CHAPTER 6.

SIMULATION OF CASE STUDY AND ITS VALIDATION

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6.1 General

Validation is the process of assessing how well mining models perform against field data. These validations of mining model are essential, before deploying them in different geo-mining condition.

In this chapter, two cases were taken for numerical simulation study using FLAC^{3D}. The model is prepared which replicate the actual situation of the mine. Instrumentation data of the bord and pillar panel will give sufficient information of the roof behaviour and load developed on the bolt during the depillaring process.

6.2 Numerical Modelling

In the present scenario, numerical modelling is frequently used to solve a rock mechanics problem. It deals with a complex problem, converted into a simpler one, combining them through the discretization technique. The total domain area of the problem is discretising into smaller zones/meshes of different shapes and sizes. After discretization of the area, all the relevant conditions are defined and incorporated. All necessary boundary conditions are applied for the considered specific problem before the application of basic governing equations for each element. Since all the elements are connected to each other, the resulting equations are also in relation to each other. Typical parameters are considered for the analysis of geo-technical problems are the geometry of the area, rock properties (RMR), and in-situ stress field.

Various numerical model techniques have been developed (Jing, 2003; Cogan et al., 2006, Hudson and Feng, 2007), utilizing a wide spectrum of algorithms and mathematics. These are the Finite Element Method (FEM), Finite Difference Method

(FDM), Boundary Element Method (BEM), Distinct Element Method (DEM), Discrete Fracture Network (DFN), and Hybrid models are available. So, out of which suitable approach is chosen for the planned investigation is very important.

FLAC^{3D} software is a three-dimensional explicit Finite Difference Method (FDM) based program for engineering mechanics computation. This package has the capability to simulate the behaviour of three-dimensional structures built with soil, rock, or other materials that undergo plastic flow when their yield strength is reached. The mixed discretization zoning technique coupled with the explicit, Langragian calculation scheme ensures that plastic collapse and flow are modelled accurately.

6.3 Mohr Columb Failure Criteria

It is very important to decide the material model as it plays a crucial role in numerical modelling. Normally, the models are classified into two parts elastic and plastic. The elastic model is based on Hooks Law of elasticity, and the plastic model comprises Mohr – Columb (MC) and Mohr Columb Hardening/softening (MCSS) models.

Rock is behaving as strain-softening material. In this case, immediate strata and coal is being taken as Mohr Columb strain softening (MCSS). FLAC gives the facility to simulate the model using MCSS. For calibration of the material which replicates the actual behaviour of rock strata is required to observe the results in an effective manner. The methodology is being developed to achieve the appropriate material property.

Empirical Failure Criterion (Sheorey, 1997), established for Indian geo–mining conditions, gives reasonable results. As per this failure criteria, the parameters are taken into consideration for simulating the model are RMR, compressive strength, and tensile strength of rock. Sheorey has proposed empirical Rock Mass Failure Criteria in 1989

(Sheorey, P. R., et al., 1989). It has been used in the present analysis. It is expressed as follows.

$$\sigma_1 = \sigma_{cm}\left(1 + \left(\frac{\sigma_3}{\sigma_{tm}}\right)^{b_m}\right) \tag{6.1}$$

$$\sigma_{cm} = \sigma_c exp\left(1 + \left(\frac{RMR-100}{20}\right)\right) \tag{6.2}$$

$$\sigma_{tm} = \sigma_t exp\left(1 + \left(\frac{RMR-100}{20}\right)\right) \tag{6.3}$$

$$b_m = b^{\frac{RMR}{100}} \quad b_m < 0.95 \tag{6.4}$$

where, σ_1 is triaxial strength of rock mass and σ_3 is confining stress in MPa, σ_c and σ_{cm} is the uniaxial compressive strength for intact rock and rock mass respectively in MPa, σ_t and σ_{tm} is the tensile strength of intact and rock mass in MPa, RMR is Bieniawski Rock Mass Rating and b and b_m are the exponent in failure criterion for intact rock and rock mass, respectively. The value of b for coal measure rock has been taken as 0.5.

In strain-softening material, peak strength parameters have been deduced by an empirical failure criterion prepared by Sheorey failure criteria (SFC) for Indian coalfields. So, using the above criterion converted into Mohr Columb Failure Criteria (MC FC). This will give the peak value of c and ϕ for different values of RMR using the below expression and best fit graph shown in figure 6.1.

$$\sigma_1 = 2c\left(\frac{cos\phi}{1 - sin\phi}\right) + \sigma_3\left(\frac{1+ sin\phi}{1 - sin\phi}\right) \tag{6.5}$$

