P.R. Helm, C.T. Davie, S. Glendinning. Numerical modelling of shallow abandoned mine working subsidence affecting transport infrastructure, *Engineering Geology* 2013, (154), 6-19.

Copyright:

This is the author's version of a work that was accepted for publication in Engineering Geology. Changes resulting from the publishing process, such as peer review, editing, corrections, structural formatting, and other quality control mechanisms may not be reflected in this document. Changes may have been made to this work since it was submitted for publication. A definitive version was subsequently published in Engineering Geology, 154, (2013)

DOI link to article:

<http://dx.doi.org/10.1016/j.enggeo.2012.12.003>

Date deposited:

03/03/2015

This work is licensed under a

[Creative Commons Attribution-NonCommercial-NoDerivs 3.0 Unported License](https://creativecommons.org/licenses/by-nc-nd/3.0/)

Numerical modelling of shallow abandoned mine working subsidence affecting transport infrastructure

Authors: P.R. Helm^{a*}, C.T. Davie^a, S. Glendinning^a

^aSchool of Civil Engineering and Geosciences, Newcastle University, Newcastle Upon Tyne, UK.

NOTICE: this is the author's version of a work that was accepted for publication in *Engineering Geology*. Changes resulting from the publishing process, such as peer review, editing, corrections, structural formatting, and other quality control mechanisms may not be reflected in this document. Changes may have been made to this work since it was submitted for publication. A definitive version was subsequently published in *Engineering Geology*, 154, (2013) DOI:10.1016/j.enggeo.2012.12.003

Notes:

Article first published online: 23 December 2012

Article allocated to a volume and published: 28 February 2013

Journal: *Engineering Geology*

Year: 2013

Volume: 154

Pages: 6-19

DOI link: <http://dx.doi.org/10.1016/j.enggeo.2012.12.003>

Comments:

This document is the post-peer review, pre-journal typesetting version of the paper. In this instance it includes the text, figure and table corrections as specified following the peer-review process.

Numerical modelling of shallow abandoned mine working subsidence affecting transport infrastructure

Authors: P.R. Helm^{a*}, C.T. Davie^a, S. Glendinning^a

^aSchool of Civil Engineering and Geosciences, Newcastle University, Newcastle Upon Tyne, UK.

***Corresponding author contact information:**

Email: P.R.Helm@dunelm.org.uk

Abstract

This work presents details of a shallow mining subsidence event that occurred in the summer of 2001 causing the formation of crown-holes at the surface which affected the East Coast Main Railway line in the UK. This subsidence event caused significant disruption and the remediation effort required the construction of a 1.8 km long diversion built on a piled, reinforced concrete raft. Details of the ground investigation are summarised along with a large parametric numerical modelling study undertaken in FLAC 3D into the potential causes of the instability, including the role of variations of the level of the groundwater table, the influence of the structure of the rock mass and also the potential geometry of the abandoned workings. Ultimately the modelling allowed constraints to be placed on the likely excavation width of the workings at the site along with bedding spacing and strength of the overlying rock mass. The modelling also suggests that the increase in the ground water table may also have been a factor in the occurrence of instability on the site.

Keywords: Shallow mining subsidence; Abandoned colliery workings; Crown-holes; Numerical modelling

1. Introduction

Coal mining has played a significant role in shaping the economic development of the United Kingdom (UK), most significantly during the industrial revolution. Although vital to the economic development of the UK in the past, there are negative impacts of mining and of concern is the threat to safety posed by the collapse of subsurface voids. As such the legacy of past mining is an increasingly important issue, where the threat of mining subsidence is recognised to be of concern not just in the UK but globally (Bell, 1992 and Jones et al., 2005).

Mining began on a significant scale in the UK in the 13th century (Bell and De Bruyn, 1999). By the 15th and 16th centuries the pillar and stall method of extraction became common (Bell and De Bruyn, 1999 and Healy and Head, 2002). In this process, a mineral is extracted, commonly in a regular pattern, while pillars of the mineral deposit (in this case coal) are left in place to support the roof of the mine (Attewell and Taylor, 1984). Pillar and stall mining has largely been replaced in the UK by longwall mining where the roof of the workings is allowed to collapse in a controlled manner as mining progresses. As such any subsidence due to pillar and stall workings within the UK is likely to be associated with mine workings abandoned over a century ago (Attewell and Taylor, 1984 and Waltham, 1989). Due to the large number of abandoned mine workings in the UK (with estimates as high as 70,000 (Deb and Choi, 2006)) where the structural integrity/stability of the workings is unknown it is clear that these pose a significant engineering and safety issue.

Best practice in assessing the subsidence hazard posed by shallow mine workings is currently based on the empirical evidence available from previous failures (Bell, 1975, Attewell and Taylor, 1984, Waltham, 1989, Whittaker and Reddish, 1989 and Healy and Head, 2002). These may be useful tools where data is limited, but do not easily provide a rigorous assessment of collapse hazards at a particular site. Where increased site investigation data is available it is possible to undertake more detailed analysis and, as demonstrated in this work, this can better constrain the hazard posed by abandoned workings. In this paper an example of shallow abandoned mining subsidence that had a significant impact on transport infrastructure in Scotland, is investigated through a numerical modelling study in order to better understand the causal mechanisms and significant factors in this event. The findings of this work are used to inform wider applicability to practising engineers when considering the stability of shallow abandoned mine workings, and the modelling, monitoring and site investigation of such phenomena, and in identifying the key factors and triggers of void migration, subsidence and collapse.

2. Shallow Abandoned Colliery Working Subsidence

The typical geometry of Scottish pillar and stall (or stoop and room) workings can be seen in Figure 1 where the stoops (coal pillars) supporting the roof are square in plan and the rooms are of a regular repeating pattern (Healy and Head, 2002).

Figure 1: Typical geometry of Scottish pillar and stall / stoop and room workings.

There are three main mechanisms of deterioration and collapse of abandoned mine workings (floor heave, pillar crushing and roof collapse) and of these, roof collapse due to the disintegration and deformation of roof strata is the primary closure mechanism in shallow workings, and is considered the greatest problem in subsidence engineering practice (Healy and Head, 2002).

