a separate site at Welbeck the RF instrument has indicated that potentially broken bolts were likely to be intact.

Following the work under this project the RF testing has now been successfully undertaken within the following rock environments:

(i) Slate,  
(ii) Salt,  
(iii) Limestone,  
(iv) Granite,  
(v) Coal Measures limestone,  
(vi) Coal Measures sandstone,  
(vii) Coal Measures siltstone.

The RF system has now reached a stage where it is a practical tool to locate broken reinforcement in European mines between tendon pairs, though it suffers from the disadvantage of requiring electrical isolation between the tendon pairs tested.

However, despite this success there have also been some inconsistent results, notably when testing SAT bolts and when the mesh is used as the ‘ground plane’ in the orthogonal test geometry. Laboratory test work has indicated a potential cause for
these inconsistencies at some sites. Further work is required to test the potential explanation for the anomalous results and examine ways of overcoming the problem when it is encountered.
4. INSTRUMENTATION DEVELOPMENTS

4.1 INTRODUCTION

The remote reading tell tale system developed by RMT was described in a previous HSE report, (HSE, 2003).

The principle of remotely reading dual height transponder units indicating roof dilation has been further developed to suite a wider range of specific requirements:

(a) Remote reading dual height tell tale system with a portable read out unit, section 4.2
(b) Dual height blast proof tell tale/extensometer for local reading, section 4.3
(c) Upgraded dual height blast proof tell tale/extensometer for local reading incorporating a ‘flying PCB cable’, section 4.4
(d) Four wire remote reading extensometer, section 4.5

The other main instrumentation development was with respect to instrumentation for detection of roof shear. These developments are described in Section 4.6.

4.2 REMOTE READING DUAL HEIGHT TELL TALE SYSTEM WITH PORTABLE READ OUT UNIT

A portable readout unit has been constructed and tested for manual reading of remote reading telltale systems underground. Figure 48(a) shows the remote reading telltale unit and Figure 48(b) the conventional set up. Figure 48(c) shows the battery powered readout box that was initially developed to substitute for the underground interrogation unit and physical link to a surface computer.

The unit was tested with an array of remote reading telltales installed in an Indian room and pillar coal mine during pillar extraction, Figure 48(d). The system allowed connection to the transducers in different geometries and readings to be taken from different points. The user recorded the data manually which was then input into a specially tailored spread sheet on a surface PC for analysis and interpretation.

The system operated well and the telltales continued to provide very accurate and stable readings of roof deformation well into the goaf area, providing valuable data on the caving characteristics. An example of an installation in ground with very little movement is shown in Figure 49(a), which indicates only slight movement (<1.5mm) occurring in the strata during the period of monitoring. The last reading taken on 08/04/03 shows indications of movement, but the instrument was overrun by the goaf before any further readings were taken. The rate of movement before failure was less than 1mm/hour up to the point that the instrument was monitored. The junction was abandoned approximately two days before the initial failure occurred. It will be seen that the repeatability and consistency of read-out data was excellent.

Figure 49(b) indicates the cumulative movement for the remote reading telltale no. 11, which was installed in the junction of 17 Level, B Rise. This set of data is one of the most significant as the readings were taken at an increasing frequency towards the end of the instrument’s life, up to the point of complete roof failure. Figure 49(c) indicates the last 48 hours of measurement before failure occurred. It shows movement occurring above and within the bolted height. Within the bolted height (AA), 28.3mm
movement occurred. More movement (75.9mm) occurred above the bolted height (BB). The calculated rate of dilation, in the upper strata peaked at 15.9mm/hour, approximately 2 hours before complete failure of the junction.

**Figure 50** shows the cumulative total roof movement and roof movement rates from the telltales from which significant roof movements were recorded. These instruments, and in particular instruments 11 and 12, were read at reduced intervals as the goaf approached the position of the instruments and read up to the point that roof failure occurred.

This exercise has demonstrated the feasibility of using a hand held, battery powered readout with the remote reading telltale transponders for intensive scientific studies in coal mines and the excellent stability and accuracy attainable. It has also provided valuable data on rates of roof dilation prior to failure that can be used when programming a system to give warning of roof failure. The experience suggests that it would be worthwhile investigating obtaining permission to use the system under EAWR19 for UK coalmines.

### 4.3 DUAL HEIGHT ‘BLAST PROOF’ TELL TALE/EXTENSOMETER FOR REMOTE READING

#### 4.3.1 Mark 1

The ‘blast proof’ version of the remote reading tell tale has the dual height transponders bonded up the borehole. The dual height visual indicators are not present and the instrument is flush or slightly recessed into the hole. Plug connections for the cable are present at the underside of the instrument. Thus, until the advance of the face is sufficient distance from the instrument such that damage due to flying rock is past, the instrument can be read by plugging in the cable each time a reading is required. Eventually the cable can be left permanently plugged in. **Figure 51** shows a schematic cross section and photograph of the instrument.

This dedicated portable reader is plugged in to the underside of the sensor body (or via a short length of cable) and the two readings, AA and BB, noted together with the location in the tunnel for later analysis. The six and five digit readings provided by the portable reader are converted to millimetres by dividing by standard factors. **Figure 52** shows a photograph of the readout unit and its use underground.

In the UK 7 of these instruments were successfully installed in a jointed hard rock tunnel, **Figure 52**, in order to confirm that the 4m long rockbolts were providing the required inter-block stability. The nominal heights for anchors were at 8.0m, and 3.5m. However the top anchor was not always installed to the full 8.0m due to length restrictions of the drill boom drifter when used with the two 4m long drill rods. The typical hole length was 7.6m. Each sensor element has a working range of 40 mm. **Figure 53(a)** shows a typical trace of recorded data over the 8 month period.

As the sensor elements are based on an inductive principle and provide a repeatable accuracy of ±0.02 mm where the ambient temperature is constant, or within ±2°C; for increased accuracy and automatic equipment check it is normal to measure a ‘fixed’ or dummy sensor located in the region of the other free moving sensors. Temperature variation can then be compensated for in the post analysis. This was undertaken in the
hard rock tunnel and the results shown in Figure 53(b). This shows ‘stability’; no temperature compensation has yet been required.

The disadvantage of this version of the blast proof dual height extensometer is that the PCB board normally on board the tell tale itself is incorporated within the read out unit. Thus the readout unit is interrogating an inductance from the instrument rather than a frequency. The inductance is sensitive to lead length and thus care must be taken to ensure the lead length remains constant in order to avoid having to make compensatory adjustments in the analysis.

This same instrument was also installed in a platinum mine in South Africa. Initially 2 instruments were relatively successfully installed, but didn’t show any movement over the monitored period. Further instruments were also installed in a platinum mine, however these installations highlighted potential problems with the instrument:

(a) Due to the high quantity of dust in the atmosphere, the bobbins could become stuck inside the body of the instrument as a result of insufficient clearance.

(b) The glue used in the hole was not viscous enough and hence could run out the hole if the packing at the hole mouth was insufficient.

(c) The ferrules were difficult to crimp up the hole and if they stuck out too far they were damaged by the blast.

Consequently a ‘Mark 2’ instrument was developed to overcome some of these problems.

4.3.2 Mark 2

The Mark 2 instruments were installed in a platinum mine in South Africa and in the hard rock tunnel in the UK. Despite the modifications there were still problems which required attention.

(i) A build up of dust still caused the bobbins to become stuck despite a spring return on the bobbin anchors,

(ii) There were still concerns about the method and position of fitting the extensometer into the hole,

(iii) The installation tool for the anchors and extensometer body was too soft.

These aspects were investigated and the MARK 3 instrument along with a new readout box were developed, as described in Section 4.3.3 below.

4.3.3 Mark 3 Upgraded Dual Height Blast Proof Tell Tale/Extensometer for Remote Reading – Incorporating the New Readout Unit with ‘Flying PCB Lead’

The Mark 3 blast proof extensometer was developed to take into account the problems identified with the Mark 2 version.

Figure 54(a) and (b) shows the installation trial of the new extensometer body in the laboratory. A new more viscous resin was sourced and this is now pumped through a tube that runs up the centre of the instrument and exits to the section of hole for
bonding, thus preventing pinching of the tube which had caused problems in the past. The top and bottom sealing rings have been designed to accommodate different hole diameters, (Figure 54(b)).

Figure 54(c) shows a photograph of the new readout unit, the RMT Portable Reader RRTT-1342-PR. The difference with this system is that the PCB board can be plugged into the instrument via a short length of cable, which in turn can be plugged into the readout box. Thus the instrument can be used for reading multiple blast proof extensometers or remote reading tell telltales.

By the end of this Project the Mark 3 version of the blast proof extensometer had been developed and produced but not installed underground.

4.4 FOUR WIRE REMOTE READING EXTENSOMETER

A 4 wire remote reading extensometer has also been developed using the same principles as the dual height remote reading tell tale/extensometer. Figure 55 shows a schematic cross section. This instrument has not yet been tested in an underground mining environment.

4.5 INSTRUMENTATION FOR DETECTION OF SHEAR

Following the fall of ground at Welbeck Colliery in PG312’s Loader Gate, the detection of roof shear became an urgent priority. The devices currently available in the UK for this purpose are as follows:

(a) Sentinel rock bolt. This device comprises a standard rockbolt with a groove machined along it length, containing fine insulated wires which have sufficient ductility not to fail until the rockbolt itself fails. As the wires are terminated against the rockbolt at different heights, measurement of the electrical continuity between the bolt and wire will indicate whether the bolt has been broken in situ. The device will therefore register bolt failure through any combination of shear, bending and tensile loading. The smaller cross-sectional area of the bolt compared with a standard bolt will ensure that the Sentinel bolt fails before a standard rockbolt in the same position would fail.

(b) Visual/manual examination of an open borehole. This can be undertaken using either a special probe rod to “feel” for lips in the hole or with a borescope. Both methods are highly manpower intensive, subjective and the borescope is also a relatively expensive instrument.

(c) Roof shortening meter. This is a simple and low cost wire operated extensometer which can be mounted between 2 rockbolts and indicates the amount of width reduction in the roof between the anchorage points. Like the Sentinel bolt, the device was developed and has been available for many years but has not been taken up in any quantity by the UK industry.

During the course of this Project, a number of alternative shear detection methods were considered. These included the “shear tale”, which was developed and has been used on a limited basis in Germany, and the use of an optical borehole shuttle.
The “shear tale” comprises 2 or more concentric, insulated steel tubes, point anchored in a roof borehole. Measurement of electrical continuity between successive tubes will indicate whether shear in the borehole has caused any of the tubes to come into contact. The main problems with this device are envisaged to be that it would need to be mounted in a relatively large diameter hole if significant levels of shear are to be detected and there is no indication of the position of the shear within the hole. It is feasible that height information could be obtained by modifying the instrument with successive insulated steel tubes at different heights but it is envisaged that the instrument would then become expensive to manufacture and difficult to read.

DMT of Germany are currently developing an Intrinsically Safe Borehole Shuttle containing a camera and recording equipment, which has the potential of automating the process of examining borehole condition. However the disadvantages of the system are that it will be an extremely expensive instrument and it will only run in a borehole with minimum shear. Any significant shear will block the hole for the instrument.

A further problem highlighted by the Welbeck fall and subsequent surveys throughout UK Coal is that, when telltale holes completely close through shear, the wires become trapped and further roof dilation above the shear horizon may not be registered.

A number of ways of addressing this problem have been considered and it currently appears that the most feasible method it to use a wire with a helical spring located immediately below the spring anchor. When the telltale is installed, if the wire is pulled downwards gently, the spring provides an indication of whether or not the wire is trapped (if no “give” is felt, then the wire is trapped). Two alternative designs of this type of telltale modification are currently being tested by UK Coal. One uses a completely separate additional wire and the other incorporates the helical spring on the upper anchor wire.

Reports to date indicate that both systems work well, although the additional wire system may have the advantages of allowing the wire to be extended to a more accessible level in high roadways and there is less chance of the telltale readings being affected by repeated use. Both modified systems are now available commercially, the additional wire system being the more expensive. Currently, 3 mines are using the spring on the upper indicator anchor system and 1 mine is using the additional wire system.

Current conclusions from these investigations are that the problems associated with telltale readings and bolt failure due to roof shear can be addressed by appropriate application of a combination of the following instrumentation and management procedures:

(a) Greater use, and further development as required, of the risk assessment procedures devised after the fall in Welbeck PG301’s Loader Gate in 1996, particularly in the Parkgate seam. These have continued to be applied at Thoresby colliery over the last 7 years.

(b) Application of a row of Sentinel bolts alongside telltales in areas identified during the risk assessments as at risk of broken bolts.

(c) Wide application of telltales with an additional spring to identify roof shear which could compromise the telltale readings.
(d) Incorporation of the above in a proper way into the Managers Scheme for the Assessment of Ground Control Measures.
5. RISK ASSESSMENT

5.1 INTRODUCTION

The purpose of a risk assessment is to identify areas of significant risk in relation to roof or rib instability so that the risk can be reduced by applying appropriate additional support or reinforcement. A methodology for this has been developed under previous ECSC projects, AB838 (ECSC 1997), AB149 (ECSC 2000[1]), AB147 (ECSC 2000[2]), and AB 058 (ECSC 2003[1]).

