SESSION 19: MINING

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APPLICATIONS OF VARIOUS ROCK MASS CLASSIFICATIONS TO UNSUPPORTED OPENINGS AT MT ISA, QUEENSLAND : A CASE STUDY
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NUMERICAL ANALYSIS OF FAILED CEMENT FILL AT ZC/NBHC MINE, BROKEN HILL
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THREE-DIMENSIONAL ANALYSIS OF ROCK FAILURE ZONES AROUND RECTANGULAR MINE OPENINGS IN ROOM AND PILLAR WORKINGS
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EXPERIENCE WITH THE MONITORING OF CROWN PILLAR PERFORMANCE IN TWO AUSTRALIAN MINES
G. Worotnicki, J.R. Enever, B. McKavanagh, A. Spathis and R. Walton; Vol 2, 161-168

Paper by N.R.P. Baczynski

Mr W. Bamford suggested that, in Fig 1, for the benefit of non-geologists, the orientation of each joint set should have been shown. Perhaps a block diagram would also have been useful. He felt that the author had made a good point about correlations being a function of rock type; perhaps these were rather a function of the unit block shape. In sections 5.1 and 5.2, Table V estimates had been given for field strength and modulus. Figures for laboratory strengths quoted during the presentation had indicated the following reduction factors: Strength = 70/185 = 0.4 and Modulus = 20/100 = 0.2. He asked whether there was any significance in this greater reduction factor for the modulus. Dr Baczynski agreed with some of the suggestions made by Mr Bamford. However, he noted that in section 5.1, the unconfined compressive strength referred to the hanging wall of the slope, while the pillar strength came from a different area. At the time of writing there was no field data to check up on modulus values. Later work had shown them to be 50 percent low. He also pointed out that it was not a single value problem since the modulus changed.

Mr P. Pells commented that the author had made a very valuable contribution in his critical evaluation regarding the mass classification systems. There was a very good correlation between Bieniawski's Geomechanics System and Barton's System and this was because they both relied very heavily on RQD or on factors related to RQD. A statistical evaluation of seventeen cases by B. McMahon had shown that RQD contributed about 80% to the geomechanics system and 69% to Barton's system. Bieniawski's system was so strongly related to RQD that one questioned whether in general his more complicated system had any greater value. It was also very important to note that the relationship published by Bieniawski between RQD and RMR had been taken directly from Deere et al's RQD relationship and had no greater intrinsic accuracy. Furthermore, the relationship between RMR and rock mass strength (cohesion and friction) was based on Bieniawski's opinion, and not on factual data. From the point of view of the determination of tunnel support and unsupported spans, Barton's system seemed consistently superior to the geomechanics system.

Dr Baczynski replied that he had not looked at the problem from this point of view, but felt that there might be merit in it.

Mr G. Worotnicki felt that the rock classification scheme for tunnels could be applied to mines, owing to the similarity.

Paper by B.N. Whittaker and A.S. Grant

Mr W.E. Bamford noted that Table II gave an unconfined compressive strength of 36 MPa for the ironstone, while Fig 5 showed maximum vertical stresses of 19 MPa. Fig 6 showed failure in the roof, and he asked how stresses of 19 MPa could cause failure in a material with an unconfined compressive strength of 36 MPa. He asked whether any possible effects of floor heave or floor failures had been considered during the analyses. He was not sure whether roof bolts had been anchored in the mudstone; in view of the author's comments about extensometers, this appeared to be difficult. He asked the author to describe the lengths, spacings, and anchorage methods for the installed system. In view of the extensive presentation of extensometer and roof sag measurement techniques, he asked why the authors had not presented any correlations between the measured and computed deformation data.

Mr A.S. Grant replied that in section 4 it could be seen that the mode of failure was determined according to the value of σ_3/σ_1 Failure in a loaded rock mass did not just occur when the major principal stress exceeded the uniaxial compressive strength and he cited the possibility of failure in tension. He pointed out that Fig 4 and Fig 5 showed the vertical component of stress and not σ_1 or σ_3. In any case, the value of 19MPa occurred at the intersection of the roof and pillar (Y=3m) and no failure was indicated at that point at 100m depth.

With regard to the absence of correlations between measured and computed deformations he said that the paper described a test case (with a different thickness for the ironstone roof beam) the object of which was to observe how closely the failure prediction model tied up with reality. The introduction to the paper dealt with the use of extensometers only in respect of the general stability assessment procedures employed at the mine. It had been stated in the paper that the stresses required for the failure prediction programme had been obtained by means of a gravity turn on method. Thus whilst the final stresses were independent of the extraction method, the displacements were not, since they would include those due to the loading of the intact strata before mining began. In order to obtain the displacements associated with the formation of rooms a multi-sequence finite element modelling technique would need to be employed.

