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SYNOPSIS: Construction of a 23m wide, 57m high, and 210m long underground power house cavern is in progress as a part of the multi-purpose Sardar Sarovar Project in India. The rock mass around the cavern is basalt which is intruded by a number of dolerite dykes. In view of the high side walls of the cavern, and the presence of a 1 to 2m thick shear zone running across the cavern width, a comprehensive approach was worked out for estimation of the wall support requirements. The approach included estimation of the roof support requirements using the four available approaches, and comparison of these requirements with the roof support system actually provided, and established as safe and adequate by the instrumentation data of six years. A favourable comparison established the reliability of the approaches used, and the most reliable of these approaches, i.e., the Barton's approach was then used with confidence for estimation of the wall support requirements.

INTRODUCTION

An underground power house cavern is being constructed as a part of the multi-purpose Sardar Sarovar Project in the state of Gujarat in India. The D-shaped cavern, which is located on the right abutment of the Sardar Sarovar Dam, is aligned N 10° E-S 10° W and is 23m wide, 57m high and 210m long. The rock mass around the cavern is basalt which has been intruded by a number of dolerite dykes (Fig.1). A shear zone runs across the cavern. The excavation has so far (August 1992) progressed up to about 85 percent of the cavern height. A comprehensive approach was worked out to estimate the roof and the wall support requirements. The approach and the results obtained are presented here.

GEOLOGY

The rocks in the area are of Deccan trap group under a thin soil cover of about 30 cm. They consist of different lava flows, viz., porphyritic basalt, amygdaloidal basalt and agglomerate (Fig.1).

The depth of weathering in basalt varies from 3m to 22m. Below this weathered zone, the rocks are fresh but jointed and fractured. The presence of thin calcified veins along the fractured planes have adversely affected the strength of the rock mass. The basalts are intruded by two dolerite dykes, varying in width from 40m to 50m.

The first southern dolerite dyke trends in N70° E-S70° W direction with a dip of 60°-65° towards the river side. Its two contacts with the adjacant basalt are sheared. The first shear contact is thin and does not intersect the cavern, whereas the second sheared contact is 1-2m wide and intersects the cavern roof at chainage 1492m. This shear zone is traversed by one set of closely spaced strike joints which are intensely iron stained and are almost

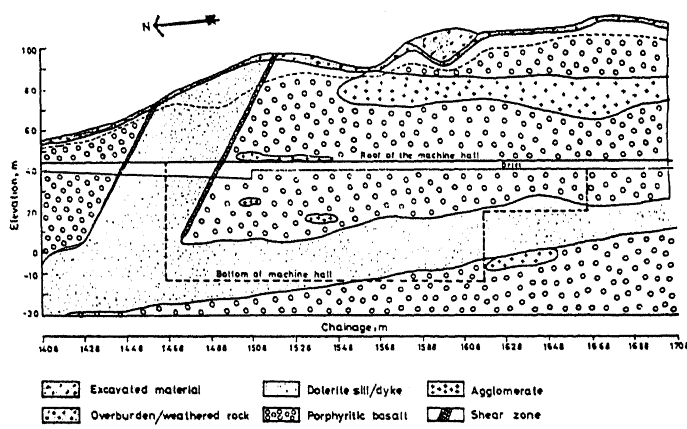

Fig. 1. Geological L-Section along the Cavern

parallel to the attitude of the dolerite dyke. The gouge within this shear zone is soft (but strongly consolidated), jointed, fractured, weathered and calcified, and contains small quantities of clays (2 to 4 percent) and fragments of dolerite dyke. Another near vertical dyke at the right end of the cavern trends in N55°E-S55°W direction. Its southern contact with basalts is calcified, while the northern one has not been ascertained.

The cavern is generally dry except in the shear zone area where minor water flow occurs during the monsoon season.