One of the aims of the work within this project was to integrate the NDT survey work with risk assessments. This was demonstrated for PG42’s Loader Gate at Thoresby Colliery. The site details have been described in Section 1.2. The NDT work was carried out during the recovery of the gate road in order to assess the original support system, Section 2.2. This was followed by the more conventional risk assessment to determine roadway stability and subsequent risks associated with face retreat, Section 5.2.

5.2 PG42’S LOADER GATE RISK ASSESSMENT

5.2.1 Introduction

A risk assessment was carried out for 42’s Loader Gate in the Parkgate Seam at Thoresby Colliery. The purpose of the assessment was to identify areas of significant risk in relation to roof or rib instability so that the risk could be reduced by applying appropriate additional support or reinforcement. The site details have been described in Section 1.2.

At the request of the mine, the risk assessment procedure, as developed for the earlier panels at the mine and described in previous ECSC reports (3, 5), was applied to PG42’s Loader Gate. However modifications were required to allow for the support procedures used in PG42’s Loader Gate in the light of its reuse and the outcome of the NDT measurement:

(a) Additional roof support was placed on recovery in the form of 5 x 1.8m rockbolts at 1m spacings installed between the “W” straps containing the original 7 x 2.4m long AT roof bolts placed on drivage, with new mash to replace the corroded original. Where the roadway passed beyond the sandstone channel which dominated the roof up until the 864MM, following a short section supported by steel, the additional bolts installed were increased in length to 2.4m. It was necessary to decide whether these bolts should be considered to be additional to the original bolting pattern or whether they should be considered to be replacements. As described in Section 2.2 above, the NDT survey in collaboration with bolt overcoring had indicated that a significant proportion of the bolts were likely to be intact, fully encapsulated and consequently performing ‘as new’. This was confirmed by the generally very good roof condition and low movements recorded on the telltales on recovery of the gate road. Hence it was decided to consider these new bolts as additional. With respect to the risk assessment this additional support was classified as ‘Category B’ rather than ‘Category A’, thus downgrading
the amount of risk reduction on their installation to compensate for the remaining uncertainties of the quality of the original bolting.

(b) Rib support in the form of ‘Para’ mesh was installed in order to replace the corroded original steel mesh. This was held to the ribside by 1.5m long GRP bolts on both the face and solid side. These bolts were not considered to be additional as they were only 1.5m long. Where the original ribbolts were damaged they were replaced. With respect to the risk assessment the rib support was considered to be ‘as original’.

(c) Triple Height Telltales had been installed where the additional bolts installed on roadway recovery were 1.8m long. The telltale anchors were thus A = 1.5m, B = 2.1m and C = 5.0m. This had to be considered in the analysis which previously had only catered for dual height telltales. For the risk analysis, the new triple height readings were converted back to dual height readings by adding the new A and B readings for an equivalent dual height A, and the C reading was treated as equivalent to the dual height B. It was not possible to split the original A reading into an A and B reading as it was not known in which portion of the bolted height the movement had occurred. Consequently the action levels used in the analysis were based on the combined A and B reading from the triple height telltales and were:

(a) Low Risk A = <20mm, B = <10mm,
(b) Medium Risk A = >19mm or B > 9mm and not stable, and
(c) High Risk A = > 24mm, B = >24mm.

5.2.2 Risk Assessment Survey System

The same survey method and booking sheet style was adopted for PG42’s Loader Gate risk assessment as that applied to previous Parkgate Seam panels at Thoresby. Figure 56 shows the modified booking sheet for PG42’s Loader Gate.

Information collected on the sheet includes support condition, monitoring data, the presence of specific identified risk factors and a roadway condition rating. Figure 57 defines the condition rating values used to assess roadway condition in PG42’s Loader Gate on the booking sheets, recorded as R1 to R6 for the roof and S1 to S6 for the sides.

5.2.3 Risk Assessment Logic

Separate logic routines have been developed for assessing the risk of falls of material from the roof and from the ribs. These are described in the form of a flow chart or logic tree as shown in Figure 58. Essentially, the assessment logic can be divided into three areas of assessment :-

(a) Visual Assessment (for roof and ribs)
(b) Roof Deformation (Monitoring) Assessment (for roof)
(c) Potential Risk Assessment (for roof and ribs)

A risk classification is determined for the roof and ribs for each survey zone based on the risk ratings assigned in Sections A) and B), whilst, separately, an assessment of the potential for the risk level to increase in the future is made based on the rating assigned in section C).
Data from the survey is processed using an Excel computer spreadsheet which has been developed to apply the logic described above and in Figure 58.

5.2.4 Risk Rating and Classification of Risk

For each risk factor identified a risk rating is assigned. Risk Rating values range from 0 to 2 where 0 = LOW risk, 1 = MEDIUM risk and 2 = HIGH risk. These values are then assessed collectively, as described in the logic tree, to give an overall risk level for the zone surveyed.

The next stage of the risk assessment is to evaluate the controls applied. In practical terms this means assessing the effectiveness of any additional or remedial support measures put in place to control the identified hazard. Appropriate remedial support is discussed in section 5 below. The risk assessment developed for Thoresby allows the surveyed zones to be re-classified and given a Residual Risk Level, which takes into account the extra controls provided, by the additional support.

5.2.5 Appropriate Remedial Support

Due to the wide variety of types of remedial support available, advice was not specifically given in this analysis. It was considered that the appropriateness of remedial support for a particular risk should be ascertained by the Colliery Rockbolting Engineer. The engineer should visit high risk areas, confirm the assessment by detailed examination of the conditions and available monitoring data and determine the type and density of remedial support based on this secondary assessment.

For guidance the tables shown in Figures 59 and 60, were produced in consultation with Thoresby Colliery staff. These tables contain a list of potential remedial support measures and relates these, in terms of their support effectiveness, to the range of hazards identified.

5.2.6 Results

Figures 61 and 62 show the risk assessment output results for 42's Loader Gate roof and ribs respectively.

**Roof**

The results for the roof indicated that based on the drivage support only and ignoring the benefits of the additional support installed on roadway recovery, 2 zones (4%) were assessed as HIGH RISK, 22 zones (44%) as MEDIUM RISK and 26 zones (52%) as LOW RISK. When the additional support was included this indicated that 0 zones (0%) were now classified as HIGH RISK, 2 zones (4%) as MEDIUM RISK, and, 44 zones (96%) as LOW RISK. Further additional support may be required for the zones classified as medium risk.

The additional support was classified as category B rather than A, reducing the amount of reduction in risk to compensate for the uncertainty of the quality of the original bolting, following flooding and subsequent rehabilitation.

For face retreat 22 zones were identified as likely to develop an increased risk due to the expected front abutment effects. Areas of increased width and zones where the B telltale was indicating greater than 10mm of movement constitute these zones. The
small number of zones indicating potential for ongoing deterioration is a reflection that the majority of the gate roof comprised massive sandstone.

**Ribs**
The rib risk assessment was based mainly on observable condition. General rib condition, rib deformation, shelving and upper rib overhang were included amongst the features in 42’s Loader Gate ribsides that contributed toward increased risk. Overall, the rib zones were 24% low risk, 42% medium risk and 34% high risk.

In general, the left hand rib (face side) presented a higher risk for rib control problems than the right hand rib. This was considered to be primarily related to the orientation of the gate road to the main cleat of the Parkgate Seam.

Looking at the potential effects of face retreat and interaction influences on rib condition, there were a high number of zones classified as HIGH RISK that were also identified as having a HIGH potential for the risk level to increase. This included 31 zones where the left hand rib had a HIGH potential for increased risk and 2 zones where the right hand rib had a HIGH potential for increased risk.

**5.2.7 Conclusions and Recommendations**

It was recommended that roof and rib zones classified by the risk assessment as HIGH RISK should be inspected by the Colliery Roofbolt Engineer to determine the level of additional support required in these zones. The tables in Figures 59 and 60 offer guidance in determining the type and level of additional support that would be appropriate to each specific hazard feature identified.

The zones identified as having a high potential for increased risk during face retreat should also be inspected by the Roofbolt Engineer before these zones were influenced by the face front abutment.

In view of the potential for support problems involving the ribs it was recommended that rib support measures were made available for use underground at short notice. This could include a readily available supply of props that could be set along high risk ribsides.

**5.3 GROUND CONTROL RISK ASSOCIATED WITH STEEL AND MIXED SUPPORT ROADWAYS IN UK COAL (MINING) LTD. COAL MINES**

**5.3.1 Introduction**

Following a fall of ground at Rossington Colliery, Rock Mechanics Technology Ltd (RMT) were asked to examine the ways in which roadways supported by steel alone or a mixture of rockbolts and steel were designed and managed by UK Coal, and, if appropriate, to develop improvements to the system to reduce ground control risk.

In order to examine the ways in which these roadways were designed and managed in UK Coal mines, data on current practices were obtained from selected mines. UK Coal Mining Ltd. utilise a range of steel support types of differing profiles. In Section 5.4.2 the current support types and their application are reviewed in conjunction with the current design and management practices.
To compare the relative ground control risks associated with each type of steel support, records of previous fall of ground incidents were examined, Section 5.3.3. It was envisaged that this would allow the identification of particular support types, support/reinforcement combinations and/or design or management practices which may be associated with increased risk.

Using data from Sections 5.3.2 and 5.3.3 it was then possible to identify potential areas where risk reduction could be undertaken, this is described in Section 5.3.4.

In Section 5.3.5 the findings from the study are listed and recommendations made to reduce ground control risks in steel and mixed support roadways.

5.3.2 UK Coal Mining Ltd. Steel Supports and their Associated Design Practices

Current Steel Support Systems

UK Coal uses ‘H’ section steel supports in roadways at all its mines, in the form of either arched or rectangular profiles. There is a range of variants within each form, as illustrated in Figure 63.

UK Coal have rationalised the number of support variants purchased to seven standard types. However many existing roadways contain non-standard supports and some of these are still installed from existing stocks. The seven standard support types are summarised in Table 14. As part of the rationalisation process two sections have been standardised upon:

(a) 152mm x 127mm (6’’x 5’’),
(b) 204mm x 152mm (8’’ x 6’’).

<table>
<thead>
<tr>
<th>UK Coal Standard Number</th>
<th>Support Type</th>
<th>Dimensions (m)</th>
<th>Support Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Standard Delta</td>
<td>4.727 wide x 3.40 high</td>
<td>Flat top Delta</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(15.5’ x 11’)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Standard Delta</td>
<td>4.700 wide x 3.00 high</td>
<td>Flat top Delta</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(15.5’ x 10’)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Standard Delta</td>
<td>6.100 wide x 3.00 high</td>
<td>Flat top Delta</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(20’ x 10’)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Standard Delta</td>
<td>5.739 wide x 3.003 high</td>
<td>Cambered Delta</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(19’ x 10’)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Standard Arch</td>
<td>5.791 wide x 3.810 high</td>
<td>Double Radius Arch</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(19’ x 12’)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Standard Arch</td>
<td>5.180 wide x 3.660 high</td>
<td>Semi Circular Arch</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(17’ x 12’)</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Standard Arch</td>
<td>5.755 wide x 3.810 high</td>
<td>Semi Circular Arch</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(19 x 12.5’)</td>
<td></td>
</tr>
</tbody>
</table>

The Application of Steel and Mixed Support in UK Coal Mine Roadways

Arch supports are predominantly used in long life main trunk and district access roads, including cross measure drivages. Rockbolts are often used as additional support to assist in maintaining the arch profile.

Delta profile supports are usually used for shorter life in seam drivages, almost always in conjunction with a full pattern of rockbolts, (mixed support). This form of support is
frequently used in the transition zone where bolted support is being established at the
start of gate road drivages. Mixed support is also used on faceline drivages where
support by bolts alone can be more difficult than in the corresponding gate roads.

Steel support is also used by UK Coal as remedial support in rockbolted roadways.

**Previous Design and Management Practice for UK Coal Mine Steel Supported
Roadways: The Regulations**
The design and management of all steel supported roadways, and indeed rockbolted
roadways, need to comply with the Mines (Control of Ground Movement) Regulations
1999; an Approved Code of Practice and Guidance covering these regulations has also
been published (HSE, 1999).

Subsequent to the publication of the regulations, DMCIAC Guidance was published for
the design, installation and use of free standing support systems in coal mines (HSE,
2002).

In October 2002 UK Coal issued a company directive on the Management Process to
be adopted to ensure Compliance with the Control of Ground Movement Regulations,
(UK Coal 2002). This document set out guidelines on Ground Control Assessment,
Design Documentation, Ground Control Rules, and Assessing the Adequacy of the
Ground Control Measures.

A review of the above regulations indicated that:

(a) The control of ground movement in mines, the Mines (Control of
Movement Regulations) 1999 provides guidance on the design and
management of roadway support, however there are no specific
guidelines with respect to the design and management of mixed support
roadways.

(b) The DMCIAC guidance on the design, installation and use of free
standing supports systems indicates that a more detailed ground
assessment survey may be necessary for mixed support but does not
give additional guidance on their management and monitoring.

(c) A brief review of UK Coal Mining Ltd. procedures indicated that Mines
had adopted procedures which were intended to comply with the Mines
(Control of Ground Movement) Regulations 1999.

(d) The ground assessment and design document for steel supported
roadways tended to be detailed, with comments made on all the factors
indicated in the Mines (Control of Ground Movement) Regulations 1999
for detailed assessments, despite the DMCIAC Guidance indicating this
documentation could be brief.

