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STABILITY ANALYSIS AND OPTIMUM SUPPORT DESIGN OF A ROADWAY IN A FAULTED ZONE DURING LONGWALL FACE RETREAT - CASE STUDY: TABAS COAL MINE

Ali Sahebi[1], Hossein Jalalifar[2], and Mohammad Ebrahimi[1]

ABSTRACT: Stability analysis of a longwall, East 1 tailgate (E1TG) of Tabas underground coal mine is presented. The mine extracts coal by both longwall and room and pillar methods. The mine is designed to produce 1.5 million tonnes of coal annually. The roadways have a rectangular profile of 4.5m width and 3.5m height. The field investigations and the geomechanical characteristics of rocks showed that the rock masses are weak, requiring a suitable support system. The roadway is intersected by a major fault zone. To investigate the effect of the fault zone on roadway stability, in particular during the face retreat, extensive numerical simulations were carried out. It was found that the stability of the tailgate was severely affected by major structures such as faults and crushed zones. In addition to this, it was revealed that the situation gets worse during the face retreat. An optimised support system was determined.

INTRODUCTION

Tabas underground coal mine is located some 85km south of Tabas town, Yazd province, Iran. Figure 1 shows the mine location. This mine is the first fully mechanised coal mine in Iran that produces 4000 tonnes of coal per day.

A 4.13 m long roof core sample taken in E1TG revealed a frequency of siltstone and sandstone layering above the coal seam. The coal seam was up to 2.2 m thick (average thickness of 2 m) dipped in the range 14°–26°. At the end of year 2007 the first longwall panel was commissioned. The longwall face was operated with a double drum shearer and shield supports. The rock mass rating (RMR) based on the roof core and rock mass confinement method (Daws 1992) was used to estimate the appropriate bolting pattern. Prior to bolt installation, the roadways were supported with conventional props and bars, which underwent severe deformation with the approaching longwall face. Figure 2 shows location of the extraction panels (East1 and East2).

LONGWALL MINING IN TABAS COAL MINE

Site description

The E1 longwall panel had a face width of 180 m and panel length of 1200m. The C1 working seam thickness varied from 1.8 to 2.2 m with dip varying between 11˚ to 26˚. The roof of the coal seam contained 0.1 to 0.2 m mudstone, siltstone/sandstone interfaces and sandstone. The C1 seam had a uniaxial compressive strength of less than 5 MPa. The other seams in the vicinity of the C1 seam were C2 and D1 above and B1 and B2 below (IRITEC 1992). A 4.13 m long roof core taken in E1TG at MM of 180.7 revealed sequential layers of siltstone, sandy siltstone and silty sandstone immediately above the roof in the tailgate. The core data is summarised in Table 1 together with other observed parameters to calculate RMR values (Taghipoor 2008). Figures 1 and 2 show the location of the mine and the plan of the extraction panels East 1 and East 2. Figure 3 shows double drum shearer and shield supports in East 1 panel.

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Figure 1- Tabas coal mine location (not to scale)

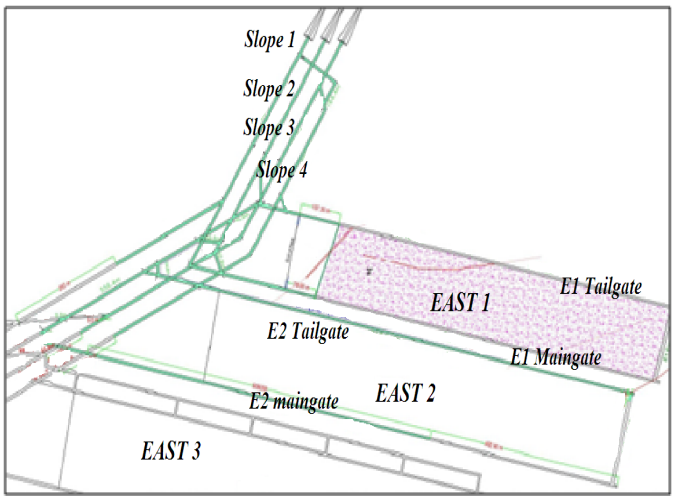

Figure 2- Extraction panels of East 1 and East 2 (not to scale)

