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FIELD AND SIMULATION STUDY FOR ROCK BOLT LOADING CHARACTERISTICS UNDER HIGH STRESS CONDITIONS

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ABSTRACT: Rhomboid shaped coal pillars (35 m x 30 m to 26 m x16 m) were formed by a modified Room and Pillar method below 850 m depth from surface at the CSM mine in the Czech Republic. The pillars were developed in a shaft protective pillar by driving roadways of 3.5-4.5 m in height and 5.2 m in width within Panel V of Seam No. 30. Development of pillars at such great depth is prone to spalling/fracturing (pillar rib dilation) due to redistribution of the high stress regime. The induced stress driven dilation was measured during partial extraction of the coal seam within the shaft protective pillar using rib extensometers. In order to stabilize the pillar ribs, four rows of rock bolts with 2.4 m length were installed into the pillar from all sides at different heights. The immediate roof was also supported by rock bolts at a 1 m grid pattern. Three-way intersections were made to control the deformation of developed pillars and other underground structures. Further, an attempt was made to understand the rock bolt loading characteristics at different stages of rib dilation using numerical modelling with the available properties of rock mass and reinforcement for the studied site. Elastic and Mohr Coulomb strain-softening constitutive models are considered in FLAC3D to evaluate the performance of the rock bolts. Results obtained on numerical models were found to be in good tune with the rock bolt loading characteristics monitored during the field study. This paper presents a discussion about the impact of rib bolting on pillar safety factor and induced load on rock bolt with respect to the dilation/spalling of pillar ribs at the studied site.

INTRODUCTION

To extract blocked coal reserves from the CSM mine of the Czech Republic, locking high grade of coal in shaft protective pillar at great depth (850 m) was partially extracted by a modified Room and Pillar method (only development). The aim of developing the protective pillar was to avoid the incidences of any surface and sub-surface subsidence in order to maintain intactness of the shaft (Waclawik *et al*., 2018). The mine is situated in the Karvina sub-basin of the Upper Silesian Coal Basin of the country and having a complex geological structures (Waclawik *et al*. 2013, Grygar and Waclawik 2011). Further, at great depth consisting of higher strength of rock mass has led to high stress conditions over the working seam. Near the trialled area, the ratio of major and minor *insitu* horizontal stresses to vertical stress ranged from 0.6 to 2 (Waclawik *et al*. 2017). However, *in situ* stress measured by CCBO stress overcoring cells (Obara and Sugawara, 2003; Stas, Knejzlik and Rambousky, 2004; Waclawik *et al*. 2016) found to be low due to influence of present roadways excavation during measurements. The ratios of the major and minor *in situ* horizontal stresses to vertical stress (calculated from these overcoring measurements) were 1.3 and 0.64 respectively. Thickness of coal seam No. 30 is around 4 m and contains an inter-burden parting of 0.5 m of siltstone. Average uniaxial

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compressive strength of coal, roof and floor strata is 14 MPa, 105 MPa and 60 MPa respectively (Waclawik *et al*. 2017). First panel V was developed in seam No. 30 on a trial basis. The development was done panel-wise by keeping only two pillars along the width. The pillars are developed considering the Mark-Bieniawski strength formula and stability factor (Das, 2012; Mark and Chase 1997). The immediate roof and pillar rib from all sides were supported by resin grouted rock bolts of 2.4 m with wire mesh. Load induced over instrumented rock bolts in immediate roof was also measured during the different stages of development. Considering the geo-mining condition of the Panel V, numerical modelling was done to understand the loading characteristics of the rock bolts using FLAC3D tool. Elastic and Mohr Coulomb strain-softening (MCSS) constitutive models were considered to evaluate the performance of the rock bolts inside the rib and immediate roof. The result of numerical modelling revealed that the induced load over roof bolts installed in the roof is relatively less than rib bolts which were found to be closer to the values of field observations.