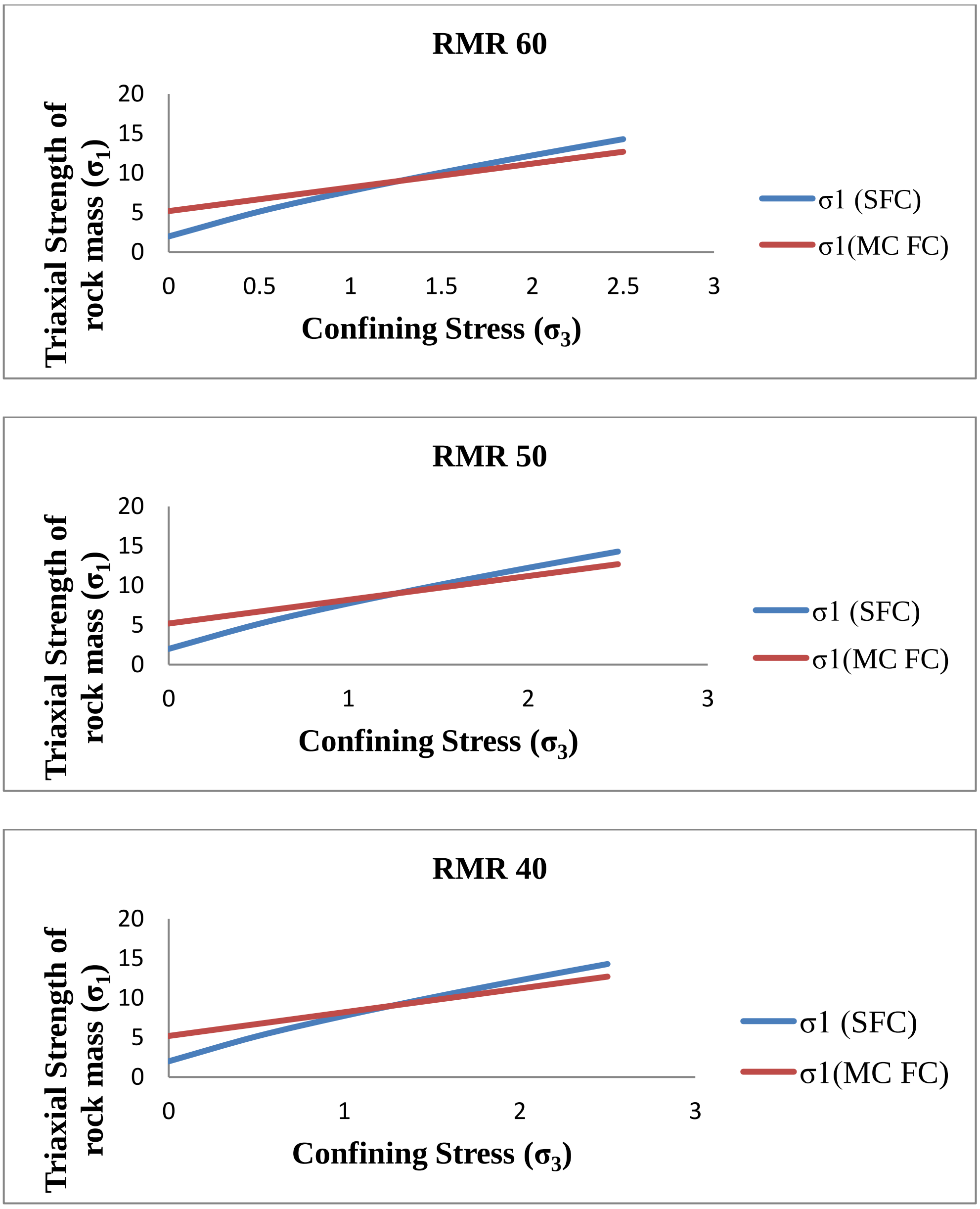

Figure 6.1 Best fit curve to deduce peak value of strength in MCSS

6.4 Calibration of Strain softening model

The rate of reduction of residual strain in the strain-softening model is derived with the help of extensive literature review and hit and trial considering various geo mining parameters of different mines shown in table 6.1. The reduction of strain-softening parameters has been determined by calibration of the model with a CMRI – RMR based support design approach. The reduction of strength with plastic strain for different values of RMR is shown in figure 6.2.

Table 6.1 Name of mine with geo-mining and geo-technical parameters

Sl. No.	Colliery Name	Seam Thickness (m)	Depth (m)	Gallery Width (m)	RMR	UCS (MPa)	Immediate Roof Thickness (m)	RLH (m)
1	lACHIPUR	2.4	220	4.2	25.02	37.5	1.8	1.6
2	BAHUlA	5.4	96	4	43.94	34.9	3.22	1.6
3	JAMBAD	2.12	71	3.6	40.32	74.5	0.6	0.6
4	JAMADOBA	8.84	386	4.2	35.00	31.5	4	3
5	KHARKHAREE	2.2	160	4	54.00	51.0	2	3
6	RAKHIKOl	5.5	275	3.65	21.31	18.3	3	3.25
7	BANKOLA	4.67	105	4.5	32.60	24.2	6.1	2.5
8	SUDAMDIH	5.5	300	3.6	66.00	52.9	1.5	1.5
9	JHANJRA	3.64	18	4	27.92	26.5	2	1
10	KUJU	6.07	30	3.2	15	18	3.1	1.2
11	LALMATIA	8.97	30	3.6	16	35	1.3	2