(e) In practice steel support design, especially for arched shaped roadways
is effectively a selection based on operational requirements and
experience. Design calculations cannot realistically be undertaken,
because the loads which will be imposed on the support system are not
known.
In roadways with mixed support the steel is invariably considered to be the system of support meeting the requirements of the Mines (Ground Control Movement) Regulations 1999. However in reality the rockbolts may be providing the ground support, especially in roadways with a rectangular or shallow arch profile.

Existing UK Coal Mining Ltd. practice consisted of the use of roadway inspection under MASHAM regulations as the monitoring scheme in conjunction with steel supports. This was also the general practice where steel supports and rockbolts were used together. However a recent UK Coal Company Directive now requires a telltale monitoring scheme to be used where rectangular section roadways were used with steel support and rockbolts. The use of flat top and cambered delta arch supports in shallow arched roadways was not included in this requirement.

5.3.3 Relative Ground Control Risks For Steel and Mixed Support Systems

The aim of this part of the study was to compare the relative ground control risk associated with each standing support option, used with or without supplementary bolting, in order to identify if any support options were associated with unacceptably high risk.

The frequency with which roof falls occurred for each standing support type, taking into account the numbers used, relates to the residual ground control risk, because risk reducing procedures (normally roadway inspection under MASHAM rules) would have been in place when the falls occurred. Therefore, if the number of roof falls associated with a given support suggested that this risk was unacceptable, either a more effective management procedure was required to reduce the risk of falls, or the support itself must be modified in some way to make it safer. If neither of these could be achieved, then the support type in question should not be used, or its use should be restricted to defined circumstances under which it could be used safely.

The relative risk for bolted support was a useful benchmark, as it was the most widely used support system. Bolting as sole support within UK Coal was always used with a roof movement monitoring system complying with the DMC/AC Guidance Document (HSE, 1996). The associated level of risk for this system was currently accepted, and must therefore be considered ‘low’. Comparing the frequency of roof falls for bolts only with the steel support/mixed support options, it would be possible to identify which, if any, support options had an unacceptably high risk.

Coal mine roof fall records were examined in order to identify the support type in use. In practice, comprehensive records existed only from 1996. Available information was examined back to 1987. Estimates of the percentage of mine roadway supported by each type had to be made from this date in order to compare the relative risks.

Appendix 1.1 tabulates the roof fall data collected and also goes on to give an analysis of the roof falls for the different roof support systems.

The roof fall data analysis indicated that:
(a) Rectangular profile supports (including delta arches), used with or without supplementary bolting, were associated with a significantly higher ground control risk than either D shaped arch profile steel support, or rockbolt supported roadways.

(b) Particular attention should be paid to long term deterioration of roadways with steel support. Roadway repair work, or geometry change such as dinting or thirling are also associated with increased ground control risk. Faceline drivages appeared to be associated with a higher ground control risk than other roadways, but this may be due to the frequent use of rectangular profile steel supports on facelines.

(c) Improvements in the management of rockbolt monitoring schemes and the development of flexible bolts as additional reinforcement had probably made a significant contribution to the reduced ground control risk demonstrated by recent statistics for rockbolt support.

5.3.4 The Potential For Ground Control Risk Reduction In Steel Supported Roadways

Using data from Sections 5.3.2 and 5.3.3 it was then possible to identify potential areas where risk reduction could be undertaken.

The Strength and Deformation Behaviour of Steel Supports

The main differences between D shaped arch and rectangular profile supports are that the former are significantly stronger, and exhibit more progressive failure behaviour. Arch systems retain significant strength at high deformations, giving obvious indication of the need for remedial support/repair. With overloaded rectangular profile supports there is an increased risk of relatively sudden collapse, possibly with limited visual warning as was indicated in the review of roof fall data. These differences explain the increased ground control risk associated with rectangular profile supports, compared with D shaped arches, with the current roadway inspection monitoring procedures.

It was unclear what proportion of fall incidents involved cambered profile supports. However strength testing had shown that cambered profile supports were no stronger than the equivalent flat topped section without a very high end restraint. In practice this was unlikely to be present, as it is difficult to accurately form the required roadway profile. It was therefore considered than the cambered support should be classed with rectangular profile rather than arched types.

One important point raised during the mine visits was that in mixed support roadways the steel was invariably considered to be the system of support meeting the requirements of the Mines (Ground Control Movement) Regulations 1999. However in reality the effect of roofbolts in rectangular profile roads was to reduce roof deformation and maintain a flat roof. In this situation the steel supports may not be packed tight to the roof. Subsequent failure of the rockbolt support may result in dynamic loading of the steel support. Even without dynamic loading, subsequent failure developing at the top of the bolted height would result in gross overloading and collapse of all commonly used rectangular profile steel supports.

This also had implications for steel used as remedial support in rectangular profile roadways as it would only be effective where it had the capacity to support the weight of failing rock. For most steel support systems this would equate to little more than the
immediate roof. The clear conclusion was that where mixed support was used in a rectangular profile, the rockbolt system must remain effective as roof support, to maintain safe conditions. The role of steel support, if required, should be limited to support of loose immediate roof.

Ground Assessment and Design Practices

Ground assessment and design practices are very similar for arched and mixed support roadways. When referring to the Mines (Control of Ground Movement) Regulations 1999 however the DMCIAC does indicate that mixed support roadways may require a more detailed ground investigation although the reason why is not given, the implication is that they are higher risk.

Mixed support is high risk for roadway profiles which are either rectangular in profile or near rectangular in profile, with existing inspection procedures. It would therefore seem prudent that more detailed ground investigations are undertaken when designing support for these types of roadway. The design of the rockbolt system should be to full rockbolt support standards and also take into account the function of the steel support. A monitoring scheme, which confirms the continuing effectiveness of the bolting system, needs to be specified in order to reduce the ground control risk.

For D shaped arch profile roadways the ground control risk is low and the ground assessment and design document could be simplified and based mainly on existing knowledge and experience.

Management and Monitoring Practices

D Shaped Arch Roadways

The review of falls of ground indicated that the management and monitoring of D shaped arched roadways, using MASHAM, with or without rockbolts, was satisfactory as the residual risk of their failure could be considered to be low. Despite the relatively low risks associated with arched shaped roadways many have a long life and the roof fall data did point to long term deterioration as a potential problem. Thus risks associated with these roadways could be further reduced by improving the monitoring and inspection system to identify any areas of deterioration requiring repair. This could be achieved by systematic ‘audit’ type roadway inspections by suitably experienced mine personnel at appropriate intervals, for example six monthly or yearly. These inspections could be targeted towards roadway sections of particular concern, such as wet areas.

Rectangular Profile Roadways with Steel Support

The roof fall data indicated that roadways supported by rectangular profile steel supports (including Delta types), without rockbolts, carried a relatively high risk of ground control failure. In these circumstances the MASHAM monitoring scheme was not reducing the residual risk to an acceptably low level. This was because of the relatively low strength of rectangular steel support and the possibility of sudden failure when it is overloaded. For this reason it was recommended that rectangular and near rectangular profile steel support, without supplementary rockbolt support, should no longer be used, except in special circumstances, for example the use of heavy section junction type supports, or in very narrow roadways where the risk of overloading could be shown to be low. (Note that junction support was not specifically examined in this study).
Rectangular Profile Mixed Support Roadways

The review of falls of ground showed that rectangular profile steel supported roadways with rockbolting were high ground control risk. These roadways had also relied on MASHAM inspections for monitoring. Although roof deformation monitoring with telltales may also have been carried out, this had not necessarily followed the procedures or used the same action levels as for primary bolted roadways. The introduction of roof movement monitoring and associated remedial support measures to primary rockbolt support standards would reduce the ground control risk to the same as that for fully rockbolted roadways. In these mixed support rectangular profile roadways, the monitoring scheme could be modified to suit the steel support element of the support system, by including a modified action level for movement within the height of roof, which the steel could support.

Monitoring of Rectangular Mixed Support Roadways

The roof monitoring scheme for effectively rectangular profile roadways with mixed system support should reflect the capacity of the steel support. It is possible to calculate how much of the immediate roof the steel can support based on steel strength, roadway width and support spacing. A modified monitoring system could make use of the triple height telltale, with the ‘A’ telltale reading representing the roof height that the steel can support. No specific ‘A’ action level would be required, although visual confirmation of effective support would be prudent where large deformations were recorded. The ‘B’ telltale would be anchored at the rockbolted height and could be used to indicate the roof deformation between the top of the strata supported by the steel and the bolted height. The ‘C’ telltale would represent movement above the rockbolted height. Action levels and remedial actions would need to be defined. In normal circumstances these should be similar to those for the equivalent fully rockbolted roadway.

Steel Support as a Remedial Measure in Primary Reinforced Roadways

A separate concern was the use of steel support as a remedial support measure for fully rockbolted roadways. This could include the use of centre legs.

As already indicated only a limited height of roof strata can be adequately supported by the steel, dependant upon the roadway width and steel support type and spacing, and in some cases the use of centre legs. Consequently when steelwork is set for remedial roof support, it is important that it should have adequate capacity to support the height of deforming roof indicated by monitoring data. Guidance on the use of steel support as a remedial supporting measure should be drawn up based on information on support capacity and equivalent maximum roof height that could be supported. This could be included with wider guidance on the use of steel in mixed support systems.

5.3.5 Conclusions

This research indicated that, for design and management purposes for risk reduction, steel supported roadways could be divided in to 2 groups:

(a) Effectively rectangular profile steel supported roadways, UK Coal Delta type supports 1, 2, 3 and 4,

(b) D shaped arched roadways, UK Coal Standard Arch support types 5, 6 and 7.
**D Shaped Arched Roadways**

D shaped arched roadways with or without supplementary rockbolts could be considered relatively low risk in the normal range of coal mine support conditions and there was no indication of any need to change existing practice for monitoring based on MASHAM inspections. However in order to reduce the risk of fall of ground due to long term deterioration of the steel it was recommended that consideration should be given to introducing systematic ‘audit type’ roadway inspections at suitable intervals.

As roadways supported by D shaped arched supports with or without bolts could be considered low risk, the ground assessment and design documentation procedures could be simplified whilst still complying with the Ground Control Movement Regulations 1999 and DMCIAC Guidance and suggested draft forms for this purpose were drawn up.

**Effectively Rectangular Profile Roadways with Delta Type Supports**

Steel supports with an effectively rectangular profile both with and without supplementary rockbolts should be considered relatively high risk with current MASHAM monitoring and inspection procedures. This was because they are significantly weaker than arch profile steel supports and there is an increased risk of relatively sudden collapse, possibly with limited visual warning. Steel supported roadways with an effectively rectangular profile without rockbolts should not be used except in special circumstances where reduced risk can be demonstrated, for example narrow roadways.

Where steel and rockbolt support (mixed support) are used in an effectively rectangular profile roadway, the reinforcement system must remain effective as roof support, to maintain safe conditions. The role of steel support, if required, should be to support a limited height of the immediate roof commensurate with the steel support capacity. Where steel support is used with rockbolts in an effectively rectangular profile the reinforcement system should be designed and monitored using the same procedures as for a fully rockbolted roadway. A suitable roof movement monitoring scheme should therefore include appropriate monitoring of the roof measures supported by the steel. This could be done by installing triple height telltales or additional telltales to differentiate between deformation in the height of roof that the steel can support and that above, which is effectively supported only by the rockbolt system. Data for use in the development of such a monitoring scheme was provided.

Flat-topped steel is also set for remedial support in rockbolted roadways. However it would only support the immediate roof unless it was centre legged. Clear guidelines were therefore needed to ensure that it was used in an appropriate manner.

**5.3.6 Guidance Notes on the Selection and Use of Steel and Mixed Support in UK Coal (Mining) Ltd. Mines**

At UK Coal’s request supplementary notes were produced for guidance on the selection and use of steel and mixed support, including appropriate measures for assessing support adequacy, when installed on drivage. They were aimed at supplementing the UK Coal Mining Ltd Company Directive 6/02 which was issued to ensure compliance with the Control of Ground Movement Regulations, 1999.

The supplementary guidance notes covered the following:
Selection of Steel and Mixed Support Systems
The support for selection was indicated to be based on the UK Coal Mining Ltd. Standard Steel work outlined in Table 14.

Potentially Low Risk Ground Control systems were:
- D shape arches used with MASHAM Regulations
- D Shape arches and rockbolts used with MASHAM regulations
- Rectangular roadways with rockbolts used with the DMCIAC Guidance
- Rectangular roadways with rockbolts and steel support used with DMCIAC Guidance

Potentially Higher Risk Ground Support Systems were:
- Rectangular or effectively rectangular with steel with no bolts
- Rectangular or effectively rectangular steel and rockbolts without a monitoring system.

It was recommended that these potentially higher risk ground support systems no longer be used except in circumstances in which low ground control risk could be demonstrated by a detailed risk assessment.

Design of Steel Arch Systems
As D shaped supports used in arched roadways are normally low risk simplified ground assessment and support design documentation could be used. To this end a tabulated format was proposed where all relevant geotechnical factors could be considered.

Assessment of Adequacy of Ground Control Measures for D Shaped Steel Arch Supported Roadways
Once installed D shaped steel arches are associated with low ground control risk with typically progressive and visually obvious deformation. However there are some additional ground control risk factors which are associated with falls of ground in arched roadways:

(a) long term deterioration/corrosion,
(b) wet areas,
(c) geometry change, for example roadway enlargement, junction formation, and
(d) roadway repair.