Figure 3- Double drum shearer and shield support in East 1

The excavated roadways serving the longwall face were generally of cross sectional area which varied between 15 m^2 and 18 m^2. The basic support pattern system consisted of 13 x 2.4 m long AT roof bolts (7+6 pattern) per metre length of the roadway. The ribs in the tailgate were supported with four 1.8m fibreglass bolts in the right hand side and three 1.8 m AT bolts in the left hand side. Several trial sites were established to examine the performance of different bolting patterns. The first site was a 15 m length of the roadway containing 13 roof bolts plus IPB-160 steel frames, set one at 1m spacing. The sides of the roadway were packed with corrugated iron sheets and sand bags. The second trial section, some 40 m long, used a basic pattern of roof and side bolts with IPB 160 steel frames set at 1 m spacing. In the third trial site section of about 37 m length, the same bolting pattern was used plus frames set at 2 m spacing (Taghipoor 2008).

DEVELOPMENT OF A FDM MODEL

FLAC ^{2D} software, based on a FDM[1] analysis, was specially developed to calculate 2D stresses and displacements induced by underground excavations. FLAC can be used to solve a wide range of mining and civil engineering problems. Materials in the model can be linear elastic and non-linear (Mohr–Coulomb and Hoek–Brown failure criterion) and discontinuities may be defined into the model. This feature was used to model the movements of blocks in the roadway.

[1] *Finite Difference Method*

Table 1 - Core data and RMR parameters

Depth into roof(m)	Rock Type	UCS (MPa)	RQD (%)	Discontinuity Spacing (m)	Discontinuity Condition	Ground Water	Discontinuity orientation	RMR
0 – 2.8	Siltstone	**32**	**29.2**	**0.06 – 0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III–IV
Rating		**4**	**8**	**8**	**23**	**4 - 15**	**-12**	(35 – 46) **40.5**
2.8 -3.7	Sandstone	**73**	**69.3**	**0.06 – 0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III - IV
Rating		**7**	**17**	**8**	**23**	**4 - 15**	**-12**	(47 – 58) **52.5**
3.7-4.13	Siltstone	**32**	**34**	**0.06 – 0.2**	Slightly rough, separation<1mm	Dripping to dry	Very unfavourable	Class III - IV
Rating		**4**	**8**	**8**	**23**	**4 - 15**	**-12**	(35 – 46) **40.5**

Geometry of the model

The geometry of the area modelled was 40 m by 40 m with a roadway width of 4.5 m and height of 3.5 m. The coal seam was modelled as 2 m thick and dipping at 20˚. The E1TG immediate roof stratification sequence consisted of siltstone and sandstone above the roof. The geometry of the model defined is shown in Figure 4.

Boundary conditions

The model assumes plane strain state, nil displacements at the boundaries and constant field stresses. If the model is used to simulate convergence without longwall influence, then it is assumed that only the stress due to the pressure of the overburden at that depth is acting.

$$\sigma_v = \gamma \times H \quad (1)$$

$$\sigma_h = k \times \sigma_v \quad (2)$$

Where γ is the unit weight (kN/m^2), H is the depth (m). E1TG was located at a depth of 150 m around the coring position. The vertical stress of 3.2 MPa and the ratio of horizontal to vertical stress k = 0.4 were determined for the site, according to the tectonic history of the region.

FLAC ^{2D} was used to analyse the efficiency of the old and new roof bolting patterns. To provide input parameters for the models, the RocLab program (working based on GSI classification, GSI=RMR-5) was used to estimate the parameters of rock mass surrounding the roadway (Rocscience, 2002) and to provide input parameters for the models. The results are listed in Table 2. Short encapsulation pull test results were used to define roof bolt bond properties as shown in Table 3.