Dr B.N. Whittaker added that bolts were needed as the main form of support in the mine described in the paper. Since the clay overlying the ironstone was very weak, an ironstone beam was needed to bridge the mining rooms. The clay would not have been capable of support by the roof bolts alone. The roof beam which gave satisfactory stability with roof bolts and W-straps was 1.5m thick, while the roof bolts needed were only 1.4m long to avoid penetrating the clay. Polyester resin (full column bond) anchorages were used throughout the mine. Roof bolts were generally spaced at 1.4m. Experiments were under way which involved the incorporation of 3m long bolts inclined over the corners of the pillars in order to achieve increased stability at the junctions. Truss supports had proved successful. The floor has tended to lift out and accounted for between 60 and 75 percent of the room convergence. The heavy mine vehicles tended to disturb the floor. The pillars were stable.

Prof Endersbee commented that the failure described sounded like a case of punching shear failure, which was a brittle failure. Dr Chappell commented that punching shear was a secondary phenomena since beam bending allowed roof cracks to open and collapse to occur. Thus, bending was the primary mode of failure.

Mr M.A. Coulthard commented that the technique used in the paper - assessing stability by testing stresses from linear finite element analyses against a yield criterion - was a valuable one. However, the reliability of the assessment depended upon the accuracy of the calculated stresses. The authors had truncated their mesh 5.5m above the roof, which was less than the room height. It was more usual to place mesh boundaries 5 or more excavation diameters away so as to avoid spurious effects. To check such boundary effects for this system, he had performed two analyses using 21-node three-dimensional elements, a slightly coarser mesh than the authors. Because of some data errors made in haste, his geometry had been similar to, but not identical with that used by the authors. Even so, his vertical stresses for a 1.5m ironstone roof beam and 5.5m of roof strata agreed reasonably with those given in Figs 4 and 5. The vertical displacements under the applied uniform overburden load varied by more than a factor of 2 from above the pillar centre to above the room intersection, suggesting that the boundary was too close. When the roof strata had been (crudely) extended to a depth of 22m by adding one layer of 21-node elements the calculated displacement variation under the load became only 1 percent and the variation at the previous boundary had been reduced by one third. The highest stresses in the pillar and in the roof immediately above the rooms had all been lowered by about 10 percent. It therefore appeared that the boundary effects might not be too important.

Mr G. Worotnicki asked if the length of the bolts at the intersection could be increased and by how much? Dr Whittaker replied that increasing length at junctions was being monitored on an experimental basis. To date the bolt forces could not be generated when the bolt penetrated into the clay. He suggested that the trussing effect could be significant but cautioned that the whole roof could still collapse as a block.

Paper by G. Worotnicki, J.R. Enever, B. McKavanagh, A. Spathis and R. Walton

Mr W.E. Bamford commented that it was impressive to see the innovative instrumentation techniques which were being developed. However, he presumed that the authors must have had some probable mechanisms for expected deformations and failures, and magnitudes for expected deformations and stresses in mind when they designed the systems. It would have been interesting to learn of these, and how well they agreed with the eventual observations.

Prof L.A. Endersbee made the comment that, in crown pillars, crushing usually occurred rather than the brittle failure that was observed. He asked where this failure occurred in particular. Mr Enever replied that the slope was old and there had been overbreak occurring. Horizontal cracks with 20-30m dilation and looking like brittle failure were occurring with an attendant increase in noise. Predominant east/west stress build up occurred. Then a dramatic stress change occurred. Later it was found that σ_V and σ_{N-S} had increased but σ_{E-W} had remained the same. The hypothesis was that brittle failure had occurred but had been confined by the walls.

Dr B.N. Whittaker invited the authors to comment on the pattern of transmission of rock stress to the foot and hanging walls with inclined slopes where crown pillars were used. The dissipation of stress from the crown pillar in inclined slope conditions greatly influenced the positioning access and haulage tunnels. He asked what form of stress bulb had been indicated by their studies, and what specific recommendations could be made for the location of a footwall drivage adjacent to a crown pillar in order to avoid stress effects. Mr Enever said that Mr Isa was more typical in this area, and invited his co-author to reply. Mr Worotnicki replied that stresses were investigated not in the walls but in the middle of the seam. He also pointed out the influence of graphite bands in the walls and suggested that their low frictional properties would give a different stress than that predicted by Boussinesqu.

Dr H. Bock commented that the relative

intensity of noise was not surprising but that the displacement seemed to be insensitive to this phenomenon. Mr Enever replied that it was surprising that no displacement had been noted. Mr Worotnicki added that measurements of the changes in stress, displacement and noise in open slopes gave good results, whereas in pillars only the stress measurement was useful.

Dr Bock suggested that noise generation was not a continuous process and was therefore of limited predictive value. Mr Worotnicki commented that experienced people were the best predictors of failure.