In these case systematic ‘audit type’ roadway inspections were recommended in addition to the MASHAM inspections.

Design of Mixed Support Systems in Rectangular Profile Roadways
In a mixed support system the rockbolts should be considered to be primary support and should be designed using the same procedures as for fully rockbolted roadways. The role of the steel should be considered, in normal circumstances, to be limited to support of the immediate roof to a maximum of one third the bolted height; this is to ensure that the rockbolt system is still effectively supporting the roof above the immediate roof supported by the steel support. The one third bolted height limit can be assumed valid for specified conditions based on:

(a) the type of steel support set,
(b) roadway width,
(c) maximum bolt length.
If one or more of the limits is exceeded the safe supported height should be calculated. The details of the calculations that need to be undertaken were provided and these are given in Appendix 1.2.

**Monitoring and Management of Mixed Support Systems in Rectangular Profile Roadways**

Guidelines were provided on a suitable roof movement monitoring scheme for rectangular roadways with mixed support. This involved the use of multi-height telltales and their appropriate action levels. In addition, to complement this, guidance on the selection of suitable remedial measures if movement developed above action levels was given. Data on the support capacities of wooden legs and cribs were detailed and reinforcement strategies outlined.
6. CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

**NDT - Ultrasonics**
Although relevant research was undertaken there was no success in sourcing a better instrument for the ultrasonic testing of rockbolts. Thus work continued with the Krautkramer USD10 NF, with which there had been some success under the previous research project.

Under this project it was used with some success to survey selected rockbolts at Thoresby Colliery along the length of PG42’s Loader Gate that was being recovered for re-use following 7 years of abandonment. The survey aimed to determine the proportion of roofbolts likely to be intact such that decisions could be made as to a suitable level of additional reinforcement required to maintain roof stability for the gate’s re-use.

The survey was carried out on 50 roofbolts. 19 tests were able to successfully detect the end of the bolts and thus indicate they were intact. Of the other tests there was no indication that the bolts were either broken or badly bent. It was likely that the reason that bolt ends were not detected was the unfavourable chamfer angle at the back end of the bolts.

At a separate UK colliery, Welbeck, the instrument was used to survey bolts in a gate road where there were known to be bolts broken at approximately 0.6m into the roof. Of the bolts tested reflections were observed from approximately 0.6m into the roof. However it was not possible to determine, using the ultrasonics method alone whether these were related to bends or breaks, in this case the survey required the use of the RF system in order to detect the difference.

**NDT – RF**
Success had been achieved with the RF NDT system under the ECSC project AB 149 (ECSC 2000[1]), with the original instrument, (MFJ-259). However this had been very limited in Coal Measures strata and it was suspected that the probable reason was that the instrument was unable to read at the lower frequency ranges which were characteristic of tendons in this strata type. The bottom range of the MFJ–259 was originally 1.4MHz. As attempts to reduce the range of the MFJ-259 were only partially successful, a new instrument was specifically built to read to 0.2MHz.

The new LRF instrument appeared to work well with promising results recorded from datum bolts of different lengths up to 2.4m installed within a coal mine. However there were inconsistencies in results and it was determined that the instrument was sensitive to lead length. Modifications were made and further tests conducted, but results were still inconsistent.

At the time difficulties were being experienced with the LRF a new instrument became available that had potential advantages over the LRF, it had a frequency range as low as that of the LRF but also was able to display the SWR, record the X, R and Z components and was potentially likely to be more stable than the LRF. The new instrument proved to work well although it has only had relatively limited use due to it being obtained somewhat late in the course of the project.
At Thoresby Colliery the RF system worked well and the LRF and new instrument were both able to determine the length of certain 2.4m long original AT rock bolts. However the system did not work on all the original in-situ bolts tested. The probable cause was considered to be a change in ground conditions along the gate road with strata water affecting the results. However when bolts were specifically installed in the area of poor results positive test results were obtained. The reason is still unclear, but is probably associated with changes to the conductivity of the rockbolt encapsulating resin with time in wet conditions.

RF testing was also undertaken at Welbeck Colliery in a gate road where shear movement was known to have broken bolts approximately 0.6m into the roof at specific locations. The new instrument was able successfully to determine the length of existing 2.4m long AT bolts and 1.8m long AT lifting bolts. The results correlated very well with those obtained from Thoresby. The instrument was also able to confirm the presence of a broken bolt. At a separate site at Welbeck the RF instrument has indicated that potentially broken bolts were likely to be intact.

Following the work under this project the RF testing has now been successfully undertaken within the following rock environments:

- Slate,
- Salt,
- Limestone,
- Granite,
- Coal Measures limestone,
- Coal Measures sandstone,
- Coal Measures siltstone.

The RF system has now reached a stage where it is a practical tool to locate broken reinforcement in European mines between tendon pairs, though it suffers from the disadvantage of requiring electrical isolation between the tendon pairs tested.

However, despite this success there have also been some inconsistent results, notably when testing SAT bolts and when the mesh is used as the ‘ground plane’ in the orthogonal test geometry. Laboratory test work has indicated a potential cause for these inconsistencies under certain circumstances. The availability of the X, R and Z parameters on the new instrument means that it is now considerably easier to differentiate between valid and invalid results when using the RF bolt integrity method. Further work is required to test the potential explanation for the anomalous results and examine ways of overcoming the problem when it is encountered.

**Instrumentation**

Instrumentation developments have led to the production of a portable readout unit for local reading of a remote reading tell tale system. This avoids the need for cabling the system to the mine surface in circumstances where this would not be practical. This system has worked well in an Indian coal mine where roof movement was monitored during pillar extraction.

A ‘blast proof’ version of the dual height remote reading tell tale has been developed. The transponders are bonded up the hole and there are no bobbins external to the hole for visual reading or which can be damaged by flying rock during blasting. The extensometer is read locally by plugging it into a dedicated readout unit. Once the danger from blast damage has passed the readout cable can be left plugged into the instrument in order to read it from the roadway floor rather than having to access the
Improvements had to be made to the initial design including spring loading of the bobbins to prevent them from sticking at the mouth of the hole and also to the method for bonding the unit into the hole.

A 4 height remote or local reading extensometer has been developed based on the dual height version.

An instrument for the detection of roof shear became a priority after a roof fall at Welbeck Colliery where shear had caused the normal dual height telltale to cease functioning and thus give misleading results once the wires were trapped. The simplest solution was to install a telltale with an additional spring that could be used to check for roof shear jamming of the wires. This should be used in combination with more rigorous risk assessment, to identify high risk areas, and sentinel bolts in these areas to detect roof shear.

**Risk Assessment**

The purpose of a risk assessment is to identify areas of significant risk in relation to roof and rib stability so that the risk can be reduced by applying appropriate additional support or reinforcement.

A risk assessment was carried out for PG42’s Loader Gate at Thoresby Colliery where the gate road had been recovered following abandonment 7 years earlier. The previously developed risk assessment procedure was modified to take into account the additional reinforcement installed and the modified action levels used following the subsequent installation of triple height telltale.

Following a fall of ground at Rossington Colliery in a mixed support system on a faceline an analysis was undertaken of how these support systems were designed and managed in order to develop improvements to the system to reduce the ground control risk.

Examination of statistics on falls of ground indicated that steel supports with an effectively rectangular profile both with and without supplementary rockbolts should be considered relatively high risk with current MASHAM monitoring and inspection procedures. This is because they are significantly weaker than arch profile steel supports and there is an increased risk of relatively sudden collapse, possibly with limited visual warning. It was concluded that steel supported roadways with an effectively rectangular profile without rockbolts should not be used except in special circumstances where reduced risk can be demonstrated, for example narrow roadways.

Where steel and rockbolt support (mixed support) is used in an effectively rectangular profile roadway, the reinforcement system must remain effective as roof support, to maintain safe conditions. The role of steel support, if required, should be to support a limited height of the immediate roof commensurate with the steel support capacity. Where steel support is used with rockbolts in an effectively rectangular profile the reinforcement system should be designed and monitored using the same procedures as for a fully rockbolted roadway. A suitable roof movement monitoring scheme should therefore include appropriate monitoring of the roof measures supported by the steel. This could be achieved by installing triple height telltale or additional telltale to differentiate between deformation in the height of roof that the steel can support and that above, which is effectively supported only by the rockbolt system.
6.2 INTEGRATED STRATEGY TO REDUCE RISK OF FALLS OF GROUND

One of the objectives of this project was to integrate results of the work undertaken in order to come up with an overall strategy in order to reduce the risk of falls of ground. The research areas for inclusion are listed below and these are integrated in a single flow chart in Figure 64.

(i) Non Destructive Testing of Rockbolts
   - Ultrasonic technique
   - Radio Frequency technique
(ii) Instrumentation Developments
    - Local electronic reading of a telltale system
    - Local electronic reading of a 2 or 4 wire extensometer
    - Instrumentation and strategy for detecting shear movement - the spring anchored ed telltale and/or sentinel bolts
(iii) Risk Assessment Methods.

6.3 RECOMMENDATIONS

The following recommendations are made relating to the various research areas:

Non Destructive Testing of Tendons ; Ultrasonics

Although the instrument used under this current research, the Krautkrammer USD10 NF, was used with some success at both Thoresby and Welbeck Collieries, it and the associated probes used with it, have been shown to have a limited range of use, especially in UK coal mines where the bolts typically have a chamfered distal end.

A newly available portable instrument, the Wavemaker 16, was trailed but with disappointing results. Another prototype instrument, developed by NDT Solutions, was planned to be evaluated, but this did not prove possible within the timescale of the project. It is recommended that this be evaluated in further research. Also with improving technology it is possible that a more suitable instrument could still be specifically developed.

The method has the disadvantage of requiring end preparation of bolt ends to obtain good signal transmission and further research should address this problem by either attempting to remove the need for end preparation or make it simpler to achieve.

Non-Destructive Testing of Tendons : Radio Frequency

During the course of the project there was rapid and successful instrument evolution from the MFJ, to the LRF and then the VIA Analyser. This has led to the ability to be able to determine rockbolt lengths in Coal Measures strata previously unattainable at the start of the Project.

However, despite this success there were also some inconsistent results when testing AT bolts in certain situations, SAT bolts and when the mesh is used as the ‘ground plane’ in the orthogonal test geometry. Laboratory tests have given some indication to the likely cause, this being leakage resistance between the tested pairs under certain conditions. Further work is required to test this explanation of anomalous results and examine ways of overcoming the problem when it is encountered. For example, direct measurement of the leakage resistance between tendon pairs or modifications to the instrument to allow for a range of internal resistances.
Further work is recommended on investigating the use of the orthogonal method for coal mines in order to reduce the need for bolt isolation, which is currently a disadvantage of the successful parallel method.

It is recommended that the parallel RF method is now used on a much wider scale as a tool to identify broken reinforcement tendons in mines, tunnels and caverns and that it is integrated into routine, fall of ground risk assessment procedures at UK coal mines.

**Instrumentation Developments**

*Remote reading dual height tell tale system with portable readout unit*: The hand held, battery powered readout developed for use with the remote reading telltale transponders was demonstrated to work very well in an Indian room and pillar mine for a site specific intensive scientific study in coal mine demonstrating excellent stability and accuracy attainable. The experience suggests that it would be worthwhile investigating obtaining permission to use the system under EAWR19 for UK coal mines for site specific studies.

*Dual height and four wire remote reading extensometers*: The dual height version of this extensometer has been shown to work well and the four wire version has been developed. The 4 wire version now needs field installation in order that its design can be fully assessed in the field. This has the potential for use in UK coal mines to replace the 4 wire extensometers where greater accuracy is required or when access for reading is problematic. The principle also has the potential for development into an extensometer with more anchors which could be a potential replacement for the sonic extensometer.

*Instrumentation for Detection of Shear*: A practical combination of instrumentation (modified telltales, sentinel bolts and roof shortening meters) has been developed to identify when excessive shear is occurring in a mine roof. It is strongly recommended that this is now implemented on a much wider basis in UK coal mines. Also, it is recommended that the new version of telltale, incorporating an additional wire to detect shear, is employed rather than the version with the shear detection spring being incorporated in the B indicator wire. Although more expensive, this version involves less risk of disturbing the telltale and allows the additional wire to be extended to an accessible level in high roadways.

**Risk Assessment**

Routine risk assessment of a district for falls of ground hazards prior to face retreat has been proven to be a practical and effective means of ensuring that potential hazards are dealt with in a timely manner. In neighbouring mines working the same seam, one has applied the procedure consistently over an extended period with no roof falls and the other, which has not applied the method, has suffered from at least one major fall in a retreating face gateroad. It is recommended that the procedure is applied much more widely and incorporates NDT testing of tendon integrity where appropriate.

**Integrated strategy**

It is recommended that all the techniques and instruments developed under the Project are applied at UK mines in an integrated manner as suggested in the flow chart shown in Figure 64.
7. REFERENCES


ECSC 2000 (1), European Coal and Steel Community, Geotechnical studies for rockbolting - part 2, Project Number: 7220-AB/149.


ECSC 2003 (1), European Coal and Steel Community, Improved support systems for highly stressed roadways, Project Number 7220-PR058.

