Ground Support Design for Olympic Dam Expansion Project

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Summary: Olympic Dam is currently undergoing an expansion project, costing an estimated $AUS 1.5 billion. Included as a part of the expansion project is 24km of tunnels as well as 6km of underground rail system. BFP is currently working under contract to Western Mining Corporation (WMC), to provide ground support design and advice relating to the expansion project. The two main approaches being used for the support design was an overall rockmass one using the Rock Tunnelling Quality Index, Q by Barton et al. (1974) and a structural failure one using the geotechnical programs DIPS and UNWEDGE. Both these approaches have been used to derive a support design for all the proposed development drives, shafts and chambers located in varying orientations, depths and geological materials for the Olympic Dam Expansion Project.

1. INTRODUCTION

Olympic Dam, so named after a livestock watering dam on the Roxby Downs pastoral lease, was first discovered in 1975 by the Western Mining Corporation (WMC). It was not until 1976 however that drill hole RD10 producing an intersection of 170 metres containing 2.1% Cu and 0.6kg/t U308 that the economic potential of the deposit was realised.(1) Estimated ore reserves are currently put at 569 million tonnes of mineralisation, requiring a mine life of 100 years. The expansion project will triple the output of the Olympic Dam Mine from 3 MT a year to 9 MT a year, making it one of, if not the largest uranium mine in the world.

Design of the tunnels and associated chambers has resulted in varying levels of support using rock bolts, cable bolts and fibre reinforced shotcrete. The actual design process will be detailed further in the following sections.

The proposed crusher and associated development are located in massive granite and granite breccias, having an average UCS value of 129 MPa. The major openings are located well away from the ore body.

2. DESIGN APPROACH

The BFP support design, technical specification and contract drawings were required by WMC and the contractor well in advance of any major openings or exposure. Hence the data provided to BFP consisted of structural mapping information from the decline which was advancing towards the area of interest and some specific geotechnical holes drilled from the decline towards two proposed crusher sites.

From the database provided, the BFP ground support approach accounted for two discrete failure modes, i.e. an overall rockmass failure or discrete structural wedge failures. The rock mass approach used the rock mass classification system of Barton et al (1974) (2) together with the latest support classification chart of Grimstad and Barton (1993). In contrast the structural approach used the Canadian program UNWEDGE to examine the occurance, size and weight of potential wedges in the roof and sidewalls of all openings.(3)

The two methods vary significantly in their philosophy, with the Q system being primarily concerned with overall rockmass properties such as rock strength, block size and inter-block shear strength while UNWEDGE is primarily concerned with the formation of discrete structural wedges.

3. POTENTIAL ROCK MASS FAILURES

Rock mass classification schemes have been continuously developing for over 100 years, with the first reported use of a classification system for the design of tunnel support in a paper by Terzaghi (1946).

The Rock Tunneling Quality Index or Q system as it is more commonly known is used for the determination of rock mass characteristics and tunnel support requirements. The numerical value of Q varies on a logarithmic scale from 0.001 to a maximum of 1000 and is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Where:

RQD	: is the Rock Quality Designation.
J_n	: is the joint set number.
J_r	: is the joint roughness number.
J_a	: is the joint alteration number.
J_w	: is the joint water reduction factor.
SRF	: is the stress reduction factor.

This equation can be broken down into three components relating to both geology and geometry as follows:

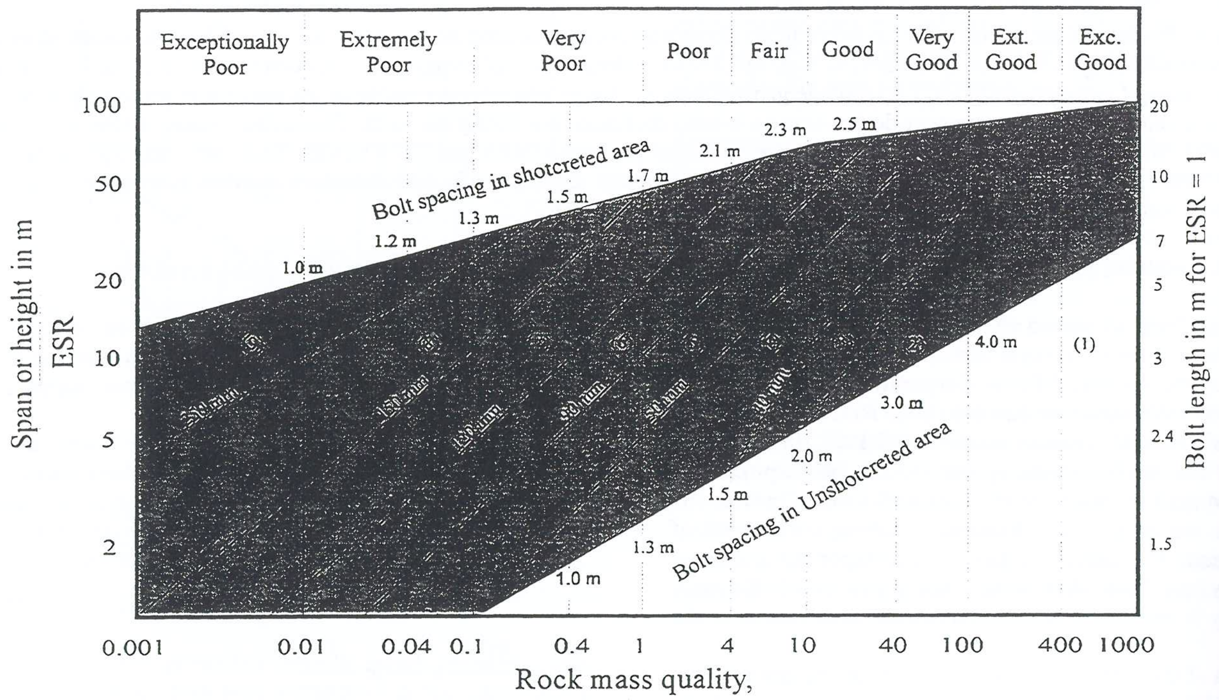
- Block Size (RQD/J_n)
- Inter-block shear strength (J_r/J_a)
- Active Stress/Strength (J_w/SRF)

The first quotient (RQD/J_n), can be seen as a crude measure of block or particle size with a maximum value of 200 and minimum value of 0.5.

The second quotient (J_r/J_a) represents the roughness and frictional characteristics of the joint walls or filling materials. The quotient is heavily weighted in favour of rough unaltered joints in direct contact.

The third quotient (J_w/SRF) can be regarded as the total stress parameter. It is a complex empirical factor describing the 'active stress'. (4)

In order to obtain Q, either logging of core on site, or underground exposure mapping is required to obtain a figure for RQD and the various J factors. The value of the aforementioned factors may vary throughout the same drill hole or opening as different materials are intersected. In



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40 - 100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and boltin
7) Fibre reinforced shotcrete, 120 - 150 mm, and boltin
8) Fibre reinforced shotcrete, >150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Figure 1. Estimated support categories based on Q (After Grimstad and Barton, 1993)

designing a large opening for example, the data from one drill hole might be separated into different regions. The core from the sidewall and roof would be geotechnically logged at intervals governed by the material encountered whilst drilling. This geotechnical logging includes such information as;

- Core loss
- Rock type
- Weathering
- Alteration
- RQD
- Number of defects
- Number of defect sets
- Defect type
- Roughness
- Infill
- Infill width

Once the core has been logged the values for the various Q factors can be obtained from the tables produced by Barton et al.

Once Q is established, the next step is to determine the *Excavation Support Ratio* or ESR. The value of ESR is related to the intended use or duty of the excavation and to the degree of security which is demanded of the support system to maintain the stability of the excavation. Barton et al. (1974) suggested the following values:

they are planar and continuous, ensuring that the largest possible wedges are enabled to form. UNWEDGE assigns a factor of safety to each potential wedge, allowing the design engineer to make an assessment of the support required. UNWEDGE allows the user to select one of the wedges, starting with the wedge with the lowest factor of safety and re-running the program with various ground support options simulated (end anchored bolts, shotcrete, etc.). Normally the first run will simulate the effect of the "first pass" support design as determined from the rockmass approach outlined in Section 3. UNWEDGE then provides a new Factor of Safety, taking into account the increased resisting force provided by the support. Multiple analyses are performed until a suitable Factor of Safety is reached (normally ≥ 1.5).