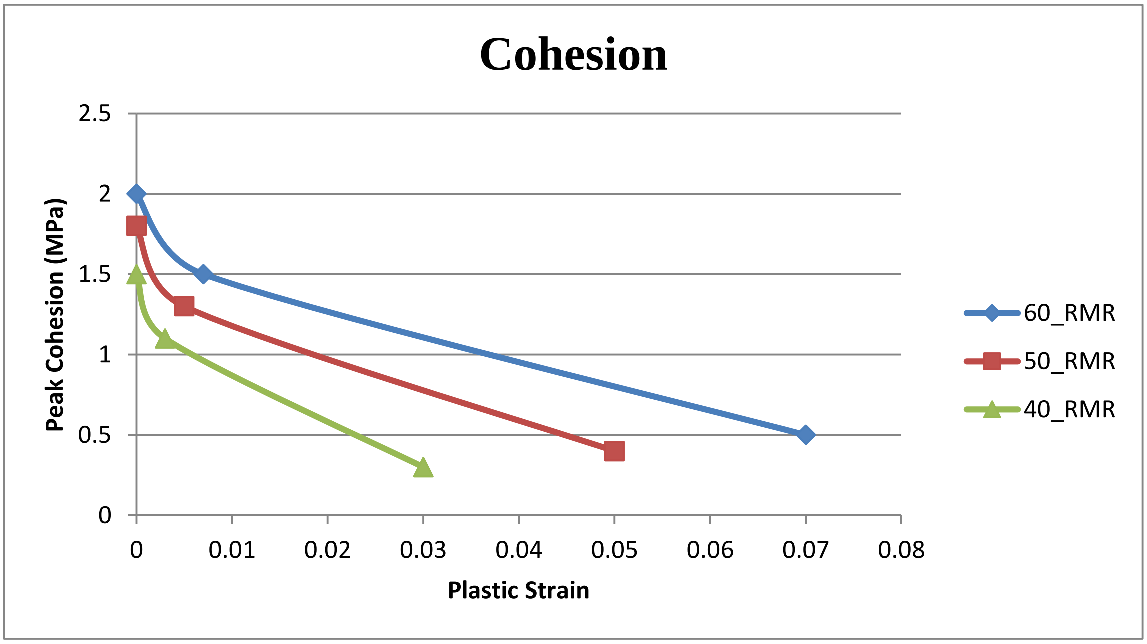

Figure 6.1 Rate of reduction of residual strain for different values of RMR

The observed values will give the reduction of residual strain (shown in figure 6.2), which will help to validate the two cases that were taken into consideration. A detailed description of the two cases is illustrated in the below section.

6.5 Simulation of CASE 1 and CASE II

In this section, the three-dimensional model of Pinoura mines is prepared to analyses the rock load height (RLH) and axial load developed on the bolt. The detailed description of mine has already been discussed in chapter 4. The following steps have been adopted to validate the model based on observed instrumented rock bolt data.

6.5.1 Model preparation

Three-dimension model consists four numbers of layers, i.e., floor, coal, immediate roof, and main roof. The dimension of the three-dimensional model is taken as coal seam thickness 5 m; the immediate roof is taken as 0.5m coal, 5 m shale, and 25 m main roof taken as sandstone. The floor height is taken as 30 m shown in figure 6.3.

For simplicity and saving time, the dimension of the actual panel is 12 rows of pillar × 4 pillars in a single row is reduced to 6 rows of pillar × 4 pillars as shown in figure 6.4. The results will not change because the actual study location (marked as red in figure 6.4) is before three rows of an intact pillar.