ECSC 2003 (2), European Coal and Steel Community, Development and demonstration of automatic ground hazard monitoring systems, Project Number 7220-PR059.


UK Coal Mining Ltd. 2002, Management process to be adopted to ensure compliance with the control of ground movement regulations 1999, Company Directive 6/02, issued October 2002,
FIGURES
General location plan for 42’s Loader Gate, Parkgate Seam, Thoresby Colliery

Figure 1
Detailed layout 42's District, Parkgate Seam, Thoresby Colliery

Figure 2
Geological variation along 42’s Loader Gate,
Parkgate Seam, Thoresby Colliery

Figure 3

Height above Parkgate Seam (m)

<table>
<thead>
<tr>
<th>Height (m)</th>
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<tbody>
<tr>
<td>6.0</td>
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<tr>
<td>5.0</td>
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<tr>
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<tr>
<td>3.0</td>
</tr>
<tr>
<td>2.0</td>
</tr>
<tr>
<td>1.0</td>
</tr>
<tr>
<td>0</td>
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Distance along 42’s Loader Gate (m)

<table>
<thead>
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<th>Distance (m)</th>
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<tr>
<td>100</td>
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<td>300</td>
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</tr>
<tr>
<td>800</td>
</tr>
<tr>
<td>900</td>
</tr>
<tr>
<td>1000</td>
</tr>
</tbody>
</table>

Geology from Roofbolt Overcore
Roof Cores 5m

- **Sandstone**
- **Sandstone / Occasional Laminea**
- **Sandstone / Siltstone 50:50 – 90:10**
- **Siltstone / Sandy**
- **Ferruginous Patches**

Coal Parkgate

Roof Horizon
Schematic layout of 312’s District, Parkgate Seam, Welbeck Colliery

Figure 4
**ROOF STRATA ASSESSMENT**

**CLIENT:** UK Coal Ltd  
**SITE:** WELBECK, Parkgate Seam  
312’s Loader Gate 154RM

---

**LOGGED BY:** L. Kent  
**DATE CORED:** 16th October 2001  
**DRILLING:** Cored by Colliery, TT56 core barrel with liners, Water flush, 42mm nominal diameter core

---

<table>
<thead>
<tr>
<th>MATERIAL PROPERTIES</th>
<th>LITHOLOGICAL AND FRACTURE LOG</th>
</tr>
</thead>
<tbody>
<tr>
<td>U.C.S. (MPa)</td>
<td>Modulus (GPa)</td>
</tr>
<tr>
<td>-------------------</td>
<td>----------------</td>
</tr>
<tr>
<td>80</td>
<td>25</td>
</tr>
<tr>
<td>74</td>
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<td>62</td>
<td>-</td>
</tr>
<tr>
<td>67</td>
<td>-</td>
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</tbody>
</table>

---

**DESCRIPTION**  

**NOT CORED** Core ends at 4.84m.  

**SILTSTONE**  
Grey, fine-medium.  

**CORE LOSS**  
Grey, medium.  

**SILTSTONE**  
Grey, fine.  

**SILTSTONE**  
Grey, fine, with occasional mudstone bands.  

**MUDSTONE/SILTSTONE**  
Grey, interbanding of occasional mudstone with silty mudstone/fine siltstone.  

**MUDSTONE**  
Grey.  

---

Figure 5  
Roof core from the 154RM, 312’s Loader Gate, Parkgate Seam, Welbeck Colliery
Layout plan for 217’s District and the South Return, Deep Soft Seam, Welbeck Colliery

Figure 6
## GEOLOGICAL STRATA SECTION

**COLLIERY:** WELBECK  
**SEAM:** ROOF/DEEP SOFT  
**DISTRICT:** 217’s RETURN GATE R. H. UPBORE  
**DISTANCE:** 178.3 m  
**FROM:** South Intake  
**N. G. COORDINATES:** E 461 000 N 368 586  
**LEVEL:** 744.7m BOD  
**EXAMINED BY:** J.D. FREEMAN  
**DATE:** 15-15/94  

Section of core taken in 217’s Return Dev. Hdg. Upbore

### GRAPHIC LOG

<table>
<thead>
<tr>
<th>THICKNESS (cm)</th>
<th>NATURE OF STRATA</th>
<th>ESTIMATED COMP. STR. (PSI)</th>
<th>ESTIMATED I. T. P.</th>
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</thead>
<tbody>
<tr>
<td>318</td>
<td>Siltstone medium, few sandstone rippled laminae and thin layers, 95/05, few ripple sets, &lt;passage&gt;</td>
<td>9000</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>223</td>
<td>Siltstone fine, poorly laminated, locally muddy, small burrows near top and base, &lt;passage&gt;</td>
<td>6000</td>
<td>LOW</td>
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<tr>
<td>304</td>
<td>Siltstone medium - fine, few sandstone laminae and ripples, 95/05, few small burrows &lt;passage&gt;</td>
<td>8000</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>103</td>
<td>Siltstone medium, poorly laminated, common dark thin burrows, Some colonial, &lt;passage&gt;</td>
<td>8000</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>52</td>
<td>Siltstone, fine, few dark thin burrows, few oblique hackly fractures</td>
<td>START OF UPBORE CORE</td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>Siltstone, fine, poorly laminated, rare sandy laminae, barren</td>
<td>6000</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>54</td>
<td>Coal, mainly bright, dirty near base</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>Mudstone, friable in parts</td>
<td>4500</td>
<td>LOW</td>
</tr>
<tr>
<td>105</td>
<td>Coal, mainly bright, 0.09 dull band near middle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Mudstone, silty, dark</td>
<td>5000</td>
<td>LOW</td>
</tr>
<tr>
<td>23</td>
<td>Coal, mainly bright</td>
<td></td>
<td></td>
</tr>
<tr>
<td>75+</td>
<td>Mudstone, silty, seatearth, locally friable, locally coarser</td>
<td>5/6000</td>
<td>LOW</td>
</tr>
</tbody>
</table>

Roof core from 217’s Return Gate, Deep Soft Seam, Welbeck Colliery  

Figure 7
Ultrasonic traces from the 2.4m datum bolt with the high frequency and low frequency probes, 42's Loader Gate, Parkgate Seam, Thoresby Colliery

Figure 8
Calibration tests undertaken with the high frequency and low frequency probes, 42’s Loader Gate, Parkgate Seam, Thoresby Colliery

Figure 9
Examples of the ultrasonic results from the in-situ rockbolts tested in 42’s Loader Gate, Parkgate Seam, Thoresby Colliery

Figure 10
Figure 11

4MHz probe trace results from the anchor bolts tested at the road cutting
100kHz probe trace results from the anchor bolts tested at the road cutting

Figure 12
Reflections from a slot cut in a Dywidag anchor

Figure 13
Schematic diagram to show the layout of the 20mm diameter resin encapsulated calibration bolts at the evaporite mine

Figure 14
Ultrasonic results from the 1.4 and 1.0m long test bolts installed into salt ribside at the evaporite mine

Figure 15
Ultrasonic results from the 0.6m long test bolts installed into salt ribside at the evaporite mine

Figure 16
Examples of ultrasonic test results using the 2 MHz probe on the four machine anchor bolts at the power station

Figure 17
Ultrasonic traces from the 4MHz (lg.) probe from the two potentially broken and the one potentially intact 2.4m rockbolts at the 524 and 527RM, PG312’s Loader Gate, Welbeck Colliery

Figure 18
Ultrasonic traces from the 70kHz probe from the two potentially broken and the one potentially intact 2.4m rockbolts at the 524 and 527RM, PG312’s Loader Gate, Welbeck Colliery

Figure 19
Ultrasonic traces from the 850kHz probe from the two potentially broken and the one potentially intact 2.4m rockbolts at the 524 and 527RM, PG312’s Loader Gate, Welbeck Colliery

Figure 20
Ultrasonic traces from the 4MHz (large) probe from the three potentially broken and one potentially intact 2.4m rockbolts Deep Soft Seam South Return, 217’s R/G Junction, Welbeck Colliery

Figure 21
Ultrasonic traces from the 850kHz probe from the three potentially broken and one potentially intact 2.4m rockbolts Deep Soft Seam South Return, 217’s R/G Junction, Welbeck Colliery

Figure 22
Photographs of the original RF instrument the MFJ-259 and the replacement LRF-200

Figure 23
Resonances for a tendon pair (10m + 12 long) taken using the original MFJ instrument and the new LRF instrument

Figure 24
Resonance troughs recorded by the MFJ and LRF instruments for the datum bolt pairs, PG42’s Loader Gate, Thoresby Colliery

Figure 25
(a) Alternative connection methods
   Top - crocodile clip
   Middle - jubilee clip
   Bottom - custom designed nut connector

(b) Effect of bolt separation and connector type on frequency for 2.4m long rockbolt pairs, 42’s Loader Gate, Thoresby Colliery

Alternative connection methods and results from tests with the different connection methods and bolt spacing, PG42’s Loader Gate, Thoresby Colliery

Figure 26
Resonance troughs recorded by the LRF instrument on the Thoresby datum bolts before and after the LRF instrument modifications

Figure 27
(a) Observed resonances from the 2.4m datum bolts at Thoresby Colliery with the original, modified and re-modified LRF instrument

(b) Plot of 1/frequency versus mean length for the original and re-modified LRF instrument

Plots from the datum bolts at Thoresby in 42’s Loader Gate to compare results from the original, modified and re-modified LRF instrument

Figure 28
Data from the cables at Middleton Mine comparing the results from the new VIA instrument with those from the LRF and MFJ instruments.
(a) Data results from the 0.6m and 1.2m bolt combination with the VIA Analyser

(b) Comparison of the LRF, MFJ Aand VIA results from the datum bolt combinations

Data from the datum bolts at Thoresby Colliery comparing the results from the new VIA instrument with those from the LRF and MFJ instruments
RF interrogation of test bolts installed into a dry salt ribside at an evaporite mine

Figure 31
(a) Resonance data from Bolt RA 2 No.124 in parallel with Bolt RA2 No. 82

(b) $1/f(0)$ plotted against the mean nominal lengths

RF results from the anchor bolts at the road cutting

Figure 32
(a) Null balance (%) versus frequency for pairs of bolts (LRF leads)

(b) Frequency versus mean bolt length

RF results from the LRF instrument on datum bolts at 178m mark, PG42’s Loader Gate, Thoresby Colliery

Figure 33
Comparison of the site calibration data recorded at Thoresby with other site data

Figure 34
(a) Null balance (%) versus frequency for pairs of bolts (standard leads)

(b) 1/Frequency versus mean bolt length

RF results from the LRF instrument on datum and in-situ bolts at 178m mark, PG42’s Loader Gate, Thoresby Colliery

Figure 35
RF results from the LRF instrument on the additional datum bolts at 457m mark
PG42’s Loader Gate, Thoresby Colliery

Figure 36
(a) Example of a valid result from a datum to existing bolt (177MM, test THO1002 2.4m datum to 2.4m long existing spotbolt)

(b) Example of an invalid result from an existing to existing bolt (245MM, test THO1008 2.4m existing (isolated) to 2.4m long existing in strap)

RF results from the VIA instrument to illustrate the good and poor results on datum and existing bolt combinations in PG42’s Loader Gate, Thoresby Colliery
Effect of bolt plates on RF resonances at the 456MM with the modified LRF meter (hence shallow resonance curves)

Figure 38
(a) Example of a result from two 1.8m long steel rib bolts in coal, PG 42’s Loader gate, Thoresby Colliery

(b) Summary of the rib bolt tests plotted via bolt distance apart and location PG 42’s Loader gate, Thoresby Colliery

RF results from the VIA instrument to illustrate a typical resonance from a pair of bolts in a coal rib and a summary of all the rib bolt test results, PG42’s Loader Gate, Thoresby Colliery
RF results from the VIA instrument on a pair of 2.4m long SAT bolts in PG312’s Loader Gate, Welbeck Colliery

Figure 40
Schematic diagram of tested bolts at the 523-524RM, PG312’s Loader Gate, Welbeck Colliery

Figure 41
(a) Example of a valid resonance from a pair of 2.4m long intact AT roof bolts

(b) Comparison of the Welbeck PG312’s L/G Results 523/524RM with those from Thoresby PG42’s Loader Gate

Successful RF results from the 523/524RM PG312’s Loader Gate, Welbeck Colliery
(a) Valid resonance from the potentially broken bolt to a potentially intact 2.4m long AT roofbolt

(b) Plot indicating the mean length of the broken to intact bolt pair at the 527RM, PG312’s, Loader Gate to be 0.6m

RF results indicating a broken 2.4m long AT bolt at the 527RM, PG312’s Loader Gate, Welbeck Colliery

Figure 43
Schematic diagram of tested bolts in the Deep Soft South Return at 217’s Return Gate Junction, Welbeck Colliery

Figure 44
(a) Weak resonance from the potentially broken bolt in the Deep Soft Return

(b) Plot showing positions of the Bolt 1 and Bolt 2 test pair in the Deep South Return indicating that they are likely to be Intact

Potentially successful RF results from the Deep Soft Return, 217’s Return Gate Junction, Welbeck Colliery

Figure 45
Schematic diagram and photographs of the intact No. 1 bolt recovered from the Deep Soft South Return, 217’s R/G Junction, Welbeck Colliery
Laboratory test with the VIA connected to a bolt pair simulator with 10 and 2.2 Ohm resistors to show the effect of potential leakage between test bolt pairs in the field