FIELD STUDY

Field study was conducted during development of Panel V of the CSM mine. The roadways were developed by Bolter Miner with 1.5-2.5 m cut-out distance followed by installation of rock bolts in roof and pillars rib. Rock bolts were used as a primary means of support in the development working considering standard design methods in the Ostrava–Karvina coal basin (OKD a.s, 2012). Rock bolts in the roof were installed at 1 m grid pattern. However, four rows of rock bolts were installed into the pillar from all sides at different heights keeping a 1 m interval between columns of bolts. Further, the immediate roof was also additionally supported by the two bolts in a row at a 1 m grid pattern up to 25 m distance from the working face. Intersections were also supported by flexi bolts. A number of geo-technical instruments including laser scanning equipment in roadways were used in this panel to monitored strata behaviour (Waclawik *et al*. 2017). Two developed pillars in the Panel V were monitored continuously for around 40 months. Considering the performance evaluation of rock bolts, this paper is discussing observation of roof/rib extensometers and strain-gauged based instrumented rock bolt as shown in part plan of Panel V (Figure 1).

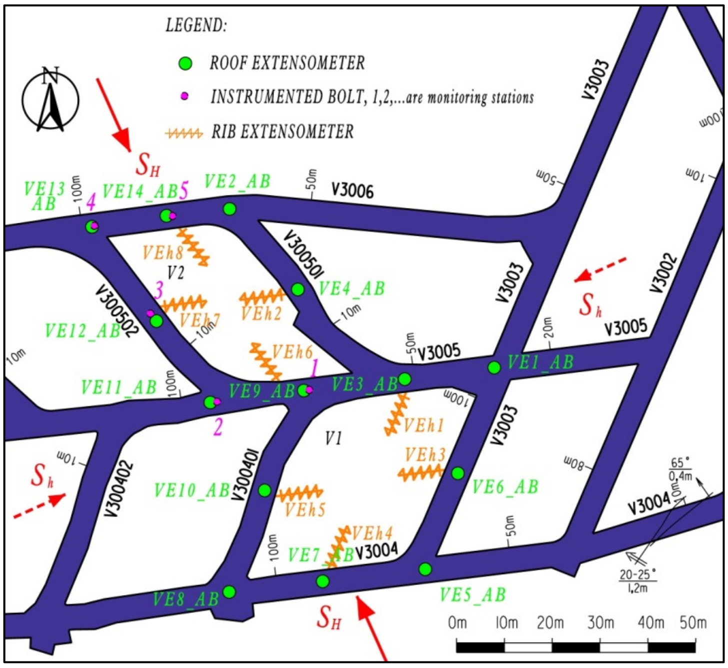

Figure 1: Part plan of Panel V showing locations of geo-technical instruments with orientation of major (SH) and minor (Sh) horizontal stress components

Roof displacement was observed at different locations (Figure 1) by multipoint extensometers at five different roof horizons (1-8 m) from the ceiling of the roadways. No significant roof displacement was observed in roadways of the Panel V. Maximum 7.8 mm roof displacement was observed at an intersection. Except for a few locations, which were influenced by geological disturbances, immediate roof was found to be quite intact (Figure 2). Floor heaving was not measured by geo-technical instruments due to machine operational constraints, but from 3D laser scanning was interpreted as more than 1 m. Due to efficient rib rock bolts, pillar dilation was, relatively, less in the region between 0-1.5 m. However, significant pillar dilation was observed at the depth of 1.5-5 m. A range of 212-300 mm dilation was observed in Pillar V2 of Panel V. Induced load over nine pairs of strain gauged based instrumented rock bolt installed in the immediate roof strata (Figure 3) was observed. A range of 1-6.8 ton induced axial load over instrumented rock bolts was observed. The instrumented rock bolt could not be installed in pillar rib due to poor strength of coal under the high stress conditions.

Figure 2: Field observation of roadways in Panel V

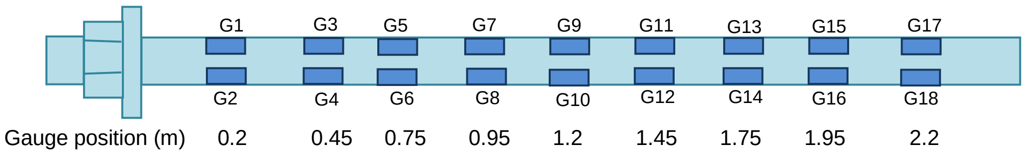

Figure 3: Instrumented rock bolts with nine pair strain gauges, placed along length of the bolt