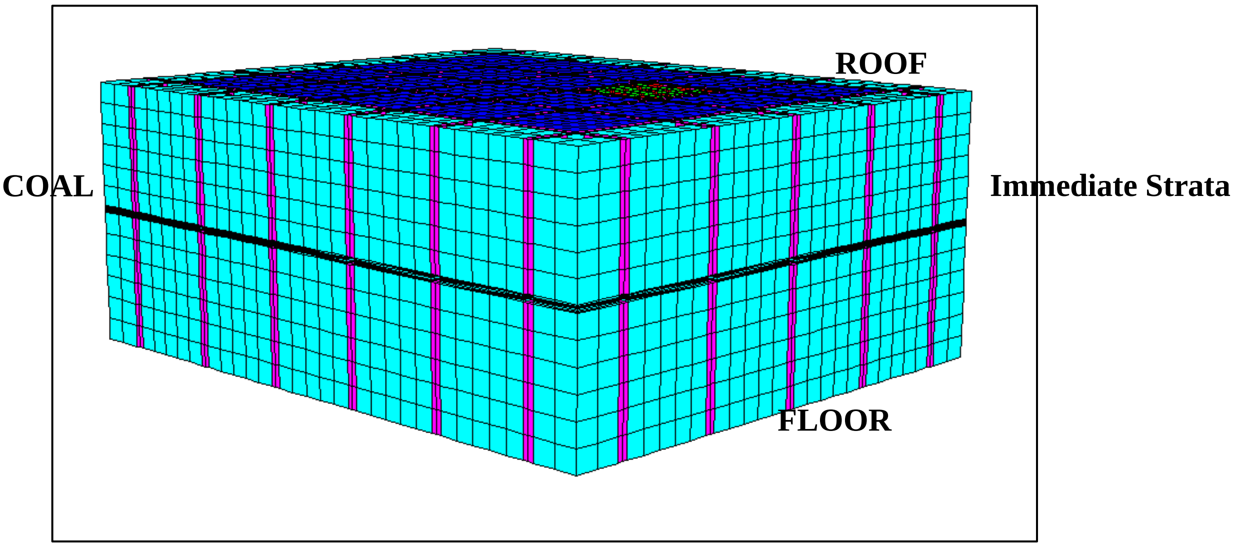

Figure 6.3 Three-dimensional view of bord and pillar panel

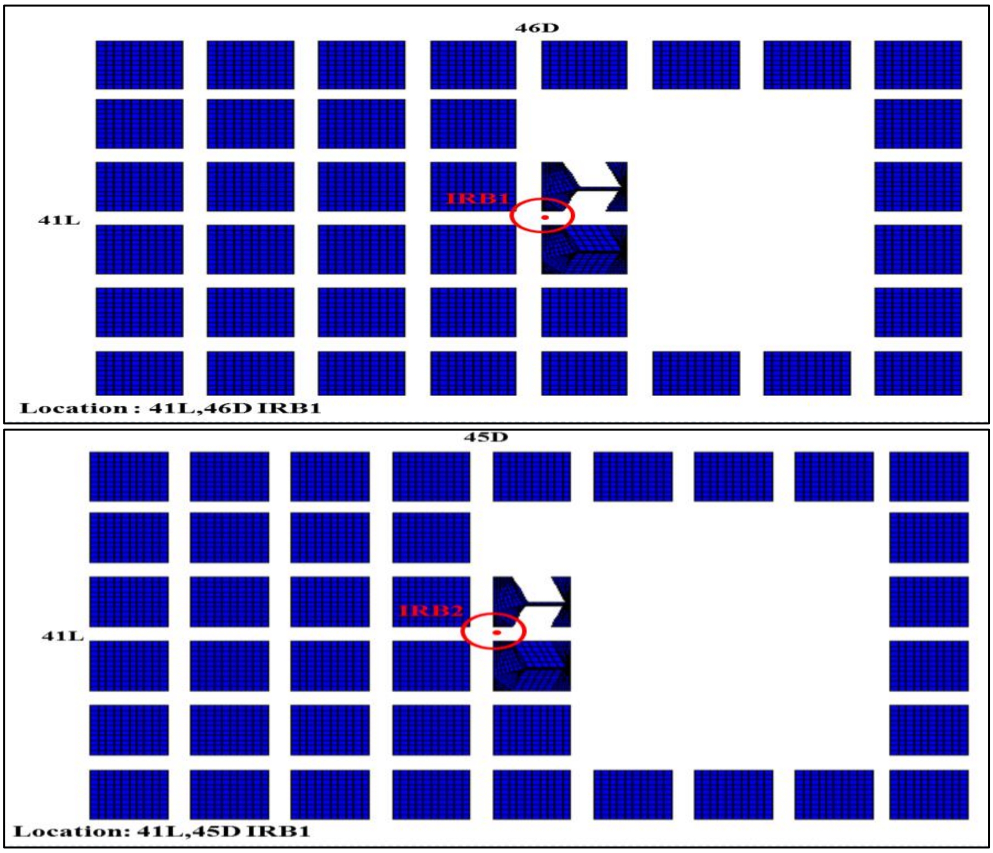

Figure 6.4 Plan view of bord and pillar panel of Pinoura mine showing instrumented rock bolt location

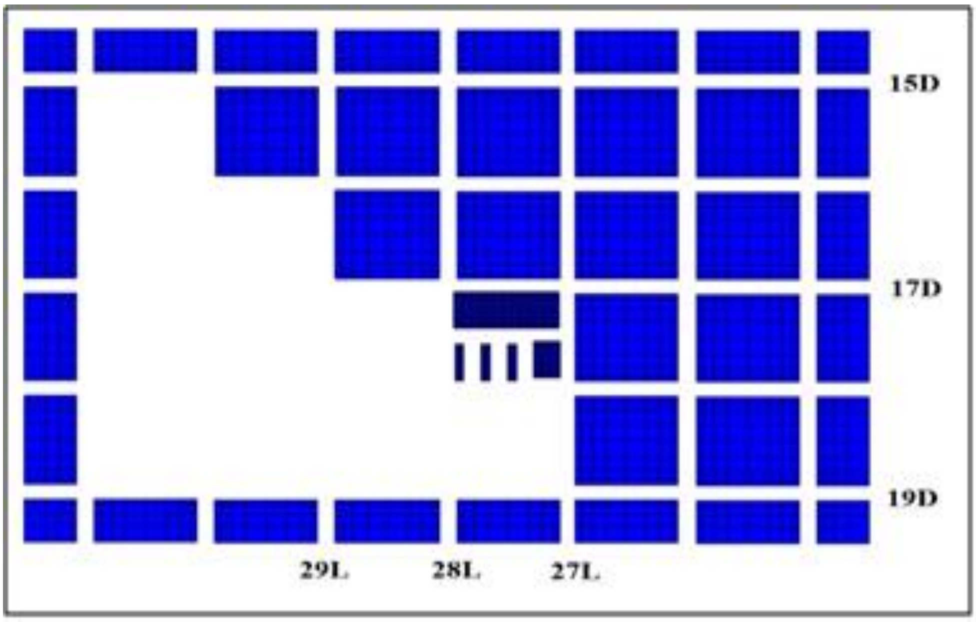

Figure 6.5 Plan view of bord and pillar panel of GDK 5A Incline, SCCL mine

The discretization of the pillar is 2 m×2 m in the panel except in the focused study area. In the focused area, fine discretization has been done to observe effective splitting and slicing during simulation. The immediate strata are finely discretized compare to the main roof and floor, as shown in the three-dimensional view of the model in figure 6.3. The model geometry of GDK 5A, the incline of SCCL coalfield, is shown in figure 6.5.