An agglomerate band, which is about 40m long and 2-3m thick and dipping at 8° towards the hill side, is present about 1m above the roof of the cavern between chainage 1501m and 1541m. The agglomerate rock is lensoid in nature. It is fresh, hard and jointed, and is grey in colour. The joints, mainly inclined, are iron stained and

Table 1. Estimated Values of Q and Ultimate Support Pressure, p_v, from Barton's Approach

Rock Mass Category	RQD	Jn	Jr	Ja	Jw	SRF	Q	p_v kg/cm sq
Jointed Basalt	55	12	1.5	0.75	1.0	1.0	9.16	0.73
Jointed Dolerite	65	12	1.5	0.75	1.0	1.0	10.88	0.69
Shear zone	25	2	1.0	4.0	1.0	2.5	1.25	0.88

Table 2. Estimated Roof Support Requirements and Comparison of Estimated Ultimate Roof Support Pressure with Available Roof Support Capacity

Rock Mass Category	Bolt Length m					Bolt Spacing m				Estimated Ultimate Support Pressure kg/cm^2	Available Support Capacity kg/cm^2		
	After Barton	After Cording	After USCE	After Hoek & Brown	Used	After Barton	After USCE	After Hoek & Brown	Used	p_v	p_{sv}	p_{bv}	Total p_{cv}
Jointed Basalt	5.5	5.9	5.7	5.75	6.0	2.0	> 2	> 2	1.75	0.73	0.30	0.58	0.88
Joited Dolerite	5.5	5.9	5.7	5.75	6.0	2.0	> 2	> 2	1.75	0.69	0.30	0.58	0.88
Shear Zone	5.5	5.9	5.7	5.75	6.0	1.75	> 2	> 2	1.75	0.88	0.30	0.58	0.88

filled with calcite. The lower contact of the agglomerate band with the basalt is gradational while the upper one is open.

THE APPROACH

The presence of a 1-2m thick shear zone running across the cavern, and a 1.5m thick agglomerate band running just above the roof of the cavern, raised doubts about its stability. Therefore, a comprehensive approach was worked out to estimate the roof and wall support requirements for the cavern. The approach consisted of the following steps:

(i) Estimation of the roof support requirements and the ultimate support pressure from the available approaches.

(ii) Comparison of the roof support estimated from the available approaches with the roof support actually provided, and monitored by instrumentation for six years, to establish the reliability of these approaches.

(iii) Estimation of the wall support requirements from the most reliable approach.

ESTIMATION OF ROOF SUPPORT REQUIREMENTS

For the purpose of estimating the support requirements, the rock mass encountered in the cavern has been classified in the following three categories:

(i) jointed basalts,
(ii) jointed dolerites, and
(iii) shear zone.

Bolt Length, Bolt Spacing, and Ultimate Support Pressure

The bolt length and the bolt spacing were estimated for the cavern roof from the approaches of:

(i) Cording et al (1971),
(ii) U.S. Corps of Engineers (1980),
(iii) Hoek & Brown (1980), and
(iv) Barton et al. (1980).

The ultimate roof support pressure was also worked out from Barton's approach. This required the determination of Q values. Table 1 contains the estimated values of Q and the ultimate support pressure, p_v, for the three rock mass categories. The estimated support requirements from the above four approaches for these rock mass categories are given in Table 2.

Bolt Pre-tension

There are two opinions regarding the desirability of the application of the pre-tension to the rock bolts. One school of thought feels that the pre-tension helps in stabilising the underground openings, whereas the other advocates that it cannot be preserved for long durations and is,therefore, unnecessary, and the bolts get tensioned automatically with passage of time. Since the rock bolts loose pre-tension with time and it is not possible to restore the lost tension once the bolts are covered with shotcrete, the long-term advantages of pre-tension are questionable. However, the desirability of pre-tension as a short-term measure cannot be denied. Pre-tension is normally applied to the rock mass around a