In order for void migration to occur, the roof of the workings must fail. Prior to the formation of any cavity, the stresses in the earth will reach an equilibrium state. Formation of a void causes the overburden load to be taken by the pillars causing tensile stresses to develop in the immediate roof, and compressive stresses to build at the upper corners of the workings (Attewell and Taylor, 1984, Twiss and Moores, 1992 and Dyne, 1998). This can cause the overlying strata to become fractured. This fractured material may be made stable by the confining stress; however any disturbance in the in-situ stress at this point can then lead to further roof failure (Thigpen, 1984). It is the potential changes in these conditions that led to a disturbance in the initially stable excavations at Dolphingstone that are investigated herein.

Typically once void migration/roof failure has commenced it will continue unless arrested by natural arching, choking of the void by collapse debris or encountering a high strength layer capable of spanning the void (Bell, 1992).

The geometry of the failure is controlled by the presence and orientation of joints in the rock mass, particularly bedding spacing and the presence of sub-horizontal steeply dipping joints. If steeply dipping joints are present with an unfavourable orientation, then roof failure may occur by shearing along these discontinuity surfaces (see Whittles et al., 2007 for a summary of differing roof failure mechanisms). In situations where steeply dipping discontinuities are

not present then the failure mechanism is dominated by snap through or buckling failure and occurs because as a roof layer fails, it flexes downwards due to yielding (the degree of actual deformation is dependent on the thickness and stiffness properties of the roof beam). This leads to the development of cracks at the point where the roof strata meets the pillar (Goodman, 1989). Fractures also develop at the roof centreline on the base of the layer forming the roof. The initial yielding and sagging of the strata can be seen in Figure 2 and due to the nature of the stress field at the ends of the roof layers, these cracks propagate diagonally away from the pillars into the strata above the excavation (Goodman, 1989). Ultimate collapse of a roof beam leaves a pair of cantilevers as abutments for the overlying roof strata, effectively reducing the span of the excavation. This progression can be seen in Figure 3.

Figure 2: Yielding of roof strata over an excavation.

Figure 3: Progressive failure leading to void stabilisation by arching.

Continued failure and roof collapse will naturally lead to a stable, conical void assuming sufficient height to rock head (Goodman, 1989) and that choking by bulking of collapse debris or bridging of the void by high strength strata in the overlying rock mass do not occur. Where there is insufficient height to rock head (normally considered to be cases where the rock mass

overlying the void is less than 30 m thick or 10 times the worked seam thickness) and where neither bridging or bulking occur then the void may migrate to surface forming a crown-hole. This mode of failure has been observed in the field (Healy and Head, 2002) and has also been successfully demonstrated in laboratory physical modelling (Bieniawski, 1984).

3. Subsidence at Dolphingstone

During May and June 2001 a pair of crown-holes of approximate diameter 1.5–2 m were found adjacent to the East Coast Main Line (ECML) track at a site near Dolphingstone between Prestonpans station and Wallyford Cutting (Dolphingstone is a village located approximately 4 km East of Edinburgh in East Lothian in Scotland, Ordnance Survey Grid Reference NT 381732). The location can be seen in more detail in Figure 4.

Figure 4: Map showing the location of Dolphingstone, the ECML, Prestonpans Station and Wallyford Cutting (Helm, 2010) © Crown Copyright/database right 2012. An Ordnance Survey/EDINA supplied service.

The presence of these crown-holes prompted the commissioning of a site investigation in order to better understand the causes of the subsidence on site.

During the desk study phase of the site investigation, it was found that historical mining had taken place on or adjacent to the site. The presence of crown-holes and the confirmation of historical mining activity and subsidence at the site, coupled with the risk of further subsidence, prompted the instigation of a ground investigation (GI) which commenced in February 2002.

A preliminary GI was undertaken with rotary open and cored holes on both the north and south bound lines during night time possessions. This confirmed the presence of mine voids at shallow depths (≈ 10 m) below the railway with only minimal rock cover.

The presence of these voids was considered a significant hazard and prompted Railtrack (now Network Rail) to impose a temporary 32 km/h speed limit along the affected portion of the line as well as commissioning a detailed ground investigation with the aim of further

investigating subsurface conditions. Based on the presence of subsurface voids and the occurrence of further crown-holes it was established that the most likely cause of the crown-hole subsidence was the collapse of abandoned mine workings beneath the area (Donaldson Associates Ltd., 2002). Ultimately, Network Rail diverted the ECML over a distance of 1.8 km at a cost of £56 million (Network Rail, 2003). In order to investigate the possible trigger for this subsidence event, a numerical modelling investigation was also undertaken based on the results of the intrusive ground investigation.

3.1 Dolphingstone Site Geology

The site investigation report (Donaldson Associates Ltd., 2002) along with the available geological maps indicate that the site is covered by Devensian age superficial deposits composed of high raised beach deposits to the north of the railway line and glacial till to the south. These superficial deposits are underlain by the Upper Carboniferous age Limestone Coal Formation composed of sandstones, siltstones, mudstones, coals and ironstone.

Trial pits and cable percussion holes were used to investigate the superficial deposits which in general comprise a topsoil layer of between 0.3 and 0.6 m thickness overlying fine to medium grained loose to moderately dense silty sands of approximately 1 to 2 m thickness in turn overlying a stiff to very stiff glacial till of 2 to 3 m thickness.

Rotary coring revealed that the subsurface strata predominantly consisted of interbedded sandstones, siltstones and mudstones with frequent coal seams. These coal seams ranged in thickness from 0.1 to 2 m and workings were found in the vast majority of them. In all cases, the workings were filled with groundwater.

A review of the crown-hole location plan and comparison with the rotary borehole location plan indicated that a number of crown-holes (4 No.) had formed approximately 50 m to the west of Preston Grange crossing and were also located in close proximity to a rotary borehole. Three of these crown-holes were recorded as having formed during the intrusive GI process (possibly as a result of vibration or high pressure drill flush) and so were not considered further. However the fourth, described as typical of the type found at the site (surface diameter of 2 m; depth of 0.3 m) was believed to have formed before the SI process as it was over grown with vegetation when discovered unlike the other crown-holes.

It was decided to use the log from this borehole (see Figure 5) to derive the stratigraphy for the numerical model as a number of crown-holes had developed in close proximity and as this bore hole appeared to capture the general geology and structure of the site (i.e. voids at shallow depth with limited rock cover).

3.2 *In-situ* and Laboratory Test Results on Soils and Rocks

The results of in-situ and laboratory testing undertaken on the superficial deposits and solid geology is summarised in Table 1.

Figure 5: Core log from borehole located in close proximity to crown-hole of interest.

Table 1: Summary of *in-situ* and laboratory test results on soils and rocks.