NUMERICAL MODELLING

Attempted field measurement for the performance of rock bolting provided valuable information, but a systematic parametric study was also needed to support the findings during the field studies. Numerical modelling is a popular tool (Ram *et al*. 2017; Singh *et al*., 2016; Murali Mohan *et al*., 2001) to design mining structures under varying geo-mining conditions. Numerical modelling provides an idealised laboratory condition to study the influence of different geotechnical parameters over stability of an underground structure. An investigation was done using FLAC^{3D} package, which adopts finite difference method. In this study, an elastic constitutive model was used for strongest strata. In elastic model, loading characteristics of rock bolting in the numerical model is found to be closer to their actual field behaviour (Basarir *et al*. 2015) due to development of maximum elastic stress. The field observation revealed that the immediate roof strata hardly deformed due to its higher strength value. However, the immediate floor started deteriorated with time. Under such circumstances, the immediate floor including coal seam and other strata were given MCSS and elastic properties respectively in the numerical model. The elastic model of FLAC^{3D}, incorporating the Sheorey failure criterion (1997) for the rock mass was used for this study. Basically, this criterion uses the 1976 version of rock mass rating (RMR) of Bieniawski (1976) for reducing the laboratory strength parameters to give the corresponding rock mass values. This criterion is defined as:

$$\sigma_1 = \sigma_{cm}(1 + \frac{\sigma_3}{\sigma_{tm}})^{b_m} \qquad \text{MPa} \tag{1}$$

$$\sigma_{cm} = \sigma_c \ \exp(\frac{RMR-100}{20}) \qquad \text{MPa} \tag{2}$$

$$\sigma_{tm} = \sigma_t \ \exp(\frac{RMR-100}{27}) \qquad \text{MPa} \tag{3}$$

$$b_m = b^{RMR/100} \qquad b_m<0.95 \tag{4}$$

where, σ_1 is tri-axial strength of rock mass or major principal stress in MPa, σ_3 is confining stress or minor principal stresses in MPa, σ_c is compressive strength of intact rock in MPa, σ_t is tensile strength of intact rock in MPa, *b* is exponent of intact rock (0.5), which controls the curvature of tri-axial curve, σ_{cm} is compressive strength of rock mass in MPa. In the above equations, the subscript m stands for the rock mass.

The factor of safety is defined as:

$$SF = \frac{\sigma_1 - \sigma_{3i}}{\sigma_{1i} - \sigma_{3i}} \qquad \text{when } -\sigma_{3i} > \sigma_{tm} \tag{5}$$

Otherwise,

$$SF = \frac{\sigma_{tm}}{-\sigma_{3i}} \tag{6}$$

where, σ_{1i} is induced major principle stress in MPa, σ_{3i} is induced minor principle stress in MPa.

Prior to simulation of rock bolt loading chracteristics, the model is validated with pillar design strength formula used in this mine. Stability factor of pillars in the panel of the CSM mine was estimated using Mark-Bieniawski pillar strength formula (Mark and Chase 1997). Further, a recent study (Kumar *et al*. 2018) was carried out for performance of pillar at great depth considering Sheorey pillar strength formula (Sheorey, 1992), which incorporated the influence of depth on pillar strength was also validated here. The properties of rock mass used for the modelling were determined in the laboratory using core samples collected from the field (Table 1). These properties were used to simulate single rhomboid pillars and attempted to validate with Mark-Bieniawski and Sheorey pillar strength formulae (Figure 4). As per the Mark-Bieniwaski and Sheorey formulae, strength of the pillar is 35.32 MPa and 34 MPa respectively. Results (32 MPa) obtained from the simulated study found to be close to these empirical pillar strength formulae. The same rock mass properties are used for simulation of Panel V.

Table 1: Properties intact core specimen and rock mass

Strata	E	v	K	G	d	σ_c	σ_t	RMR	σ_{cm}	σ_{cm}	Φ
Roof siltstone	23	0.14	10.65	10.09	2650	128	12.8	60	17.323	2.909	38.36
Roof sandstone	35	0.15	16.67	15.22	2690	155	15.5	65	26.935	4.24	39.57
Floor Siltstone	18	0.18	9.38	7.63	2600	60	6	50	4.925	0.942	35.90
Floor sandstone	35	0.15	16.67	15.22	2690	155	15.5	60	20.977	3.523	38.36
Coal	2.6	0.25	1.73	1.04	1400	14	1.4	40	0.697	0.152	33.36

E= Young modulus in GPa, v= Poission ratio, K= Bulk modulus in GPa, G= Shear modulus in GPa, d=density of rock mass in kg/m^3, σ_c= Uniaxial compressive strength of intact rock in MPa, σ_t= Tensile strength of intact in MPa, RMR= Rock Mass Rating, σ_{cm}= Uniaxial strength compressive of rock mass in MPa, σ_{tm}= tensiles trength of rock mass in MPa and Φ= Angle of internal friction in degree.