Despite the advantages in using UNWEDGE to perform structural analyses there are also disadvantages. UNWEDGE is primarily designed for use where the in situ stresses are low and where their influences can be neglected without the introduction of significant errors. In cases of high in situ stresses, the factor of safety provided by the program may be too high or too low, depending on the shape of the wedge. In the case of a long thin wedge, the high in situ stress will tend to clamp the wedge in place, the result being a factor of safety too low. The reverse is true for shallow flat wedges which may be forced out by the high stresses.

The following case study will more clearly illustrate the methodology behind the Barton et al. Q system and UNWEDGE.

5.0 CASE STUDY

5.1 Rock Mass

The following example is taken from the Olympic Dam Crusher Chamber Geotechnical Investigation produced for WMC by BFP Consultants Pty Ltd.

A total of six holes were drilled with the geotechnical logging of 1000m of BQ core. Samples were selected for Uniaxial Compression Strength tests (UCS). From this data, the following values for the various components of the Q system were obtained;

RQD: almost of the logged core had an RQD of 100%. The lowest RQD value recorded was 93%.

J_n: varied from 0.5 to 9 (i.e. massive rock with no defined joint sets to 3 for well defined joint sets).

J_r: most of the observed defects had undulating rough surfaces. Occasional slicksides were noted, a value of 3 was used for J_r.

J_a: the majority of defect surfaces were unaltered or contained hard rock infills that would not be likely to weaken the rock mass. A value of 1 was used for J_a

J_w: very low water flows were observed at the drilling site. J_w has therefore been set to 1 for all Q calculations.

SRF: for the sake of this example, SRF has been set to 150 based on the recent work by Grimstad and Barton (1993).

The mean value for Q 3.8 was selected for use in the rock mass design.

In order to use the chart in Figure 1, the Span/ESR factor is required. The initial unsupported span is 7.0 m while an ESR figure of 1.3 was used from Category C, Table 1. These two values provide a Span/ESR figure of 5.4.

Using the mean value for Q and the Span/ESR of 5.4, the support design from Figure 1 is rockbolts about 2.8 metres in length at about 2 metre spacing with 50mm of fibre reinforced shotcrete.

5.2 Structural Analysis

This case study involves the design of the Crusher Chamber at Olympic Dam, Roxby Downs. A plan view of the crusher chamber is shown in Figure 2.

As the dimensions in Figure 3 indicate, the crusher opening is very large and surrounded by development. The crusher's orientation was selected to minimize any stress effects. Previous insitu stress measurements in the area resulted in a predominately NE-SW principal stress direction, hence the long axis of the crusher was aligned in this direction. Due to the size, orientation and location of the crusher several additional problems needed to be overcome, primarily;

- The length and location of any rock bolts would be governed by the location of surrounding openings as well as any potential structural and rock mass requirements.
- The crusher excavation sequence.