Material	Depth	Description	c'	ϕ'
Sands	0.25 m–3.0 m	Standard and cone penetrations tests (SPT and CPTs) indicative of loose to very loose sand below the topsoil down to 1 m underlain by medium dense to dense sand. Particle size analysis indicated silty sand with the silt content increasing with depth.	2–11 kPa	35.5° to 38.5°
Glacial Till	3.0 m–6.2 m	SPTs suggested a C_u of 150-450 kPa. CPTs gave cone resistance values from 3-10 MPa which indicate firm to very stiff clay.	10 kPa	29.5°
Moderately Strong Sandstone	6.2 m–8.0 m	Slightly weathered, containing sub-horizontal clay infilled joints. Uniaxial compressive strength (UCS) testing gave a strength of 11.4 MPa. Clay filling material ϕ' from 15° to 30° with residual values as low as 11° (Brady and Brown, 1993; Goodman, 1989; Zhang, 2005)		
Interbedded Sandstones, Siltstones and Mudstones ^a	> 9.2 m	Sandstone and siltstone described as moderately strong to strong and contain mudstone lamina, suggesting a UCS value of 50-100 MPa. The mudstone is described as moderately strong to moderately weak suggesting a UCS of 5-50 MPa (British Standards Institution, 1999).		

^a No laboratory test data was available for these rock types

3.3 Groundwater Table

The Dolphingstone hydrogeological report (Edmund Nuttall Ltd., 2002) indicates that the seasonal variation of the water table for Prestonpans (to the east of the site) is approximately 1.5 m. Groundwater levels at the Wallyford site (near the western boundary of the area of interest) are plotted in Figure 6 showing a minimum water table during August 2004 of around 26 m (Piezometer 1), increasing to a maximum of approx. 27.8 m in mid-March 2005. This again shows a seasonal variation of a little over 1.8 m. It is possible that the water level increases beyond this value, however there is missing data in March 2005.

The mine workings at the site and in the surrounding area are all below the level of the groundwater table and are recorded as being flooded in both the site investigation and hydrogeological reports and originally required drainage to maintain a water table below the base of the excavations.

Figure 6: Variations in the groundwater table at Wallyford cutting at the west of the Dolphingstone site (Personal communication, Keith Whitworth).

4. Numerical Modelling of the Dolphingstone Collapse

In order to investigate the conditions that may have led to the initiation of shallow mine subsidence at the Dolphingstone site a numerical modelling study was undertaken using FLAC 3D 3.0 which includes the inbuilt programming language FISH that allows the user to customise model behaviour. Due to the large number of unknowns (for example the width of workings) and the potential for variability of the coal measures strata in terms of strength and stiffness properties along with bedding/discontinuity spacing, the influence of these different variables was investigated in a parametric study.

4.1 Model Geometry and Mesh Discretisation

The numerical models used in this work represent a void of varying width at 8 m depth below surface, with 10 m of underlying rock mass material. The analysis considered two-dimensional sections of the problem geometry assuming plane strain conditions and was undertaken using FLAC 3D. The geometry parameters are outlined in Table 2.

Table 2: Numerical model geometry parameters.

Parameter	Value
Room Width	1, 2 and 3 m
Resultant Model Excavation Span (corner to corner)	1.41, 2.42 and 4.24 m
Excavation Height	1 m
Pillar Width	1, 2 and 3 m
Finite Difference Zone Size	0.1 m ²
Number of Discrete Strata within overburden rock mass	2, 4, 10, 20
Discontinuity Spacing	1.0, 0.5, 0.2 and 0.1 m

As roof collapse occurs via the failure of individual roof bedding planes in a progressive manner as discussed in Section 2 and demonstrated in Figure 2 and Figure 3, interface elements along which sliding or separation can occur were incorporated into the model (for more details see Itasca, 2005) to allow the model to behave in a discontinuous manner so as to more closely approximate the collapse behaviour of layered strata such as those that compose coal measures rocks. The discontinuity spacing values chosen broadly correspond to the thin, medium and thick descriptors used in BS5930:1999 (British Standards Institution, 1999) for the description of bedding plane thickness. The model geometry including the geological profile and excavated mine void is illustrated in Figure 7. The vertical boundaries represent planes of symmetry and were fixed to prevent horizontal displacements and the base of the model was fixed to prevent vertical displacements. Due to the likely occurrence of large strains/displacement within the model, modelling was undertaken in large strain mode.

Figure 7: Numerical representation of the site geology and mine void. Phreatic surface 3.0 m below surface.

4.2 Soil Parameters

The soil parameters were derived from information in the site investigation report where possible. Where this was impossible, they are assumed from published literature (Tomlinson, 2001, Craig, 2004 and Itasca, 2005) and are summarised in Table 3.

Table 3: Soil Properties as used in the Dolphingstone numerical modelling work.

	Bulk Modulus	Shear Modulus	Cohesion	Friction Angle	Tensile Strength	Density
Silty Clay	3.33	2.0	11.0 ^a	38.5 ^a	0.0	1540.0
Silty Sand	33.3	20.0	2.0 ^a	30.5 ^a	0.0	1540.0
Sand, Gravel & Clay	33.3	20.0	3.0 ^a	35.5 ^a	0.0	1620.0
Silty Gravel	14.0	8.4	3.0 ^b	29.5 ^b	0.0	1620.0
Glacial Till	33.0	20.0	10.0 ^a	29.5 ^a	0.0	1620.0
	(MPa)	(MPa)	(kPa)	(°)	(kPa)	(kgm⁻³)

Strength parameters^a - Donaldson Associates, 2002; density values from Tomlinson, 2001; Strength parameters^b and stiffness values from Itasca, 2005.

4.3 Rock Mass Strength, Stiffness and Density Parameters

The medium strength sandstone overlying the void/coal seam has a uniaxial compressive strength of 11.4 MPa. As this bed is immediately overlying the mine void, and is the only layer of competent rock before the superficial deposits, it is likely to be the key factor in determining the stability or otherwise of the excavation roof and whether void migration occurs. The sandstone in this region of the site contains sub-horizontal joints with clay infills which are likely to act to reduce the shear strength properties of the rock mass. As such, a suite of rock mass strength and stiffness properties were created in ROCLAB (Rocscience, 2010) using the Hoek–Brown criterion (Hoek and Brown, 1997) for a range of rock mass quality values estimated based on a range of Geological Strength Index values (Hoek et al., 2005) to give a full potential range of rock mass strength properties.