6.5.2 Material Properties

The strain-softening model is considered for roof and coal pillars because the behaviour of coal and roof is not perfectly elastic. It behaves as elastoplastic behaviour. The physico-mechanical properties and geo–technical properties are considered for the simulation process as shown in table no. 6.1 and table no. 6.2 respectively for Pinoura mines. The physico-mechanical properties and geo–technical properties are considered for the simulation process as shown in table no. 6.3 and table no. 6.4 respectively for GDK 5A Incline, SCCL mines.

Table 6.2 Physico-mechanical properties of the rock strata (Pinoura Mine, SECL)

Rock Type	Modulus E, (MPa)	Poisson's Ratio	Density (kg/m3)	UCS MPa	Tensile Strength MPa
Shale	4000	0.41	2270	24.50	1.64
Sandstone	1000	0.31	1970	32.50	2.17
Coal	4000	0.27	1350	20.50	1.37

Table 6.3 Geo-technical properties for the numerical model (Pinoura Mine, SECL)

Rock Strata	Shear Modulus (GPa)	Bulk Modulus (GPa)	Friction angle (degree)	Cohesion (MPa)	RMR
Top Layer	1.98	3.47	35	1.5	42
Coal	1.57	2.89	25	1.3	-

Table 6.4 Physico-mechanical properties of the rock strata (GDK 5A Incline, SCCL mine)

Rock Type	Modulus E, (MPa)	Poisson's Ratio	Density (kg/m3)	UCS MPa	Tensile Strength MPa
Grey- pyrite Sandstone	15,000	0.25	2500	18.4	1.85
White Sandstone	10,000	0.25	2500	15.0	1.5
Coal	4000	0.27	1350	20.50	1.37

Table 6.5 Geo-technical properties for the numerical model (GDK 5A Incline, SCCL mine)

Rock Strata	Shear Modulus (GPa)	Bulk Modulus (GPa)	Friction angle (degree)	Cohesion (MPa)	RMR
Top Layer	4.0	6.6	35	2.5	51
Coal	1.57	2.89	35	2.0	-

6.5.3 In-situ stresses

The in-situ stresses are well-known factors that influence the stability of an underground structure during mining. For Indian coalfield, (Sheorey et al., 2001) proposed an equation for average in–situ vertical stress and horizontal stress given below in equation (6.6) and equation (6.7), respectively. In this study, these values are taken into consideration during the simulation process.

$$\sigma_v = \rho g H \tag{6.6}$$

$$\sigma_h = \sigma_v \left(\frac{\upsilon}{1-\upsilon}\right) + \frac{\beta E G}{1-\upsilon}\,(H + 1000) \tag{6.7}$$

where, σ_v is vertical stress in MPa, ρ is density in t/m^3, H is depth in m, g is acceleration due to gravity in m/s^2, σ_h is horizontal stress in MPa, υ is Poisson's ratio, β is the coefficient of thermal expansion in /°C, E is young's modulus in MPa, G is the thermal gradient °C/m

6.5.4 Boundary Condition

The model height is taken as 30 m. So, the remaining extra vertical stresses are applying to the top of the model as per the depth of the coal seam. All six faces of the model are fixed.

6.5.5 Input data for the rock bolt model

The FLAC^{3D} package has the capability to incorporate different properties of a roof bolt for the reinforcement of rock mass. It provides a structural element of pileSel which are two node, straight finite elements with one axially oriented translational degree of freedom per node (Itasca, 2012). It provides information on each node at any interval along the length of the bolt. Generally, the manufacturer of the bolt supplies information regarding the area, modulus of yield force resistance of the bolt. However, the properties related to grout are more difficult to estimate. In FLAC, the grout annulus is assumed to behave as an elastic-perfectly plastic solid. The manual of FLAC describes the process for estimation of grout properties (Itasca, 2012). Properties of different elements of reinforcement used in the modelling are derived from the existing norms of designing the reinforcement (DGMS Circulars, 209 and 2010). Grout stiffness (Cs_nstiff) and the cohesive strength (Cs_ncoh) are determined using the below equation.

$$\mathrm{Cs}_{\mathrm{nstiff}} = \frac{2\pi G}{10\ \ln\left(1+\frac{2t}{D}\right)} \tag{6.8}$$

$$\mathrm{Cs}_{\mathrm{ncoh}} = \pi\ (D + 2t)\tau_{peak} \tag{6.9}$$

where, G is grout shear modulus, t is annulus thickness, D is diameter of roof bolt, τ_{peak} is shear strength of rock/grout interface.