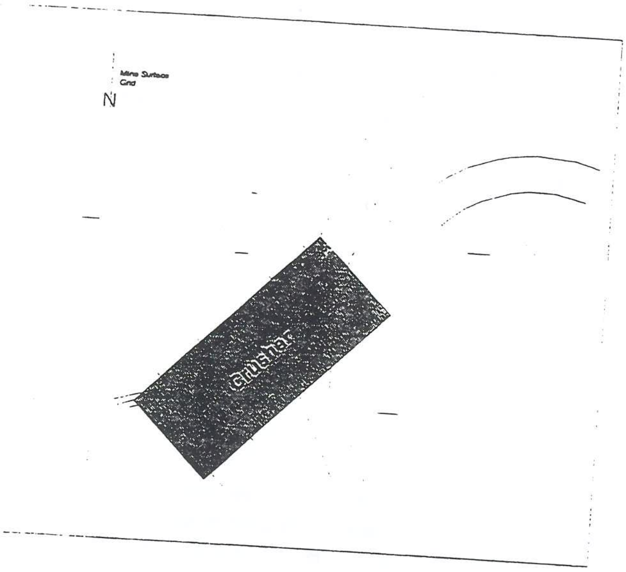


Figure 2. Crusher Chamber – Plan Elevation

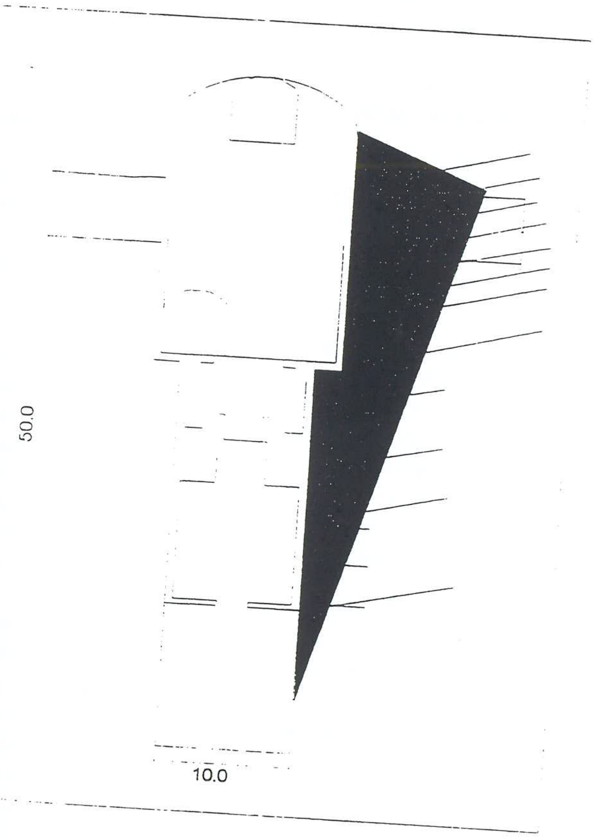


Figure 3 Crusher Chamber – Cross Section

Having being provided with structural mapping data by WMC from the nearby decline, a DIPS structural file provided the following result

Joint Set	Dip (degrees)	Dip Direction (degrees)
1	50	241
2	84	350
3	26	90

The UNWEDGE analysis indicated the potential for a large wedge on the South-East wall of the crusher chamber. Due to the size and shape of this wedge (see Figure 3), the primary concentration of bolts would have to be in the upper chamber as most of the mass of the wedge is contained in the upper portion. The maximum depth of the potential wedge in this region is approximately 8 metres, and this potential wedge meant that cable bolts would need to be used to ensure adequate anchorage length into solid rock. Cable bolts however do not offer immediate support and usually require the time taken for the grout to cure, before offering their maximum level of support.

The use of end anchored rock bolts was also recommended in conjunction with shotcrete. These end anchored rock bolts offer immediate support of the surface skin, allowing men and machinery to safely operate in the supported openings. The shotcrete provides additional safety against block failures between the rock bolts.

As can be seen in Figure 2, there are drives and openings located on the South-East wall. The bolting pattern was also tailored to suit the location of these openings. Another practical consideration was the excavation stages. The crusher chamber was to be excavated using a "top down" approach. With this method, the support would have to be installed as excavation progressed, hence the use of the end anchored bolts to ensure immediate support particularly in the roof.

Ultimately, the support installed consisted of cable bolts, end anchored bolts and shotcrete, with a concentration of cable bolts in the top of the South- East wall to support the potential wedge.

The ground support is performing well but the design process has not stopped. All openings are geologically mapped, not only for filing records but with the purpose of checking against the original DIPS master file and UNWEDGE analyses. In this way the insitu structures for a particular opening are used to update and change, if necessary, the UNWEDGE input data. Further analysis can then be re-run so that the original support design is checked and the "design loop" is closed off.

6. CONCLUSIONS

Analyses using both a structural and rock mass approach have a significant role to play in the design of underground openings. The contrast between the two, i.e. the *rockmass approach* based on the inherent properties of the rock, ie. rock strength, the presence of water, the number of joints and defects etc.. and a *structural* analysis based on the potential formation of wedges caused by the intersection of joints and/or bedding planes. Both of these methods are required to cover all potential failure modes when designing any underground opening in rock.

7. REFERENCES

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8. ACKNOWLEDGEMENTS

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