The rock mass is modelled as a strain softening material which uses the standard Mohr–Coulomb parameters with tensile cutoff with strain softening in the form of user specified reductions in friction, cohesion and tensile strength values at user specified plastic shear and tensile strain increments.

The confining stress dependent tunnel form of the Hoek–Brown criterion (Hoek et al., 2002) was used to derive the Mohr–Coulomb input parameters for the model due to the shallow depth of the workings involved. The intact uniaxial compressive strength of the sandstone layer as measured in the laboratory was also used as an input to the programme. Finally an estimate of the intact elastic modulus of sandstone of 13 GPa (Goodman, 1989, Brady and Brown, 1993 and Zhang, 2005) was used as an input parameter to allow estimates of the rock mass elastic modulus at varying GSI values.

As both the uniaxial compressive strength and tensile strength of rock are affected when it is saturated with water, it was necessary to account for this when deriving the strength

properties. Zhang (2005) reports that the strength reduction can vary from as little as 3% to as much as a 50%. Vasarhelyi (2003) and Li and Reddish (2004) report a 25% to 26% reduction in uniaxial compressive stress between dry and saturated samples of sandstone. In the modelling work undertaken here, both dry and saturated strength parameters are derived for a given rock mass. In this case the uniaxial compressive strength of the sandstone undergoes a 25% decrease (selected as a mean of the values reported above) as the water table rises through it and an automated function was written to alter the rock mass strength properties to the saturated values below the water table.

The post peak strength and friction properties were estimated from literature (Goodman, 1989, Brady and Brown, 1993 and Zhang, 2005) along with the methodology suggested by Cai et al. (2007) where the residual strength of a rock mass can be estimated based on the minimum value of GSI. The strains at which shear softening occurs, are derived based on a review of parameters used for the modelling of similar problems in the literature (Badr et al., 2003, Esterhuizen and Karacan, 2005 and Singh and Singh, 2009). All use values in the range of 10–50 millistrains. The tensile softening parameter was derived from values reported by Zipf, 2005 and Zipf, 2006 where tensile strength dropped to zero at 1 millistrain which is in broad agreement with the laboratory results reported for various rock types by Okubo and Fukui (1996), with tensile strength loss occurring between 0.25 and 0.75 millistrains. Based on the above, the cohesion and friction properties are assumed to drop to a minimum residual value equivalent to a GSI of 5 for the rock mass, at a plastic shear strain of 0.05 (50 millistrains or 5% strain) and the tensile strength falls to a residual value of zero at a plastic tensile strain of 0.01 (10 millistrains or 1% strain).

It should be noted that as a strain softening model was used in this work, the issue of mesh dependency in the results had to be addressed. Mesh dependency can be considered to present two issues. Firstly, in a model composed of a mesh of varying zone sizes, localisation of softening tends to be attracted to smaller zones and failures may therefore be predicted in areas where they would not in reality occur. This problem was avoided here by developing meshes in which the zone sizes were uniform throughout the region where strain softening was likely to occur.

Secondly, however, the chosen size of zones in a uniform mesh can also influence the overall development of softening and the model parameters must be consistent with the mesh size in order to capture realistic behaviour. This issue is addressed in this work by tuning the zone size and the strain softening parameters using published data (Badr et al., 2003, Esterhuizen and Karacan, 2005, Zipf, 2005, Zipf, 2006 and Singh and Singh, 2009). The potential effects of mesh dependency are therefore minimised.

The normal stiffness (k_n) and shear stiffness (k_s) (GPa m^{-1}) for differing rock mass strengths which are a required input property of the interface elements were estimated from the Young's moduli of the intact rock and rock mass (E_r and E), the Shear moduli of the intact

rock and rock mass (G_r and G) and the joint spacing (Int_s) respectively using the following equations (Itasca, 2005):

$$k_n = \frac{EE_r}{Int_s(E_r - E)} \quad k_s = \frac{GG_r}{Int_s(G_r - G)} \quad 1 \& 2$$

The interface cohesion (based on typical joint properties for British coal measures rocks) was taken from Zhang (2005) as a value of 12 kPa. Friction angles for the interfaces were derived from published values of common joint fills (Brady and Brown, 1993 and Zhang, 2005).

The interface friction angles were scaled relative to the strength of the surrounding numerical model zones representing the rock mass where the lowest friction angle (45°) for the matrix, at a GSI of 10, would have a joint friction angle of 20° (chosen as an average of the lowest common peak friction angles in sedimentary rocks as summarised by Zhang, 2005). The GSI values equivalent to higher strength rock masses (GSI 60–100, suggesting a rock mass with rough, relatively fresh, unweathered joint surfaces) were allocated joint friction angles equivalent to the maximum peak values for quartz rich sedimentary rocks with rough unfilled, closely spaced joint surfaces (38°) as suggested by Hoek and Bray (see Zhang, 2005). The intermediate interface friction angles were scaled accordingly.

The coal was assumed to have a uniaxial compressive strength (UCS) of 15 MPa (Bell, 1975) and the UCS of the underlying rock was estimated from driller's descriptions using BS5930:1999 to be 37.5 MPa. The Mohr–Coulomb parameters were estimated using the Hoek–Brown method (Hoek et al., 2002) as applied by ROCLAB (Rocscience, 2010) and are summarised in Table 4. The full range of Mohr–Coulomb rock mass strength and stiffness properties as derived by ROCLAB and used in this modelling for the moderate strength sandstone can be seen in Table 5.

Table 4: Strength and stiffness properties as used for the coal and underlying rock mass

Rock Type	Bulk Modulus	Shear Modulus	Cohesion	Friction Angle	Tensile Strength
Coal	1.47	0.38	0.498	49	0.26
Underlying Rock Mass	9.39	2.45	1.21	51	0.651
	(GPa)	(GPa)	(MPa)	(°)	(MPa)

Table 5: Strength and stiffness property suite as used for the sandstone during the parametric study.