Different properties are to be specified in table 6.6 shows the properties for the rock bolt considered in model for simulation.

Table 6.6 Input data for rock bolt element used for numerical model

Properties	**Symbol**	**Unit**	**Value**
Rock Mass:			
Bulk Modulus	K	Pa	1e9
Shear Modulus	G	Pa	1e9
Density	Dens	Kg/m^3	2500
Rock Bolt and Grout:			
Elastic Modulus of the rock bolt steel	E	GPa	125e9
Cross sectional area of the rock bolt bar	Area	m^2	2e-4
Moment of Inertia of rock bolt bar	I	m^4	3.22e-9
Exposed perimeter of the rock bolt		m	5e-2
Maximum tensile force	Yield	N	7.8e5
Interaction between the rock bolt and rock mass:			
Stiffness of the normal coupling spring	Cs_nstiff	N/m^2	1.35e10
Cohesive strength of the normal coupling spring	Cs_ncoh	N/m	2.4e6
Friction strength of the normal coupling spring	Cs_nfric	Degree	45°

6.5.6 Results of CASE I:

At Pinoura mine, the MD is done at a shallow depth of cover and under relatively weak, laminated strata having RMR 42. Therefore, the caving inside the goaf was regular without any overhang except in the case of left-out pillars/fenders inside the goaf. The observed value of induced stresses is generally low, and there is less spalling in the natural supports in and around the goaf edge. It is observed that the installed support pattern in and around the goaf area are sufficiently designed and working effectively.

a. The height of roof yield (RLH) is 1 m observed at the location where instrumented rock bolt (IRB1) is installed, shown in figure 6.6. Different types of failure and maximum axial load exerted on the bolt are also depicts in the figure.

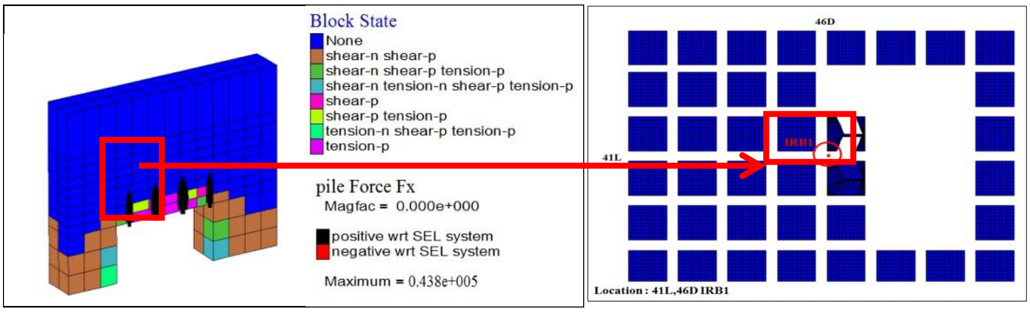

Figure 6.6 Maximum value of axial load on bolt and roof yield at Pinoura Mine at IRB1

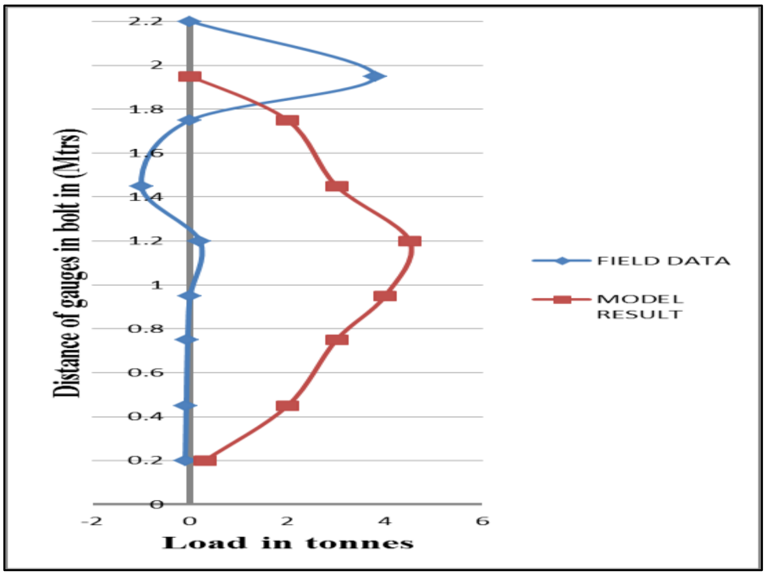

Figure 6.7 Axial bolt profile along its length observed from field and model result at (IRB_1), Pinoura Mine

The results that the maximum axial load at IRB1 is 4.38 tonne observed from the model are reasonably matching the field observation 3.7 tonne as shown in figure 6.7. It also represents the variation of axial load at different horizontal sections along the length of the bolt.

b. The failure of immediate strata, also termed as rock load height (RLH), is 1.5 m observed at the location where an instrumented rock bolt (IRB2) is installed, shown in figure 6.8. Different types of failure and maximum axial load exerted on the bolt are also depicts in the figure.