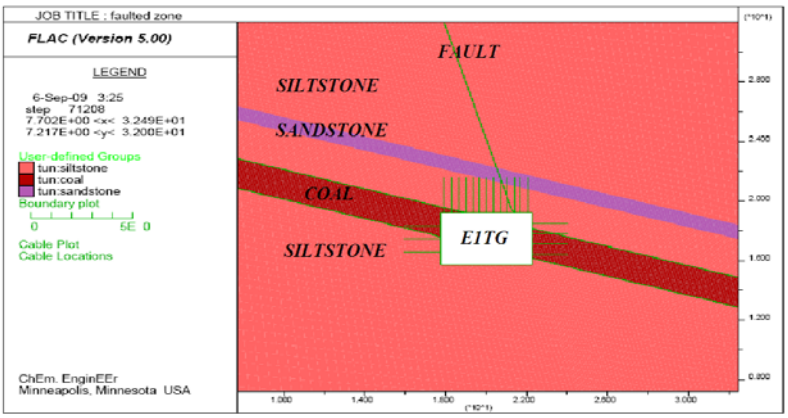

Figure 4 – The model roadway profile, layers and rockbolt pattern numerical modelling

Numerical Modelling

Table 2 - Intact rock and rock mass parameters using Roclab program

Rock Type	Depth into roof (m)	Intact Rock			Rock Mass							
		UCS (MPa)	m*	GSI	m	s	C (Mpa)	ϕ (deg)	Tensile Strength (MPa)	Compressive Strength (MPa)	E (GPa)	ν
Coal	0	5	1	25	0.06	0.0002	0.063	9	0.018	0.06	0.53	0.25
Siltstone	0 -2.8	32	7	35.5	0.69	0.0008	1	23.4	0.035	0.79	2.4	0.26
Sandstone	2.8- 3.7	73	13	47.5	1.99	0.0029	3.7	32	0.107	3.79	7.4	0.26
Siltstone	3.7- 4.2	32	7	35.5	0.69	0.0008	1	23.4	0.035	0.79	2.4	0.26

Table 3 - Bolt and bond properties (IRITEC 1992)

Parameter	unit	value
Diameter	mm	21.7
Elastic Modulus	GPa	207
Tensile Yield Load	tone	32
Shear Stiffness of Bond	KN/mm	65
Compressive Strength of Bond	MPa	5
Normal Stiffness of Bond	MPa	100

MODELLING WITH FAULTED ZONE

In this section, the stability analysis of roadway E1TG within the faulted zone was carried out. Material properties such as friction angle, cohesion, dilation angle, bulk modulus, shear modulus and in-situ stresses were used as inputs into the model defined with displacements and shear strains as outputs. The simplest and best-known failure criterion for rocks, the Mohr-Coulomb criterion, was used. In FLAC^{2D}, the Mohr-Coulomb plasticity model is one of the built-in constitutive material models. It is used for materials that yield when subjected to shear loading but the yield strength depends solely on the major and minor principal stresses; the intermediate principal stress has no effect on the yield. For the Mohr-Coulomb model, the following material properties are required: rock mass density, bulk modulus, shear modulus, friction angle, cohesion, dilation angle and tensile strength. Interfaces here mean joints, faults, or bedding planes in rock masses. In FLAC, interfaces are characterised by Coulomb sliding and/or tensile separation. They have properties of friction, cohesion, dilation, normal and shear stiffness, and tensile strength. For bedding planes to be modelled, slip along the plane and bed separation are the major desired features (Itasca Group 2000).

Bolt representation

There are several structural elements in FLAC for simulating structural supports. One of them is called cable element, which is a one-dimensional axial element. It can be point-anchored or grouted to the surrounding material so that the cable element develops forces along its length as the surrounding media deform. Therefore, cable element is the best structural element for modelling rock bolts. The cable element requires the physical and mechanical rock bolt parameters as input data. The properties associated with the grout are more difficult to estimate. In many cases, the following expression provides a reasonable estimation of *kbond* (Itasca Group 2000):

$$K_b = \frac{\pi G}{5Ln\left(1+\frac{2t}{d}\right)} \qquad (3)$$

Where;

G = Grout shear modulus;
t = Annulus thickness;
d = Diameter of the bolt.