GSI	Bulk Modulus	Shear Modulus	Cohesion ^a		Friction Angle ^a		Tensile Strength		Interface Friction Angle
			Dry	Sat.	Dry	Sat.	Dry	Sat.	
10	0.55	0.14	0.026	0.023	45.0	43.0	0.76	0.57	20.0
20	0.82	0.22	0.039	0.036	51.0	49.0	1.6	1.0	26.0
30	1.47	0.38	0.054	0.048	55.0	53.0	3.5	3.0	30.0
40	2.88	0.75	0.071	0.062	57.0	56.0	7.3	6.0	33.0
50	5.55	1.45	0.098	0.083	60.0	58.0	16.0	12.0	35.0
60	9.39	2.45	0.15	0.12	61.0	60.0	33.0	25.0	37.0
70	13.23	3.45	0.25	0.19	62.0	61.0	70.0	50.0	38.0
80	15.90	4.15	0.45	0.34	62.0	61.0	150.0	110.0	37.0
90	17.31	4.52	0.84	0.64	61.0	60.0	320.0	240.0	37.0
100	17.95	4.68	1.60	1.21	60.0	59.0	680.0	510.0	35.0
	(GPa)	(GPa)	(MPa)		(°)		(kPa)		(°)

^aMohr-Coulomb shear strength parameters are fitted from the Hoek-Brown criterion at low confining stress leading to high friction angles and low cohesion.

4.4 Groundwater Table

The water table at Dolphingstone was recorded at approximately 3 m below the surface in the borehole used to develop the ground model. The site investigation and hydrogeological reports (Donaldson Associates Ltd., 2002 and Edmund Nuttall Ltd., 2002) state that the groundwater recharge at the site is predominantly reliant on precipitation, which is inherently variable and had demonstrated seasonal variability of up to 1.8 m. As such, the phreatic surface was modelled at varying levels from 2 m below the excavation/void (simulating the conditions at mine abandonment) to the surface, which would capture the full potential range of groundwater levels on site from mine abandonment onwards.

In this modelling the water pressure is assumed to be hydrostatic, and to account for the support of the water in the void, a normal stress equal to the weight of the overlying water column is applied to the excavation floor and roof. A normal stress gradient is also applied to the excavation wall (i.e. the coal pillar) which varies linearly from the maximum value calculated for the floor to the minimum value calculated for the roof. The above process is automated using a FISH function during the modelling runs. For more information on the effects of internal and external pore water pressures on tunnel lining support and resultant deformations see Carranza-Torres and Zhao (2009).

4.5 Modelling Methodology

The parametric study to investigate the instability that occurred at Dolphingstone was run using an automated FISH routine which varied the geometry along with the strength and stiffness properties of the model automatically while stepping to either static equilibrium, which represents a stable solution without collapse (the model is assumed to be stable when the unbalanced force ratio is less than 1×10^{-5} — see Itasca, 2005 for more details), or to an unstable state where a prescribed level of vertical displacement within the roof strata was used as a cut off to represent roof instability. This value was chosen as a limiting value corresponding to a midspan deflection of 10% of the beam thickness as is commonly recommended as the limit of stability for voussoir arches in mine roof design (Hutchinson and Diederichs, 1996, Diederichs and Kaiser, 1999 and Swart and Handley, 2005). In order to ensure that deformation and yielding were as a result of collapse a minimum cut off value of ten times this value (100% vertical tensile strain within the excavation roof) was selected as a criterion for failure.

In this work, the modelling iterated through a number of parameters, nested as follows:

- A. Bedding thickness (0.1 m, 0.2 m, 0.5 m, 1.0 m)
- B. Excavation Width (1 m, 2 m, 3 m)
- C. Rock mass strength (10, 20, 30, 40, 50, 60, 70, 80, 90, 100)
- D. Rising water table (1 m increments)

Ultimately this resulted in 1320 model runs ($4 \times 3 \times 10 \times 11$).

5. Results of the Parametric Study on the Stability of the Site at Dolphingstone

In this section the results of the parametric study are summarised. Parameter sets where the instability or collapse criteria were exceeded, or those of further interest, are reported in more detail.

Based on the relatively wide variation in the degree and type of failure that can occur within a numerical representation of a rock mass with variable interface spacing, a number of terms were developed to describe the degree of yielding within the rock mass. These were derived after examining the modelling results in order to allow a way of rapidly categorising the large number of results in terms of their degree of stability and are summarised in Table 6 and illustrated in Figure 8.

Table 6: Descriptive terms used for varying degrees of failure in the modelling.

	Descriptive Term	Yield Description
1.	Stable	No significant deformations (or plastic yielding of the rock mass). See Figure 8A
2.	Yielding	The model has exceeded the user defined yield criterion for axial strain in the roof centre line. However this does not involve significant deformations of the whole rock mass (<i>i.e.</i> displacements < 0.2 m). See Figure 8B
3.	Moderate Yielding or Delamination	As per description for yielding, however the roof strata have also undergone moderate deformations (0.2-0.5 m). See Figure 8C. Where delamination occurs, there is visible bed separation extending 0.5-1.0 m into rock mass. See Figure 8D
4.	High Degree of Yielding or Delamination	As per description for moderate yielding, however the region of yielding and deformation extends > 1 m into the rock mass. See Figure 8E. Where delamination occurs, there is visible bed separation extending > 1 m into rock mass. See Figure 8F
5.	Collapse	Yielding and visible delamination of the whole of the overlying rock mass with moderate or large displacements (>0.5m). May also be visible deformation at upper boundary of model representing the surface. See Figure 8G

Figure 8: Examples of the differing yield descriptions summarised in Table 6.

Initially a large number of parameter sets were found to exceed the instability and collapse criteria and in order to refine the list of feasible scenarios for the Dolphingstone site, a number of further criteria were adopted. The additional criteria are summarised in Table 7 and the modelling process and assessment of failure using these criteria is summarised in Figure 9.

Table 7: List of criterion used to refine list of feasible failure scenarios

Number	Name	Description
1	Vertical Strain Criterion	Inclusion criterion: vertical strain criterion as described in section 4.5. Parameter sets passing this criterion were then examined for compliance with criterion 2.
2	Immediate Collapse Criterion	Exclusion criterion: models where the roof collapse occurred immediately upon excavation of the void space and so the excavations had an initial factor of safety of less than 1 and as such were considered not to accurately reflect the collapse that occurred at Dolphingstone. Parameter sets that failed to be excluded were then examined for compliance with criterion 3
3	Full Overlying Rockmass Failure Criterion	Inclusion criterion: the plots of block yield state and interface shear slip were examined and where the yield plot indicated failure of elements through the entire sandstone bed and the interface failure plot also displayed current shear failure of the bedding surfaces from the base of the sandstone to the upper boundary (indicative of the formation of shear fractures from the roof pillar intersection into the rockmass necessary for roof collapse to occur) then there was the potential for roof failure leading to void migration to occur.
4	Rising Water Failure Criterion	Inclusion criterion: parameter sets passing criterion 1 but failing criterion 3. These sets were compared to those at a later stage of the model run with increasing levels of phreatic surface, to identify if there was any potential for failure to occur within the roof strata to the full thickness of the sandstone roof layer leading to the possibility of void migration occurring. An example of a case where increases in the water table lead to failure propagating through the full thickness of the roof layer is shown in Figure 11a and Figure 11b.