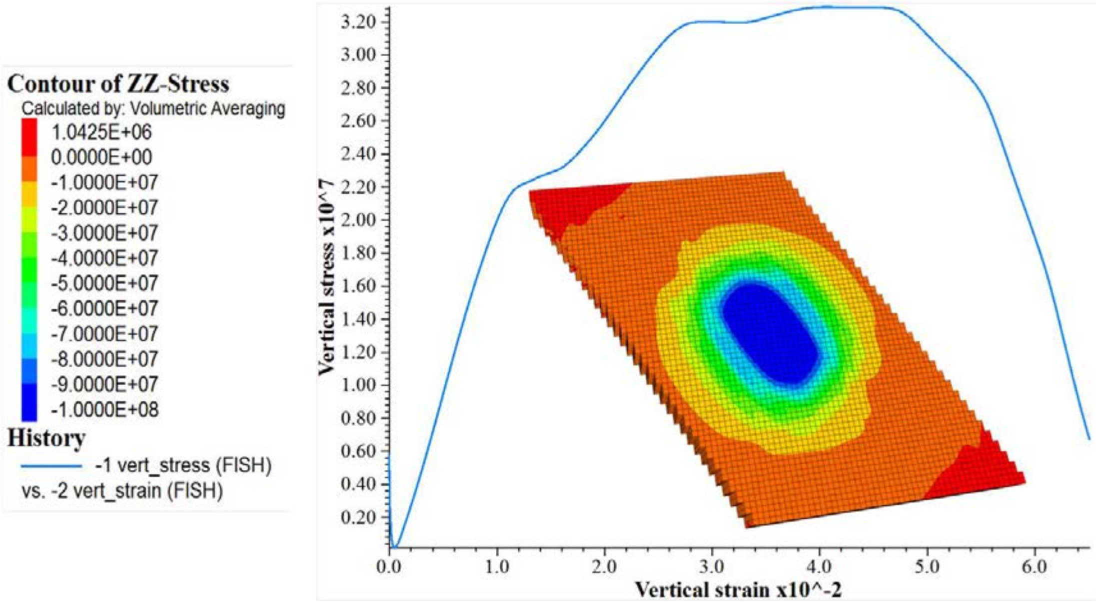

Figure 4: Stress-strain relationship of rhomboid pillar at 850 m by numerical modelling

Generation of model

Considering the site conditions of the studied Panel V, a model 195 mlong, 113 m wide and 104 m high was developed (Figure 5) for the study. In this model, theoretical value of vertical *in situ* stress (0.025 x depth of cover, MPa/m) was applied. As per the actual measured horizontal *in situ* stress near the working panel, major and minor horizontal to vertical *in situ* stress set to ratio of 1.3 and 0.64 respectively in the simulated panel. Directions of the major and minor horizontal stresses are applied along with the width and length of the panel. The calibrated MCSS and elastic properties used for simulating one pillar were directly used in the developed numerical model. The size of pillars was kept as per actual development in this panel. Width and height of gallery was fixed at 5 m and 4 m respectively. A truncated load of 20 MPa (0.025 x depth of cover) for the unmodelled portion of the overlying strata was applied over the model. The sides and bottom boundaries of the model were fixed and the top was kept free. Development of the coal seam in the simulated model has been carried out till the strata monitoring period. Rock bolts of 2.4 m in roof and in pillars were installed (Figure 6) in the model as per actual practice in field.

Properties of different elements of reinforcement and other parameters reported by Holy (2018) were used in CSM mine were considered during the simulation. Grout stiffness, $\mathrm{K_g}$ and cohesive strength, $\mathrm{C_g}$ are determined (FLAC3D, 2012) using equations 7 and 8 respectively.

$$K_g \cong \frac{2\pi G}{10\ln\left(1+\frac{2t}{D}\right)} \tag{7}$$

$$C_g = \pi(D+2t)\tau_{peak} \tag{8}$$

where *G* is grout shear modulus, *t* is annulus thickness, *D* is diameter of roof bolt, τ_{peak} is shear strength of grout/rock or bolt/grout interface. Considered properties of reinforcement materials are given in Table 2.