Given the failure of the bolting system occurs at the grout/rock interface, *sbond* can be approximated by the following equation:

$$S_{bond} = \pi\left(d + 2t\right)\tau_1 Q_B \tag{4}$$

Where;

τ_1 = One-half of the uniaxial compressive strength of the weaker of the rock and grout;

Q_B = the quality of the bond between the grout and rock (Q_B = 1 for perfect bonding);

If it is believed that the failure occurs at the bolt/grout interface rather than at the grout/rock interface, then the shear stress should be evaluated at this interface by replacing ($d + 2t$) by *d* in Equation 4. For partially grouted bolts, the free portion of the bolt does not have any bond with the rock. Under such circumstance, the values of *kbond* and *sbond* are set to zero (Itasca Group 2000).
To investigate the underground stability the Sakurai and et al. (1994) method was used. The method evaluates the critical strain in the elastic region. Since the rock mass is under triaxial stress, it is logical using the maximum critical strain for investigation of tunnel stability. They suggested the following equations (Sakurai 1993).

$$log\ \varepsilon_c = -0.25\, log\ E - 1.22 \tag{5}$$

$$\gamma_c = (1+\nu)\,\varepsilon_c \tag{6}$$

Where;

E = Young's modulus of intact rock $\left({}^{kgf}\!/_{cm^2}\right)$

ε_c = critical strain in uniaxial compressive strength

γ_c = critical strain

ν = Poisson's ratio.

Critical displacement values based on the critical strain are obtained by following equation:

$$\varepsilon_c = \frac{U_c}{a} \tag{7}$$

Where;

U_c = Allowable displacement

a = radius of the roadway

Displacement vectors in Figure 5 show that higher deformations occur at the top of left rib and bottom of right rib of the roadway within the coal. Similar results can be found in the maximum shear strain increment plot (Figure 6). These two figures reveal the potential failure mechanism which can lead to failure of wedges at left hand side part of roof and top part of right hand side rib. The displacement around E1TG is high and shows that E1TG was unstable and needed to be supported. To find the optimum support system, different patterns and spacing of rockbolts were modelled. Using Sakurai's equations and results of analysis it was determined that patterns 8+6 and No flexi 4+2 are better than other patterns in the faulted zone.

Figure 7 presents axial forces in rockbolts in the two patterns. Results of vertical and horizontal displacements and maximum shear strain around E1TG are summarised in Table 4 and Table 5.

MODELLING WITH RETREATING FACE

In the longwall mining method, the roof strata at the longwall face is undermined and allowed to collapse behind the longwall face shields. When the face is far enough advanced from the face starting position, the immediate roof collapses at a certain distance depending on the geological conditions. Failure of the roof continues until the roof and the caved material are in contact. The natural stress distribution in the rock strata is disturbed with the excavation; high pressure zones are created in the adjacent coal because of the transfer of stresses. Figure 8 shows state in the ground after the excavation of a panel (Yavuz 2004).