Figure 9: Flow chart showing modelling methodology and the process of failure assessment.

The results for all the parameter sets are summarised in Figure 10. Some broad trends in terms of stability can be identified based on the results:

- Stability increases with increasing rock mass strength and stiffness properties.
- Stability decreases with increasing excavation width which can be seen in Figure 10 where, for example, a 1 m wide excavation with 0.2 m bedding spacing and a GSI of 20 is initially stable (although increased water table promotes instability) however increasing excavation widths for an equivalent water table lead to increasing instability and at a 3 m excavation width cause total collapse.
- Stability decreases with increasing height of the ground water table above the excavation base due to an increase in the pore pressures within the rock mass promoting yielding, the increase in the overburden load due to the presence of the water and also due to the strength reduction that occurs as a result of saturation of coal measures rocks.
- Stability decreases with decreased bedding thickness, which can be seen quite clearly in, for example, Figure 10 where the increase in bedding thickness leads progressively from a collapse that causes surface deformation at a 0.1 m spacing with a rock mass GSI of 30 to increasing stability at 0.2 m and 0.5 m spacing where a high and moderate degree of yielding occur respectively.

Another factor of note is that a number of the excavations (at low rock mass strengths, with low values of bedding thickness and with larger excavation widths) underwent collapse, or a high degree of strata delamination, when the water table was below the base of the void upon initial excavation. These models were therefore removed from the list of scenarios that may feasibly have occurred at Dolphingstone as per criterion 2 in Table 7.

Models where failure was initiated by a rise in groundwater were also considered important, as there have been reported seasonal variations in the ground water level of 1.5 to 1.8 m, and it is not clear, due to the variable seasonal nature of precipitation, as to whether a depth of 3 m below the ground surface represented a maximum, median or minimum value of water table. From this it was assumed that scenarios that showed yielding occurring with a water table at a depth above the excavation roof (8 m below surface), up to a depth of 1 m below surface would span the full range of potential scenarios that may have triggered the Dolphingstone collapse. An example of this is seen in Figures 11a and b.

Figure 10: Plot showing results of full modelling study in terms of type of failure (as per Table 6) and the ground water level at which it occurred relative to excavation base.

Figure 11a: Plot of scenario which initially fails the full span thickness failure criterion (initially fails criterion 3).

Figure 11b: Plot of scenario where failure criterion is met when level of water table increases (passes criterion 4).

The parameter sets which fit the new criteria are summarised in Table 8 in which there are thirteen scenarios where void migration may occur; of which nine have geometry with a 3 m wide room/excavation and the remaining four, a 2 m wide excavation geometry. This indicates a minimum excavation width of 2 m for the failure to occur but suggests that 3 m is more likely.

Table 8: Scenarios fitting the revised criterion for failure of the full thickness of the sandstone bed.

Scenario	Bedding separation (m)	GSI	Excavation Width (m)	Hydraulic head above excavation roof at instability (m)
1	0.1	60	2	+2 m – Yielding ^a +8m exceeds criterion 4
2	0.1	70	2	+7 m – Yielding
3	0.1	70	3	+2 m – Yielding
4	0.1	80	3	+3 m – Yielding
5	0.2	40	2	+2 m – Yielding ^a +8m exceeds criterion 4
6	0.2	60	3	+3 m – Yielding
7	0.2	70	3	+3 m – Yielding ^a +6m exceeds criterion 4
8	0.5	30	2	+2 m – Yielding ^a +8m exceeds criterion 4
9	0.5	40	3	+2 m – Yielding
10	0.5	50	3	+3 m – Yielding ^a +4 m exceeds criterion 4
11	0.5	60	3	+4 m – Yielding ^a +6 m exceeds criterion 4
12	0.5	70	3	+7 m – Yielding ^a +8 m exceeds criterion 4
13	1.0	40	3	+2 m – Yielding ^a +6 m exceeds criterion 4

^aIndicates that the criterion was exceeded on raising the water table. Value is height above excavation base

The results also indicate that the sandstone roof strata contained bedding or discontinuity surfaces and were most likely to be of less than 1 m separation. The majority of scenarios where yielding occurred through the full thickness of the sandstone and where the water table was above the excavation roof had a bedding spacing of 0.5 m (except for one potential failure scenario — GSI 40, 3 m wide excavation, 1 m bedding thickness) which is equivalent to a thickly bedded sandstone.

For the majority of excavations and bedding thicknesses within the constraints defined (*i.e.* a water table above the level of the workings), the maximum strength at which roof failure could occur was at a GSI of 70 (there was one exception to this where failure occurred in a GSI 80 model with a 3 m wide excavation with 0.1 m bedding spacing).

It can also be seen from Table 8 that seven of the scenarios show initial roof yielding that does not span the full thickness of the strata until the water table level is raised. A number of those are triggered at a water level table of between 5 and 7 m above the excavation base (2 to 4 m below the surface). This is within plus or minus 1 m of the water table as measured in the borehole of interest. This result is significant as it is very strongly indicative that the rise in the groundwater table at Dolphingstone after the abandonment of the mine workings was a significant factor in causing the initiation of the collapse.

This would suggest that the stability of the roof strata of the abandoned workings in these scenarios were marginal (factor of safety approaching 1) and that the seasonal fluctuations in the ground water table may have been responsible for triggering the collapse. As the

groundwater table at Dolphingstone is recharged solely by precipitation (Edmund Nuttall Ltd., 2002) this potentially suggests that there may be a meteorological/climate trigger related to unseasonably high rainfall at the site which was responsible for triggering the collapse.

An example of a scenario where a rise in water table to present day levels prompted failure is presented in Figure 12.

Figure 12: Yield and interface shear failure plot of the failed rock mass above the excavation. Pore pressures are monitored in roof centreline.