Figure 47
To spring anchors

Spring Anchor
Coil “A”
Core “A”
Stabiliser
Drip tray
Electronics
Core “B”
Coil “B”
“A” Indicator
“B” Indicator
Water seals

Surface computer
Local Interrogation Unit
Up to 100 passive transponders
Up to 4 per system

(b) Normal layout for the remote reading tell tale system

(a) Tell tale transponder unit

(b) Portable read out unit to replace local interrogation unit

Remote reading tell tale system and modifications for an Indian room and pillar coal mine

Figure 48
Example readings from the remote reading tell tale system in the Indian room and pillar coal mine
(a) Selected roof movement data from the remote reading telltales

(b) Selected roof movement rates from the remote reading telltales

Example readings from the remote reading tell tale system in the Indian room and pillar coal mine

Figure 50
(a) Schematic cross section of the remote reading ‘blast proof’ dual height extensometer

(b) Photograph of the remote reading blast proof dual height extensometer

Schematic cross section and photograph of the Mark 1 blast proof remote reading dual height extensometer

Figure 51
Photographs of the Mark 1 blast proof remote reading extensometer readout unit and its use underground

Figure 52
(a) Typical roof movement data recorded from a blast proof extensometer

(b) Roof movement data alongside the complementary dummy extensometer installed to monitor possible effects of temperature variation

Typical data recorded by the Mark 1 blast proof remote reading extensometer
Mark 3 blast proof remote reading extensometer laboratory trials and readout unit RRTT-1342-PR

Figure 54
To spring anchors

Water seals

Retaining anchor

Extensometer body

Stabilising fins

Drip tray

Connecting cables

4, 3, 2, and 1 anchor wire end

Schematic diagram and photograph of the 4 wire remote reading extensometer  Figure 55
### General Rockbolted Roadway Risk Assessment Survey – Thoresby Colliery (RISKTHPG42LG.doc)

**Survey sheet for PG42’s Loader Gate risk assessment, Thoresby Colliery**

<table>
<thead>
<tr>
<th><strong>District:</strong></th>
<th>PG42’s Loader Gate</th>
<th><strong>Date:</strong></th>
<th><strong>Surveyed Section:</strong></th>
<th><strong>Sheet No.:</strong></th>
</tr>
</thead>
</table>

- Original support system  
  - ROOF: 7 x 2.4m AT at 1.0m cycles  
  - RIBS: 3 x 1.8m faceside and solid side at 1.0m cycles
- Original bolt installation system  
  - Hand held Wombats
- Original heading machine type  
  - Boom Header
- Additional Support:  
  - ROOF: 5 x 1.8m bolts at 1.0m Cycles + Mesh cage: RIBS: Parra Mesh

### Monitoring / Deformation

<table>
<thead>
<tr>
<th>Tell-tale No.</th>
<th>Tell-tale position (m)</th>
<th>A (mm)</th>
<th>B (mm)</th>
<th>C (mm)</th>
<th>stable Y/N</th>
<th>Are tell-tales operating correctly, signs posted, readable Y/N?</th>
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</thead>
</table>

**Comments**

### VISUAL CONDITION RATING (looking Inbye)

<table>
<thead>
<tr>
<th>Roadway Sketch (looking Inbye)</th>
<th>Left Hand Rib</th>
<th>Right Hand Rib</th>
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</thead>
</table>

**Additional information, sketches etc**

### RISK FACTORS (looking Inbye)

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<tr>
<th>Structure</th>
<th>Roadway Dimensions</th>
<th>Strata Water</th>
<th>Stress Effects – NB, cumulative effects, therefore add values if more than one stress feature present (circle relevant factors for future Reference).</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Roadway developed in line with H max (roof = 0, ribs = 0); Roadway developed across H max (roof = 1, ribs = 0)</th>
<th>Roof =</th>
<th>L/H rib =</th>
<th>R/H rib =</th>
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<table>
<thead>
<tr>
<th>Development</th>
<th>Retreat</th>
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<tbody>
<tr>
<td>ROOF = ...............</td>
<td>RIBS = ...............</td>
</tr>
</tbody>
</table>

Top Coal seam split – Not Applicable to PG42’s Loader Gate

N/A

- Orientation of roadway to Main Cleat – Parallel 0-30° (P), Intermediate 30-60° (I), Normal 60-90° (N)

### SUPPORT STANDARDS / ADDITIONAL SUPPORT

<table>
<thead>
<tr>
<th>Compliance with Manager’s Support Rules</th>
<th>Does support system comply with Manager’s Support Rules Y/N</th>
</tr>
</thead>
</table>

(If N specify)

<table>
<thead>
<tr>
<th>Additional Support (NB in addition to that specified in Support Rules, record details of type / side / pattern / spacing and if remedial or on development)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>RIBS</td>
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<table>
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<th>ROOF RISK REDUCTION CATEGORY</th>
<th>RIB RISK REDUCTION CATEGORY</th>
<th>L/H RIB</th>
<th>R/H RIB</th>
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**Figure 56**
<table>
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<tr>
<th>RATING</th>
<th>CATEGORY</th>
<th>ROOF</th>
<th>Features consistent with rating category</th>
<th>RATING</th>
<th>CATEGORY</th>
<th>SKETCH</th>
<th>Features consistent with rating category</th>
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</thead>
<tbody>
<tr>
<td>R1</td>
<td>GOOD</td>
<td></td>
<td>Flat roofline, and No overbreak or guttering, and No visible stress effects, and No deformation between straps , and No flat bolt plates</td>
<td>S1</td>
<td>GOOD</td>
<td></td>
<td>Original rib profile - rib face intact No spalling Minor visible deformation / movement</td>
</tr>
<tr>
<td>R2</td>
<td>GOOD</td>
<td></td>
<td>Slight flaking or undulating of the roof, or Minor localised shear in immediate roof, and No flat bolt plates, and No significant deformation between straps</td>
<td>S2</td>
<td>GOOD</td>
<td></td>
<td>Visible rib minor movement Coal rib generally intact Superficial fracturing in overlying stone Minor decoupling at top of seam OR at natural parting in stone rib near roof level</td>
</tr>
<tr>
<td>R3</td>
<td>FAIR</td>
<td></td>
<td>Visible roof shear/bulking or minor guttering Roof strap shortening / weathering between straps Small number of flat bolt plates (&lt; 10%) These may be present in flat or uneven roof profiles</td>
<td>S3</td>
<td>GOOD</td>
<td></td>
<td>Visible rib movement Induced fractures in upper stone rib Some spalling in lower rib between bolts Increased decoupling at roof level or in upper stone rib</td>
</tr>
<tr>
<td>R4</td>
<td>FAIR</td>
<td></td>
<td>Visible roof shear Significant overbreak or guttering Roof bulking between straps Several flattened bolt plates (&gt;10% &lt;20%) Moderate localised / isolated cavities on excavation</td>
<td>S4</td>
<td>FAIR</td>
<td></td>
<td>Moderate rib movement Upper stone rib fractured / weathered Some spalling of lower rib Local bulging in lower rib Increase decoupling evident Loss of rib profile / differential rib movement Signs of compressive / shear failure Increased collar loading on rib bolts</td>
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<tr>
<td>R5</td>
<td>POOR</td>
<td></td>
<td>High deformation Moderate to large localised cavities Extensive moderate depth cavities Significant guttering Roof sagging / bending Visible tensile cracks (near centre line) Large number of flattened bolt plates (&gt;20%) Bolts pulling through plates/ failed on threads High collar loads on flexible bolts/ cable bolts</td>
<td>S5</td>
<td>POOR</td>
<td></td>
<td>Seam highly fractured Significant spalling between and around bolts Significant upper rib fracture / spall Overhanging upper rib Ribside leaning into roadway Bulging rib side (significant tension breaks evident) Rib bolts failed or pulling through end plates</td>
</tr>
<tr>
<td>R6</td>
<td>POOR</td>
<td></td>
<td>High deformation / Roof lowering Large cavities Significant tensile cracks Roof straps / mesh tearing Large open gutters extending into immediate roof Roof sagging or cantilevering towards gutter Large number flat bolt plates / large number of bolts pulling through plates or failed on threads Cable / Flexi bolts pulling through end plates</td>
<td>S6</td>
<td>POOR</td>
<td></td>
<td>Gross rib failure / deterioration Severe spalling / rib overbreak Rib flushing or bulging High movement Significant loss of width Increase in roadway span if rib spalls Significant rib lean or overhang (stabbing / toppling potential) Failed rib bolts or bolts pulling through plates</td>
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### Category Of Remedial / Additional Support Set

<table>
<thead>
<tr>
<th>Reason for Increased risk</th>
<th>Level Of Risk</th>
<th>Steel Work</th>
<th>Cribs</th>
<th>Cable Bolts (Min: 8m Long)</th>
<th>Flexible Bolts (Min: 4m Long)</th>
<th>Spotbolts (Standard Length)</th>
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<tr>
<td></td>
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<td>LD</td>
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<td>A</td>
<td>B</td>
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<td>B</td>
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<tr>
<td>'B' Tell Tale &gt; action levels</td>
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<td>R5 - R6 Roof</td>
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<td>F1 Fault (minor faulting)</td>
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<td>A</td>
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<tr>
<td>F2 Fault (major faulting)</td>
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<tr>
<td>Disturbed lithology (DL)</td>
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<td>Top Coal seam split (-4)</td>
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### RISK REDUCTION CATEGORY

- **A** = Reduces 'High' risk to 'Low' residual risk
- **B** = Reduces 'High' risk to 'Medium' residual risk, or 'Medium risk to 'Low residual risk
- **C** = Does not reduce perceived risk but may improve conditions
- **NA** = Not Applicable - OR - Not Appropriate support for the perceived risk

#### Definitions:

- **LD Steelwork** = Light Duty Steel Sets (6" x 5" section) at standard spacings
- **HD Steelwork** = Heavy Duty Steel Sets (6" x 5" section) 0.6m spacings or closer
- **LD Steelwork** = Heavy Duty Steel Sets (8" x 6" section)
- **LD Cribs** = Light Duty Cribs = Narrow 4 point cribs, narrow set concrete or star block cribs
- **HD Cribs** = Heavy Duty Cribs = Hercules, 9 pt Hardwood, Wide set Link & Locks, wide set concrete or star blocks etc.
- **LD Cable Bolts** = Low Density Cable Bolt Pattern = < 0.4 cables per square metre of roof
- **HD Cable Bolts** = High Density Cable Bolt Pattern = > 0.4 cables per square metre of roof
- **LD Flexible Bolts** = Low Density Flexible Bolts = < 0.8 flexibolts per square metre
- **HD Flexible Bolts** = Low Density Flexible Bolts = > 0.8 flexibolts per square metre
- **LD Spotbolts** = Low Density Spotbolts = < 1 bolt per square metre (excluding bolts in existing pattern)
- **HD Spotbolts** = Low Density Spotbolts = > 1 bolt per square metre (excluding bolts in existing pattern)

### Guide for 5.0m wide roadway

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<tr>
<th></th>
<th>0.3 TBC/m²</th>
<th>1.5 TBC/m²</th>
<th>0.4 TBC/m²</th>
<th>0.8 flexbolts/m²</th>
<th>0.4 - 0.8 flexbolts/m²</th>
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<td>MIN 2 TBC/m²</td>
<td>2 - 4 flexbolts/m²</td>
<td>MIN 4 flexbolts/m²</td>
<td>2 - 5 bolts/m²</td>
<td>MIN 5 bolts/m²</td>
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## PG42’s Loader Gate risk assessment

### Appropriate additional support for the ribs

<table>
<thead>
<tr>
<th>Category Of Remedial / Additional Support Set</th>
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<tbody>
<tr>
<td>Reason for Increased risk (Single Factors)</td>
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<tr>
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<td>S5 - S6 Rib</td>
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<tr>
<td>F1 Fault (minor faulting)</td>
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<td>F2 Fault (major faulting)</td>
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<tr>
<td>Disturbed Lithology (DL)</td>
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<tr>
<td>Roadway Overheight</td>
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<td>Roadway Excessive Height</td>
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<tr>
<td>Strata Water</td>
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### RISK REDUCTION CATEGORY

A = Reduces 'High' risk to 'Low' residual risk
B = Reduces 'High' risk to 'Medium' residual risk, or 'Medium risk to 'Low residual risk
C = Does not reduce perceived risk but may improve conditions
NA = Not Applicable - OR - Not Appropriate support for the perceived risk

### Definitions:

**LD Steelwork** = Light Duty Steel Sets (6" x 5" section) at standard spacings  
**HD Steelwork** = Light Duty Steel Sets (6" x 5" section) 0.6m spacings or closer  
**HD Steelwork** = Heavy Duty Steel Sets (8" x 6" section)

**LD Spotbolts** = Low Density Spotbolts = < 0.8 bolts per square metre (excluding bolts in existing pattern)  
**HD Spotbolts** = Low Density Spotbolts = > 0.8 bolts per square metre (excluding bolts in existing pattern)  
**LD Long Tendons** = Low Density Cable Bolts or Flexible Bolts or Fibreglass Dowels = < 0.4 tendons per square metre of rib  
**HD Long Tendons** = Low Density Cable Bolts or Flexible Bolts or Fibreglass Dowels = > 0.4 tendons per square metre of rib

**NB. Rib over/excessive height**  
With regard to rib overheight, additional spotbolts to be installed at face on development  
With regard to excessive height, additional spotbolts to be installed at face on development and area examined by Rockbolting Eng.  
**Long tendons include:**  
5m long full column grouted cable bolts (steel or fibreglass), hollow fibreglass dowels, ishabech dowels, and fully encapsulated flexible bolts.