Table 3. Comparison of Required and Applied Rock Bolt Pre-tension in Cavern Roof

Rock Type	Ultimate Roof Support Pressure kg/cm^2	Short-term Roof Support Pressure kg/cm^2	Capacity of Shotcrete kg/cm^2	Required Pre-tension (3-4) kg/cm^2	Area of Influence of Bolts cm^2	Required Pre-tension (5x6) tonnes	Applied Pre-tension tonnes
1	2	3	4	5	6	7	8
Jointed Basalts	0.75	0.43	0.30	0.13	175x175	4.0	7.0
Jointed Dolerites	0.69	0.41	0.30	0.11	175x175	3.4	7.0
Shear zone	0.88	0.52	0.30	0.22	175x175	6.7	7.0

cavern so that a pre-stressed rib of rock mass is created soon after excavation. This pre-stressed rock rib is relatively more rigid and, therefore, helps in controlling the convergence.

It is recommended, therefore, to apply pre-tension only for short-term advantages. When the pre-tensioned rock bolts are used with shotcrete, the sum of the applied pre-tension and the shotcrete capacity must be greater than the short-term support pressure. Thus,

$$p_b + p_s > p_{vi} \qquad (1)$$

where, p_{vi} is the short-term roof support pressure, p_b is the bolt pre-stress and p_s is the shotcrete capacity.

The required bolt pre-tension would, therefore, be:

$$p_b > p_{vi} - p_s \qquad (2)$$

Short-term Roof Support Pressure

The short-term roof support pressure, p_{vi}, from the approach of Barton et al.(1980) is given by:

$$p_{vi} = \frac{p_v}{1.7} \qquad (3)$$

where p_v is the ultimate roof support pressure.

The short-term roof support pressure, the capacity of shotcrete for the roof, and the required roof bolt pre-stress, for the three types of rock masses are given in Table 3. A comparison of the applied pre-tension with the recommended pre-tension shows that it is safe for all the three rock mass categories.

EVALUATION OF ESTIMATED ROOF SUPPORT REQUIREMENT

Bolt length

In the case of caverns, once the rock wedges have been taken care of, the bolt length for roof depends only on the width of the cavern (Barton, 1980 - bolt length, L = 2 + 0.15 H/ESR, where H = cavern width, and ESR = excavation support ratio). The estimated bolt length from the available approaches range from a minimum of 5.5 m (Barton's approach) to a maximum of 5.9 m (Cording's approach) as shown in Table 2. The bolt length of 6m actually used is, therefore, safe.

Bolt spacing

The estimated values of bolt spacing from all the approaches is more than the bolt spacing actually provided (Table 2). The adopted bolt spacing for all the three rock mass categories covering the entire length of the cavern roof is, therefore, adequate.

Support pressure

The ultimate roof support pressure was worked out from Barton's approach only, as the other available approaches do not facilitate estimation of the support pressure. The available roof support capacities (Table 2) were found to be slightly greater than the ultimate roof support pressure for the jointed basalts and the jointed dolerites, whereas these values were just equal for the shear zone.

Instrumentation

The performance of the roof supports has been monitored for six years to establish their adequacy. The instrumentation scheme was formulated to take up the the monitoring work in two phases. In the first phase, the instruments were installed to monitor the construction stage behaviour of the roof and the walls of the cavern in the shear zone and the agglomerate band areas. Single-point borehole extensometers (SPBXs) and multi-point borehole extensometers (MPBXs) were installed in addition to the closure studs (for measuring closure by tape extensometer), load cells, pore pressure cells, and stress meters.

For monitoring the post-construction behaviour of the cavern, MPBXs, SPBXs and points for closure measurements are being installed regularly during the second phase of the instrumentation. All the instruments in this phase are connected to a computerised data-logging system.

The details and the results of the instrumentation have been given by Verman et al. (1992). The instrumentation results indicate that the cavern roof is stable and the supports provided are adequate.

The following points emerge from the above discussion:

(i) A reliable estimate of bolt length and spacing is possible from the four available approaches.