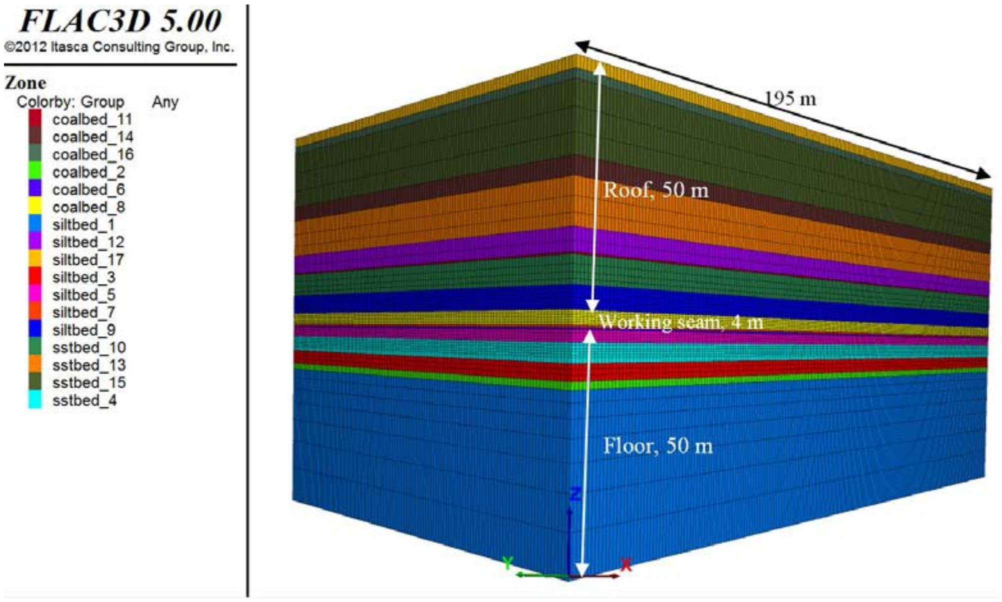

Figure 5: Block formation for the considered site in FLAC3D

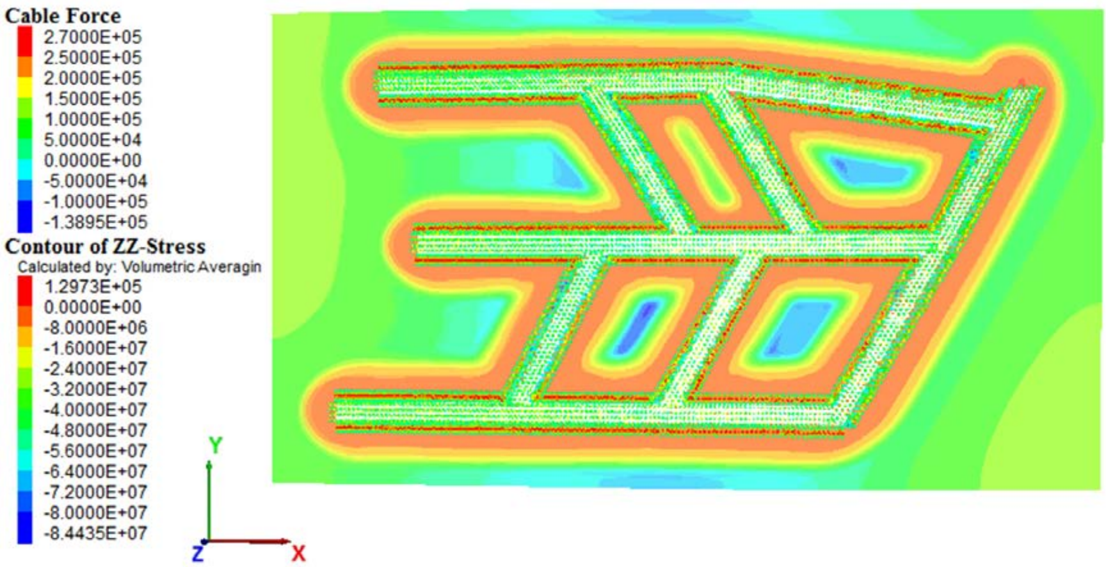

Figure 6: Development of coal seam in panel V

Table 2: Properties of reinforcement

Reinforcement	Parameter	Value
Grout	Grout stiffness per unit length (N/m^2)	1.7e^{10}
	Grout cohesive strength (N/m)	2.6e^{5}
	Grout exposed perimeter (m)	8.95e^{-2}
	Shear strength of grout/rock or bolt/grout interface (MPa)	3
Rock bolt	Cross sectional area (m^2)	3.8e^{-4}
	Young's modulus (GPa)	200
	Tensile Yield strength (N)	2.43e^{5}
	Pretension (N)	2.94e^{4}