### Guide for 3.6m high roadway

<table>
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<tr>
<th>Definition</th>
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<td>0.3 - 0.8 bolts/m²</td>
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<tr>
<td>0.3 tendons/m²</td>
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Roof risk assessment results for PG42's Loader Gate, Thoresby Colliery

Figure 61
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<tr>
<th>Task Taken (zone) No.</th>
<th>Position (m)</th>
<th>RIBS RISK RATING (based on visual/soft indications, not including rib count/rodding support)</th>
<th>REASON FOR INCREASED RISK</th>
<th>And/or where support already installed</th>
<th>REMEDIAL RIBS RISK RATING (aimed at reducing loads)</th>
<th>POTENTIAL RISK RATING FOR FACE RETREAT (potential for ongoing (de)stabilisation)</th>
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<td>766</td>
<td>HIGH</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>B LD</td>
</tr>
<tr>
<td>41</td>
<td>784</td>
<td>HIGH</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>B LD</td>
</tr>
<tr>
<td>42</td>
<td>802</td>
<td>HIGH</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>B LD</td>
</tr>
<tr>
<td>43</td>
<td>821</td>
<td>HIGH</td>
<td>H</td>
<td>H</td>
<td>M</td>
<td>B LD</td>
</tr>
<tr>
<td>44</td>
<td>845</td>
<td>HIGH</td>
<td>H</td>
<td>M</td>
<td>M</td>
<td>B LD</td>
</tr>
<tr>
<td>45</td>
<td>864</td>
<td>HIGH</td>
<td>H</td>
<td>M</td>
<td>M</td>
<td>B LD</td>
</tr>
</tbody>
</table>

Ribs risk assessment results for PG42's Loader Gate, Thoresby Colliery

Figure 62
a) Squarework, ‘Flattop’

Variation in size, section, splay and joint type and position.
Straight legs, flat topped.

b) Cambered (-----) or Cranked (……)

Straight legs.

c) Delta

Upper section of legs angled to reduce length of roof beam.
Beam can be flat, cambered, (-----) cranked (……)
Leg splay can vary.

d) Donisthorpe Arch

e) Double Radius Arch

D shaped.

f) Semi-Circular Arch

D shaped.

Summary of steel work profiles used in UK Coal Mines

Figure 63
ROADWAY SUPPORT DESIGN

- Fully Rockbolted
- Mixed Support
- Steel Arches

REDUCE RISK OF FALL OF GROUND BY:

- Appropriate Ground Assessment and Design Document
- Fully Rockbolted Roadway – Detailed Ground Assessment and Design
- Mixed Support – Detailed Ground Assessment and Design as per Fully Bolted Roadway
- Additional Monitoring as Necessary
- Steel Arches – Simplified Ground Assessment and Design may be Appropriate

Plan for Appropriate Additional Support

ROADWAY DRIVAGE

- Normal Monitoring Procedures
- May Indicate Areas of Increased Risk

HIGH RISK AREAS

- Additional Support
- Additional Monitoring – which may be installed at Face of Heading Until Conditions Improve:
  - Sentinel Bolts
  - Spring Anchored Telltale
- Detailed Monitoring - Local Electronic Reading Telltale System/4 Wire Extensometers
- NDT methods for Rockbolt Integrity
  - Ultrasonics
  - Radio Frequency

COMPLETE ROADWAY

RISK ASSESSMENT FOR FACE RETREAT

- Identify High Risk Areas
- Additional Support
- and/or
- Additional Monitoring

Flow chart to show integrated design and monitoring strategy to reduce the risk of falls of ground
APPENDICES
APPENDIX 1
Risk Assessment for Mixed Support System Roadways

Appendix 1.1
Falls of ground in UK Coal Mine Roadways 1987-Date

Coal Mine Roof Fall Data 1987-2002

Since 1996, roof falls have been reportable incidents and The Mines Inspectorate therefore have a record of the number occurring, the type of excavation and the support system in use.

Prior to this, they only came to general attention if there were casualties, or men trapped, or major disruption to mining operations, or if rockbolts formed part of the support system. British Coal TSRE’s Rock Mechanics Branch, and latterly as RMT, investigated and retain information on most major roadway falls which occurred between 1987 and 1996, together with all falls involving rockbolt support. British Coal also collated records of these falls between 1991 and 1994, for the Mines Inspectorate (British Coal, 1994). This information includes details of the support system in use.

It is therefore possible to estimate the frequency with which falls occurred for each support system type, covering the period 1987-present.

The summary list of roof fall incidents from 1987 onwards in roadways has been compiled from UK Coal, BCC, RMT reports and HMI records. The full list is tabulated in Table A1 below. From April 1996 it should be complete. Before that it includes all roadway falls in which rockbolts formed part of the support, but many smaller falls with standing support may not have been recorded. This is particularly likely to be the case prior to 1991. To this extent the data prior to 1996 is less useful for comparison, but does indicate the range of support circumstances in which falls have occurred.

It should be noted that falls in advancing gate roads, at junctions and in partial extraction room and pillar districts and in general, at small mines, have not been listed.

There are a very large number of falls in partial extraction districts involving prop and bar support, and a few instances with rockbolts or mixed support. At Ellington, for example, there were approximately 65 falls with prop and bar support in room and pillar districts between 1991 and 1994. Since the introduction of rockbolting at Ellington these incidents have been almost eliminated, but continue at small mines using props and bars and timber supports. Small mine fall incidents have therefore not been listed unless the circumstances and support system are relevant to the current study. These instances however remind us that traditional prop and bar support is high risk, even in shallow depth ‘easy’ support conditions, typical of small mines.
<table>
<thead>
<tr>
<th>Year</th>
<th>Site</th>
<th>Support</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>2002/03 (to date)</td>
<td>Rossington B102’s Faceline</td>
<td>Flat top deltas &amp; supplementary rockbolts</td>
<td>Road size 6.3m x 3.3m. 152x127 section at 1.2m centres plus 10 roofbolts and 4 rib bolts. Fall 214-230MM.</td>
</tr>
<tr>
<td>2001/02</td>
<td>Riccall Stanley Main Intake</td>
<td>Rockbolts</td>
<td>Road size 5.0m x 3.2m. 6 x 2.1m roofbolts and 6 x rib bolts per m. Fall from 1740MM of unknown length.</td>
</tr>
<tr>
<td>2000/01</td>
<td>Ellington Loco Road</td>
<td>Delta supports</td>
<td>Supports 5 x 3.5m 18m fall in roadway remodelled 40 yrs ago. Seam convergence a factor.</td>
</tr>
<tr>
<td>2000/01</td>
<td>Hatfield Loco Road</td>
<td>D shaped arches</td>
<td>Supports 3.66 x 3.05m at 0.9m spacing. 5m fall. Road driven 50 years ago, later under ringed. Corrosion a factor.</td>
</tr>
<tr>
<td>2000/01</td>
<td>Longannet Surface drift</td>
<td>D shaped arches</td>
<td>20m fall 700m from top of old drift. Wet conditions. Rotten timber lagging. Arches still in place.</td>
</tr>
<tr>
<td>2000/01</td>
<td>Tower Surface drift</td>
<td>D shaped arches</td>
<td>4.2 x 3.2m arches at 0.9m spacing with concrete slab lagging. 14m fall. 23m cover, very wet. Long term deterioration obscured by internal sheeting.</td>
</tr>
<tr>
<td>1999/2000</td>
<td>Ellington Drift</td>
<td>D shaped Arches</td>
<td>Light section arches at 1.2m spacing. 4m fall in old drift installed &gt;50 yrs. Heavy corrosion.</td>
</tr>
<tr>
<td>1998/9</td>
<td>Aberpergwm In seam development</td>
<td>Steel square work</td>
<td>Fall from face 6m long. 5m x 3.5m steel of 126 x 104mm section 1m spacing. Faulting parallel to advance.</td>
</tr>
<tr>
<td>1998/9</td>
<td>Annesley Return roadway</td>
<td>3 piece steel arches D shape?</td>
<td>6m long fall in 5yr old road due to corrosion at fishplated joints. Water from fault plane. Supports 4.8m x 3.6m at 1.2m spacing.</td>
</tr>
<tr>
<td>1997/8</td>
<td>Calverton Cross measures drift</td>
<td>Steel arches D shape?</td>
<td>4.8 x 3.6m arches. 4m long fall due to long term corrosion-water present.</td>
</tr>
<tr>
<td>1997/8</td>
<td>Wistow 94’s Faceline</td>
<td>Flat top deltas &amp; supplementary rockbolts</td>
<td>6.2m x 3m supports at 1m spacing. Fall 124 –135MM at dint position.</td>
</tr>
<tr>
<td>1997/8</td>
<td>Maltby Maingate</td>
<td>Rockbolts</td>
<td>Rib fall 7m long on retreat.</td>
</tr>
<tr>
<td>1996/7</td>
<td>Riccall Gate road</td>
<td>Rockbolts</td>
<td>Rib fall on development.</td>
</tr>
<tr>
<td>Year</td>
<td>Site</td>
<td>Type</td>
<td>Details</td>
</tr>
<tr>
<td>-------</td>
<td>---------------</td>
<td>-----------------------------</td>
<td>------------------------------------------------------------------------</td>
</tr>
<tr>
<td>1996/7</td>
<td>Blenkinsopp</td>
<td>Steel girders and legs</td>
<td>4.2m x 1.7m road 11m fall at head due to clay filled joints in limestone roof. Supports at 1.2m intervals.</td>
</tr>
<tr>
<td>1996/7</td>
<td>Baddesley</td>
<td>D shape steel arches</td>
<td>13m fall 30m from adit. Water, soft strata, roof cavity. Fall initiated by dinting.</td>
</tr>
<tr>
<td>1994/5</td>
<td>Asfordby</td>
<td>Rockbolts 13 2.4m roof &amp; 6 rib per 1.2m</td>
<td>Road size 4.6 x 2.7m. Fall 674-688MM after 5 months.</td>
</tr>
<tr>
<td>1994/5</td>
<td>Asfordby</td>
<td>Circular Steel rings –large diameter supports</td>
<td>610 x 305 rings plus supplementary bolting. Asymmetric loading –thick seatearth zone.</td>
</tr>
<tr>
<td>1993/4</td>
<td>Kiveton</td>
<td>Arches</td>
<td>12ft x 9 ft arches. 2 falls due to rotten timber lagging.</td>
</tr>
<tr>
<td>1994/5</td>
<td>Kellingley</td>
<td>Rockbolts</td>
<td>Fall on face side from face end to 8m outbye.</td>
</tr>
<tr>
<td>1993/4</td>
<td>Bilsthorpe</td>
<td>Rockbolts</td>
<td>Large fall in skin to skin roadway.</td>
</tr>
<tr>
<td>1993/4</td>
<td>Welbeck</td>
<td>Rockbolts</td>
<td>Small fall in front of face on retreat 866MM.</td>
</tr>
<tr>
<td>1993/4</td>
<td>Thoresby</td>
<td>Rockbolts</td>
<td>Small fall in front of face on retreat 696MM.</td>
</tr>
<tr>
<td>1993/4</td>
<td>Markham</td>
<td>Rockbolts</td>
<td>4m long fall at face due to faulting.</td>
</tr>
<tr>
<td>1992/3</td>
<td>Kiveton</td>
<td>Square work plus wood legs</td>
<td>Old road left open for ventilation.</td>
</tr>
<tr>
<td>1992/3</td>
<td>Longannet</td>
<td>Steel square work plus centre legs supplementary roofbolts</td>
<td>5m long fall during back-ripping repair. Roofbolts 6 x 2.1m at 0.6m.</td>
</tr>
<tr>
<td>1992/3</td>
<td>Silverdale</td>
<td>Steel flat top arches</td>
<td>5.6 x 3.6m arches. 28m long fall interaction effect?</td>
</tr>
<tr>
<td>1992/3</td>
<td>Stillingleaf</td>
<td>Flat top deltas &amp; supplementary roofbolting</td>
<td>Delta size 5.2m x 3.0m 152x127 section at 1.2m centres 127 x 114 legs plus 5 roofbolts. 17m fall.</td>
</tr>
<tr>
<td>1992/3</td>
<td>Ellington: No.4 road</td>
<td>‘Flat top Hollybanks’ steel square work</td>
<td>50yds from D3E conveyor. No other information. 5m long fall.</td>
</tr>
</tbody>
</table>
### Support System Risk Estimates

To allow a fair comparison of the risk associated with each support type we need, ideally, to calculate the total length of roadway in use for each type in each year. To fully achieve this, whilst not impossible, would be a major undertaking requiring lengthy research. However some summary records exist from which we can at least estimate the relative proportions of bolted, arched and rectangular section roads from 1987 up to privatisation at the end of 1994. After that we have been able to make a similar estimate for each support type from RJB/UK Coal mine plans and Supply and Contracts records.