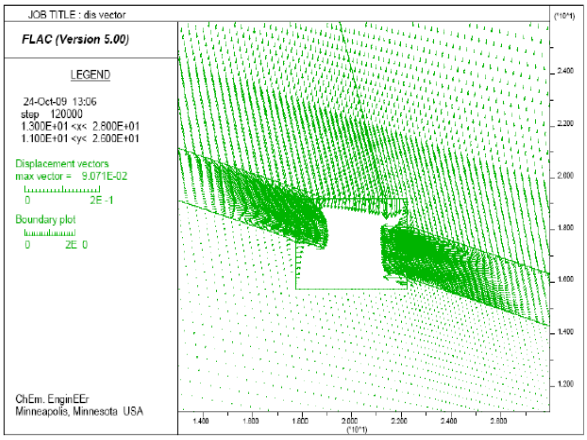

Figure 5 - Displacement vector plot

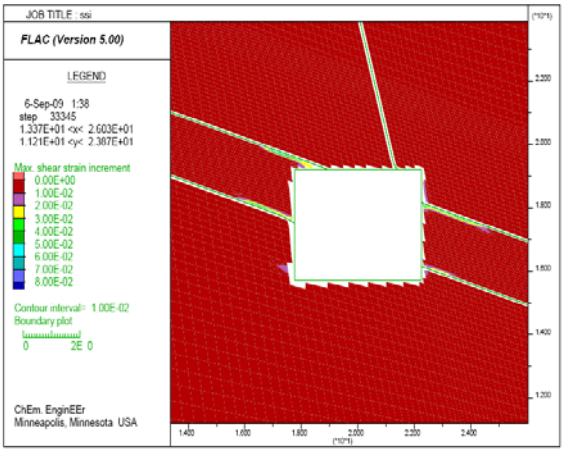

Figure 6 - Maximum shear strain increment plot

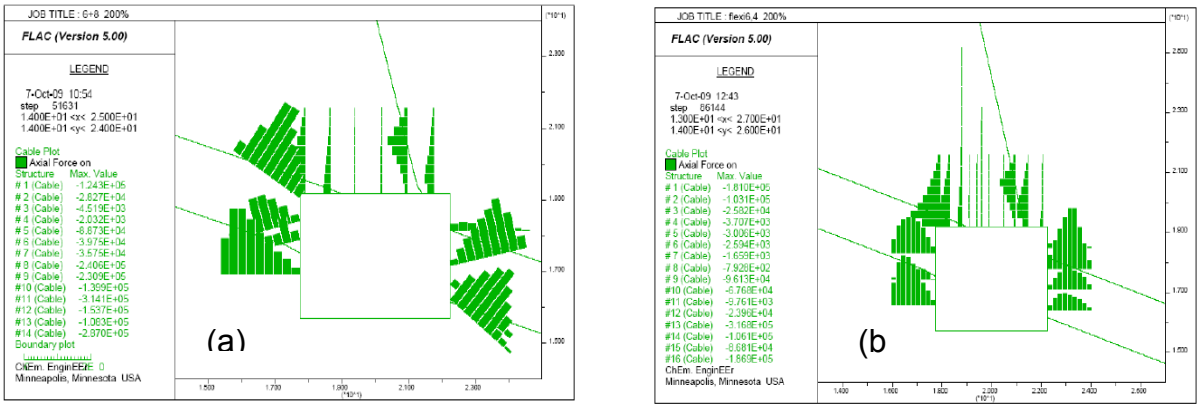

Figure 7 - Axial loads (Units: Newtons) in the bolts: (a) Pattern 6+8 and (b) Pattern Flexi

Table 4 - Horizontal and vertical displacements of around E1TG (mm)

Model of fault	Roof	Right hand rib	Left hand rib
No Support	25.2	66.6	87.4
Pattern 6+3	5.96	20.1	41.6
Pattern 6+7	4.76	15.7	36.7
Pattern 6+8	3.38	12.9	16.3
Pattern No Flexi 4+2	2.70	9.95	12.2
Pattern Flexi	5.69	1.99	40.1
Critical Displacement	10.2	18.3	18.3

Table 5 - Maximum shear strain increment around E1TG ($\times 10^{-3}$)

Model of fault	Roof	Right hand rib	Left hand rib
No Support	10.4	10.7	18.7
Pattern 6+3	0.05	1.61	33.4
Pattern 6+7	0.04	0.6	84.2
Pattern 6+8	0.07	1.4	4.93
Pattern No Flexi 4+2	0.07	0.7	4.30
Pattern Flexi	0.04	1.59	38.1
Sakurai Critical Strain	4.54	6.48	6.48