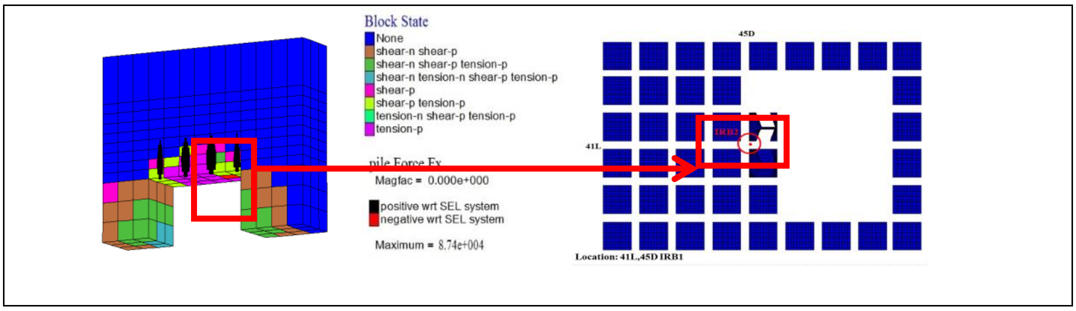

Figure 6.8 Maximum value of axial load on bolt and roof yield at Pinoura Mine at IRB2

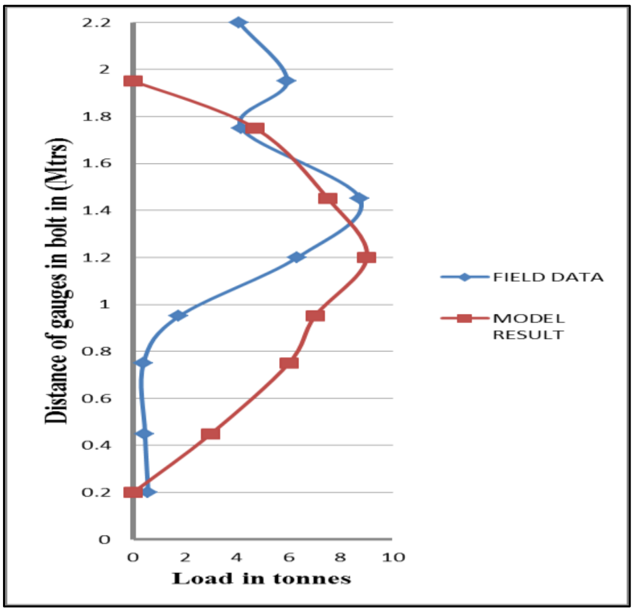

Figure 6.9 Axial bolt profile along its length observed from field and model result at (IRB_2), Pinoura Mine

The results that the maximum axial load at IRB2 is 8.74 tonne observed from the model are reasonably matching the field observation 8.21 tonne as shown in figure 6.9. It also represents the variation of axial load at different horizontal sections along the length of the bolt.

6.5.7 Results of CASE II:

At GDK-5A Incline, SCCL, the conventional Bord and Pillar mining is doing with LHD/SDL. The depth of cover is 180 m, having laminated overlying strata of sandstone of RMR 51. At GDK-5A incline of SCCL, the conventional split and slice method of extraction was being performed for developed pillars.

Three bolts in a row at a spacing of 1.2 m in split galleries and two wooden props in a row at a spacing of 1.2m in each slice were used as a support system during the depillaring operation. In addition to that, a wooden cog was also erected at the center of the slice junction. At goaf edge, three sets of wooden cogs were erected skin to skin. Induced caving was performed at regular intervals of face advance, so that chances of overriding could be avoided.

Strain gauged bolt GSG1was installed in the level gallery when the position of the face was about four pillars away from the site, as shown in figure 6.10. The observation was continued over a period of 90 days till the face reached there. From figure 6.12, it is clear that the maximum load developed in the bolt was around 13.4 tonne at a distance of 0.95 m from the roof level when the slicing operation in front of GSG1was being performed.

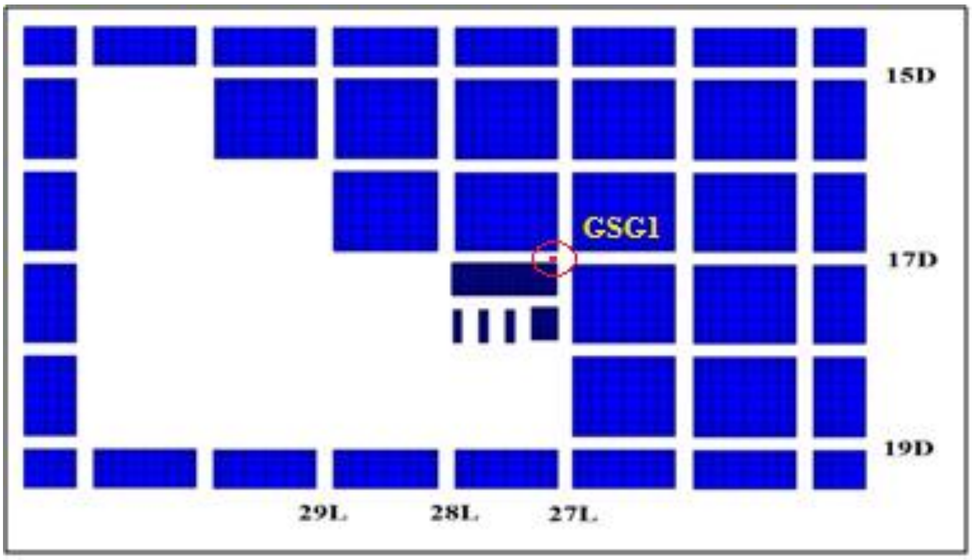

Figure 6.10 Bord and Pillar no. 31 at GDK-5A, Incline, SCCL with instrumented rock bolt location GSG1