**1996 - present**

The roof fall information does not always distinguish between the two arch profiles, nor between the cambered and flat topped delta supports. In practice there is little difference between the strength of the latter two supports (see section 4.2) and it is considered logical to include cambered supports with flat topped supports, as effectively rectangular profile supports, and express current UK Coal steel support usage data (Section 2.3) as follows:

(a) D shaped Arch profile support: 67%
(b) Rectangular profile support: 33%

Steel arch supported roadways are almost always district and main access roads and therefore long life. Many arched roads forming the basic mine roadway infrastructure are now quite old. Bolted and rectangular profile steel supported roads tend to be gate roads, associated connecting roads or facelines and have a relatively short life, typically up to two years.

For this reason, the percentage of drivage currently undertaken with each support type will not be the same as the percentage of existing roadways with each support.
Inspection of a sample of colliery plans suggest that the relative proportions of support types in existing UK Coal mine roads are:

(a) Rockbolts, 40%
(b) Steel - Arch profile, 50% (more recent with supplementary bolts)
(c) Steel - Rectangular profile, 10% (mainly with supplementary bolts).

These figures are compatible with current drivage statistics, when differences in roadway life are taken into account, and are considered to represent the proportion of each support type in use, from April 1996, with sufficient accuracy to allow risk estimates to be made.

The support system in use for the fall incidents listed in appendix 3 was as follows:
(a) D shaped arches 7
(b) Rectangular profile with supplementary bolting 2
(c) Rectangular profile without bolting 3
(d) Rockbolts 3

The relative risk can be estimated by dividing the number of falls by the percentage of roadway supported. The relative risk ratios (counting the risk in rockbolted roads as 1) are therefore as follows:
(a) D shaped arches  1.9
(b) Rectangular profile with or without supplementary bolting  6.7

Taken at face value, the data suggests that the relative ground control risk associated with D shaped arched profile steel supports is about twice the risk for bolted support, with the risk associated with rectangular profile steel support more than six times greater. However closer examination of the details of the roof fall incidents leads to some revision of the relative risk level estimates.

Six of the seven falls with arched support occurred in old roadways following long term deterioration, and significant support corrosion. If these incidents are discounted for the moment, the revised ground control risk estimate for arch profile roads is very small, suggesting that D shaped arches are potentially a very safe form of support.

The large majority of rectangular profile steel supports are used with supplementary bolting. As there were two roof fall incidents with supplementary bolting and three without, it is possible to conclude that the use of rectangular profile steel support without bolting carries a very high ground control risk.

There were two roof falls involving rectangular profile steel supports used with supplementary bolting, both in faceline drivages. Even assuming all 10% of rectangular profile steel supported drivage was supplementary bolted, the relative risk estimate of this support combination would still be nearly three times higher than for full rockbolt support.

This suggests either that use of rectangular profile roadways with mixed support is relatively high risk, or that the drivage of facelines carries a higher risk. Whilst the latter may be true to an extent, many faceline drivages have been completed with bolted support, without incident. This suggests that the use of rectangular profile steel support with supplementary bolting is associated with significantly increased ground control risk.
1987-1995

The roof fall data prior to 1996 is not necessarily comprehensive, and is therefore less useful for making risk estimates. It is however worth examining in order to check the conclusions reached above, and to examine the range of support circumstances in which falls have occurred.

The relative proportions of roadway support types in use changed significantly between 1987 and 1995. Rockbolts were first introduced as sole support in 1987, but their use did not become widespread until 1991/92, when the AT bolting system became available. However bolts were widely used as supplementary support in conjunction with rectangular profile steel support from the mid 1980s. Consequently the use of rectangular profile steel support was increasing by 1987.

Figures given in a 1991 research report (ECSC 1991) indicate that from 1986 to 1991 rectangular profile steel supported roadways increased from 26% to 57% of roadways driven. This figure probably peaked in that year, as many rectangular gate roads progressed from supplementary to primary rockbolting. Rectangular profile roads would also have been short life, so the arch shaped steel support would have continued to be the main system in existing roads.

Based on the figures above, estimates of the proportions of each system in use in existing roads are as follows:

(a) Rockbolts increasing from zero (1987) to about 30% (1995)
(b) Steel - Arch profile reducing from 85% (1987) to about 55% (1995)
(c) Steel –rectangular profile increasing from 15% in 1987 to 25% in 1991, then reducing again to 15% by 1995, mainly with supplementary bolting

Fall incidents recorded:
(a) D shaped Arches 5
(b) Rectangular profile steel supports with bolting 4
(c) Rectangular profile steel supports without bolting 6
(d) Bolts 6

There were remarkably few roof falls documented in steel arch supported roadways, taking into account the large number in existence over this period. Full details are not available for all the incidents, but long term deterioration was again a factor in several cases. Several falls in arched roads also occurred whilst repair work was in progress.

There was again a disproportionately high number of incidents involving rectangular profile steel supported roads, both with and without supplementary bolting. Four of the recorded incidents occurred on faceline drivages, including two in which mixed support was in use. However two major falls involving mixed support also occurred in roadways being developed for retreat. Taken with the cases in which bolting was not used, they confirm that rectangular profile supports, both with and without supplementary bolting, are associated with increased ground support risk. Although the data also suggests that faceline drivages may be associated with increased risk, this could be explained simply by the use on facelines of ‘high risk’ rectangular profile supports.

The disproportionately high number of reported incidents over this period involving rockbolt support reflects a requirement for all falls of ground involving rockbolts to be reported to BCC headquarters, and on inspection it can be seen that four of the incidents were relatively minor, and similar incidents involving steel supports have
probably not come to attention. In addition the fall at Bilsthorpe occurred in a skin to skin retreat roadway, a system which is no longer used.

During the period 1993/4, procedures for the management of rockbolt monitoring schemes were improved, and these improvements were incorporated into the subsequent Code of Practice (HSE, 1996). It is likely that this change, together with the introduction of flexible bolts as additional support in 1996, have contributed to the reduced ground control risk indicated by post 1996 statistics for rockbolt support.
APPENDIX 1
Risk Assessment for Mixed Support System Roadways

Appendix 1.2
Design of Mixed Support Systems in Rectangular Profile Roadways and Calculation of Steel Support Capacity and Maximum Supported Roof Height

DESIGN OF MIXED SUPPORT SYSTEMS IN RECTANGULAR PROFILE ROADWAYS

Where rockbolts and steel supports are used together in an effectively rectangular profile, it is essential for safety that the rockbolt system remains effective as support. Therefore rockbolts should be considered to be the primary support in a mixed support system and designed using the same procedures as for a fully rockbolted roadway. The role of the steel support should be considered, in normal circumstances, to be limited to support of the immediate roof up to a maximum of one third of the bolted height (the safe supported height). The safe supported height may be less than this in exceptional circumstances (wide roadway, low capacity steel support, and long bolts) and in this situation would need to be determined by calculation.

The one third bolted height limit can be assumed to be valid within the following limits:

(i) Use of UK Coal standard supports No.’s 1-4 or other similar supports with minimum of 152 x 127mm section, 37.2kg/m weight and fishplated joints, properly installed and well packed to the roof and sides. *(note: excludes props and bars, and squarework with clip legs, corner brackets, or single bolt joints).*

(ii) Maximum roadway width of
   (a) 5.9m with supports at 1.2m maximum centres,
   (b) 6.5m with supports at 1.0m maximum centres and
   (c) 8m with supports at 0.6m centres.

(iii) Maximum bolt length of 2.4m

If one or more of these limits is exceeded, the safe supported height should be checked by calculation as detailed below.

*Note: The reason for the one third bolt height limit is to ensure the bolt system is still effectively supporting the roof above the immediate roof zone supported by the steel support.*

CALCULATION OF STEEL SUPPORT CAPACITY AND MAXIMUM SUPPORTED ROOF HEIGHT

The role of rectangular profile steel support in a mixed support roadway should be considered to be limited to support of the immediate roof up to one third of the bolted height, or to the maximum supported roof height, whichever is less. This value is the safe supported roof height.

The maximum supported roof height is the maximum height of roof strata, which the steel can support before failure. In most cases this will be greater than one third bolt
height, in which case it does not need to be calculated. However, if one or more of the
limits listed in Appendix 4 are exceeded, there is a possibility that the maximum
supported roof height will be less than one third the bolt height, and this needs to be
checked by calculation using the method below.

The method of calculation was derived by British Coal and described in ECSC final
approximate collapse load of the support in question, using assumptions as follows:

\[
\text{Collapse Load } W_c \ (kN) = W_o + 4M_r/x
\]

Where:
- \(W_o\) = collapse load of a simply supported beam
- \(M_r\) = moment of resistance of beam end restraint (kNm)
  (For fishplated joints, \(M_r\) is assumed to be 90kNm)
- \(x\) = beam length (m)

\[
W_o \ (kN) = 1.98Z/x
\]

Where:
- \(Z\) = section modulus \((\text{cm}^3)\)
  - For 152 x 127mm, 37.2kg/m beams, the section modulus is 248 \(\text{cm}^3\)
  - For 203 x 152mm, 52.3kg/m beams, the section modulus is 472 \(\text{cm}^3\)

The calculation assumes beam failure by bending due to point loading at it’s centre,
which for most practical situations will represent a ‘worst case’ giving the lowest failure
load.

The equivalent height that the steel can support is based on the roof area supported
(roadway width x support spacing) and the unit weight of rock as follows:

\[
\text{Height supported} = \frac{\text{Collapse Load(kN)}}{\text{Area Supported(m}^2\)} \times \frac{\text{Unit weight of Rock(kN/m}^3\)}
\]

Where:
- Unit Weight of Rock = 25kN/m\(^3\)

This assumes that a rectangular cross section of roof of thickness equal to the
supported height is point loading the steel support beam at its centre. This again
represents a ‘worst case’ situation, and no additional factor of safety has therefore
been applied. In order to achieve the calculated capacity, the steel support needs to be
properly installed, be undamaged and in good condition and be well packed to the roof
and sides. Tables A2 and A3 below list for information the calculated support capacity
and equivalent maximum supported roof height for UK Coal standard supports
Numbers 1 to 4 for 6”x5” and 8”x6” steel sections respectively.
Table A2 Approximate steel support capacity and maximum supported roof height for 152 x 127mm (6"x5") Section Steel

<table>
<thead>
<tr>
<th>UK Coal Standard Number</th>
<th>Effective Roadway Width (m)</th>
<th>Restrained Collapse Load (kN)</th>
<th>Maximum Supported Roof Height (m) 0.6m spacing</th>
<th>Maximum Supported Roof Height (m) 1.0m spacing</th>
<th>Maximum Supported Roof Height (m) 1.2m spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Standard Delta 4100/3400</td>
<td>5.1</td>
<td>208</td>
<td>2.72</td>
<td>1.63</td>
</tr>
<tr>
<td>2</td>
<td>Standard Delta 4100/3000</td>
<td>5.1</td>
<td>208</td>
<td>2.72</td>
<td>1.63</td>
</tr>
<tr>
<td>3</td>
<td>Standard Delta 5500/3000</td>
<td>6.5</td>
<td>155</td>
<td>1.59</td>
<td>0.95</td>
</tr>
<tr>
<td>4</td>
<td>Standard Delta - Cambered</td>
<td>6.2</td>
<td>190</td>
<td>2.04</td>
<td>1.22</td>
</tr>
</tbody>
</table>

It can be seen that the calculated maximum supported roof height exceeds one third of the bolted height for bolts 2.4m long or less in each case, with the exception of Standard Support 3 at 1.2m centres.

If 2.4m long bolts are used, the safe supported roof height is therefore 0.8m (one third of the bolted height) in all cases except for Standard Support 3 at 1.2m spacing where it is 0.79m.

If 2.1m bolts are used, the safe supported roof height is 0.7m (one third of the bolted height) in all cases.

Table A3 Approximate Steel Support Capacity and Maximum Supported Roof Height for 203 x 152mm (8"x6") Section Steel

<table>
<thead>
<tr>
<th>UK Coal Standard Number</th>
<th>Effective Roadway Width (m)</th>
<th>Restrained Collapse Load (kN)</th>
<th>Maximum Supported Roof Height (m) 0.6m spacing</th>
<th>Maximum Supported Roof Height (m) 1.0m spacing</th>
<th>Maximum Supported Roof Height (m) 1.2m spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Standard Delta 4100/3400</td>
<td>5.1</td>
<td>315</td>
<td>4.12</td>
<td>2.47</td>
</tr>
<tr>
<td>2</td>
<td>Standard Delta 4100/3000</td>
<td>5.1</td>
<td>315</td>
<td>4.12</td>
<td>2.47</td>
</tr>
<tr>
<td>3</td>
<td>Standard Delta 5500/3000</td>
<td>6.5</td>
<td>235</td>
<td>2.41</td>
<td>1.45</td>
</tr>
<tr>
<td>4</td>
<td>Standard Delta - Cambered</td>
<td>6.2</td>
<td>289</td>
<td>3.11</td>
<td>1.86</td>
</tr>
</tbody>
</table>
It can be seen that the calculated maximum supported roof height exceeds one third of
the bolted height for bolts 3m long or less in every case.

If 2.4 m long bolts are used, the safe supported roof height is therefore 0.8m (one third
of the bolted height) in all cases.

If 2.1m bolts are used, the safe supported roof height is 0.7m (one third of the bolted
height) in all cases.