This shows plots of the shear failure along the interfaces and the yield state of the rock mass over the excavation roof (Scenario 7, Table 8) where it can be seen that a water table 3 m above the excavation floor (6 m below surface) triggers displacements that exceed criterion 1 (marked by displacements co-incident with a pore pressure of 20 kPa in the roof centreline), however a further 3 m rise in the water table (indicated by a 50 kPa pore pressure in the roof centreline in Figure 12 at maximum displacement) is required to cause the failure to propagate through the full thickness of the sandstone layer to allow the model to meet criterion 4 (see Table 7) which further reinforces the evidence that even small increases in the water table appear to have an effect on excavation roof stability at the site in question and so possibly may also be significant in other shallow abandoned workings.

6. Conclusions

The work undertaken here in the investigation of the instability that occurred at the site at Dolphingstone highlights the significant impact that shallow abandoned mine workings can have on transport infrastructure and the potentially large financial cost of remediation. The work also suggests that future modelling work when investigating geotechnical problems with significant unknowns is best undertaken as a parametric study and that rather than expecting to find a single cause of failure, a suite of potential scenarios that all fit the observed failure may be found. These can be used to place constraints on the likely ground conditions in terms of rock mass properties (both rock mass strength and the presence and spacing of discontinuity surfaces) as well as on the subsurface stress conditions likely to contribute to failure (in this case a function of the groundwater table, and the void size).

The modelling work also highlights the importance of engineering judgement in the interpretation of numerical modelling results (in this case specifically the requirement to refine the failure criteria used in assessing which parameter sets represented a realistic analogue to the failure that occurred at Dolphingstone) and may serve as a reminder to the numerical modeller and practising engineer that a model result is not an end in of itself unless it accurately captures the behaviour of the system being modelled and so care must be taken in developing a model to ensure that this behaviour is being captured and also in specifying what constitutes a valid end state of modelling (in this case failure within the constraints known to have existed based on the site history and ground investigation results).

As discussed in Section 4.3, the results produced from this work were potentially affected by the chosen strain softening parameters and the density of the mesh. This was mitigated in this work through the selection of appropriate parameters and zone sizes derived from data in the literature. But, it should be noted that, unless the model parameters are appropriately and consistently adjusted, an increasing density of mesh would result in softening of the strata within the model which form the roof beams occurring at ever smaller displacements. This would imply lower stability than observed and would no longer match the expected behaviour.

The specific results of the modelling undertaken here suggests that the Dolphingstone collapse was a result of roof failure in a room width of at least 2 m span (although the majority of scenarios resulting in the potential for roof collapse occurred in rooms with a width of 3 m (equivalent to a maximum corner to corner span of approximately 4.24 m).

The modelling indicates that the sandstone bed would have been stable at a rock mass strength equivalent to a GSI of 80 at all levels of ground water table and all bedding plane spacings tested with one exception, as such this is assumed to mark an upper bound of the strength of the rock mass overlying the abandoned workings. Lower bounds for the rock mass properties are assumed to be those cases where failure occurred immediately upon excavation.

The modelling also indicates that the rise in the groundwater table at Dolphingstone after the abandonment of mine workings was a significant factor in causing the initiation of the collapse. Of particular interest are the scenarios where the full failure of the roof strata did not occur until the water table reached a level in close proximity to that of the present day water table. This indicates that at these groundwater levels and resultant pore water pressures (which act to reduce the effective stress in the rock mass) combined with the decrease in material strength due to saturation as described in Section 4.3, the excavation stability was very marginal (close to a factor of safety of 1). This in turn suggests that the seasonal fluctuations in the ground water table may have contributed to the instability and so be responsible for triggering the collapse.

Another implication from this work is the potential impact of even small increases in the ground water table on the stability of shallow voids. Normally dewatering is considered to be the primary hydrogeological risk factor for the failure of old abandoned workings, however this work suggests that small seasonal fluctuations in the groundwater table can also have an impact on stability. This may be of particular importance in cases (such as here) where the stability of the overlying strata is marginal and where the potential for void migration to be halted by bulking or arching is low. In such cases, the routine monitoring of groundwater levels/pore pressures in the rock mass and overburden over shallow abandoned mine workings that are a potential threat to infrastructure may be a sensible precaution, with a trigger for intervention set where any changes that are outside a norm derived for that area act as a potential warning of increased subsidence risk. More broadly the case study summarised here along with the modelling study suggests that numerical modelling allows the practising engineer to make a detailed assessment of potential trigger mechanisms that may lead to subsidence occurring or may trigger further subsidence events. In turn this information may potentially allow the development of a monitoring regime at sites where there may be a subsidence risk.

Acknowledgements:

The authors would like to thank Network Rail for providing the funding for this research project along with access to the ground investigation data necessary to undertake this work. The authors would also like to thank Mr Keith Whitworth for providing access to the piezometer data for Wallyford as used in Figure 6.

7. References

- Attewell, P.B., Taylor, R.K., 1984. Ground Movements and their Effects on Structures. Blackie & Son Ltd, London.
- Badr, S., Ozbay, U., Kieffer, S., Salamon, M., 2003. Three-Dimensional Strain Softening Modeling of Deep Longwall Coal Mine Layouts, in: Andrieux, P., Brummer, R., Detournay, C., Hart, R. (Eds.), Third International Symposium on FLAC and FLAC3D Numerical Modelling in Geomechanics. Taylor & Francis, Sudbury, Ontario, pp. 233-239.
- Bell, F.G., 1975. Site Investigations in Areas of Mining Subsidence. Butterworth and Co., London.
- Bell, F.G., 1992. Ground Subsidence: A General Review, COMA: Construction Over Mined Areas. South African Institution of Civil Engineers, Pretoria, South Africa, pp. 1-23.
- Bell, F.G., De Bruyn, I.A., 1999. Subsidence Problems due to Abandoned Pillar Workings in Coal Seams. Bulletin of Engineering Geology and the Environment 57, 225-237.
- Bieniawski, Z.T., 1984. Rock Mechanics Design in Mining and Tunnelling. Balkema, Rotterdam.
- Brady, B.H.G., Brown, E.T., 1993. Rock Mechanics for Underground Mining, 2 ed. Chapman and Hall, London.
- British Standards Institution, 1999. BS 5930:1999 - Code of Practice for Site Investigations. BSI, Milton Keynes.
- Cai, M., Kaiser, P.K., Tasaka, Y., Minami, M., 2007. Determination of Residual Strength Parameters of Jointed Rock Masses Using the GSI System. International Journal of Rock Mechanics and Mining Sciences 44, 247-265.
- Carranza-Torres, C., Zhao, J., 2009. Analytical and numerical study of the effect of water pressure on the mechanical response of cylindrical lined tunnels in elastic and elasto-plastic porous media. International Journal of Rock Mechanics & Mining Sciences, 46, 531-547
- Craig, R.F., 2004. Craig's Soil Mechanics, 7 ed. Taylor and Francis, London.
- Deb, D., Choi, S.O., 2006. Analysis of Sinkhole Occurrences Over Abandoned Mines Using Fuzzy Reasoning: A Case Study. Geotechnical and Geological Engineering 24, 1243-1258.
- Diederichs, M.S., Kaiser, P.K., 1999. Stability of Large Excavations in Laminated Hard Rock Masses: The Voussoir Analogue Revisited. International Journal of Rock Mechanics and Mining Sciences 36, 97-117.
- Donaldson Associates Ltd., 2002. ECML Dolphingstone Remediation Works Geotechnical Interpretive Report. Edmund Nutall Ltd.
- Dyne, L.A., 1998. The Prediction And Occurence Of Chimney Subsidence In Southwestern Pennsylvania, Mining and Minerals Engineering. Virginia Polytechnic Institute and State University, Blacksburg, p. 109.
- Edmund Nuttall Ltd., 2002. Hydrological Summary Report – Dolphingstone Remediation Works. Report No. SC/MG/512/8.