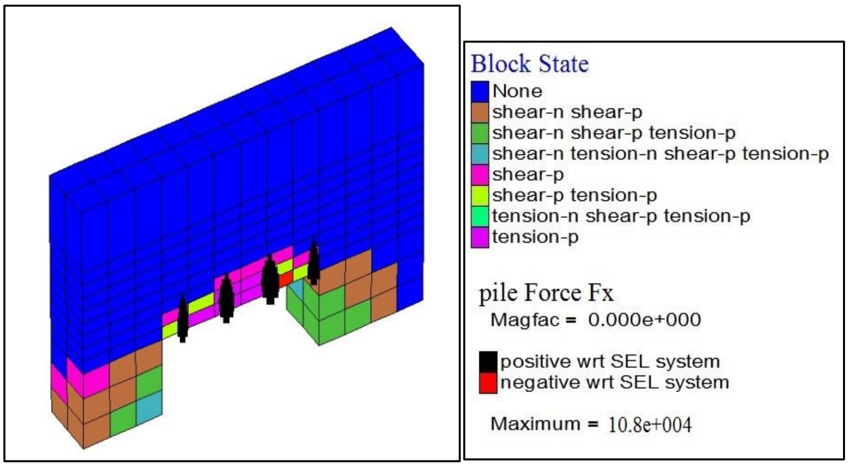

Figure 6.11 Maximum value of axial load on bolt and roof yield at instrumented rock bolt location GSG1 for GDK-5A, Incline, SCCL

The roof yield (RLH) is 1.5 m observed at the location where the instrumented rock bolt (GSG1) is installed, shown in figure 6.11. Different types of failure and maximum axial load exerted on the bolt are also depicts in the figure.

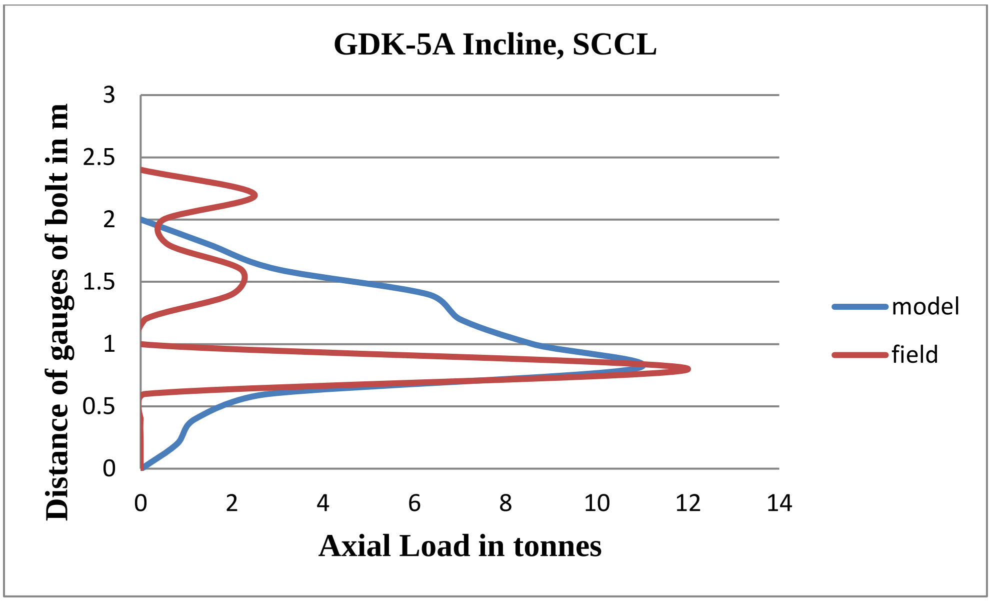

Figure 6.12 Bolt profile along its length observed from field and model result at (GSG1), GDK-5A, Incline, SCCL

The results that the maximum axial load at GSG1 is 11.8 tonne observed from the model are reasonably matching the field observation 13.4 tonne, as shown in figure 6.12. It also represents the variation of axial load at different horizontal sections along the length of the bolt.

6.6 Summary

In this chapter, various numerical simulation methods are discussed. Out of which best suitable numerical technique is adopted based on literature survey. The calibration of the material properties is being established to represent the reasonable results on analyzing roof behaviour in terms of RLH and axial load developed on the bolt. The established best curve is being deduced and tested with the different field data. Validation of the model is carried out using the observed value of instrumented bolt at various locations in the panel. Observed results show the model is reasonably matching with the field data in both cases.