(ii) Only Barton's approach provides reliable estimates of roof support pressure and it can,therefore, be used to estimate required rock bolt pre-tension for the cavern roof.

(iii) The input parameters (RQD, Jn, Jr, Ja, Jw, SRF) obtained at the site, and used for estimating the support requirements from Barton's method, are reliable, since the estimated roof support requirements compare favourably with the used support, which has been established as safe by the instrumentation data of six years.

ESTIMATION OF WALL SUPPORT REQUIREMENTS

Having established the usefulness of Barton's Q-system, and the reliability of the input parameters obtained, the wall support requirements have been worked out using this approach. The results are presented in Table 4 for the three rock mass categories.

EVALUATION OF RECOMMENDED BOLT LENGTH FOR SIDE WALLS

The recommended rock bolt length for the side walls was critically examined in view of the fact that the design of cavern wall support is often more difficult than roof support design for several reasons (Cording,1971). Also, doubts were raised on the adequacy of the recommended bolt length (10m) on account of high sidewalls and presence of the shear zone. Therefore, the sidewall bolt length was worked out using the other available approaches also, namely, Cording's approach, Hoek & Brown's guidelines, U.S. Corps of Engineer's approach, and from the case-histories.

Case-histories

Case-histories of caverns, some of them with comparable wall heights, have been compiled in Table 5. It is clear from the table that anchors of much longer lengths than the suggested length of 10m have been provided in most instances. It may be seen that in most of the cases, shorter rock bolts used as temporary support have later been supplimented by longer tensioned bolts/ cables/tendons for long-term requirements.

In case of Sardar Sarovar cavern, only 10m long tensioned rock bolts have been suggested. Therefore, the recommended bolt length of 10m is not off the practice.

Cording's approach

Based on past experience, Cording et al. (1971) proposed that the bolt length should be scaled in proportion to the wall height. Some of

Table 4. Estimated Ultimate Wall Support Pressure and Wall Support Requirements from Barton's Approach

S.No.	Rock mass description	Q_v	Wall Factor	Q_h	P_h kg/cm sq	Support Category	Estimated Wall support	Recommended Wall support
1.	**Jointed Basalt**	9.166	2.5	22.91	0.542	16	B(tg)1.75m (9.8T) S(mr)8.5cm	B(tg)1.75m (10T) S(mr)8.5cm
2.	**Jointed Dolerite**	10.88	2.5	27.2	0.51	16	B(tg)1.75m (9.2T) S(mr)8.5cm	-do-
3.	**Shear zone**	1.25	2.5	3.125	0.645	20	B(tg)1.75m (11.7T) S(mr)8.5cm or B(tg)1.6m (10T) S(mr)8.5cm	B(tg)1.6m (10T) S(mr)8.5cm

Note - (i) Length of rock bolts = 2 + 0.15 H/ESR = 9.65m (H = 51m; ESR = 1.0)
Recommended bolt length = 10 m

(ii) If felt necessary, bolt spacing may be increased with corresponding increase in the amount of pre-tension, keeping the ratio of the pre-tension and square of bolt spacing equal to the support pressure.

(iii) Notations used in the last two columns are: B(tg) - tensioned rock bolts; B(tg)1.75 - tensioned rock bolts with 1.75m spacing (in both directions);S(mr)8.5cm - 8.5cm thick shotcrete with mesh reinforcement; (9.2T),(9.8T), etc. - bolt pre-tension in tonnes.