Esterhuizen, G.S., Karacan, C.O., 2005. Development of Numerical Models to Investigate Permeability Changes and Gas Emission around a Longwall Mining Panel Proceedings of the 40th U.S. Symposium on Rock Mechanics (USRMS), Anchorage, University of Alaska, pp. 25-29.

Goodman, R.E., 1989. Introduction to Rock Mechanics, 2 ed. Wiley, New York.

Healy, P.R., Head, J.M., 2002. Construction Over Abandoned Mine Workings, CIRIA Special Publication 32. CIRIA, London, p. 94.

Helm, P., 2010. ECML between Wallyford and Prestonpans [PNG map], Scale 1:10,000 (OS 1:25,000 Raster [geospatial data], updated May 2010; Land-Form PROFILE [geospatial data], updated August 2003), Ordnance Survey, GB. Using: EDINA Digimap Ordnance Survey Service, <<http://edina.ac.uk/digimap>>, Created: December 2010.

Hoek, E., Brown, E.T., 1997. Practical estimates of rock mass strength. International Journal of Rock Mechanics and Mining Sciences 34, 1165-1186.

Hoek, E., Carranza-Torres, C., Corkum, B., 2002. Hoek-Brown Failure Criterion - 2002 Edition, Proceedings of the NARMS-TAC Conference, Toronto, pp. 267-273.

Hoek, E., Marinos, P.G., Marinos, V.P., 2005. Characterisation and Engineering Properties of Tectonically Undisturbed but Lithologically Varied Sedimentary Rock Masses. International Journal of Rock Mechanics & Mining Sciences 42, 277-285.

Hutchinson, D.J., Diederichs, M.S., 1996. Cablebolting in Underground Mines. Bitech Publishers Ltd, Vancouver.

Itasca, 2005. FLAC3D - Fast Lagrangian Analysis of Continua in 3 Dimensions Version 3.0, Users Guide. Itasca Consulting Group, Minneapolis.

Jones, C.J.F.P., Lamont-Black, J., Davies, O., 2005. Review of Current Practice Associated with the Management and Treatment of Subsurface Voids as they Influence Transport Networks. University of Newcastle upon Tyne, Newcastle upon Tyne.

Li, Z., Reddish, D.J., 2004. The Effect of Groundwater Recharge on Broken Rocks. International Journal of Rock Mechanics and Mining Sciences 41, 280-285.

Network Rail, 2003. Annual Return to the Rail Regulator. [online] Available at: http://www.networkrail.co.uk/uploadedFiles/networkrailcouk/Contents/Publications/Annual_Return/2003%20annual%20return.pdf [Accessed 13 April 2012].

Okubo, S., Fukui, K., 1996. Complete Stress-Strain Curves for Various Rock Types in Uniaxial Tension. International Journal of Rock Mechanics and Mining Sciences Abstracts 33, 549-556.

Rocscience, 2010. Rocscience Products – Roclab [online] Available at: <http://download.rocscience.com/products/RocLab.asp> [Accessed 18 November 2010].

Singh, G.S.P., Singh, U.K., 2009. A Numerical Modelling Approach for Assessment of Progressive Caving of Strata and Performance of Hydraulic Powered Support in Longwall Workings. Computers and Geotechnics 36, 1142-1156.

Swart, A.H., Handley, M.F., 2005. The Design of Stable Stope Spans for Shallow Mining Operations. *The Journal of the South African Institute of Mining and Metallurgy* 105, 275-286.

Thigpen, L., 1984. On the Mechanics of Strata Above Underground Openings. *American Society of Civil Engineers Journal of Geotechnical Engineering* 122, 14-44.

Tomlinson, M.J., 2001. *Foundation Design and Construction*, 7 ed. Pearson Education Ltd, Harlow, Essex.

Twiss, R.J., Moores, E.M., 1992. *Structural Geology*. W.H. Freeman, New York.

Vasarhelyi, B., 2003. Some Observations Regarding the Strength and Deformability of Sandstones in Dry and Saturated Conditions. *Bulletin of Engineering Geology and the Environment* 62, 245-249.

Waltham, A.C., 1989. *Ground Subsidence*. Blackie & Son Ltd, London.

Whittaker, B.N., Reddish, D.J., 1989. *Subsidence: Occurrence, Prediction and Control*. Elsevier, Amsterdam.

Whittles, D.N., Reddish, D.J., Lowndes, I.S., 2007. The Development of a Coal Measure Classification (CMC) and its Use for Prediction of Geomechanical Parameters. *International Journal of Rock Mechanics and Mining Sciences* 44, 496-513.

Zhang, L., 2005. *Engineering Properties of Rocks*. Elsevier, London.

Zipf, K., 2005. Failure Mechanics of Multiple Seam Mining Interactions, in: Peng, S.S., Mark, C., Finfinger, G.L., Tadolini, S.C., Heasley, K.A., Khair, A.W. (Eds.), *Proceedings of the 24th International Conference on Ground Control in Mining*. West Virginia University, Morgantown, West Virginia, pp. 93-106.

Zipf, K., 2006. Numerical Modelling Procedures for Practical Coal Mine Design, in: Yale, D.P. (Ed.), *Proceedings of the 41st U.S. Symposium On Rock Mechanics - Golden Rocks 2006* Curran Associates, Golden, Colorado.