IMPORTANT FINDINGS

Under the existing high stress conditions and strong overlying strata, the observed value of pillar dilation (up to 300 mm) was more than the displacement of immediate roof (7.8 mm). Further, development of axial load on rock bolts depends on the bonding strength between the bolt/rock-grout interfaces and deformation within rock bolted zonealso. In Panel V, axial load over instrumented bolts was measured at five different locations including three-way intersections. A range of 1-6.8 tons axial load developed over instrumented bolts which was around 70% less than the bearing capacity of the installed rock bolts. Maximum roof displacement and pillar dilations were observed to be 10.36 mm and 292 mm respectively (Figures 7 and 8). In the simulated model, higher axial load over rock bolts observed in rib bolts than roof rock bolts, which might be due to the influence of displacement within rock bolted zone. In the simulated mode, maximum values of tensile axial load were found to be in the middle of the rock bolts. Generally, average rock load over roof rock bolts was observed to be less than 10 tons, at some location it was observed up to 22 tons. Generally, more axial load observed in rib rock bolts, compared to the roof rock bolt. Maximum observed values of the axial load over rib bolts found to be 27 tonnes. It was found that rib bolts was installed parallel to the direction of major horizontal stress, received relatively higher value of axial load than those installed along minor horizontal stress direction (Figure 9). It might be due to the release of higher strain energy in the major horizontal stress direction compared to the minor direction of horizontal stress with rib dilation due to discontinuity of the coal seam by roadways. However, in order to establish such unique observation, extensive modelling is required to be simulated. Axial loads observed over rock bolts at five different locations are given in Figure10.