Table 5. Case-histories of Caverns

Cavern Location and Dimensions B=width, H=Height, L'=Length, D=Overburden (m)	Rock types & conditions	Wall support details	Roof Support Details
1. Paulo Afonso IV Power Station, Brazil B=24,H=54,L'=210,D=50	Good quality migmatite containing granite, biotite, gneiss, amphibolite and biotite schist.	Rock bolts and 18m long tendons	4cm thick shotcrete, 9m long 32mm dia bolts on 1-5m grid tensioned to 22t
2. Lago Delio Power Station, Italy B=21,H=60.5,L'=195.5, D=130	Gneiss with sub-vertical foliation at right angles to cavern axis, several joint sets in rock mass.	Wall heavily reinforced with 3-5m long tensioned rock bolts, 5-25m long prestressed cables of up to 80 T capacity and reinforced shotcrete	Concrete arch roof
3. San Fiorano Power Station, Italy B=19,H=64.7,L'=96.7, D=210	Phyllite with sub-vertical closely spaced schistosity planes forming major discontinuities.	Cables up to 33m long tensioned upto 80 T and 5m long, 5 T perfo bolts used for wall reinforcement.	Concrete arch roof
4. Okutataragi Power Station, Japan B=24.9,H=49.2,L'=133.4, D=200	Quartz porphyry, diabase and rhyolite	Walls supported by 5 to 15 m long rock bolts,one per 3 sq m of wall area, tensioned from 8 to 35t	Concrete arch roof
5. Shintakasegawa Power Station, Japan B=27,H=54.5,L'=163, D=250	Good quality granite with major faults nearby. Average joint spacing 20 cm. Horizontal in-situ stress 1.8 times vertical.	Walls reinforced by bolts and 15-20m long anchors tensioned to 120t,16-24cm thick mesh reinforced shotcrete on upper walls. Lower walls concreted.	5m long 25mm dia and 2m long 22mm dia rock bolts
6. El Toro Power Station, Chile B=24.4,H=38.4,L'=102	Granodiorite with orthogonal jointing.	15.2m long tendons on 6.1m pattern in walls	15-17m long tendons on 6.1m pattern tensioned to 1.8MN and 4m long bolts on 2.4m pattern tensioned to 180kN
7. Chhibro Power Station, India B=18.2, H=32.5, L'=113, H=230	Thinly beeded limestones and slates, joints shear zones and bedding planes isolated potentially unstable blocks. Beddings dips at 45 to 50	Walls supported by 350 prestressed anchors,avg. length 23.5m,capacity 60t, spacing 2-5m; 7.5cm reinforced shotcrete where necessary	Steel arch support
8. Okuyoshino Power Station, Japan B=20.5,H=41.6, L'=157.8,D=180	Sandstones & shales, sometimes interbedded, dipping at 40^{o}.Ten fault zones of maxm.width=1.5m	Wall support by 5m long prestressed bars and 10-20m long prestressed cable anchors.	Concrete arch support
9. Ronco Val Grande, Lake Maggiore B=20, H=58.5, L'=187	Gneiss, partially clefted.	Upper 15m wall supported by 200k,16m long tendons, spacing 3x3m.Lower 42m of wall supported by 80k,16m long tendons,spacing 6x6m	0.75m thick concrete arch support
10. Hongrin power station, Veytaux, Switzerland. B=30.5, H=27.5 L'=140, D=65.150	Marly limestone & limestone schist.Undulating, near-horizontal bedding (20-150 cm spacing)cut by three systems of fractures & faults,some filled with clay-like material, or mylonites. Water bearing rock contains considerable amount of clay. RQD=50-75	11-13m long, tensioned anchors, 6-7m spacing and 4m long bolts (mostly prestressed) 2-3m spacing.	Details not available