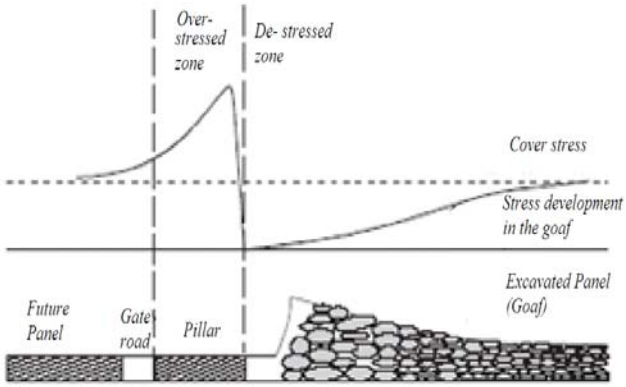

Figure 8 - Stress state in the ground after the excavation of a panel (Yavuz 2004)

To investigate the potential influence of the face retreat, the stresses applied to the model were increased in several stages to simulate the increase in stress ahead of the retreating face. Since the roadways were aligned with the maximum horizontal stress, roadways were not expected to be subject to a severe stress notch during the face retreat. To reflect this, the applied increases were mainly vertical. The stress increases simulating face retreat were applied in several stages. The results indicated that large amounts of roadway closure, mainly in the form of rib squeeze and floor heave, could be expected ahead of the retreating face. With the roof horizon in the seam, there was a large increase in the displacements within the coal roof as the stresses were increased.

The displacements above the coal seam did not show a significant increase nor did the height of softening increase (Bigby, *et al*, 2004). Taking into account the influence of the retreating longwall, the stresses are approximately 2.3 times the pre-mining (or virgin) stress magnitudes. This is in agreement with the calculation results based on the Wilson's theory (Wilson 1980). In this research, it was found that the stress increases to about 2.4 times the pre-mining values. An independent study suggested stress magnitudes of the order 2 to 3 times pre-mining stresses were to be expected (IRITEC 1992).

Figure 9 shows roof layer deformation due to the increasing stress around E1TG. The total displacements of ribs and roof of roadway under different condition are shown in Figure 10.

Results of vertical and horizontal displacements and maximum shear strain around E1TG are shown in Table 6 and Table 7. The results show patterns 8+6 and No flexi 4+2 perform better than other patterns in the faulted zone during the longwall retreat.

Table 6 - Horizontal and vertical displacements of around E1TG in face retreat(mm)

Left hand rib	Right hand rib	Floor	Roof	Model of fault (240%)
85.2	137.2	2.82	31.9	No Support
36.4	25.8	1.31	13.2	Pattern 6+3
33.3	25.5	1.31	12.6	Pattern 6+7
23.9	14.9	1.11	10.2	Pattern 6+8
19.5	8.7	1	8.16	Pattern No Flexi 4+2
31	25.8	1.68	19.7	Pattern Flexi
18.3	18.3	10.2	10.2	Critical Displacement

Table 7 - Maximum shear strain increment around E1TG in face retreat($\times 10^{-3}$)

Left hand rib	Right hand rib	Floor	Roof	Model of fault (240%)*
19.7	13.5	1.55	13.7	No Support
11.8	2.14	0.14	0.1	Pattern 6+3
14.2	2.11	0.12	0.1	Pattern 6+7
4.55	1.87	0.13	0.2	Pattern 6+8
2.67	0.82	0.14	0.21	Pattern No Flexi 4+2
21	2.7	0.18	0.13	Pattern Flexi
6.48	6.48	4.54	4.54	Sakurai Critical Strain