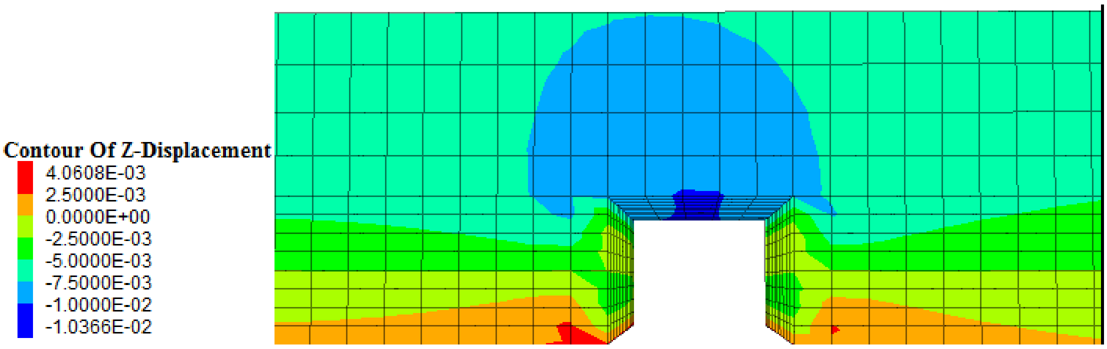

Figure 7: Vertical roof displacement in roadways

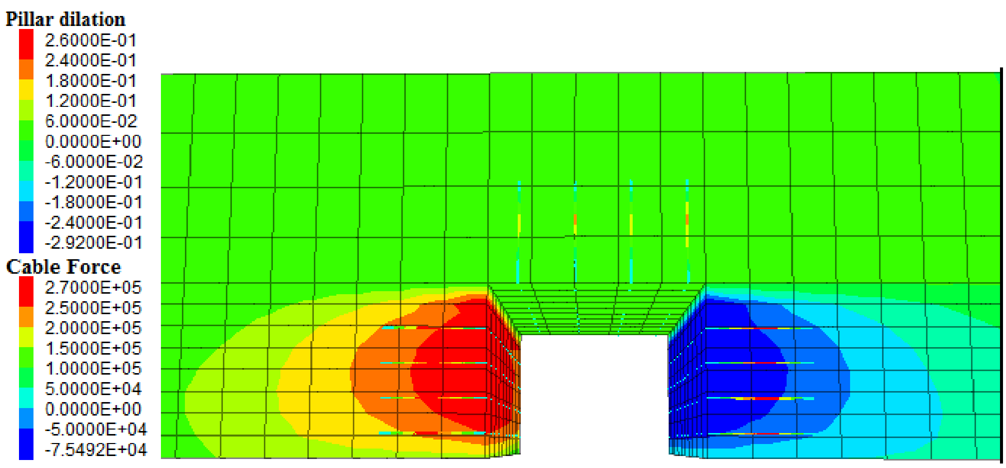

Figure 8: Pillar dilation observed in simulation of panel V

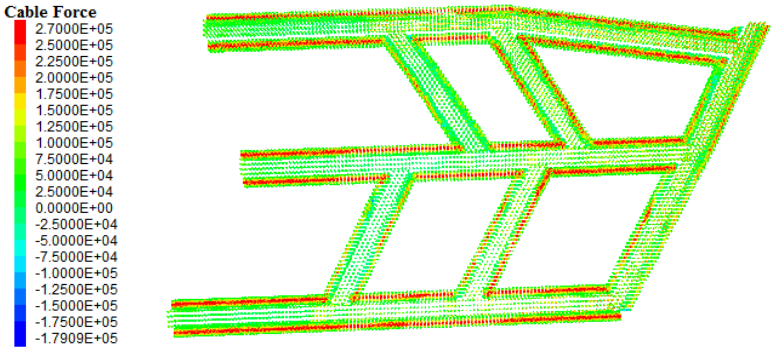

Figure 9: Induced axial load over rib and roof rock bolts in the simulated model

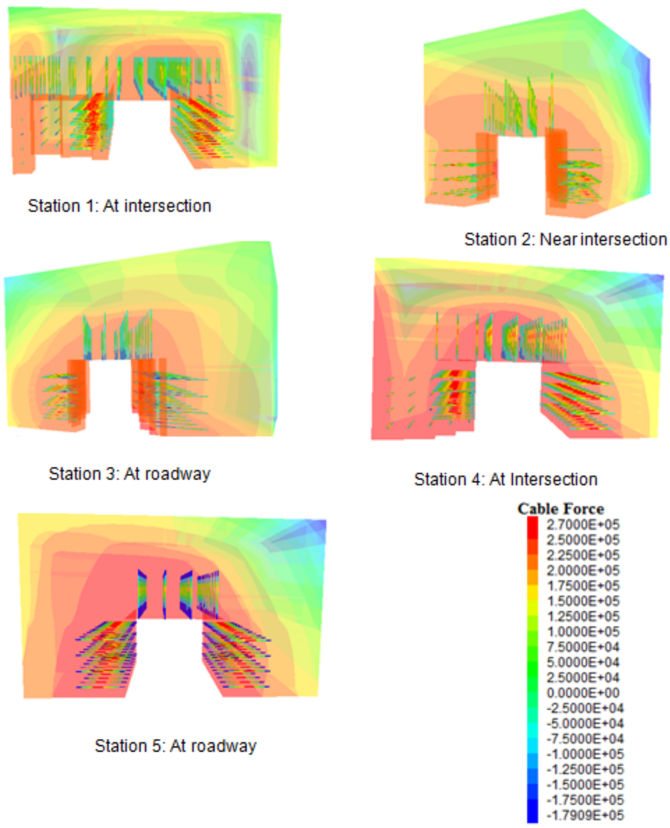

Figure 10: Axial load observed near five different locations of fields monitoring

CONCLUSIONS

Loading characteristics over rock bolts under higher stress conditions were examined using numerical models. Less roof displacement was observed in roof compared to pillar dilation, which might have occurred due to the presence of competent roof having higher uniaxial compressive strength. The numerical modelling revealed that more axial load was observed in rib rock bolts than roof rock bolts. The simulation was done by considering only four roof rock bolts in a row (excluding additional supports) and an average 10 tonnes axial load was observed, which seems to be competent enough to support the existing site conditions of the panel. In the simulated model, the maximum 27 tonnes load observed over rib rock bolts with 60-292 mm pillar dilation. This is a preliminary study on numerical modelling based on performance of rock bolts at higher stress conditions. Extensive and intensive studies are required by replicating the actual site conditions in order to establish the effect of major and minor horizontal stress.

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