Table 6. Cording's Case-histories

S.No.	Cavern Location and Dimensions B=Width,H=Height, L'=Length,D=Overburden (m)	Rock type & properties	Bolt Length, L (m)	Cavern Height, H (m)	L/H
1.	Nevada cavities I & II B=24, H=42, L'=36,D=390 (σ_v=67kg/sq cm, σ_h=33 kg/sq cm)	Tuff RQD:95-100% qc=100 kg/sq cm	7.2	42	0.17
2.	Tumut I,Australia B=23, H =33, L'=90, D=330 (σ_v=100 kg/sq cm)	Granite,Granite Gneiss RQD: Fair-Good qc= 1333 kg/sq cm	3.6	33	0.11
3.	Morrow Point power plant, Colorado B=17, H=30-41 L'=62, D=120 (σ_v=27-134 kg/sq cm)	Micaceous Quartzite, Mica Schist RQD: Good to Excellent qc=400-1067kg/sq cm	3.6	30-41	0.12
4*	Oroville Point Power Plant B=20.7, H=36 L'=165 (σ_v = σ_h = 34 kg/sq cm)	Amphibolite RQD: Fair to Good	6.0	36	0.17
5.	Poatina Power Station, Tasmania B=13.5, H.25.5 L'=90, D=150 (σ_v =80 kg/sq cm σ_h=120-160 kg/sq cm)	Thin to massive bedded mudstone qc = 333 kg/sq cm	4.2	25.5	0.16
6*	Norad, Cheyenne Mountain,Colarado B=13.5, H=18, L'=180 (σ_v=80 kg/sq cm)	Biotite Granite qc = 333 to 667 kg/sq cm	3	18	0.17

* Rock mass description close to that encountered in the Sardar Sarovar cavern.

the case-histories included by Cording et al. are compiled in Table 6. According to this approach, the L/H ratio (Table 6) is lower for a competent rock mass and higher for an inferior rock mass. Needless to mention that the L/H ratio should be constant for rock masses of comparable quality. It may be seen that the rock masses in cases 4 and 6 of Table 6 are closest to the case of the power house cavern under consideration (represented by RQD value and strength). The L/H ratio for these two cases is 0.17. The RQD, as observed in the cavern under consideration is slightly lower than the values reported for cases 4 & 6, and its average value may be described as ¶fair'. Therefore, the L/H ratio for the present case should be slightly more than 0.17. It is felt that a range of 0.17 to 0.20 may be chosen. The corresponding range of rock bolt length, therefore, is 8.67 to 10.2 m, with an average value of 9.5m. This approach, therefore, also supports the recommendation (based on Barton's approach) of providing 10m long rock bolts.

Guidelines of Hoek & Brown

Hoek & Brown (1980) have given the following guidelines for determining the rock bolt length:

(a) Tensile stress criterion

Estimate the maximum sidewall boundary stress in the rock surrounding the excavation by using the following equation:

$$\text{maximum sidewall stress, } \sigma_s = \sigma_v \,(B'-k) \qquad (4)$$

where,

σ_v = vertical in-situ stress,

B' = shape constant
= 1.5 for the shape of Sardar Sarovar powerhouse cavern, and

$$k = \frac{\text{horizontal in-situ stress}}{\text{vertical in-situ stress}} \qquad (5)$$

Table 7. Estimation of Boundary Stress

Rock type	σ_v (kg/cm sq)	σ_h (kg/cm sq)	B'	k= σ_h/σ_v	$\sigma_s = \sigma_v(B'-k)$ (kg/cm sq)
Basalt	13.79	11.71	1.5	0.85	+8.96(compressive)
Dolerite	12.84	9.47	1.5	0.74	+9.76(compressive)

The values of σ_s calculated from Eq.4 are given in Table 7 for the basalts and the dolerites. The positive signs of σ_s values indicate that there would be no tensile stresses on the sidewalls. Therefore, the extent of tensile stress zone can not be the criterion for determining sidewall bolt length in this case.

(b) Compressive stress criterion

If only compressive stresses are present around the opening (as is the case in this situation), compare the σ_s values with the unconfined compressive strength of the rock mass σ_c which can be obtained from the following equation:

$$\sigma_c = q_c \sqrt{s} \quad (6)$$

where,

s = material constant which depends on the rock mass properties and is related to the Q value and the rock type,

q_c = uniaxial compressive stregth of the intact rock material,

The values of vc have been worked ot to be 73.4 kg/cm^2 for basalt and 31.96 kg/cm^2 for dolerite. If the maximum boundary stress (8.96 kg/cm^2 for Basalt, 9.76 kg/cm^2 for dolerite) does not exceed the v_c values (as is the case here), the bolt length should be adequate only to prevent the structurally controlled instability, i.e., the possibility of formation of wedges and blocks. Such a situation of structural instability has, however, already been avoided in this case by providing the immediate support.