* 240% of pre-mining stress values applied

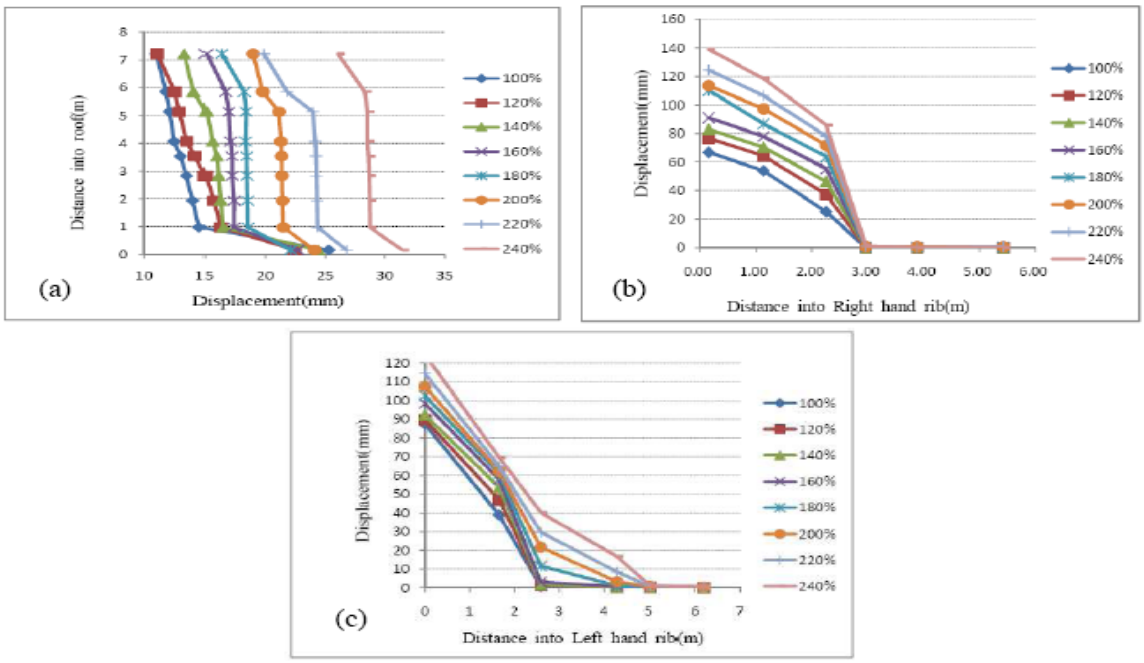

Figure 9 - Displacement around E1TG with increasing stresses due to face retreat: (a) Roof displacement (b) Displacement in right hand rib (c) Displacement in left hand rib Percentages for series indicate proportion of pre-mining stresses applied in modelling

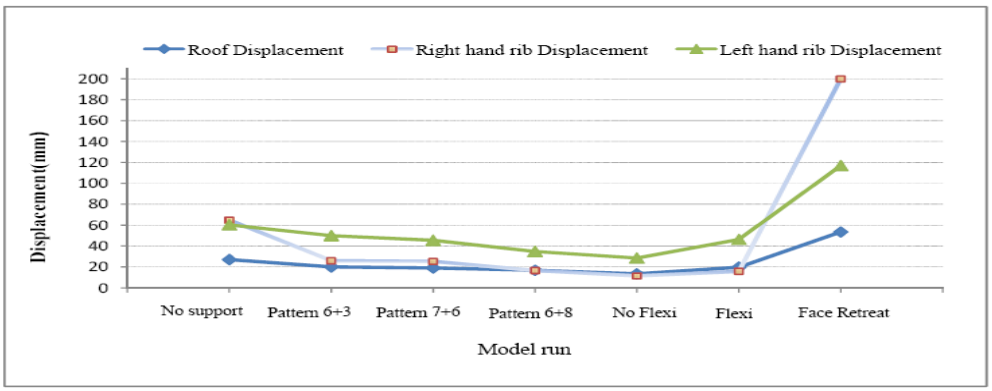

Figure 10 - Displacement for various support patterns, at the faulted zone and the face in retreat state

CONCLUSIONS

Stability analysis and optimum support design of E1TG roadway in the faulted zone during face retreat in Tabas coal mine indicate that:

- There is a need to install strong support system to counter high ground deformation and low safety factor around the roadway.
- Floor heave of TG is independent of the reinforcement in ribs and roof and also face retreat.
- More roof bolts will be needed to control roof movement during face retreat.
- Left side rib will need more reinforcement during retreat. Fully grouted resin bolts are a good choice for this purpose.
- $FLAC^{2D}$ analysis indicate that patterns 8+6 and No flexi 4+2 are better than other patterns within the faulted zone.

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