(c) Minimum bolt length criterion

The last guideline is to check for the minimum bolt length to be provided. Since, in this case, the bolt length is not to be based either on the stabilisation of the tensile stress zone (such a zone does not exist around the walls), or on the stabilisation of the potential failure zone resulting from the maximum boundary stress exceeding the unconfined compressive strength of the rock mass (maximum sidewall boundary stress is much less than the unconfined compressive strength of both basalt and dolerite), or on the prevention of the structurally controlled instability (such an instability has already been avoided by providing immediate support), the only criterion left to determine the bolt length is to provide the minimum required length.

The followng empirical rules provide a check for the estimated bolt length:

Minimum bolt length should be the greatest of:

(i) Twice the bolt spacing

= 2 x 1.75 = 3.5 m

(ii) Three times the width of critical and potentially unstable rock blocks defined by average joint spacing in the rock mass

= 3 x 0.40 = 1.20 m

(iii) For excavation heights greater than 18 m, the length of sidewall bolts to be one fifth of wall height

= (1/5) x 51 = 10.20 m

Thus, according to Hoek & Brown's guidelines, the minimum sidewall bolt length should be 10.2m. This also supports the recommended rock bolt length of 10m.

Criterion of U.S. Corps of Engineers

U.S. Corps of Engineers (1980) use the following criterion to determine the sidewall bolt length:

Sidewall bolt length, L = 0.2 x Height of wall

According to this approach the sidewall bolt length works out to be 10.2m. Thus, this approach also supports the recommendation for providing 10m bolt length.

The lengths of sidewall rock bolts, determined by different approaches, are given in Table 8.

One may have a feeling from Table 8 that the approaches of Hoek & Brown as well as that of U.S. Corps of Engineers suggest slightly longer bolts as compared to the recommended length(10m)

Table 8. Sidewall Rock Bolt Lengths Estimated from Different Approaches

Recommended (Barton's approach)	Case-histories	Cording's Approach	Hoek & Brown's Approach	US Corps of Engineers' Approach
10.0m	Bolts up to 15m Anchors up to 25m	9.5m	10.2m	10.2m

which is based on the Barton's approach. Case studies also indicate the need of bolts longer than 10m. Looking into the basis of the approaches of Hoek & Brown and the U.S corps of Engineers, and comparing these with Barton's approach, it is felt that Barton's approach is relatively more scientific. In view of this, and the fact that the length of the rock bolts for walls estimated from Barton's approach does not vary siginificantly from that estimated from these two approaches, the bolt length has been selected as 10m. This value also tallies with the estimate of the sidewall bolt length from Cording's approach.

CONCLUSIONS

(i) A reliable estimate of the cavern support requirements is possible from the four available approaches. Out of these, only Barton's approach gives estimate of the support pressure and has, therefore, been selected for estimation of the wall support requirements.

(ii) Performance monitoring of the supports for six years has established that the roof support system, comprising of 123mm thick shotcrete and 6.0m long staggered rock bolts with a spacing of 1.75m x 1.75m, and tensioned upto 7 tonnes, is adequate for the entire cavern length.

(iii) After having established the reliability of Barton's approach and the input parameters used on the basis of the adequacy of roof support system provided, as indicated by instrumentation, the approach was used to estimate the wall support requirements. The recommended wall support system consists of 85 mm thick shotcrete with 10 m long rock bolts, tensioned upto 10 tonnes. The recommended bolt spacing is 1.6x1.6m in the shear zone and 1.75x1.75m in the rest of the cavern. The bolt length was evaluated by other appraches as well, and was found to be adequate.

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